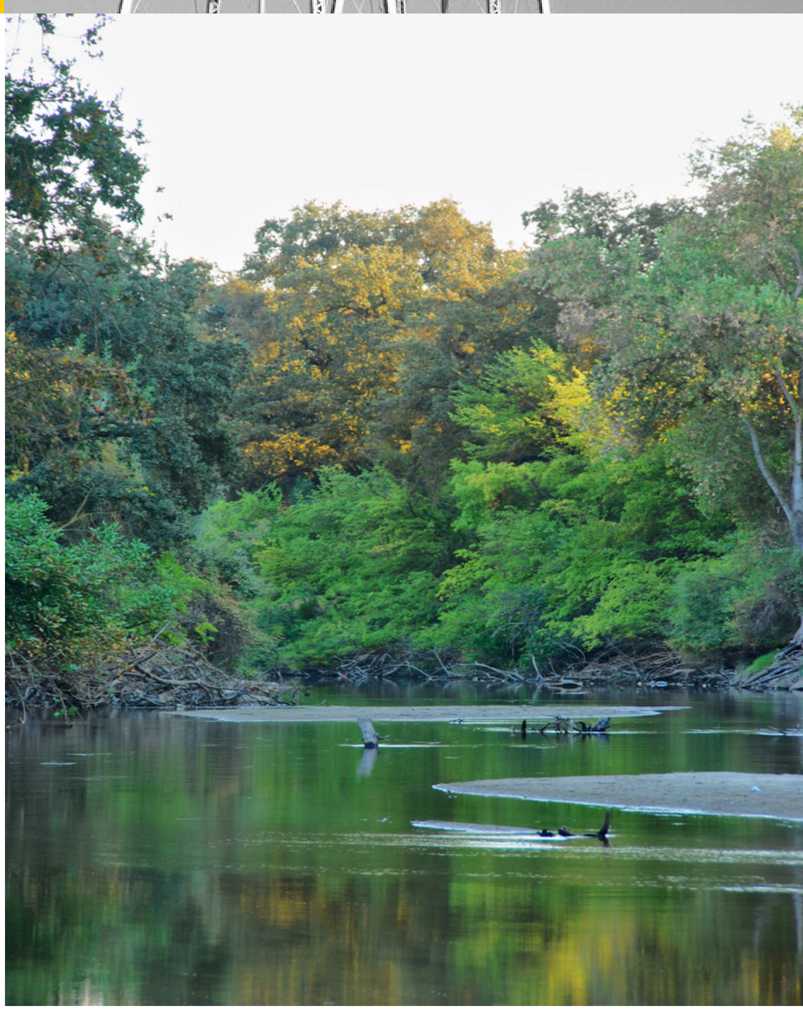
June
2014



Integrated Master Plan

for Potable Water, Sanitary Sewer, and Storm Drainage Systems

City of Reedley, CA

HDR

Integrated Master Plan
For Potable Water, Sanitary Sewer, and Storm Drainage Systems

City of Reedley

June 2014

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Executive Summary

This Integrated Master Plan (Master Plan) provides a comprehensive program for City of Reedley's (City) potable water, sanitary sewer and storm drainage systems. This Master Plan supplements the City's General Plan 2030, adopted in February 2014, by addressing how the City's existing water-related utilities will be upgraded and new infrastructure will be installed in order to provide desired levels of service for both the City's existing residents and expected future development. This executive summary provides an overview of the background, basis of planning, and recommended improvements described in this Master Plan.

ES.1. Background

The City is located along the Kings River in the central San Joaquin Valley portion of California, lying just inland between the State's coastal mountain ranges and the Sierra Nevada Mountains. The City is situated approximately 25 miles southeast of the City of Fresno and 20 miles northwest of Visalia. The City covers approximately 5-square miles and serves a population of approximately 24,000 through about 6,000 active water service connections[1].

ES.2. Basis of Planning

The City recently adopted The City of Reedley General Plan 2030 in late February, 2014. The General Plan 2030 provides a critical foundation for planning of the City's utility infrastructure. Decisions on where to expand the water, sewer and storm drainage systems are made with both current and future needs in mind. This Master Plan includes an analysis of utility system conditions and needs, first in terms of current conditions, and then in terms of planned future conditions. The planned future conditions are based on the Land Use Element of the General Plan 2030. As such, the recommended improvements contained within this Master Plan are consistent with the General Plan 2030 and its associated programmatic Environmental Impact Report (EIR), which was also adopted by the City in February 2014.

The facilities required to serve the development envisioned in the General Plan 2030 will be time-phased to correspond with projected growth as it occurs. Due to the uncertainty around the timing of the actual build out of the General Plan 2030, the following phases have been established for evaluation of the respective systems:

[1] Based on Department of Water Resources Public Water System Statistics report submitted in 2011.

- Existing Phase: Baseline Conditions
- Phase 1: Period between Existing (Baseline) and Approximately 2020
- Phase 2: Period beyond 2020 through build out of the City's SOI

The fundamental planning basis for developing water demands, projected sanitary sewer flows, and storm water runoff is the planned land use presented in the General Plan 2030, and as illustrated in Figure ES-1 for the Phase 1 and Phase 2 planning phases, respectively.

Although the projected 2030 land area presented in Figure ES-1 is based on the General Plan, when the City conducted the environmental review, three alternatives were considered. As described in the City's program EIR for the General Plan 2030, Alternative 3 was selected as the environmentally superior alternative. Alternative 3 includes fewer acres in the future SOI. As a result, the basis for the analysis of the potable water, sanitary sewer, and storm drainage systems under future (Phase 2) conditions is considered a worse case scenario since the selected environmentally superior alternative included development of less land.

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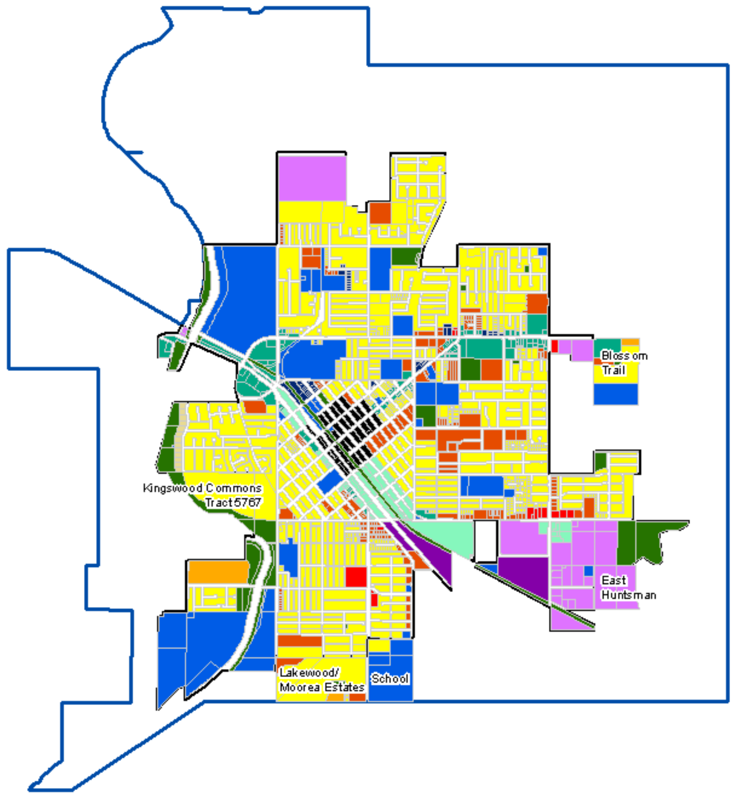
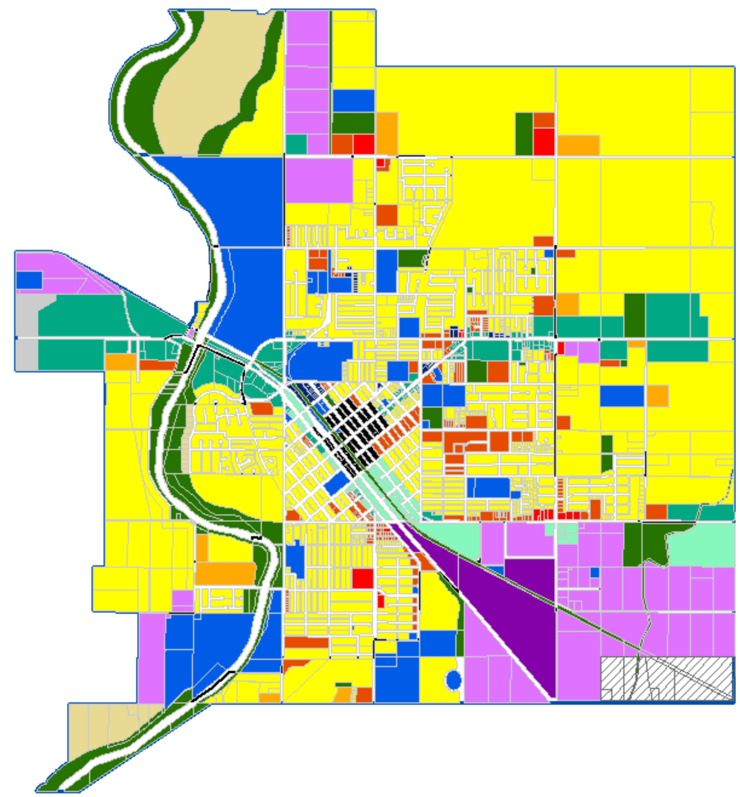
Figure ES-1. Future Land Use

Phase 1: Projected 2020 Land Use



Phase 2: Projected 2030 Land Use



Legend

Reedley Land Use Areas

- Suburban Residential
- Low Residential
- Medium Residential
- High Residential
- Neighborhood Commercial
- Community Commercial
- Service Commercial
- Office
- Central Downtown
- Public/Institutional Facility
- Light Industrial
- Heavy Industrial
- Open Space
- Buffer
- Future Development Area
- Reedley City Limits
- Reedley Proposed SOI

ES.3. Potable Water System

The City's existing potable water system includes approximately 82 miles of pipelines, two existing elevated storage tanks, and six operational wells. In addition, the Sports Park Water Tower and associated new well will be operational later this year.

The City depends on groundwater as its sole source of potable water supply. Groundwater is withdrawn from the Kings Groundwater Basin, a basin which has historically been in overdraft. As described in the City's General Plan 2030, the Reedley Municipal Code (RMC) has implemented regulations for the conservation of potable water including a reduction of water use and reduction of unnecessary use of potable water supplies. The RMC, coupled with the goals and policies of the General Plan 2030 and supporting plans, such as the City's 2010 Urban Water Management Plan, represent an effort by the City to effectively manage groundwater as a valued resource and to ensure the avoidance of a critical overdraft of the finite water resource.

Land use data from the City's General Plan 2030 was used to forecast potable water demands. The average daily demand for water is expected to grow from approximately 5.3 mgd under baseline conditions, to 6.8 mgd in Phase 1 and approximately 17.8 mgd in Phase 2, which corresponds to build out of the City's General Plan 2030 sphere of influence.

Figure ES-2 illustrates the average day demand (ADD) and the maximum day demand (MDD) in comparison to the City's existing groundwater well capacity and projects the supply deficit under the existing, Phase 1, and Phase 2 demand conditions.

Based on Figure ES-2, it is clear that additional groundwater wells will be required in the future to serve future growth. This Master Plan identifies potential locations for the recommended new wells for Phase 1 and Phase 2. The City should continue to track planned development and the associated demand, particularly for Phase 2, to better define the exact timing and location for new wells. In addition, future conservation efforts and implementation of a recycled water system could help to offset the need for new wells.

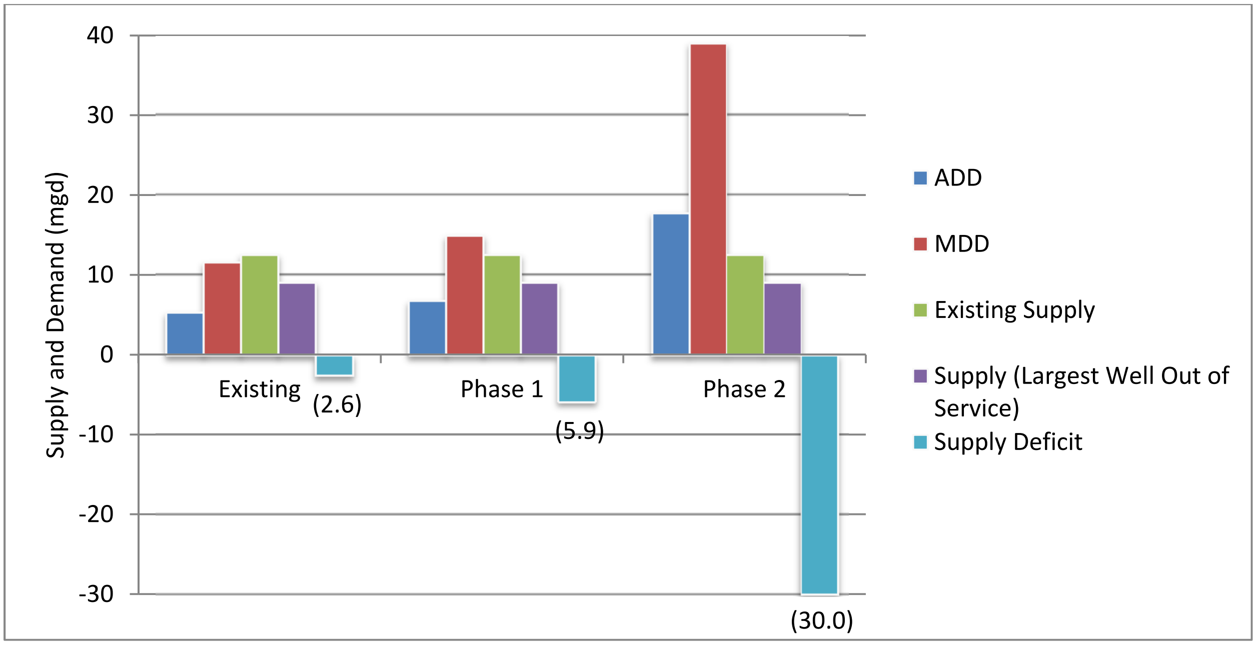


Figure ES-2. Water Supply and Demand Comparison

In addition to new wells, new water storage facilities and pipeline improvements will also be need to correct existing system deficiencies and to serve the future growth, as envisioned in the General Plan 2030.

The recommended improvements for the potable water system, including new wells, new storage tanks, and pipeline improvements, are illustrated in Figure ES-3 and Figure ES-4, for Phase 1 and Phase 2, respectively. As shown, the new supply and storage facilities were distributed throughout the potable water system to provide coverage across the system. The proposed well locations are intended to provide a general location only. Future studies will be required to identify actual locations and those studies should consider property acquisition needs, proximity to demands and other system facilities (e.g., storage), and proximity to other wells in order to minimize localized groundwater drawdown issues.

It is also important to note that in conducting water quality tests for the Sports Park Well, the City determined there was a TCP contamination plume below the site, and as a result GAC treatment was required for the new well. It is possible that future wells could have similar contamination issues; however, for the purposes of developing the cost estimates presented in this Master Plan, additional treatment has not been included. Additional studies will be required to determine if future wells will have similar requirements. In addition, the City should continue to monitor the development of the Chromium VI Rule to determine if additional treatment will be required.

Figure ES-3. Recommended Potable Water System Improvements through Phase 1

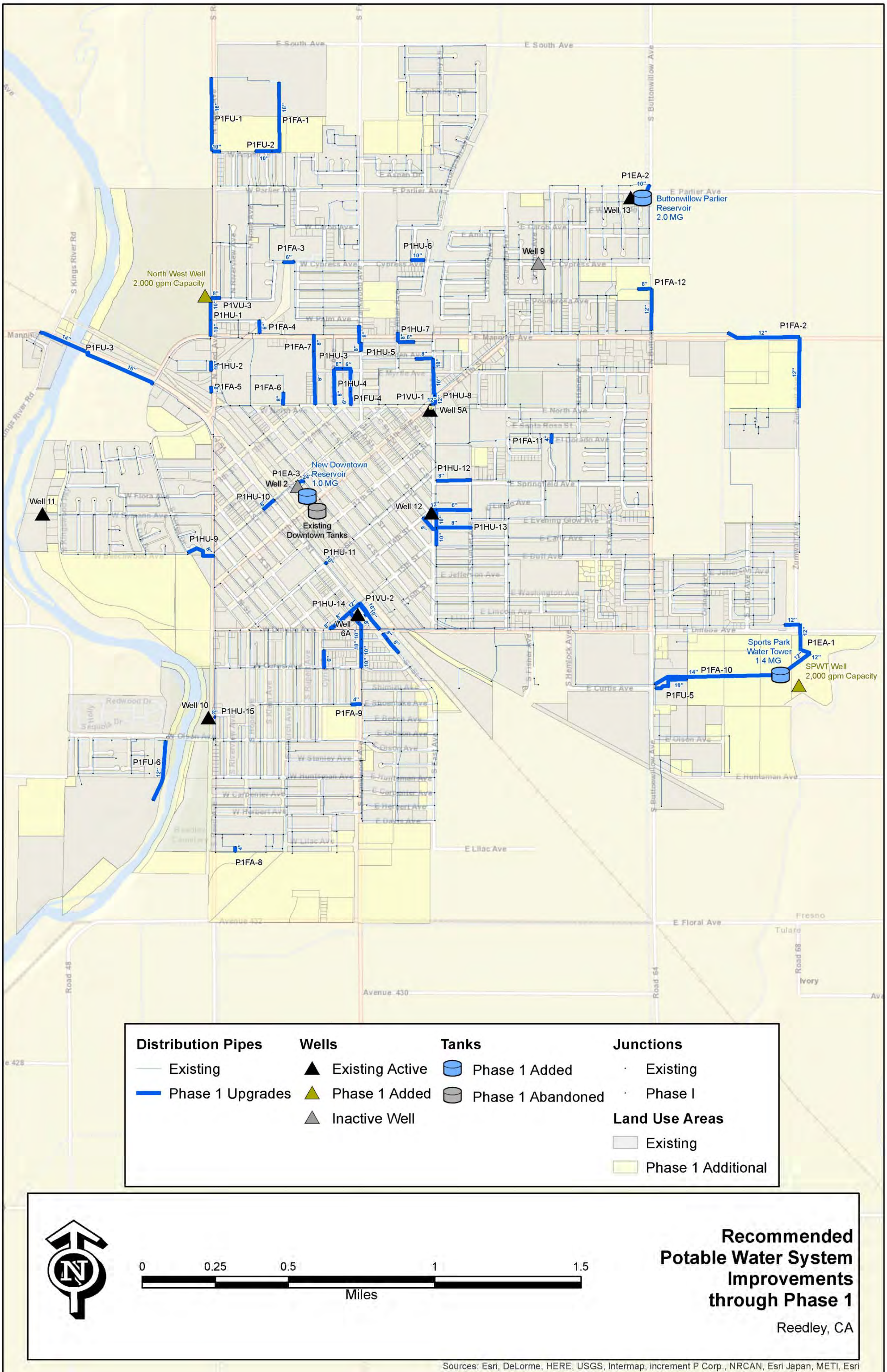
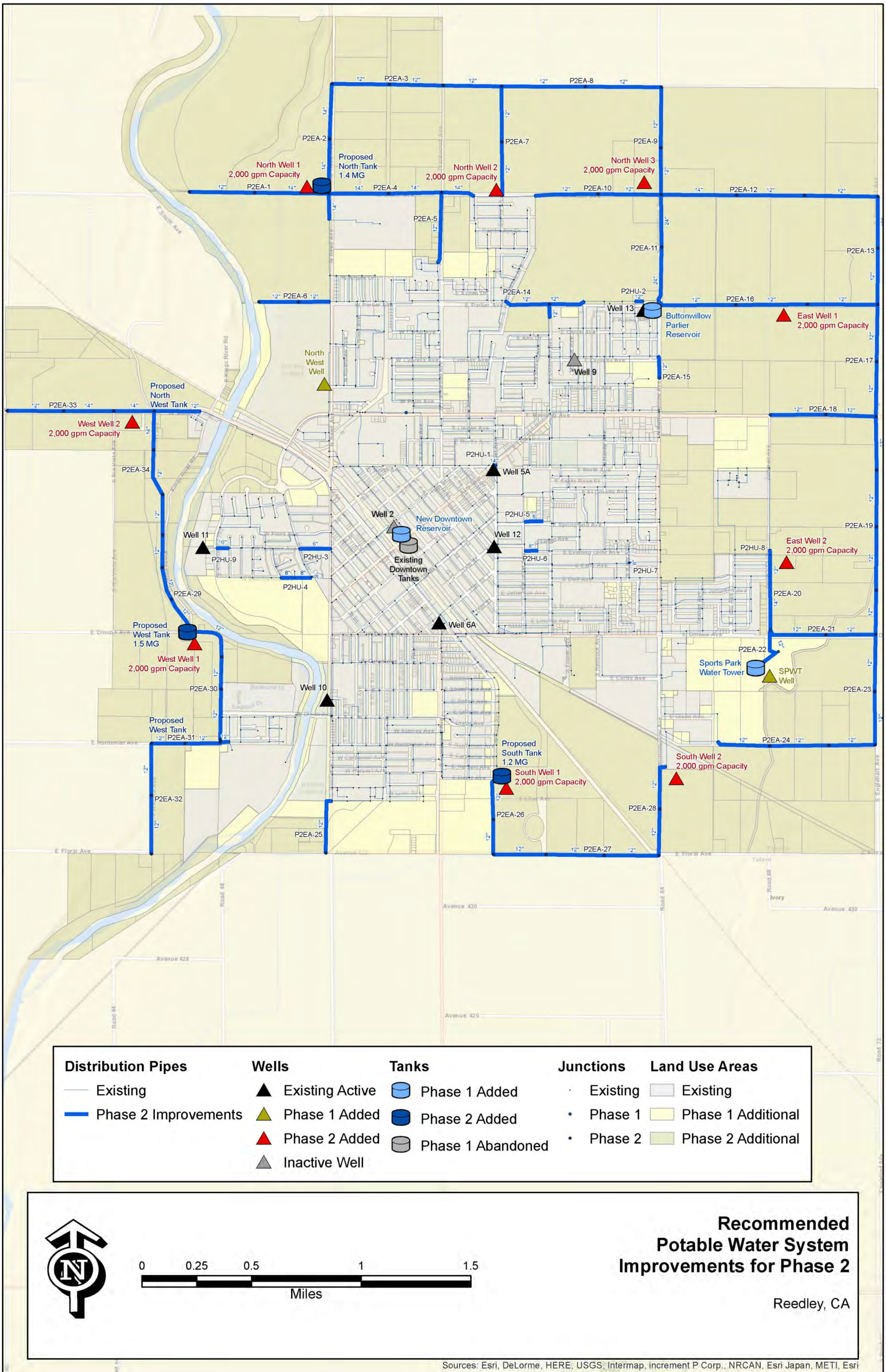


Figure ES-4. Recommended Potable Water System Improvements for Phase 2



ES.4. Sanitary Sewer System

The City's existing sanitary sewer system includes approximately 72 miles of pipelines, including both gravity mains and pressurized forcemains, and four existing lift stations.

Sewer flows were projected based on the projections of potable water demand, which, as described above, were based on the land use projections in the City's General Plan 2030. Based on the growth forecast in the General Plan 2030, total flows entering the sanitary sewer system are projected to grow from approximately 2.5 mgd under existing conditions, to approximately 3.2 mgd in Phase 1, and up to approximately 8.2 mgd at the buildout of Phase 2.

A hydraulic model of the City's sanitary sewer system was developed to evaluate hydraulic deficiencies under existing and future conditions. The recommended improvements for the City's sanitary sewer system are illustrated in Figure ES-5 and Figure ES-6, for the Phase 1 and Phase 2 planning periods, respectively.

Approximately 26,000 linear feet of pipeline are recommended to improve the existing system and an additional 11,000 linear feet are needed to accommodate the additional flows from Phase 1. These improvements are largely restricted to existing trunk mains to improve the system capacity and enable gravity flow to the City's wastewater treatment plant without backwater effects, surcharging, and sanitary sewer overflows.

The Phase 2 recommendations include upgrades to the existing system trunk mains in Reed Avenue, Manning Avenue and Columbia Avenue, as well as expansion beyond the existing system to serve the Phase 2 growth. As shown, trunk sewers, ranging from 12- to 30-inches, are planned for arterial roads. The sanitary sewer systems feeding into these trunk sewers would be constructed as part of future developments and are therefore not included. In total, approximately 120,000 linear feet of pipeline are recommended to serve the build out flows in Phase 2.

For the Existing system, priority for improvements has been given to downstream bottlenecks that result in backwater effects in the upstream pipelines. For these areas, improvements should be prioritized from downstream to upstream.

Figure ES-5. Recommended Sanitary Sewer System Improvements through Phase 1

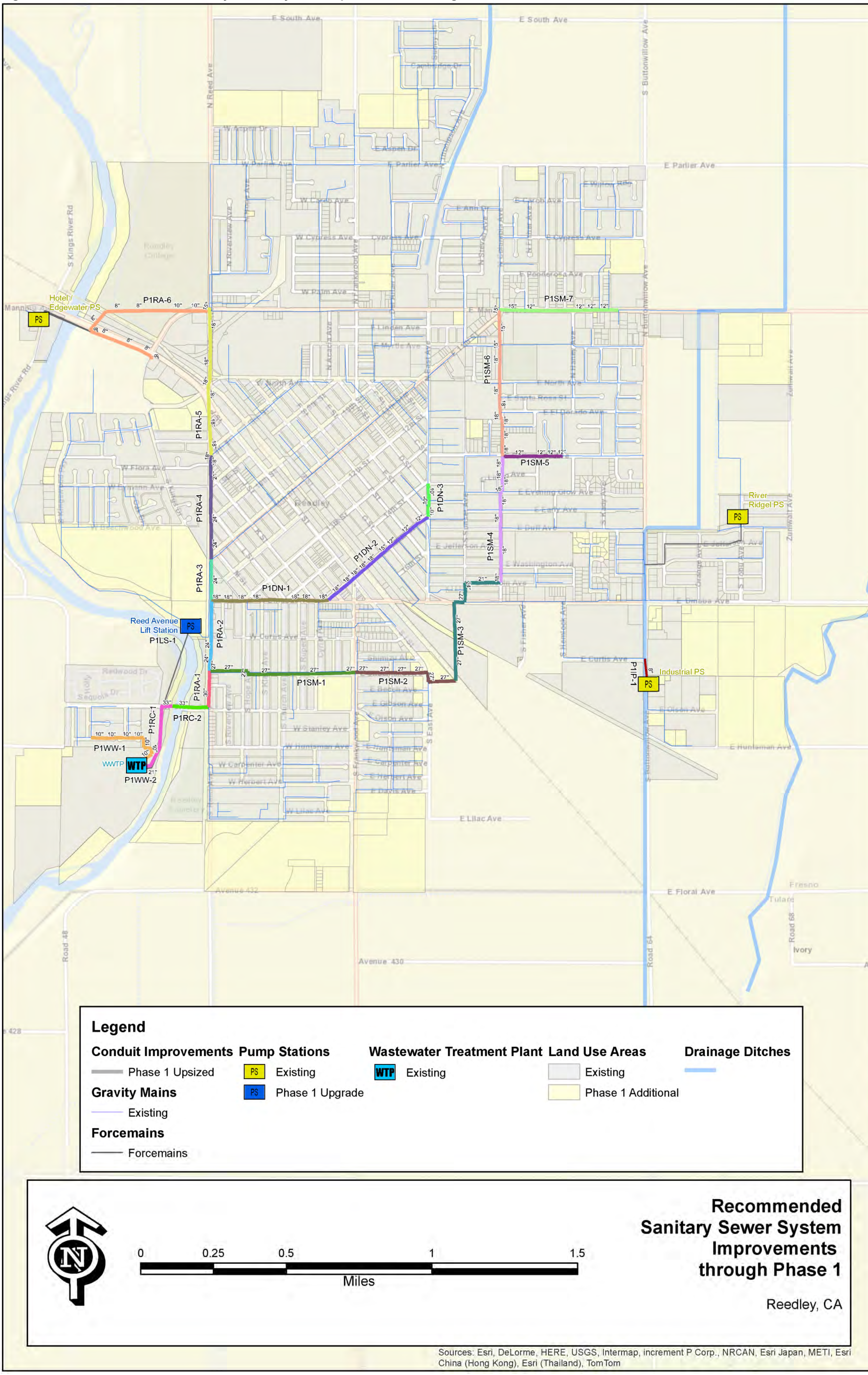
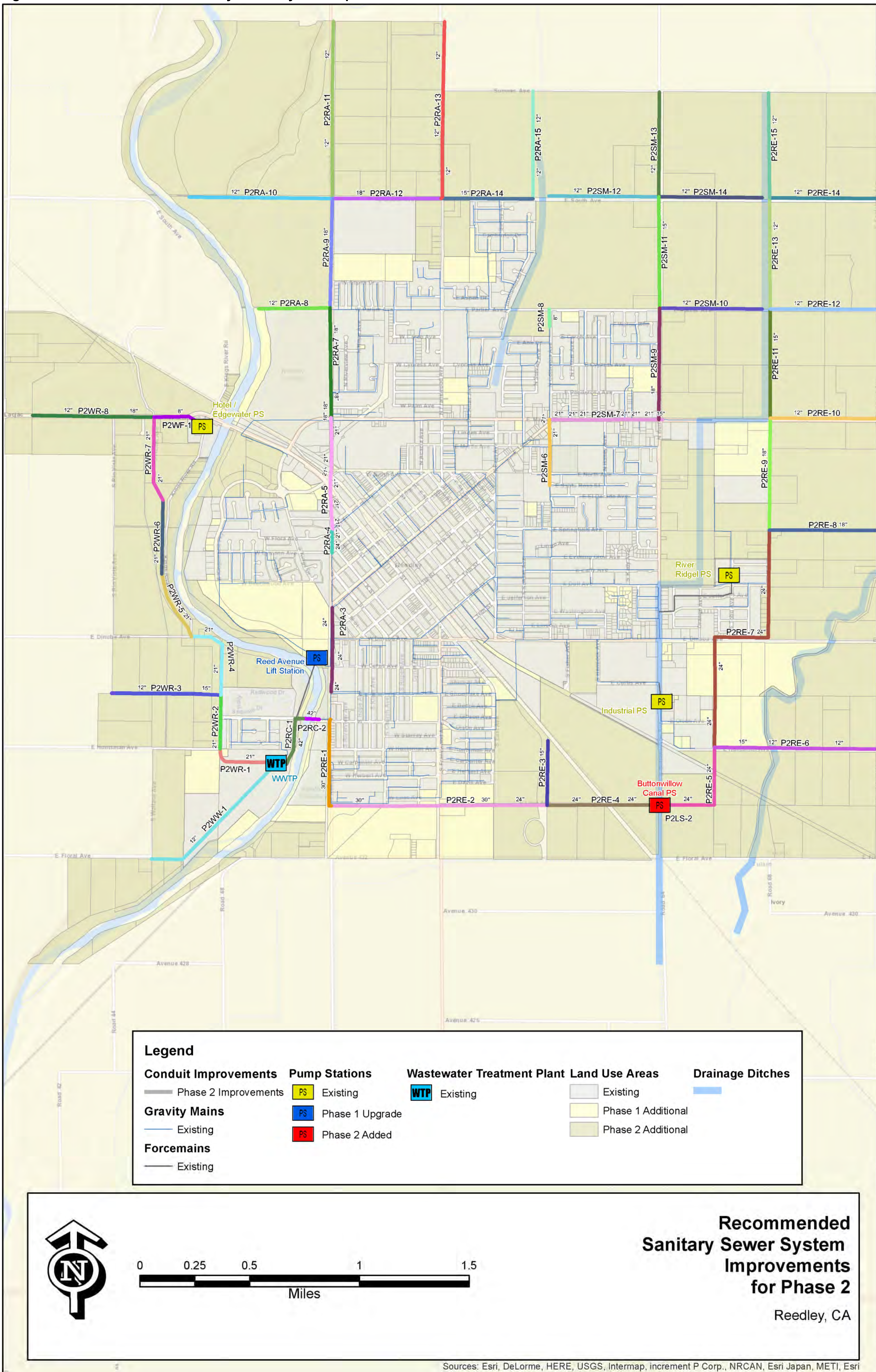


Figure ES-6. Recommended Sanitary Sewer System Improvements for Phase 2



ES.5. Storm Drainage System

The City's existing storm drainage system is divided into 17 sub-basins. The existing system also includes 13 outfalls, of which ten outfalls discharge directly to the Kings River and three discharge to an Alta Irrigation District drainage canal. The drainage system includes nearly 150,000 linear feet of pipeline, or 28 miles. In addition, the system includes three lift stations and ten storage facilities. Seven of these storage facilities are retention basins that collect runoff water and rely on infiltration to dispose of stormwater.

The existing storm drainage system drains approximately 2,590 acres. As development occurs, the system will expand to drain approximately 550 additional acres in Phase 1 and an additional 3,550 acres in Phase 2.

A hydraulic model of the City's storm drainage system was developed to evaluate hydraulic deficiencies under existing conditions and to plan for new drainage system infrastructure to accommodate growth in the future. The recommended improvements for the City's storm drainage system are illustrated in Figure ES-7 and Figure ES-8, for the Phase 1 and Phase 2 planning periods, respectively.

Approximately 23,000 linear feet of pipeline are recommended to improve the existing system and an additional 12,000 linear feet are needed to accommodate the additional flows from Phase 1. In addition, new retention basins are needed for Basins L, P and Q.

To serve the development for Phase 2, approximately 77,000 linear feet of pipeline is recommended in five new, separate storm drainage collection basins. Each basin would have its own retention basin as well.

Figure ES-7. Recommended Storm Drainage System Improvements through Phase 1

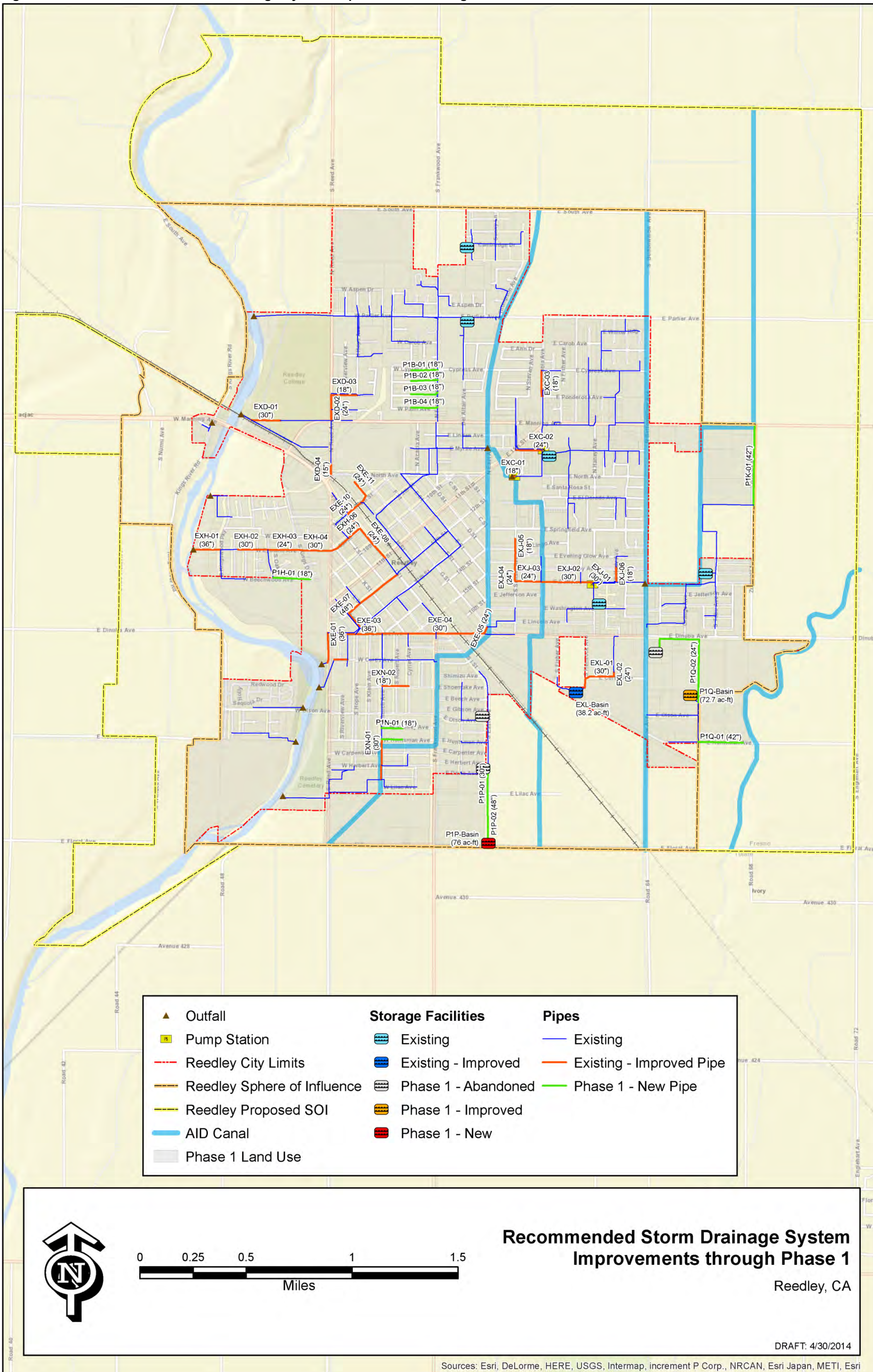
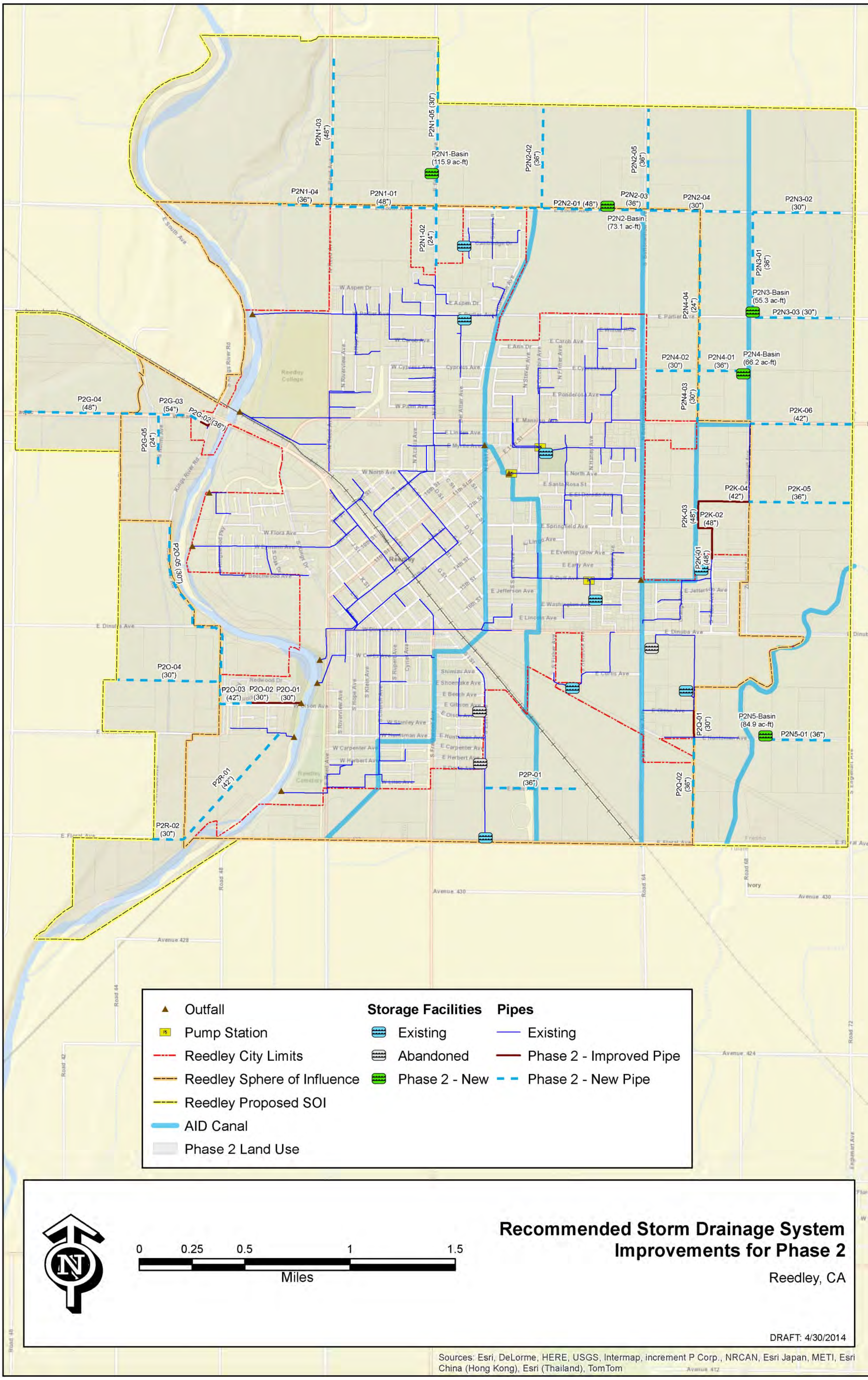


Figure ES-8. Recommended Storm Drainage System Improvements for Phase 2



ES.6. Recommended CIP

Table ES-1 presents a summary of the costs for the recommended projects for the potable water, sanitary sewer, and storm drainage system components. All costs are presented in 2014 dollars.

Table ES-1. Summary of Recommended Combined CIP

System[(a)]	Phase			Total
	Existing	Phase 1	Phase 2	
Potable Water				
Wells	39,000	2,555,000	22,995,000	25,589,000
Storage Tanks	5,510,000	4,057,000	13,538,000	23,105,000
Pipelines	6,948,000	1,431,000	35,674,000	44,053,000
Subtotal	12,497,000	8,043,000	72,207,000	92,747,000
Sanitary Sewer				
Lift Stations	64,000	0	2,134,000	2,198,000
Pipelines	21,923,000	5,914,000	68,101,000	95,938,000
Subtotal	21,987,000	5,914,000	70,235,000	98,136,000
Storm Drainage				
Retention Basins	1,648,000	10,457,000	29,899,000	42,004,000
Pipelines	28,154,000	12,332,000	97,928,000	138,414,000
Subtotal	29,802,000	22,789,000	127,827,000	180,418,000
Total	64,286,000	36,746,000	270,269,000	371,301,000

(a) All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

As summarized, the potable water system improvements total approximately $92.7 million and the sanitary sewer improvements total approximately $98.1 million, while the recommended storm drainage improvements total approximately $180.4 million, nearly double that of the water and sewer systems.

A time-phased CIP was prepared based on the projects that address needs or deficiencies in the potable water, sanitary sewer, and storm drainage systems and a 15-year implementation schedule of the projects was then developed for those projects needed to address existing deficiencies as well as Phase 1 deficiencies. Due to the uncertainty of the timing associated with future development in Phase 2, the projects needed to serve Phase 2 growth have not been scheduled.

Generally, projects needed to address existing system deficiencies were scheduled for implementation within the ten-year planning horizon between 2015 and 2025, while those projects required to serve Phase 1 growth were scheduled for implementation between 2025 and 2030. In addition, for the City's sanitary sewer and storm drainage systems, downstream pipeline improvements have been scheduled for improvement before their upstream tributary pipelines to avoid bottlenecks.

Table ES-2 presents a summary of the projected annual cash flow for the 15-year CIP. As summarized, the annual cash flow ranges from a minimum of $2.2 million in 2015 to a high of over $8 million, with an average annual projected cash flow of approximately $6.7 million.

Table ES-2. Summary of Annual Cash Flow for the 15-Year Combined CIP

Year	System[a]			Total
	Potable Water	Sanitary Sewer	Storm Drainage	
2015	985,000	574,000	667,000	2,226,000
2016	2,156,000	2,196,000	3,308,000	7,660,000
2017	1,144,000	1,275,000	3,097,000	5,516,000
2018	1,413,000	2,875,000	3,115,000	7,403,000
2019	2,217,000	1,686,000	4,509,000	8,412,000
2020	2,486,000	2,840,000	3,020,000	8,346,000
2021	1,427,000	2,511,000	3,725,000	7,663,000
2022	2,277,000	2,664,000	3,516,000	8,457,000
2023	996,000	1,765,000	5,155,000	7,916,000
2024	435,000	1,900,000	5,566,000	7,901,000
2025	1,523,000	1,591,000	5,685,000	8,799,000
2026	3,272,000	804,000	2,834,000	6,910,000
2027	165,000	1,003,000	2,007,000	3,175,000
2028	9,000	2,031,000	3,285,000	5,325,000
2029	35,000	2,186,000	3,102,000	5,323,000
Average	1,369,000	1,860,000	3,506,000	6,735,000

(a) All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

ES.7. Next Steps

The following subsections describe the next steps in implementing the Master Plan recommendations, including engineering, environmental compliance and permitting, coordination with ongoing projects and programs, financing, and continued use of Master Plan tools.

Engineering

The technical work completed for this Master Plan provides a framework for the recommended improvements to the potable water, sanitary sewer, and storm drainage facilities previously described in this chapter. The locations and pipeline alignments of these new facilities, shown in Figures ES-3 through ES-8, are preliminary and final locations should be determined during predesign work.

The purpose of the predesign studies is to finalize locations and alignments, refine design criteria and sizing, identify land requirements, evaluate operational requirements, and update cost estimates. Following completion of predesign studies additional engineering will include design, construction management, testing and startup.

Many of the proposed improvements will be phased and the engineering work should be scheduled accordingly. Construction contract packaging should be evaluated to provide the greatest opportunities for competitive bidding by contractors.

In addition, there are some common corridors in which water, sewer, and/or storm drainage pipeline projects are needed. Where appropriate, and as financially feasible, such projects in common corridors should be designed and packaged together to provide greater economies of scale. In addition, opportunities to leverage other capital improvement programs such as pavement renewal projects or parks improvements should be coordinated with the recommendations in this Master Plan to take advantage of economies of scale and minimize construction activities.

Environmental Compliance and Permitting

The recommended facilities will require compliance with the California Environmental Quality Act (CEQA) and possibly the National Environmental Policy Act (NEPA) to evaluate the environmental impacts of the specific projects. The required environmental compliance documents should be completed in conjunction with the engineering preliminary design studies.

Numerous federal, state and local permits will also be required for project implementation. The required permits will be identified during the preparation of the engineering preliminary design studies and environmental compliance documents. A permitting strategy should be developed to minimize project delays and potential mitigation costs.

Coordination with Ongoing Projects and Programs

Implementation of the Master Plan should be coordinated with other ongoing projects and programs. Specifically, the Master Plan should be coordinated with the following:

- Water Conservation Program
- Kings Groundwater Basin Management
- Asset Renewal and Replacement
- Sewer System Management Plan under the State's General WDR Permit
- Storm Water Management Plan under the State's General MS4 Permit

Financing

The estimated capital costs by phase were summarized in Table ES-1. The recommended facilities should be incorporated into the City's five-year capital improvement program in accordance with the proposed phasing plan. Specific project financing, including escalation, can then be addressed as part of the City's regular budgeting, rates, and facility capacity/connection fee program updates.

Project costs associated with the expansion of the existing systems to accommodate future growth, particularly for Phase 2, should be included in the City's facility capacity/connection fees such that future growth pays for the respective facilities they need.

Use of Master Plan Tools

The City has invested substantial resources in the completion of this Master Plan. The tools developed as part of this work should be utilized in the future evaluation of proposed new developments, proposed land use changes, refinements to the recommended facilities, and potential regional projects and programs. Some of the tools to be utilized by the City include the following:

- Planning criteria established for evaluation of facilities
- Potable water distribution system hydraulic model

- Sanitary sewer collection system hydraulic model
- Storm drainage collection system hydraulic model

Future Updates

The recommendations presented in this Master Plan include infrastructure upgrades to improve the existing system as well as to accommodate future growth as envisioned in the City's General Plan 2030. The recommendations represent a substantial CIP, particularly to accommodate the growth anticipated in Phase 2. As such, the City should regularly evaluate actual system conditions, including the number of new connections per year, conservation savings, development of recycled water, and other changes that may impact the growth in the potable water demand, and the generation of sanitary sewer flows and storm water runoff. Based on these regular updates, the annual CIP should be adjusted as needed. The City should also prepare a formal update to this Master Plan in approximately five years.

1. Introduction

The City of Reedley (City) has developed an Integrated Master Plan for Potable Water, Sanitary Sewer and Storm Drainage Systems (Master Plan). This Master Plan supplements the City's *General Plan 2030,* adopted in February 2014, by addressing how the City's existing water-related utilities will be upgraded and new infrastructure will be installed in order to provide desired levels of service for both the City's existing residents and expected future development. This Master Plan replaces prior plans for the three water-related utility systems.

1.1. Background

The City is located along the Kings River in the central San Joaquin Valley portion of California, lying just inland between the State's coastal mountain ranges and the Sierra Nevada Mountains. The City is situated approximately 25 miles southeast of the City of Fresno and 20 miles northwest of Visalia. The City covers approximately 5-square miles and serves a population of approximately 24,000 through about 6,000 active water service connections[1].

Civil war hero Thomas Law Reed settled in Reedley to provide wheat for Gold Rush miners in the mid 1800's. His donation of land for a railroad station site established the town as the center of the Valley's booming wheat business. Railroad officials commemorated his vision by naming the fledgling City in his honor. When mining fever began to fade, wheat demand slackened. Kings River water was diverted for crop irrigation, and the region began its over 100-year tradition of bountiful field, tree, and vine fruit harvests.

With water and railroad services in place, farming families of European immigrants were recruited, and the settlement was incorporated in 1913, with Ordinance No. 1 adopting and prescribing the style of a Common Seal on February 25, 1913.

The valley floor is still the richest intensive agricultural production area in the world. The City's economy continues to be predominantly based upon agricultural production and agriculturally-oriented industry. Although it has diversified in recent years, the local economy continues to be significantly dependent upon the underlying agricultural character of the region.

[1] Based on Department of Water Resources Public Water System Statistics report submitted in 2011.

Since 1946, the City has been known as the Fruit Basket of the World because it leads the nation in the shipping of fresh fruit. Thirty fruit and vegetable packing and cold storage facilities, including the world's largest plant, along with nearby wineries, supply tree and vine fruit products of consistently high quality. Related manufacturing industries in the City include boxes and packing machinery, and automatic packing equipment.

The Council-Manager form of government administers a general fund operating budget of over $4,700,000 with a total budget in excess of $15,000,000. The City has had a Planning Commission since the 1940's and provides a full range of City services including a municipal airport, water system, sewer plant, and trash collection.

1.2. Scope of Work

Detailed technical and economic analyses were completed to develop this Master Plan. This Master Plan was originally conceived as two separate contracts. The first contract envisioned only the water system analysis. However, as part of a second contract, analysis of both the sanitary sewer collection and storm drainage systems were added.

The following tasks and subtasks comprise the Scope of Services for this Master Plan:

- Water System
  - Data Collection and Inventory of Existing Facilities
  - Potable Water Planning Criteria
  - Projected Water Supply and Demand Requirements
  - Water Supply Alternatives
  - Existing Hydraulic Model Update
  - Water Model Validation
  - Water System Infrastructure Evaluation
  - Water System Master Plan Preparation
- Sewer Collection System
  - Data Collection and Inventory of Existing Facilities
  - Design Standards
  - Flow Monitoring
  - Current and Future Flow Projections
  - Hydraulic Model Development
  - Lift Station Assessment

- Existing and Future Deficiencies Analysis
- Capital Improvement Program
- Sewer System Master Plan Preparation
- Storm Drainage System
  - Data Collection and Inventory of Existing Facilities
  - Runoff Calculation Procedures and Design Standards
  - Hydraulic Model Development
  - Existing and Future Deficiencies Analysis
  - Capital Improvement Program
  - Storm Drainage Master Plan Preparation

1.3. Report Organization

This Master Plan provides a summary of pertinent background information, an evaluation of existing facilities and future needs, and a recommended plan for the potable water, sanitary sewer, and storm drainage systems. Separately bound appendices to this report provide additional background information, detailed technical and economic data, and further documentation for conclusions and recommendations.

This Master Plan is organized into the following sections:

- **Executive Summary.** The Executive Summary precedes Section 1 for use in communicating the Master Plan results. This section provides a high level summary of the Master Plan contents, including the CIP recommendations and costs.

- **Section 1.0 Introduction.** This section provides an overview of the Master Plan background and scope of services.

- **Section 2.0 Basis of Planning.** This section provides details of the study area and land use, planning period, population forecast, and the basis of cost estimates.

- **Section 3.0 Potable Water System.** This section provides a summary of the existing potable water system, water demand forecasts, regulatory requirements, system design criteria, water supply alternatives evaluation, and an analysis of the existing system under current and future conditions and the corresponding recommendations to overcome system deficiencies.

- **Section 4.0 Sanitary Sewer System.** This section provides a summary of the existing sanitary sewer system, wastewater flow generation forecasts, regulatory requirements, system design criteria, and an analysis of the existing system under current and future conditions and the corresponding recommendations to overcome system deficiencies.

- **Section 5.0 Storm Drainage System.** This section provides a summary of the existing storm drainage system, rainfall/runoff forecasts, regulatory requirements, system design criteria, and an analysis of the existing storm drainage system under current and future conditions and the corresponding recommendations to overcome system deficiencies.

- **Section 6.0 Capital Improvement Plan.** This section presents the combined capital improvement plan for the potable water, sanitary sewer, and storm drainage systems as well as the estimated costs and next steps necessary to implement the recommended upgrades.

- **Appendices (Bound Separately).** Appendices A through J include references, technical memoranda, and other relevant data and background information.

1.4. Project Team

This Master Plan was developed by staff from HDR Engineering, Inc. and ADS Environmental (for flow monitoring services), and with support from the City's Community Development and Public Works Departments. Plan development was coordinated closely with development of the *General Plan 2030*.

1.5. Abbreviations

To conserve space and improve the text, the following abbreviations have been used in this Master Plan:

AAF	average annual flow
ac	acre
ac-ft/yr	acre-feet per year
ac-ft	acre-feet
ADD	average day demand
ADWF	average dry weather flow
AID	Alta Irrigation District

BSF	base sanitary flow
CCR	Consumer Confidence Report
CEQA	California Environmental Quality Act
cfs	cubic feet per second
CID	Consolidated Irrigation District
City	City of Reedley
City Council	Reedley City Council
CMOM	capacity, management, operation and maintenance
County	Fresno County
Cr(VI)	hexavalent chromium
CWA	Clean Water Act
d/D	ratio of water depth to pipeline diameter
DBCP	dibromochloropropane
DBPR	Disinfection and Disinfectant Byproducts Rule
DEM	digital elevation model
DPH	California State Department of Public Health
DWR	California State Department of Water Resources
EIR	Environmental Impact Report
ENR	Engineering News Record Construction Cost Index
fps	feet per second
ft	feet
GAC	granular activated carbon
gal	gallon
GIS	geographic information system
GMP	Groundwater Management Plan
gpd	gallons per day
gpm	gallons per minute
GWI	groundwater infiltration
GWR	Ground Water Rule
HAA5	haloacetic acids
HDR	high density residential
HGL	hydraulic grade line
hp	horsepower
hr	hour
IDSE	Initial Distribution System Evaluation
I/I	inflow and infiltration
IESWRT	Interim Enhanced Surface Water Treatment Rule

in	inch
IOC	inorganic chemicals
LCR	Lead and Copper Rule
LDR	low density residential
LOS	level of service
LT2ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
Master Plan	Integrated Master Plan for Potable Water, Sanitary Sewer and Storm Drainage Systems Master Plan
MCL	maximum contaminant level
MCLG	maximum contaminant level goals
MDR	medium density residential
MDD	maximum day demand
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
mrem	millirems
MS4	Municipal Separate Storm Sewer System
n	Manning's coefficient
NOM	natural organic matter
NPDES	National Pollution Discharge Elimination System
NRCS	National Resource Conservation Service
NTU	nephelometric turbidity unit
O&M	operation and maintenance
PDWF	peak dry weather flow
PHD	peak hour demand
PNR	Public Notification Rule
psi	pounds per square inch
PWWF	peak wet weather flow
q/Q	ratio of flow in a pipeline compared to pipeline flow capacity
RDI	rainfall-dependent infiltration
RDI/I	rainfall-dependent inflow and infiltration
RMC	Reedley Municipal Code
RWQCB	California Regional Water Quality Control Board, Central Valley Region
SDWA	Safe Drinking Water Act
SOC	synthetic organic chemicals

SOI	Sphere of Influence
SPWT	Sports Park Water Tower
SSMP	Sewer System Management Plan
SSO	sanitary sewer overflow
State	State of California
SWI	storm water inflow
SWMP	Storm Water Management Plan
SWRCB	State Water Resources Control Board
SWTR	Surface Water Treatment Rule
TCP	trichloroproprane
TCR	Total Coliform Rule
TDS	total dissolved solids
TM	Technical Memorandum
TMDL	Total Maximum Daily Load
TTHM	trihalomethane
USBR	United States Bureau of Reclamation
USEPA	U.S. Environmental Protection Agency
UWMP	Urban Water Management Plan
VOC	volatile organic chemicals
WDR	Waste Discharge Requirements
WTP	water treatment plant
WWTP	wastewater treatment plant
yr	year

2. Basis of Planning

This section of the Master Plan summarizes the basic planning considerations that are common to all three of the systems analyzed. More specific data for each system (potable water, sewer collection, and storm drainage) can be found in the sections that follow.

The basis of planning includes assumptions with respect to the Study Area, planning period, land use, population, and cost estimates.

2.1. General Plan 2030

The City recently adopted The City of Reedley General Plan 2030 in late February, 2014. Every city in California is required to have an active general plan, which serves as a blueprint for future development and describes the City's development goals and policies. A general plan serves to:

- Identify the community's land use, circulation, environmental, economic, and social goals and policies as they related to land use and development.
- Provide a basis for local government decision-making, including decisions on development approvals and exactions.
- Provide citizens with opportunities to participate in the planning and decision-making processes of their communities.
- Inform citizens, developers, decision-makers, and other cities and counties of the ground rules that guide development within a particular community.

The Reedley General Plan 2030 provides a critical foundation for planning of the City's utility infrastructure. Decisions on where to expand the water, sewer and storm drainage systems are made with both current and future needs in mind. This Master Plan includes an analysis of utility system conditions and needs, first in terms of current conditions, and then in terms of planned future conditions. The planned future conditions are based on the Land Use Element of the General Plan 2030, as described in Subsection 2.4. As such, the recommended improvements contained within this Master Plan are consistent with the General Plan 2030 and its associated programmatic Environmental Impact Report (EIR), which was also adopted by the City in February 2014.

2.2. Study Area

The City is located adjacent to the Kings River in Fresno County, California. Figure 2-1 displays the City's existing City limits and current, adopted land uses and planned Sphere of Influence (SOI).

The Study Area for this Master Plan consists of lands that are both currently served and those which are planned for future development that may require municipal water, sewer, and storm drainage services. The Study Area includes the SOI for the City's General Plan 2030.

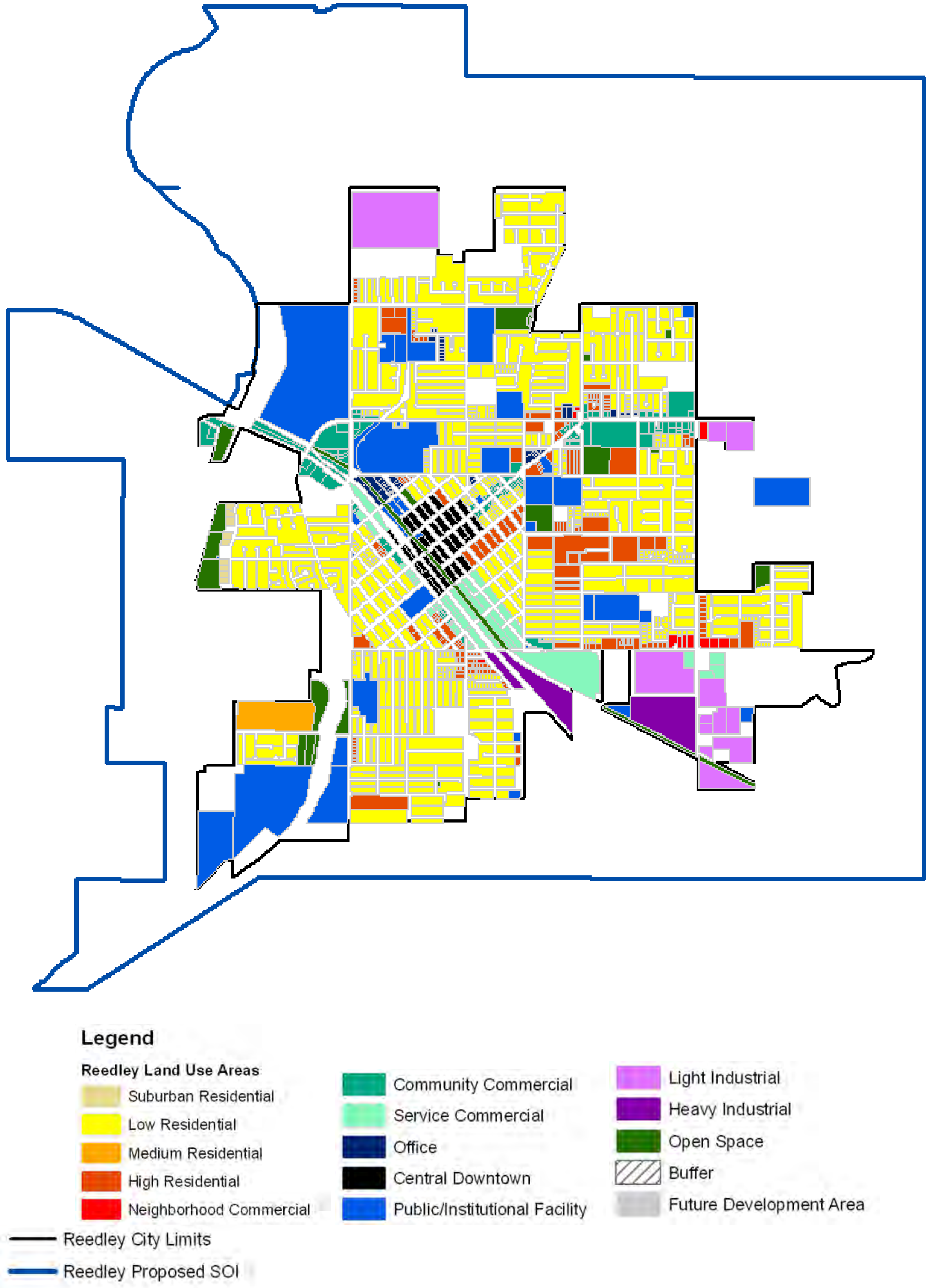
2.3. Planning Period

The planning period for this Master Plan extends from the baseline through build out of the General Plan horizon, which is envisioned as 2030. Since the Master Plan was prepared over the course of multiple years, the planning data used to characterize the "existing" system relied on information from 2008 to 2013. Therefore, the "existing" conditions referred to in the Master Plan are not a snap shot in time, but rather represent an average baseline for which deficiencies and future improvements have been identified in the capital improvement plan.

The facilities required to serve the build out demand for each system will be time-phased to correspond with projected growth as it occurs. Due to the uncertainty around the timing of the actual build out of the General Plan 2030, the following phases have been established for evaluation of the respective systems:

- Existing Phase: Baseline Conditions
- Phase 1: Period between Existing (Baseline) and 2020
- Phase 2: Period beyond 2020 through build out of the City's SOI

Figure 2-1. Existing Land Use



While the system will be analyzed for the phases above, recommended capital improvement projects (CIP) will be prioritized and phased over time such that improvements required to address existing deficiencies will occur earlier in Phase 1, while improvements needed to address deficiencies associated with future development will be phased over time to occur as development is expected.

2.4. Land Use

The objective of this Master Plan is to identify the recommended water, sewer collection, and storm drainage infrastructure needed to serve the future growth defined by the City's General Plan 2030. The fundamental planning basis for developing water demands, projected sanitary sewer flows, and storm water runoff is the planned land use presented in the General Plan.

Figure 2-2 displays planned future land uses and the City's SOI, as envisioned in the General Plan. As previously described, an interim planning period, Phase 1, was developed to reflect the near term development that is anticipated in the City between now and approximately 2020. Phase 2 corresponds to build out of the City's General Plan 2030. As development occurs in the future, the City anticipates the City limits will expand and utility services will be extended as necessary to serve the new development beyond the City's current limits.

Table 2-1 presents the acreages planned for different land uses, as proposed in the General Plan 2030. As will be presented in later sections, these planned future land uses and acreages are used to project expected future needs in terms of quantity of potable water delivered to customers, sewer flows received from customers, and storm water runoff from different categories of developed land. Those projections are described in Chapters 3, 4 and 5, respectively.

Figure 2-2. Future Land Use

Phase 1: Projected 2020 Land Use

Phase 2: Projected 2030 Land Use

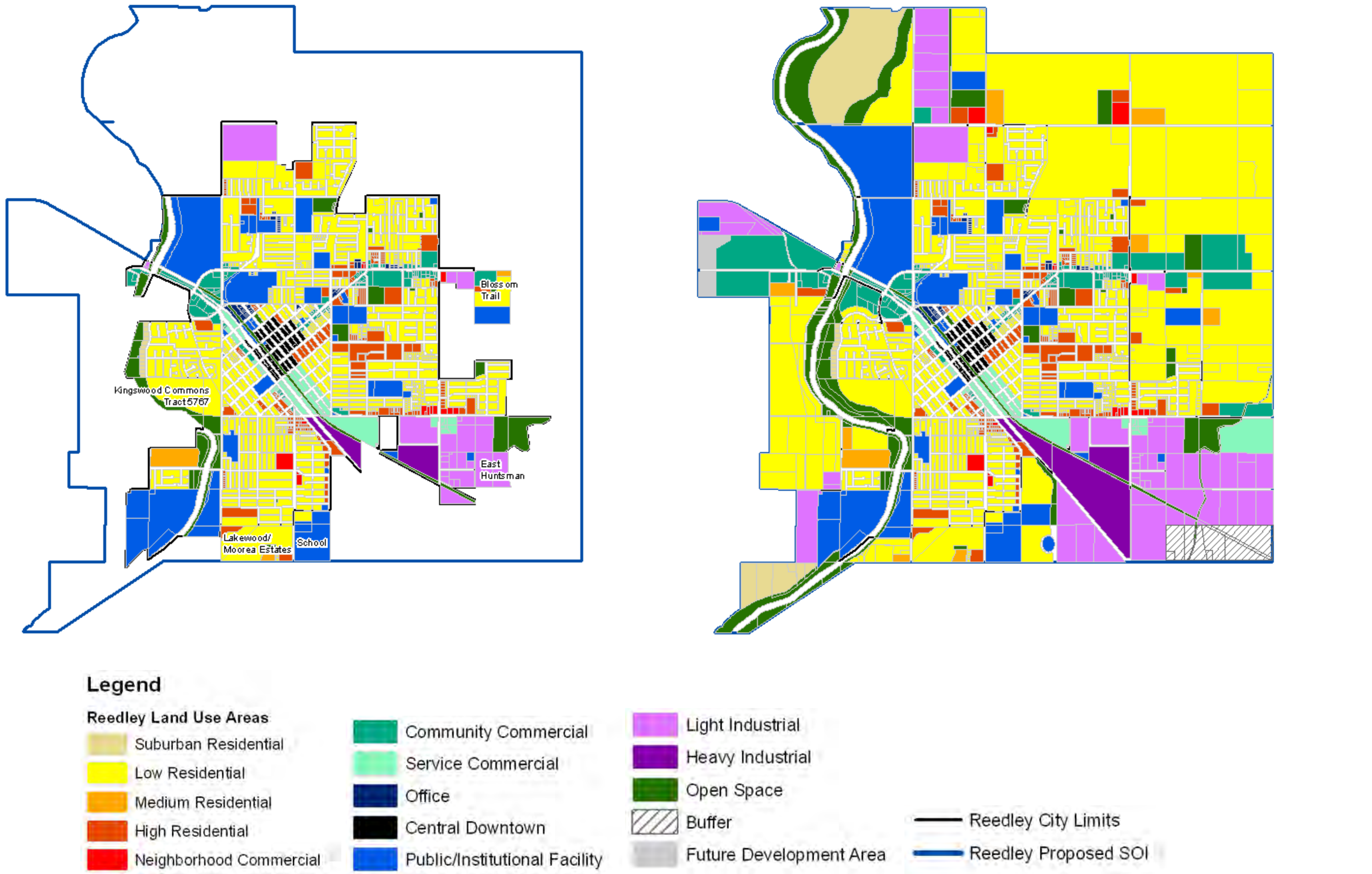


Table 2-1. Existing and Projected Land Uses

General Plan Land Use	Existing Land Area (acres)	Projected 2020 Land Area[(a)] (acres)	Projected 2030 Land Area[(b)] (acres)
Residential			
Suburban Residential	7	10	276
Low Density Residential (LDR)	959	1,181	4,075
Medium Density Residential (MDR)	27	35	111
High Density Residential (HDR)	156	202	251
Commercial and Industrial			
Central Downtown	40	40	40
Community Commercial	92	111	434
Neighborhood Commercial	11	23	44
Service Commercial	75	78	140
Office	16	17	17
Light Industrial	142	225	809
Heavy Industrial	54	54	179
Other			
Community Buffer	-	-	112
Open Space	96	188	636
Public/Institutional Facility	408	511	752
System-Wide Total	2,083	2,675	7,876

(a) Source of data: City Development and Public Works Department.
(b) Source of data: *General Plan 2030*.

As shown in Table 2-1, the City anticipates the addition of approximately 600 acres in Phase 1, the period between the baseline conditions and approximately 2020.

The projected 2030 land area presented in Table 2-1 is based on the General Plan 2030. However, when the City conducted the environmental review, three alternatives were considered. As described in the City's program EIR for the General Plan 2030, Alternative 3 was selected as the environmentally superior alternative. Alternative 3 includes fewer acres in the future SOI. At 7,057 acres, Alternative 3 has approximately 820 fewer acres of future development compared to the 7,876 presented in Table 2-1. As a result, the basis for the analysis of the potable water, sanitary sewer, and storm drainage systems under future (Phase 2) conditions is considered a worse case scenario since the selected environmentally superior alternative included less development.

2.5. Population

Growth in the City, mainly residential in nature, has averaged about 3 percent per year since World War II. The 1990s and early 2000s had a declining trend from 3.6 percent to 2.4 percent per year. Population estimates for the City's service area were developed for the City's 2010 Urban Water Management Plan (UWMP) in accordance with the State's Department of Water Resources (DWR) guidelines.

Census data for the City was used to determine the population for the existing service area. Table 2-2 shows the historical population within the incorporated City and service area.

Table 2-2. Historical Population

Year	Population[(a)]
1920	2,447
1930	2,589
1940	3,170
1950	4,135
1960	5,850
1970	8,131
1980	11,071
1990	15,791
1995	18,757
2000	20,756
2005	21,447
2010	24,194
2011	24,407

(a) Source of data: *City of Reedley 2010 Urban Water Management Plan.*

Table 2-3 presents the estimated population for the Study Area for 2010 through 2030. The projected population is based on an estimated growth rate of 3 percent per year for a 20 year projection period, and is consistent with future population growth rates presented in the City's General Plan 2030.

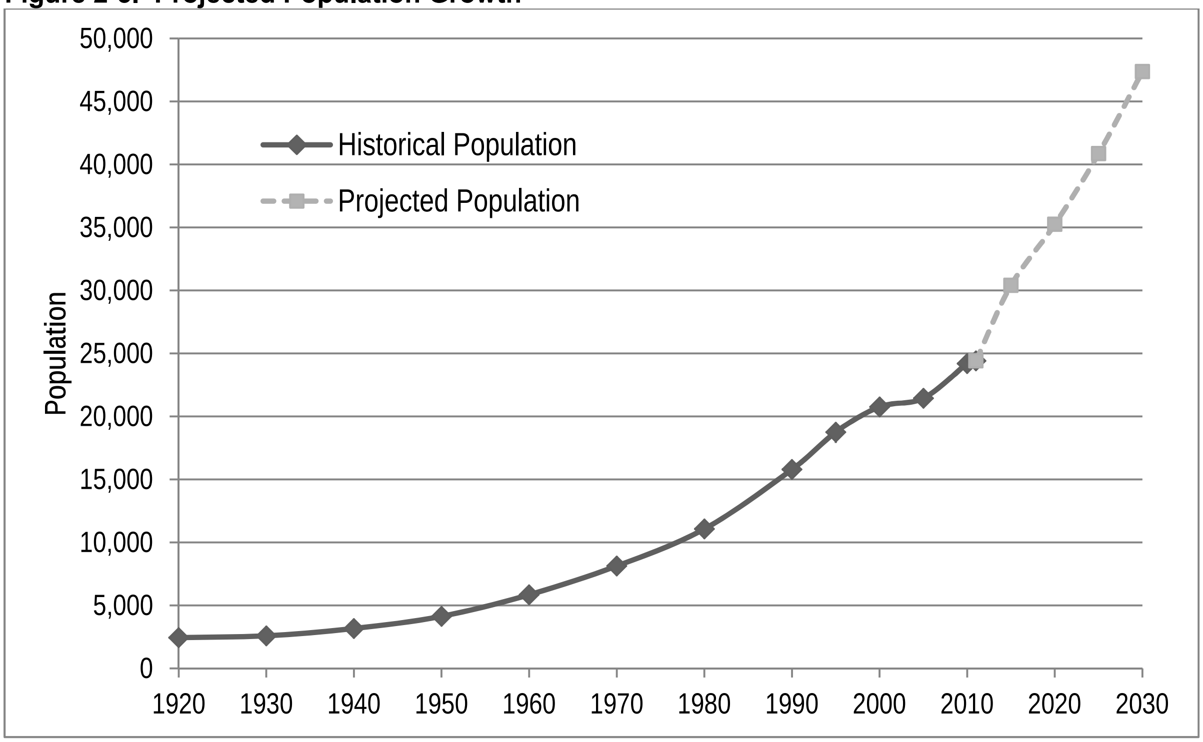
Table 2-3. Current and Projected Population

Year	Population[(a)]
2010	24,194
2015	30,404
2020	35,247
2025	40,861
2030	47,369

(a) Source of data: *City of Reedley 2010 Urban Water Management Plan.*

The historical population and the projected population growth are also illustrated in Figure 2-3.

Figure 2-3. Projected Population Growth



2.6. Basis of Cost Estimates

Preliminary cost estimates were developed for the infrastructure needs identified in this Master Plan. Capital cost estimates were prepared by applying unit costs and cost curve data to the estimated quantities or capacities for proposed improvement projects. Allowances were added for contingency (30 percent) and engineering, administration, and permitting (25 percent).

Where possible, construction costs have been based on actual bid data. Where prior bid results are not available, new facility costs were developed using techniques deemed appropriate for a project planning-level cost estimate (i.e., +40 percent, -20 percent). Construction costs were limited to the following major facilities:

- Water System: wells, storage tanks, transmission and distribution pipelines, and supporting infrastructure (e.g., pumping stations) if needed.
- Sewer Collection System: collection pipelines, lift stations, and supporting infrastructure if needed.
- Storm Drainage System: collection pipelines, retention and detention basins, outfalls, and supporting infrastructure if needed.

All preliminary cost estimates have been adjusted to represent current dollars. The basis for the estimates presented in this Master Plan is the Engineering News Record (ENR) 20 Cities Construction Cost Index for January 2014, 9681.

3. Potable Water System

This section of the Master Plan describes the City's potable water sources and distribution system. It begins with an inventory of system components, provides information on current and future demands and supplies, as well as regulatory requirements that affect infrastructure requirements for the potable water system. It concludes with an analysis of the hydraulics of the water distribution system, under current and projected future conditions, as well as recommended improvements to overcome system deficiencies.

For information on the proposed capital improvement plan, see Section 6 of this Plan.

3.1. Inventory of Existing Water System

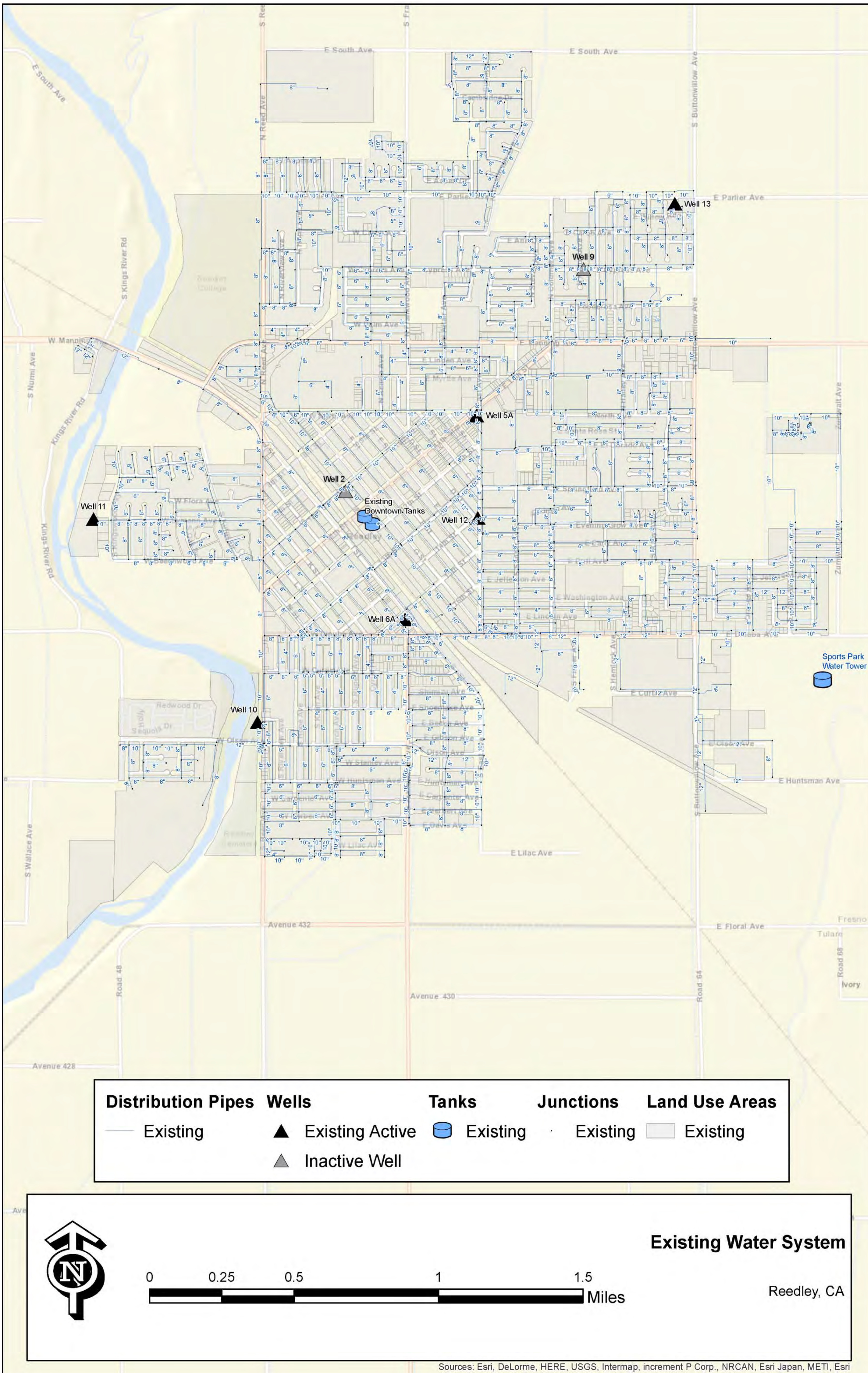
This section provides an inventory of the existing potable water distribution system. The primary data source was the H2OMap hydraulic model of the existing distribution system. The hydraulic model was developed by Boyle Engineering and was documented in the July 2006 *Computerized Hydraulic Model and System Analysis Report*.

The information contained in the system inventory in the hydraulic model was compared to the City's water system AutoCAD drawings and minor additions were incorporated to develop an updated system inventory. The existing system is shown in Figure 3-1.

The City's existing distribution system includes approximately 82 miles of pipeline. The sizes and lengths of distribution pipelines are summarized in Table 3-1.

Table 3-1. Existing Potable Water Distribution Pipe

Diameter (inches)	Length (feet)	Length (miles)
2	420	0.1
4	24,756	4.7
6	118,351	22.4
8	198,862	37.7
10	71,354	13.5
12	21,488	4.1
Total	435,231	82.4


Well 13
Well 9
Well 5A
Well 2
Existing Downtown Tanks
Well 12
Well 11
Well 6A
Well 10
Sports Park Water Tower
E South Ave
E Parlier Ave
E Huntsman Ave
E Lilac Ave
E Floral Ave
Avenue 432
Avenue 430
Avenue 428
S Buttonwillow Ave
N Reed Ave
S Kings River Rd
S Wallace Ave
Road 48
Road 64
Road 68
Fresno
Tulare
Ivory
Distribution Pipes
Existing
Wells
Existing Active
Inactive Well
Tanks
Existing
Junctions
Existing
Land Use Areas
Existing
Existing Water System
Reedley, CA
0
0.25
0.5
1
1.5
Miles
Sources: Esri, DeLorme, HERE, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), TomTom

The City's system has two existing elevated storage tanks. The tanks provide operational storage, help meet peak hour demands, and continuously pressurize the system depending on water levels in the tanks. In addition, a third elevated storage tank is under construction, the Sports Park Water Tower (SPWT). Characteristics of these tanks are summarized in Table 3-2.

Table 3-2. Existing Potable Water Storage Tanks

Location	Ground Elevation at Base (ft)	Low Water Elevation (ft)	High Water Elevation (ft)	Approx. Diameter (ft)	Volume[a] (gal)
10th & Reedley Parkway	346	453	463	21	25,900 / 50,000
10th & Reedley Parkway	346	453	463	21	25,900 / 50,000
Sports Park Water Tower	344	460	492	88	1,400,000

(a) Operational storage / nominal storage volumes

Due to the flat topography of the City's service area, the entire distribution system is served by a single pressure zone, and there are no pressure reducing valves or control valves in the system. Hydraulic grade line and static pressure information are summarized in Table 3-3.

Table 3-3. Hydraulic Grade Line and Static Pressures

Parameter	Existing Value[a]	Future with SPWT[b]
Nominal Hydraulic Grade Line Elevation	460 feet	500 feet
Minimum Ground Surface Elevation	305 feet	305 feet
Maximum Ground Surface Elevation	358 feet	358 feet
Maximum Static Pressure	67 psi[c]	85 psi
Minimum Static Pressure	44 psi	62 psi

(a) Represents system condition prior to the SPWT coming online.
(b) Represents system condition after the SPWT is brought online.
(c) psi = pounds per square inch

The system is supplied by groundwater produced by eight wells, of which six are in service and two are out of service. Information on the City's wells is summarized in Table 3-4.

Table 3-4. Existing Groundwater Well Operation

Well	Depth (ft)	Discharge[(a)] (gpm)	Control Type	Control Node	On Setting	Off Setting	Backup Power
2	240	Out of Service	N/A	--	--	--	N/A
5A	485	1,810	Pressure	580	47 psi	56 psi	No
6A	540	2,440	Pressure	1974	45 psi	60 psi	No
9	--	Out of Service	N/A	--	--	--	N/A
10	374	880	Pressure	1628	51 psi	56 psi	No
11	565	1,090	Time	--	5:50 AM	11:30 PM	No
12	540	1,620	On 24 hours	--	--	--	Yes
13	640	880	On 24 hours	--	--	--	Yes

(a) Represents discharge under existing HGL condition without SPWT online. Discharge will decrease approximately 1,750 gpm when the HGL is increased due to pump efficiencies.
(b) gpm = gallons per minute; psi = pounds per square inch; N/A = Not Applicable

In total, the active wells shown in Table 3-4, have a combined capacity of approximately 8,720 gpm. With the largest well out of service, Well 6A, the combined capacity is approximately 6,280 gpm. It is important to note that once the SPWT is brought online and the HGL is increased, the capacity will decrease approximately 1,750 gpm to 6,970 gpm due to a decline in well pump efficiencies (refer to Appendix E for additional information).

Each of the active wells is equipped for disinfection using sodium hypochlorite. Well 9 was removed from service in 2011 due to nitrate levels just below the maximum contaminant level (MCL) of 45 mg/L. Well 2 was similarly removed from service in late 2012 due to nitrate levels (and was subsequently destroyed in March 2013). In addition, Well 2 had elevated levels of trichloropropane (TCP) -- above the State’s drinking water notification level of 0.005 µg/L.

In addition to the wells noted above, a new well is being designed and will be operational in the near future. However, test wells at the site of the new well (Well 14, collocated with the SPWT) have suggested potential groundwater contamination from dibromochloropropane (DBCP) and TCP. Water from the new Well 14 will be treated with granular activated carbon (GAC) to remove contaminants of concern to levels below State and Federal requirements.

Groundwater levels in the Reedley area in recent years showed a marked decrease during the last drought (2007-2009) but have somewhat recovered since that time. Groundwater contour maps, prepared by Alta Irrigation District, show that depth to groundwater levels increased from

55-65 ft in 2007 (first year of the drought) to 70-85 ft in 2009 (last year of the drought) and decreased back down to 50-60 ft in 2011[1].

Long-term water level measurements for eight wells near the City obtained from the Department of Water Resources indicate an average annual water level decline of 0.4 ft/year for the Reedley area. Based on groundwater level declines for these wells, an overdraft of approximately 350 ac-ft/yr is estimated for the City's SOI[2].

As described in the City's General Plan 2030, the Reedley Municipal Code (RMC) has implemented regulations for the conservation of potable water including a reduction of water use and reduction of unnecessary use of potable water supplies. The RMC, coupled with the goals and policies of the General Plan 2030 and supporting plans, such as the City's 2010 UWMP, represent an effort by the City to effectively manage groundwater as a valued resource and to ensure the avoidance of a critical overdraft of the finite water resource.

Subsection 3.4 presents an analysis of the feasibility of adding surface water as a source of potable supply for the City's system to augment the existing groundwater supply.

3.2. Current and Projected Water Demands

Current and projected water demands were estimated based on land use zoning and current production rates. The following subsections describe the development of the current and future demands.

3.2.1. Current Water Demands

Data on metered water uses from January 2005 to February 2008 was used to estimate baseline demands. The baseline average day demand for the City's potable water distribution system is approximately 5.3 million gallons per day (mgd).

3.2.2. Projected Future Water Demands

Land use data from the City's General Plan 2030 was used to forecast potable water demands for the Phase 1 and Phase 2 planning horizons. The method used was to multiply estimated water duties per acre of land in different land use categories by the acreages forecast in the General Plan for those same land use categories. Water duties were obtained from the 2006

[1] City of Reedley, 2010 Urban Water Management Plan, 2013.
[2] Schmidt, Reedley Water Budget Report, May 16, 2013.

Boyle Hydraulic Model & System Analysis Report and iteratively adjusted to match total observed system demand. Water duty factors used by similarly sized cities in the area were used to check that the adjusted factors were reasonable. The water duties, acreages and results for Average Day Demand (ADD) are presented in Table 3-5.

Based on the forecast presented in Table 3-5, average daily demand for water is expected to grow from approximately 5.3 mgd under baseline conditions, to 6.8 mgd in Phase 1 and approximately 17.8 mgd in Phase 2, which corresponds to build out of the City's General Plan 2030 SOI.

Table 3-5. Water Demand Forecast

General Plan Land Use	Duty Factor (gpd/acre)	Existing Scenario		Phase 1 Scenario		Phase 2 Scenario	
		Acres[(a)]	ADD (gpd)	Acres[(a)]	ADD (gpd)	Acres[(a)]	ADD (gpd)
Residential							
Suburban Residential	2,110	7	14,000	10	20,100	274	578,200
Low Residential	2,820	959	2,703,500	1,181	3,329,200	3,403	9,597,700
Medium Residential	2,740	27	74,300	35	94,500	110	300,700
High Residential	4,200	156	656,700	202	849,900	246	1,031,300
Commercial and Industrial							
Central Downtown	2,990	40	119,700	40	119,700	40	119,700
Community Commercial	2,530	92	233,300	111	281,100	430	1,087,900
Neighborhood Commercial	2,840	11	31,600	23	63,900	44	124,400
Service Commercial	2,670	75	201,200	78	208,500	129	345,300
Office	2,090	16	34,100	17	34,500	17	34,500
Light Industrial	1,650	142	234,900	225	371,200	805	1,328,500
Heavy Industrial	2,850	54	154,000	54	154,600	177	505,100
Other							
Open Space	3,180	96	305,900	188	598,300	563	1,791,500
Public/Institutional Facility	1,270	408	518,100	511	648,500	721	915,300
System-Wide Total ADD			5,281,300		6,774,000		17,760,100

(a) The presented acreages do not include "Community Buffer" areas or interstitial areas (roadways, alleys, railways, etc.) that do not have an associated water demand.

Corresponding projections for maximum day demand (MDD) and peak hour demand (PHD) were developed by multiplying the ADD by peaking factors. The MDD peaking factor of 2.2 is based on the 2006 Boyle Hydraulic Model & System Analysis Report. The PHD factor is based on the diurnal pattern illustrated in Figure 3-2.

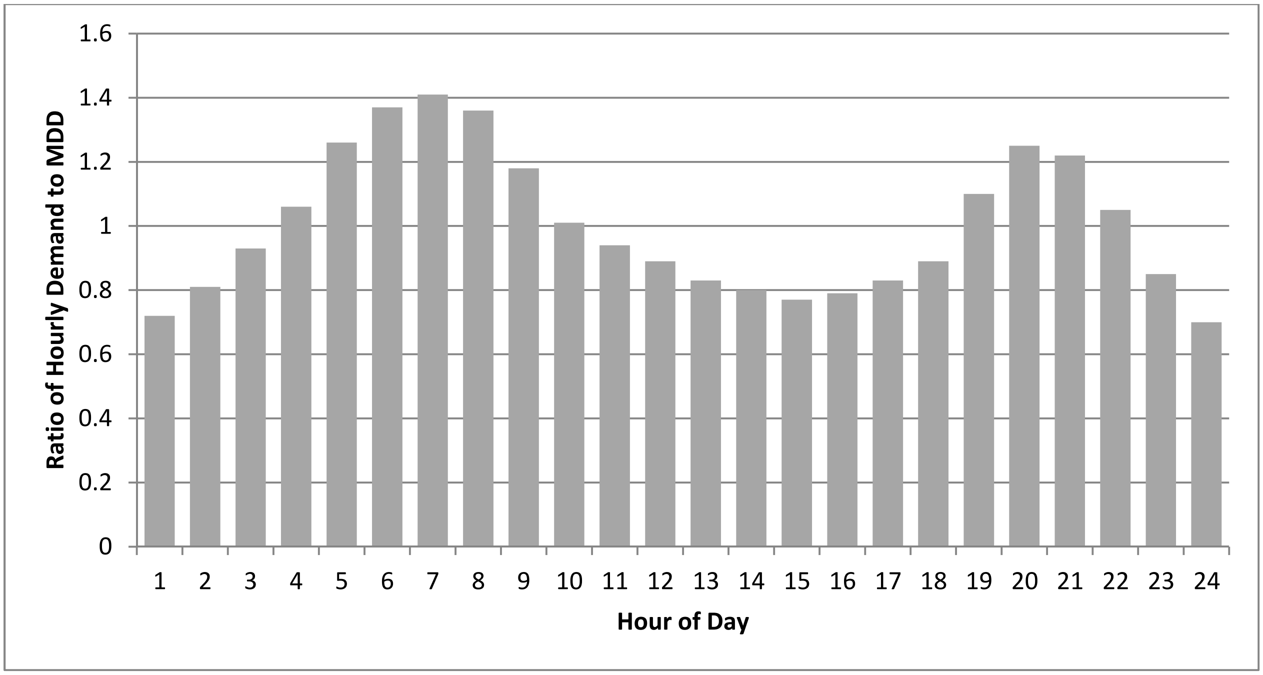


Figure 3-2: Diurnal Demand Pattern

The peaking factors and corresponding MDD and PHD forecasts are summarized in Table 3-6.

Table 3-6. Water Demand Forecasts for ADD, MDD, and PHD

Demand Type	Peaking Factor	Existing Scenario		Phase 1 Scenario		Phase 2 Scenario	
		(gpd)	(mgd)	(gpd)	(mgd)	(gpd)	(mgd)
ADD	1	5,281,300	5.3	6,774,100	6.8	17,760,100	17.8
MDD	2.2	11,618,800	11.6	14,903,000	14.9	39,072,300	39.0
PHD	3.1	16,372,000	16.4	20,999,700	21.0	55,056,400	55.1

3.3. Regulatory Requirements

Understanding regulatory requirements is essential for developing a comprehensive and successful long-term plan for water supply and treatment. This section provides an overview of federal and state regulations currently in effect, describes how the City is meeting regulatory standards, identifies anticipated changes in the regulatory environment and provides suggestions for how the City might prepare for and adapt to these changes.

Drinking water regulations are in place to ensure that harmful water constituents are kept below threshold levels established to protect public health. Numerous laws are in place to regulate potential public health risks including pathogenic microbes, carcinogens such as disinfection byproducts, toxins such as lead, and synthetic contaminants such as pesticides and industrial by-products, among a range of other contaminants. All municipalities must maintain compliance with the drinking water regulations established by these laws.

The U.S. Environmental Protection Agency (USEPA) has promulgated numerous regulatory rules under the Safe Drinking Water Act. Federal law requires drinking water systems to comply with the minimum requirements of these rules. Each state retains primacy for enforcement of federal drinking water regulations, and may promulgate and enforce regulations more stringent that the federal requirements. The California Department of Public Health (CDPH) retains primacy in the State of California, and has in many cases adopted more stringent regulations.

For a water supply and distribution system to remain in compliance with regulatory rules, prescribed treatment, monitoring and reporting requirements, and water quality standards must be met. Municipalities are also required to produce annual summary Consumer Confidence Reports (CCRs). The reports provide valuable data to the public regarding drinking water sources and all levels of contaminants found during compliance monitoring over the preceding year. CCRs summarizing the City's 2011 and 2012 water quality data are contained in Appendix A. The CCRs demonstrate that the existing City water supply consistently provides high quality drinking water with respect to current federal and state regulations.

As previously described, the City currently depends entirely on groundwater for its supply and therefore the following federal surface water regulations do not apply:

- The Surface Water Treatment Rule (SWTR),
- The Interim Enhanced Surface Water Treatment Rule (IESWTR), and
- The Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR).

The remainder of this section addresses only groundwater-specific drinking water rules currently in effect, and applicable to the City's water supply and distribution system as enforced by the CDPH. A brief overview of each applicable rule is provided below. The overviews contain summaries of current requirements and indications of anticipated revisions to the rules where applicable. Table 3-7 summarizes requirements, City monitoring activities, regulatory compliance status, and planned actions for each of the currently applicable rules.

Total Coliform Rule (TCR) and Planned Revisions – The 1989 Total Coliform Rule set both health goals and legal limits for total coliform levels in drinking water to prevent public exposure to harmful microbial pathogens. The rule also detailed the type and frequency of monitoring requirements. Previously, no more than 5% of distribution system total coliform samples conducted over a month could be positive.

Proposed revisions to the Total Coliform Rule Revisions were published in Federal Register in July of 2012, and implementation activity is currently under way. The revisions include:

- Establishing an MCL of 0 for E. coli, a more specific indicator of fecal contamination and potential harmful pathogens than total coliform. Total Coliform will likely be removed as an indicator because many of the organisms detected by total coliform methods are not of fecal origin and do not have any direct public health implication.
- A utility that incurs an E. coli MCL violation must conduct an assessment and correct any sanitary defects found.

The City will coordinate with CDPH to prepare for implementation of the TCR revisions. Further information is contained in the EPA Draft *Proposed Revised Total Coliform Rule Assessments and Corrective Actions Guidance Manual*.

Lead and Copper Rule (TCR) and Planned Revisions (Long-term LCR) – The Lead and Copper Rule (LCR) establishes action levels, monitoring, and compliance requirements for lead and copper levels at customers' taps. To meet the established action levels under the current LCR, 90 percent of all residential samples collected must have lead levels equal to or less than 0.015 mg/L and copper levels equal to or less than 1.3 mg/L. If these action levels cannot be met, systems must implement public education and a corrosion control treatment strategy for meeting these levels.

The USEPA is currently revisiting monitoring criteria that have been in place since the rule was promulgated in 1991. Draft revisions are expected soon; initially they had been planned for 2012. Integration of the revisions will constitute the new Long-Term LCR. The revisions will most likely entail a re-arrangement of the City's current residential sampling sites, as the new monitoring criteria will likely place new emphasis on sites where lead service lines are absent, but solder and plumbing components that may contain lead are likely to be present. The City will continue to track the progress of the Long-term LCR development, and will prepare for changes to their current monitoring as information becomes available.

Consumer Confidence Report (CCR) and Public Notification Rules - The Consumer Confidence and Public Notification Rules require systems to provide customers with water quality information on an annual basis, and when a regulatory violation occurs. Under the

Consumer Confidence Report (CCR) Rule promulgated in 1998, community water systems are required to provide an annual CCR on the source of their drinking water and levels of any contaminants found. The annual report must be supplied to all customers and must include:

- Information on the source of drinking water.
- A brief definition of terms.
- If regulated contaminants are detected: the maximum contaminant levels goal (MCLG), the maximum contaminant level (MCL), and the level detected.
- If an MCL is violated: information on health effects.
- If the USEPA requires it: information on levels of unregulated contaminants.

While the CCR provides an annual “state-of-the-water” report, the Public Notification Rule (PNR) directs utilities in notifying customers of acute violations when they occur. The PNR was revised in May 2000 and outlines public notification requirements for violations of MCLs, treatment techniques, testing procedures, monitoring requirements, and violations of a variance or exemption. If violations have the potential for “serious adverse effect,” consumers and the State must be notified within 24 hours of the violation. The notice must explain the violation, potential health effects, corrective actions, and whether consumers need to use an alternate water source. Notice must be made by appropriate media or posted door-to-door. Less serious violations must be reported to consumers within 30 days in an annual report, or by mail or direct delivery service within 1 year, depending on the severity of the violation.

The City is current in its CCR reporting, and plans to continue meeting the following annual schedule:

- Provide annual report to retail customers and CDPH by July 1 of each year,
- Certify report information before October 1 of each year.

Stage 1 and 2 Disinfection and Disinfection Byproducts Rules (DBPR I and II) - Disinfection byproducts (DBPs) result from the reaction of natural organic matter (NOM) and various inorganic precursors with chemical disinfectants. Some DBPs, such as trihalomethanes, have been shown to cause cancer and negative reproductive health effects. In 2002, the federal Stage 1 Disinfectant/Disinfection By-Product Rule was implemented requiring running annual averages of distribution system sample of total trihalomethanes (TTHM) to be less than 80 µg/L and the total of five haloacetic acids (HAA5) to be less than 60 µg/L. The City has been in compliance with the Stage 1 rule, and currently collects monitoring samples at 4 distribution sample sites quarterly.

The final Stage 2 DBPR was promulgated on January 4, 2006. The Stage 2 DBP Rule has been developed by the USEPA to further reduce exposure to DBPs linked to bladder, rectal, and colon cancers. The Stage 2 DBPR revises reporting of running average TTHM and HAA5 concentrations to locational running annual averages. Preparation for the Stage 2 DBPR included development of a sampling plan to revise monitoring sites, collecting a year of monitoring data at the sites to select permanent Stage 2 monitoring locations, and an Initial Distribution System Evaluation (IDSE) reporting these results to EPA and finalizing the new monitoring locations. The City completed the Sampling and IDSE in 2010, and began monitoring at the new sites in 2013, reporting locational running averages to CDPH.

Phase I, II, and V Rules - Monitoring requirements and MCLs for inorganic (IOC), synthetic organic (SOC), and volatile organic (VOC) chemicals are addressed by federal Organic, Synthetic Organic and Inorganic Chemicals Phases I, II, and V Rules. Under Phases II and V, MCLs are set for 16 inorganic, 30 synthetic, and 21 volatile organic contaminants with monitoring of IOCs, VOCs, and SOCs at each source on 12- to 36-month sampling cycles, depending on the contaminant and source type. Nitrate and nitrite are measured each year and monitoring cannot be waived. City distribution system nitrate levels tend to have the widest range of reported values and on occasion, samples have been over 40 mg/L, though still lower than the CDPH MCL of 45 mg/L. This is a parameter that the City is aware of, and has made efforts to inform its customers of potential health impact of high nitrate levels in its past CCRs. The City will continue this approach. The City has also has isolated incidents of positive sampling for the VOCs TCP and DBCP. Sampling levels observed do not present a meaningful level of risk with respect to the MCLs for these VOCs. The City has nonetheless taken a proactive public education approach, providing information on these VOCs in the CCRs.

Radionuclide Rule – The Radionuclide Rule, since its 2003 revisions, includes MCLs for the sum of radium-226 and radium-228 (5 pCi/L), adjusted gross alpha emitters (15 pCi/L), gross beta and photon emitters (4 millirems per year [mrem/year]), and uranium (0.03 mg/L). Systems were required to conduct initial monitoring between 2003 and 2007, unless earlier radionuclide data can be used as grandfathered data. Monitoring for radionuclides must be conducted at each entry point to the distribution system. The City is in compliance with requirements of the Radionuclide rule, and no changes to current activity are expected.

Arsenic Rule – Revision to the Arsenic rule of 1975 became effective January 2006. These reduce the arsenic MCL to 0.010 mg/L and identify several best available treatment technologies (BATs) for compliance. Compliance with the new MCL is based on the running annual average of monitoring results at each entry point to the distribution system. The rule makes arsenic monitoring requirements consistent with monitoring for other IOCs regulated under the Phase II/V standardized monitoring framework. The City is in compliance with requirements of the Arsenic rule, and no changes to current activity are expected.

The Groundwater Rule (GWR) – The GWR was initially published in 2006 to provide increased protection against microbial pathogens in systems serving groundwater. The basic components of the rule are:

- Periodic sanitary surveys of systems and the identification of significant deficiencies (e.g., a well located near a leaking septic system);
- Triggered source water monitoring when a system identifies a positive sample during its (Revised) Total Coliform Rule monitoring and assessment monitoring at CDPH discretion.
- Corrective action is required for any system with a significant deficiency or source water fecal contamination;
- Compliance monitoring to ensure that treatment technology installed to treat drinking water reliably achieves 99.99 percent (4-log) inactivation or removal of viruses.

The City conducted its first sanitary survey starting in 2009, and has been conducting all required monitoring. The City is in compliance with requirements of the GWR, and no changes to current activity are expected.

Table 3-7. Summary of Applicable Regulations and Compliance Status

Rule	Requirement	Required Compliance Date	Status	City Compliance	Planned Actions
TCR	Max 5% positive total coliform per month.	1989	City conducts required monitoring.	Yes	Prepare for Implementation of Revisions
RTCR	Zero Positive E. Coli samples	TBD	Tracking new requirements	Future	Coordinate with CDPH
Lead and Copper Rule	No more than 0.015 mg/L lead 1.3 mg/L copper in 90th percentile residential sampling.	1992	City conducts residential sampling.	Yes	Track status of Long-term Revisions.
CCR and Public Notification Rules	CCR sent to all customers and CDPH by July of each year.	April 1999	CCRs published annually.	Yes	Continue current schedule
Stage 1 DBPR	TTHM/HAA5 $\leq$ 80/60 µg/l as running annual average	Jan 2002	Monitor at four distribution system locations quarterly.	Yes	Continue with existing monitoring until 2013 transition to Stage 2 approved sites.
Stage 2 DBPR	TTHM/HAA5 $\leq$ 80/60 µg/l as locational running annual average at new sampling sites	2013 Stage 2 transition	IDSE Approved	Yes	Prepare for Stage 2 monitoring transition.
Phase I, II, V Rules	Monitoring for 16 inorganic (IOC), 30 synthetic (SOC), and 21 volatile organic contaminants (VOC).	1989-1993	Meets monitoring requirements	Yes	Continue with current monitoring.
Radionuclide Rule	Beta/photon emitters $\leq$ 4 mrem/hr; Alpha emitters $\leq$ 15 pCi/L; Combined radium $\leq$ 5 pCi/L; Uranium $\leq$ 30 µg/L.	Dec 2003	Meets monitoring requirements	Yes	Continue monitoring per CDPH requirements.
Arsenic Rule	Arsenic $\leq$ 10 µg/L arsenic.	Jan 2006	Meets monitoring requirements	Yes	Continue with existing monitoring.
Groundwater Rule	Monitoring for 4-log removal of viruses. Sanitary survey every 3 years.	Dec 2009	Meets monitoring requirements	Yes	Continue with existing monitoring

3.3.1. Other Potential Regulatory Changes

Planned revisions to the Total Coliform Rule and Lead and Copper Rule are discussed above. In addition to preparing for these revisions, the City is tracking other potential changes in the regulatory environment. Two potential changes, the proposed Radon Rule and activity around regulation of Hexavalent Chromium in California are discussed below.

Radon Rule - A proposed Radon Rule was released in 1999 that provides two options for the maximum level of radon allowable in public drinking water supplies. The Safe Drinking Water Act directed the USEPA to propose and finalize an MCL for radon in drinking water, but also to make available a higher alternative MCL (AMCL) accompanied by a multimedia mitigation (MMM) program to address risks of radon exposure due to its occurrence in air. The proposed MCL and AMCL for radon are 300 pCi/L and 4,000 pCi/L, respectively. The drinking water standard that would apply to the City depends on whether or not CDPH develops an MMM program. Development of a final radon rule has been delayed numerous times since the rule was first proposed. At present, it is unclear when it will be finalized.

Hexavalent Chromium – Independent of EPA studies, California is considering its own specific regulation for Hexavalent chromium (Cr(VI)) in drinking water. In 2011 the California Office of Environmental Health Hazard Assessment (OEHHA) established a Public Health Goal (PHG) for Cr(VI) of 0.02 µg/L. The PHG is a non-enforceable standard set at a one-in-one-million excess cancer life time risk level due to exposure to Cr(VI) in drinking water. In 2013, CDPH proposed an MCL for Cr(VI) of 0.010 mg/L. Comments on the proposed rule were due in October 2012. CDPH has reviewed the comments that were provided by interested parties and responded to them in the final statement of reasons. On April 15, 2014, CDPH submitted the Cr(VI) MCL regulations package to the Office of Administrative Law (OAL) for its review for compliance with the Administrative Procedure Act. OAL has 30 working days to complete its review, and upon completion of the review, will either approve or reject the regulations. If approved, the regulations will be transmitted to the Office of the Secretary of State, and become effective as early as July 1, 2014. The City should continue to monitor the rulemaking process to determine what impacts the new MCL could have on the City's water supply.

3.4. Potable Water Supply Alternatives Analysis

As described in Subsection 3.2 of this Master Plan, water demands are forecasted to more than triple between current conditions and projected build out (Phase 2) of the City's General Plan 2030. The City's current source of supply is groundwater from municipal wells. As a part of this planning effort, the City considered whether increased water supplies should also be drawn from groundwater or whether other sources should be developed. This section summarizes the future water supply need and the findings, and Appendix C contains additional information.

The primary alternative to pumping additional groundwater would be diversion of surface water from the Kings River. For this analysis, potential new facilities were identified that would be required to divert, treat and deliver water from the Kings River, and these were compared with facilities needed to add additional groundwater wells to the City's supply system. For purposes of this comparison, several assumptions were made, as summarized below:

- Expanded groundwater supply to fully meet demands at buildout (Phase 2) would require up to eight new wells, each capable of pumping up to 2,500 gpm.
- Groundwater would require treatment using granular activated carbon (GAC). This assumption is based on the City's experience with the Sports Park Water Tower well and the uncertainty regarding the future expansion/dispersal of the TCP contamination plume. In other words, this assumption was included to provide a conservative assessment of the future cost of groundwater development and associated treatment.
- Wells could be distributed throughout the City as needed to serve developing areas without needing major improvements to the distribution piping system.
- In order to use water from the Kings River, the City would need to purchase a senior surface water right from a willing seller within the Kings River Basin.
- Surface water development would require a screened intake on the river, a conventional surface water treatment plant, and a new transmission main looped around the City to distribute water throughout newly developing areas. While a portion of these facilities could be phased over time, such as treatment plant capacity, much of the infrastructure would need to be in place up front, in order to begin delivering surface water to customers.

Six criteria were developed and used to compare the surface water and groundwater alternatives. These criteria included capital cost, annual operations and maintenance (O&M) cost, availability of supply, reliability of supply, ability to phase costs to match growth, and permitting complexity. The results were as follows:

- **Capital Costs.** Based on preliminary cost estimates, surface water development would cost approximately $249 million while groundwater would cost approximately $102 million (costs are in 2012 dollars).

- **Annual Costs.** O&M costs for surface water treatment and transmission were estimated at approximately $5 million per year, compared with $10 million per year for groundwater (assuming GAC treatment is needed for all new wells – a very conservative assumption). However, since the capital costs would require bonding, debt service was also considered. Total annual costs when debt service on the bonds is included would be on the order of $22 million per year for surface water compared with $16 million for groundwater. This assumes all facilities would be built and financed in Phase 1. In reality annual costs with debt service for groundwater development would be much less, because groundwater supplies can be developed and paid for in stages as demands increase over time, while most of the surface water development costs would be incurred up front (i.e., water rights, diversion facilities, basic treatment infrastructure, and transmission facilities). As a result, it is clear that the total annual costs for surface water are significantly greater than those for groundwater.

- **Availability of Supply.** Surface water is much less available in the Kings River Basin than groundwater. Due to over-appropriation of surface water resources, it may be difficult or impossible to purchase a suitable surface water right. While groundwater resources in the Kings River Basin also face significant management challenges due to overdraft, it appears far more likely that the City could secure the right to pump additional groundwater.

- **Supply Reliability.** Groundwater provides a more reliable source of supply from year to year, because it is less affected by drought than surface water.

- **Ability to Phase Infrastructure.** As noted previously, groundwater can be developed incrementally to match growth, while a new surface water source would need to be constructed, for the most part, all at once.

- **Permitting Complexity.** The complexity of the permitting process is expected to be much greater for surface water than for groundwater.

The role of water conservation and reuse of treated wastewater in meeting the City's needs for additional water supplies were also considered. Water conservation is addressed in the City's 2010 UWMP. Conservation can potentially push the need for new supplies farther into the future, and can also reduce the quantity needed. Similarly, treatment of wastewater to allow for reuse can help to meet a portion of the future water demand. Both of these approaches would have roughly equal effects on needs regardless of whether the City develops new surface water or groundwater resources. However, a strategy of incremental development of groundwater can be matched better with actual water needs over time, as conservation and reuse lower the City's water demand profile.

Based on the comparative analysis of both costs and non-economic criteria, it was recommended that the City continue to rely on groundwater as its primary source of potable water to serve future growth.

While the State does not require a permit to access new groundwater supplies, the Kings River Groundwater Basin has a well-documented overdraft problem. This problem is described in the 2007 *Upper Kings Basin Integrated Water Management Plan* issued by the Upper Kings Basin Water Forum (of which Reedley is a member); and Kings River Conservation District. The Upper Kings Basin Plan recognizes that cities in the Kings basin need water for growth, and identifies a variety of potential strategies to enable this to occur. The plan is specifically intended to improve management of groundwater resources, including meeting new needs or replacement of supplies. A number of initiatives are proposed in the plan. However it is not clear how the plan will directly affect the City's need for increased groundwater supplies.

City staff will continue to engage with the State and other parties in the Basin with regard to new groundwater supply development, particularly with respect to opportunities to enhance groundwater recharge to minimize potential overdraft of the groundwater basin. For example, it may be possible for the City to collaborate with agencies such as CID and AID to use existing and future stormwater retention basins (described in Section 5) as recharge basins when surface water is available.

3.5. Water Distribution System Evaluation

As part of this planning process, a hydraulic model of the City's water distribution system was developed. This model was based on a previously completed model (Boyle 2006) and updated to reflect current infrastructure conditions and new demand forecasts (existing, Phase 1, and Phase 2). A detailed description of the distribution system model update is included in Appendix D.

The distribution system model was then used to analyze each planning scenario (existing, Phase 1, and Phase 2) and determine required improvements. The modeling results also provided information used in determining the phasing of recommended system additions and improvements. Results were evaluated based upon planning metrics established for fire flow, velocity, head loss, and pressure.

The following subsections describe the planning criteria used to evaluate the system as well as the results of the analyses for the Existing, Phase 1, and Phase 2 planning periods.

3.5.1. Planning Criteria

Planning criteria were established to have a common set of metrics to evaluate the existing system and to use as the basis for developing system upgrades to address deficiencies and expand the system to serve future growth. These planning criteria are summarized in Table 3-8.

Table 3-8. Water System Planning Criteria

Parameter		Value
Minimum Pressure	ADD	40 psi
	MDD	40 psi
	PHD	40 psi
	MDD plus fire flow	20 psi
Maximum Pressure	All conditions	70 psi
Maximum Desirable Pipeline Velocity	PHD[(a)]	5 fps
Maximum Headloss	Pipe diameter < 16 in	6 ft / 1,000 ft
	Pipe diameter > 16 in	2 ft / 1,000 ft
Required Fire Flow	Residential	1,500 gpm
	Commercial	2,500 gpm
	Industrial	3,000 gpm
	Fire flow duration	2 – 4 hours
Storage[(b)]	Operating volume	20% of MDD
	Emergency volume	Provided by Wells
	Fire volume	4 hours at 3,000 gpm
Supply	MDD	Meet demand with largest well out of service

(a) Higher velocities allowed during fire flow.
(b) Storage criteria was revised following the June 29, 2012 Water Distribution and Sewer Collection Systems Criteria memo in order to reduce overall storage requirements while recognizing that emergency supply can be provided by backup wells with emergency generators.

3.5.2. Supply Analysis

Using the water demand forecast described in Section 3.2 and the current available supply, comparisons were completed to identify future water needs. As previously discussed, it is recommended that the City continue to rely on groundwater as its primary source of water supply due to the significant capital investments and uncertainty associated with the development of a surface water supply from the Kings River.

Figure 3-3 illustrates the growth of the water demand (ADD and MDD) from the existing condition through Phase 1 and Phase 2 and compares that against the City's existing groundwater supply which is nearly 13 mgd under current conditions (9 mgd with the largest well out of service, which is the design criteria per Table 3-8). It is also important to note that the existing groundwater supply decreases from the Existing to Phase 1 condition. This decrease in supply capacity is due to the increase in the system HGL once the Sports Park Water Tower is brought online, as described in Section 3.1.

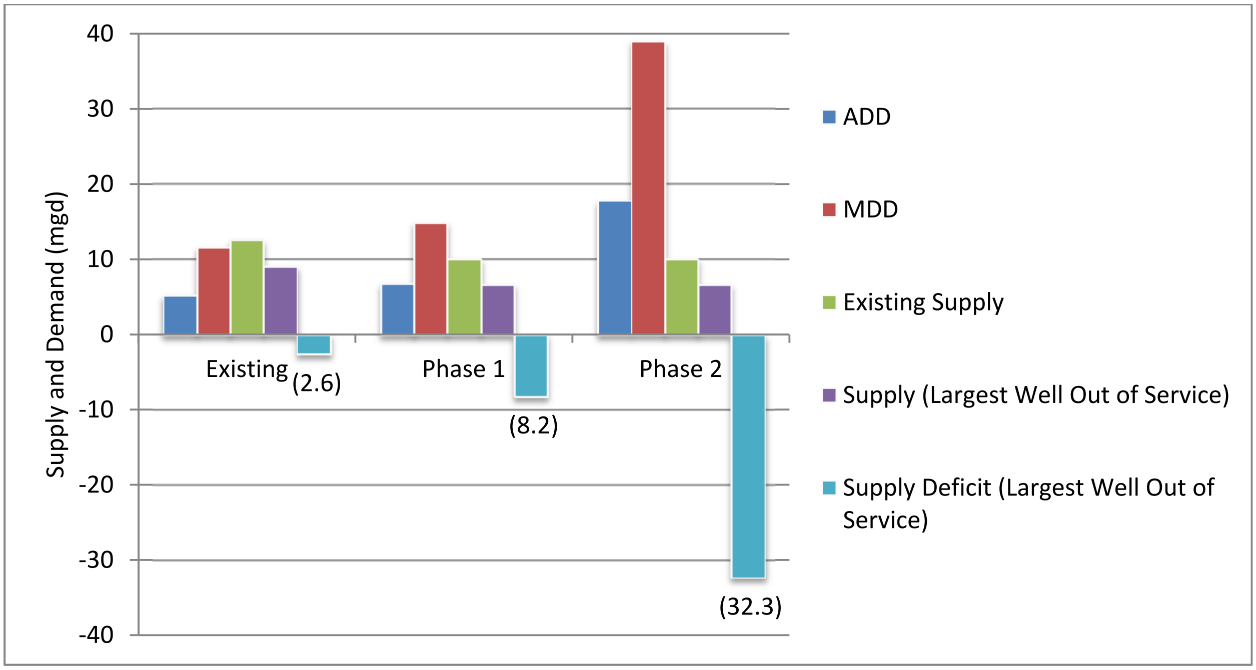


Figure 3-3. Water Supply and Demand Comparison

Increasing the HGL from 468 ft (current HGL) to 500 ft (HGL with SPWT) requires the existing pumps to provide approximately 32 ft of additional lift. Due to pump efficiencies, this results in a total reduction in discharge capacity of approximately 1,750 gpm. As described in Appendix E, the existing well pumps must be retrofitted to operate more efficiently in order to maintain their existing capacity. As a result, to maintain the existing capacity, it is recommended that Wells 5A, 10 and 12 be augmented with an additional bowl to provide the additional lift to accommodate the new, higher HGL. Figure 3-4 illustrates the comparison of the supply and demand with these changes in place.

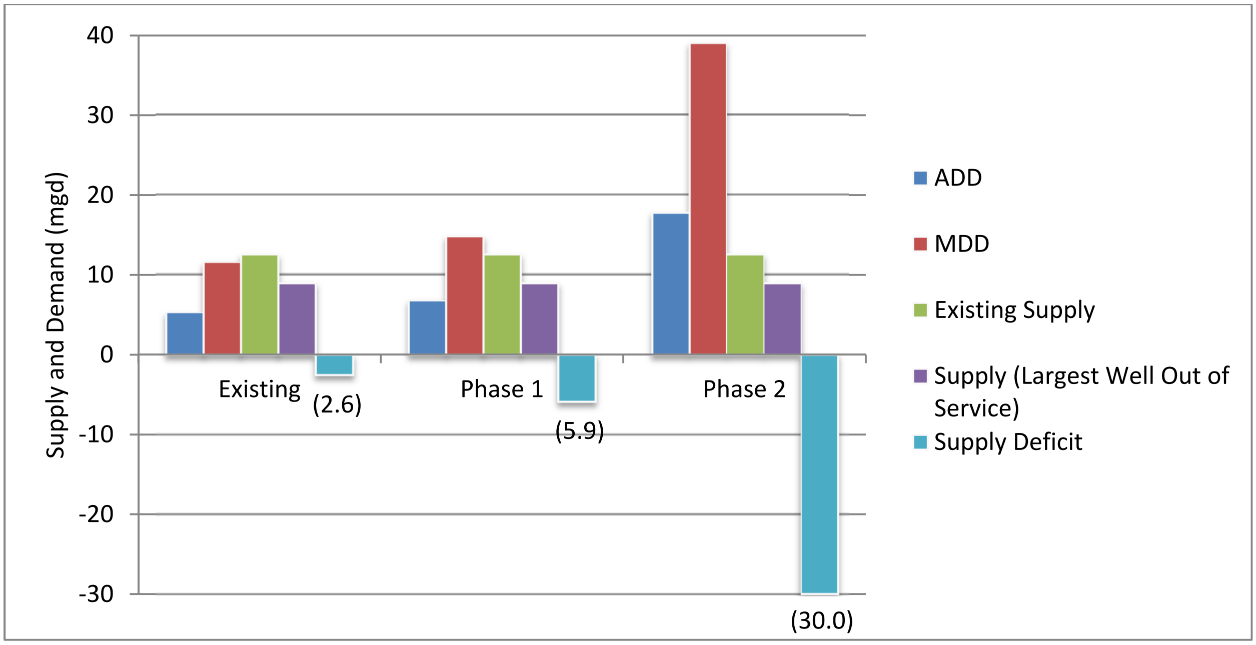


Figure 3-4. Water Supply and Demand Comparison with Existing Well Augmentation

As shown in Figure 3-4, under existing conditions, there is already a small deficit (approximately 2.6 mgd) when the largest well is out of service. However, that deficit grows over time to approximately 5.9 mgd in Phase 1 and to approximately 30 mgd in Phase 2. As a result, additional wells will be needed to augment the City's existing supply.

Once construction of the Sports Park Water Tower well is completed in 2014, the deficit under existing conditions will be resolved. However, a second new well, with similar capacity (i.e., 2,000 gpm), will be needed in Phase 1 to resolve the Phase 1 deficiency.

As the City continues to expand and realize the planned growth in Phase 2, additional wells will be required. Assuming a capacity of 2,000 gpm each, nine additional wells will be needed to meet the full demand in Phase 2. The City should confirm the actual capacity of new wells based on additional planning for well specific sites. If the capacity of future wells varies from 2,000 gpm (up or down), the number of new wells should be adjusted accordingly.

This Master Plan identifies potential locations for the recommended new wells (as shown later in Section 3.5.4 and again in Section 6) for Phase 1 and Phase 2. The City should continue to track planned development and the associated demand, particularly for Phase 2, to better define the exact timing and location for these new wells. In addition, future conservation efforts and implementation of a recycled water system could help to offset the need for new wells.

3.5.3. Storage Analysis

Storage needs were evaluated using the criteria presented in Table 3-8. For the purposes of this evaluation, it was assumed that the Sports Park Water Tower Tank is in service.

As shown in Table 3-9, even with the new Sports Park Water Tower Tank in place, the existing system still has a deficiency of approximately 1.5 MG based on the criteria presented in Table 3-8. Furthermore, the deficiency grows to approximately 2.3 MG in Phase 1 and 7.1 MG in Phase 2, bringing the total system storage requirement to 8.5 MG.

Table 3-9. Water System Storage Analysis

Parameter	Unit	Existing	Phase 1	Phase 2
Max Day Demand	mgd	11.6	14.9	39.1
Operating Volume (20% of MDD)	MG	2.3	3.0	7.8
Emergency Volume (Provided by Wells)	MG	-	-	-
Fire Volume (4 hours @ 3,000 gpm)	MG	0.7	0.7	0.7
Total Required Volume	MG	3.0	3.7	8.5
Current Storage Volume[a]	MG	1.5	1.4	1.4
Storage Surplus / Deficiency	MG	(1.5)	(2.3)	(7.1)
Recommended New Storage	MG	2.0	1.0	4.1
Cumulative New Storage	MG	2.0	3.0	7.1
Total System Storage	MG	3.5	4.4	8.5

(a) Current storage includes the 1.4 MG SPWT Tank and 0.1 MG Downtown Tanks. The Downtown Tanks will be abandoned before Phase 1 due to the new, higher HGL in the distribution system.

To reduce the cost, future storage can be provided in ground level tanks coupled with booster pump stations. However, if this is the desired approach, emergency generators should also be provided with the tanks to ensure that operational and fire volumes are available during a power outage.

3.5.4. Hydraulic Deficiencies

The results of the scenario runs for the existing and future (Phase 1 and Phase 2) systems are shown in Figure 3-5 through Figure 3-10. It is important to note that some new infrastructure (e.g., wells, tanks, major transmission mains) is illustrated on the Phase 1 and Phase 2 deficiencies maps because this infrastructure is necessary to run the model (e.g., sufficient supply must be provided to meet the system demand). The figures present the following information:

- **Figure 3-5. Existing Water System Pressure Deficiencies**: simulations indicate that existing system pressures in the northern part of the system may fall below the 40 psi minimum pressure requirement under peak hour demand conditions.

- **Figure 3-6. Existing Water System Deficiencies**: simulations indicate that 35 locations in the existing system may not meet minimum fire flow requirements. However, 14 of these locations are estimated to meet fire flow requirements once the system HGL is raised when the Sports Park Water Tower is brought online. Approximately 775 LF of pipeline is estimated to exceed the maximum velocity requirement of 5 fps during peak hour demand conditions. In addition, 7,195 LF are estimated to exceed the maximum headloss rate of 6 feet per thousand feet (ft/kft) for pipes with less than 16 inch diameter during peak demand conditions.

- **Figure 3-7. Phase 1 Water System Pressure Deficiencies**: once the Sports Park Water Tower is brought online, which is included in the scenario geometry for Phase 1, the system pressures meet the minimum requirement of 40 psi during peak hour demand conditions. Thus, *there are no minimum pressure deficiencies for Phase 1*.

- **Figure 3-8. Phase 1 Water System Deficiencies**: 22 locations in the Phase 1 system may not meet minimum fire flow requirements. Pipelines totaling 1,703 LF are estimated to exceed the maximum velocity requirement of 5 fps during Phase 1 peak hour demand conditions. In addition, pipelines totaling 12,650 LF are estimated to exceed the maximum headloss rate of 6 ft/kft for pipes with less than 16 inch diameter during peak demand conditions. This map also illustrates the following new infrastructure:

  - Wells: The new Sports Park Water Tower well will be online before Phase 1. In addition, a second new well is required to provide emergency backup supply when the largest well is offline. As illustrated, this well, Northwest Well (NW1), has been located in the northwest area of town near the intersection of Reed Ave and South Ave.

  - Tanks: In addition to the Sports Park Water Tower, two additional storage tanks are required in Phase 1. As illustrated in Figure 3-8, one tank has been located in downtown Reedley in order to replace the existing elevated storage tanks that will be abandoned after the HGL is raised. The second tank is located near the intersection of Buttonwillow and Parlier Avenue in order to maintain pressures in the northeast portion of the system.

- **Figure 3-9. Phase 2 Water System Pressure Deficiencies**: with the new wells and tanks required to provide supply and storage for the Phase 2 system, there are *no minimum pressure deficiencies expected for Phase 2*.

- **Figure 3-10. Phase 2 Water System Deficiencies**: 11 locations in the Phase 2 system may not meet minimum fire flow requirements. In addition, 11 pipes totaling 3,829 ft are expected to exceed the maximum velocity requirement of 5 fps during Phase 2 peak hour demand conditions. In addition, 33 pipes totaling 8,278 ft are estimated to exceed the maximum headloss rate of 6 ft/kft for pipes with less than 16 inch diameter during peak demand conditions.

    - Wells: As previously described, assuming a capacity of 2,000 gpm, nine new wells are needed to serve the Phase 2 demand. These wells are shown spread throughout the system in a distributed fashion to minimize localized impacts on the groundwater basin and to provide a source of supply in relatively close proximity to demand location (thereby reducing water age).

    - Tanks: 4.7 MG of new storage are required in Phase 2. As a result, three new reservoirs were included as shown in Figure 3-10. These are located in the northwest (near Reed and South Avenue), the west (near Huntsman and Wallace Avenue), and the south (near Davis and East Avenue). As illustrated, tanks were generally collocated with one or two wells to minimize the cost of property acquisition and streamline future operation and maintenance requirements.

Figure 3-5. Existing Water System Pressure Deficiencies

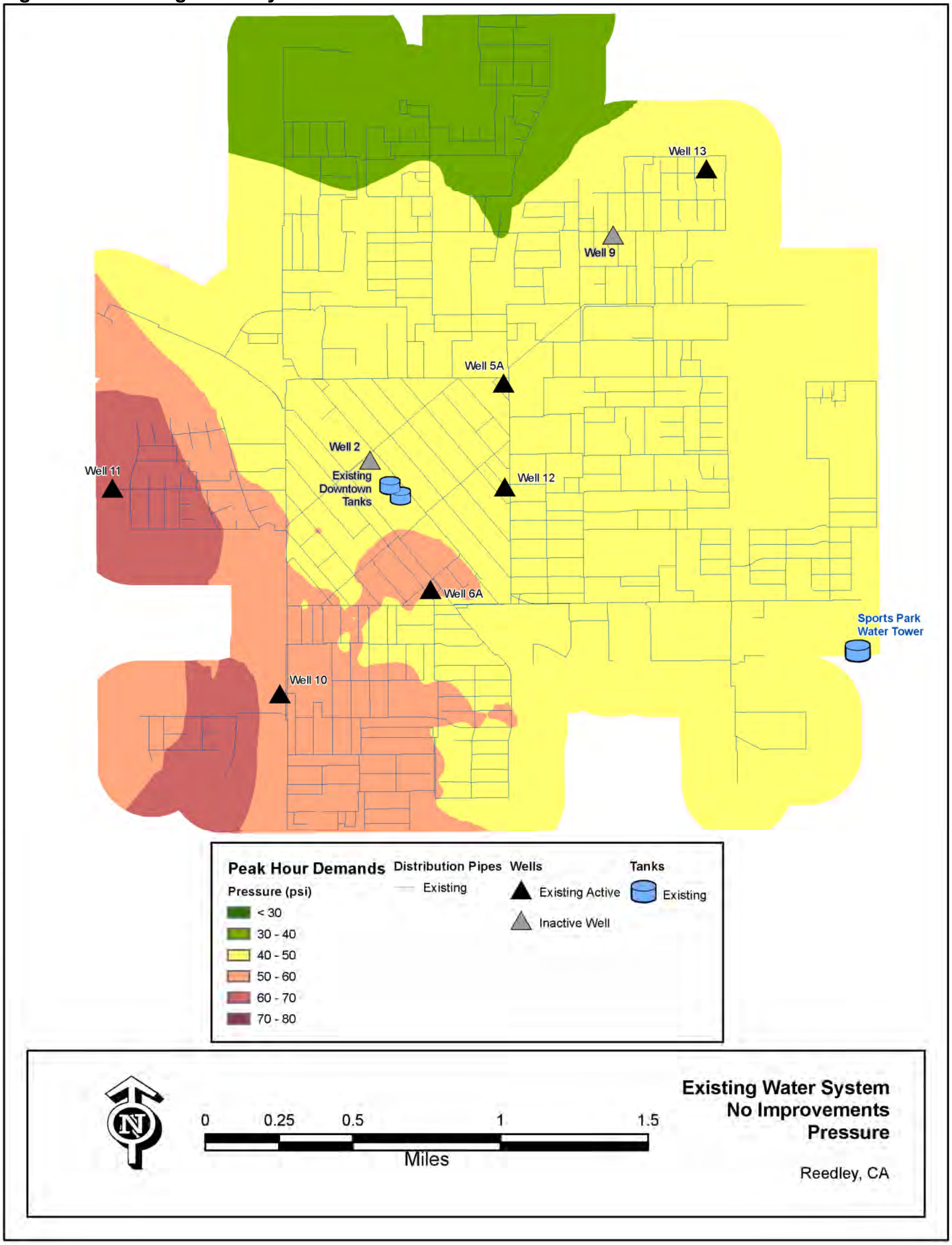


Figure 3-6. Existing Water System Deficiencies

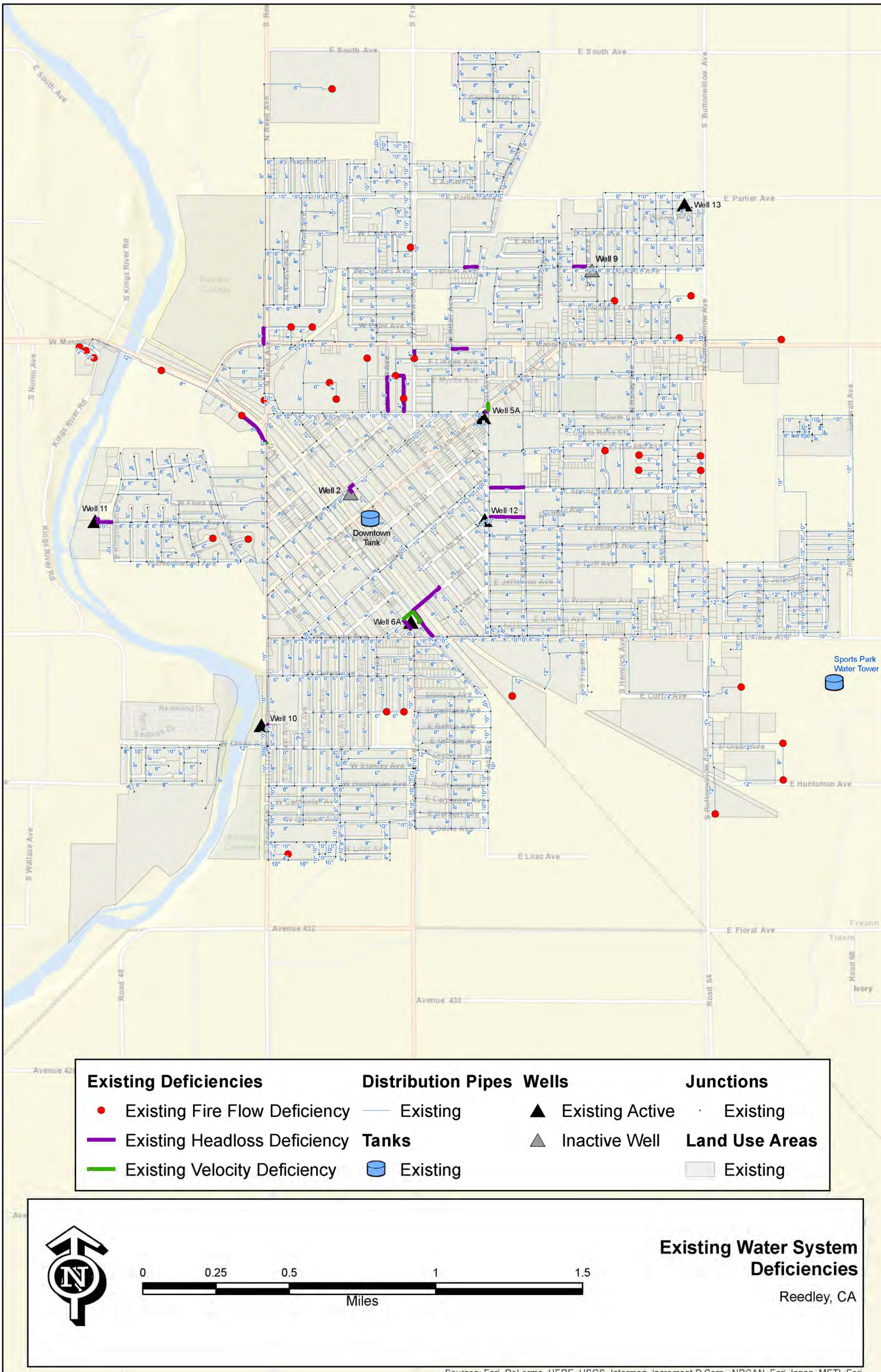


Figure 3-7. Phase 1 Water System Pressure Deficiencies

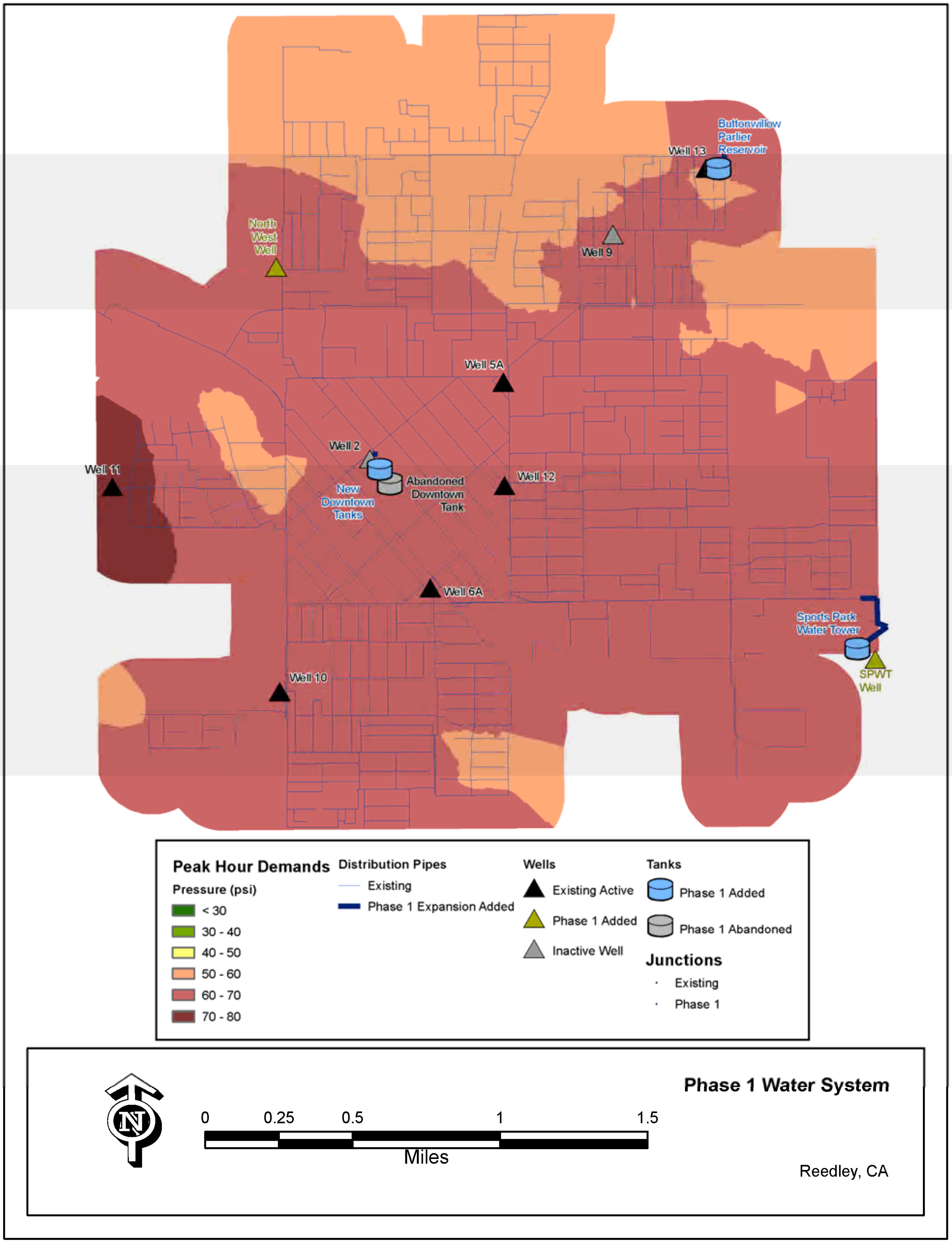


Figure 3-8. Phase 1 Water System Deficiencies

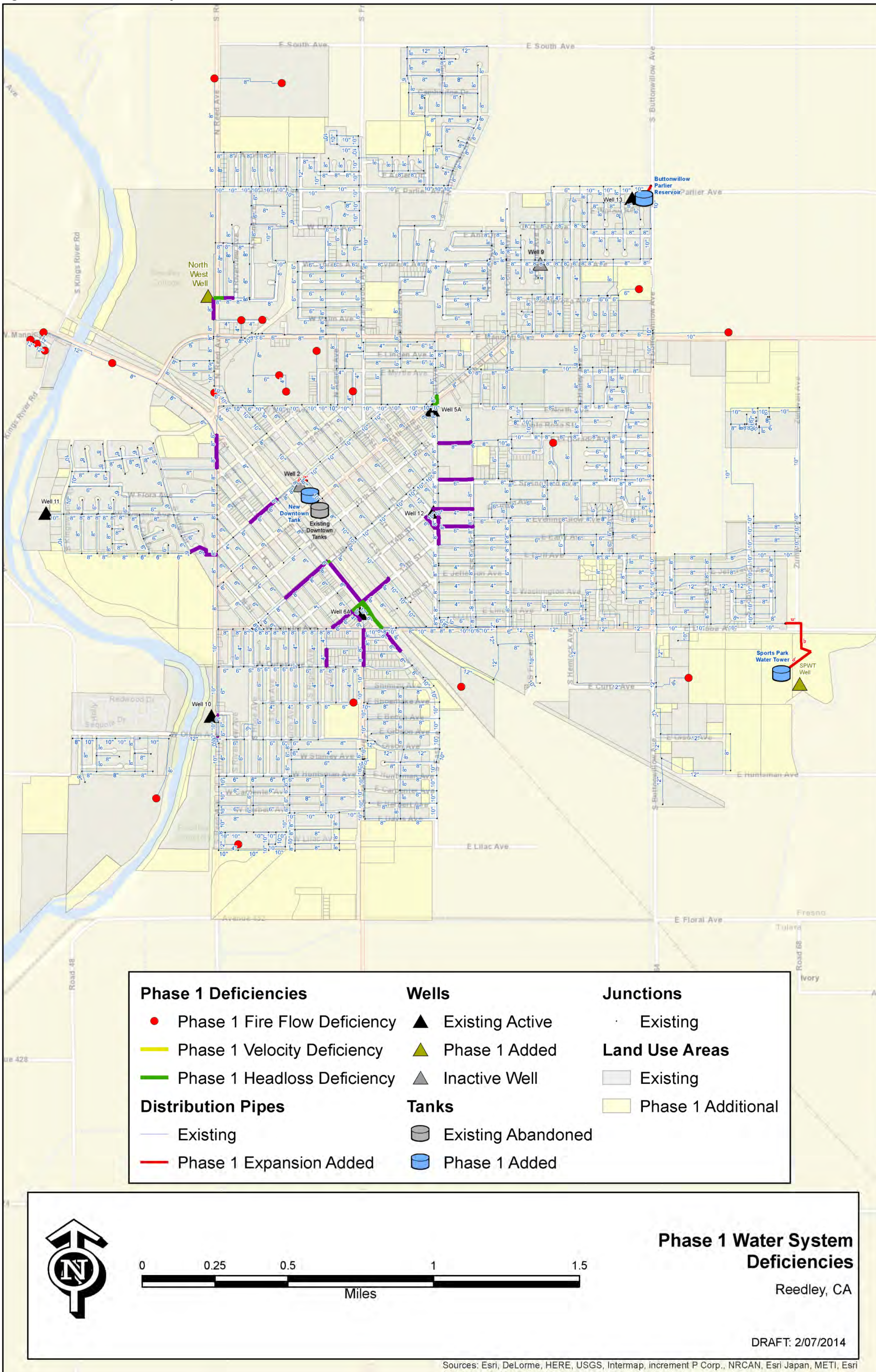


Figure 3-9. Phase 2 Water System Pressure Deficiencies

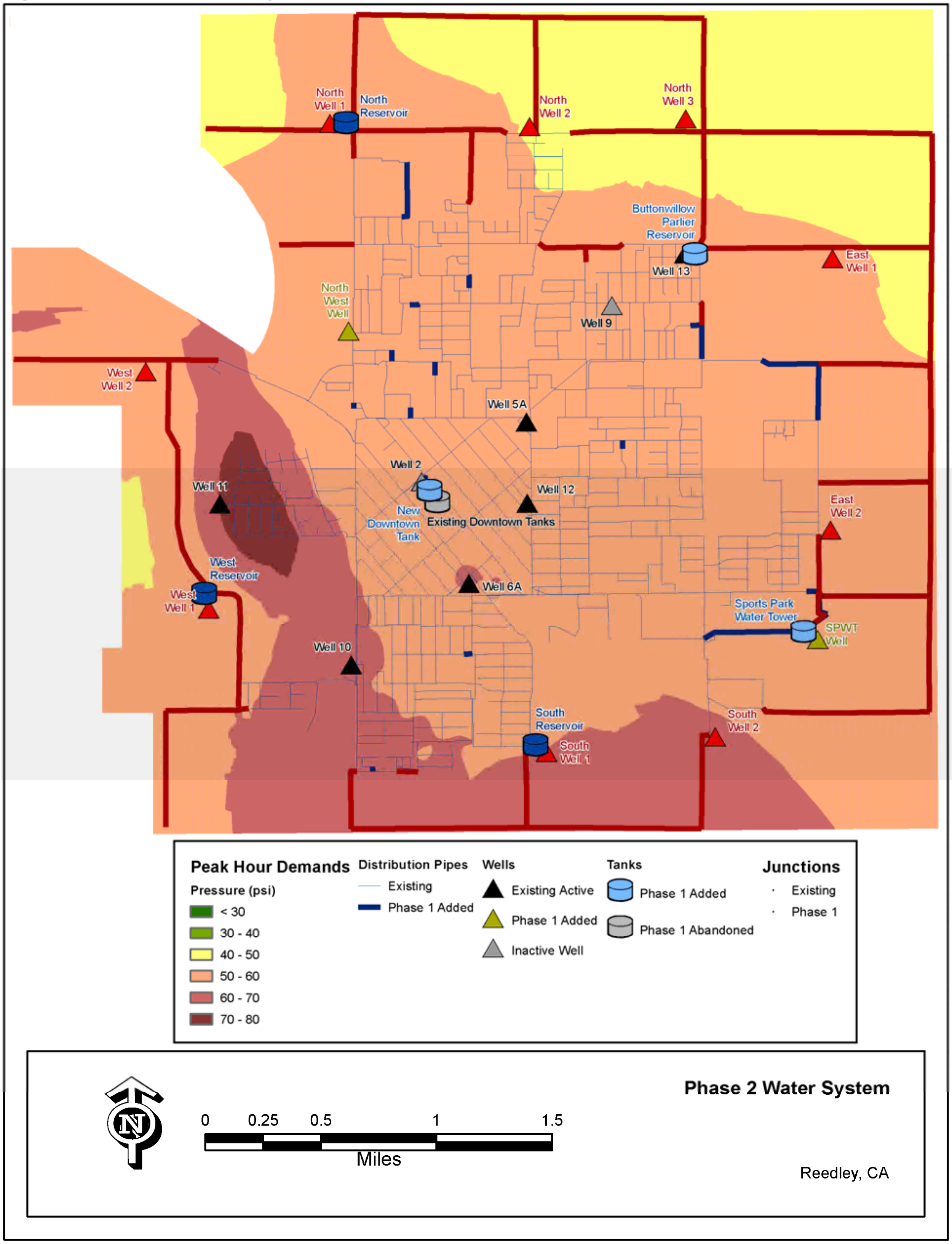
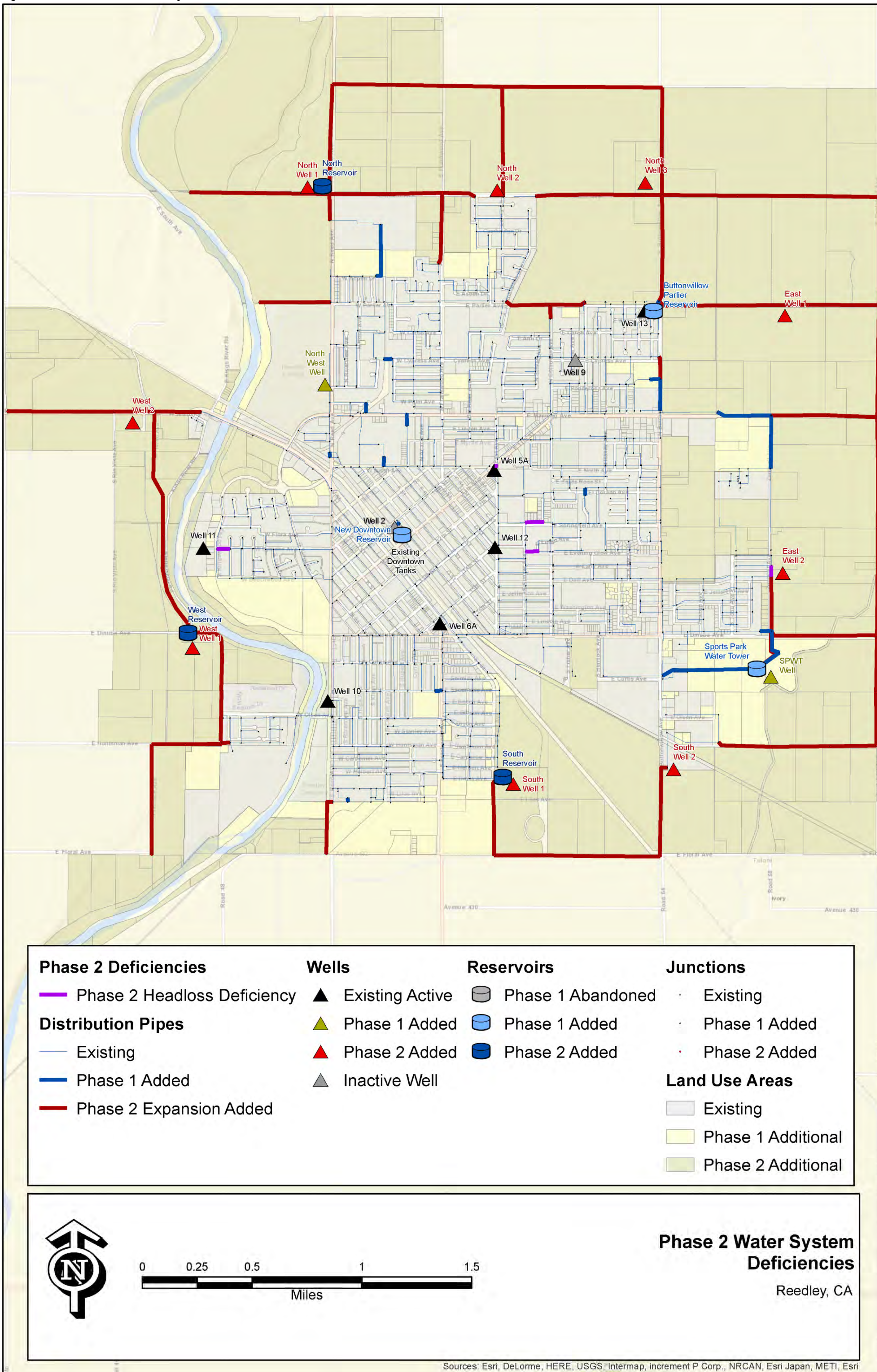


Figure 3-10. Phase 2 Water System Deficiencies



3.5.5. Proposed Improvements

Based upon the deficiencies analysis described in the previous subsection, recommended improvements were developed to achieve the design criteria presented in Table 3-8. In addition, new supply and storage facilities were distributed throughout the water system to provide coverage across the system. However, where possible, storage and well facilities are proposed to be collocated to minimize future operation and maintenance requirements and to reduce property acquisition costs. The proposed upgrades for each phase are shown in Figure 3-11 and Figure 3-12 and are described below.

Proposed Well Improvements

Table 3-10 summarizes the new well facilities that are required for the Existing, Phase 1 and Phase 2 planning periods and their proposed locations. As shown, it was assumed that each new well could achieve a capacity of 2,000 gpm. Additional studies will be needed to determine the potential yield for all future well facilities. If the yield is determined to be less than 2,000 gpm per well on average, additional wells will be required to make up the difference.

Table 3-10. Recommended Water System Well Upgrades

Phase	Project ID	Well Name	Proposed Location	Well Capacity (gpm)
Existing	P1WE-1	Sports Park Well	Zumwalt Ave and Dinuba Ave	2,000
Existing	PEWU-1[(a)]	Well 5A	Add an additional bowl to existing well pump	--
Existing	PEWU-2[(a)]	Well 10	Add an additional bowl to existing well pump	--
Existing	PEWU-3[(a)]	Well 12	Add an additional bowl to existing well pump	--
Phase 1	P1WE-2	North Central Well	Reed Ave between Kip Patrick and Ponderosa	2,000
Phase 2	P2WE-1	North Well 1	Reed Ave and South Ave	2,000
Phase 2	P2WE-2	North Well 2	South Ave and Sunny Ln	2,000
Phase 2	P2WE-3	North Well 3	Buttonwillow Ave and South Ave	2,000
Phase 2	P2WE-4	East Well 1	Zumwalt Ave and Parlier Ave	2,000
Phase 2	P2WE-5	East Well 2	Zumwalt Ave and Springfield Ave	2,000
Phase 2	P2WE-6	South Well 1	Church Ave and Lilac Ave	2,000
Phase 2	P2WE-7	South Well 2	Buttonwillow Ave and Huntsman Ave	2,000
Phase 2	P2WE-8	West Well 1	Huntsman Ave and Wallace Ave	2,000
Phase 2	P2WE-9	West Well 2	Manning Ave and Nurmi Ave	2,000
			Total New Well Capacity	22,000

(a) Proposed upgrades allow existing wells to essentially maintain their existing capacity once the system HGL is increased following the completion of the SPWT project.

Figure 3-11. Existing and Phase 1 Water System Upgrades

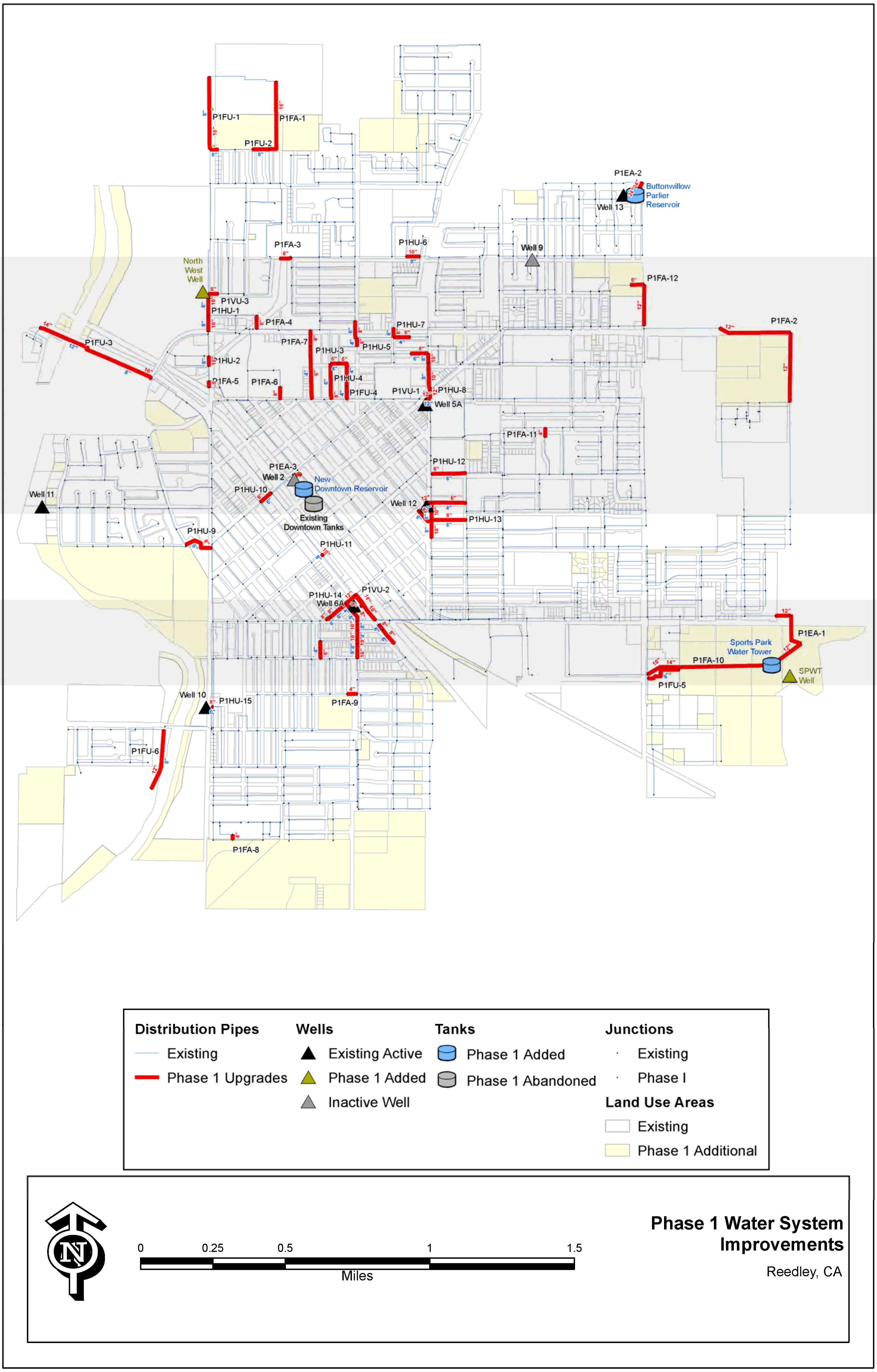
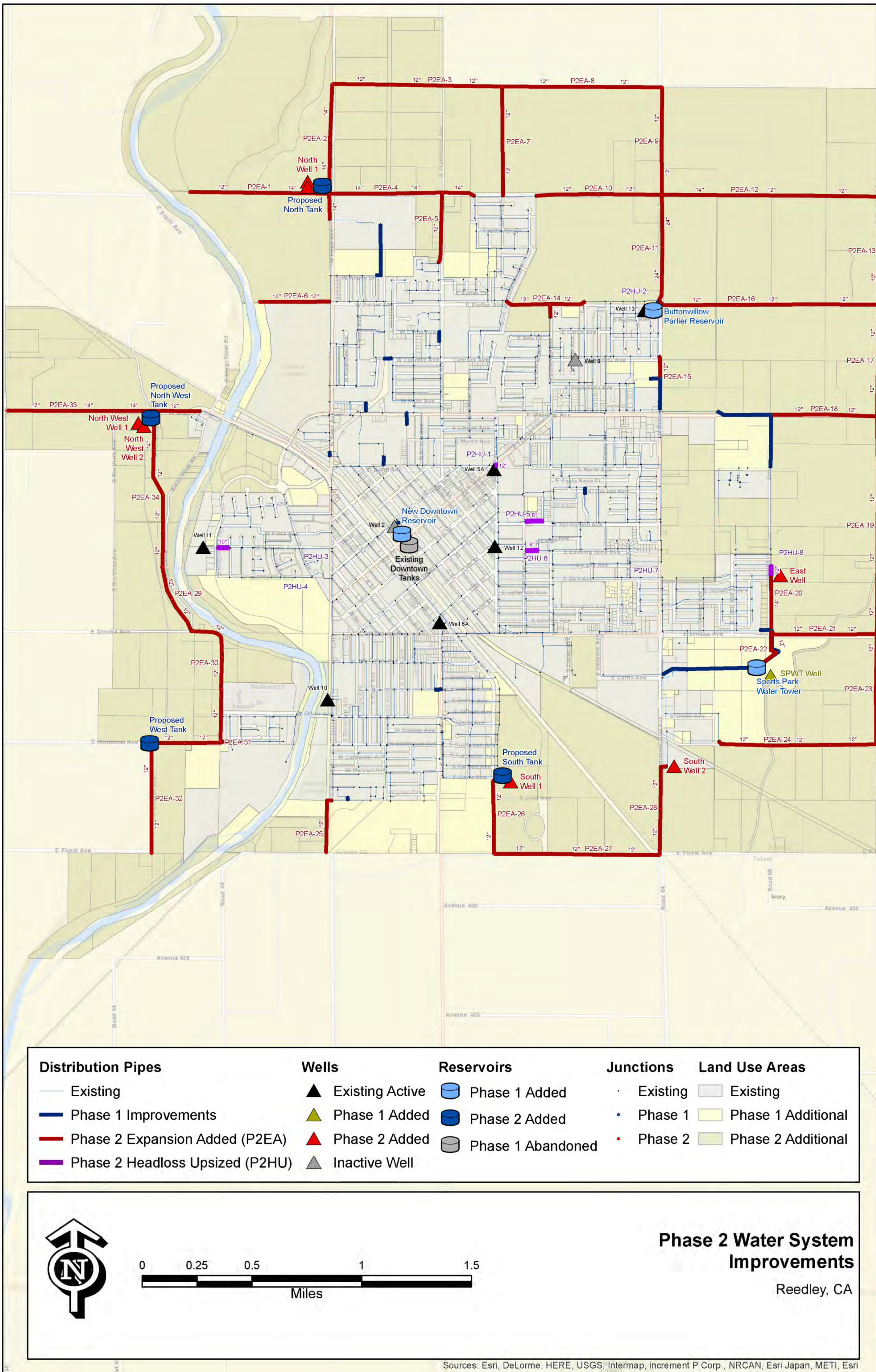


Figure 3-12. Phase 2 Water System Upgrades



The proposed well locations presented in Table 3-10 are intended to provide a general location only. Future studies will be required to identify actual locations and those studies should consider property acquisition needs, proximity to demands and other system facilities (e.g., storage), and proximity to other wells in order to minimize localized groundwater drawdown issues.

It is also important to restate that in conducting water quality tests for the Sports Park Well, the City determined there was a TCP contamination plume below the site, and as a result GAC treatment was required for the new well. It is possible that future wells could have similar contamination issues; however, for the purposes of developing the cost estimates presented in Section 6, additional treatment has not been included at this time. Additional studies will be required prior to design to determine if future wells will have similar requirements. In addition, the City should continue to monitor the development of the Chromium VI Rule to determine if additional treatment will be required.

Proposed New Storage Facilities

Table 3-11 summarizes the recommended new storage facilities for the Existing, Phase 1, and Phase 2 planning periods and their proposed locations. The location and volume of storage facilities, particularly for Phase 2, should be coordinated with the location and water demand of new developments as they are planned to come online. Additional studies will be needed to determine the specific location of these facilities as well. For example, it may be possible and more cost effective to locate the New Downtown Reservoir at the site of the City's existing (and abandoned) Well 2.

Table 3-11. Recommended Water System Storage Upgrades

Phase	Project ID	Tank Name	Proposed Location	Volume (MG)
Existing	P1RE-2	Sports Park Water Tower[(a)]	Zumwalt Ave and Dinuba Ave	1.4
Existing	P1RE-3	Buttonwillow Reservoir	Buttonwillow and Parlier Ave	2.0
Phase 1	P1RE-1	New Downtown Reservoir	10th St and H St	1.0
Phase 2	P2RE-1	North Reservoir	Reed Ave and South Ave	1.4
Phase 2	P2RE-3	West Reservoir	Huntsman Ave and Wallace Ave	1.5
Phase 2	P2RE-2	South Reservoir	Davis Ave and East Ave	1.2
			Total New Storage	8.5

(a) The Sports Park Water Tower is under construction.

Proposed Pipeline Improvements

Table 3-12 provides a list of the recommended distribution pipeline upgrades for the Existing, Phase 1, and Phase 2 planning periods. For each project, the project driver (i.e., fire flow, velocity, headloss, or expansion) is identified. In addition, the new pipeline diameter, length, and a unique Project ID label are also provided.

To upgrade the existing system to meet the design criteria presented in Table 3-8, approximately 22,000 LF of pipeline needs to be upgraded. In Phase 1, approximately 12,000 LF are needed to meet the design criteria and expand the system to serve new development. As expected, Phase 2 has the most pipeline recommendations (approximately 144,000 LF) because the system must be significantly expanded with new transmission mains to serve future development outside the City's current boundary. Note that the location and length of future distribution pipelines will be driven by actual development, and as a result, has not been included in this analysis.

The upgraded pipelines range in diameter from 6- to 24-inch. Additional studies will be required to determine the appropriate method(s) and materials for construction for each project.

For the Existing phase, priority for improvements should be given to those improvements that are needed to improve fire flow in the distribution system. Although velocity and headloss criteria were also defined in Table 3-8, these improvements are secondary as they are recommended to improve system efficiencies.

In order to determine the relative priority of the recommended upgrades, the existing system was evaluated under Existing and Phase 1 demands to determine where fire flow deficiencies were most severe. Priority is given to those recommended upgrades that resolve deficiencies in which less than 75 percent of the required fire flow is being delivered at a hydrant. Following that, other fire flow deficiencies have the second priority, followed by velocity and headloss deficiencies. The recommended upgrades are illustrated according to these priorities in Figure 3-13. These priorities are also identified in Table 3-12.

Integration of Expected Development with Proposed Improvements

For all of the projects planned for Phase 2, as identified in Table 3-10, Table 3-11, and Table 3-12, the City should carefully track the timing, location, and demand of future developments such that projects can be tailored, prioritized, and phased to meet the demands of the system where and when they occur.

Section 6 of this Master Plan presents the integrated CIP for the water, sewer, and storm drainage facilities, as well as cost estimates for the recommended projects.

Figure 3-13. Existing and Phase 1 Pipeline Improvement Priorities

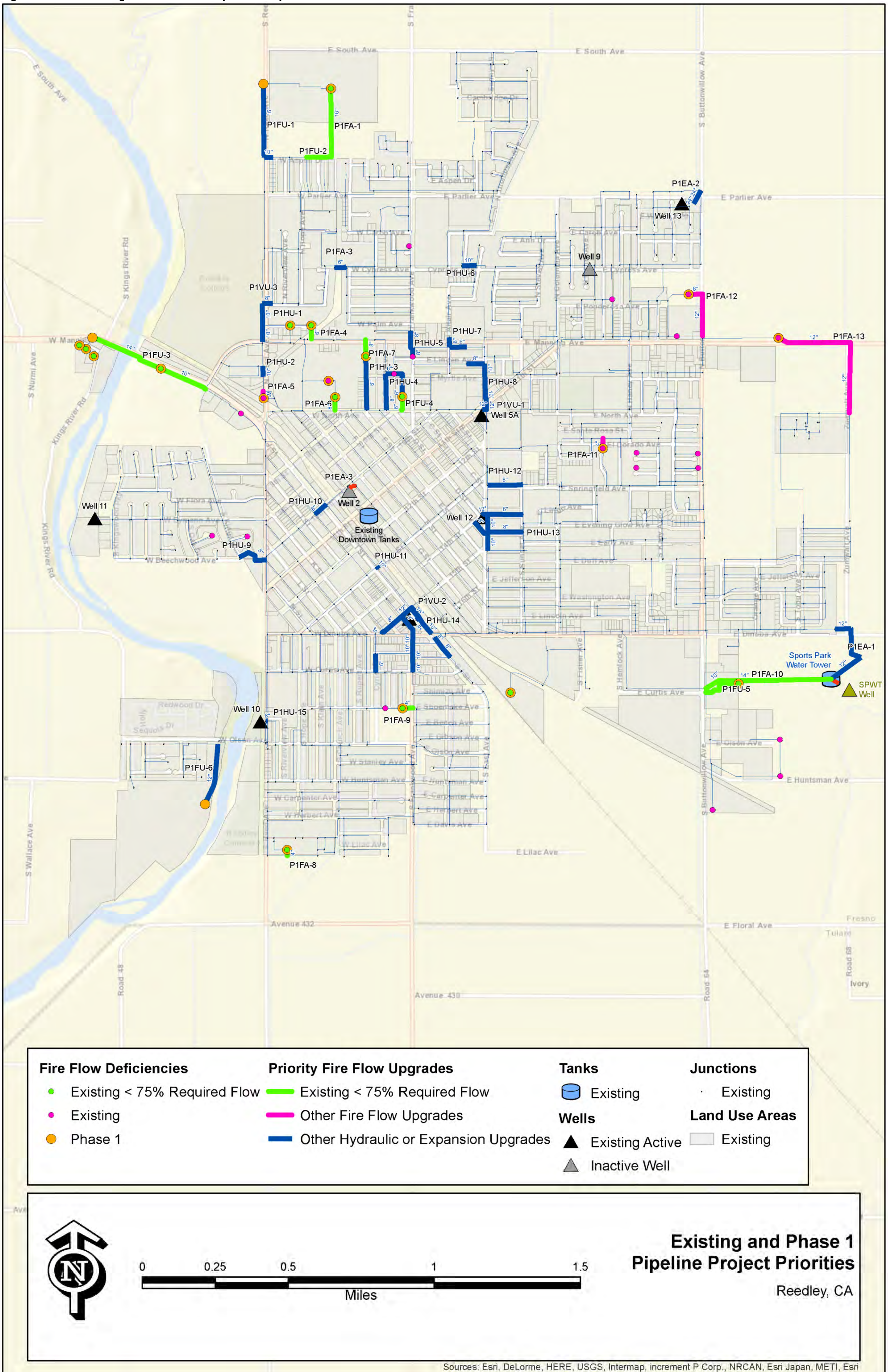


Table 3-12. Recommended Water System Distribution Pipeline Upgrades

Phase	Project ID	Priority[a]	Driving Criteria	Location	Diameter (in)	Length (feet)
Existing	P1EA-1	1	Existing Storage Deficiency	Sports Park Water Tower Appurtenance Piping	12 - 24	1,320
Existing	P1EA-2	1	Existing Storage Deficiency	Buttonwillow Parlier Tank Appurtenance Piping	10	720
Phase 1	P1EA-3	1	Phase 1 Expansion	Downtown Tank Appurtenance Piping	24	160
Existing	P1FA-1	1	Fire Flow Loop Added	W Aspen Dr & N Church Ave	16	1,380
Phase 1	P1FA-3	3	Fire Flow Loop Added	West Cypress Ave & N Hollywood Dr	6	230
Existing	P1FA-4	1	Fire Flow Loop Added	West of W Palm Ave & W Manning Ave	6	250
Existing	P1FA-5	2	Fire Flow Loop Added	North of N Reed Ave & W North Ave	6	130
Existing	P1FA-6	1	Fire Flow Loop Added	W North Ave & F St	8	240
Existing	P1FA-7	1	Fire Flow Loop Added	East of W Palm Ave & W Manning Ave	8	310
Existing	P1FA-8	1	Fire Flow Loop Added	Palm Village Retirement Community	6	110
Existing	P1FA-9	1	Fire Flow Loop Added	E Shoemake Ave & S Frankwood Ave	6	200
Existing	P1FA-10	1	Fire Flow Loop Added	E Curtis Ave & S Buttonwillow Ave	14	1,680
Existing	P1FA-11	2	Fire Flow Loop Added	El Dorado Ave	6	180
Existing	P1FA-12	2	Fire Flow Loop Added	E Manning Ave & N Buttonwillow Ave	6 - 12	1,020
Existing	P1FA-13	2	Fire Flow Loop Added	E Manning Ave & Zumwalt Ave	12	5,160
Phase 1	P1FU-1	3	Fire Flow Upsized	North Reed Ave & W Aspen Dr	10 - 16	1,500
Existing	P1FU-2	1	Fire Flow Upsized	W Aspen Dr	10	320
Existing	P1FU-3	1	Fire Flow Upsized	West Manning Ave	14 - 16	2,300
Existing	P1FU-4	1	Fire Flow Upsized from 4" to 6"	N Birch Ave	6	250
Existing	P1FU-5	1	Fire Flow Upsized from 8" to 10"	E Curtis Ave & S Buttonwillow Ave	10	720
Phase 1	P1FU-6	3	Fire Flow Upsized from 8" to 12"	W Olson Ave & West Bank of King's River	12	1,130
Existing	P1HU-1	3	Headloss Upsized from 8" to 10"	North of N Reed Ave & W Manning Ave	10	710
Phase 1	P1HU-2	3	Headloss Upsized from 8" to 10"	South of N Reed Ave & W Manning Ave	10	200
Phase 1	P1HU-3	3	Headloss Upsized from 4" to 6"	N Hollywood Dr	6	980
Existing	P1HU-4	3	Headloss Upsized	W Myrtle Ave & N Acacia Ave, N Birch Ave	6 - 8	1,400
Existing	P1HU-5	3	Headloss Upsized from 4" to 8"	Manning Ave & N Frankwood Ave	8	520
Existing	P1HU-6	3	Headloss Upsized from 8" to 10"	Cypress Ave & Concord Ave	10	270

Phase	Project ID	Priority[a]	Driving Criteria	Location	Diameter (in)	Length (feet)
Existing	P1HU-7	3	Headloss Upsized	E Manning Ave & Del Altoir Ave	6 - 8	490
Existing	P1HU-8	3	Headloss Upsized	E 11th St & N East Ave	8 - 12	1,180
Phase 1	P1HU-9	3	Headloss Upsized from 6" to 8"	S Kings Drive Cir & Beechwood Ave	8	600
Phase 1	P1HU-10	3	Headloss Upsized from 6" to 8"	1st St & 10th St	8	280
Phase 1	P1HU-11	3	Headloss Upsized from 8" to 10"	1st St & 13th St	10	50
Existing	P1HU-12	3	Headloss Upsized from 6" to 8"	S East Ave & E Springfield Ave	8	640
Existing	P1HU-13	3	Headloss Upsized	S East Ave & E August Ave, E Myra Ave	6 - 12	2,420
Existing	P1HU-14	3	Headloss Upsized	W Dinuba Ave & S Frankwood Ave	6 - 12	3,410
Phase 1	P1HU-15	3	Headloss Upsized from 6" to 8"	South of S Reed Ave & Beech Ave - Well 10	8	40
Existing	P1VU-1	3	Velocity Upsized from 10" to 12"	E 11th St & N East Ave	12	80
Existing	P1VU-2	3	Velocity Upsized from 12" to 16"	1st St & S Frankwood Ave	16	290
Phase 1	P1VU-3	3	Velocity Upsized from 8" to 10"	Klein Ave at Reed Ave	10	180
Phase 2	P2EA-1		Phase 2 Expansion	E South Ave West of Reed Ave	12 - 14	4,950
Phase 2	P2EA-2		Phase 2 Expansion	Reed Ave North and South of E South Ave West	14	4,610
Phase 2	P2EA-3		Phase 2 Expansion	Sumner Ave between Reed Ave and Sunny Ln	12	9,130
Phase 2	P2EA-4		Phase 2 Expansion	South Ave between Reed Ave and Concord Ave	14	4,820
Phase 2	P2EA-5		Phase 2 Expansion	Frankwood Ave South of South Ave	12	1,730
Phase 2	P2EA-6		Phase 2 Expansion	W Parlier Ave West of Reed Ave	12	2,680
Phase 2	P2EA-7		Phase 2 Expansion	Sunny Ln Expansion North	12	3,970
Phase 2	P2EA-8		Phase 2 Expansion	Sumner Ave between Sunny Ln and Buttonwillow Ave	12	4,860
Phase 2	P2EA-9		Phase 2 Expansion	Buttonwillow Ave between Sumner Ave and South Ave	12	3,720
Phase 2	P2EA-10		Phase 2 Expansion	South Ave West of Buttonwillow Ave	12	2,040
Phase 2	P2EA-11		Phase 2 Expansion	Buttonwillow Ave between South Ave and Parlier Ave	24	3,830
Phase 2	P2EA-12		Phase 2 Expansion	South Ave between Buttonwillow Ave and Englehart Ave	12 - 14	8,640
Phase 2	P2EA-13		Phase 2 Expansion	Englehart Ave between South Ave and Parlier Ave	12	3,970
Phase 2	P2EA-14		Phase 2 Expansion	Parlier Ave and Columbia Ave	12	2,330
Phase 2	P2EA-15		Phase 2 Expansion	Buttonwillow Ave at Cypress Ave	12	550

Phase	Project ID	Priority[(a)]	Driving Criteria	Location	Diameter (in)	Length (feet)
Phase 2	P2EA-16		Phase 2 Expansion	Parlier Ave between Buttonwillow Ave and Englehart Ave	12	8,760
Phase 2	P2EA-17		Phase 2 Expansion	Englehart Ave between Parlier Ave and Manning Ave	12	3,970
Phase 2	P2EA-18		Phase 2 Expansion	Manning Ave between Zumwalt Ave and Englehart Ave	12	2,560
Phase 2	P2EA-19		Phase 2 Expansion	Englehart Ave between Manning Ave and Dinuba Ave	12	9,300
Phase 2	P2EA-20		Phase 2 Expansion	Zumwalt Ave between Duff Ave and Dinuba	14	1,310
Phase 2	P2EA-21		Phase 2 Expansion	Dinuba Ave between Zumwalt Ave and Englehart Ave	12	3,820
Phase 2	P2EA-22		Phase 2 Expansion	Sports Park Water Tower	12	1,260
Phase 2	P2EA-23		Phase 2 Expansion	Englehart Ave between Dinuba Ave and Huntsman Ave	12	3,930
Phase 2	P2EA-24		Phase 2 Expansion	Huntsman Ave West of Englehart Ave	12	6,380
Phase 2	P2EA-25		Phase 2 Expansion	Reed Ave North of Ave 432	12	1,400
Phase 2	P2EA-26		Phase 2 Expansion	East Ave South of Davis Ave	12	2,760
Phase 2	P2EA-27		Phase 2 Expansion	Floral Ave/ Ave 432 between East Ave and Buttonwillow Ave	12	5,450
Phase 2	P2EA-28		Phase 2 Expansion	Buttonwillow Ave North of Floral Ave	12	2,300
Phase 2	P2EA-29		Phase 2 Expansion	Kings River Road North of Dinuba Ave	12	1,730
Phase 2	P2EA-30		Phase 2 Expansion	Kings River Road between Dinuba Ave and Huntsman Ave	12	3,220
Phase 2	P2EA-31		Phase 2 Expansion	Huntsman Ave between Wallace Ave and Kings River Rd	12	2,800
Phase 2	P2EA-32		Phase 2 Expansion	S Wallace Ave	12	2,670
Phase 2	P2EA-33		Phase 2 Expansion	W Manning Ave between Lac Jac Ave and Kings River Rd	12 - 14	6,150
Phase 2	P2EA-34		Phase 2 Expansion	Nurmi Ave to Kings River Rd	12 - 14	8,450
Phase 2	P2HU-1		Headloss Upsized 12" to 14"	11th St and East Ave	14	70
Phase 2	P2HU-2		Headloss Upsized 10" to 12"	Parlier Ave East of Cedar Ave	12	210
Phase 2	P2HU-3		Headloss Upsized 4" to 6"	Eymann Ave between Kings Dr and Reed Ave	6	780
Phase 2	P2HU-4		Headloss Upsized 6" to 8"	Beechwood Ave between Oak Dr and Kings Dr	8	790
Phase 2	P2HU-5		Headloss Upsized 6" to 8"	Springfield Ave between Sunset Ave and Justine Ave	8	900
Phase 2	P2HU-6		Headloss Upsized 6" to 8"	Evening Glow Ave between Sunset Ave and Lingo Ave	8	640
Phase 2	P2HU-7		Headloss Upsized 6" to 8"	Early Ave and Kady Ave	8	30
Phase 2	P2HU-8		Headloss Upsized 10" to 12"	Zumwalt Ave and Duff Ave	12	270

Phase	Project ID	Priority[a]	Driving Criteria	Location	Diameter (in)	Length (feet)
Phase 2	P2HU-9		Headloss Upsized 8" to 10"	Eymann Ave between Kingswood Ave and Willow Glenn Dr	10	320
					Subtotal Existing Phase[b]	27,700
					Subtotal Phase 1	5,350
					Subtotal Phase 2	144,060
					Total	177,110

(a) Priority is provided only for the Existing and Phase 1 system. Upgrades for Phase 2 should be planned and prioritized based on the actual timing and location of new development.

(b) Includes projects identified for the Existing Phase and Existing/Phase 1.

4. Sanitary Sewer System

This section of the Master Plan describes the City's sanitary sewer system. It includes an inventory of existing system components, and provides information on existing and projected future wastewater flows. It then provides an analysis of the hydraulics of the sanitary sewer system under existing and future conditions and concludes with recommended improvements to overcome system deficiencies. The Master Plan addresses only the sanitary sewer system, not the City's wastewater treatment plant. The City prepared a separate Wastewater Treatment Plant Master Plan in 2006.

For information on proposed capital improvements, see Section 6 of this Plan.

4.1. Inventory of Existing Sanitary Sewer System

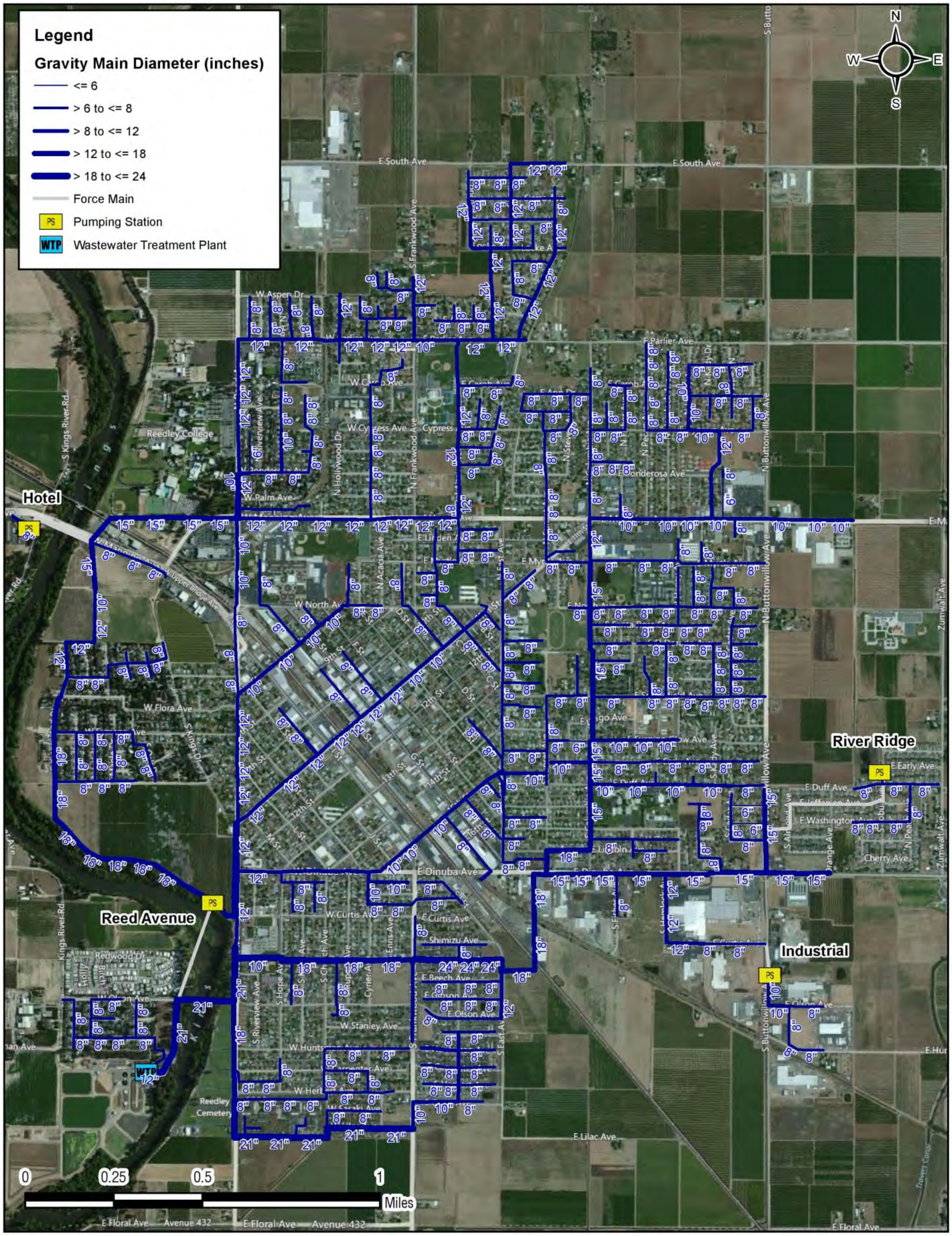
An inventory of the existing sanitary sewer system was developed for this Master Plan. The primary data source was the City's AutoCAD drawings of the existing sanitary sewer system. The existing system is shown in Figure 4-1.

The City's existing sanitary sewer system includes approximately 72 miles of pipeline. Pipe diameters were obtained from the City's AutoCAD drawing and from additional information provided by City staff. The sizes and lengths of sewer system piping are summarized in Table 4-1.

Table 4-1. Existing Sanitary Sewer Pipe

Diameter (inches)	Gravity Sewers (feet)	Force Mains (feet)	Total (Combined)	
			Length (feet)	Length (miles)
4	--	1,132	1,132	0.2
6	125,009	533	125,542	23.8
8	148,576	3,649	152,225	28.8
10	27,781	--	27,781	5.3
12	37,658	1,509	39,167	7.4
15	12,836	--	12,836	2.4
18	13,851	--	13,851	2.6
21	6,254	--	6,254	1.2
24	2,069	--	2,069	0.4
Total	374,034	6,823	380,857	72.1

Figure 4-1. Existing Sanitary Sewer System



Manhole rim and pipe invert elevations were obtained from the City's AutoCAD drawings. Manhole rim elevations were entered as attributes of the manholes, and upstream and downstream invert elevations were entered as attributes of the pipelines. Since these elevations are not considered to be completely reliable, key manhole locations were identified for subsequent field survey to validate and update the available AutoCAD data. As part of this project, 100 manholes, as identified in Figure 4-2, were surveyed by Douglas Johnson Land Surveying in May 2008. Invert elevations in the hydraulic model were then updated based on these survey results.

The locations of the City's four existing pump stations are illustrated in Figure 4-1. Site visits were conducted at the four pump stations to collect information pertinent to the hydraulic model, assess the existing condition of the pumping stations, and identify necessary improvements.

The Reed Avenue Pump Station is the City's largest pump station. Located on the east side of the Kings River, it pumps a significant portion of the City's flow across the river to the Wastewater Treatment Plant (WWTP). The other three pump stations are smaller facilities that pump flow from outlying areas. The sanitary sewer basins for each of the pumping stations are illustrated in Figure 4-3. Areas not shaded flow by gravity to the WWTP.

Characteristics of the existing pumping stations are summarized in Table 4-2. More detailed information collected during the site visits is included in Appendix F.

Table 4-2. Existing Sanitary Sewer Pump Stations

Pump Station	No. of Pumps	Design Flow (gpm)	Design Head (ft)	Wet Well Size (gal)	Other Info
Reed Avenue	3	550	40	3,430 gallons; 3 – 48-inch dia., 12 ft - 2 inches in depth	automatically controlled by ultrasonic level sensor
Hotel	2	150	45	3,970 gallons; 78-inch dia, 16 ft deep	automatically controlled by ultrasonic level sensor
Industrial	2	500	30	2,960 gallons; 72-inch dia, 14 ft deep	automatically controlled by ultrasonic level sensor
River Ridge	2	500	30	3,810 gallons; 72-inch dia, 18 ft deep	automatically controlled by ultrasonic level sensor

All flow from the sanitary sewer system is conveyed to the City's WWTP. The City recently completed an upgrade of its treatment plant, including the headworks.

Figure 4-2. Survey Locations for Manhole Rim and Invert Elevations

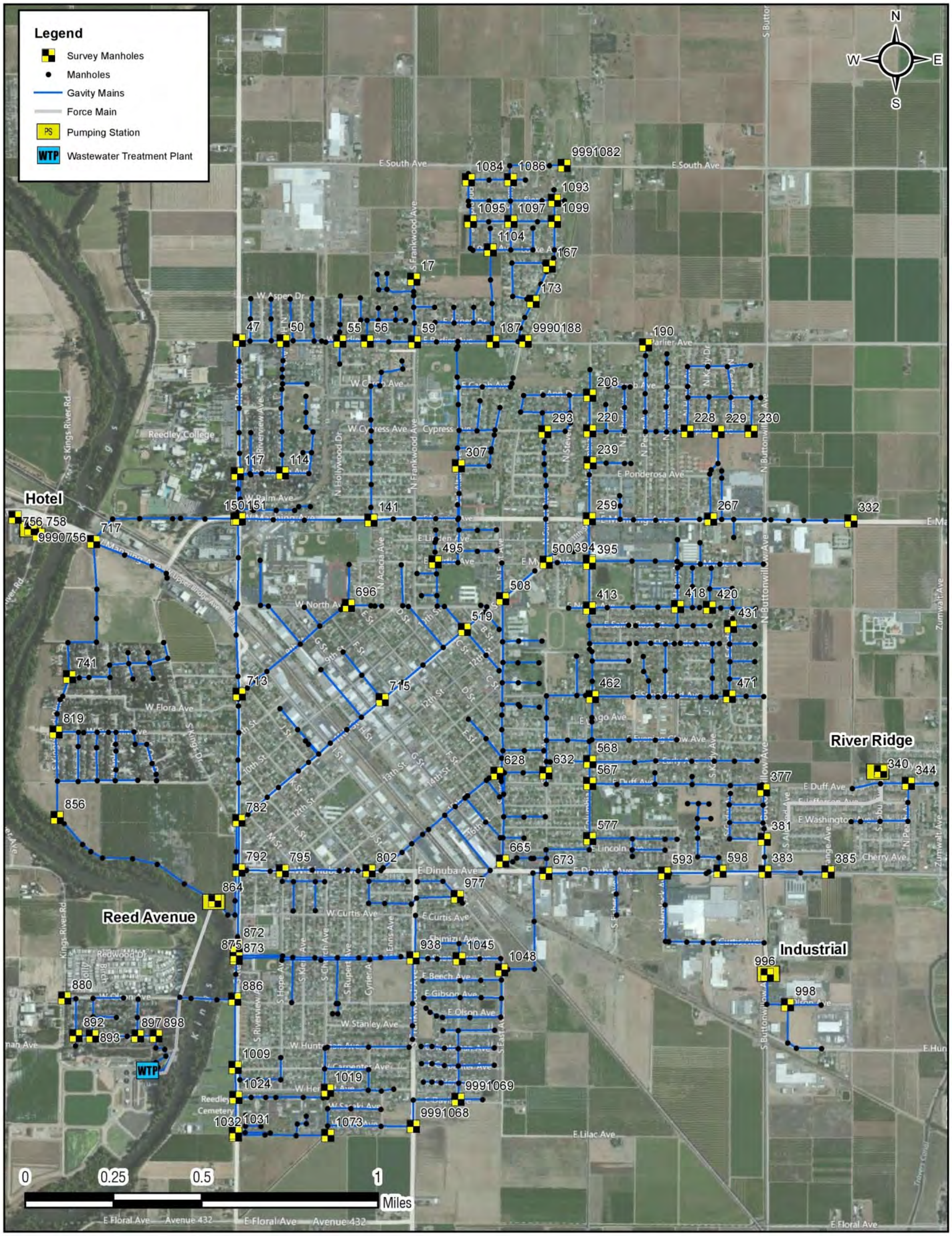
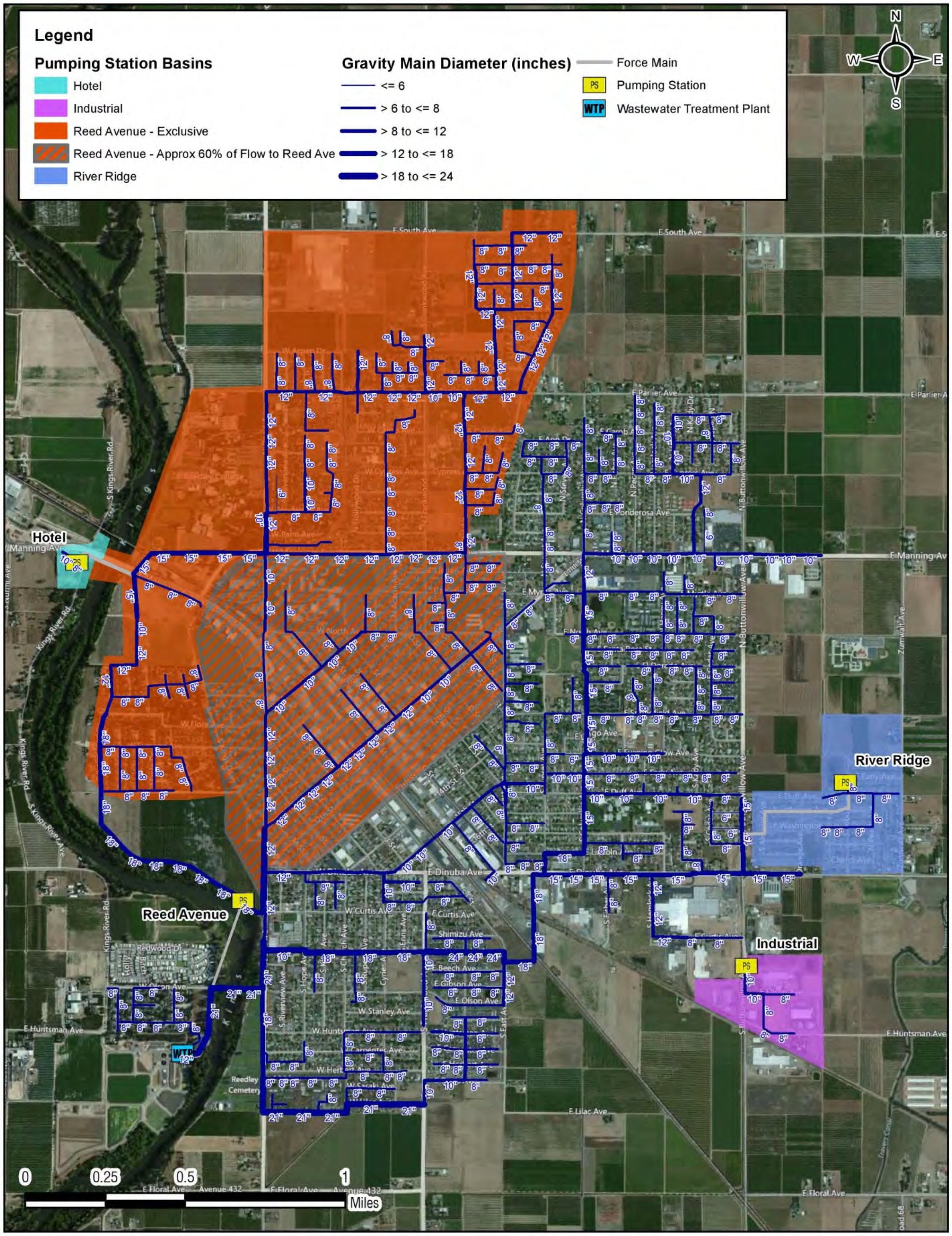


Figure 4-3. Sanitary Sewer Pumping Station Basins



4.2. Baseline and Projected Wastewater Flows

The following subsections describe the flow components that make up the sanitary sewer flow and the flow monitoring that was conducted to understand existing flows; it also presents the analysis of historical flows and presents future projected flows.

4.2.1. Flow Components

There are four components that make up the influent flow to the City's WWTP. These components include base sanitary flow (BSF), groundwater infiltration (GWI), rainfall-dependent infiltration (RDI), and storm water inflow (SWI). These four components are illustrated conceptually in Figure 4-4.

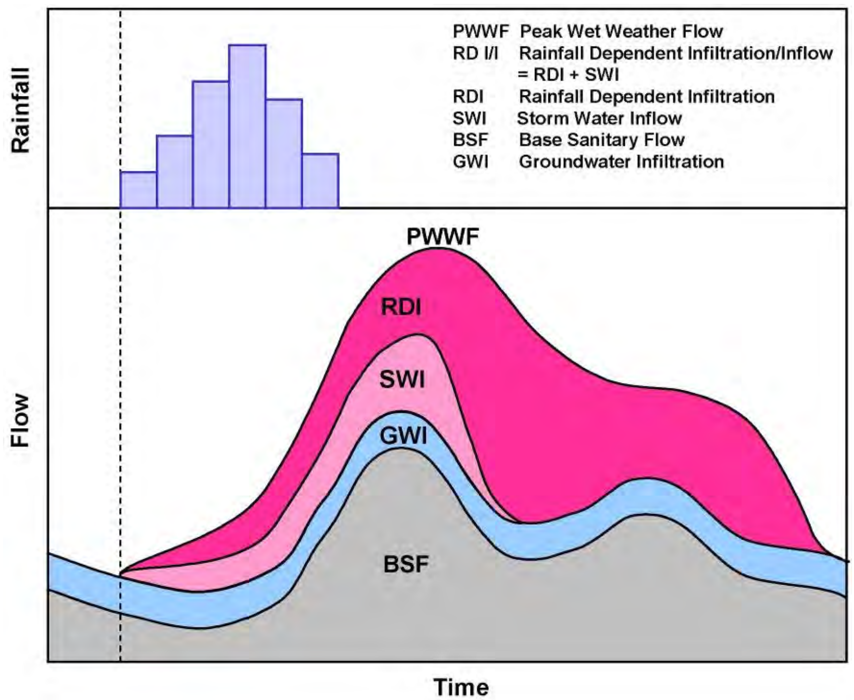


Figure 4-4. Flow Components[1]

Each of the four components illustrated in Figure 4-4 are described below.

- Baseline or base sanitary flow is composed of wastewater produced by facilities associated with residential, commercial, industrial, and institutional land uses. This is usually calculated by multiplying the flow generation factors (gallons per acre per day, gpad) for each land use category by the total acreage.

[1] V&A Consulting Engineers, Inc.

- Groundwater infiltration is defined as stormwater and/or groundwater that enters the sewer system through defects such as cracked pipes, offset joints, and leaky manholes.
- Storm water inflow is defined as storm water that enters the sewer system through improperly connected storm drains, down spouts and sump pumps.
- Rainfall dependent infiltration is infiltration that occurs in response to a storm event.

While each of these components makes up part of the flow to the WWTP, the identification and characterization of each component can require extensive data collection and manipulation. For this Master Plan, the components have been grouped into two categories: dry weather and wet weather flows.

- Dry weather flow (DWF) includes BSF and GWI. It includes typical flows in the system when no storm event is occurring. The average flow during these conditions is considered average dry weather flow (ADWF). The Regional Water Quality Control Board (RWQCB) typically defines ADWF as being equal to the lowest average of three consecutive dry season months.
- Wet weather flow (WWF) during storm events includes RDI and SWI (in addition to the dry weather flow). RDI and SWI are combined and termed rainfall-dependent inflow and infiltration (RD I/I). During a storm event, the peak wet weather flow (PWWF) will consist of the dry weather flow plus the RD I/I.

The wastewater flows were estimated using existing and future land use data, WWTP flow data, temporary flow monitoring and rain gauge data, and comparisons with other similar communities (e.g., Selma, Galt, and Riverbank). The daily WWTP influent flow data for February and March 2008 were analyzed and used to further refine the ADWF generation factors.

As described in the following section, although flow monitoring was conducted in the wet season in order to help develop the PWWF criteria, there was not a significant rainfall event. As a result, the PWWF was estimated by using a peaking factor based on annual average flow (AAF). When the new WWTP was designed, a peaking factor of 2.5 was selected based on an analysis of historical flow influent to the plant. Peak flows in the sewer system are generally attenuated as it moves downstream towards the WWTP. Therefore, it is common to see a higher peaking factor used for the collection system analysis than at the treatment plant. As a result, for this

Master Plan, the PWWF in the collection system was estimated using a peaking factor of 3.0 times AAF, which is similar to surrounding communities.

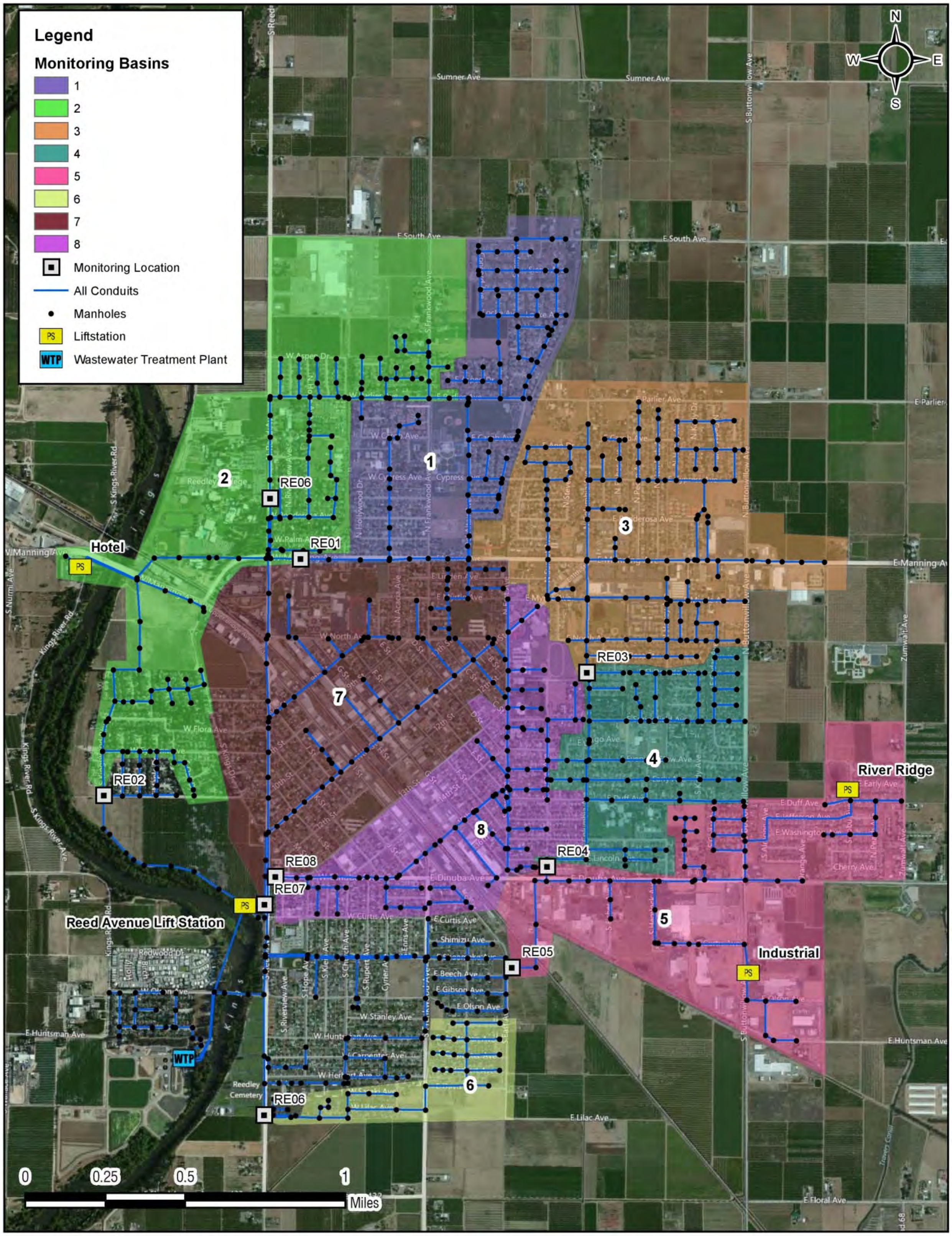
4.2.2. Temporary Flow Monitors and Rain Gauges

As part of this Master Plan, a temporary flow monitoring program was conducted. The monitoring period began on February 13, 2008. The purpose of the temporary flow monitoring program was to capture a significant storm event and use the resulting data to determine RD I/I. ADS Environmental Services (ADS) performed the temporary flow monitoring in the sanitary sewer system. A total of eight flow monitors were installed around the City and maintained for 28 days. The locations for the monitors are shown in Figure 4-5. Each monitor recorded velocity, depth, and flow every 15 minutes. During the monitoring period, ADS visited the monitors once a week to perform maintenance and verify that high-quality data were being collected. ADS also maintained one tipping-bucket rain gauge to record precipitation during the 28-day monitoring period.

According to the monitoring report from ADS, during the monitoring period, there were only two small storms, one on February 20 with 0.72 inches and the other on February 23 with 0.5 inches of rain accumulation. In addition, the sanitary sewer system showed minimal to no response to these rain events. Due to the lack of a significant rainfall event during the monitoring period, it was determined that the monitoring data were insufficient to estimate RD I/I.

The WWTP influent data during this period was provided by the City in a graphical format. In order to make use of this information for model calibration, the charts were interpreted into numerical format in 1-hour increments. Based on the interpreted hourly flow data, the average dry weather WWTP influent flow was approximately 2.3 mgd, which is very close to the known existing system ADWF. Therefore, for the purpose of this Master Plan, it is assumed that the flow monitoring data during this period was representative of the existing dry weather flow conditions and was suitable for model calibration.

Figure 4-5. Temporary Flow Monitoring Locations



HDR

4.2.3. Analysis of Historical Wastewater Flows

Daily wastewater flow data at the WWTP were compiled for 2005 through 2007. These data are shown in Table 4-3 and graphically in Figure 4-6. The average daily flow (arithmetic average) to the treatment plant was approximately 2.3 mgd. Aside from specific storm events, the flows are higher in the summer than in the winter, when most of the rain events occur. It is not clear if this trend is due to reduced water use in the winter, outdoor water use in the summer leading to increased infiltration, food processing water use in the summer, or some combination of these factors. The highest daily flow, with the exception of an anomaly, was 3.2 mgd on April 4, 2006, due to rain from a storm event.

Table 4-3. Summary of Wastewater Treatment Plant Flows, 2005 - 2007

Year	ADWF[(a)] (mgd)	Peak Daily Flow (mgd)	Notes
2005	2.2	2.8	
2006	2.3	3.2	Peak flow due to rain from storm event.
2007	2.2	3.5[(b)]	Pumped return activated sludge (RAS) through Headworks due to RAS pumping station problem.

(a) ADWF is based on lowest average plant influent of three consecutive dry season months.
(b) Not representative of high influent flow.

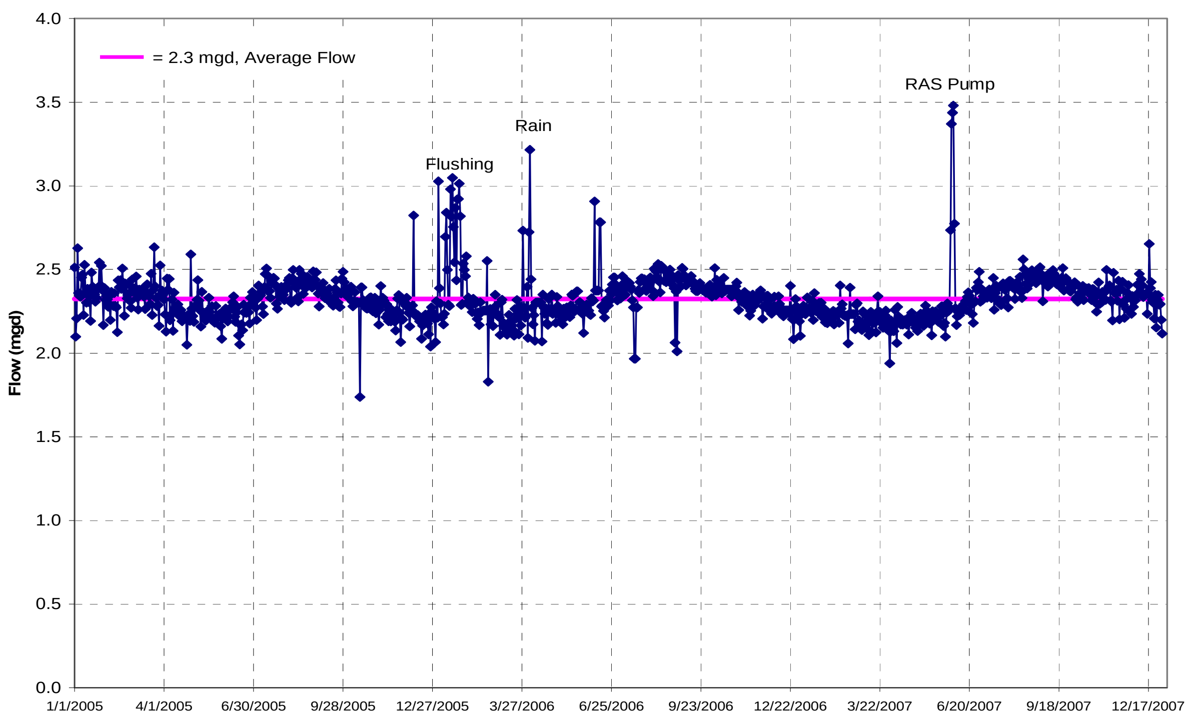


Figure 4-6. Wastewater Treatment Plant Influent Flows, 2005 - 2007

4.2.4. Projected Wastewater Flows

Wastewater flows were projected based on the projections of potable water demand described in Chapter 3. Potable water demands were first forecast using land use projections from the City's General Plan 2030.

For the various land use categories, estimates were developed of the percentage of water that enters the wastewater system, known as "return rate." Potable water demands were then multiplied by those return rates to estimate wastewater flows. The return rates and resulting flows are shown in Table 4-4.

Based on the growth forecast in the General Plan 2030, total flows entering the sanitary sewer system are projected to grow from approximately 2.5 mgd under existing conditions, to approximately 8.2 mgd at the buildout of Phase 2. Similarly, PWWF is projected to grow from approximately 7.6 mgd to over 24.7 mgd in Phase 2.

Table 4-4. Sanitary Sewer Flow Forecast

General Plan Land Use	Return Rate[a]	Sanitary Flow Generation Rate (gpd/acre)	Existing DWF (gpd)	Phase 1 DWF (gpd)	Phase 2 DWF (gpd)
Residential					
Suburban Residential	60%	1,140	7,500	10,800	312,200
Low Residential	60%	1,520	1,459,900	1,797,800	5,182,700
Medium Residential	60%	1,480	40,100	51,100	162,400
High Residential	60%	2,270	354,700	458,900	556,900
Commercial and Industrial					
Central Downtown	50%	1,350	53,900	53,900	53,900
Community Commercial	50%	1,140	105,000	126,500	489,600
Neighborhood Commercial	50%	1,280	14,200	28,800	56,000
Service Commercial	50%	1,200	90,600	93,800	155,400
Office	50%	940	15,300	15,500	15,500
Light Industrial	50%	740	105,700	167,000	597,800
Heavy Industrial	50%	1,280	69,300	69,600	227,300
Other					
Open Space	0%	-	-	-	-
Public/Institutional Facility	50%	570	233,100	291,800	411,900
Remainder of the Study Area	0%	-	-	-	-
System-Wide Total			2,549,300	3,165,500	8,221,600

(a) Sewer flows are calculated as a percentage (return rate) of the forecasted water demands listed in Chapter 3, after subtracting 10% from water demands to account for losses in the potable water distribution system.

4.3. Regulatory Requirements

Federal, state, and local regulatory agency policies and procedures affect the installation, upgrades, and operation of the City's sanitary sewer system. The impacts of these policies and procedures on wastewater management planning in the City are described below. The discussion of regulations presented is not exhaustive and is focused on those regulations and laws that are relevant to wastewater conveyance.

4.3.1. Federal Policies

Federal policies that will affect the planning process include the Federal Water Pollution Control Act/Clean Water Act; Safe Drinking Water Act; and the proposed Capacity, Management, Operation and Maintenance (CMOM) Rules.

Federal Water Pollution Control Act / Clean Water Act

Since its enactment, the Federal Water Pollution Control Act, also known as the Clean Water Act (CWA), has formed the foundation for regulations detailing specific requirements for response measures and pollution prevention. The CWA requires states to adopt water quality standards consistent with federal limitations on pollutant and thermal loading. The standards are to take into consideration the use of the waters for public water supplies; propagation of fish and wildlife; recreational purposes; and agricultural, industrial, and other beneficial uses. The City's sanitary sewer system conveys wastewater to the WWTP, where it is treated before being discharged to the Kings River. State policies specifically regulate pollutant and thermal loading to comply with federal policy as detailed in the CWA.

Safe Drinking Water Act

The Safe Drinking Water Act (SDWA) authorizes the USEPA to set national health-based standards for drinking water to protect against contaminants that may be found in drinking water. Wastewater flows that are collected in or travel through substandard sanitary sewer systems have the potential to contaminate drinking water systems. State and local regulations are designed to comply with the SDWA, and prohibit activities that could cause an adverse impact on existing or potential beneficial use of groundwater.

Proposed Capacity, Management, Operation, and Maintenance Rule

The USEPA has proposed rules that will govern the manner in which municipalities and special service districts manage and operate sanitary sewer systems. The proposed CMOM Rule, depending on its final form, may have a significant effect on sanitary sewer system development and O&M for the City. Under the proposed rule, sanitary sewer overflows (SSOs) would be prohibited unless caused by severe natural disasters. Owners of sanitary sewer systems would be required to provide adequate capacity for peak flows in all parts of the system, monitor and report on SSOs, and make the SSO control program and reports available for public review.

The proposed CMOM rule is vague on the design threshold to which SSOs must be controlled. For instance, the proposed rule does not specify the recurrence interval of a storm event above which SSOs would be allowed. As such, the USEPA offers wastewater agencies no clear guidance regarding the amount of additional pipeline and pump station construction that would be required under CMOM, nor an understanding about the amount of additional maintenance effort required to ensure elimination of SSOs.

The CMOM rule has not yet been implemented and the timeline for implementation is uncertain. How the CMOM rule will eventually be interpreted and applied in California is also uncertain. One possibility is that California's Waste Discharge Requirements (WDR) (see Section 4.3.2), will be used to set a minimum threshold for SSO prevention.

4.3.2. State Requirements

National Pollutant Discharge Elimination System

Section 402 of the CWA provides the legal basis for the NPDES permit program, which regulates point and nonpoint discharges. The State Water Resources Control Board (SWRCB) and the RWQCB are authorized by the USEPA to administer the NPDES program. These rules and statutes include regulations for wastewater collection, treatment, control, and disposal. Under the conditions of NPDES permits, permittees are allowed to construct, install, modify, or operate these systems only in conformance with the CWA and the State statutes that set forth requirements, limitations, and conditions for such activities.

Waste Discharge Requirements Program

As noted previously, EPA is currently considering proposed CMOM rules. While the proposed CMOM rule is silent on sanitary sewer system design criteria, California has already adopted a program to address sewer overflows. This program, commonly referred to as the Sanitary Sewer Overflow Reduction Program, provides guidance to design engineers and sanitary sewer system owners. To provide a consistent, statewide regulatory approach to address SSOs, the SWRCB adopted Statewide General Waste Discharge Requirements for Sanitary Sewer Systems. The Sanitary Sewer Systems WDR requires public agencies that own or operate sanitary sewer systems to develop and implement sewer system management plans and report all SSOs to the State Water Board's online SSO database.

To facilitate proper funding and management of sanitary sewer systems, each agency must develop and implement a system-specific Sewer System Management Plan (SSMP). The SSMP must include provisions to provide proper and efficient management, operation, and maintenance of sanitary sewer systems, while taking into consideration risk management and cost benefit analysis. Additionally, an SSMP must contain an overflow response program that establishes standard procedures for immediate response to a SSO in a manner designed to minimize water quality impacts and potential nuisance conditions. The City completed an SSMP in 2009 covering all required elements and is compliant with SSO reporting requirements.

4.4. Sanitary Sewer System Evaluation

A hydraulic model was used to identify system capacity deficiencies during peak wet weather flow for both existing and future conditions. The model was also used to evaluate improvements that could address key system deficiencies. This subsection describes the planning criteria used to perform the evaluation of the sanitary sewer system, the hydraulic model development and calibration, system deficiencies, and recommended system improvements.

4.4.1. Planning Criteria

Planning criteria were established to have a common set of metrics with which to evaluate the existing system and to use as the bases for developing system upgrade recommendations to address deficiencies and to expand the system to serve future growth. These planning criteria are summarized in Table 4-5. For additional information refer to Appendix B.

Table 4-5. Sanitary Sewer System Planning Criteria

Parameter		Value
Evaluation Criteria for Gravity Sewers	PDWF	d/D > 0.5
	PWWF	d/D >= 1.0
Design Criteria for New Gravity Sewers	PWWF	d/D >= 0.75
Evaluation Criteria for Pump Stations	Design Flow	Pump design flow with largest pump out of service
Hydraulic Planning Criteria	Manning's Roughness Coefficient for Gravity Sewers	0.013
	Hazen-Williams Roughness Coefficient for Force Mains	100

4.4.2. Model Development and Calibration

The hydraulic model was developed using the Innovyze's InfoSWMM version 9.0 hydraulic modeling platform (model). InfoSWMM uses the US EPA's SWMM V5 simulation engine, which is considered a fully dynamic (as opposed to steady state) engine that can account for various complex hydraulic phenomena in the sanitary sewer system, such as surcharging and backwater effects in gravity mains.

The model was loaded with (annual) average dry weather flow and calibrated against the flow data observed in early spring of 2008. The calibration focused primarily on matching the observed peak dry weather flow (PDWF) and observed hydrograph volume and secondarily on matching the observed hydrograph shape and the time of the peak. For additional information refer to Appendix H.

Model Development

In InfoSWMM, the sanitary sewer system is modeled as a network of nodes and links. Manholes, connectivity nodes, fittings, outlets and wet wells are modeled as nodes; and links are used to represent gravity mains, force mains, pumps and flow diversion structures.

As a starting point for model development, the existing pipe network was imported from a GIS dataset, including a link and a node layer. The GIS dataset was created from an AutoCAD drawing provided by the City. Refer to Appendix H for additional information regarding the original development and calibration of the sanitary sewer system hydraulic model.

Using information provided by the City in May 2012, the model was updated and re-calibrated. Many of the updates made to the model were based on as-built drawings provided by the City.

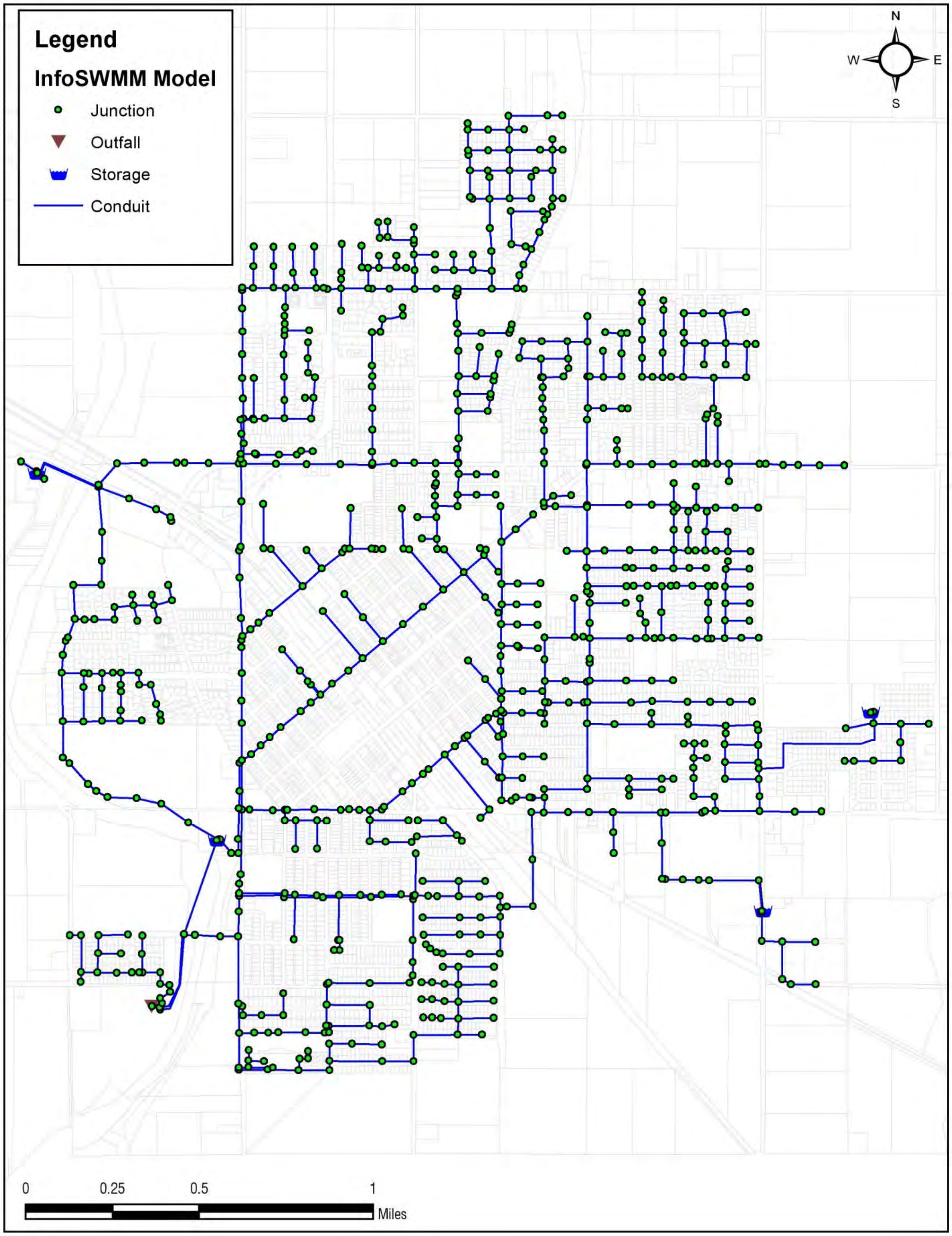
Some invert elevations were interpolated in order to maintain the desired general slope and flow paths indicated in the materials provided by the City. Because some invert elevations in the model were initially based on ground elevation and assumed pipe depth, some as-built inverts do not exactly match what is in the model. For these cases, assumptions were made in order to satisfy the intended hydraulics of the system.

The active existing system modeled pipe network includes 861 pipe segments, of which seven pipe segments are force mains. Figure 4-7 shows the modeled sanitary sewer system and Table 4-6 summarizes the pipe segments included in the model.

Table 4-6. Summary of Modeled Pipes by Diameter

Type	Diameter (inch)	No. of Pipes (count)	No. of Pipes (%)	Length (ft)	Length (mile)	Length (%)
Gravity Main	6	5	1	2,565	< 1	1
	8	526	61	148,514	28	58
	10	94	11	27,781	5	11
	12	126	15	35,649	7	14
	15	43	5	12,836	2	5
	18	39	5	13,851	3	5
	21	15	2	6,254	1	2
	24	6	1	1,462	< 1	1
	Subtotal	854	99	248,911	47	97
Force Main	4	1	< 1	1,132	< 1	< 1
	6	1	< 1	533	< 1	< 1
	8	2	< 1	3,641	1	1
	12	1	< 1	1,509	< 1	1
	21	2	< 1	1,451	< 1	1
	Subtotal	7	1	8,265	2	3
Grand Total		861	100	257,176	49	100

Figure 4-7. Modeled Elements of Existing System



There is only one outlet in the model and it represents the headworks at the City's WWTP. The outlet also defines the boundary conditions of the model. It is assumed that the wastewater is freely discharged into the wetwell in the headworks and therefore, the boundary water surface condition at the outlet is set to be the normal flow at the connecting pipe instead of a set hydraulic grade line. As a result, any backwater effects of the headworks, if present in the system, are not included in the model.

As previously described, the sanitary sewer system includes four lift stations -- the Reed Avenue, Hotel (Edgewater Inn), Industrial, and River Ridge Lift Stations. The Hotel and Industrial lift stations are equipped with constant speed pumps, while the Reed Avenue and River Ridge lift stations have variable speed pumps. The wet well dimensions associated with each lift station are shown in Table 4-7.

Table 4-7. Wet Well Dimensions

Lift Station Name	Diameter (ft)	Bottom Elevation (ft)	Depth (ft)
Reed Ave	6.9[(a)]	293	12
Hotel	6.5	298	16
Industrial	6	325	14
River Ridge	6	325	18

(a) This is the equivalent diameter of three 48-inch chambers.

Wastewater Flow Allocation

In order to represent the existing dry weather flow conditions in the model, the ADWF is allocated to the model nodes. Based on the literature, there are several methods to allocate the wastewater flows. The adopted allocation approach for this task is based on land uses, water demands and applicable water return rates (the fraction of potable water that enters the sewers). As described in Subsection 4.2.4, for various land use categories, the estimated wastewater flow generated was developed by applying a return rate (i.e., percentage of water that enters the wastewater system) to the potable water demand for a specific parcel (based on land use type). Refer to Table 4-4 for a summary of the return rates and resulting flows.

Model Calibration

As previously described, flow monitoring was conducted for a 28-day period from February 14 to March 18, 2008. The flow monitoring data was used in conjunction with the WWTP influent flow data during that period to calibrate the model.

The goal of the model calibration was to minimize the difference between the model-predicted flow and the observed flow in the field. It is expected that the model can predict reasonably accurate wastewater depths after the flow in the model is calibrated to the field conditions and therefore can be used for deficiency analysis. The calibration focused primarily on matching the observed PDWF and ADWF and secondarily on matching the hydrograph shape and the time of the peak. The goal of the model calibration was to match the PDWF (hydrograph peak) and ADWF (hydrograph volume) at the eight flow monitoring locations plus the WWTP headworks to within 5 percent of the observed values.

The calibration was performed for each basin in the sanitary sewer system. The ADWF was first verified and adjusted if necessary before the PDWF calibration was performed. Table 4-8 summarizes the calibration results at the eight flow monitoring locations and the WWTP headworks. As shown, the calibration was successful in matching the modeled flows to within 5 percent of the observed flows.

Table 4-8. Sanitary Sewer Collection Model Calibration Summary

Location	Pipe ID in Model	Pipe Size (in)	Target ADWF (mgd)	Model ADWF (mgd)	% Difference in ADWF	Target PDWF (mgd)	Model PDWF (mgd)	% Difference in PDWF
RE01	95	12	0.15	0.15	< 1	0.24	0.24	2
RE02	54	18	0.47	0.47	< 1	0.68	0.64	5
RE03	578	15	0.61	0.61	< 1	0.87	0.87	1
RE04	532	18	0.94	0.94	< 1	1.32	1.30	1
RE05	1105	18	1.06	1.04	1	1.46	1.43	2
RE06	971	21	0.01	0.01	1	0.03	0.03	3
RE07	123	18	0.33	0.33	1	0.46	0.44	4
RE08	281	12	0.29	0.29	1	0.43	0.41	5
WWTP	CDT-11	N/A	2.34	2.47	5	3.20	3.23	1

RE = monitoring location (see Figure 4-5)

System Evaluation

Following calibration, the existing system was evaluated under PWWF conditions to analyze the current capacity of the system. PWWF conditions were simulated by loading PWWF flows to the sanitary sewer system in the InfoSWMM model and running the scenario in steady state. PWWF was estimated by using a peaking factor based on the AAF. When the City's WWTP was designed, a peaking factor of 2.5 was selected based on an analysis of historical flow influent to the treatment plant. Peak flow in the sanitary sewer system is generally attenuated as it moves through the system, downstream to the WWTP. Thus, it is common to see a higher peaking factor used for a sewer system analysis than for the downstream WWTP. For this Master Plan, the PWWF in the sewer system was estimated by using a peaking factor of 3.0 times the AAF (refer to Appendix B for a discussion of the PWWF peaking factor). The resulting flows loaded into the model are shown in Table 4-9 for each scenario, respectively.

Table 4-9. Modeled Flows by Scenario

Flow Scenario	Peaking Factor	Existing (mgd)	Phase 1 (mgd)	Phase 2 (mgd)
Average Annual Flow	1	2.55	3.16	8.21
Peak Wet Weather Flow	3	7.65	9.48	24.63

4.4.3. System Deficiencies

As noted in the previous subsection, the sanitary sewer system was evaluated under PWWF conditions to identify deficiencies in the existing system and under Phase 1 and Phase 2 conditions. The results of the analyses are described below.

Lift Stations

As described in Subsection 4.4.1, the evaluation criteria applied for existing lift stations is to provide the design flow of the lift station with the largest pump out of service. Each of the existing lift stations were evaluated to determine if they could pump the design flow with the largest pump out of service. Based on this analysis, it was determined that the Reed Avenue Lift Station should be expanded to provide an additional standby pump.

Sanitary Sewer Pipelines

The results of the hydraulic analysis of the sanitary sewer pipelines for the existing, Phase 1 and Phase 2 conditions are shown in Figure 4-8 through Figure 4-13 and summarized below.

To better understand the deficiencies in the existing system, the results for the existing and Phase 1 systems were analyzed with both d/D and q/Q factors to determine available capacity during PWWF. The d/D ratio refers to the ratio of water depth in the pipeline to the full diameter of the pipeline, with a full pipeline having a d/D equal to one. Similarly, q/Q refers to the ratio of modeled flow in a pipeline to the calculated maximum flow the pipeline can accommodate. Maximum flow is calculated based on the flow equation using only the pipe properties and disregarding the flows in the upstream and downstream pipes (e.g., backwater effects, bottlenecks, etc.). The q/Q factor is often used to help better understand the underlying cause of the d/D factor being greater than 1.0. Specifically, the q/Q factor identifies whether a particular pipe segment is capacity deficient (e.g., q/Q > 1.0) or merely experiencing backwater effects due to downstream bottlenecks. Thus, the q/Q factor can be helpful in prioritizing recommended upgrades.

- **Figure 4-8. Existing Sewer System PWWF d/D Deficiencies**: identifies the location of flow restrictions for the existing system under PWWF conditions. Approximately 259 pipe segments (30 percent) were found to exceed the d/D capacity criteria; however, much of this surcharging was due backwater conditions caused by downstream flow conditions.

- **Figure 4-9. Existing Sewer System PWWF q/Q Deficiencies**: identifies the location of capacity deficiencies for the existing system under PWWF conditions. For those pipelines shown in red (approximately 55 segments), where the q/Q exceeds 1.0, manholes are surcharging and could result in SSOs if the HGL in surcharged pipelines exceeds the ground level elevation. The model currently predicts that the existing system would experience one SSO under PWWF conditions. Given that SSOs can result in monetary fines, the improvements needed to correct these deficiencies should be a high priority.

- **Figure 4-10: Phase 1 System PWWF d/D Deficiencies**: illustrates the capacity deficiencies in the existing system under the Phase 1 flow conditions. As illustrated, more than half of the pipe segments in the system (476 segments) are expected to experience surcharging and backwater effects. In addition, 15 manholes are expected to overflow.

- **Figure 4-11: Phase 1 System PWWF q/Q Deficiencies**: illustrates that while much of the system is experiencing backwater effects (per Figure 4-10), a much smaller portion of those same pipelines are actually capacity deficient (approximately 96 pipelines).The major areas of concern are in the trunk sewers in 15th Street and Dinuba Avenue, in S Columbia and Shoemake Avenues, W Manning Avenue, and in the pipeline from S Kingswood Parkway to the Reed Avenue Lift Station. In addition, the pipelines upstream of the WWTP from the Reed Avenue Lift Station and the Olson Avenue Bridge crossing will also be capacity deficient under Phase 1 flow conditions. Given that these are the only pipelines conveying wastewater to the WWTP, they will create a bottleneck and an upgrade will be required.

- **Figure 4-12. Phase 2 System PWWF d/D Deficiencies**: illustrates the deficiencies in the system under Phase 2 flow conditions following the upgrades recommended to address the deficiencies for the existing and Phase 1 conditions. Despite those upgrades, 93 pipelines were found to exceed capacity under Phase 2 PWW flow conditions.

  This figure also illustrates the proposed new trunk mains to serve the new areas beyond the City's current service area. As shown, much of the area to on the East side of the City is proposed to connect to a new trunk main in Zumwalt Avenue that would convey flow southward to Lilac Avenue. This was selected in order to minimize the impact on the existing system.

- **Figure 4-13. Phase 2 System PWWF q/Q Deficiencies**: illustrates that under Phase 2 conditions, Reed Avenue has capacity limitations due to the new areas in the northwest area of the City that will connect to the existing system. In addition, there are some capacity bottlenecks in E Manning, Columbia, and Shoemake Avenues.

Figure 4-8. Existing Sewer System PWWF d/D Deficiencies

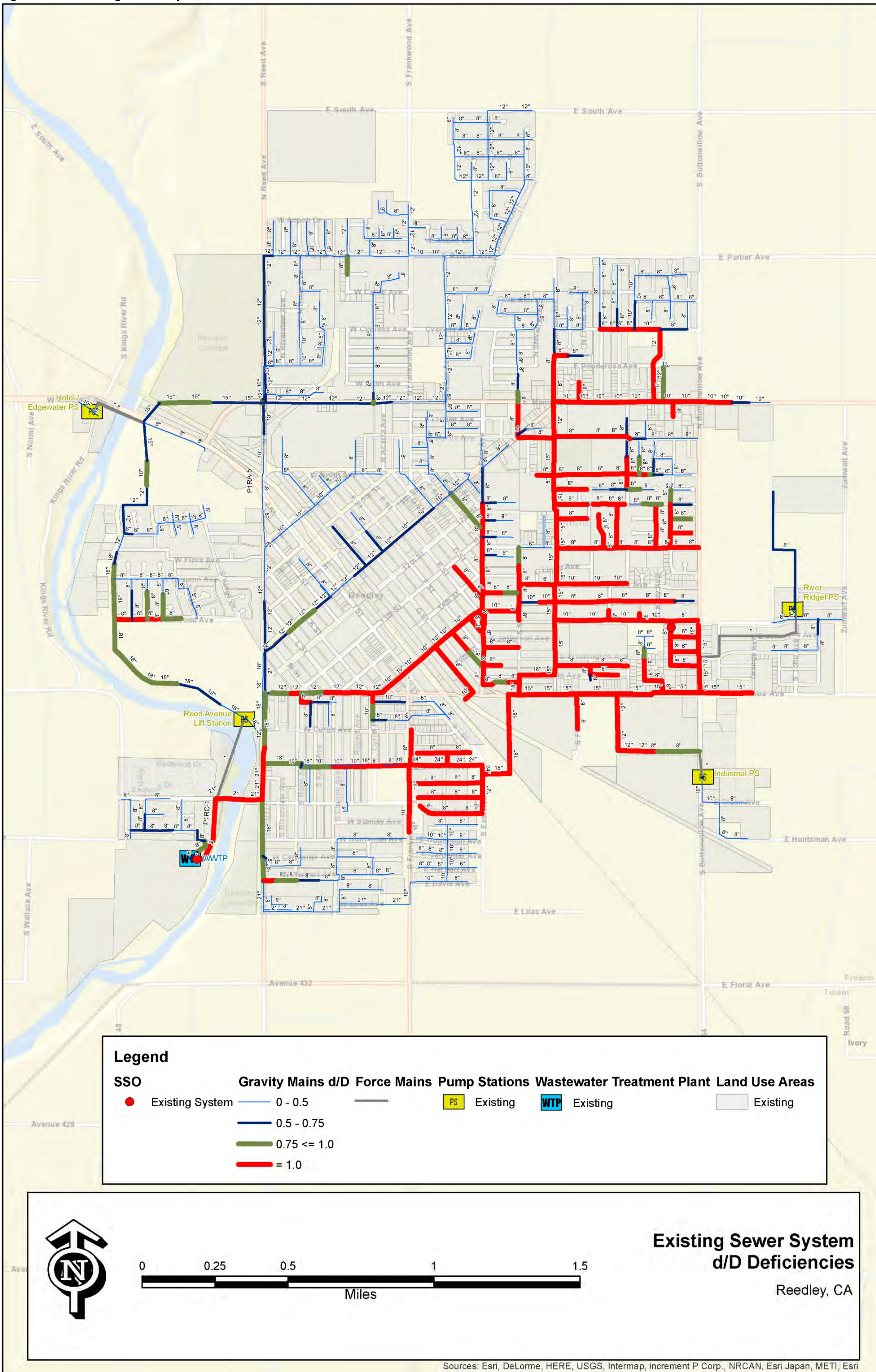


Figure 4-9. Existing Sewer System PWWF q/Q Deficiencies

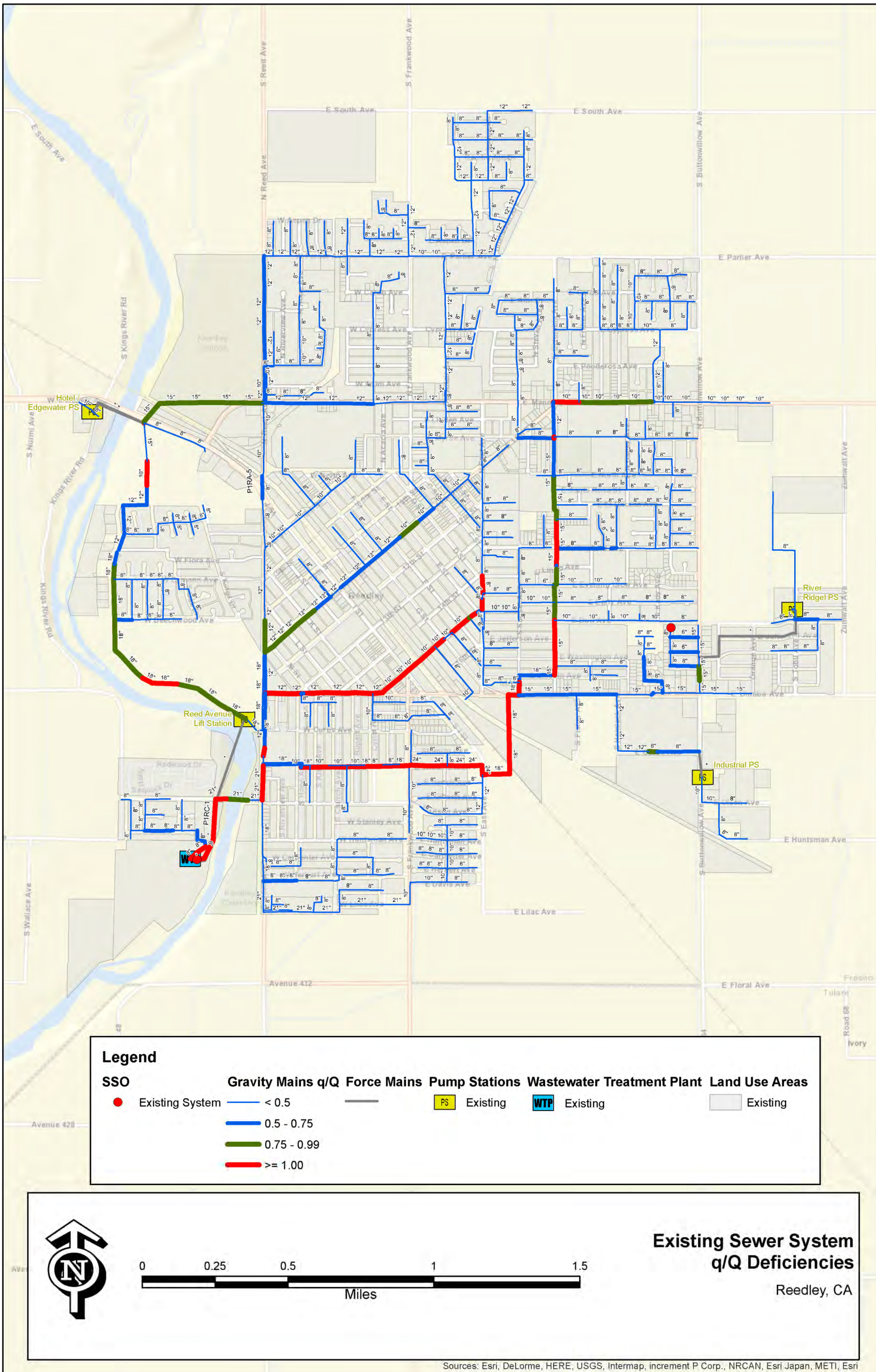


Figure 4-10: Phase 1 System PWWF d/D Deficiencies

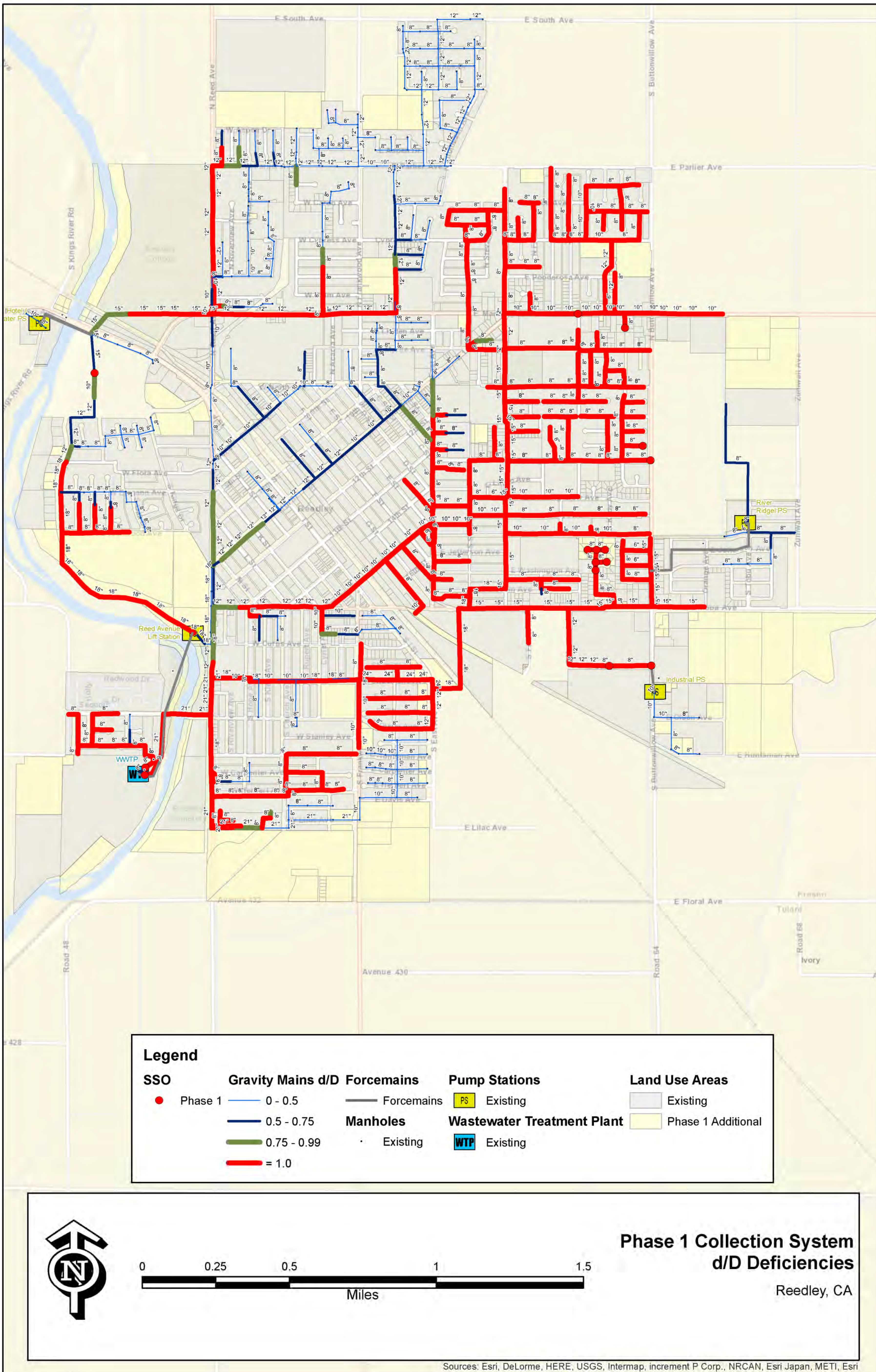


Figure 4-11: Phase 1 System PWWF q/Q Deficiencies

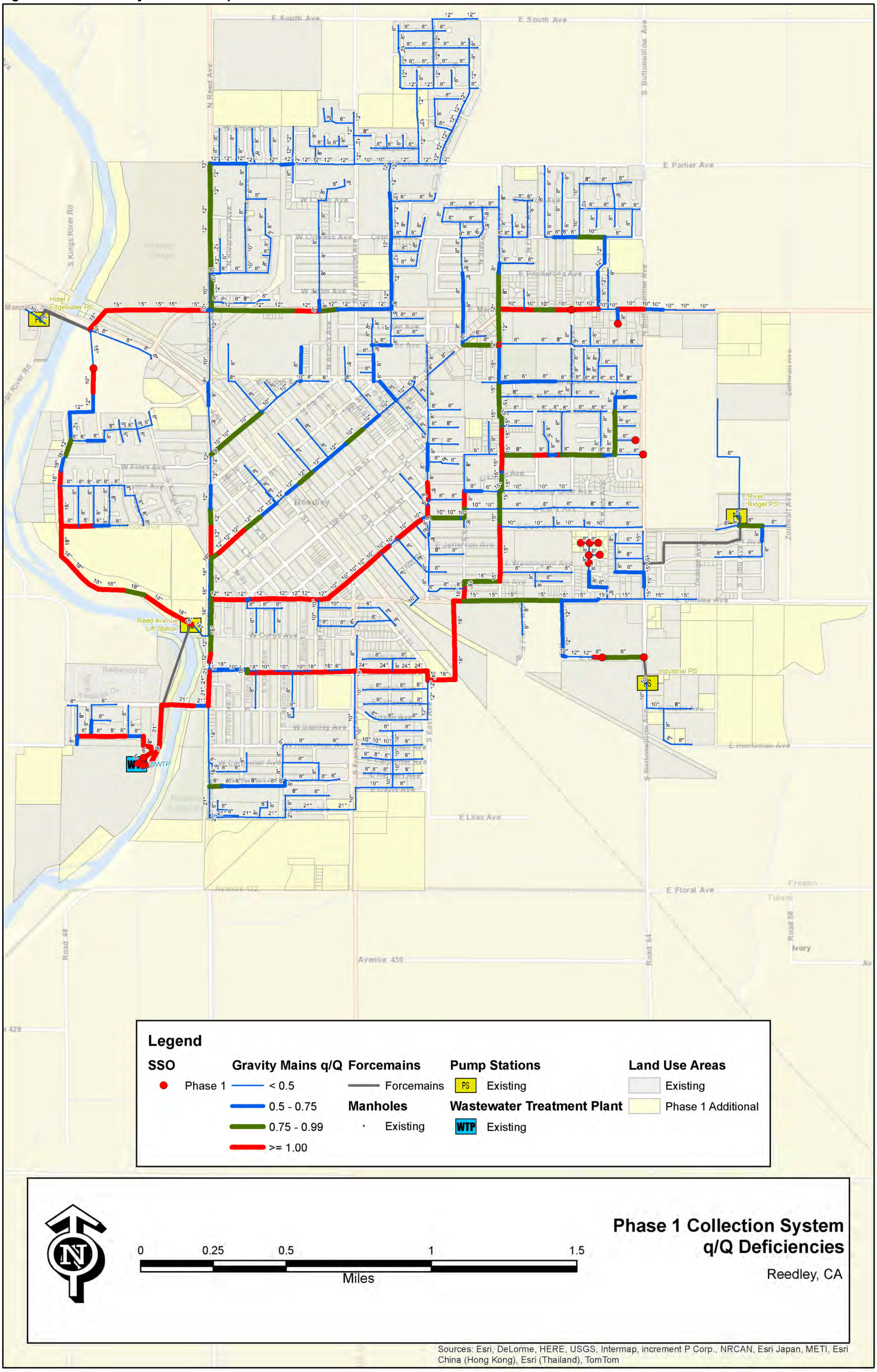


Figure 4-12. Phase 2 System PWWF d/D Deficiencies

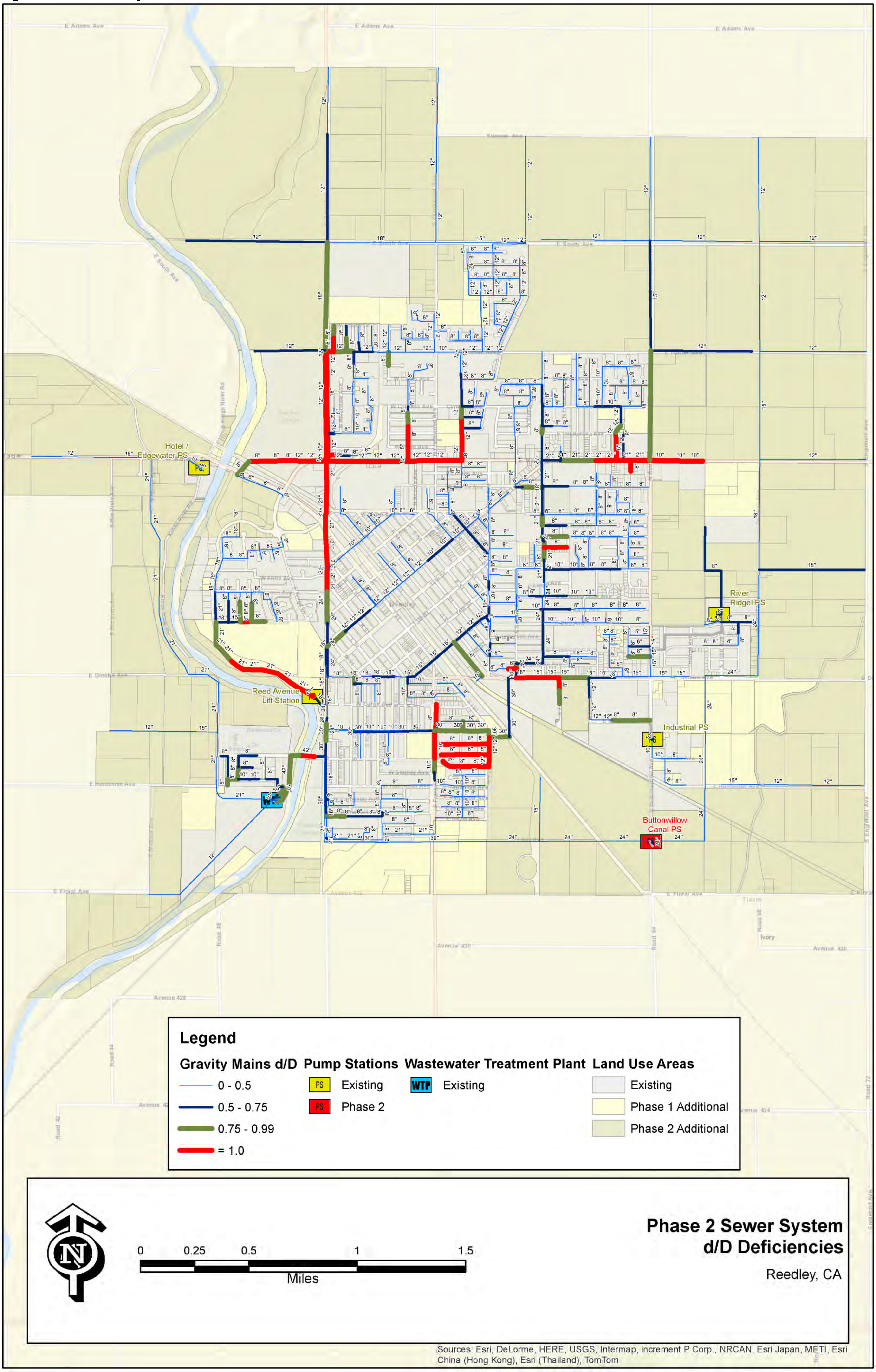
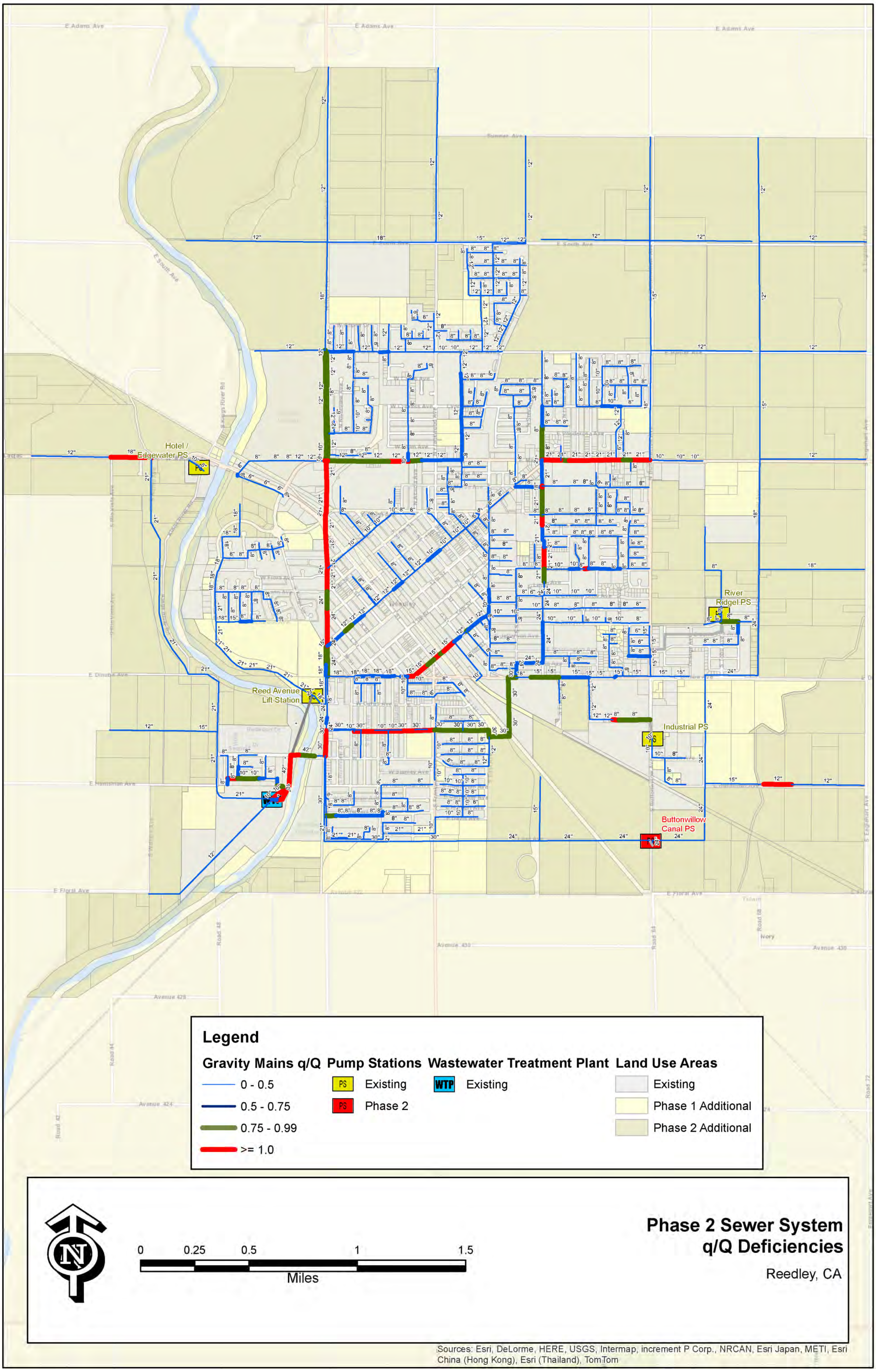


Figure 4-13. Phase 2 System PWWF q/Q Deficiencies



4.4.4. Proposed Improvements

Recommended improvements to the existing system were developed using the following guiding principles and considerations:

- Upsize sanitary sewer system pipelines in the existing system to address the existing and Phase 1 capacity issues while achieving the design criteria (d/D >= 0.75),
- In a second round of upgrades, upsize pipelines in the existing system to address the Phase 2 capacity issues while achieving the design criteria (d/D >= 0.75). This second round of upgrades for Phase 2 was provided because the additional capacity associated with the larger, build out flows may not be needed for many years, and the cost of the additional capacity may not be recoverable for a long time.
- Size new pipelines for the design criteria (d/D >= 0.75).
- Where possible, route new flows, particularly for Phase 2, around the existing system to avoid significant capacity issues.
- Avoid the creation of new lift stations where possible to minimize associated O&M labor and power requirements. As a result of these criteria, the new pipeline network for Phase 2 was oriented to avoid the Alta Irrigation District (AID) canal system, where possible.
- Where new lift stations are required, provide a standby pump such that the lift station can pump the design flow with the largest pump out of service.

Using these considerations to address the deficiencies described in the previous subsection, recommended improvements were developed. These recommended improvements are illustrated in Figure 4-14 and Figure 4-15, and summarized in Table 4-10 and Table 4-11.

As shown in Figure 4-14, the recommended improvements to resolve the existing and Phase 1 deficiencies have been combined. However, the higher priority upgrades are prioritized for earlier implementation, whereas the others can be implemented later in Phase 1. Approximately 26,000 linear feet of pipeline are recommended to improve the existing system and an additional 11,000 linear feet are needed to accommodate the additional flows from Phase 1. These improvements are largely restricted to existing trunk mains to improve the system capacity and enable gravity flow to the City's WWTP without backwater effects, surcharging, and SSOs. Of particular interest are the recommended improvements in the Reed Avenue trunk main and the Kings River Crossing.

Multiple alternatives were considered to convey flow from the north down to the WWTP, particularly considering the future growth anticipated in Phase 2. One option that was considered was pumping new flows westward across the Kings River at the Manning Avenue Bridge and then conveying them southward in a new trunk line on the west side of the river. However, this option was eliminated from further consideration because the timing of the Phase 2 development on the west side of the river is uncertain and thus the City did not want to invest in the new trunk line there until it would be needed. As a result, upgrades to the Reed Avenue pipeline were carefully considered. A significant stretch of the existing trunk main in Reed Avenue, the portion from Manning Avenue southward to 11th Street, is deep, with depths up to 12 feet below the ground surface. This depth makes typical open cut construction more difficult. In addition to the depth, the slope of the alignment in the lower segments is relatively flat, resulting in backwater effects and, under Phase 2 flow conditions, significant surcharging in the manholes. As a result of the existing conditions in the Reed Avenue trunk line, realignment is needed to adjust the slope. In addition, the pipe diameters need to be increased as well.

With the additional flow being routed down Reed Avenue, the diversion structure at the intersection of Reed Avenue and 11th Street must be adjusted such that the additional flow is routed away from the existing Reed Avenue Lift Station to avoid a significant capacity upgrade for that facility. Instead, the flow should be routed to the Kings River Crossing at the Olson Avenue Bridge, as the capacity of this crossing will need to be increased due to the new flows from the east. In this manner, only one of the river crossings will need an upgrade. While this additional capacity in the existing Olson Avenue crossing could be provided by upsizing the existing pipeline, it may be desirable to add a new crossing (e.g., a second pipeline across the bridge or a new pipeline under the river) to provide redundancy in the event one of the pipelines needs to be taken out of service for maintenance.

Figure 4-15 illustrates the recommended improvements for Phase 2. These recommendations include upgrades to the existing system trunk mains in Reed Avenue, Manning Avenue and Columbia Avenue, as well as expansion beyond the existing system to serve the Phase 2 growth. As shown, trunk sewers, ranging from 12- to 30-inches, are planned for arterial roads. The sanitary sewer systems feeding into these trunk sewers would be constructed as part of future developments and are therefore not included. In total, approximately 120,000 linear feet of pipeline are recommended to serve the build out flows in Phase 2.

On the east side of the City, wastewater will be collected in Zumwalt Avenue and conveyed southward to Lilac Avenue and westward toward Reed Avenue where it will be conveyed north to the Kings River Crossing at the Olson Avenue Bridge. A new lift station is required to serve this new alignment where it crosses the existing AID canal near Buttonwillow Avenue. A gravity flow option was also considered for the canal crossing; however, that would have resulted in a significant drop in the HGL and a lift station would have been required downstream to cross the Kings River or to enter the WWTP.

On the west side of the Kings River, once development starts there and the new trunk main is constructed near Kings River Road, flows from the existing Hotel/Edgewater Lift Station can be redirected, such that the existing river crossing can be eliminated.

For the Existing system, priority for improvements should be given to downstream bottlenecks that result in backwater effects in the upstream pipelines. For these areas, improvements should be prioritized from downstream to upstream.

Table 4-10. Recommended Lift Station Upgrades

Phase	Project ID	Lift Station	Capacity	Description
Existing	P1LS-1	Reed Avenue	2.4 MGD	Add an additional 550 gpm pump for reliability
Phase 2	P2LS-2	Buttonwillow Canal	1.5 MGD	New lift station at Lilac Avenue and the Buttonwillow Canal

Figure 4-14. Existing and Phase 1 Sewer System Upgrades

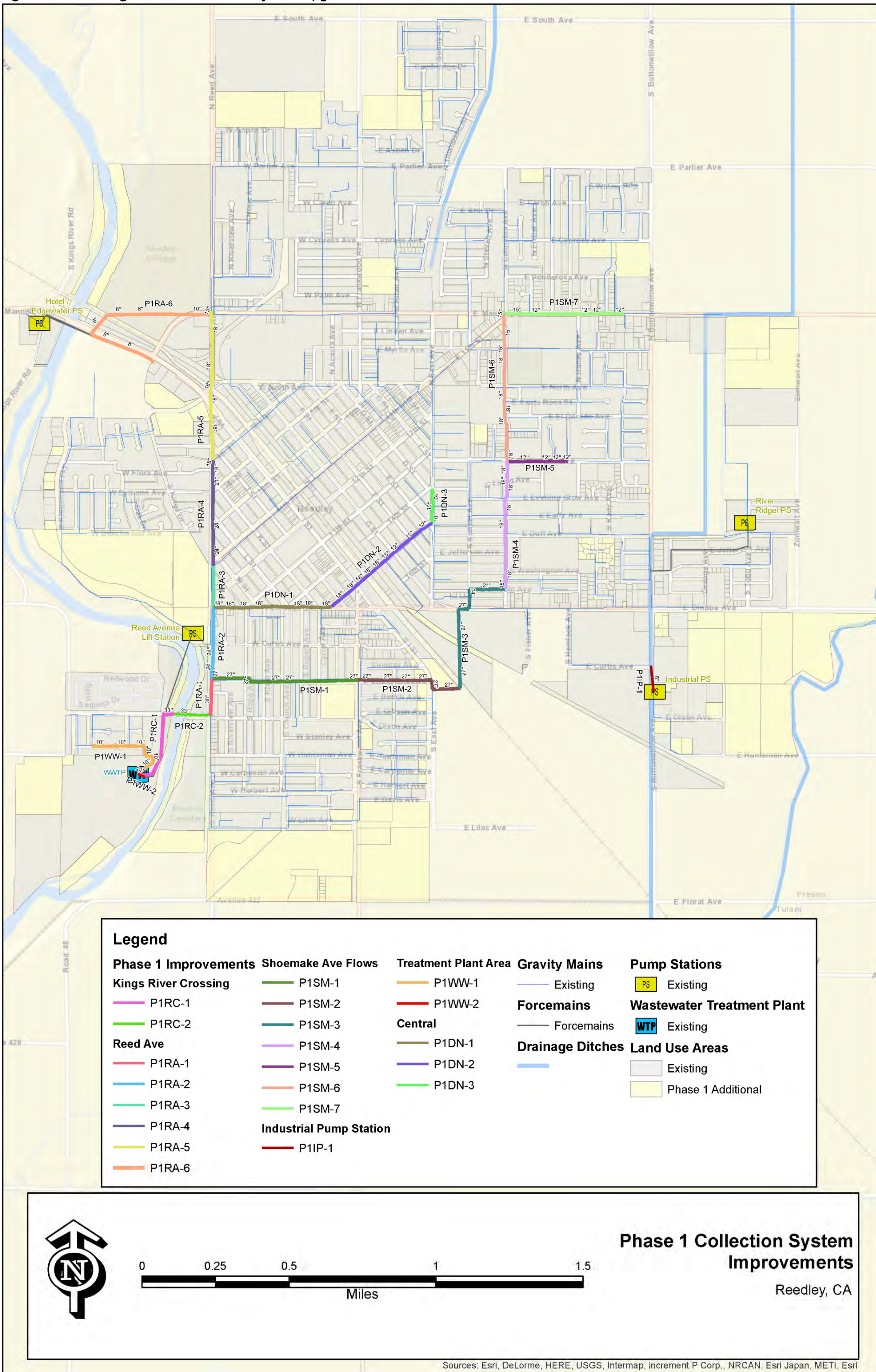


Figure 4-15. Phase 2 Sewer System Upgrades

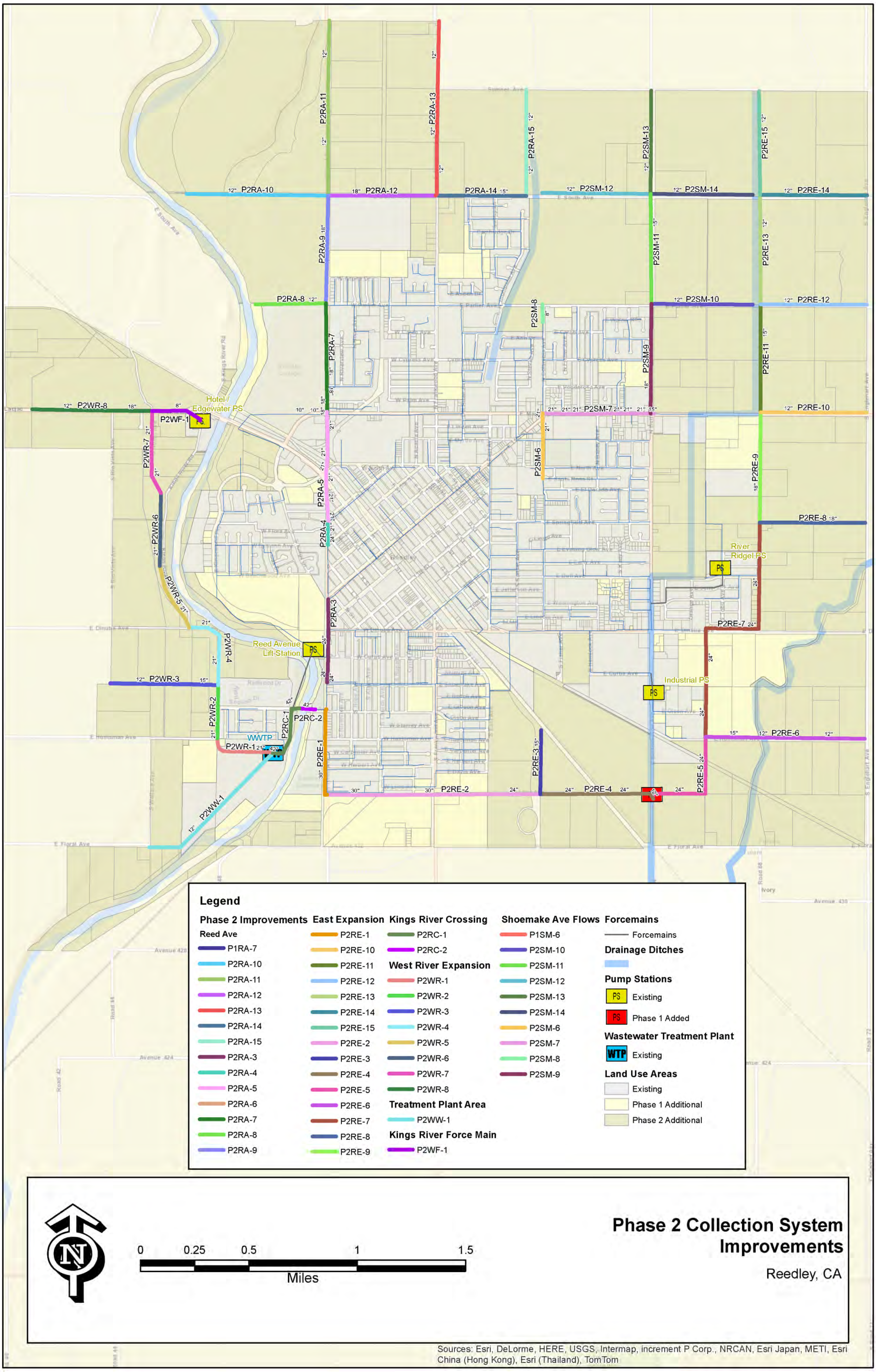


Table 4-11. Sewer Pipeline Improvements

Phase	Project ID	Location	Pipe Diameter (in)	Length (ft)	Description
Existing	P1DN-1	W Dinuba Ave between Reed Ave & 15th St	18	2,130	Upsized from 12" to 18"
Existing	P1DN-2	15th St between W Dinuba Ave & S East Ave	12 - 18	2,360	Upsized from 10" to various
Existing	P1DN-3	15th St between W Dinuba Ave & S East Ave	10	610	Upsized from 8" to 10"
Existing	P1IP-1	Industrial Pump Station Force main	8	540	Upsized from 6" to 8"
Existing	P1RA-1	Reed Ave between Olson Ave & W Shoemake Ave	30	660	Upsized from 21" to 30"
Existing	P1RA-2	Reed Ave between Dinuba Ave & W Shoemake Ave	24	1,280	Upsized from 12" to 24”
Existing	P1RC-1	South of Olson Ave at WWTP	33	1,570	Upsized from 12"/21" to 33”
Existing	P1RC-2	W Olson Ave - Kings River Crossing	33	660	Upsized from 21" to 33”
Existing	P1SM-1	Shoemake Ave between Reed Ave & S Frankwood Ave	27	2,720	Upsized from 18" to 27"
Existing	P1SM-2	Shoemake Ave S Frankwood Ave & Railroad	27	1,990	Upsized from 18"/24" to 27"
Existing	P1SM-3	Columbia Ave between Dinuba Ave & Springfield Ave & South of Dinuba Ave	21 - 27	2,630	Upsized from 18" to various
Existing	P1SM-4	Columbia Ave between Dinuba Ave & Springfield Ave	18	2,320	Upsized from 15" to 18"
Existing	P1SM-5	E Springfield Ave east of N Haney Ave	12	1,090	Upsized from 8" to 12"
Existing	P1SM-6	Columbia Ave btn E Manning Ave & E Springfield Ave	15 - 18	2,680	Upsized from 12"/15" to various
Existing	P1SM-7	E Manning Ave between E 11th St & Buttonwillow Ave	12 - 15	2,160	Upsized from 10" to various
Existing	P1WW-2	WWTP Headworks	24	150	Upsized from 21" to 24"
Phase 1	P1RA-3	Reed Ave between Dinuba Ave & 11th St	24	750	Upsized from 12" to 24"
Phase 1	P1RA-4	Reed Ave between 11th St & 8th St	18 - 24	1,900	Upsized from 12" to various
Phase 1	P1RA-5	Reed Ave between W Manning Ave & 8th St	18	2,630	Upsized from 8"/10" to 18"
Phase 1	P1RA-6	Manning Ave between Reed Ave and Upper Bridge Ave	8 - 10	3,600	Profile Reversed and upgraded
Phase 1	P1WW-1	W Henley Creek Rd	10	1,730	Upsized from 8" to 10"
Phase 2	P2RA-4	Reed Ave between 11th St & 8th St	21 - 24	570	Upsized from 18" to various
Phase 2	P2RA-5	Reed Ave between W Manning Ave & 8th St	21	2,630	Upsized from 18" to 21"
Phase 2	P2RA-7	Reed Ave between Parlier Ave and Manning Ave	18	2,710	Upsized from 10" to 18" & P2 Expansion
Phase 2	P2RA-8	Parlier Ave West of Reed Ave	12	1,720	Phase 2 Expansion
Phase 2	P2RA-9	Reed Ave between Manning Ave and South Ave	18	2,640	Phase 2 Expansion

Phase	Project ID	Location	Pipe Diameter (in)	Length (ft)	Description
Phase 2	P2RA-10	South Ave West of Reed Ave	12	3,440	Phase 2 Expansion
Phase 2	P2RA-11	Reed Ave between Adams Ave and South Ave	12	4,270	Phase 2 Expansion
Phase 2	P2RA-12	South Ave between Reed Ave and Frankwood Ave	18	2,630	Phase 2 Expansion
Phase 2	P2RA-13	Frankwood Ave between South Ave and Adams Ave	12	4,270	Phase 2 Expansion
Phase 2	P2RA-14	South Ave between Frankwood Ave and East Reedley Ditch	15	2,210	Phase 2 Expansion
Phase 2	P2RA-15	West Bank of East Reedley Ditch at South Ave	12	2,600	Phase 2 Expansion
Phase 2	P2RC-1	South of Olson Ave at WWTP	42	1,570	Upsized from 30" to 42"
Phase 2	P2RC-2	W Olson Ave - Kings River Crossing	42	390	Upsized from 30" to 42"
Phase 2	P2RE-1	Reed Ave between Olson Ave and Lilac Ave	30	2,150	Phase 2 Expansion
Phase 2	P2RE-2	Lilac Ave between Reed Ave and Columbia Ave	24 - 30	5,200	Phase 2 Expansion
Phase 2	P2RE-3	Columbia Ave between Railroad and Lilac Ave	15	1,560	Phase 2 Expansion
Phase 2	P2RE-4	Lilac Ave West of Buttonwillow Canal	12 - 24	2,830	Phase 2 Expansion with new force main
Phase 2	P2RE-5	South of Buttonwillow Canal and South of Huntsman Ave	24	2,710	Phase 2 Expansion
Phase 2	P2RE-6	Huntsman Ave West of Englehart Ave	12 - 15	3,880	Phase 2 Expansion
Phase 2	P2RE-7	West of Travers Canal btn Springfield and Huntsman Ave	24	6,530	Phase 2 Expansion
Phase 2	P2RE-8	Springfield Ave between Zumwalt Ave and Englehart Ave	18	2,570	Phase 2 Expansion
Phase 2	P2RE-9	Zumwalt Ave between Manning Ave and Springfield Ave	18	2,700	Phase 2 Expansion
Phase 2	P2RE-10	Manning Ave between Zumwalt Ave and Englehart Ave	12	2,590	Phase 2 Expansion
Phase 2	P2RE-11	Zumwalt Ave between Parlier Ave and Manning Ave	15	2,630	Phase 2 Expansion
Phase 2	P2RE-12	Parlier Ave between Zumwalt Ave and Englehart Ave	12	2,590	Phase 2 Expansion
Phase 2	P2RE-13	Zumwalt Ave between South Ave and Parlier Ave	12	2,680	Phase 2 Expansion
Phase 2	P2RE-14	South Ave between Zumwalt Ave and Englehart Ave	12	2,620	Phase 2 Expansion
Phase 2	P2RE-15	Zumwalt Ave between Sumner Ave and South Ave	12	2,530	Phase 2 Expansion
Phase 2	P2SM-6	Columbia Ave btn E Manning Ave & E Springfield Ave	21	1,560	Upsized from 18" to 21"
Phase 2	P2SM-7	Manning Ave between Columbia Ave and Buttonwillow Ave	21	2,720	Upsized from 10"-18" to 21”
Phase 2	P2SM-8	Columbia Ave South of Parlier Ave	8	430	Phase 2 Expansion
Phase 2	P2SM-9	Buttonwillow Ave between Parlier Ave and Manning Ave	18	2,680	Phase 2 Expansion

Phase	Project ID	Location	Pipe Diameter (in)	Length (ft)	Description
Phase 2	P2SM-10	Parlier Ave between Buttonwillow Ave and Zumwalt Ave	12	2,470	Phase 2 Expansion
Phase 2	P2SM-11	Buttonwillow Ave between South Ave and Parlier Ave	15	2,680	Phase 2 Expansion
Phase 2	P2SM-12	South Ave btn East Reedley Ditch and Buttonwillow Ave	12	2,650	Phase 2 Expansion
Phase 2	P2SM-13	Buttonwillow Ave between Sumner Ave and South Ave	12	2,530	Phase 2 Expansion
Phase 2	P2SM-14	South Ave between Buttonwillow Ave and Zumwalt Ave	12	2,490	Phase 2 Expansion
Phase 2	P2WF-1	Manning Ave between Kings River and Nurmi Ave	8	1,130	New 8" Force Main
Phase 2	P2WR-1	Between Huntsman Ave and Treatment Plant	21	1,590	Phase 2 Expansion
Phase 2	P2WR-2	Kings River Rd between Huntsman Ave and Redwood Dr	21	1,300	Phase 2 Expansion
Phase 2	P2WR-3	West of Kings River Rd parallel to Redwood Dr	12 - 15	2,630	Phase 2 Expansion
Phase 2	P2WR-4	Kings River Rd between Redwood Dr and Dinuba Ave	21	2,010	Phase 2 Expansion
Phase 2	P2WR-5	Kings River Rd North of Dinuba Ave	21	1,700	Phase 2 Expansion
Phase 2	P2WR-6	Kings River Rd South of Nurmi Ave	21	1,780	Phase 2 Expansion
Phase 2	P2WR-7	Nurmi Ave between Kings River Rd and Manning Ave	21	2,060	Phase 2 Expansion
Phase 2	P2WR-8	Manning Ave between Lac Jac Ave and Nurmi Ave	12 - 18	2,930	Phase 2 Expansion
Phase 2	P2WW-1	West Bank Kings River between Wallace Ave and WWTP	12	3,980	Phase 2 Expansion
			Subtotal Existing Upgrades	25,550	
			Subtotal Phase 1 Upgrades	10,610	
			Subtotal Phase 2 Upgrades	118,740	
			Total Recommended Improvements	154,900	

5. Storm Drainage System

This section of the Master Plan describes the City's storm drainage system. It includes an inventory of existing system components and provides information on current and projected future storm water flows. It then provides an analysis of the hydraulics of the storm drainage system under current and future conditions, as well as recommended upgrades to improve system deficiencies.

For information on the capital improvement program, see Section 6 of this Plan.

5.1. Inventory of Storm Drainage System

Basin delineation and existing storm drainage infrastructure is shown in Figure 5-1. The storm drainage system has multiple discharge points, including to the Kings River, Alta Irrigation District (AID) agricultural canal system and retention storage facilities. Major components of the City's storm drainage infrastructure include pipe networks, pump stations, retention and detention storage facilities.

The existing storm drainage system is divided into 17 sub-basins. The basin dimensions were determined using best available aerial imagery and GIS data. The existing system also includes 13 outfalls. Ten outfalls discharge directly to the Kings River and three discharge to an AID drainage canal. Information on each drainage basin, include the size, outfall location and total pipe length, is summarized in Table 5-1.

Figure 5-1. Storm Drainage System

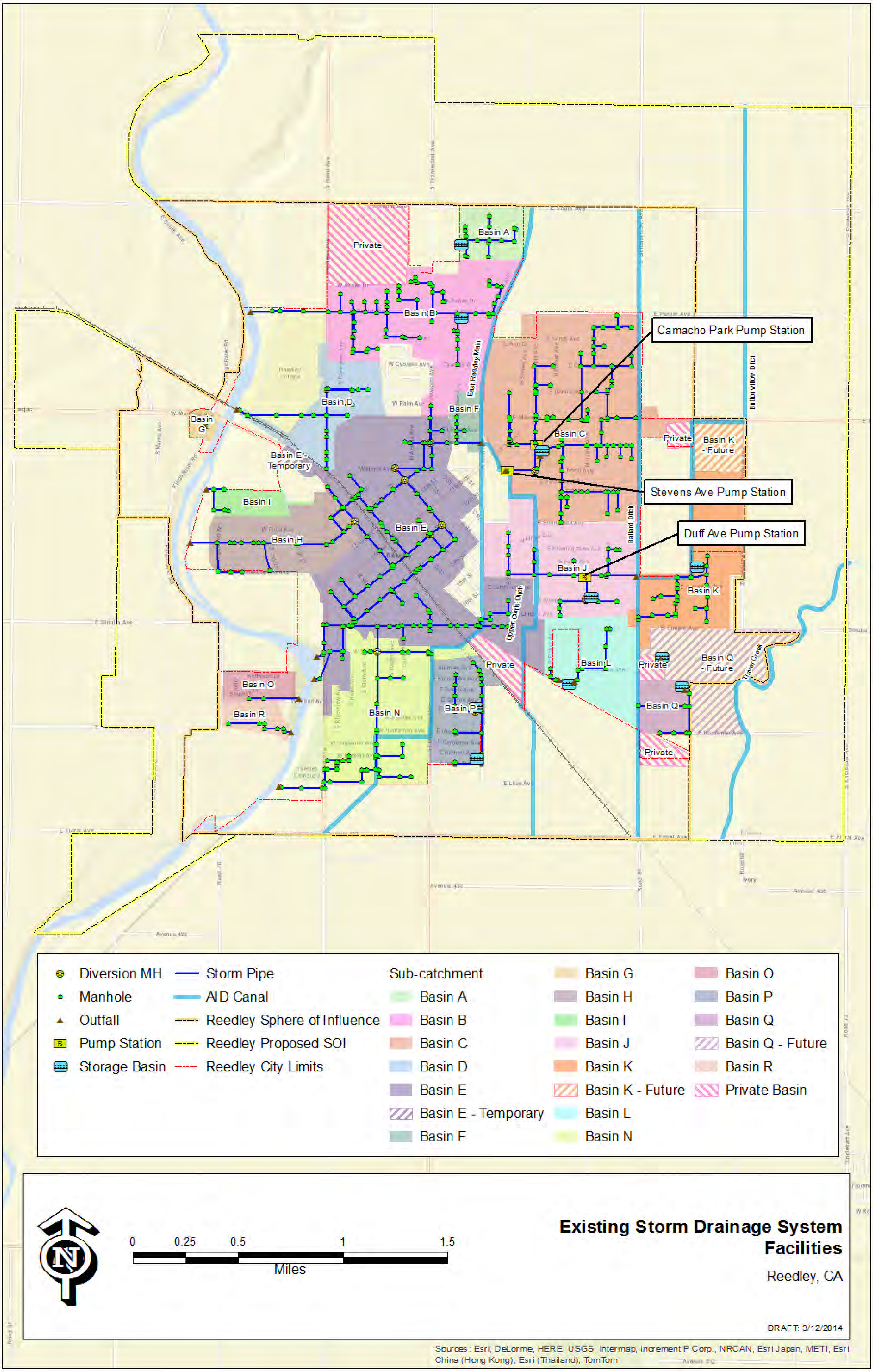


Table 5-1. Storm Drainage Basins

Drainage Basin[(a)]	Acres	Outfall Location	System Pipe Diameters (in)	Length of Pipe (ft)
A	53.4	Retention Basin	18, 21	3,585
B	246.9	Detention Basin Kings River	15,18, 24, 30, 36, 42	17,400
C	454.3	Detention Basin Upper Curtis Ditch	8, 12, 15, 18, 21, 24, 30, 36	27,050
D	91.4	Kings River	12, 15, 18, 24, 27	9,220
E	521.0	Kings River[(b)]	10,12, 14, 15, 16, 18, 20, 24, 30, 36	35,360
F	25.5	East Reedley Main Ditch	15, 18, 30	1,460
G	12.2	Kings River	15,24	280
H	129.5	Kings River	18, 24, 27, 30	7,225
I	28.5	Kings River	18	1,315
J	191.5	Detention Basin Ballard Ditch	15, 18, 21, 24, 31, 36	9,490
K	155.1 39.6[(c)]	Retention Basin	18, 24, 36	5,545
L	116.7	Retention Basin	18, 24	3,010
N	209.9	Kings River	8, 14, 15, 24, 30, 36	15,565
O	32.2	Kings River	18, 21	1,240
P	88.0	2 Retention Basins	18, 24, 30	5,895
Q	39.0 129.9[(c)]	Retention Basin	21, 24, 30	3,275
R	27.9	Kings River	24	1,765

(a) Drainage basin M is not listed. It is a private drainage basin that is not expected to connect to the City's drainage system.
(b) Two outfall locations.
(c) Additional future acreage for Basins K & Q. These basins will be expanded as the area is developed in the future.

There are two locations where flows can be diverted between two basins.

- Between Basin E and Basin H – Diversion located at 9th St and I St.
- Between Basin E and Basin N – Diversion located at Dinuba Ave. and Church Ave.

There are three existing pump stations: the Camacho Park Pumping Facility, Duff Avenue Pump Station, and Stevens Avenue Pump Station. Information on each pump station is summarized in Table 5-2.

Table 5-2. Pump Station Information

Pump Station	Drainage Basin	Number of Pumps	Design Flow per Pump (gpm)	Design Head (ft)
Camacho Park	C	1	700 [(a)] (1.5 cfs)	20
Stevens Ave.	C	1	700 [(a)] (1.5 cfs)	Unknown
Duff Ave.	J	3	2,45 (5.5 cfs)	43.6

(a) Data on pump not available. The value is estimated.

The City has ten storage facilities in the existing system. Seven of these storage facilities are retention basins that collect runoff water and rely on infiltration to dispose of stormwater. The remaining three storage facilities operate as detention basins, storing stormwater to reduce the magnitude of peak flows. As the rainfall peak passes, stored flows are released back into the storm drainage system to be discharged to Kings River or to AID's canal system. Information on the storage facilities is summarized in Table 5-3.

Table 5-3. Storm Drainage Storage Facilities

Drainage Basin	Discharge Location	Est. Available Facility Depth (ft)	Est. Available Facility Volume (ac-ft)	Est. Facility Bottom Elevation (ft)	Est. Facility Top Elevation
A	Retention Facility	11.0	12.2	348	359
B	Kings River	13.0	85.1	341	354
C	Alta Irrigation System	26.5	102.4	310	336
J	Alta Irrigation System	6.0	6.7	334	341
K	Retention Facility	20.0	250.5	326	346
L	Retention Facility	14.0	16.4	326	340
P[(a)]	Temporary Retention Facility	14.0	13.4	333	347
P[(a)]	Temporary Retention Facility	14.0	5.2	332	346
Q	Retention Facility	14.0	18.0	323	337
Q-Future[(b)]	Temporary Retention Facility	6.0	0.8	334	341

(a) There are two temporary storage facilities in Basin P. These storage facilities will be replaced with a permanent facility to be constructed downstream.

(b) Temporary facility located at Heritage Mini Storage Site. This area will be connected to existing storage facility in Basin Q in the future.

5.2. Hydrologic Conditions

This section describes the relevant soil, land cover and rainfall information needed to characterize the drainage basins, and determine the parameters and values to be used to model and analyze the storm drainage system. The analysis method (see Section 5.4) relies on a hydrologic model that simulates the response of the City's storm drainage system to rainfall and runoff. Runoff from a rainfall event is a function of the amount and intensity of precipitation, together with infiltration and other losses that reduce the amount of runoff reaching the storm drainage system.

Each major drainage basin is divided into multiple sub-catchments to facilitate the estimation of runoff and allocation of flows into the storm drainage system. Section 5.4.2 describes the analytical methods used to determine the runoff patterns. The hydrologic model considers the following sub-catchment characteristics in determining response to rainfall events:

- Sub-catchment area
- Percent impervious area
- Infiltration parameters
- Runoff coefficients
- Average sub-catchment gradient
- Width of sub-catchment

The topography generally defines each sub-catchment's area, gradient and width; while land use and soil types generally define the percent impervious area, infiltration parameters and runoff coefficients.

5.2.1. Existing Conditions

The City is located in the central San Joaquin Valley of California, lying just inland between the State's coastal mountain ranges and the Sierra Nevada mountain range. The City is located 22 miles southeast of Fresno with ground elevations ranging from 300 to 400 feet (NGVD 29). Topography within the City's service area is flat. The normal annual precipitation in this region is approximately 13 inches, with the majority of the precipitation occurring during the winter months. The area drained by the storm drainage system has approximately 2,590 acres.

The Natural Resources Conservation Service (NRCS) classifies soils into four groups for hydrologic analysis as summarized in Table 5-4. Within the Reedley area shallow soils are generally clay loams with slow infiltration rates and about two feet of hardpan that varies from two to four feet below the surface. In general, for the City's storm drainage service area the soils are consistent with soil Group C as presented in Table 5-4.

Table 5-4. NRCS Hydrologic Soil Group Definitions

Soil Group	Definition
A	Low runoff potential. Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures. E.g., shallow loess sandy loam.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures. E.g., clay loams, shallow sandy loam.
D	High runoff potential. Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay-pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Land Use

The major land use classifications within the City's system include residential, commercial, industrial and other (e.g., open space). The major subcategories are as follows:

- Residential
  - Low Density Residential
  - Suburban Residential
  - Medium Density Residential
  - High Density Residential
- Commercial
  - Central Downtown
  - Community Commercial
  - Neighborhood Commercial
  - Office
  - Public/Institutional Facility
  - Service Commercial

- Industrial
    - Light Industrial
    - Heavy Industrial
- Other
    - Open Space
    - Open Space (Park)

Percent imperviousness for each land use type is shown in Table 5-5. The percent impervious area for each sub-catchment is calculated based on a weighted average of the land use types contained within its boundaries.

Infiltration in the model is estimated using the NRCS Curve Number method. This method generates a peak runoff rate and a hydrograph for flow routing using a curve number to define the relationship between accumulated rainfall and accumulated runoff. This curve number represents a runoff factor indicating an area's runoff potential; a higher curve number corresponds with a higher runoff potential. Curve numbers vary for different land use types and the four NRCS soil groups. For the City, a soil of Group C was assumed because it is the predominant soil type. The curve number defined for each land use type is also shown in Table 5-5.

Runoff is determined using the U.S. Environmental Protection Agency (USEPA) Nonlinear Reservoir method in the hydraulic/hydrologic model. This method performs a water balance for each sub-catchment accounting for all inflows (precipitation and flow from upstream sub-catchments) and outflows (infiltration, evaporation and surface runoff) to and from the sub-catchment. Parameters that define surface runoff include the sub-catchment characteristics (area, percent impervious, width and sub-catchment slope) and Manning's *n* value for overland flow. The values used in the model are presented in Table 5-5.

As with determining the percent impervious for each sub-catchment, a weighted average of land use types contained within its boundary is used to determine the curve number and Manning's *n* value to be used. The weighted curve number is used to estimate the amount of infiltration and the weighted Manning's *n* value is used to estimate the runoff that results from a specific precipitation event.

Table 5-5. Percent Impervious and Curve Number by Land Use Category

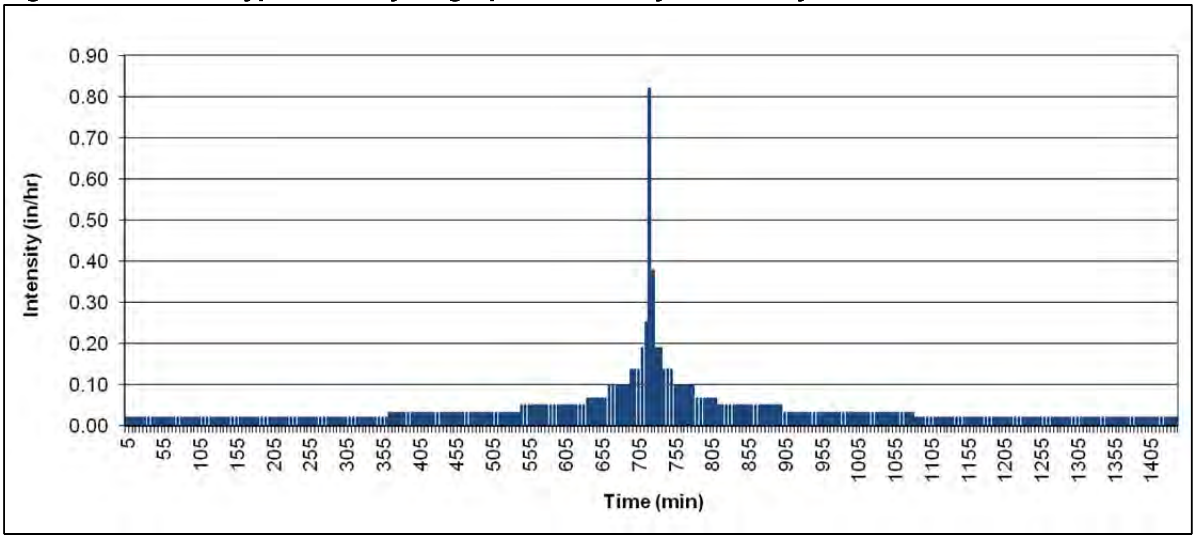
Land Use Category	Land Use Subcategories	Percent Impervious[a]	Curve Number[b]	Manning's *n* for Impervious Area[c]	Manning's *n* for Pervious Area[c]
Low Residential	Low Residential	30	80	0.013	0.30
Medium Residential	Medium Residential	70	81	0.013	0.20
High Residential	High Residential	80	90	0.013	0.20
Suburban Residential	Suburban Residential	15	79	0.013	0.40
Commercial	Central Downtown Community Commercial Neighborhood Commercial Office Public/Institutional Facility Service Commercial	90	94	0.013	0.11
Industrial	Heavy Industrial Light Industrial	85	91	0.013	0.11
Other	Open Space Open Space (Park)	2	79	0.013	0.40

(a) Based on *City and County of Sacramento Drainage Manual (Table 7-1).*
(b) Assumes Type C soils (soils with slow infiltration rates when thoroughly wetted (e.g., clay loams, shallow sandy loam)). Reference (included Appendix J): *SCS Urban Hydrology for Small Watersheds, 2nd Ed., (TR-55), June 1986.*
(c) Based on *McCuen, R. et. al. (1996), Hydrology, FHWA-SA-96-067, Federal Highway Administration, Washington DC.*

Rainfall

As noted above, the normal annual precipitation in this region is approximately 13 inches, with the majority of the precipitation occurring during the winter months. Rainfall hyetographs (charts showing rainfall distribution) from Sacramento City/County Drainage Manual "Sac Calc" program (2005) were modified to represent the equivalent rainfall hyetograph for the City. For analysis purposes, the NRCS Type I rainfall unit hydrograph is used to represent the rainfall pattern (shown in Figure 5-2).

Figure 5-2. NRCS Type I Unit Hydrograph for the City of Reedley



5.2.2. Projected Future Conditions

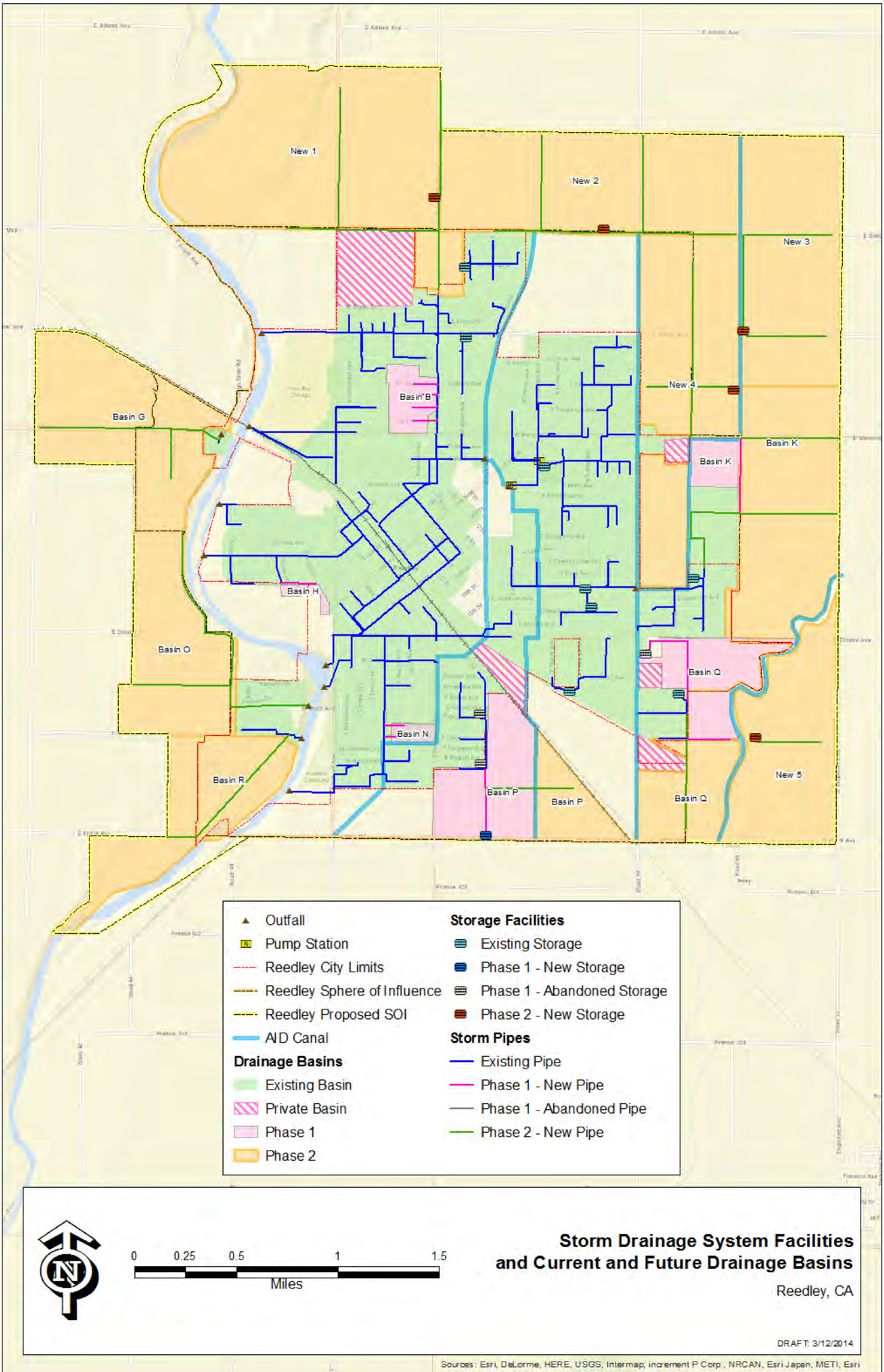
Section 5.2.1 described the land use, soil and rainfall for current conditions. For purposes of the storm drainage system analysis, future conditions assume the storm drainage system needs to serve two phases of expansion as shown in Figure 5-3:

- Phase 1 – corresponds approximately to the year 2020, and includes approximately 550 acres of additional drainage area (21 percent increase over the existing area). In addition, connections to existing storm inlets are anticipated within Basins B, H and N. These inlets are currently tied to the AID irrigation system

- Phase 2 – corresponds to build-out conditions based on the City's General Plan 2030, and includes approximately 3,550 acres of additional drainage area (137 percent increase over the existing area).

Future conditions assume that soil types and level of imperviousness within the existing sub-catchment areas do not change significantly despite infill and other land use changes. Based on input from the City, the understanding is that few infill areas within the existing City Limits will require connection to existing storm drain pipelines.

Figure 5-3. Storm Drainage Facilities and Current and Future Drainage Basins



Furthermore, the storm drainage system serving the Phase 1 and Phase 2 expansion areas are assumed to be independent, i.e. not connected to the existing storm drainage system. This was assumed because preliminary assessment indicated that the existing storm drainage system is already significantly under-sized and has capacity limitations. New storm drain piping for new areas that topographically drain to existing sub-catchments were laid out such that they drain to retention facilities rather than to the existing downstream system. Additional retention facilities were included at the downstream end of the Phase 1 and Phase 2 sub-catchments to keep runoff from those areas from flowing into the existing system.

5.3. Regulatory Requirements

The federal Clean Water Act includes provisions for National Pollutant Discharge Elimination System (NPDES) permits. This includes requirements for Municipal Separate Storm Sewer System (MS4) providers through a two-phase permit process. The State of California administers this program through the SWRCB. The Phase I permit applies to large- and medium-sized municipalities with populations greater than 100,000. The Phase II Permit is applicable to smaller jurisdictions with populations of greater than 10,000 and for certain census-defined urban areas. The City has a population of approximately 24,000 and is listed by the State as a Phase II community.

The SWRCB adopted a General Permit for the Discharge of Storm Water from Small MS4s in WQ Order No. 2003-0005-DWQ. The General Permit requires each regulated MS4 to develop and implement a Storm Water Management Plan/Program (SWMP) with the goal of reducing the discharge of pollutants to the maximum extent practicable. The SWMP specifies Best Management Practices to be applied in various program areas, including:

- Public education and outreach;
- Illicit discharge detection and elimination;
- Construction site stormwater runoff control;
- Post-construction stormwater management for new development and redevelopment;
- Pollution prevention and good housekeeping for municipal operations.

The City issued the *City of Reedley Storm Water Management Implementation Plan* in 2007 to comply with the General Permit requirements. The SWRCB is undertaking a revision to the

Phase II General Permit, and this may change the specific requirements affecting the City and its SWMP.

Other regulatory requirements that may affect the City's storm water management system and program include, but are not limited to:

- Clean Water Act, Section 303(d) List and Total Maximum Daily Load (TMDL) Program, affecting limits on discharges of certain pollutants;
- Clean Water Act Sections 10 and 404 Permits regulating placement of fill in waters of the United States, including some wetland areas;
- Endangered Species Act, with listings of certain species as threatened or endangered and associated prohibitions on actions that could harm certain species and their habitats.
- National Flood Insurance Program, affecting land use and zoning and associated local regulations in mapped flood hazard areas.

City staff should remain aware of these regulatory programs and consider how they relate to continued development and operation of the storm drainage system.

5.4. Storm Drainage System Evaluation

A preliminary assessment of the City's storm drainage system showed that the existing system was under-sized relative to the City's historic design criteria. Therefore, the approach for the storm drainage system analysis was to define a "level of service" (LOS) for the City's existing storm drainage system that is different than the design criteria used to develop recommended system improvements. This approach allows the City to cost-effectively improve the overall performance (and level of service) of the system over time without undertaking prohibitively expensive projects to upgrade systems in older neighborhoods that were not designed to meet today's standards.

Capacity of the storm drainage system was evaluated using Innovyze's InfoSWMM hydraulic model (version 12.0, SP1). The existing system in the model includes the planned Frankwood Avenue Reconstruction Project.

The following subsections describe the planning criteria used to perform the storm drainage system evaluation, the modeling analysis, identification of system deficiencies, and recommended system improvements.

5.4.1. Planning Criteria

As noted above, the existing system was evaluated using a level-of-service approach. The minimum level-of-service evaluation conditions adopted to evaluate the performance of the storm drainage system are:

- For the conveyance system:
  - 2-year, 24-hour storm for existing system
  - 10-year, 24-hour storm for Phase 1 expansion areas
  - 10-year, 24-hour storm for Phase 2 expansion areas
- For the storage facilities:
  - 100-year, 10-day storm for retention facilities
  - 100-year, 2-day storm for detention facilities

Shorter duration events were considered, but the 24-hour duration is consistent with Fresno Metropolitan Flood Control District (FMFCD) criteria, which is commonly used in Fresno County.

In addition, the following criteria were used to evaluate the capacity of the existing infrastructure:

- For storm pipes, velocities shall be greater than 2 feet per second (fps) and less than 10 fps for the calculated peak flow. This velocity range minimizes the deposition of solids in the pipelines, and minimizes the effects of abrasion and turbulence on the pipe and at pipe joints. The maximum hydraulic grade line (HGL) will be 1-feet below ground surface for existing pipes. New pipes will be designed to flow full at peak flow.

- Storage facilities shall maintain at least 1.5 feet of freeboard under design storm conditions.

- Storage facility infiltration will be considered for multi-day rainfall events. Storage facility infiltration shall be based on an assumed Group C soil type with an estimated infiltration rate of 0.75 inch per day (*Source: 1982 Storm Drainage Master Planning Report*).

Since pipe material information was not available, all gravity pipelines were assumed to have a Manning's friction coefficient of 0.013. This value is consistent with the most common types of pipe material used for storm drainage construction (e.g., reinforced concrete pipe, cast-in-place concrete pipe, and asbestos cement pipe). For the forcemain pipelines associated with each pump station, a Hazen-Williams friction coefficient, C-factor, of 120 was assumed. This value corresponds with a typical forcemain pipe material of ductile iron.

Table 5-6 summarizes the design criteria used to evaluate the storm drainage system.

Table 5-6. Storm Drainage Planning Criteria Summary

Design Element	Evaluation Conditions and Performance Criteria
Hydrologic Criteria	
Rainfall events used to establish Level of Service and evaluate the collection system	2-, and 10-year event of 24-hour duration
Rainfall event used to evaluate retention facilities	100-year; 10-day storm
Rainfall event used to evaluate detention facilities	100-year; 2-day storm
Rainfall Pattern	SCS Type I unit hydrograph
Rainfall	Hyetographs from the County of Sacramento will be adjusted proportionally to obtain rainfall for the City of Reedley
Hydraulic Criteria	
Pipeline friction coefficients	Manning's roughness coefficient of 0.013 for all gravity pipes. Hazen-Williams C-factor of 120 for forcemains.
Velocity	Minimum of 2 feet per second (fps); Maximum of 10 fps at peak flow
Depth of flow	Hydraulic grade line (HGL) is 1-foot below ground surface for existing pipes; Pipe full at peak flow for new pipes.
Storage facility freeboard	1.5-foot minimum freeboard to top of facility
Storage facility infiltration	Rate assumed to be 0.75 inches per day

5.4.2. Hydraulic Modeling Analysis

Evaluation of the City's storm drainage system was conducted using Innovyze's InfoSWMM (version 12.0, SP1). InfoSWMM is a fully dynamic, geospatial wastewater and stormwater modeling and management software application. It can be used to model the entire land phase of the hydrologic cycle and applied to urban stormwater and wastewater collection systems. The model can perform single event or long-term (continuous) rainfall-runoff simulations, accounting fro climate, soil, land use, and topographic conditions of the watershed.

Model Development

The InfoSWMM model of the storm drainage system was generated using data obtained from a CAD map of the system. Invert elevations – the lowest elevation of a pipe or other hydraulic structure at a given location – were provided on the map. Where invert elevations were not known, they were inferred based on the slope of the nearest upstream or downstream pipeline. Ground elevations were estimated using a California digital elevation (DEM) model hosted by ESRI. The DEM contains elevation in meters.

Information on the pumps and storage facility geometry was provided by the City through as-built documentation or other available reports.

The resulting model is intended only for planning-level analysis. If a more detailed analysis is necessary it is recommended that verification of information included in the model should be performed. In particular, the inferred invert elevations should be verified.

Model Validation

The storm drainage system model was validated using two rain events that occurred in 2011:

- City flooding on March 20, 2011 – represents a drawn out event with a steady rain occurring over several hours. The rain total for this event was 2.06 inches.
- City flooding on June 5, 2011 – represents a much shorter event, with the bulk of the rain occurring in 3 to 4 hours. The rain total for this event was 1.64 inches.

Results from the validation runs compared well to the anecdotal observations that the City provided. City staff concurred with the comparisons and confirmed the system network represented in the model matches the actual storm drainage system.

Scenario Analyses

A series of model runs were completed to evaluate the existing and future system and conditions, as follows:

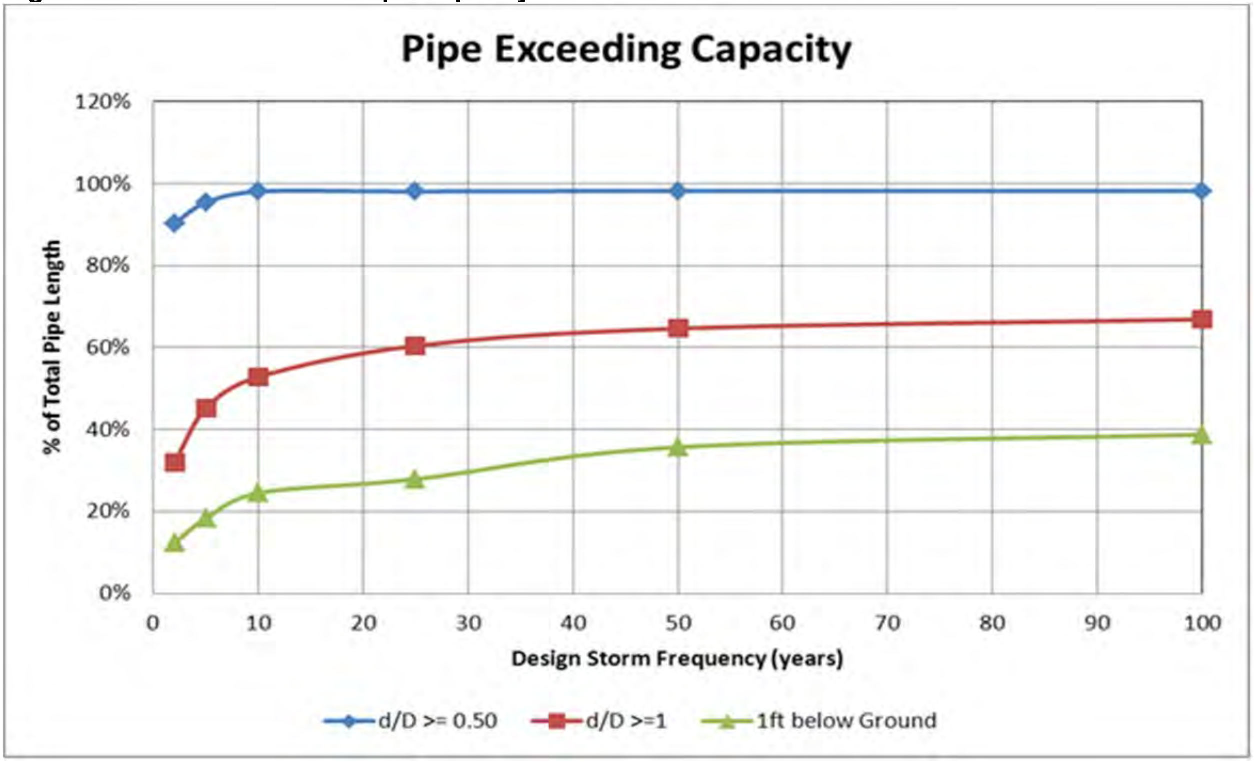
- **Existing System: Level of Service Runs** – used to identify the adopted level-of-service criteria identified in Section 5.4.1. The following storm frequencies (all with 24-hour durations) were evaluated to compare the extent of deficiencies identified in the system:

2-year, 5-year, 10-year, 25-year, 50-year, and 100-year. For each storm frequency different capacity criteria were also evaluated: (i) pipe 50% full; (ii) pipe full; and (iii) hydraulic grade line (HGL) at one foot below ground surface. *Note: the results from this series of runs were not used to identify deficiencies or improvements, but rather to confirm the LOS criteria.*

The LOS runs showed that system capacity was exceeded for a large portion of the storm drainage system for the larger storms when the capacity was limited to 50% and full pipe capacity (Figure 5-4). For example, at the 10-year frequency the capacity of nearly all of the storm drainage pipes was exceeded using the 50% pipe full criterion. Thus, for planning purposes the City agreed that for the existing system the LOS criteria would be defined as a 2-year, 24-hour storm event with the maximum HGL at one foot below ground surface.

- **Existing System: Deficiency Runs and Improvements** – used to identify deficiencies in the existing system using the adopted LOS criteria for the existing system, and to develop improvements to address existing system deficiencies relative to the following LOS requirements:
  - Evaluate conveyance system against the defined LOS: 2-year, 24-hour
  - Evaluate storage facilities against defined LOS: 100-year, 10-day for retention facilities, or 100-year, 2-day for detention facilities. This analysis was completed using spreadsheet analysis rather than the hydraulic model.
- **Future System Phase 1: Deficiency Runs and Improvements** – used to confirm that the existing system still meets the 2-year, 24-hour LOS; and to size new pipes added to the expanded areas under Phase 1 to meet the 10-year, 24-hour LOS for conveyance, and 100-year, 10-day or 100-year, 2-day LOS for storage facilities.
- **Future System Phase 2 (Build-out): Deficiency Runs and Improvements** – used to confirm that the existing system still meets the 2-year, 24-hour LOS; and to size any new pipes added to the expanded areas under Phase 1 and Phase 2 to meet the 10-year, 24-hour LOS for conveyance, and 100-year, 10-day or 100-year, 2-day LOS for storage facilities.

Figure 5-4. Level of Service Pipe Capacity Results



Storage Facility Analysis

For each storage facility the necessary volume was evaluated using either a 100-year, 10-day event for retention facilities, or a 100-year, 2-day event for detention facilities. For those events, volume requirements are dependent on drainage area and a land-use-based runoff coefficient using the following equation:

$$Volume = St \cdot C \cdot A$$

where:

- *Volume = Storage volume needed (acre-feet)*
- *St = Storm event factor; depth of rainfall accumulation for a given storm minus infiltration losses (feet)*
- *C = Runoff coefficient*
- *A = Drainage area (acres)*

The storm event factor for the two different LOS storm events is as follows[1]:

- For the 100-year, 10-day event: *St* = 0.55 ft
- For the 100-year, 2-day event: *St* = 0.28 ft

[1] Source: Department of Water Resources, CA Climate Rainfall Depth Duration Frequency, Goodridge 2007 (using Lat.=36°35'59.1"N and Lon.=119°26'45.36"W)

The runoff coefficient for different land use types is shown in Table 5-7. For each drainage area contributing to a storage facility the runoff coefficient was comprised of the weighted average of the different land use categories that comprised the contributing drainage area.

Table 5-7. Runoff Coefficients for Storage Volume Calculation

Land Use Category	Land Use Breakdown	Runoff Coefficient [(a)]
Low Residential	n/a	0.25
Medium Residential	n/a	0.28
High Residential	n/a	0.40
Suburban Residential	n/a	0.28
Public	n/a	0.30
Commercial	Central Downtown Community Commercial Neighborhood Commercial Office Service Commercial	0.70
Industrial	Heavy Industrial Light Industrial	0.65
Other	Open Space Open Space (Park)	0.12

(a) Based on *Table 1 in Appendix D of the 1982 City of Reedley Storm Drainage Master Planning Report.*

Improvement Criteria

The improvements to the existing system and the Phase 1 and Phase 2 system were developed using the following guiding principles and considerations:

- Upsize storm pipes in the existing system to address the capacity issues using the 2-year, 24-hour storm event, while including flow contributions from build-out areas where applicable.
- Size new pipes (in new areas for Phases 1 & 2) using the 10-year, 24-hour event.
- Direct all new piping to new retention facilities to the extent possible and away from the existing system, since there are significant capacity limitations in the existing system.
- Evaluate whether new pipes should flow to existing retention facilities, or be routed to new retention facilities if the additional flow causes capacity issues within a drainage basin.
- For Phase 1, locate new pipes in areas where development is expected (mainly in Basins K and Q on the east side of town).

- New retention facilities were generally located in close proximity to the AID canals, such that in the future, the City could purchase ditch water, if available, and use the retention facilities for percolation to augment groundwater storage. Generally the preference was to locate new storage facilities in the north and east of the City, because the groundwater is generally flowing southward and slightly west.

The improvements identified address the deficiencies relative to the design criteria in Table 5-6 to meet the LOS requirements.

5.4.3. System Deficiencies

The results of the scenario runs for the existing and future (Phase 1 and Phase 2) systems are shown in Figure 5-5 through Figure 5-8. The figures present the following information:

- **Figure 5-5. Existing System Capacity Limitations:** identifies the location of capacity limitations for the existing system for the LOS storm (2-yr, 24-hour). Locations where performance issues were identified generally correlate with observed system flooding conditions.

- **Figure 5-6. Existing System Capacity Limitations with Pipe Improvements:** shows capacity analysis of the existing system with the recommended pipeline improvements in place for the LOS storm. In some cases, it was necessary to make changes to pipelines downstream of the identified capacity limitations shown in Figure 5-5 to improve transmission of flow to downstream facilities.

- **Figure 5-7. Phase 1 System Capacity Limitations:** identifies the capacity limitations for the Phase 1 system for the LOS storm. This analysis assumes that the existing system pipeline improvements have been implemented. No improvements to the existing drainage system were identified to meet Phase 1 growth.

- **Figure 5-8. Phase 2 Capacity Limitations**: identifies the capacity limitations for the Phase 2 system for the LOS storm. This analysis assumes that the existing system pipeline improvements have been implemented. Some of the new pipes in the Phase 2 analysis show up as capacity limited. This is caused by backwater from downstream (due to limited capacity in downstream pipes). Once the downstream capacity limitations are addressed, any limitations indicated in the new Phase 2 pipelines are resolved.

Each map shows the deficiencies for the identified growth condition for the 2-year, 24-hour storm LOS condition. A capacity deficiency was identified if the HGL was within 1 foot of ground surface elevation.

Calculated storage facility volumes are shown in Table 5-8. The analysis shows there are two existing storage facilities that do not currently meet the volume requirements. These are the facilities in Basins L and P. Currently there are two temporary facilities in Basin P. These are to be replaced with a new facility in Phase 1. New facilities are also planned in Phase 2 for locations in Basins designated as New 1, New 2, New 3, New 4 and New 5.

Table 5-8. Storage Facility Evaluation Results

Drainage Basin	Existing Storage Facility Volume (acre-ft)	Storage Facility Volume Needed (acre-ft) [a]			Notes
		Existing	Phase 1	Phase 2	
Basin A	12.2	10.7	10.7	10.7	Retention facility
Basin B	85.1	48.3	58.3	58.3	Retention facility
Basin C [b]	102.4	52.7	52.7	52.7	Detention facility
Basin J [b]	6.7	3.9	3.9	3.9	Detention facility
Basin K	250.5	28.4	36.4	113.2	Retention facility
Basin L	16.4	**38.2**	**38.2**	**38.2**	Retention facility
Basin P (north) [c]	13.4	9.9	**45.9**	**76.0**	New facility in Phase 1
Basin P (south) [c]	5.2	**9.2**	--	--	New facility in Phase 1
Basin Q	18.0	13.7	**49.1**	**72.7**	Retention facility
New 1	--	--	--	115.9	New facility in Phase 2
New 2	--	--	--	73.1	New facility in Phase 2
New 3	--	--	--	55.3	New facility in Phase 2
New 4	--	--	--	66.2	New facility in Phase 2
New 5	--	--	--	84.9	New facility in Phase 2

(a) Deficiencies are indicated in **bold**.
(b) Basin C and Basin J are detention facilities and were evaluated using a storm event factor of 0.28.
(c) Existing facilities in Basin P are to be abandoned and a new facility is planned in Phase 1.

Figure 5-5. Existing System Capacity Limitations

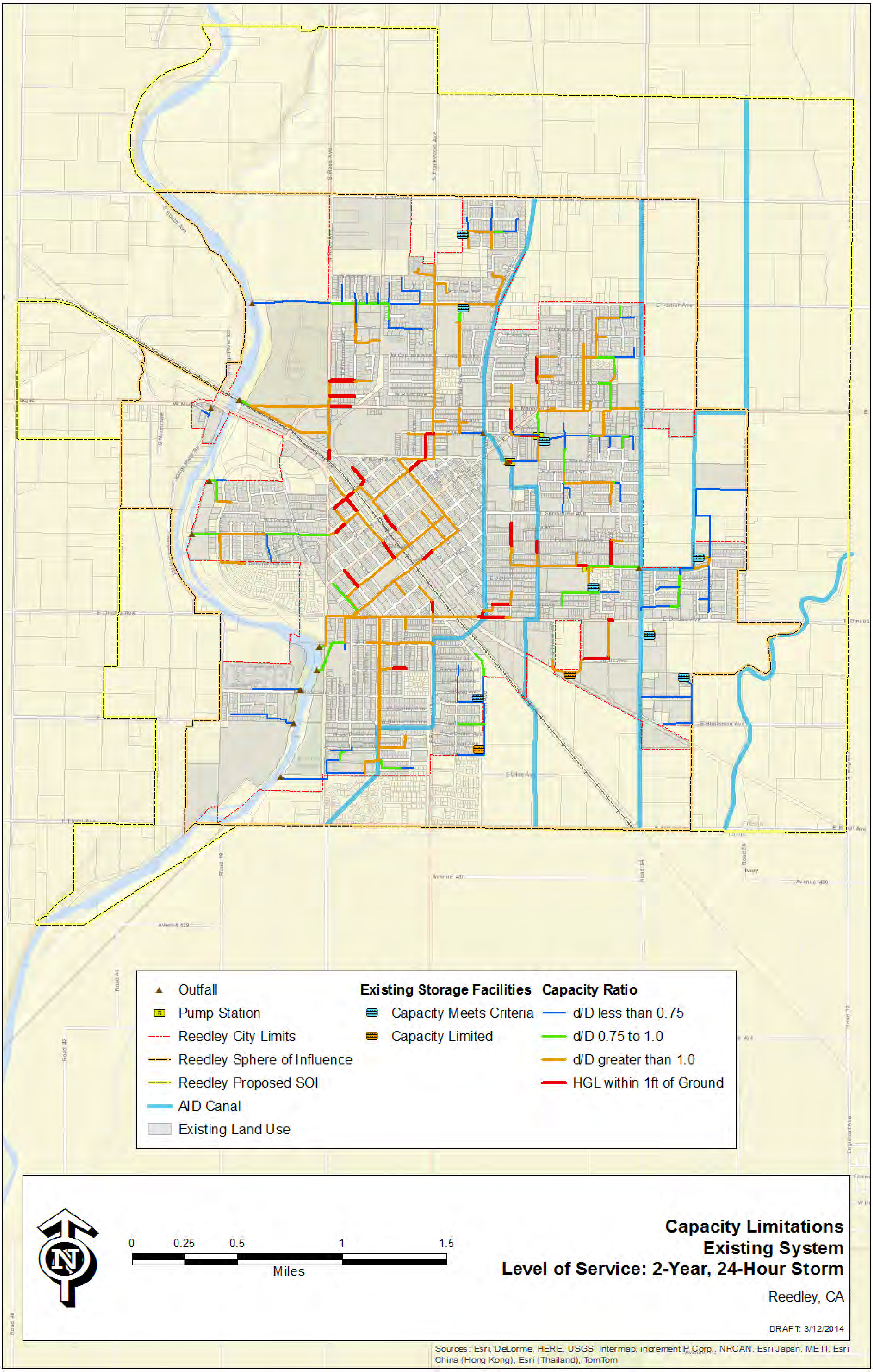


Figure 5-6. Existing System Capacity Limitations with Pipe Improvements

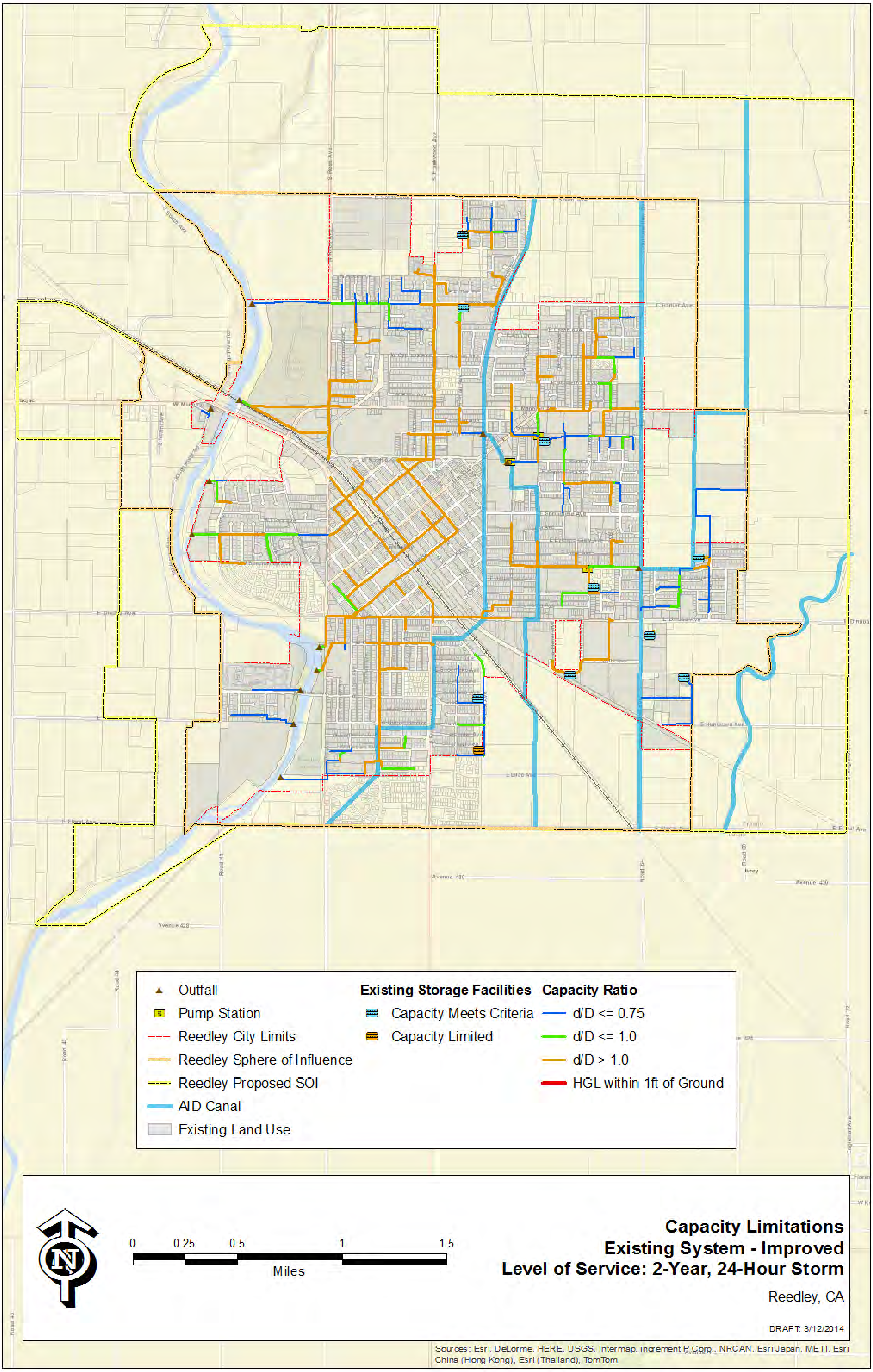


Figure 5-7. Phase 1 System Capacity Limitations

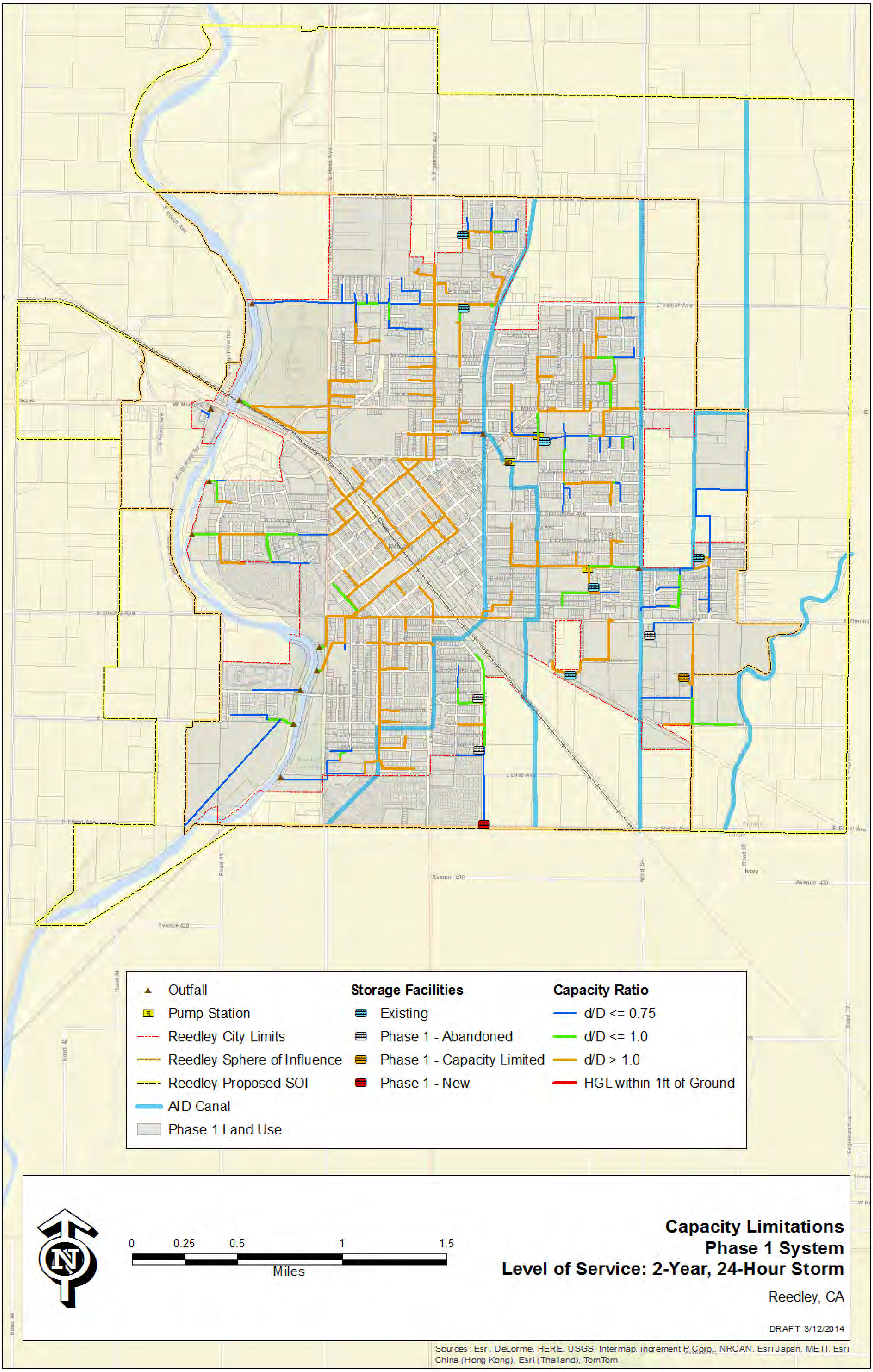
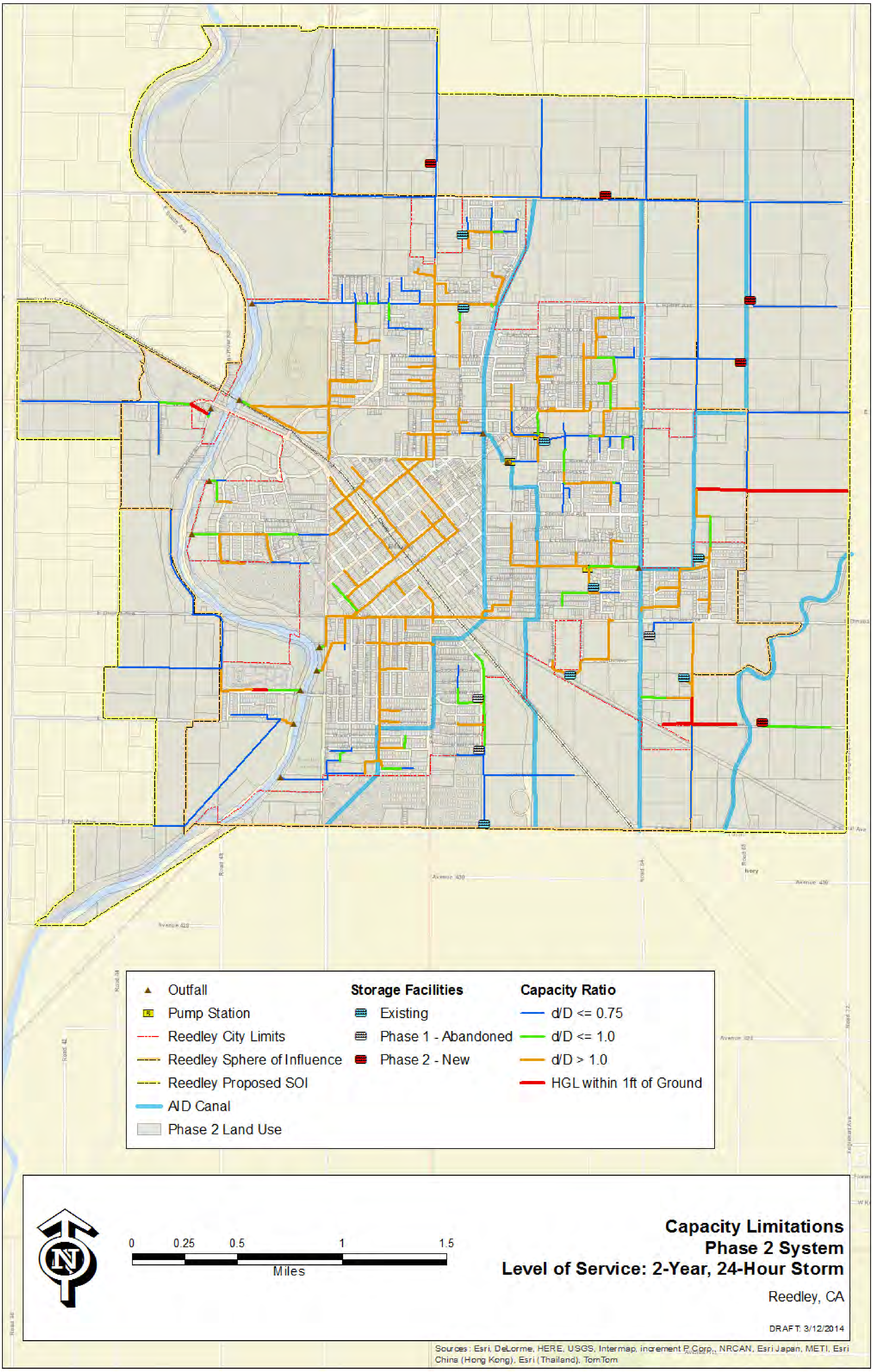


Figure 5-8. Phase 2 Capacity Limitations



5.4.4. Proposed Improvements

Using the improvement criteria described in Section 5.4.2 to address the deficiencies, the following proposed improvements were identified. Figure 5-9 shows the location of the new and improved pipelines and storage facilities.

- New pipes in Basins K and Q to address new development. New development area added in Basin Q results in the retention facility in that basin to be undersized for Phase 1 conditions.

- In Phase 1, new pipes in Basins B, H and N to connect existing storm inlets to the system (Figure 5-3). These inlets are currently tied to the AID irrigation system.

- The existing retention facility in Basin L is undersized and storage at this facility will need to be expanded.

- The southern temporary retention facility in Basin P is undersized. This facility will be replaced in Phase 1 with a new facility that replaces both temporary facilities in Basin P.

- Based on input from the City, it is anticipated that three existing retention facilities will be abandoned. Flow into the storage facility at S Buttonwillow Avenue just south of E Dinuba Avenue will be redirected to the existing facility in Basin Q. A new storage facility will be installed to replace the two temporary storage facilities off of East Avenue in Basin P.

- Five new retention facilities in the Phase 2 development areas. The retention facilities are located in areas where open space was designated in the land use zoning. The retention facilities can serve as recharge basins, since there is an interest in providing more groundwater recharge. The facilities were located based on preferences noted by the City for recharge benefits. The storage facilities can also be designed to serve as parks that include playing fields and other amenities.

Table 5-9 shows the storage facilities that will need to be improved. It is assumed that each facility will be constructed for the Phase 2 volume to serve the ultimate build out need for the City's system or that provisions for phasing to the full volume could be incorporated in the design.

Table 5-9. New Storm Drainage Storage Facilities

Phase	Drainage Basin	Storage Facility Volume (ac-ft)
Existing	Basin L	38.2
Phase 1	Basin P	76.0
	Basin Q	72.7
Phase 2	New 1	115.9
	New 2	73.1
	New 3	55.3
	New 4	66.2
	New 5	84.9

Table 5-10 provides details for the improved and new pipes listed by drainage basin and expected phase. For the Existing phase, priority for improvements should be given to areas where flooding is known to occur – in particular, Basins D, E and H. For these areas, improvements should be prioritized from downstream (e.g., closest to the discharge point) to upstream. Within each drainage basin listed in Table 5-10, pipeline improvements have been given Project IDs starting with the most downstream location.

As described in Section 3, the City has been considering ways to enhance groundwater recharge to avoid and/or minimize groundwater overdraft in the area. To that extent, in addition to the new storage basins described above, the City has considered two additional sites including the Sports Park and a 20-acre parcel located in the northeast corner of Floral and Reed Avenue, adjacent to an existing AID canal. These two properties present excellent recharge opportunities because of the soil make-up and both are in close proximity to AID facilities. Either of these facilities could be used for future storm water retention and groundwater recharge purposes, providing a dual-purpose benefit for the City. However, due their respective locations, they may not be appropriately sited to address future growth in the north and northeastern portions of the City. Based on their respective locations, they could be considered as potential locations for the future storage improvements for Basin Q and Basin P, respectively. The City has been collaborating with AID on these projects for some time. Additional analysis would be needed to determine the feasibility using these sites.

Table 5-10. Storm Drainage Pipeline Improvements

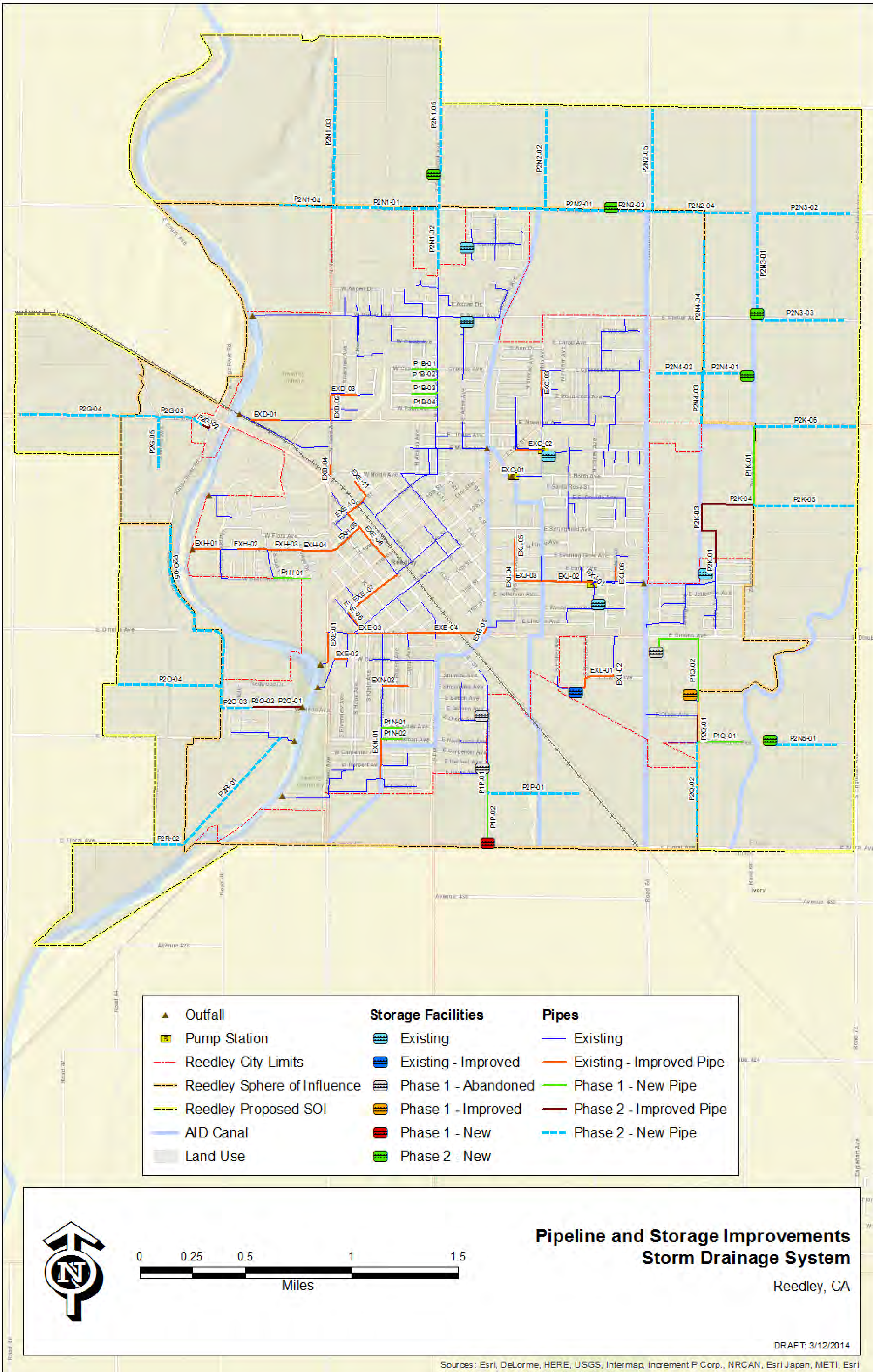
Drainage Basin	Phase	Project ID	Location	Diameter (in)	Length (ft)	Description
Basin B	Phase 1	P1B-01	W Cypress Ave	18	656	New pipe (existing system)
	Phase 1	P1B-02	W Sycamore Ave	18	661	New pipe (existing system)
	Phase 1	P1B-03	W Ponderosa Ave	18	687	New pipe (existing system)
	Phase 1	P1B-04	W Palm Ave	18	667	New pipe (existing system)
Basin C	Existing	EXC-01	Outfall to canal	18	61	Upsize 8" to 18" (outfall)
	Existing	EXC-02	E Myrtle Ave between N Sunset and N Columbia Ave	24	661	Upsize 18" to 24"
	Existing	EXC-03	N Columbia Ave between E Ponderosa to E Cypress Ave	18	665	Upsize 15" to 18"
Basin D	Existing	EXD-01	East of railroad (near outfall)	30	726	Upsize 27" to 30"
	Existing	EXD-02	N Reed Ave between W Manning and W Ponderosa Ave	24	628	Upsize 18" to 24"
	Existing	EXD-03	W Ponderosa Ave between N Reed Ave and N Hope Ave	18	648	Upsize 12" to 18"
	Existing	EXD-04	N Reed Ave north of W North Ave	15	231	Upsize 12" to 15"
Basin E	Existing	EXE-01	S Reed Ave between W Curtis Ave and W Dinuba Ave	36	748	Upsize 24" to 36"
	Existing	EXE-02	W Curtis Ave west of S Riverview Ave to alley	36	329	Upsize 30" to 36"
	Existing	EXE-03	W Dinuba Ave between alley east of Riverview to S Frankwood Ave	36	2205	Upsize 24" to 36"
	Existing	EXE-04	W Dinuba Ave between S Frankwood and S East Ave	30	1209	Upsize 24" to 30" (includes RR crossing)
	Existing	EXE-05	S East Ave between W Dinuba Ave to G St	24	170	Upsize 18" to 24"
	Existing	EXE-06	M St between 13th St and 12th St	48	412	Upsize 36" to 48"
	Existing	EXE-07	12th St between M St and I St	48	1552	Upsize 36" to 48"
	Existing	EXE-08	I St between 11th St and 9th St	24	969	Upsize 18" to 24"
	Existing	EXE-09	I St between 9th St and 8th St	30	472	Upsize 18" to 30"
	Existing	EXE-10	8th St between I St and alley east of H St	24	635	Upsize 18" to 24" (includes RR crossing)
	Existing	EXE-11	Alley east of H St between 8th St and W North Ave	24	450	Upsize 20" to 24"

Drainage Basin	Phase	Project ID	Location	Diameter (in)	Length (ft)	Description
Basin G	Phase 2	P2G-01	Outfall to Kings River (near Edgewater Inn)	36	183	Upsize 24" to 36" (outfall)
	Phase 2	P2G-02	Northwest of N Kings River Rd (near Edgewater Inn)	36	203	Upsize 15" to 36"
	Phase 2	P2G-03	W Manning Ave between S Nurmi and Edgewater Inn	54	1114	New pipe
	Phase 2	P2G-04	W Manning Ave between S Lac Jac Ave and Nurmi Ave	48	3447	New pipe
	Phase 2	P2G-05	S Nurmi Ave south of W Manning Ave	24	1293	New pipe
Basin H	Existing	EXH-01	Outfall to Kings River	36	676	Upsize 30: to 36" (outfall)
	Existing	EXH-02	W Eymann Ave between S Willow Glen Dr and alley west of S Oak Dr	30	744	Upsize 27" to 30"
	Existing	EXH-03	W Eymann Ave between alley west of S Oak Dr and S Kings Dr	24	855	Upsize 18" to 24"
	Existing	EXH-04	W Eymann Ave between S Kings Dr and S Reed Ave	30	729	Upsize 18" to 30"
	Existing	EXH-05	9th St between S Reed Ave and northeast of J St	24	718	Upsize 18" to 24"
	Existing	EXH-06	9th St between alley northeast of J St and I St	24	194	Upsize 20" to 24"
	Phase 1	P1H-01	W Beechwood Ave between alley west of S Oak Dr and S Kings Dr	18	917	New pipe (existing system)
Basin J	Existing	EXJ-01	E Duff Ave between detention pond and S Hemlock Ave	36	309	Upsize 21" to 36"
	Existing	EXJ-02	E Duff Ave between S Hemlock and S Columbia Ave	30	987	Upsize 21" to 30" (ditch crossing)
	Existing	EXJ-03	E Duff Ave between S Columbia Ave and S Sunset Ave	24	648	Upsize 15" to 24"
	Existing	EXJ-04	S Sunset Ave between E Duff Ave and E Myra Ave	24	490	Upsize 15" to 24"
	Existing	EXJ-05	S Sunset Ave between E Myra Ave and north of E August Ave	18	565	Upsize 15" to 18"
	Existing	EXJ-06	S Kady Ave between E Duff Ave and E Early Ave	18	352	Upsize 15" to 18"
Basin K	Phase 1	P1K-01	Zumwalt Ave north from Silas Batsch Elementary School to E Manning Ave	42	1960	New pipe
	Phase 2	P2K-01	S Tobu Ave between Evening Glow and E Springfield Ave	48	766	Upsize 36" to 48"
	Phase 2	P2K-02	E Springfield Ave from S Tobu Ave east toward Buttonwillow Ditch	48	360	Upsize 30" to 48"
	Phase 2	P2K-03	Along Buttonwillow Ditch between E Springfield Ave to south of Silas Batsch Elementary	48	669	Upsize 30" to 48"

Drainage Basin	Phase	Project ID	Location	Diameter (in)	Length (ft)	Description
	Phase 2	P2K-04	South of Silas Batsch Elementary from Buttonwillow Ditch to Zumwalt Ave	42	1211	Upsize 30" to 42"
	Phase 2	P2K-05	East of Zumwalt Ave to S Englehart Ave	36	2527	New pipe
	Phase 2	P2K-06	E Manning Ave between Zumwalt and S Englehart Ave	42	2544	New pipe
Basin L	Existing	EXL-01	North from retention basins to E Curtis Ave; East along E Curtis Ave from S Hemlock Ave	30	950	Upsize 24" to 30"
	Existing	EXL-02	North from E Curtis Ave	24	104	Upsize 18" to 24"
Basin N	Existing	EXN-01	S Church Ave between W Sasaki and W Huntsman Ave	30	1006	Upsize 24" to 30"
	Existing	EXN-02	W Shoemake Ave between S Church and Cyrier Aver	18	657	Upsize 8" to 18"
	Phase 1	P1N-01	W Stanley Ave east from S Church Ave	18	517	New pipe (Existing system)
	Phase 1	P1N-02	W Huntsman Ave east from S Church Ave	18	507	New pipe (Existing system)
Basin O	Phase 2	P2O-01	Outfall to Kings River along W Olson Ave	30	875	Upsize 21" to 30" (outfall)
	Phase 2	P2O-02	W Olson Ave west of outfall	30	365	Upsize 18" to 30"
	Phase 2	P2O-03	W Olson Ave west of outfall to S Kings River Rd; North along S Kings River Rd	42	1353	New pipe
	Phase 2	P2O-04	West of S Kings River Rd	30	2562	New pipe
	Phase 2	P2O-05	North along S Kings River Rd from new pipe	30	4807	New pipe
Basin P	Phase 1	P1P-01	South along S East Ave between E Davis to Lilac Ave	30	488	New pipe
	Phase 1	P1P-02	S East Ave between Lilac Ave and E Floral Ave (to new retention basin)	48	1258	New pipe
	Phase 2	P2P-01	Lilac Ave between S East Ave and railroad	36	2350	New pipe
Basin Q	Phase 2	P2Q-01	Between Olson Ave and E Huntsman Ave	36	660	Upsize 21" to 36"
	Phase 2	P2Q-02	South from E Huntsman Ave	36	2375	New pipe
	Phase 1	P1Q-01	East along E Huntsman Ave from Existing pipe	42	1110	New pipe
	Phase 1	P1Q-02	North from Existing retention pond to E Dinuba Ave, west to temporary pond	24	2752	New pipe

Drainage Basin	Phase	Project ID	Location	Diameter (in)	Length (ft)	Description
Basin R	Phase 2	P2R-01	New pipeline southwest from Existing system	42	3409	New pipe
	Phase 2	P2R-02	New pipeline along E Floral Ave	30	776	New pipe
New 1	Phase 2	P2N1-01	S Frankwood Ave from new retention pond south to E South Ave; E South Ave between S Frankwood and S Reed Ave	48	3649	New pipe
	Phase 2	P2N1-02	South on S Frankwood Ave from E South Ave	24	1506	New pipe
	Phase 2	P2N1-03	North on S Reed Ave from E South Ave	48	4080	New pipe
	Phase 2	P2N1-04	West on E South Ave from N Reed Ave	36	3179	New pipe
	Phase 2	P2N1-05	North from new retention pond on S Frankwood Ave	30	3912	New pipe
New 2	Phase 2	P2N2-01	West from new retention pond on E South Ave to N Sunny Ln	48	2906	New pipe
	Phase 2	P2N2-02	North from E South Ave	36	2492	New pipe
	Phase 2	P2N2-03	East from new retention pond on E South Ave to S Buttonwillow Ave	36	1056	New pipe
	Phase 2	P2N2-04	East on E South Ave from S Buttonwillow Ave	30	2441	New pipe
	Phase 2	P2N2-05	North on S Buttonwillow Ave from E South Ave	36	2492	New pipe
New 3	Phase 2	P2N3-01	North from new retention pond along Buttonwillow Ditch to E South Ave	36	2496	New pipe
	Phase 2	P2N3-02	E South Ave between Buttonwillow Ditch to S Englehart Ave	30	2311	New pipe
	Phase 2	P2N3-03	E Parlier Ave from new retention pond to S Englehart Ave	30	2249	New pipe
New 4	Phase 2	P2N4-01	West from new retention pond along E Cypress Ave	36	1129	New pipe
	Phase 2	P2N4-02	West from new retention pond along E Cypress Ave	30	1140	New pipe
	Phase 2	P2N4-03	South from new pipe to E Manning Ave	30	1292	New pipe
	Phase 2	P2N4-04	North from E Cypress Ave	24	3288	New pipe
New 5	Phase 2	P2N5-01	E Huntsman Ave east of new detention pond	36	1736	New pipe

Figure 5-9. Pipeline and Storage Improvements



6. Recommended CIP

This section of the Master Plan presents the recommended capital improvement plan (CIP) for the City's potable water, sanitary sewer, and storm drainage systems, and presents the costs and schedules for projects planned for implementation between 2015 and 2030.

6.1. Development of CIP

The CIP was prepared by first identifying projects that address needs or deficiencies in the potable water, sanitary sewer, and storm drainage systems, as described in earlier chapters of this Master Plan. A 15-year implementation schedule of the projects was then developed for those projects needed to address existing deficiencies as well as Phase 1 deficiencies. Due to the uncertainty of the timing associated with future development in Phase 2, the projects needed to serve Phase 2 growth have not been scheduled. The schedule should be optimized for coordination with other City programs and capital project opportunities, as discussed in Section 6.6.

Generally, projects of higher priority (i.e., those that address existing system deficiencies) were scheduled for implementation within the ten-year planning horizon between 2015 and 2025, while those projects required to serve Phase 1 growth were scheduled for implementation between 2025 and 2030. In addition, for the City's sanitary sewer and storm drainage systems, downstream pipeline improvements have been scheduled for improvement before their upstream tributary pipelines to avoid bottlenecks, as described in Sections 4 and 5, respectively.

Planning level cost estimates have been developed for each capital project, including those included in the 15-year CIP as well as those required to serve Phase 2 development. Each project cost includes the following components:

- **Base Construction Cost**. Includes all labor and material costs needed to construct a project.
- **Construction Contingency**. A 30 percent construction contingency has been applied to all projects to account for uncertainties associated with estimated project costs at the planning level.
- **Engineering, Administration, and Permitting**. A 25 percent allowance was included for engineering, administration and permitting (EAP), which includes City and consultant

design costs, and other related cost items, such as permitting and construction administration.

These elements are summed to determine the total project-level cost estimate for a project. All costs have been presented in 2014 dollars. EAP costs are scheduled one year in advance of construction costs, to reflect a typical period for design and permitting.

6.2. Potable Water System CIP

Table 6-1 presents the City's schedule of potable water system CIP projects planned for implementation between 2015 and 2030, as well as the estimated project cost for those projects recommended to accommodate Phase 2 development. Figure 6-1 provides the locations for the major planned improvements through Phase 1, while the recommended improvements for Phase 2 are illustrated in Figure 6-2.

As Table 6-1 indicates, the potable water system CIP includes approximately $20.5 million for new wells, water storage, and pipeline improvements to correct existing system deficiencies and accommodate Phase 1 growth. An additional $72.2 million is needed to expand the system to accommodate Phase 2 growth in the potable water system. Refer to Chapter 3 for a description of the recommended potable water system projects.

6.3. Sanitary Sewer System CIP

Table 6-2 presents the City's schedule of sanitary sewer system CIP projects planned for implementation between 2015 and 2030, as well as the estimated project cost for those projects recommended to accommodate Phase 2 development. Figure 6-3 provides the locations for the major planned improvements through Phase 1, while the recommended improvements for Phase 2 are illustrated in Figure 6-4.

As Table 6-2 indicates, the sanitary sewer system CIP includes approximately $27.9 million for lift station and pipeline improvements to correct existing system deficiencies and accommodate Phase 1 growth. An additional $70.2 million is needed to expand the system to accommodate Phase 2 growth in the sanitary sewer system. Refer to Chapter 4 for a description of the recommended sanitary sewer system projects.

Table 6-1. Recommended Potable Water System CIP (1,000s)

Project ID	Description of Water System Improvement	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
Well Improvements																		
PEWU-1	Add an additional bowl to existing Well 5A	Existing	13	13														
PEWU-2	Add an additional bowl to existing Well 10	Existing	13	13														
PEWU-3	Add an additional bowl to existing Well 12	Existing	13	13														
P1WE-1	New 2,000 gpm Well, Sports Park Well[b]	Existing	N/A															
P1WE-2	New 2,000 gpm Well, North Central Well	Phase 1	2,555							511	2,044							
P2WE-1	New 2,000 gpm Well, North Well 1	Phase 2	2,555															
P2WE-2	New 2,000 gpm Well, North Well 2	Phase 2	2,555															
P2WE-3	New 2,000 gpm Well, North Well 3	Phase 2	2,555															
P2WE-4	New 2,000 gpm Well, East Well 1	Phase 2	2,555															
P2WE-5	New 2,000 gpm Well, East Well 2	Phase 2	2,555															
P2WE-6	New 2,000 gpm Well, South Well 1	Phase 2	2,555															
P2WE-7	New 2,000 gpm Well, South Well 2	Phase 2	2,555															
P2WE-8	New 2,000 gpm Well, West Well 1	Phase 2	2,555															
P2WE-9	New 2,000 gpm Well, West Well 2	Phase 2	2,555															
Subtotal Well Improvements			25,589	39	0	0	0	0	0	511	2,044	0	0	0	0	0	0	0
Storage Improvements																		
P1RE-1	New Downtown Reservoir, 1 MG Ground Storage	Phase 1	3,934											787	3,147			
P1RE-1	Abandon 0.1 MG Downtown Tank	Phase 1	123	123														
P1RE-2	New 1.4 MG Elevated Storage Tank, Sports Park Water Tower[b]	Existing	N/A															
P1RE-3	Buttonwillow Reservoir, 2 MG Ground Storage	Existing	5,510				1,102	2,204	2,204									
P2RE-1	North Reservoir, 1.4 MG Ground Storage	Phase 2	4,565															
P2RE-2	South Reservoir, 1.2 MG Ground Storage	Phase 2	4,250															
P2RE-3	West Reservoir, 1.5 MG Ground Storage	Phase 2	4,723															
Subtotal Storage Improvements			23,105	123	0	0	1,102	2,204	2,204	0	0	0	0	787	3,147	0	0	0
Pipeline Improvements																		
P1EA-1	Sports Park Water Tower Appurtenance Piping	Existing	350	350														
P1EA-2	Buttonwillow Parlier Tank Appurtenance Piping	Existing	141			28	113											
P1EA-3	Downtown Tank Appurtenance Piping	Phase 1	76											15	61			
P1FA-1	New pipe, W Aspen Dr & N Church Ave (1320')	Existing	431	86	345													
P1FA-3	New pipe, West Cypress Ave & N Hollywood Dr (180')	Phase 1	28											6	22			
P1FA-4	New pipe, West of W Palm Ave & W Manning Ave (1020')	Existing	29	6	23													
P1FA-5	New pipe, North of N Reed Ave & W North Ave (5160')	Existing	16			3	13											

Project ID	Description of Water System Improvement	Phase	Project Cost[(a)]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P1FA-6	New pipe, W North Ave & F St (230')	Existing	39		8	31												
P1FA-7	New pipe, East of W Palm Ave & W Manning Ave (250')	Existing	49		10	39												
P1FA-8	New pipe, Palm Village Retirement Community (130')	Existing	13		3	10												
P1FA-9	New pipe, E Shoemake Ave & S Frankwood Ave (240')	Existing	25		5	20												
P1FA-10	New pipe, E Curtis Ave & S Buttonwillow Ave (720')	Existing	460	92	368													
P1FA-11	New pipe, El Dorado Ave (160')	Existing	21															
P1FA-12	New pipe, E Manning Ave & N Buttonwillow Ave (1380')	Existing	210			42	168											
P1FA-13	New pipe, E Manning Ave & Zumwalt Ave (1680')	Existing	1,209		242	967												
P1FU-1	New pipe, North Reed Ave & W Aspen Dr (310')	Phase 1	563										113	450				
P1FU-2	New pipe, W Aspen Dr (110')	Existing	78	16	62													
P1FU-3	New pipe, West Manning Ave (200')	Existing	1,150	230	920													
P1FU-4	New pipe, N Birch Ave (1500')	Existing	38	8	30													
P1FU-5	New pipe, E Curtis Ave & S Buttonwillow Ave (320')	Existing	175	35	140													
P1FU-6	New pipe, W Olson Ave & West Bank of King's River (2300')	Phase 1	331										66	265				
P1HU-1	New pipe, North of N Reed Ave & W Manning Ave (250')	Existing	174						35	139								
P1HU-2	New pipe, South of N Reed Ave & W Manning Ave (2420')	Phase 1	49									10	39					
P1HU-3	New pipe, N Hollywood Dr (3410')	Phase 1	145									29	116					
P1HU-4	New pipe, W Myrtle Ave & N Acacia Ave, N Birch Ave (40')	Existing	240						48	192								
P1HU-5	New pipe, Manning Ave & N Frankwood Ave (200')	Existing	103								21	82						
P1HU-6	New pipe, Cypress Ave & Concord Ave (980')	Existing	66					13	53									
P1HU-7	New pipe, E Manning Ave & Del Altoir Ave (1400')	Existing	81						16	65								
P1HU-8	New pipe, E 11th St & N East Ave (520')	Existing	291								58	233						
P1HU-9	New pipe, S Kings Drive Cir & Beechwood Ave (270')	Phase 1	118									24	94					
P1HU-10	New pipe, 1st St & 10th St (720')	Phase 1	55						0	0					11	44		
P1HU-11	New pipe, 1st St & 13th St (1130')	Phase 1	13						0	0					3	10		
P1HU-12	New pipe, S East Ave & E Springfield Ave (710')	Existing	125						25	100								
P1HU-13	New pipe, S East Ave & E August Ave, E Myra Ave (280')	Existing	525						105	420								
P1HU-14	New pipe, W Dinuba Ave & S Frankwood Ave (50')	Existing	770								154	616						
P1HU-15	New pipe, South of S Reed Ave & Beech Ave - Well 10 (640')	Phase 1	9									2	7					
P1VU-1	New pipe, E 11th St & N East Ave (490')	Existing	25												5	20	0	0
P1VU-2	New pipe, 1st St & S Frankwood Ave (1180')	Existing	114												23	91	0	0
P1VU-3	New pipe, Klein Ave at Reed Ave (600')	Phase 1	44														9	35
P2EA-1	New pipe, E South Ave West of Reed Ave (80')	Phase 2	1,293															
P2EA-2	New pipe, Reed Ave North and South of E South Ave West (3970')	Phase 2	1,261															

Project ID	Description of Water System Improvement	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2EA-3	New pipe, Sumner Ave between Reed Ave and Sunny Ln (5450')	Phase 2	2,140															
P2EA-4	New pipe, South Ave between Reed Ave and Concord Ave (2670')	Phase 2	1,318															
P2EA-5	New pipe, Frankwood Ave South of South Ave (6150')	Phase 2	406															
P2EA-6	New pipe, W Parlier Ave West of Reed Ave (8450')	Phase 2	629															
P2EA-7	New pipe, Sunny Ln Expansion North (4820')	Phase 2	930															
P2EA-8	New pipe, Sumner between Sunny Ln and Buttonwillow Ave (1730')	Phase 2	1,138															
P2EA-9	New pipe, Buttonwillow between Sumner Ave and South Ave (2680')	Phase 2	871															
P2EA-10	New pipe, South Ave West of Buttonwillow Ave (290')	Phase 2	480															
P2EA-11	New pipe, Buttonwillow between South Ave and Parlier Ave (180')	Phase 2	1,794															
P2EA-12	New pipe, South Ave between Buttonwillow and Englehart (4950')	Phase 2	2,226															
P2EA-13	New pipe, Englehart Ave between South Ave and Parlier Ave (2040')	Phase 2	930															
P2EA-14	New pipe, Parlier Ave and Columbia Ave (3830')	Phase 2	548															
P2EA-15	New pipe, Buttonwillow Ave at Cypress Ave (8640')	Phase 2	130															
P2EA-16	New pipe, Parlier Ave between Buttonwillow and Englehart (3970')	Phase 2	2,053															
P2EA-17	New pipe, Englehart Ave between Parlier Ave and Manning (2330')	Phase 2	931															
P2EA-18	New pipe, Manning Ave between Zumwalt and Englehart Ave (550')	Phase 2	600															
P2EA-19	New pipe, Englehart Ave between Manning and Dinuba Ave (8760')	Phase 2	2,179															
P2EA-20	New pipe, Zumwalt Ave between Duff Ave and Dinuba (2560')	Phase 2	359															
P2EA-21	New pipe, Dinuba Ave between Zumwalt and Englehart Ave (9300')	Phase 2	895															
P2EA-22	New pipe, Sports Park Water Tower (4610')	Phase 2	296															
P2EA-23	New pipe, Englehart Ave between Dinuba and Huntsman Ave (1310')	Phase 2	921															
P2EA-24	New pipe, Huntsman Ave West of Englehart Ave (3820')	Phase 2	1,495															
P2EA-25	New pipe, Reed Ave North of Ave 432 (1260')	Phase 2	329															
P2EA-26	New pipe, East Ave South of Davis Ave (3930')	Phase 2	649															
P2EA-27	New pipe, Floral Ave between East Ave and Buttonwillow Ave (6380')	Phase 2	1,279															
P2EA-28	New pipe, Buttonwillow Ave North of Floral Ave (1400')	Phase 2	540															
P2EA-29	New pipe, Kings River Road North of Dinuba Ave (2760')	Phase 2	406															
P2EA-30	New pipe, Kings River Road between Dinuba and Huntsman (2300')	Phase 2	756															
P2EA-31	New pipe, Huntsman between Wallace and Kings River Rd (1730')	Phase 2	659															
P2EA-32	New pipe, S Wallace Ave (9130')	Phase 2	628															
P2EA-33	New pipe, W Manning between Lac Jac and Kings River Rd (3220')	Phase 2	1,585															
P2EA-34	New pipe, Nurmi Ave to Kings River Rd (2800')	Phase 2	2,199															
P2HU-1	New pipe, 11th St and East Ave (4860')	Phase 2	25															
P2HU-2	New pipe, Parlier Ave East of Cedar Ave (3970')	Phase 2	61															

Project ID	Description of Water System Improvement	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2HU-3	New pipe, Eymann Ave between Kings Dr and Reed Ave (210')	Phase 2	115															
P2HU-4	New pipe, Beechwood Ave between Oak Dr and Kings Dr (3720')	Phase 2	155															
P2HU-5	New pipe, Springfield Ave between Sunset Ave and Justine Ave (70')	Phase 2	175															
P2HU-6	New pipe, Evening Glow between Sunset Ave and Lingo Ave (780')	Phase 2	126															
P2HU-7	New pipe, Early Ave and Kady Ave (900')	Phase 2	6															
P2HU-8	New pipe, Zumwalt Ave and Duff Ave (320')	Phase 2	80															
P2HU-9	New pipe, Eymann between Kingswood and Willow Glenn Dr (790')	Phase 2	78															
Subtotal Pipeline Improvements			44,053	823	2,156	1,144	311	13	282	916	233	996	435	736	125	165	9	35
Total Water System Improvements			92,747	985	2,156	1,144	1,413	2,217	2,486	1,427	2,277	996	435	1,523	3,272	165	9	25

(a) All costs are presented in 1,000s of dollars. All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.
(b) Both the Sports Park Water Tower and the Sports Park well are currently under construction and should be complete in 2014. Thus a project cost has not been included in the recommended CIP.

Figure 6-1. Recommended Potable Water System Improvements through Phase 1

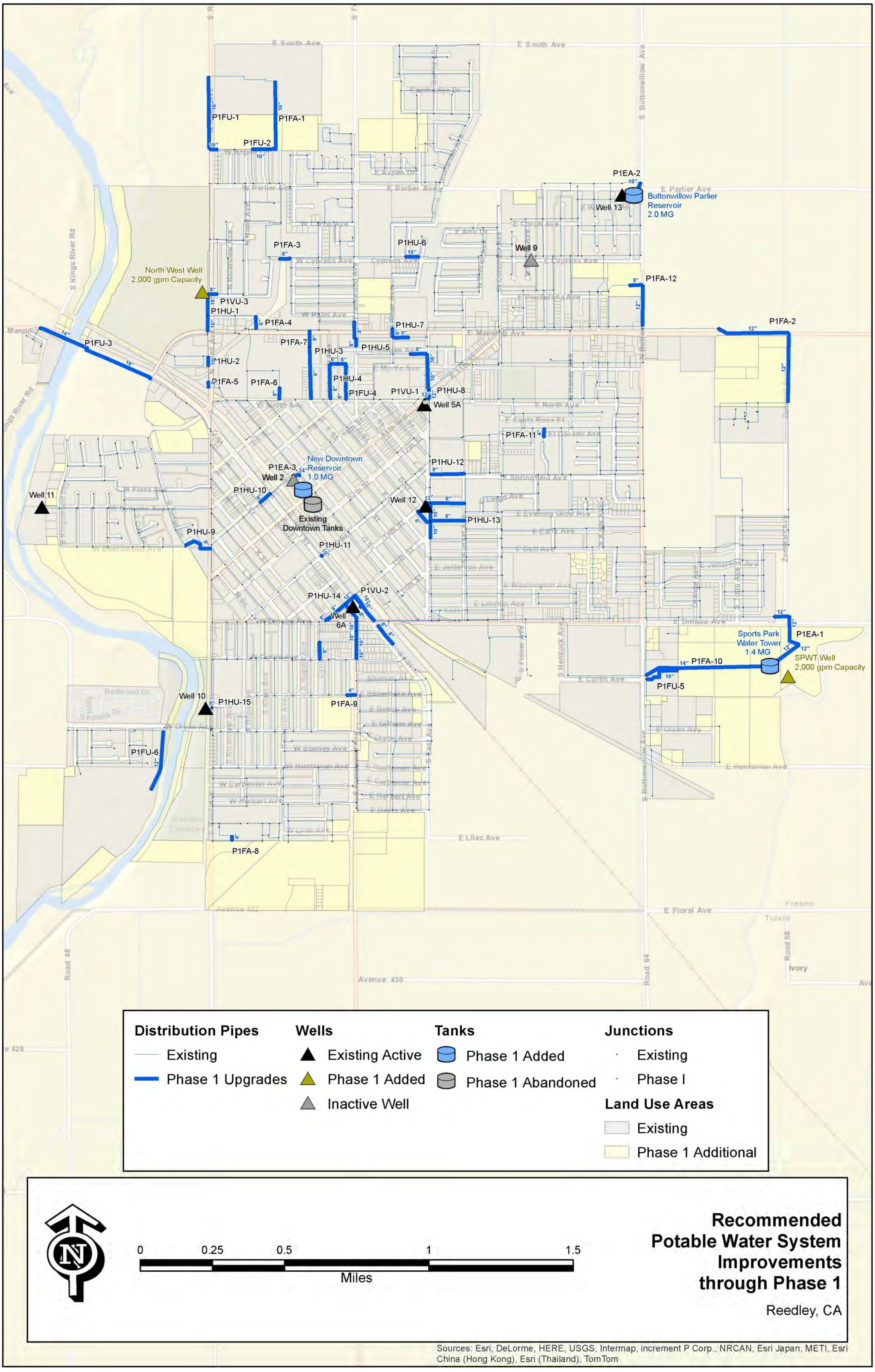


Figure 6-2. Recommended Potable Water System Improvements for Phase 2

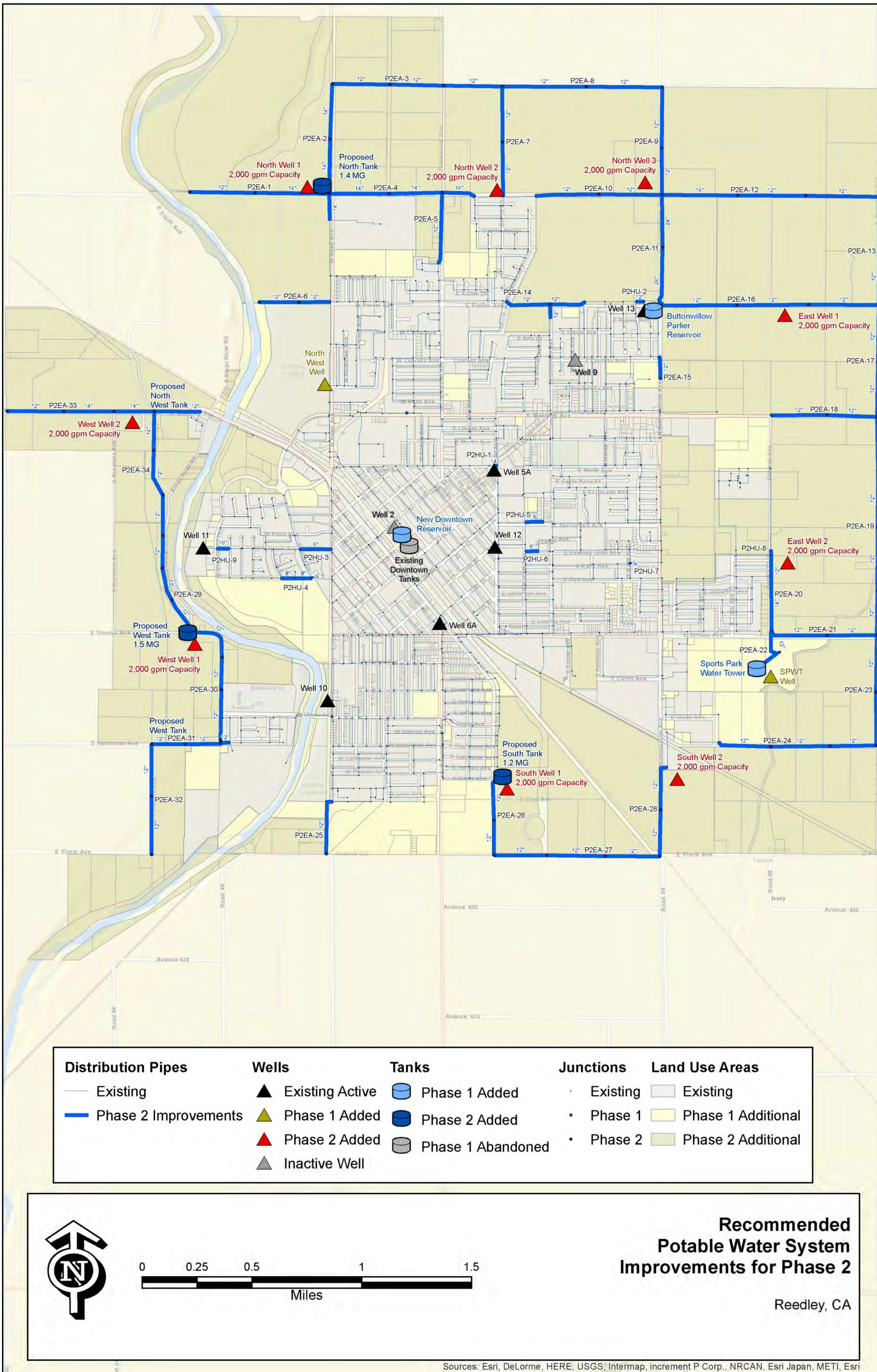


Table 6-2. Recommended Sanitary Sewer System CIP (1,000s)

Project ID	Sewer System Improvement Project Description	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
Lift Station Improvements																		
P1LS-1	Upgrade to Reed Ave Lift Station - Add 550 gpm standby pump	Existing	64	64														
P2LS-2	New lift station at Buttonwillow Canal	Phase 2	2,134															
	Subtotal Lift Station Improvements		2,198	64	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Pipeline Improvements																		
P1DN-1	W Dinuba Ave between Reed Ave & 15th St, Upsized from 12" to 18" (2130')	Existing	1,506					301	1,205									
P1DN-2	15th St between W Dinuba Ave & S East Ave, Upsized from 10" to various (2360')	Existing	1,440					288	1,152									
P1DN-3	15th St between W Dinuba Ave & S East Ave, Upsized from 8" to 10" (610')	Existing	190										38	152				
P1RA-1	Reed Ave between Olson Ave & W Shoemake Ave, Upsized from 21" to 30" (660')	Existing	871	174	697													
P1RA-2	Reed Ave between Dinuba Ave & W Shoemake Ave, Upsized from 12" to various (1280')	Existing	1,371				274	1,097										
P1RA-3	Reed Ave between Dinuba Ave & 11th St, Upsized from 12" to 24' (750')	Phase 1	790												158	632		
P1RA-4	Reed Ave between 11th St & 8th St, Upsized from 12" to various (1900')	Phase 1	1,856													371	1,485	
P1RA-5	Reed Ave between W Manning Ave & 8th St, Upsized from 8/10" to various (2630')	Phase 1	1,691														338	1,353
P1RA-6	Manning Ave between Reed Ave and Kings River, Profile Reverse and Upgraded (3600')	Phase 1	1,041														208	833
P1RC-1	South of Olson Ave at WWTP, Upsized from 12/21" to 33" (1570')	Existing	1,679	336	1,343													
P1RC-2	W Olson Ave - Kings River Crossing, Upsized from 21" to 33' (660')	Existing	781		156	625												
P1SM-1	Shoemake Ave between Reed Ave & S Frankwood Ave, Upsized from 18" to 27" (2720')	Existing	3,251			650	2,601											
P1SM-2	Shoemake Ave S Frankwood Ave & Railroad, Upsized from 18/24" to 27" (1990')	Existing	2,414						483	1,931								
P1SM-3	Columbia Ave between Dinuba Ave & Springfield Ave, Upsized from 18" to various (2630')	Existing	2,899							580	2,319							
P1SM-4	Columbia Ave between Dinuba Ave & Springfield Ave, Upsized from 15" to 18" (2320')	Existing	1,724								345	1,379						
P1SM-5	E Springfield Ave east of N Haney Ave, Upsized from 8" to 12" (1090')	Existing	513										103	410				
P1SM-6	Columbia Ave btn E Manning Ave & E Springfield Ave, Upsized from 12/15" to various (2680')	Existing	1,928									386	1,542					
P1SM-7	E Manning Ave between E 11th St & Buttonwillow Ave, Upsized from 10" to various (2160')	Existing	1,085										217	868				
P1WW-1	W Henley Creek Rd, Upsized from 8" to 10" (1730')	Phase 1	536											107	429			
P1WW-2	WWTP Headworks, Upsized from 21" to 24" (150')	Existing	96											19	77			
P1IP-1	Industrial Pump Station Forcemain Upgrade, Upsized from 6" to 8" (540')	Existing	175											35	140			
P2RA-4	Reed Ave between 11th St & 8th St, Upsized from 18" to various (570')	Phase 2	540															
P2RA-5	Reed Ave between W Manning Ave & 8th St, Upsized from 18" to 21" (2630')	Phase 2	1,975															
P2RA-7	Reed Ave between Parlier and Manning, Upsized from 10" to 18" and P2 Expansion Added (2710')	Phase 2	1,999															
P2RA-8	Parlier Ave West of Reed Ave, Phase 2 Expansion (1720')	Phase 2	671															
P2RA-9	Reed Ave Between Manning Ave and South Ave, Phase 2 Expansion (2640')	Phase 2	1,930															
P2RA-10	South Ave West of Reed Ave, Phase 2 Expansion (3440')	Phase 2	1,008															
P2RA-11	Reed Ave between Adams Ave and South Ave, Phase 2 Expansion (4270')	Phase 2	1,565															

Project ID	Sewer System Improvement Project Description	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2RA-12	South Ave between Reed Ave and Frankwood Ave, Phase 2 Expansion (2630')	Phase 2	1,443															
P2RA-13	Frankwood Ave between South Ave and Adams Ave, Phase 2 Expansion (4270')	Phase 2	1,348															
P2RA-14	South Ave between Frankwood Ave and East Reedley Ditch, Phase 2 Expansion (2210')	Phase 2	1,011															
P2RA-15	West Bank of East Reedley Ditch between Sumner Ave and South Ave, Phase 2 Expansion (2600')	Phase 2	610															
P2RC-1	South of Olson Ave at WWTP, Upsized from 30" to 42" (1570')	Phase 2	2,135															
P2RC-2	W Olson Ave - Kings River Crossing, Upsized from 30" to 42" (390')	Phase 2	720															
P2RE-1	Reed Ave between Olson Ave and Lilac Ave, Phase 2 Expansion (2150')	Phase 2	2,831															
P2RE-2	Lilac Ave between Reed Ave and Columbia Ave, Phase 2 Expansion (5200')	Phase 2	5,031															
P2RE-3	Columbia Ave between Railroad and Lilac Ave, Phase 2 Expansion (1560')	Phase 2	570															
P2RE-4	Lilac Avenue near Buttonwillow Canal, Phase 2 Expansion (2830')	Phase 2	2,186															
P2RE-5	South of Buttonwillow Canal and South of Huntsman Ave, Phase 2 Expansion (2710')	Phase 2	2,335															
P2RE-6	Huntsman Ave West of Englehart Ave, Phase 2 Expansion (3880')	Phase 2	1,756															
P2RE-7	West of Travers Canal between Springfield and Huntsman Ave, Phase 2 Expansion (6530')	Phase 2	6,363															
P2RE-8	Springfield Ave between Zumwalt Ave and Englehart Ave, Phase 2 Expansion (2570')	Phase 2	1,128															
P2RE-9	Zumwalt Ave between Manning Ave and Springfield Ave, Phase 2 Expansion (2700')	Phase 2	1,483															
P2RE-10	Manning Ave between Zumwalt Ave and Englehart Ave, Phase 2 Expansion (2590')	Phase 2	948															
P2RE-11	Zumwalt Ave between Parlier Ave and Manning Ave, Phase 2 Expansion (2630')	Phase 2	963															
P2RE-12	Parlier Ave between Zumwalt Ave and Englehart Ave, Phase 2 Expansion (2590')	Phase 2	948															
P2RE-13	Zumwalt Ave between South Ave and Parlier Ave, Phase 2 Expansion (2680')	Phase 2	785															
P2RE-14	South Ave between Zumwalt Ave and Englehart Ave, Phase 2 Expansion (2620')	Phase 2	959															
P2RE-15	Zumwalt Ave between Sumner Ave and South Ave, Phase 2 Expansion (2530')	Phase 2	741															
P2SM-6	Columbia Ave btn E Manning Ave & E Springfield Ave, Upsized from 18" to 21" (1560')	Phase 2	1,350															
P2SM-7	Manning Ave between Columbia and Buttonwillow, Upsized from 10-18" to 21" (2720')	Phase 2	1,780															
P2SM-8	Columbia Ave South of Parlier Ave, Upsized from 8" to various (430')	Phase 2	106															
P2SM-9	Buttonwillow Ave between Parlier Ave and Manning Ave, Phase 2 Expansion (2680')	Phase 2	1,471															
P2SM-10	Parlier Ave between Buttonwillow Ave and Zumwalt Ave, Phase 2 Expansion (2470')	Phase 2	904															
P2SM-11	Buttonwillow Ave between South Ave and Parlier Ave, Phase 2 Expansion (2680')	Phase 2	1,225															
P2SM-12	South Ave between East Reedley Ditch and Buttonwillow Ave, Phase 2 Expansion (2650')	Phase 2	970															
P2SM-13	Buttonwillow Ave between Sumner Ave and South Ave, Phase 2 Expansion (2530')	Phase 2	926															
P2SM-14	South Ave between Buttonwillow Ave and Zumwalt Ave, Phase 2 Expansion (2490')	Phase 2	911															
P2WR-1	Between Huntsman Ave and Treatment Plant, Phase 2 Expansion (1590')	Phase 2	1,019															
P2WR-2	Kings River Rd between Huntsman Ave and Redwood Dr, Phase 2 Expansion (1300')	Phase 2	1,110															
P2WR-3	West of Kings River Rd parallel to Redwood Dr, Phase 2 Expansion (2630')	Phase 2	1,101															
P2WR-4	Kings River Rd between Redwood Dr and Dinuba Ave, Phase 2 Expansion (2010')	Phase 2	1,724															

Project ID	Sewer System Improvement Project Description	Phase	Project Cost(a)	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2WR-5	Kings River Rd North of Dinuba Ave, Phase 2 Expansion (1700')	Phase 2	1,451															
P2WR-6	Kings River Rd South of Nurmi Ave, Phase 2 Expansion (1780')	Phase 2	1,520															
P2WR-7	Nurmi Ave between Kings River Rd and Manning Ave, Phase 2 Expansion (2060')	Phase 2	1,414															
P2WR-8	Manning Ave between Lac Jac Ave and Numri Ave, Phase 2 Expansion (2930')	Phase 2	1,678															
P2WW-1	West Bank Kings River between Wallace Ave and Treatment Plant, Phase 2 Expansion (3980')	Phase 2	1,165															
P2WF-1	Manning Ave between Kings River and Nurmi Ave, New 8" Force Main (1130')	Phase 2	294															
Subtotal Pipeline Improvements			95,938	510	2,196	1,275	2,875	1,686	2,840	2,511	2,664	1,765	1,900	1,591	804	1,003	2,031	2,186
Total Sanitary Sewer System Improvements			98,136	574	2,196	1,275	2,875	1,686	2,840	2,511	2,664	1,765	1,900	1,591	804	1,003	2,031	2,186

(a) All costs are presented in 1,000s of dollars. All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

Figure 6-3. Recommended Sanitary Sewer System Improvements through Phase 1

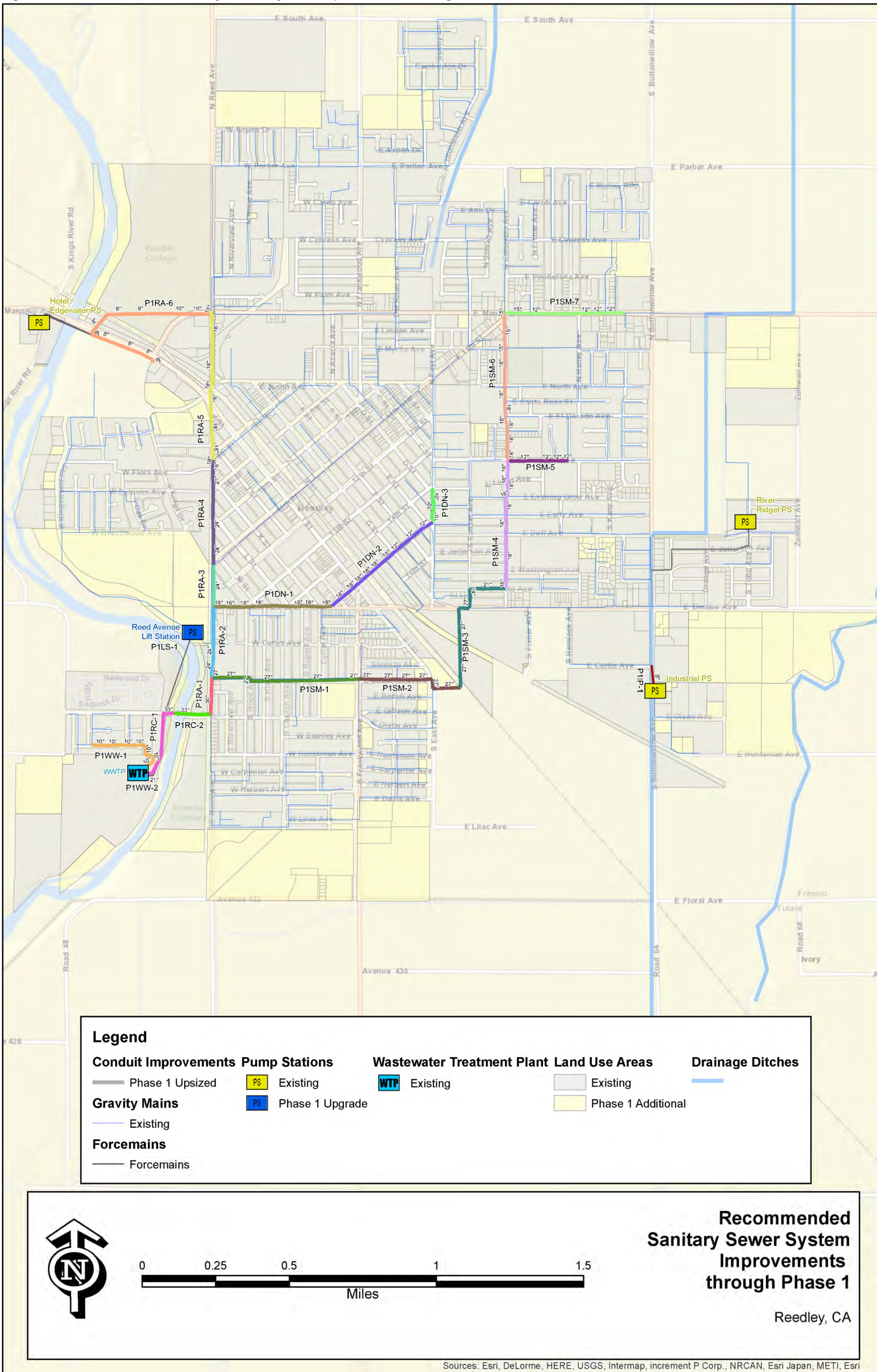


Figure 6-4. Recommended Sanitary Sewer System Improvements for Phase 2

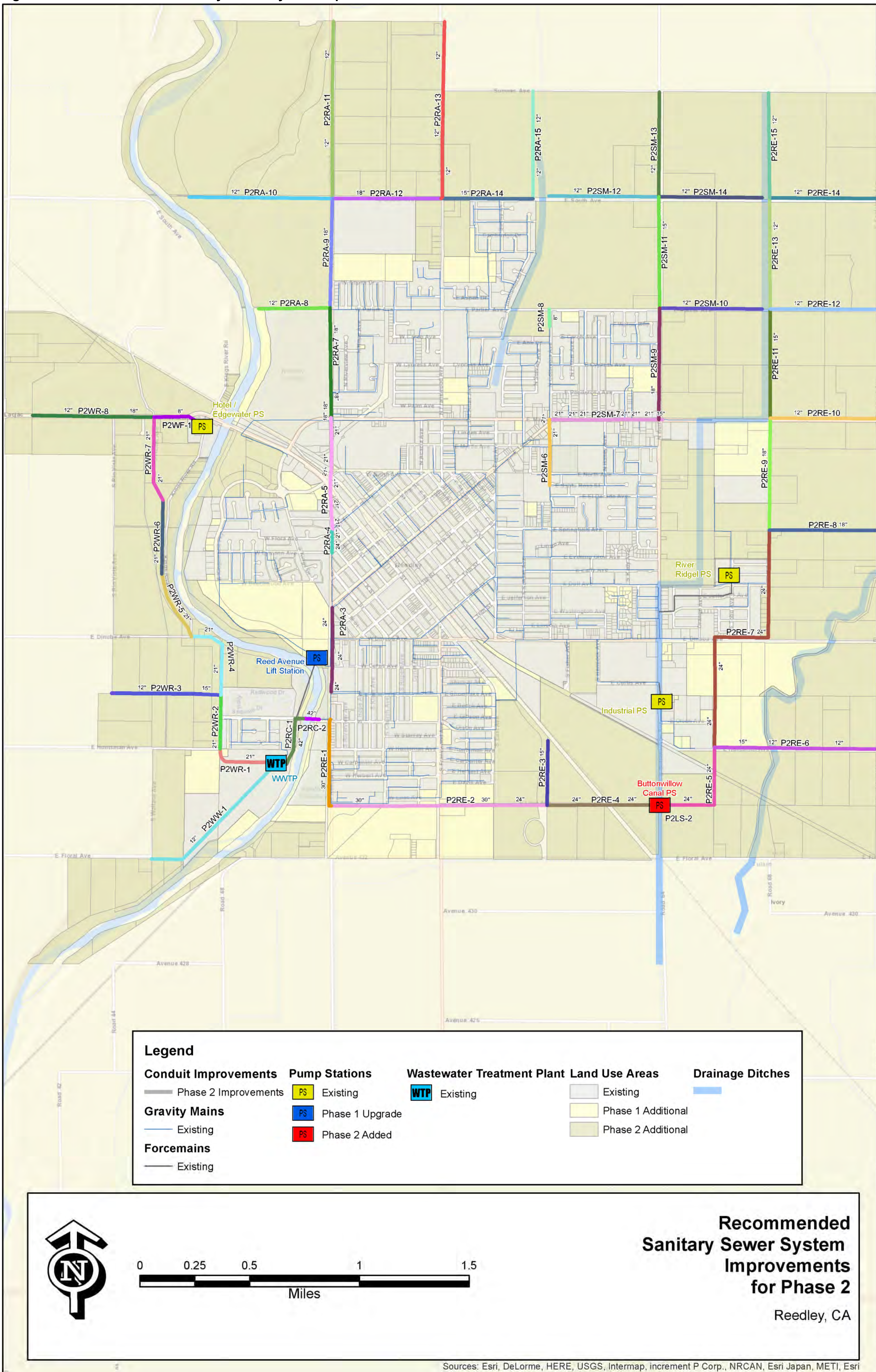


Table 6-3. Recommended Storm Drainage System CIP (1,000s)

Project ID	Storm Drainage System Improvement Project Description	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
Storage Basin Improvements																		
EXL-Basin	Expand Storage in Basin L to 38.2 ac-ft	Existing	1,648				330	1,318										
P1P-Basin	Replace Storage in Basin P, 76 ac-ft	Phase 1	6,328										1,266	5,062				
P1Q-Basin	Replace Storage in Basin Q, 72.7 ac-ft	Phase 1	4,129								826	3,303						
P2N1-Basin	New Storage in Basin 1, 115.9 ac-ft	Phase 2	8,768															
P2N2-Basin	New Storage in Basin 2, 73.1 ac-ft	Phase 2	5,520															
P2N3-Basin	New Storage in Basin 3, 55.3 ac-ft	Phase 2	4,176															
P2N4-Basin	New Storage in Basin 4, 66.2 ac-ft	Phase 2	5,010															
P2N5-Basin	New Storage in Basin 5, 84.9 ac-ft	Phase 2	6,425															
		Subtotal Storage Improvements	42,004				330	1,318			826	3,303	1,266	5,062				
Pipeline Improvements																		
P1B-01	New 18-inch Pipe in W Cypress Ave (660')	Phase 1	483													97	386	
P1B-02	New 18-inch Pipe in W Sycamore Ave (670')	Phase 1	491													98	393	
P1B-03	New 18-inch Pipe in W Ponderosa Ave (690')	Phase 1	505														101	404
P1B-04	New 18-inch Pipe in W Palm Ave (670')	Phase 1	491														98	393
EXC-01	Upsize Pipeline from Outfall to canal (70')	Existing	53							11	42							
EXC-02	Upsize Pipeline in E Myrtle Ave between N Sunset Ave and N Columbia Ave (670')	Existing	708							142	566							
EXC-03	Upsize Pipeline in N Columbia Ave between E Ponderosa Ave to E Cypress Ave (670')	Existing	491							98	393							
EXD-01	Upsize Pipeline in East of railroad (near outfall) (730')	Existing	963	193	770													
EXD-02	Upsize Pipeline in N Reed Ave between W Manning Ave and W Ponderosa Ave (630')	Existing	665	133	532													
EXD-03	Upsize Pipeline in W Ponderosa Ave between N Reed Ave and N Hope Ave (650')	Existing	476		95	381												
EXD-04	Upsize Pipeline in N Reed Ave north of W North Ave (240')	Existing	110		22	88												
EXE-01	Upsize Pipeline in S Reed Ave between W Curtis Ave and W Dinuba Ave (750')	Existing	1,185	237	948													
EXE-02	Upsize Pipeline in W Curtis Ave west of S Riverview Ave to alley (330')	Existing	521	104	417													
EXE-03	Upsize Pipeline in W Dinuba Ave between alley east of Riverview Ave to S Frankwood Ave (2210')	Existing	2,620		524	2,096												
EXE-04	Upsize Pipeline in W Dinuba Ave between S Frankwood Ave and S East Ave (1210')	Existing	1,594			319	1,275											
EXE-05	Upsize Pipeline in S East Ave between W Dinuba Ave to G St (170')	Existing	180			36	144											
EXE-06	Upsize Pipeline in M St between 13th St and 12th St (420')	Existing	886			177	709											
EXE-07	Upsize Pipeline in 12th St between M St and I St (1560')	Existing	3,286				657	2,629										
EXE-08	Upsize Pipeline in I St between 11th St and 9th St (970')	Existing	1,023					205	818									
EXE-09	Upsize Pipeline in I St between 9th St and 8th St (480')	Existing	633					127	506									
EXE-10	Upsize Pipeline in 8th St between I St and alley east of H St (640')	Existing	675					135	540									
EXE-11	Upsize Pipeline in Alley east of H St between 8th St and W North Ave (450')	Existing	475					95	380									

Project ID	Storm Drainage System Improvement Project Description	Phase	Project Cost[a]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2G-01	Upsize Pipeline in Outfall to Kings River (near Edgewater Inn) (190')	Phase 2	226															
P2G-02	Upsize Pipeline in Northwest of N Kings River Rd (near Edgewater Inn) (210')	Phase 2	250															
P2G-03	New Pipe in W Manning Ave between S Nurmi Ave and Edgewater Inn (1120')	Phase 2	2,163															
P2G-04	New Pipe in W Manning Ave between S Lac Jac Ave and Nurmi Ave (3450')	Phase 2	5,921															
P2G-05	New Pipe in S Nurmi Ave south of W Manning Ave (1300')	Phase 2	1,116															
EXH-01	Upsize Pipeline from Outfall to Kings River (680')	Existing	1,074						215	859								
EXH-02	Upsize Pipeline in W Eymann Ave between S Willow Glen Dr and alley west of S Oak Dr (750')	Existing	988						198	790								
EXH-03	Upsize Pipeline in W Eymann Ave between alley west of S Oak Dr and S Kings Dr (860')	Existing	906							181	725							
EXH-04	Upsize Pipeline in W Eymann Ave between S Kings Dr and S Reed Ave (730')	Existing	963							193	770							
EXH-05	Upsize Pipeline in 9th St between S Reed Ave and northeast of J St (720')	Existing	759								152	607						
EXH-06	Upsize Pipeline in 9th St between alley northeast of J St and I St (200')	Existing	211								42	169						
P1H-01	New Pipe in W Beechwood Ave between alley west of S Oak Dr and S Kings Dr (920')	Phase 1	673														135	538
EXJ-01	Upsize Pipeline in E Duff Ave between detention pond and S Hemlock Ave (310')	Existing	368											74	294			
EXJ-02	Upsize Pipeline in E Duff Ave between S Hemlock Ave and S Columbia Ave (990')	Existing	1,304											261	1,043			
EXJ-03	Upsize Pipeline in E Duff Ave between S Columbia Ave and S Sunset Ave (650')	Existing	686											137	549			
EXJ-04	Upsize Pipeline in S Sunset Ave between E Duff Ave and E Myra Ave (490')	Existing	516												103	413		
EXJ-05	Upsize Pipeline in S Sunset Ave between E Myra Ave and north of E August Ave (570')	Existing	418												84	334		
EXJ-06	Upsize Pipeline in S Kady Ave between E Duff Ave and E Early Ave (360')	Existing	264												53	211		
P1K-01	New Pipe in Zumwalt Ave north from Silas Batsch Elementary School to E Manning Ave (1960')	Phase 1	2,209														442	1,767
P2K-01	Upsize Pipeline in S Tobu Ave between Evening Glow Ave and E Springfield Ave (770')	Phase 2	1,621															
P2K-02	Upsize Pipeline in E Springfield Ave from S Tobu Ave east toward Buttonwillow Ditch (360')	Phase 2	759															
P2K-03	Upsize Pipeline in Buttonwillow Ditch between E Springfield to south of Batsch Elementary (670')	Phase 2	1,060															
P2K-04	Upsize Pipeline in South of Silas Batsch Elementary from Buttonwillow Ditch to Zumwalt (1220')	Phase 2	1,686															
P2K-05	New Pipe in East of Zumwalt Ave to S Englehart Ave (2530')	Phase 2	2,443															
P2K-06	New Pipe in E Manning Ave between Zumwalt Ave and S Englehart Ave (2550')	Phase 2	2,873															
EXL-01	Upsize Pipeline in North from retention basins to E Curtis Ave; East along E Curtis (950')	Existing	1,251									250	1,001					
EXL-02	Upsize Pipeline in North from E Curtis Ave (110')	Existing	88									18	70					
EXN-01	Upsize Pipeline in S Church Ave between W Sasaki Ave and W Huntsman Ave (1010')	Existing	1,331						266	1,065								
EXN-02	Upsize Pipeline in W Shoemake Ave between S Church Ave and Crier Ave (660')	Existing	483						97	386								
P1N-01	New Pipe in W Stanley Ave east from S Church Ave (520')	Phase 1	380															
P1N-02	New Pipe in W Huntsman Ave east from S Church Ave (510')	Phase 1	374											76	304			
P2O-01	Upsize Pipeline in Outfall to Kings River along W Olson Ave (880')	Phase 2	1,159											75	299			
P2O-02	Upsize Pipeline in W Olson Ave west of outfall (370')	Phase 2	488															
P2O-03	New Pipe in W Olson west of outfall to S Kings River Rd; North along S Kings River Rd (1360')	Phase 2	2,043															

Project ID	Storm Drainage System Improvement Project Description	Phase	Project Cost[(a)]	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
P2O-04	New Pipe in West of S Kings River Rd (2570')	Phase 2	2,758															
P2O-05	New Pipe in North along S Kings River Rd from new pipe (4810')	Phase 2	5,160															
P1P-01	New Pipe in South along S East Ave between E Davis Ave to Lilac Ave (490')	Phase 1	526												105	421		
P1P-02	New Pipe in S East Ave between Lilac Ave and E Floral Ave (to new retention basin) (1260')	Phase 1	2,163													433	1,730	
P2P-01	New Pipe in Lilac Ave between S East Ave and railroad (2350')	Phase 2	3,026															
P2Q-01	Upsize Pipeline in Between Olson Ave and E Huntsman Ave (660')	Phase 2	1,044															
P2Q-02	New Pipe in South from E Huntsman Ave (2380')	Phase 2	2,298															
P1Q-01	New Pipe in East along E Huntsman Ave from existing pipe (1110')	Phase 1	1,668									334	1,334					
P1Q-02	New Pipe in North from existing retention pond to E Dinuba Ave, west to temporary pond (2760')	Phase 1	2,369									474	1,895					
P2R-01	New Pipe in New pipeline southwest from existing system (3410')	Phase 2	5,120															
P2R-02	New Pipe in New pipeline along E Floral Ave (780')	Phase 2	629															
P2N1-01	New Pipe in S Frankwood Ave from new retention pond south to E South Ave; E South Ave between S Frankwood Ave and S Reed Ave (3650')	Phase 2	6,265															
P2N1-02	New Pipe in South on S Frankwood Ave from E South Ave (1510')	Phase 2	1,296															
P2N1-03	New Pipe in North on S Reed Ave from E South Ave (4080')	Phase 2	7,003															
P2N1-04	New Pipe in West on E South Ave from N Reed Ave (3180')	Phase 2	4,094															
P2N1-05	New Pipe in North from new retention pond on S Frankwood Ave (3920')	Phase 2	4,205															
P2N2-01	New Pipe in West from new retention pond on E South Ave to N Sunny Lane (2910')	Phase 2	4,994															
P2N2-02	New Pipe in North from E South Ave (2500')	Phase 2	3,218															
P2N2-03	New Pipe in East from new retention pond on E South Ave to S Buttonwillow Ave (1060')	Phase 2	1,365															
P2N2-04	New Pipe in East on E South Ave from S Buttonwillow Ave (2450')	Phase 2	1,971															
P2N2-05	New Pipe in North on S Buttonwillow Ave from E South Ave (2500')	Phase 2	2,414															
P2N3-01	New Pipe in North from new retention pond along Buttonwillow Ditch to E South Ave (2500')	Phase 2	3,218															
P2N3-02	New Pipe in E South Ave between Buttonwillow Ditch to S Englehart Ave (2320')	Phase 2	2,490															
P2N3-03	New Pipe in E Parlier Ave from new retention pond to S Englehart Ave (2250')	Phase 2	2,414															
P2N4-01	New Pipe in West from new retention pond along E Cypress Ave (1130')	Phase 2	1,455															
P2N4-02	New Pipe in West from new retention pond along E Cypress Ave (1140')	Phase 2	1,224															
P2N4-03	New Pipe in South from new pipe to E Manning Ave (1300')	Phase 2	1,394															
P2N4-04	New Pipe in North from E Cypress Ave (3290')	Phase 2	2,824															
P2N5-01	New Pipe in E Huntsman Ave east of new detention pond (1740')	Phase 2	2,241															
	Subtotal Pipeline Improvements		138,414	667	3,308	3,097	2,785	3,191	3,020	3,725	2,690	1,852	4,300	623	2,834	2,007	3,285	3,102
	Total Storm Drainage System Improvements		180,418	667	3,308	3,097	3,115	4,509	3,020	3,725	3,516	5,155	5,566	5,685	2,834	2,007	3,285	3,102

(a) All costs are presented in 1,000s of dollars. All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

6.4. Storm Drainage System CIP

Table 6-3 presents the City's schedule of storm drainage system CIP projects planned for implementation between 2015 and 2030, as well as the estimated project cost for those projects recommended to accommodate Phase 2 development. Figure 6-5 provides the locations for the major planned improvements through Phase 1, while the recommended improvements for Phase 2 are illustrated in Figure 6-6.

As Table 6-3 indicates, the storm drainage system CIP includes approximately $52.6 million for storage basin and pipeline improvements to correct existing system deficiencies and accommodate Phase 1 growth. An additional $128 million is needed to expand the system to accommodate Phase 2 growth in the storm drainage system.

Refer to Chapter 5 for a description of the recommended storm drainage system projects.

6.5. Recommended CIP

Table 6-4 presents a summary of the costs for the recommended projects for the potable water, sanitary sewer, and storm drainage system components. All costs are presented in 2014 dollars. As summarized, the potable water system improvements total approximately $92.7 million and the sanitary sewer improvements total approximately $98.1 million, while the recommended storm drainage improvements total approximately $180.4 million, nearly double that of the water and sewer systems.

In addition, Table 6-5 presents a summary of the projected annual cash flow for the 15-year CIP. As summarized, the annual cash flow ranges from a minimum of $2.2 million in 2015 to a high of over $8 million, with an average annual projected cash flow of approximately $6.7 million.

Figure 6-5. Recommended Storm Drainage System Improvements through Phase 1

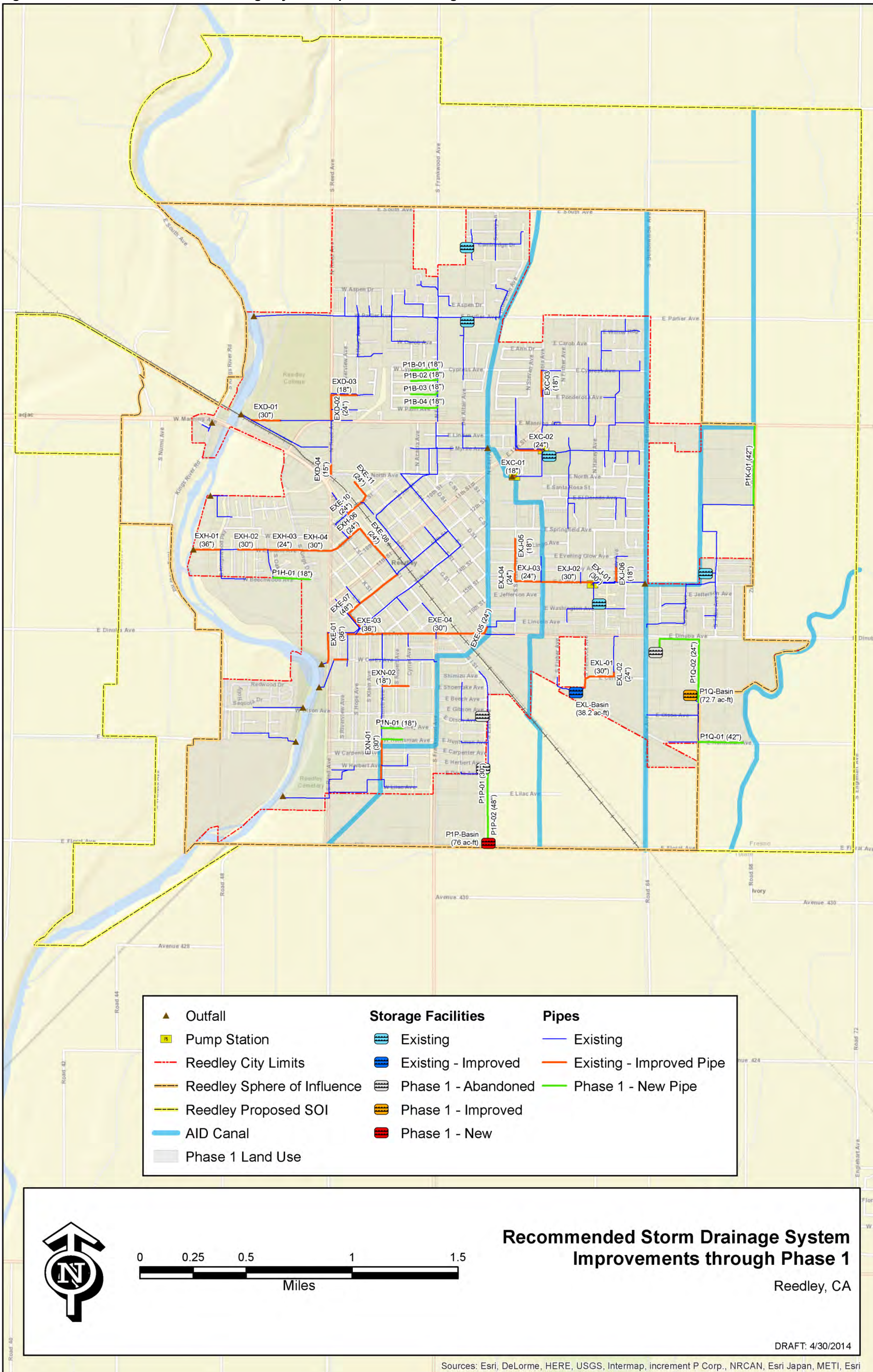


Figure 6-6. Recommended Storm Drainage System Improvements for Phase 2

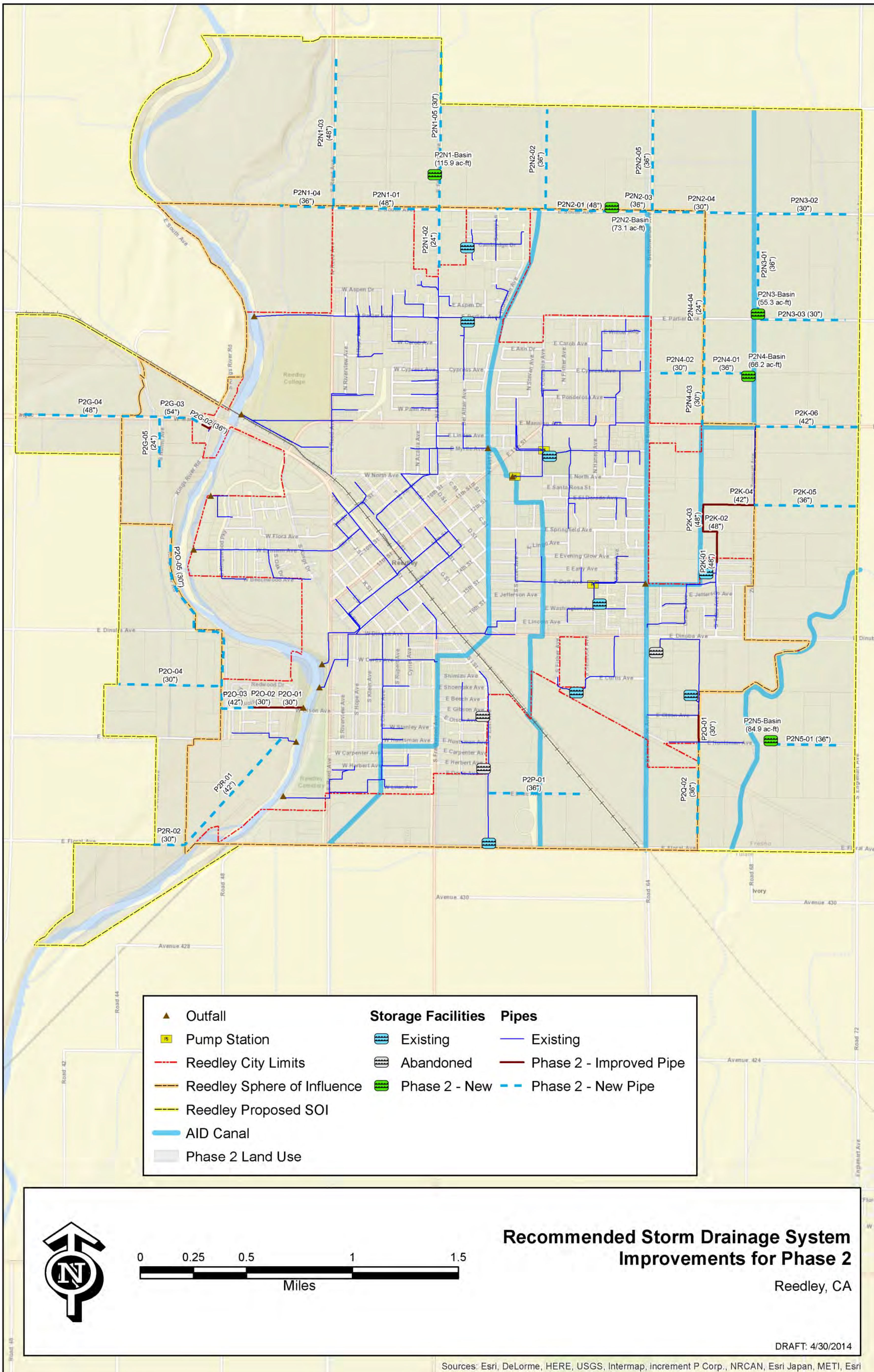


Table 6-4. Summary of Recommended Combined CIP

System(a)	Phase			Total
	Existing	Phase 1	Phase 2	
Potable Water				
Wells	39,000	2,555,000	22,995,000	25,589,000
Storage Tanks	5,510,000	4,057,000	13,538,000	23,105,000
Pipelines	6,948,000	1,431,000	35,674,000	44,053,000
Subtotal	12,497,000	8,043,000	72,207,000	92,747,000
Sanitary Sewer				
Lift Stations	64,000	0	2,134,000	2,198,000
Pipelines	21,923,000	5,914,000	68,101,000	95,938,000
Subtotal	21,987,000	5,914,000	70,235,000	98,136,000
Storm Drainage				
Basins	1,648,000	10,457,000	29,899,000	42,004,000
Pipelines	28,154,000	12,332,000	97,928,000	138,414,000
Subtotal	29,802,000	22,789,000	127,827,000	180,418,000
Total	64,286,000	36,746,000	270,269,000	371,301,000

(a) All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

Table 6-5. Summary of Annual Cash Flow for the 15-Year Combined CIP

Year	System(a)			Total
	Potable Water	Sanitary Sewer	Storm Drainage	
2015	985,000	574,000	667,000	2,226,000
2016	2,156,000	2,196,000	3,308,000	7,660,000
2017	1,144,000	1,275,000	3,097,000	5,516,000
2018	1,413,000	2,875,000	3,115,000	7,403,000
2019	2,217,000	1,686,000	4,509,000	8,412,000
2020	2,486,000	2,840,000	3,020,000	8,346,000
2021	1,427,000	2,511,000	3,725,000	7,663,000
2022	2,277,000	2,664,000	3,516,000	8,457,000
2023	996,000	1,765,000	5,155,000	7,916,000
2024	435,000	1,900,000	5,566,000	7,901,000
2025	1,523,000	1,591,000	5,685,000	8,799,000
2026	3,272,000	804,000	2,834,000	6,910,000
2027	165,000	1,003,000	2,007,000	3,175,000
2028	9,000	2,031,000	3,285,000	5,325,000
2029	35,000	2,186,000	3,102,000	5,323,000
Average	1,369,000	1,860,000	3,506,000	6,735,000

(a) All costs include 30% contingency, 25% EAP, and are presented in 2014 dollars, referenced to the ENR CCI for February 2014, 9681.

6.6. Next Steps

The following subsections describe the next steps in implementing the Master Plan recommendations, including engineering, environmental compliance and permitting, coordination with ongoing projects and programs, financing, and continued use of Master Plan tools.

6.6.1. Engineering

The technical work completed for this Master Plan provides a framework for the recommended improvements to the potable water, sanitary sewer, and storm drainage facilities previously described in this chapter. The preliminary locations of these new facilities are shown in Figures 6-1 through 6-6. These locations and pipeline alignments are preliminary and final locations should be determined during predesign work.

The purpose of the predesign studies is to finalize locations and alignments, refine design criteria and sizing, identify land requirements, evaluate operational requirements, and update cost estimates. Following completion of predesign studies additional engineering will include design, construction management, testing and startup.

Many of the proposed improvements will be phased and the engineering work should be scheduled accordingly. Construction contract packaging should be evaluated to provide the greatest opportunities for competitive bidding by contractors.

In addition, there are some common corridors in which water, sewer, and/or storm drainage pipeline projects are needed. For example, Project P1DN-1 for the sanitary sewer system is in the same alignment in Dinuba Avenue as project EXE-03 for the storm drainage system. Where appropriate, and as financially feasible, such projects in common corridors should be designed and packaged together to provide greater economies of scale. In addition, opportunities to leverage other capital improvement programs such as pavement renewal projects or parks improvements should be coordinated with the recommendations in this Master Plan to take advantage of economies of scale and minimize construction activities.

6.6.2. Environmental Compliance and Permitting

The recommended facilities will require compliance with the California Environmental Quality Act (CEQA) and possibly the National Environmental Policy Act (NEPA) to evaluate the environmental impacts of the projects. The required environmental compliance documents should be completed in conjunction with the engineering preliminary design studies.

Numerous federal, state and local permits will also be required for project implementation. The required permits will be identified during the preparation of the engineering preliminary design studies and environmental compliance documents. A permitting strategy should be developed to minimize project delays and potential mitigation costs.

6.6.3. Coordination with Ongoing Projects and Programs

Implementation of the Master Plan should be coordinated with other ongoing projects and programs. Specifically, the Master Plan should be coordinated with the following:

- Water Conservation Program
- Asset Renewal and Replacement
- Kings Groundwater Basin Management
- Sewer System Management Plan under the State's General WDR Permit
- Storm Water Management Plan under the State's General MS4 Permit

6.6.4. Financing

The estimated capital costs by phase were summarized in Table 6-4. All costs are presented in 2014 dollars.

The recommended facilities should be incorporated into the City's five-year capital improvement program in accordance with the proposed phasing plan. Specific project financing, including escalation, can then be addressed as part of the City's regular budgeting, rates, and facility capacity/connection fee program updates.

Project costs associated with the expansion of the existing systems to accommodate future growth, particularly for Phase 2, should be included in the City's facility capacity/connection fee such that future growth pays for the respective facilities they need.

6.6.5. Use of Master Plan Tools

The City has invested substantial resources in the completion of this Master Plan. The tools developed as part of this work should be utilized in the future evaluation of proposed new developments, proposed land use changes, refinements to the recommended facilities, and potential regional projects and programs. Some of the tools to be utilized by the City include the following:

- Planning criteria established for evaluation of facilities
- Potable water distribution system hydraulic model
- Sanitary sewer collection system hydraulic model
- Storm drainage collection system hydraulic model

6.6.6. Future Updates

The recommendations presented in this Master Plan include infrastructure upgrades to improve the existing system as well as to accommodate future growth as envisioned in the City's General Plan 2030. The recommendations represent a substantial CIP, particularly to accommodate the growth anticipated in Phase 2. As such, the City should regularly evaluate actual system conditions, including the number of new connections per year, conservation savings, development of recycled water, and other changes that may impact the growth in the potable water demand, and the generation of sanitary sewer flows and storm water runoff. Based on these regular updates, the annual CIP should be adjusted as needed. The City should also prepare a formal update to this Master Plan in approximately five years.