FINAL REPORT

SEWER SYSTEM MASTER PLAN UPDATE

TOWN OF APPLE VALLEY, CALIFORNIA

Prepared for

Town of Apple Valley, California

Project No.: 38001723

August 2013

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AC	Acre
AD	Assessment District
ADF	Average Daily Flow
ADWF	Average Dry Weather Flow
AV	Apple Valley
AVWD	Apple Valley Water District
BO	Build out
CCTV	Closed Circuit Television
CIP	Capital Improvement Project
C-G	General Commercial
C-R	Regional Commercial
C-S	Service Commercial
d/D	Depth over Diameter
DI	Ductile Iron
DWF	Dry Weather Flow
DU	Dwelling unit
EDU	Equivalent Dwelling Unit
FT	Feet
FPS	Feet per Second
GIS	Geographic Information Systems
GPD	Gallons per Day
GPM	Gallons per Minute
GWDR	General Waste Discharge Requirements
HDR	High Density Residential
HGL	Hydraulic Grade Line
in	Inch
I/I	Infiltration and Inflow
I-P	Planned Industrial
JR	Jess Ranch
LCER	Lewis Center for Educational Research
LDR	Low Density Residential
LF	Linear Foot
LS	Lift Station
MDR	Medium Density Residential
MG	Million Gallons

mgd	Million Gallons per Day
MH	Manhole
MU	Mixed Use
n	Manning's Roughness Coefficient
NAD	North American Datum
NAP	Not-A-Part
NASSCO	National Association of Sewer Service Companies
NAVI	North Apple Valley Interceptor
N/A	Not Applicable
OS	Open Space
O&M	Operations and Maintenance
O-P	Office Professional
OS-C	Open Space - Conservation
PACP	Pipeline Assessment and Certification Program
PDF	Peak Dry Flow
PFU	Plumbing Fixture Unit
PUD	Planned Unit Development
PVC	Polyvinyl Chloride Pipe
PWF	Peak Wet Weather Flow
R-A	Residential Agriculture
R-E	Estate Residential
R-EQ	Equestrian Residential
R-LD	Low Density Residential
R-M	Multi-Family Residential
R-SF	Single Family Residential
R-VLD	Very Low Density Residential
SCAG	Southern California Association of Governments
Sq.	Square
SSO	Sanitary Sewer Overflows
URS	URS Corporation
USGS	United States Geological Survey
VCP	Vitrified Clay Pipe
VVWRA	Victor Valley Wastewater Reclamation Authority
WWF	Wet Weather Flow

1.0 EXECUTIVE SUMMARY

The last Sewer Master Plan for the Town of Apple Valley was developed in 1993 as an accompanying document to the Town's General Plan. URS Corporation (URS) was retained by the Town to develop a Sewer Master Plan Update that is comprehensive in nature and includes the development of recommendations for all aspects of the sewer system, and that addresses the short- and long-term planning operations.

1.1 STUDY OBJECTIVES

The Sewer Master Plan prepared by URS serves as a reference document for the Town of Apple Valley for the existing wastewater system, and outlines necessary system upgrades for future wastewater system planning. The Master Plan provides a plan and schedule to properly manage, operate, and maintain all parts of the sanitary sewer system until buildout and includes the following:

- Development of a Geographic Information Systems (GIS) database of the existing Sewer System Network.
- Development and calibration of sewer system hydraulic model.
- Updated flow projections based on population growth and land use data for buildout.
- Analysis of the hydraulic capacities of the Town's existing Sewer System and identify system deficiencies.
- Condition assessment of the existing collection sewer system for segments inspected by Town staff.
- Evaluation of existing and future system deficiencies and the development of a prioritized list of Capital Improvement Projects (CIPs) and a CIP implementation schedule to address the deficiencies identified.
- Establish and summarize required CIPs for the Town's future Sewer System based on projected flows determined using anticipated growth and land use patterns.

1.2 BACKGROUND

According to the 2010 U.S. Census, Apple Valley covers an area of 73.2 square miles, and has a population of 69,139 people. Assuming growth occurred linearly between 1990 and 1993, the population at the time of the last Master Plan in 1993 was approximately 48,527 people. In the twenty years since the last Master Plan was prepared, the Town has grown by approximately 46 percent, and nearly 266 percent since 1980.

1.3 GIS DATABASE DEVELOPMENT

Prior to development of this Master Plan Update, no digital database of the Town's existing sewer system database existed. URS developed a GIS database for organizing Apple Valley's sanitary sewer system manhole and pipe data. The GIS database was developed using as-built drawings of the Town's sewer system. Until now, the bulk of the Town's sanitary sewer system network has been maintained utilizing printed maps and as-built copies of plans. The newly created GIS database will assist the Town in managing its aging infrastructure, as well as allow flexibility for future growth.

In creating the GIS database, each manhole was identified by a unique manhole number, which contains Assessment District, Tract Number, and Manhole Number (e.g., 2B-16799-015). The naming convention was developed jointly with the Town of Apple Valley staff.

The major objective of the naming convention is for ease of location of the manholes in the Town's sewer database for activities such as cleaning and video inspections.

1.4 HYDRAULIC MODEL DEVELOPMENT

As part of this Master Plan, an InfoWorks hydraulic model was developed to model the sewer system and to assist with the hydraulic evaluation. InfoWorks is a sophisticated hydraulic modeling software used around the world for modeling sewer systems. Originally developed in the United Kingdom, the software was recently acquired by Innovyze who is currently responsible for the licensing and technical support of the software.

The model developed by URS consisted over 3,500 sewer manholes and pipes 8 inches in diameter and larger. Smaller 6 inch diameter pipes were included where necessary if they are located in-between 8 inch diameter pipelines.

The hydraulic model was calibrated with flow monitoring data provided by VVWRA conducted between September 8 and September 24, 2012. Based on the calibration results, URS concluded that the model results fairly match the flow monitoring data, thus establishing confidence in the hydraulic model.

1.5 LAND USE AND FLOW PROJECTIONS

Population growth is critical to the planning of expansion of the Town's sewer system. Many issues impact the existing sewer system, including exceedance of capacity of existing facilities, funding, and aging infrastructure. The challenge facing the Town is knowing when and what to upgrade to maintain a viable and properly functioning sewer system.

URS utilized the existing Land Use Map developed for the Town of Apple Valley General Plan to determine commercial, industrial and residential flows. Historic land use of two houses per acre is consistent with the average residential development throughout the majority of the Town of Apple Valley. Previous Sewer Master Plans have indicated future development could deviate from the two household per acre norm. The current Master Plan considers the undeveloped areas of the Town according to the land use designation outlined in the 2009 General Plan.

1.6 DEVELOPMENT OF CAPITAL IMPROVEMENT PROJECTS

The computer model developed for the Town's sewer system was used to evaluate the hydraulic conditions of the system under existing and future buildout conditions. The model provided a powerful tool to analyze what if scenarios. Based on the InfoWorks computer modeling, existing and proposed sewer lines were developed to relieve existing capacity deficiencies and to provide adequate pipe capacity for future buildout to accommodate the expansion of growing areas. The following provides a summary of the results found for existing and buildout conditions.

1.6.1 Existing Conditions

Simulations of the existing system conditions of the Town's sewer system were performed using the InfoWorks model. The model simulations identified a system which is adequate, with no observed deficiencies.

1.6.1.1 Hydraulic Evaluation

The following statements summarize the findings of the hydraulic evaluation of the existing system:

- The existing system appears to have adequate capacity to convey flows under average dry weather conditions.
- The existing pump stations appear to have adequate pumping capacity.
- The system does not appear to have been heavily influenced by ingress of infiltration and inflow into the sewer system.

1.6.1.2 Condition Assessment of Existing Pipeline Segments

Overall, the pipes appear to be in good condition with the exception of certain segments assessed in the CCTV evaluation. The following locations appear to need immediate attention due to the likeliness of collapse in the foreseeable future.

- 18 North / Tao Road: Between upstream manhole 1C-3382-009 and downstream manhole 1C-3382-008.
- Rancherias / Ottawa Road: Between upstream manhole 2A-000-205 and downstream manhole 2A-000-204.

Table 1-1 provides a summary of the condition assessment performed on the CCTV data provided by the Town staff.

Table 1-1: CCTV Pipe Segment Evaluation

No.	Location	Segment	Defect Summary
1	St. Mary's Hospital / 18 North	Upstream: 1B-000-003 Downstream: 1B-000-002	• Structure defects • Sags • Blisters within the pipe lining • Grade 3
2	St. Mary's Hospital / 18 North	Upstream: 1B-000-004 Downstream: 1B-000-003	• Sag • Grade 3
3	18 North / Tao Road	Upstream: 1C-3382-009 Downstream: 1C-3382-008	• Sag • Crack • Grade 4, due to possibility of collapse
4	Wintun / Corwin	Upstream: 1C-5770-056 Downstream: 1C-008-022	• Sag • Grade 3
5	Rancherias / Ottawa Road	Upstream: 2A-000-205 Downstream: 2A-000-204	• Sag • Root Intrusion • Grade 4, due to possibility of collapse
6	Potomac Road / Quantico Road	Upstream: TOAV-AVI-4494-001 Downstream: VVWRA-AVI-063	• Sag • Spalling • Grade 3
7	American Security Bank	Upstream: JR-14310-130 Downstream: JR-MSP1-007	• Sag • Joint defects • Grade 3
8	Jess Ranch Parkway / Jess Ranch Place	Upstream: JR-14310-359 Downstream: JR-MSP1-013	• Severe Sag • Collection of debris • Grade 3
9	American Security Bank / Jess Ranch	Upstream: JR-14310-359 Downstream: JR-14310-359A	• Video submerged therefore no assessment available
10	Jess Ranch Parkway / Jess Ranch Place	Upstream: JR-14310-131 Downstream: 14310-130	• Sag • Grade 3

1.6.2 Future Buildout Conditions

The future buildout sewer system evaluation was based on the assumption that the Town will develop per land use projections defined by the Town's 2009 General Plan and Specific Plans. A model was created to represent the future buildout scenario. The projected sewer system at buildout was determined by identifying all residential, commercial, and industrial vacant and undeveloped lots and applying flow factors based on land use designation per Town's planning criteria.

The following statements summarize the findings of the hydraulic evaluation of the future system:

- The existing system piping has inadequate capacity to convey future buildout flows. Upgrades will be required to accommodate the future projected wastewater flows.

- The existing pump stations appear to have inadequate pumping capacity to convey future flows. Upgrades to the pump stations will be necessary to convey future projected wastewater flows.

- Inadequate capacity is projected in the VVWRA interceptor, AD 1, Jess Ranch, AD 3A and surrounding areas, and pipelines near LS AD 2B and LS AD 2A No. 2.

Table 1-2 provides a summary of the projected deficiencies within each of the locations.

Table 1-2: Summary of Deficiencies Found for Future Buildout Conditions

Location	Summary of Deficiencies	Proposed Recommendations	Cost Estimates
VVWRA Interceptor	The model predicted an overloading of the VVWRA interceptor during future buildout. SSOs are predicted during future buildout dry weather conditions.	The entire interceptor will require upgrading to a larger diameter to handle the projected buildout flows.	Upgrading of this pipeline is the responsibility of VVWRA; therefore, no costs are included in this report.
AD 1 and Surrounding Areas	Seven pipe segments are predicted to surcharge during future buildout scenario as a result of a nearby infill development.	Upgrade pipe sizes.	$260,000
AD 2A and Surrounding Areas	Pipeline segments and Lift Station 2A No. 2 are predicted to have inadequate capacity at buildout. The lift station surcharging is a result of inadequate pumping capacity during buildout flow conditions.	Upgrading of Lift Station 2A No. 2 and pipeline segments, along with construction of Lift Station Tr. 17247 will be necessary to accommodate the future buildout flows.	$18.0 million
AD 2B and Surrounding Areas	Pipeline segments and Lift Station 2B are predicted to have inadequate capacity at buildout. The lift station surcharging is a result of inadequate pumping capacity during buildout flow conditions.	Upgrading of Lift Station 2B and pipeline segments will be necessary to accommodate the future buildout flows.	$27.4 million
Jess Ranch, AD 3A, and Surrounding Areas	Pipeline segment upstream of Lift Station JR No. 1 in Jess Ranch Parkway is predicted to surcharge during future buildout due force main inadequate capacity.	Construction of Lift Station Tr. 17093 and upgrading of force main will be necessary to accommodate the future buildout flows.	$6.3 million
Northern Areas of Town	Flows from the Golden Triangle and the surrounding area near the airport were directed to this interceptor sewer during future buildout conditions. The hydraulic model indicated that the interceptor sewer has adequate capacity to receive	Construction of new pipelines is needed to provide services to areas that are expected to grow.	$31.9 million

Location	Summary of Deficiencies	Proposed Recommendations	Cost Estimates
	additional flows during future buildout in addition to the Golden Triangle and the Airport area flows.		
Total			**$83.9 million**

The overall cost of CIPs presented in this Master Plan is approximately $83.9 million.

1.7 Conclusions And Recommendations

The calibrated hydraulic model was used for evaluating the hydraulic deficiencies of the existing and the future systems. The existing system model was updated with future land use information and used to evaluate the system under future buildout conditions. The system appears to suffer from considerable hydraulic deficiency during the buildout conditions and warrants a need for a massive Capital Improvement Project to accommodate the anticipated land use classification outlined in the 2009 General Plan.

1.7.1 Conclusions

The existing sewer system appears to have adequate capacity to covey flows during dry weather conditions. Currently, only about 30 percent of the Town's development is connected to the sewer system. Sewer flows are projected to greatly increase during buildout conditions, thereby increasing sewer collection infrastructure requirements. The increase in flows is projected to cause tremendous stress on the existing pipe network, resulting in system hydraulic deficiencies. The hydraulic model results showed that the most critical deficiencies occur in the VVWRA interceptor. Under the future buildout scenario, surcharging is predicted for the majority of the sewers including the VVWRA interceptor, AD 1, Jess Ranch, AD 3A, and surrounding areas. On the VVWRA interceptor, SSOs are predicted during future buildout conditions. The North Apple Valley interceptor is predicted to have adequate capacity under buildout flow conditions.

Recommended CIPs were developed using the hydraulic model to simulate the replacement pipelines needed to accommodate the future flows. Although, the upgrades were simulated as replacement, URS believes that it would be best if parallel pipelines are constructed in order to continue to provide service when the construction is in progress. The parallel lines will provide a way to maintain sewer service while improvements are made. Proposed CIPs for future needs were also established to expand the sewer collection system to areas that are currently not covered, but are expected to require service. Based on pipeline replacements and construction, the overall cost of CIPs presented in this Master Plan is approximately $83.9 million.

1.7.1.1 Existing System

The following statements summarize the findings of the hydraulic evaluation of the existing system:

- The existing system appears to have adequate capacity to convey flows under average dry weather conditions.
- The existing pump stations appear to have adequate pumping capacity.
- The system does not appear to have been heavily influenced by ingress of infiltration and inflow into the sewer system during rain events.

1.7.1.2 **Future Buildout System**

The future buildout system appears to have inadequate capacity to convey flows. The deficiencies identified are the result of the new development that is projected to connect to the sewer collection system. It should be noted that the 1993 Sewer Master Plan directed flows to three (3) proposed sub-regional plant sites to be located in Jess Ranch, near AD 2A, and in the Northwestern part of Apple Valley, near Interstate 15. The Updated Master Plan prepared by URS has directed some flows away from the water reclamation plants given that these may not materialize. The direction of flows to the VVWRA sewer has resulted in overloading of the VVWRA interceptor during buildout. This result points to the fact that the VVWRA sewer may require upgrading or the construction of one of the proposed Water Reclamation Plant near Dale Evans Parkway and Otoe Road is necessary to alleviate the projected overloading of the VVWRA interceptor. The analysis also shows that the Jess Ranch sewer becomes overloaded with the proposed developments to the south of Jess Ranch.

The following provides a summary of the deficiencies within each of the locations during future buildout conditions.

- ***VVWRA Interceptor—***The VVWRA pipeline is predicted to surcharge during future buildout conditions, and SSOs are predicted along this pipeline segment. This pipeline should be upsized to accommodate the future buildout flows. The proposed upgraded pipe sizes are listed in Appendix A. In lieu of the upgrading the VVWRA pipeline segment, the proposed water reclamation plant near Dale Evans Parkway and Otoe Road should be considered. Also, the VVWRA Nanticoke AD 2 Lift Station requires upgrading at future buildout. The VVWRA pipeline segment and its Nanticoke AD 2 Lift Station are operated and maintained by VVWRA. The responsibility to upgrade this pipeline segment and its lift station is that of VVWRA, not the Town of Apple Valley.
- ***Jess Ranch, AD 3A, and Surrounding Areas—***At buildout, Lift Station JR No. 1 has inadequate force main capacity resulting in backing up flows. This force main requires upgrading at buildout to alleviate the surcharging predicted in Jess Ranch Parkway.
- ***AD 2B and Surrounding Areas—***At buildout Lift Station 2B has inadequate capacity resulting in backing up of flows to its upstream pipe segments. This lift station will require upgrade at buildout.
- ***AD 2A and Surrounding Areas —***All three pipeline segments leading to the Lift Station 2A No. 2 are predicted to surcharge due to the lift station's inadequate capacity during

future buildout. To alleviate the surcharging, Lift Station 2A should be upgraded at buildout.

1.7.2 Recommendations

Based on the analysis presented in this report, URS recommends the following:

- ***Existing Sewer System—***The existing sewer system has adequate hydraulic capacity to convey flows under existing conditions. However, the Town is advised to consider the following:
  - *Inflow Control*— The Town has a number of manhole covers that are perforated. These covers allow inflow to enter the sewer system during heavy rainfall events when ponding occurs over these manholes. Although, inflow was not observed as a problem during the analysis, URS recommends the replacement of these manhole covers or plugging the perforations. The recommended repairs involve providing water resistant lids and frames or raising the frames. The lack of evidence of inflow into the system during the analysis is attributable to the lack of adequate rainfall data concurrent with the flow monitoring data to conclusively confirm that inflow not being a problem in the system.
  - *CCTV Inspections*— The Town has been diligent in performing CCTV inspections of historic problem areas. However, consideration should be given to performing additional CCTV inspections on the remainder of the system to determine the condition of the older sewer systems. Also, it appears the Town has gathered considerable CCTV data over the years and the data should be analyzed to determine the condition of the pipes and develop a proactive pipeline renewal and replacement plan. The Town has inspected approximately 35 percent of the sewer system since September 2009.
  - *Flow and Rainfall Monitoring*—The flow and rainfall monitoring used for the analysis was provided by VVWRA and was performed during the dry weather season. The typical approach to performing a sewer master plan is to monitor flows and rainfall during both dry and wet weather conditions to ascertain the system response to rainfall. URS performed a limited assessment of the system response to rainfall using rain gauges provided by agencies within the Town. However, a more comprehensive infiltration and inflow assessment is recommended to be performed by the Town using a flow and rainfall monitoring data performed during the wet-weather season.
- ***Future Buildout System—***The hydraulic evaluation predicts a massive deficiency in the Town's sewer network to accommodate flows during future buildout conditions. Based on the General Plan, all new developments are projected to connect to the sewer collection system. To accommodate future growth, URS has developed a list of recommended pipes to provide services to areas that are expected to grow. Also, to address the deficiencies identified in the existing interceptor sewers, URS modeled the pipeline infrastructure required. The total cost of the infrastructure required to

accommodate future buildout flows, including the upgrading of existing infrastructure and the provision of new pipe segments to serve new development, is estimated at approximately $83.9 million. Planning and design criteria for the recommended capital improvements projects in this Master Plan were developed at a preliminary basis. URS recommends that the Town conduct site specific assessments and refine budget-level costs as needed as more refined data becomes available, from Specific Plans.

- ***Hydraulic Modeling Updates***—The hydraulic modeling was conducted to assess the sewer system requirements for existing and future buildout conditions. URS recommends that the Town maintain the existing sewer network database with up-to-date information (e.g., addition of new pipelines, manholes, force mains, pump stations, etc.). The General Plan and the Specific Plans will be updated periodically as new and refined data becomes available. URS recommends that the Town updates the Sewer Master Plan and the Hydraulic Model at approximately 5 year intervals or as a result of a major proposed development which will have significant impact on the system hydraulics. Also, should the proposed water reclamation plant near Dale Evans Parkway and Otoe Road be considered for construction, the hydraulic evaluation presented in this report will need to be re-evaluated. The current assumption in this Master Plan Update is that the water reclamation plant will not be constructed and therefore flows were directed to the North Apple Valley and VVWRA interceptors. Should the Town desire to construct the water reclamation plant, an immediate re-evaluation of the recommendations presented in this report regarding sewer infrastructure to accommodate future buildout should be performed.

2.0 INTRODUCTION

The Town of Apple Valley lies at the southern edge of the Mojave Desert in the County of San Bernardino. It is located approximately 90 miles northeast of the City of Los Angeles, 50 miles north of San Bernardino, and 40 miles south of Barstow. Neighboring cities include Hesperia to the southwest and Victorville to the west. The location map is shown in Figure 2-1.

According to the late Mary Hampton, local historian, the name Apple Valley came from the lavishness of apple orchards the town had in the 1920s. When the Great Depression came, the cost of irrigation water was too great, and the orchards subsequently died off during the 1930s. The modern founders of Apple Valley were Newton T Bass and B.J. “Bud” Westlund. They were partners in the oil and gas industry in Long Beach, California, and together they marketed Apple Valley as a residential community and destination resort. Apple Valley has 350 days of sunshine a year. The town has two high schools, eight elementary school and three K-8 academies. The median family income is about $47,000. State Route 18, or Happy Trails Highway, is the main road for the Town.

According to the 2010 U.S. Census, Apple Valley has an area of 73.2 square miles, and a population of 69,139 people. Assuming linear growth between 1990 and 1993, the population at the time of the last Master Plan in 1993 was approximately 48,527 people. In the twenty years since the last Master Plan, the town has grown by 46 percent, and nearly 266 percent since 1980.

Groundwater is the main source of potable water for the Town. The Apple Valley Water District (AVWD) was created in 1975 to provide residents and commercial users with a mechanism to collect, treat, and dispose of wastewater. AVWD entered into an agreement with Victor Valley Water Authority on December 13, 1977. This agreement resulted in entrusting the collection and treatment of wastewater from the Town to Victor Valley Wastewater Reclamation Authority (VVWRA). The Town is tasked with feasibility studies and analysis to optimize service by gravity sewer services, lift stations, and related facilities. The Town’s trunk sewers convey wastewater flows to the sewer interceptor lines for the VVWRA. VVWRA serves the High Desert municipalities, including Apple Valley, Victorville, Hesperia, Adelanto, Oro Grande, Spring Valley Lake, and George Air Force Base. As the Town of Apple Valley continues to grow, its sewer system will require both upkeep and expansion.

2.1 MASTER PLAN OBJECTIVES

The Sewer Master Plan Update serves as a reference document for the Town of Apple Valley for the existing wastewater system, and outlines necessary system upgrades for future wastewater system planning. The Master Plan provides a plan and schedule to properly manage, operate, and maintain all parts of the sanitary sewer system until buildout and includes the following:

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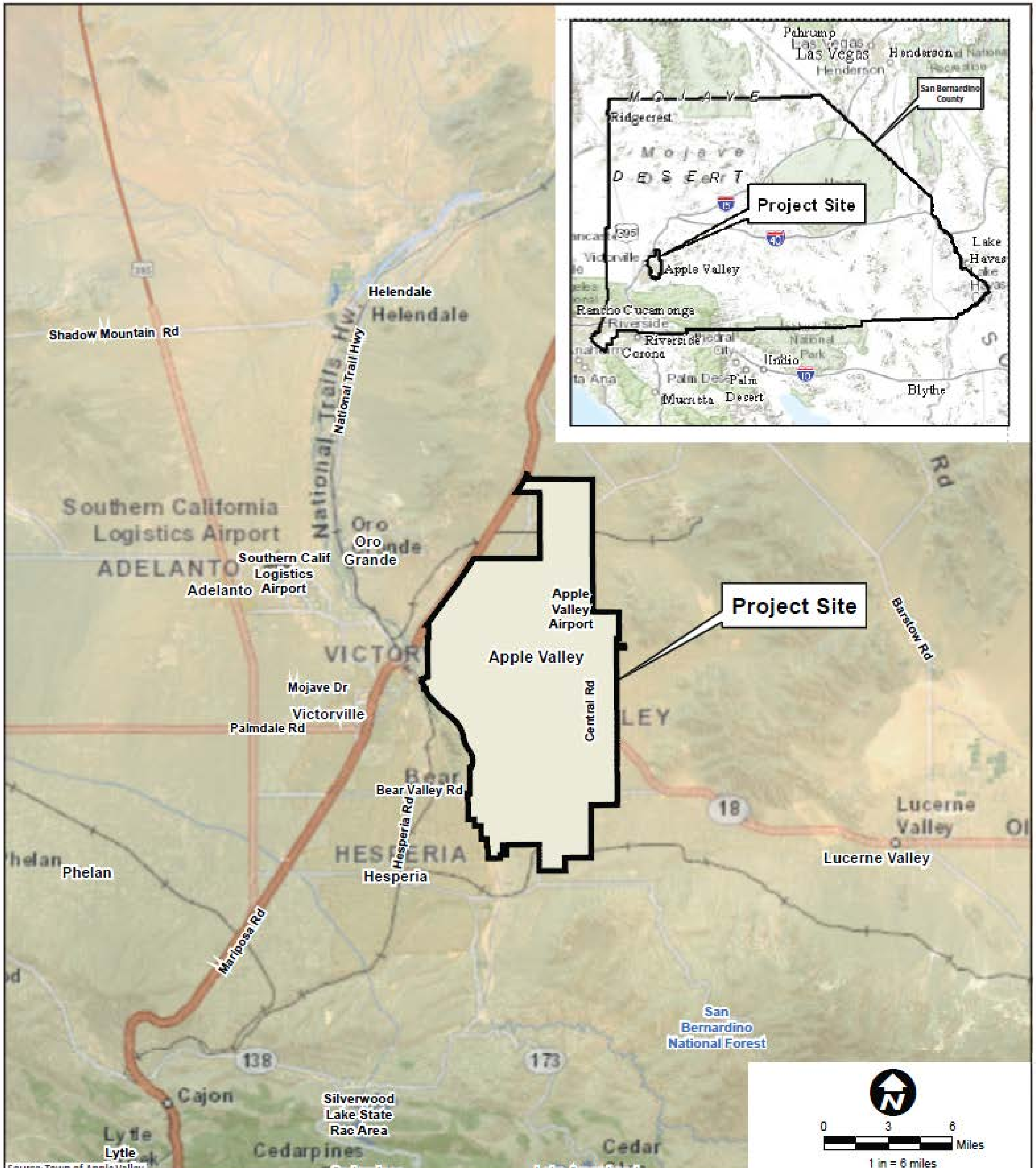
2.2 BACKGROUND

The last Sewer Master Plan for the Town of Apple Valley was developed in 1993 as an accompanying document to the Town's General Plan. URS Corporation (URS) was retained by the Town to develop a Sewer Master Plan Update that is comprehensive in nature and includes the development of recommendations for all aspects of the sewer system that addresses the short- and long-term planning operations.

Prior to development of this Master Plan Update, no digital database of the Town's existing sewer system existed. It was critical that URS develop a GIS database for organizing Apple Valley's sanitary sewer system manhole and pipe data. The GIS database of the Apple Valley Sewer System was developed using as-built drawings provided by the Town staff. Until now, the bulk of the Town's sanitary sewer system network has been maintained utilizing as-built copies of plans. The newly developed GIS database and the computer model of the sewer system will assist the Town in managing its aging infrastructure, as well as allow flexibility for future growth.

URS utilized the existing Land Use Map (Figure 2-2) developed for the Town of Apple Valley General Plan to determine commercial, industrial and residential flows. Historic land use of two houses per acre is consistent with the average residential development throughout the majority of the Town's service area. Previous Sewer Master Plans have indicated that future development could deviate from the two household per acre norm. This Master Plan Update considers the undeveloped areas of the Town according to the land use designation outlined in the 2009 General Plan.

Figure 2-1: Location Map



Town of Apple Valley
Sewer Master Plan Update

Location Map

Figure 2-1

Town of Apple Valley

URS
3500 Porsche Way, Suite # 300
Ontario, CA 91764
(909) 980 4000

2.3 Wastewater Collection System Design Criteria

Currently, the Town requires that new sewers shall be polyvinyl chloride (PVC) pipe. Hydraulic design of sewer lines depends on slope, diameter, and material. These characteristics are important in determination of sewer line capacities and velocities achieved during peak and average wastewater flow conditions. Tables 2-1 through 2-4 summarize the Town's hydraulic design criteria for sanitary sewers.

Table 2-1: d/D Criteria

Ratio	<15 inches	>15 inches
d/D	0.5	0.75

Table 2-2: Velocity Criteria

Criteria	Velocity (fps)
Minimum	2
Desired	3-5
Maximum	10

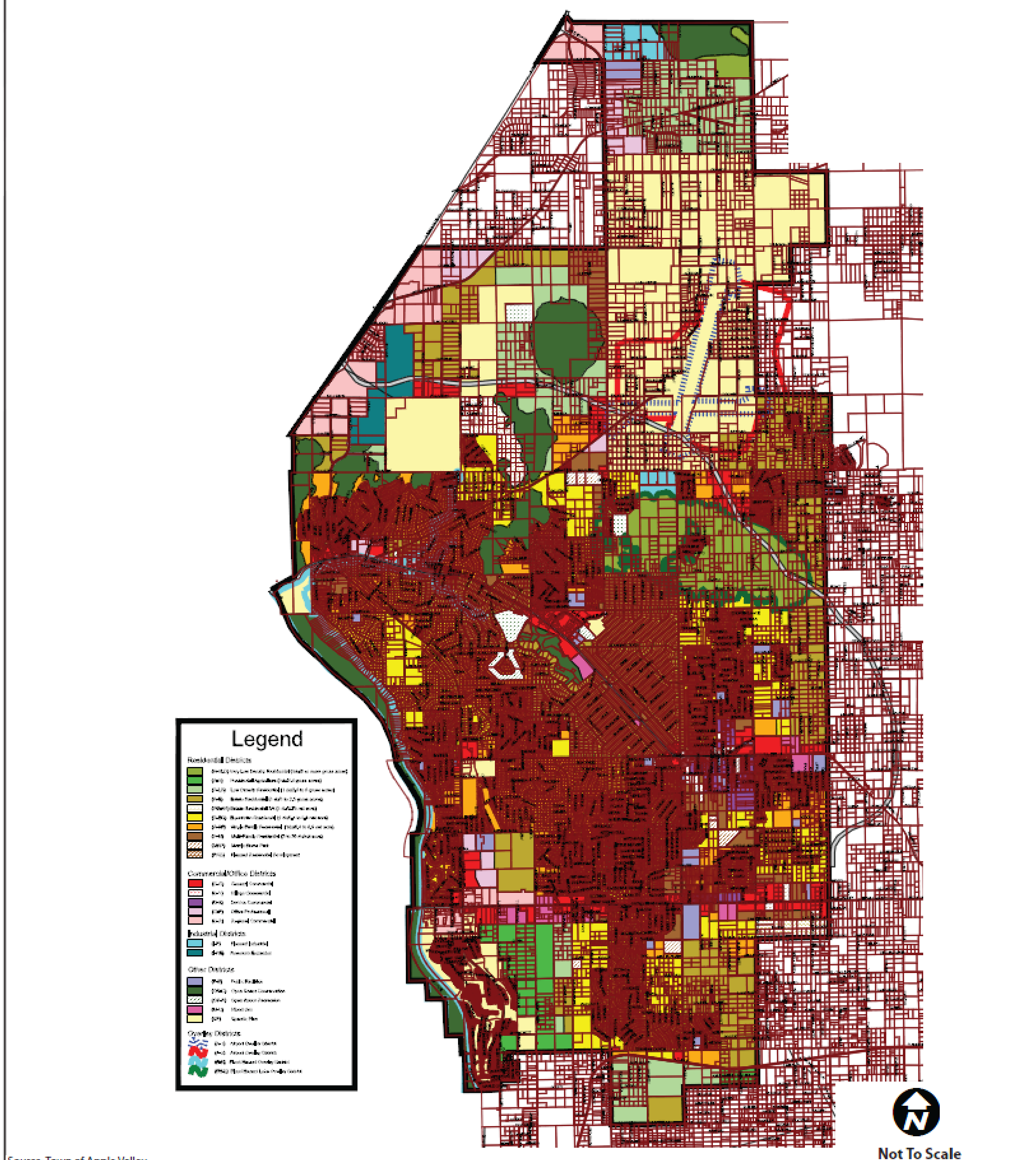
Table 2-3: Manning's Roughness Coefficient (n)

Material	n
PVC	0.009
DI	0.013
VCP	0.011

Table 2-4: Slope and Flow Depths Criteria

Sewer Pipe Size (in)	Slope (ft/ft)	Design Flow Depths at Peak Flow (%)
6	0.0060	50
8	0.0040	50
10	0.0029	75
12	0.0022	75
15	0.0016	75
18	0.0012	75
21	0.0010	75
24 or larger	0.0008	75

Figure 2-2: Town of Apple Valley Zoning Map



Town of Apple Valley
Sewer Master Plan Update

Town of Apple Valley Zoning Map

Figure 2-2

Town of Apple Valley

URS
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3.0 EXISTING SEWER COLLECTION SYSTEM

For organizational purposes, the Town of Apple Valley has been subdivided by Assessment District (AD) and surrounding areas: AD 1, AD 2A, AD 2B, Jess Ranch, AD 3A, and the Northerly portion of Apple Valley. The sewer collection system operates via gravity where possible, but due to the natural topography, there are nine lift stations (LS) scattered throughout the Town. A map of the existing sewer facilities is shown in Figure 3-1.

There are a large number of septic tanks within the Town of Apple Valley. Some of the existing sewer system was put in place in areas where septic systems were determined infeasible due to geologic constraints, or due to groundwater protection and other constraints. In the Town of Apple Valley, as of the date of this report, 70 percent of the developed residential areas by acreage are on septic systems. As the Town continues to grow over the next two to three decades, it is anticipated that most, if not all of these residences will be sewered. The Town's sewer collection system consists of approximately 175 miles of sewer, 3,500 manholes, 9 lift stations, and 9 miles of force mains.

3.1 AD 1 AND SURROUNDING AREAS COLLECTION SYSTEM

Assessment District 1 (AD 1), also known as Upper Desert Knolls, was established in the early 1980's. AD 1 occupies a portion of the mid-westerly end of Apple Valley that is immediately adjacent to Highway 18. Three Assessment Districts and two major tracts make up AD 1: AD 1A, AD 1B, AD 1C, Tract 12905, and Tract 7972. AD1 is mostly composed of single and multi-family residential homes. Much of the land bordering Highway 18 is zoned commercial or light industrial. AD 1 is bounded by Crown Valley Drive in the west, Menahka Road in the north, Rimrock Road in the east, and to the south by the back property lines of the parcels bordering the south side of Highway 18. Saint Mary's Hospital and the Corwin Medical Center are among the larger commercial wastewater dischargers in AD 1. According to the 1993 Master Plan, AD 1 was found to be incompatible with septic tanks for geologic reasons.

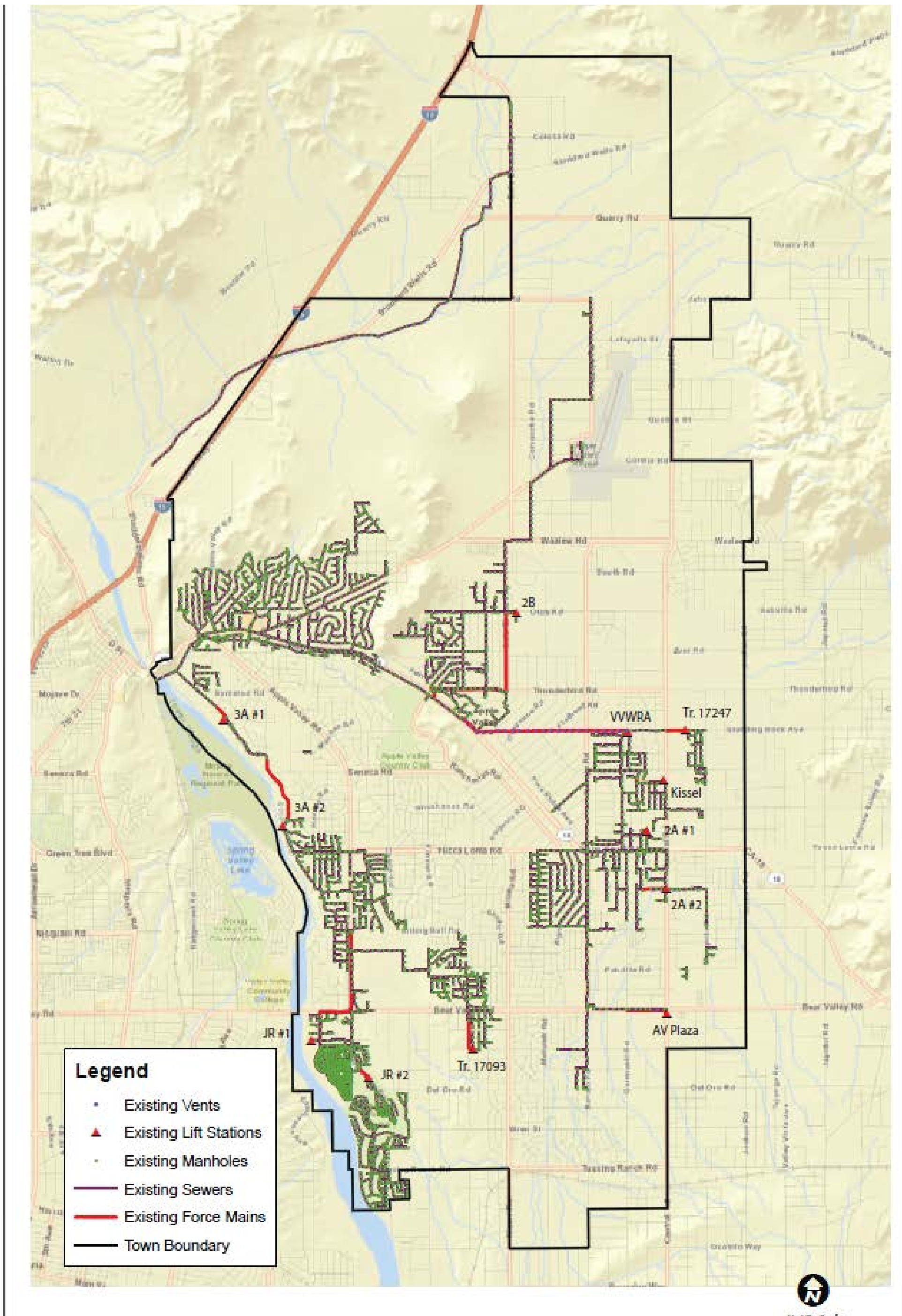
Pipe infrastructure in the AD 1 area is nearing 30 years old. The Town of Apple Valley conducted a video assessment of the sewers in this area based on historic maintenance issues. The results of the assessment are summarized in Section 11 of this Report.

The total sewer pipeline length in AD 1 and surrounding area is approximately 193,100 feet, as shown in Table 3-1. There are no lift stations in AD 1.

Table 3-1: AD 1 and Surrounding Area Sewer Pipelines

Pipe Size (in)	Length (ft)
6	1,970
8	187,125
10	1,313
12	2,447
21	194
Total	**193,048**

Figure 3-1: Existing Sewer Facilities



Town of Apple Valley
Sewer Master Plan Update
Existing Sewer Facilities

Figure 3-1

3.1.1 AD 1A

Constructed in the early 1980's, AD 1A is located at the westerly end of AD 1. It is roughly bounded on the north by Ohna Road, on the west by Chiwi Road, Highway 18 to the south, and Kamana and Kasota Roads to the east. Tract AD 1A is mainly zoned as single and multi-family residential. All sewage from AD 1A flows south by gravity to a 12-inch sewer that is bored under Highway 18, and tied in to the VVWRA interceptor trunk sewer at manhole VVWRA-AVI-061.

3.1.2 AD 1B

Constructed in the early 1980's, AD 1B is located in the center of AD 1. The area is bounded by Corwin Road to the southeast, Monache Road and Kasota Road on the west, and Majela Road to the north. Tract AD 1B is mainly composed of single family residential, with some of the land bordering Highway 18 zoned commercial. All of the wastewater generated from this area flows south to a gravity sewer on Corwin Road. The Corwin Road sewer ties in to VVWRA interceptor.

3.1.3 AD 1C

Constructed in 1982, AD 1C is wedged between Corwin Road to the northwest, Highway 18 to the south, and Rimrock Road to the east. Like the other two AD 1 developments, Tract AD 1C is mainly single family residential in nature, with some of the land bordering Highway 18 zoned as commercial or multi-family residential. Its sewer system discharges to a sewer line located on Highway 18. The line runs parallel to the VVWRA interceptor and is tributary to the VVWRA line west of AD 1C.

3.1.4 Tracts

Tracts 12905 and 7972 are located to the west and east of AD 1A, respectively. Constructed in 1986, Tract 12905 is bounded by Crown Valley Drive on the west, Crown Valley Court in the north, Ridge View Drive in the east, and Highway 18 to the south. Constructed in 1988, Tract 7972 is bounded by Viho Road in the west, Menahka Road on the north, Monache Road in the east, and Highway 18 to the south. Both tracts are composed of single-family residential homes.

3.2 AD 2 and Surrounding Areas Collection System

The primary developments and most notable areas in Assessment District 2's collection system are Assessment District 2A (AD 2A), Assessment District 2B (AD 2B), Tracts 16492 and 17247, and the Kissel Development. Located along the southeasterly portion of the Town of Apple Valley, AD 2A contains various land uses, including residential, industrial, public facilities and commercial/light industrial.

The Navajo Road sewer system is the primary trunk sewer for the southeasterly portion of AD 2A. The flow pattern for the southeasterly portion is primarily south to north. The Navajo trunk sewer extends northerly from Bellflower Road to Ramona Road, it then travels east to Quinnault Road, from Quinnault Road it heads north to Standing Rock Avenue, and then winds northeasterly to the intersection of Standing Rock Road and Nanticoke Road. The Lift Station VVWRA Nanticoke AD 2, located at this intersection, pumps the wastewater flows nearly two miles westerly along and parallel to Standing Rock Road to Highway 18 where it flows by gravity to the VVWRA interceptor. Sewage from Tract 17247 is pumped westerly along Standing Rock Avenue to manhole 2A-17247-051, located at the intersection of Central Road and Standing Rock Avenue. From here the flows travel by gravity westerly to the Lift Station VVWRA Nanticoke AD 2.

AD 2B, located northwest of AD 2A, is made up of residential developments, commercial and open space areas. The flow pattern for the residential housing is primarily south to north. Wastewater flows from this area are conveyed to Lift Station 2B, located at the intersection of Otoe Road and Dale Evans Parkway. From here the flows get directed through a force main to the VVWRA interceptor.

3.2.1 AD 2A

Established in 1982, Assessment District 2A's sewer system transports wastewater from Apple Valley's Village area. The area is bounded by Little Beaver Road at the south end, Mohawk Road along the west, Standing Rock Road to the north, and Central Road on the east side. The Navajo Road sewer system is the primary trunk sewer of AD 2A. Sewage from AD 2A flows generally north to the Lift Station VVWRA Nanticoke AD 2, located at the intersection of Standing Rock Road and Nanticoke Road. Wastewater discharge from a small portion of the central east area of AD 2A flows to pump station LS 2A No. 1. LS 2A No. 1 pumps sewage through a 10-inch diameter pipe 11,259 feet to a manhole located at VVWRA-AVI-001. Pump station LS 2A No. 2 conveys flows from the southeasterly portion of AD 2A. The flow travels westerly along Ottawa Road through a 4-inch, 1,600 feet long sewer force main. From here, the wastewater flows by gravity northerly toward the Lift Station VVWRA Nanticoke AD 2. The Lift Station VVWRA Nanticoke AD 2 pumps the wastewater flows nearly two miles westerly along and parallel to Standing Rock Road to Highway 18 where it flows by gravity to the VVWRA interceptor. Table 3-2 lists the lift stations and their respective design data located in AD 2A and surrounding areas.

Pipe infrastructure in the AD 2A area is nearing 30 years old. The Town of Apple Valley conducted a video assessment of the sewers in this area to determine the condition of the pipes. Condition assessments are listed in Section 11 of this Report.

Table 3-2: AD 2A and Surrounding Area Lift Stations

Lift Station	Status	Switch On (ft)	Switch Off (ft)	Design Flow (gpm)	Design Head (ft)	Impeller Diameter (in)	Remarks
2A No. 1	Duty	2,924.04	2,923.46	250	16.8	8 1/8	
	Assist	2,924.54	2,923.96	60	13	6 5/8	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
2A No. 2	Duty	2,927.36	2,926.86	60	36	4	
	Assist	2,927.86	2,927.36	60	36	4	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,928.36	2,927.86	60	36	4	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
AV Plaza	Duty	2,968.22	2,967.72	270	36.9	6.89	
	Assist	2,968.72	2,968.22	270	36.9	6.89	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
Kissel	Duty	2,912.43	2,910.43	200	45	3	
	Assist	2,912.93	2,910.93	200	45	3	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
VVWRA	Duty	2,904.57	2,903.57	650	74.23	13.2	
	Assist	2,905.07	2,904.07	650	186.9	13.2	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,905.57	2,904.57	650	186.9	13.2	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
Tr. 17247	Duty	N/A	N/A	N/A	N/A	N/A	Not built.
	Assist	N/A	N/A	N/A	N/A	N/A	

The total sewer pipeline length in AD 2A is approximately 223,000 feet, as shown in Table 3-3.

Table 3-3: AD 2A and Surrounding Area Sewer Pipelines

Pipe Size (in)	Length (ft)
6	52,215
8	61,168
10	43,863
12	19,613
15	20,863
18	14,465
21	5,354
24	5,447
Total	**222,988**

3.2.2 Tract 16492

Established in 2003, Tract 16492's sewer system transports sewage from the tract easterly where it joins manhole 2A-000-192 just west of the intersection of Navajo Road and Happy Trails Highway. Here, Tract 16492's flows are combined with those from AD 2A and are conveyed northerly to the Lift Station VVWRA Nanticoke AD 2. All land use for Tract 16492 is single family residential.

3.2.3 Tract 17247

Established in 2007, this small tract is located in the northeast corner of the AD 2A collection system. There are currently no properties in Tract 17247. Flows from this area are constructed to travel northerly to Standing Rock Avenue and then head west to a small sewer lift station along Standing Rock Avenue. The lift station pumps the water 1,275 feet west to the intersection of Standing Rock Avenue and Central Avenue. Here, the force main joins manhole 2A-17247-051 and gravity flows westerly to the Lift Station VVWRA Nanticoke AD 2.

3.2.4 Kissel Development

Established in 1990, this small development is located east of AD 2A, between Esaws Road and Nanticoke Road. Flows from the residential area are conveyed northeast along Hurons Road to the Kissel lift station located at Hurons Road and Central Road. From the lift station, the flow travels west to manhole 2A-000-C08 located at Pioneer Road and Hurons Road. From here the flow continues north to Ramona Road, it then heads west to Quinnault Road, north to Standing Rock Avenue, and is finally directed northeast to the Lift Station VVWRA Nanticoke AD 2.

3.2.5 AD 2B and Surrounding Area

AD 2B is located northwest of AD 2A, along Highway 18. Established in 1991, AD 2B consists predominantly of single family, multi-family, equestrian residential homes, commercial and open space areas. The residential areas are bounded by Rancherias Road on the southwest, Huasna Road to the north, Dale Evans Parkway to the east, and by the back commercial property bordering Highway 18. Surrounding areas consist of the northeastern part of Town. The major developments in this northeastern area include the Walmart Distribution Center, Apple Valley Airport and Los Ranchos Mobile Home Park. The distribution center is considered to be among the Town's largest wastewater dischargers. The Walmart Distribution Center is located adjacent to Johnson Road between Bell Mountain Road and Navajo Road, Apple Valley Airport is bounded by Waalew Road at the south end, Bell Mountain Road along the west, Johnson Road to the north, and Sycamore Lane to the west, and the Los Ranchos Mobile Home Park is located on Waalew Road. Los Ranchos Mobile Home Park is not connected to the Town's sewer system; it has its own private package treatment plant. Flows from the residential area are conveyed northeast along Otoe Road to Lift Station 2B located at the intersection of Otoe Road and Dale Evans Parkway. The wastewater flows from the large density areas travel south to Lift Station 2B. Here, a 12 inch sewer force main, 11,424 feet long conveys the wastewater flows southerly along Dale Evans Parkway to Thunderbird Road where it heads westerly to Rancherias Road and then winds southerly where it discharges to the VVWRA interceptor trunk sewer at manhole VVWRA-AVI-008. Table 3-4 lists the lift stations and their respective characteristics located in AD 2B.

Table 3-4: AD 2B Lift Stations

Lift Station	Status	Switch On (ft)	Switch Off (ft)	Design Flow (gpm)	Design Head (ft)	Impeller Diameter (in)	Remarks
2B	Duty	2,891.61	2,889.03	1200	141	14	
	Assist	2,892.11	2,889.53	680	64	9.2	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,892.61	2,890.03	725	60	9.2	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,893.11	2,890.53	1200	141	14	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.

The total sewer pipeline length in AD 2B is approximately 118,800 feet, as shown in Table 3-5.

Table 3-5: AD 2B Sewer Pipelines

Pipe Size (in)	Length (ft)
6	227
8	50,660
10	24,586
12	26,639
15	12,063
18	4,615
Total	**118,790**

3.3 Jess Ranch, AD 3A, and Surrounding Area Collection System

The Jess Ranch, AD 3A, and surrounding area sewer collection system consists predominantly of residential, single-family housing. There are large commercial pockets along Bear Valley Road. The sewer system from this area was established to collect flows and convey them northerly. Flows from Jess Ranch are conveyed from Poppy Road in the south toward Town Center Drive in the north. Lift Station JR No. 1 pumps wastewater northerly from the Jess Ranch toward the middle portion of the westerly flow area. There is also an additional lift station, JR No. 2 that pumps wastewater from the central portion of Jess Ranch over a high point and back to the trunk gravity sewer system. Lift stations LS 3A No. 1 and LS 3A No. 2 pump additional wastewater northerly where it flows into the VVWRA interceptor about a quarter mile south of Happy Trails Highway. Table 3-6 lists the lift stations located in the Jess Ranch, AD 3A, and surrounding area sewer collection system.

Table 3-6: Jess Ranch, AD 3A, and Surrounding Areas Lift Stations

Lift Station	Status	Switch On (ft)	Switch Off (ft)	Design Flow (gpm)	Design Head (ft)	Impeller Diameter (in)	Remarks
JR No. 1	Duty	2,809.30	2,808.30	900	78	10 3/32	
	Assist	2,809.80	2,808.80	900	78	10 3/32	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
JR No. 2	Duty	2,827.84	2,826.84	140	25	6	
	Assist	2,828.34	2,827.34	140	25	6	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
3A No. 1	Duty	2,759.68	2,758.42	1560	82	15	
	Assist	2,760.18	2,758.92	1560	82	15	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,760.68	2,759.42	1560	82	15	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
3A No. 2	Duty	2,738.06	2,736.80	1700	50	13	
	Assist	2,738.56	2,737.30	1700	50	13	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
	Assist	2,739.06	2,737.80	1700	50	13	All on and off elevations of assist pumps are assumed to be 0.5 ft higher.
Tr. 17093	Duty	N/A	N/A	N/A	N/A	N/A	Not built.
	Assist	N/A	N/A	N/A	N/A	N/A	

The approximate total sewer pipeline length in Jess Ranch, AD 3A, and surrounding areas is 306,430 feet, as shown in Table 3-7.

Table 3-7: Jess Ranch, 3A, and Surrounding Area Sewer Pipelines

Pipe Size (in)	Length (ft)
6	46
8	203,060
10	33,309
12	29,498
15	21,230
18	18,630
21	657
Total	**306,430**

3.4 NORTHERN APPLE VALLEY INTERCEPTOR

The Northern Apple Valley Interceptor (NAVI) transports flows from the northerly portion of the Town. This area consists of the High Desert Juvenile Detention Center located at the intersection of Dale Evans Parkway and Morro Road. The Detention Center is considered to be among the larger wastewater dischargers in Apple Valley. Sewage from the Detention Center flows south from manhole VVWRA-NAVI-001 to Stoddard Wells Road where it continues southerly along the road until reaching Interstate 15. The NAVI terminates at manhole VVWRA-NAVI-089. From there, the flow travels downstream to the Victorville sewer collection system. The total sewer pipeline length in the NAVI is 38,925 feet, with diameters ranging from 18 inches to 24 inches.

4.0 PREPARATION OF SEWER SYSTEM INVENTORY

This section describes the preparation of the sewer system inventory database for the Town of Apple Valley's sewer system. The Town provided the as-built drawings to URS, which show the existing sewer system. Based on the as-built drawings, the existing sewer system was reproduced in AutoCAD and exported into ArcGIS. The resulting GIS sewer system database file contains pipe information for all sanitary sewer pipes eight (8) inches and greater in diameter, and their respective upstream and downstream manholes in the system. In some instances, pipes less than six (6) inches in diameter were incorporated if they are located between 8 inches diameter pipes.

4.1 CONSTRUCTION AND RECORD DRAWINGS

The as-built maps provided by the Town of Apple Valley serve as the primary source of information for creating the GIS database. The as-built maps show the locations of all manholes, sewer size, length, and slope.

4.2 ATTRIBUTE DATA

For each sanitary sewer, the following information were manually extracted from the as-built drawings and entered into a GIS database:

- Plan name
- Diameter of pipe
- Upstream manholes reference number
- Downstream manholes reference number
- Upstream invert level
- Downstream invert level
- Length of pipe
- Slope of pipe
- Pipe material
- Date of construction
- Street name
- Manhole rim elevations and invert elevations

4.3 MAPPING AND MANHOLE NUMBERING

Manholes and existing sanitary sewer locations were obtained by extracting information from as-built maps provided by the Town of Apple Valley. URS utilized right-of-way and parcel map information provided by the Town to determine manhole and pipe locations. The spatial information was subsequently transferred into the GIS database. Figure 3-1 shows the layout of the existing sewer system.

Each manhole on Figure 3-1 is identified by a unique manhole number. The naming convention was developed jointly with the Town of Apple Valley staff. The main goal of the naming convention developed for the Town's system was to quickly and easily locate the manholes in the Town's cleaning and video inspection databases.

The manhole naming convention used follows this pattern, 2B-16799-015, where 2B represents the Assessment District, 16799 represents the Tract Number, and 015 represents the Manhole Number located on the as-built plans.

In the case of an Assessment District having a Tract Number, e.g., 7972, the convention used was 7972-000-001 for the plans marked Tract 7972, 7972-DK1-001 for the plans marked Desert Knolls, and 7972-DK2-001, for the plans marked Desert Knolls II.

If the plans had no Tract Number, e.g., 1B, the area designated for the Tract Number were filled with three zeros, 1B-000-001.

Some plans had a project name instead of a Tract Number, e.g., 2B Aztec Road Extension, the convention used was 2B-AZTEC-001. In some cases an abbreviation was used in the case of 2B East Side Public Safety Training Facility, 2B-ESPSTF-001.

4.4 DATUM

The projected coordinate system used to create the Town's GIS sewer system database is the North American Datum 1983 (NAD 83) California State Plane Zone V, with US Foot as the unit of measure.

The manhole inverts were recorded to different datum throughout the system. In order to ensure the pipe's flow downhill, URS made adjustments to the invert level so sewer can flow downhill. The updated elevations are documented in the GIS database URS prepared as part of this project.

5.0 LAND USE AND GROWTH PROJECTIONS

Population growth is critical to the planning of expansion of the Town's sewer system. Many issues impact the existing sewer system, including exceedance of capacity of existing facilities, funding, and aging infrastructure. The challenge facing the Town is knowing when and what to upgrade to maintain a viable and properly functioning sewer system.

5.1 BACKGROUND

According to the Town of Apple Valley's web site, its population in 1990 and 2000 were 46,079 and 54,239, respectively. Assuming linear growth between 1990 and 1993, the population at the time of the last Master Plan was 48,527. During the time since the last Master Plan, the Town has grown approximately 46 percent. The population and number of households for 2010, 2020, and 2030 from Southern California Association of Governments (SCAG) are shown in Table 5-1 and 5-2. It should be noted that the projected population and households for 2030 were estimated assuming a linear growth between 2020 and 2035 data obtained from SCAG. As the population continues to increase, there will be primarily three sources of growth for the Town's sewer collection system. These three areas are development in previously undeveloped areas of town, infill of undeveloped lots in existing subdivisions, and connection of houses currently on septic systems.

Table 5-1: Town of Apple Valley Projected Population

2010	2020[1]	2030
69,100	82,900	100,300

[1]*Information referenced from SCAG – Growth Forecast 2012-2035.*

Table 5-2: Town of Apple Valley Projected Households

2010	2020[1]	2030
23,600	28,500	34,200

[1]*Information referenced from SCAG – Growth Forecast 2012-2035.*

Both land use and population are critical for determining wastewater flows. The Town has determined the amount of flow per acre for each land use type by evaluating sewer discharge from existing areas of the same land use type (expressed as average daily flow in gallons per day per acre). This information was provided to URS. URS divided the sewer network into areas by land use type. URS calculated existing residential flows by counting the total number of households, and multiplied them by 210 gallons per household per day, per Town of Apple Valley planning criteria. URS tabulated the land use type, the acreage, and the total flow for each area for all commercial areas as well as future residential areas.

In allocating flows in the existing and future conditions for the Jess Ranch area, URS assumed a wastewater flow of 90 gallons per household per day for the west side of Apple Valley Road. Per the direction of the Town of Apple Valley, the land use for this area is designated as senior living where water consumption is less. As a result, the production of wastewater flows is lower than the average residential flows. However, the area located on the east side of Apple Valley Road in the Jess Ranch development is zoned as mostly residential and flow per household produced are the typical average flows, therefore URS was directed to use 210 gallons per household per day.

5.2 CHALLENGES

Much of the Town's existing sewer system is aging. Assessment Districts 1 and 2A were built in the early 1980's. Additionally, according to the SCAG, the Town's population is expected to grow by 45 percent between 2010 and 2030. This growth will come primarily from two sources: new residential subdivisions and the infill of vacant lots in existing communities.

The population increase creates a need for new sewers. While much of these sewer systems will be constructed by the developers of the new communities, these systems will ultimately tie into the town's existing sewer infrastructure. This infrastructure is nearing capacity, and the Town is presently working with the Victor Valley Reclaimed Water Authority (VVWRA) to create new facilities to accommodate these increased sewer flows.

Two significant trends are occurring within Apple Valley. First, the Town is requiring new developments to irrigate vegetation within street rights-of-way by using reclaimed water. To this end, the Town is working with VVWRA to construct a new wastewater treatment plant that would provide a large source of irrigation water for existing Town facilities. Second, sewer flows will increase by the construction of new sewers in existing communities currently on septic tanks. Table 5-3 presents the population and housings for 2012 and buildout. According to the Town's 2009 General Plan, only about 30 percent of the Town's households are tied into the existing sewage network. Unmarked areas shown in Figure 5-1 are expected to remain under septic systems. That means 70 percent of the Town's existing households, or 16,580 households, do not currently discharge to the Town's sewer system. Assuming all households are sewered at buildout, an extra number of 44,297 households need to be sewered as compared to 2012.

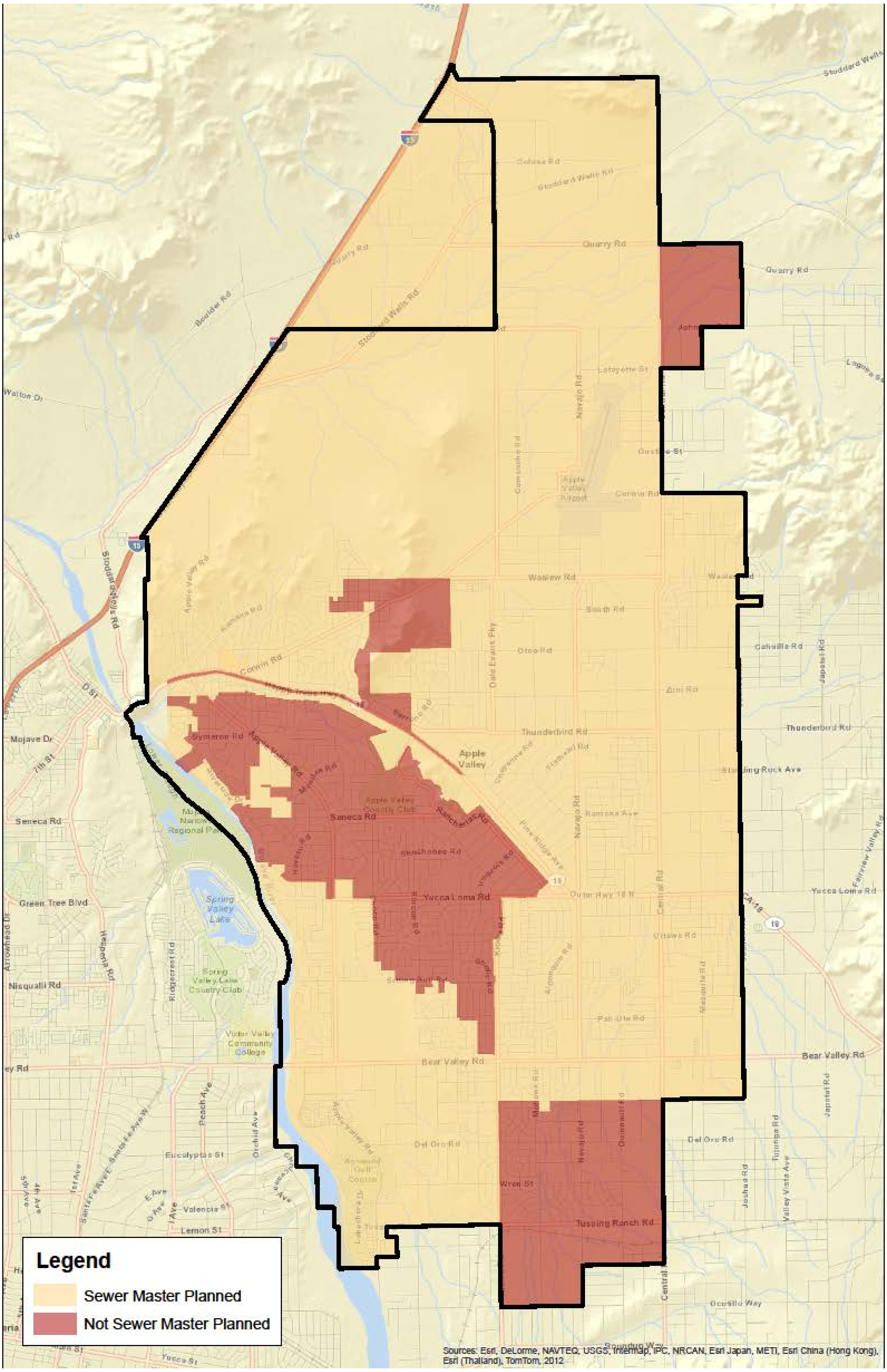
Table 5-3: Existing and Buildout Data for the Town of Apple Valley

2012		BuildOut[2]	
Population	**Housing**	**Population**	**Housing**
70,033	23,685	185,858	60,877

[2]*Source: 2009 Apple Valley General Plan.*

Contamination of groundwater is a concern for all septic tanks. It is anticipated that existing communities currently on septic systems will be served by new sewers as funding becomes available.

Figure 5-1: Sewer Master Planned and Not Sewer Master Planned



Town of Apple Valley
Sewer Master Plan Update
Sewer Master Planned and Not Sewer Master Planned

Figure 5-1

Town of Apple Valley

URS
3500 Porsche Way, Suite #300
Ontario, CA 91764
(909) 980-4000

5.3 GENERAL PLAN POLICIES AND OBJECTIVES

The Town has a rural characteristic that is aesthetically pleasing with the desert area. It is paramount that the Town preserves and protects its character for the community's long term well-being. To this end, residential Specific Plans within the Town are restricted to a maximum size of 2 units per acre. New development shall minimize grading and avoid mass grading when possible. Where conceivable, no grading is encouraged. As undeveloped areas of the Town are urbanized, these limitations on grading, along with the Town's topography may prevent sewers to flow by gravity in some areas. To alleviate this problem, the construction of new lift stations is anticipated.

New residential developments shall connect to the sewer system. Where sewer service is unavailable and lot sizes are less than one acre, the Town requires installation of a dry sewer and sewer connection fees for future sewer main extensions. Additionally, units adjacent to major roadways shall be sewered, and all commercial businesses and medium and high density housings shall also be sewered.

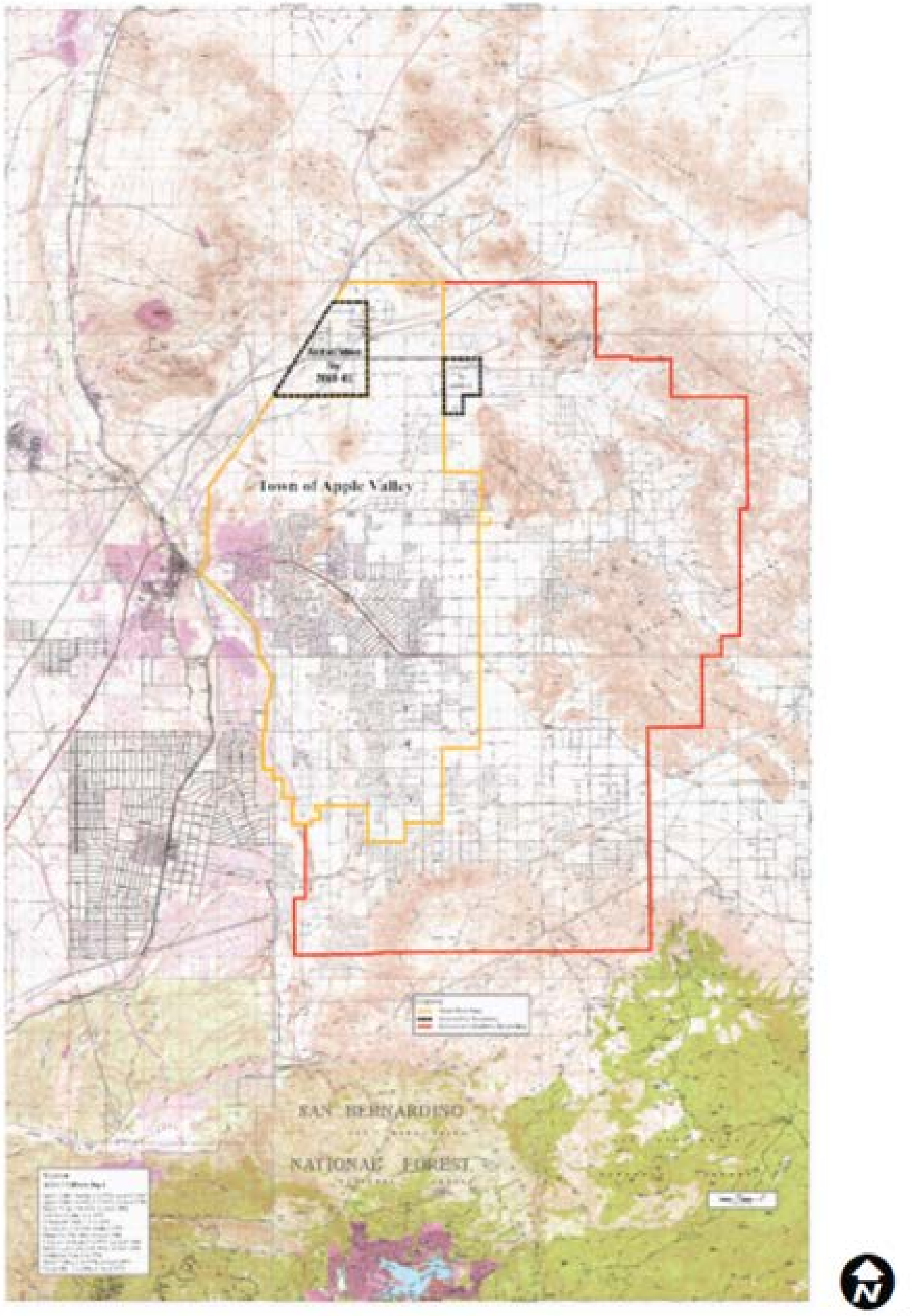
The General Plan loosely follows historic land usage trends for existing and proposed development. Where necessary, the General Plan alters historic trends to better serve the community. The General Plan was updated in 2009. One of the mandates of the General Plan was the creation of an updated Sewer Master Plan.

5.3.1 Sphere of Influence

The 2009 General Plan identified a very large sphere of influence for the Town, as shown in Figure 5-2. This sphere of influence, as determined by the Local Agency Formation Commission, spans almost 123,000 acres, or 192 square miles, and is roughly equal to the current acreage of the Town. The Town is working to annex approximately 809 acres located east of Central Road and south of Quarry Road (Annexation Number 2008-02). This area is projected to become part of the North Apple Valley Industrial Specific Plan which includes and surrounds the airport.

The second area incorporated in the General Plan, Annexation Number 2008-01, is in the northern part of Apple Valley, and is also known as the "Golden Triangle". As indicated by the Town, while the Golden Triangle will no longer be annexed, this area will ultimately tie into the Town's sewer infrastructure. Due to this, the projected flows generated by this area are also incorporated in the sewer hydraulic model. The rest of the Town's sphere of influence is not considered in this General Plan. Other parts of the sphere of influence will be addressed by the Town as they are annexed.

Figure 5-2: Sphere of Influence Boundary



Source: 2009 General Plan

Town of Apple Valley
Sewer Master Plan Update

Sphere of Influence Boundary

Figure 5-2

Town of Apple Valley

URS
1500 Porsche Way, Suite # 300
Ontario, CA 91764
(909) 980 4000

5.3.2 Water and Sewer Services

The 2009 General Plan identified that water and sanitary sewer services are provided by the Apple Valley Ranchos Water Company and other independent water companies, and the VVWRA, respectively. Lands designated for Multi-Family or Mixed Use developments are located on major roadways, which are serviced by water and sewer mains currently. The water purveyors and the sanitary sewer system have current capacity, or expansion plans sufficient to accommodate in Town, including the Town's housings which need allocation. As required, the Town will provide the water purveyors and VVWRA with copies of the adopted Hosing Element. These purveyors are also required by law to provide priority service for affordable housing projects.

5.4 Future Flow Projections

The Town covers a total area of 50,532 acres, including the two annexation areas. The land area and usage defined in the Town's 2009 General Plan plays an important role in the determination of sewer flow projections since flow generation is a function of land use designation. The General Plan defines land use designations for residential, commercial, industrial, and other land uses that are within the boundaries of the Town. The General Plan Land Use Map, Figure 2-2, was utilized to determine the future flow projections at buildout for the following categories: residential, commercial, and industrial. The following sections summarize the methodology used to calculate the projected wastewater flows.

5.4.1 Residential Flows

The buildout residential wastewater flows were projected using the land use designations established in the Town's General Plan. The allowable densities for each residential designation, summarized in Table 5-4, were used to determine the projected dwelling units (du) for undeveloped areas.

Table 5-4: General Plan's Residential Land Use Densities

Residential Designation	Maximum Density
Very Low Density Residential (R-VLD)	1 du/5 or more acres
Residential Agriculture (R-A)	1 du/2.5 or more acres
Low Density Residential (R-LD)	1 du/2.5 to 5 gross acres
Estate Residential (R-E)	1 du/1 to 2.5 gross acres
Equestrian Residential (R-EQ)	1 du/0.4 to 0.9 net acre
Single Family Residential (R-SF)	1 du/0.4 to 0.9 net acre
Multi-Family Residential (R-M)	2 to 20 du/1 acre

The undeveloped residential areas for the buildout scenario were found by intersecting the land use map and aerial Bing map. Empty residential parcel found in the aerial map were measured and multiplied by the allowable residential densities to find the future dwelling units within each assessment district. Similar to the residential flows in the existing system, URS estimated the number of residences projected and use the maximum density per Table 5-4 to arrive at a total number of EDUs which were then multiplied by 210 gallons per day per dwelling unit to compute the wastewater flows for the undeveloped residential area.

5.4.2 Future Commercial and Industrial Flow Allocations

The General Plan classifies commercial and industrial areas using the following designations: General Commercial (C-G), Service Commercial (C-S), Regional Commercial (C-R), Planned Industrial (I-P), and Office Professional (O-P). Using these designations, future commercial and industrial zones were approximated by creating a layer in GIS to represent these land use areas. The polygons were then compared to the existing commercial and industrial development in the region. The already built areas were assessed and removed from the total acreage designated as commercial and industrial. The Town's commercial and industrial criterion for wastewater flow generation is 1,200 and 1,500 gallons per acre per day, respectively. The future commercial and industrial flows were calculated by multiplying the net undeveloped area with the Town's unit flow criterion. These flows were then added to the existing model to obtain the projected build out sewer flow. The buildout flows for each commercial and industrial polygon were equally distributed to manholes identified in close proximity to the designation under consideration.

5.4.3 Specific Plans Flows

According to the Town's Development Code, Specific Plans are adopted in areas where remoteness, environmental constraints, or unique land use concerns require specific land use and/or design controls. The Specific Plans are consistent with the provisions of the adopted General Plan. The General Plan Land Use Map, Figure 2-2, contains eight (8) Specific Plan areas. The Specific Plans were used to identify current or future developmental area designations in order to calculate the expected buildout sewer flow generation. The following are brief descriptions of all the Specific Plans listed in the General Plan Land Use Map.

5.4.3.1 Bridle Path Estates

The Bridle Path Estates Specific Plan is situated in a sloping valley that is in between the hills and rock outcroppings of northern Apple Valley. The project area is located south of Falchion Road, mostly west of Chippewa Road, and east of Apple Valley Road, as shown in Figure 5-3. The Specific Plan project area covers 664-acre which consists of 60 acres of preserved natural Open Space, 53 acres of Mineral Resources and 551 acres of Community Reserve land use that support half-acre residential lot development. Table 5-5 lists the land use designation of the areas, gross acres, and maximum allowed units. The Bridle Path Estates Specific Plan was adopted by the Town Council on October 10, 2006.

Table 5-5: Land Use Summary for the Bridle Path Estates Specific Plan

Land Use	Gross Acres	Maximum Allowed Units[3]	Maximum Residential Density
Equestrian Residential (R-EQ)	99	194	2.0 du/ac. Net
Single Family Residential (R-SF)	452	904	2.0 du/ac. Net
Open Space (OS)	60	-	N/A

Land Use	Gross Acres	Maximum Allowed Units[3]	Maximum Residential Density
Mineral Resources (MR)	53	4	N/A
Total	**664**	**1,102**	

[3]*Maximum allowed units is based on gross developable area.*

5.4.3.2 Deep Creek Estates

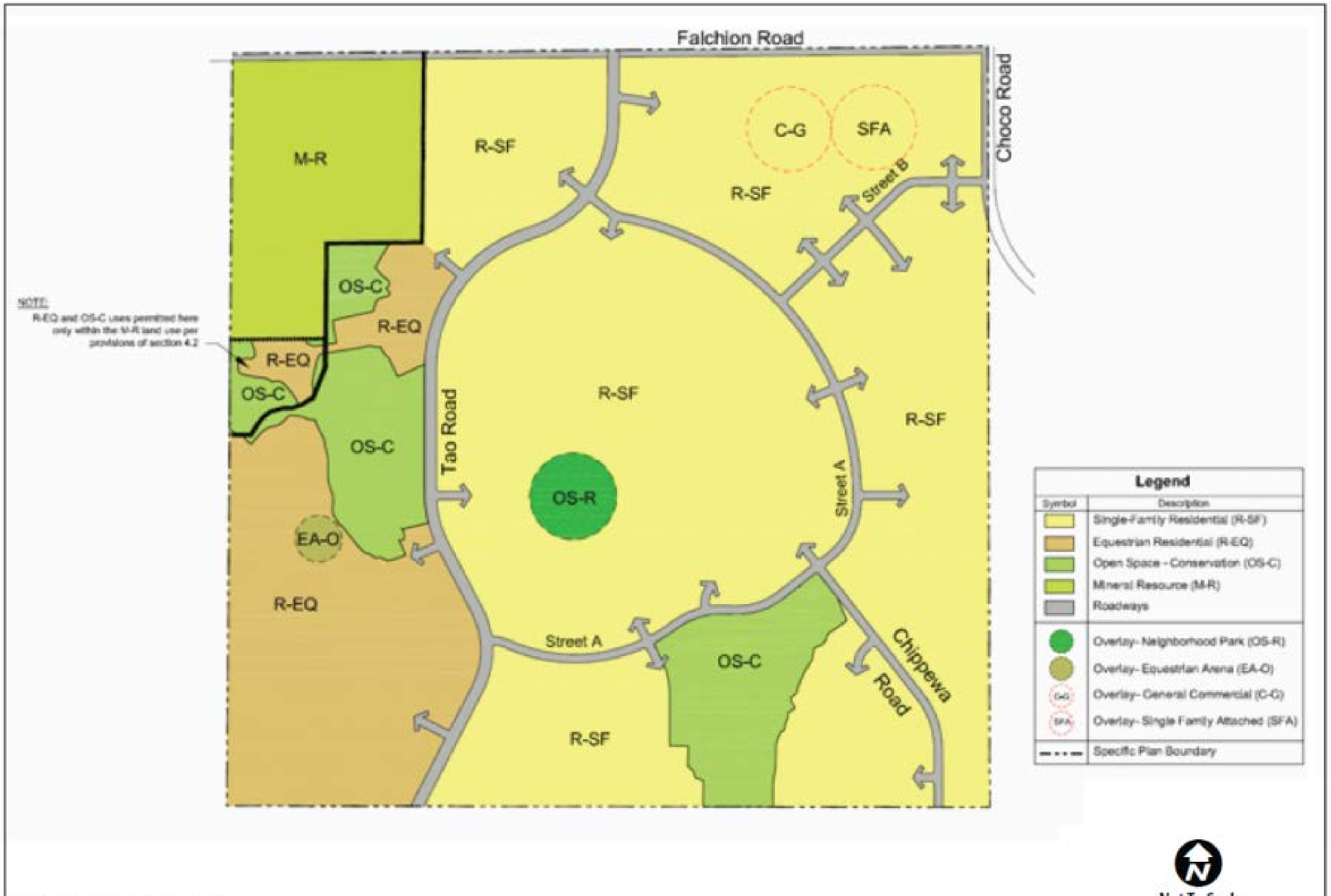
The Deep Creek Estates Specific Plan is located in the southwest corner of the Town. The project area is south of Grande Vista Street, west of Deep Creek Road, north of Tussing Ranch Road, and southeast of Jess Ranch, as shown in Figure 5-4. The project plan includes a project area of 80 acre which includes open-space and easements for recreational opportunities (e.g., community equestrian, multi-use trails, park, residential and equestrian development), and an equestrian themed residential and single-family community developments. Table 5-6 contains the maximum allowed land use summary provided in the Specific Plan.

Table 5-6: Land Use Summary for Deep Creek Estates Specific Plan

Land Use	Gross Acres	Maximum Allowed Units[4]	Maximum Residential Density
Equestrian Residential (R-EQ)	40.68	81	2.0 du/ac. Net
Single Family Residential (R-SF)	27.88	55	2.0 du/ac. Net
Drainage Area/Open Space (OS)	12.19	-	N/A
Total	**80.75**	**136**	

[4]*Maximum allowed units is based on gross developable area.*

Figure 5-3: Bridle Path Estates Specific Plan Land Use Map



Source: 2006 Briddle Path Estates Specific Plan

Town of Apple Valley
Sewer Master Plan Update

Bridle Path Estates Specific Plan Land Use Map

Figure 5-3

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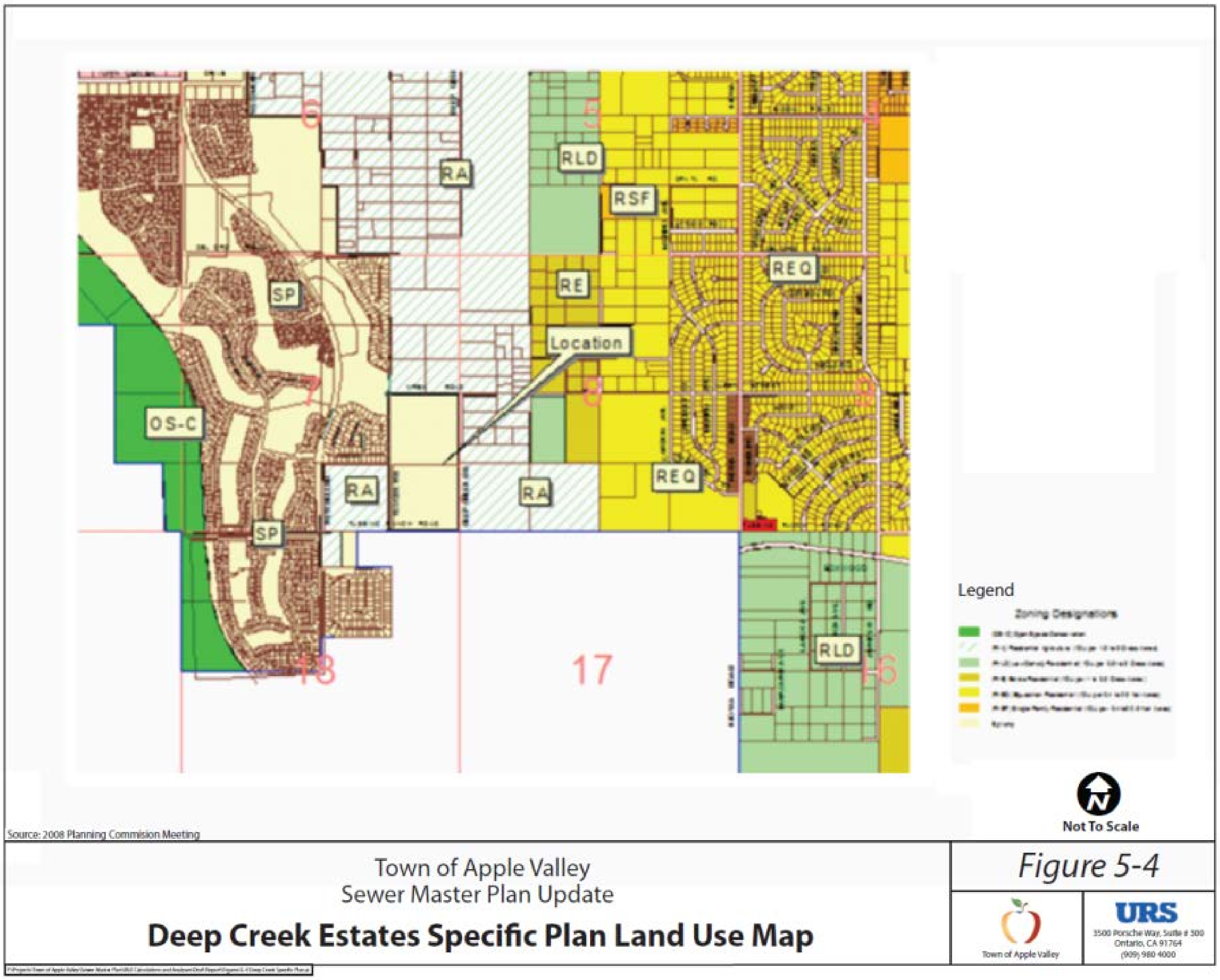
Figure 5-4: Deep Creek Estates Specific Plan Land Use Map



5.4.3.3 **Jess Ranch**

Jess Ranch consists of 1,447 acres which includes 80 acres zoned as regional commercial located at the corner of Bear Valley Road and Apple Valley Road and approximately 1,367 acres that are part of the 1998 Planned Unit Development (PUD), shown in Figure 5-5. Jess Ranch is located east of the Mojave River, south of Bear Valley Road, north of Poppy Street and east of Cottontail Lane. The PUD area was broken down into small parcels that are defined by land use designations as listed in Table 5-7 for residential and commercial areas. The PUD designations were used to approximate the buildout sewer flows generated within this area.

Table 5-7: Residential and Commercial Land Use Allocations

Land Use Designation	Undeveloped Dwelling Units	Developed Dwelling Units	Maximum Allowed Units
Residential Land Uses			
Senior Living-Medium Density Residential (MDR)	1060	2447	3507
Senior Living-High Density Residential (HDR)	906	104	1010
Low Density Residential (LDR)	25	154	179
Total	**1991**	**2705**	**4696**
Land Use Designation	**Acres Vacant**	**Acres Developed**	**Acres Total**
Commercial Land Uses			
Commercial Recreation	26	13	39.01
Office Commercial	44.2	11.9	56.1
Neighborhood Commercial	10	64	74
Total	**80.2**	**88.9**	**169.11**

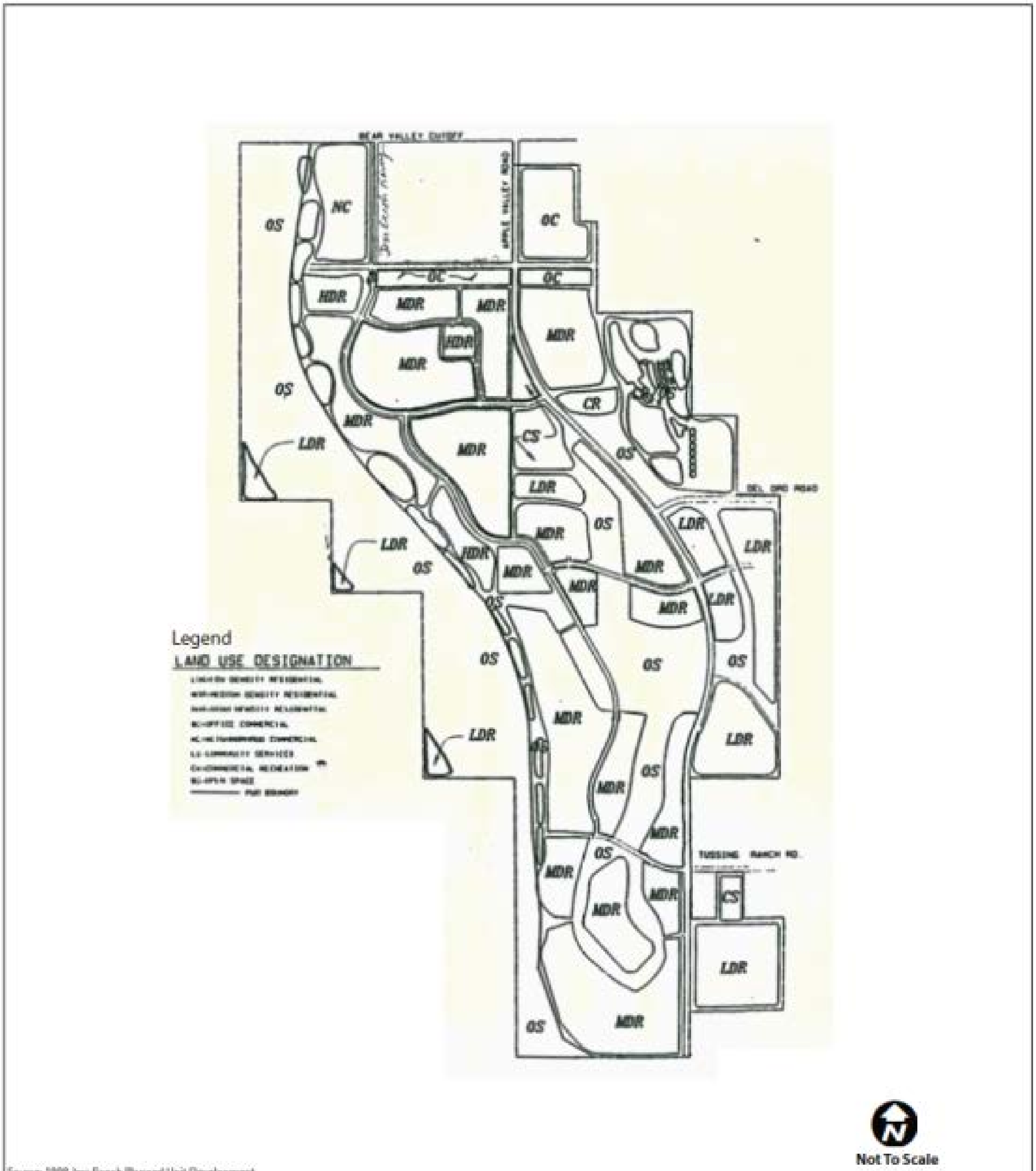
5.4.3.4 **Lewis Center for Educational Research**

The Lewis Center for Educational Research (LCER) Specific Plan outlines future development for the LCER school campus site which includes: K-12 School Zone (25 acres), College/University School Zone (11 acres), Flood Zone (7 acres), and Conservation Zone (107 acres), as shown in Figure 5-6. The area is bounded by Happy Trails Highway on the north, Owatonna Road in the east, and Mojave Narrows Regional Park to the south. Currently there is a developed K-12 campus that covers an area of approximately 10.5 acres. The land use designation buildout summary is listed in Table 5-8.

Table 5-8: Buildout Land Use Summary for LCER Specific Plan

Land Use Designation	Acres Vacant	Acres Developed	Acres Total	Existing Sq. Footage	Potential Sq. Footage	Total Sq. Footage
School Zone: 3-12	14.5	10.5	25	48,375	115,500	163,875
School Zone: College/University	11	-	11	-	120,000	120,000
Conservation Zone	107	-	107	-	-	-
Flood Zone	7	-	7	-	-	-
Total	**139.5**	**10.5**	**150**	**48,375**	**235,500**	**283,875**

Figure 5-5: Jess Ranch Planned Unit Development Specific Plan Use Map



Town of Apple Valley
Sewer Master Plan Update
Jess Ranch Planned Unit Development Specific Land Plan Use Map

Figure 5-5

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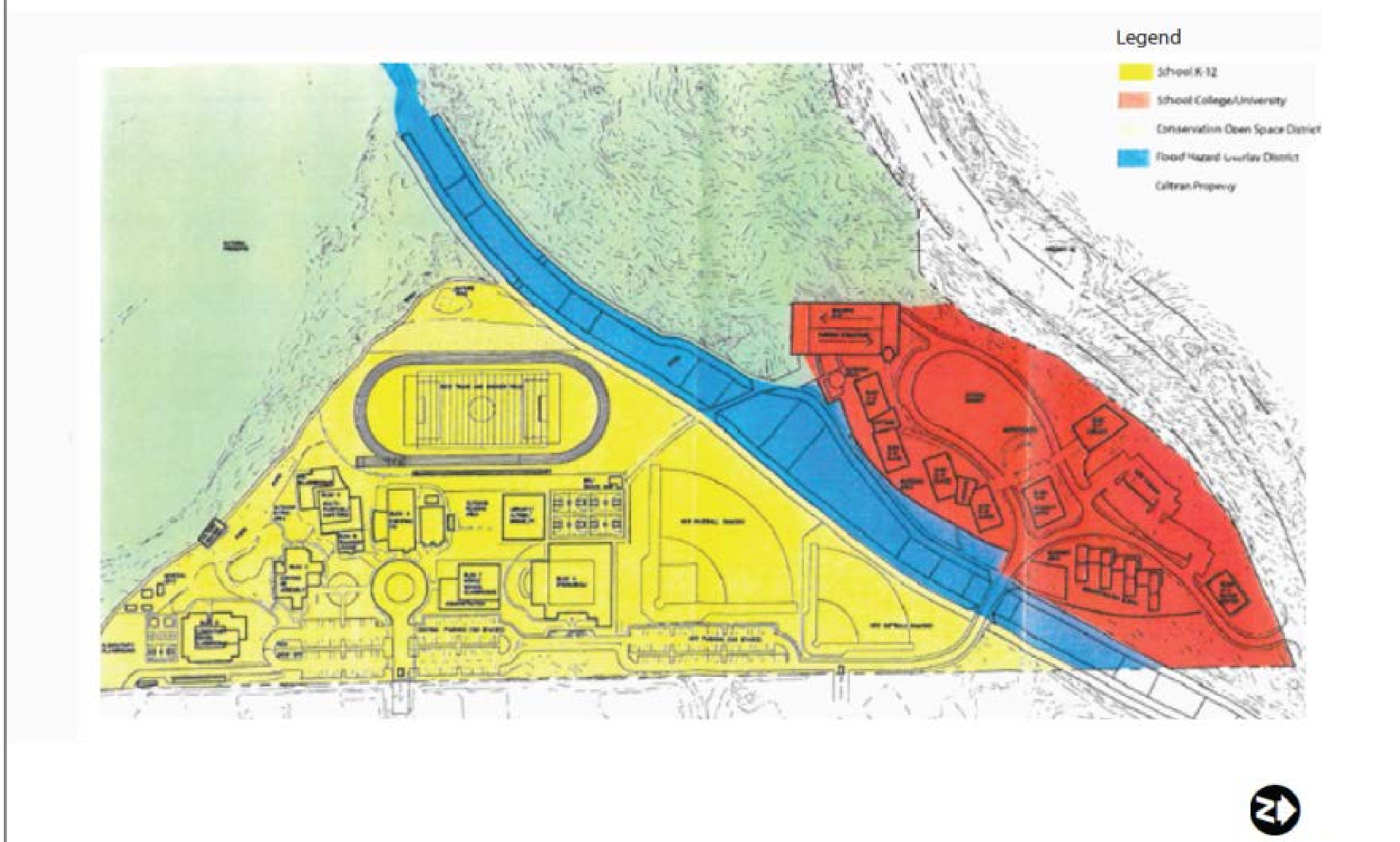
Figure 5-6: Lewis Center for Educational Research Specific Plan Land Use Map



Source: 2010 LCER Specific Plan

Town of Apple Valley
Sewer Master Plan Update

Lewis Center for Educational Research Specific Plan Land Use Map

Figure 5-6

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5.4.3.5 Meadowbrook Specific Plan

The Meadowbrook Specific Plan incorporates around 72 acres of mixed use development. The Specific Plan area is located at the southeast corner of Apple Valley Road and Yucca Loma Road, as shown in Figure 5-7. The development plan integrates residential, senior living and care facilities, commercial, and professional uses. Table 5-9 summarizes the approximate building and site areas for each development.

Table 5-9: Buildout Land Use Designations Summary for Meadowbrook Specific Plan

Land Use Designation	Vacant Acres	Developed Acres	Total Acres
Single Family Residential	42.3	-	42.3
Senior Living	9.7	-	9.7
Commercial	13.85	1.15	15
Public Facilities/School	4.03	-	4.03
Total	**69.9**	**1.15**	**71**

5.4.3.6 North Apple Valley Industrial Specific Plan

The North Apple Valley Industrial Specific Plan addresses long-term developmental goals, standards and guidelines for the 6,220 acres which includes the Airport Influence Area, the Dry Lake Flood Area, the Apple Valley Village Area located west of Central Avenue, Highway 18 Improvement Area, the I-15 Corridor, and the Bear Valley Road Improvement Area. The Specific Plan area is bounded by Langley Road on the north, Waalew Road on the south, Dale Evans Parkway on the west, and Central and Joshua Roads on the east, as shown in Figure 5-8. The Specific Plan includes industrial and commercial land uses for the area and the surrounding airport. The land use designation and buildout summary is listed in Table 5-10. The North Apple Valley Industrial Specific Plan incorporates Annexation 2008-002 which consists of approximately 809 acres. This area is located east of Central Road, south of Quarry Road, north of Lafayette Street, and west of Joshua Road.

Table 5-10: North Apple Valley Specific Plan Land Use Designation Buildout Summary

Designation	Vacant Acres	Developed Acres	Total Acres	Existing Sq. Footage[5]	Potential Sq. Footage[6]	Total Sq. Footage
General Commercial	265.7	4.9	270.6	46,958	2,546,256	2,593,214
Industrial - Airport	329.5	410.6	740.1	N/A	N/A	N/A
Industrial - Specific Plan	4445.2	343.3	4788.5	3,287,037	42,599,240	45,886,277
Industrial-General	334	6.1	340.1	58,458	3,200,789	3,259,246
High Desert Corridor	73.7	8	81.7	N/A	N/A	N/A
Total	**5,448.1**	**772.9**	**6,221**	**3,392,453**	**48,346,285**	**51,738,737**

[5] *Assumes that existing development, which is generally non-conforming under the Specific Plan, will be re-developed with up to 22% building coverage.*
[6] *Assumes new development at 22% building coverage.*

Figure 5-7: Meadowbrook Specific Plan Site Location

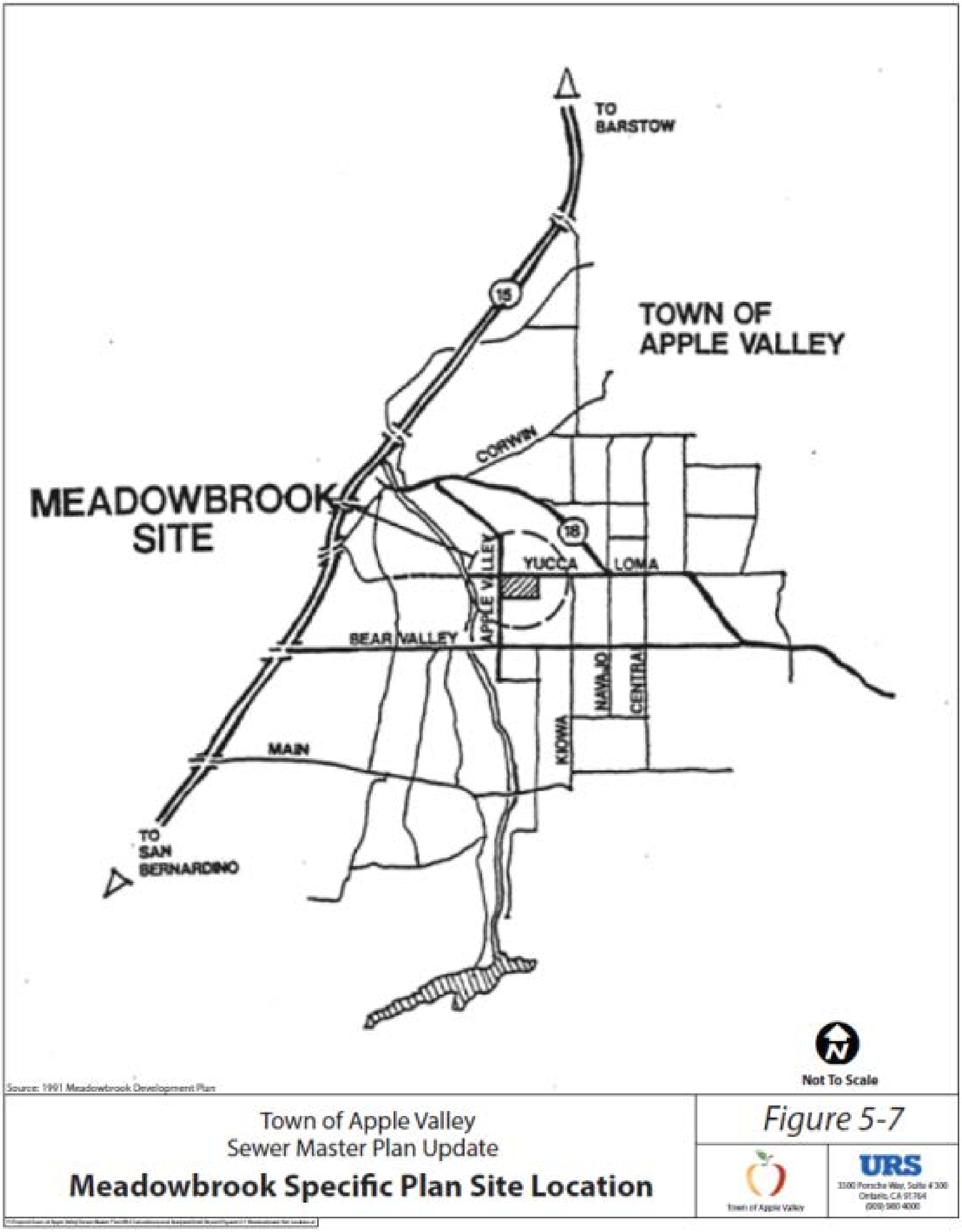
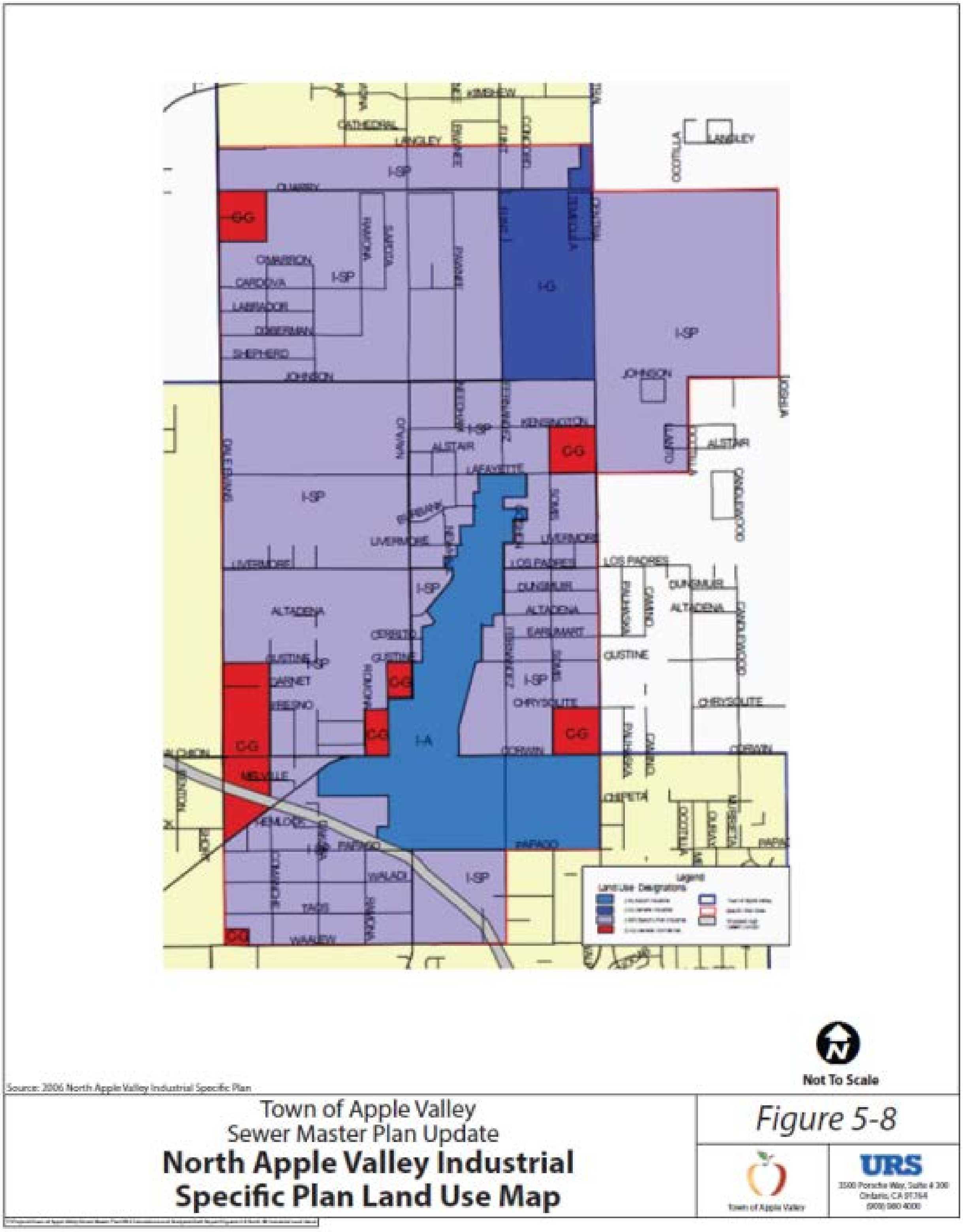


Figure 5-8: North Apple Valley Industrial Specific Plan Land Use Map



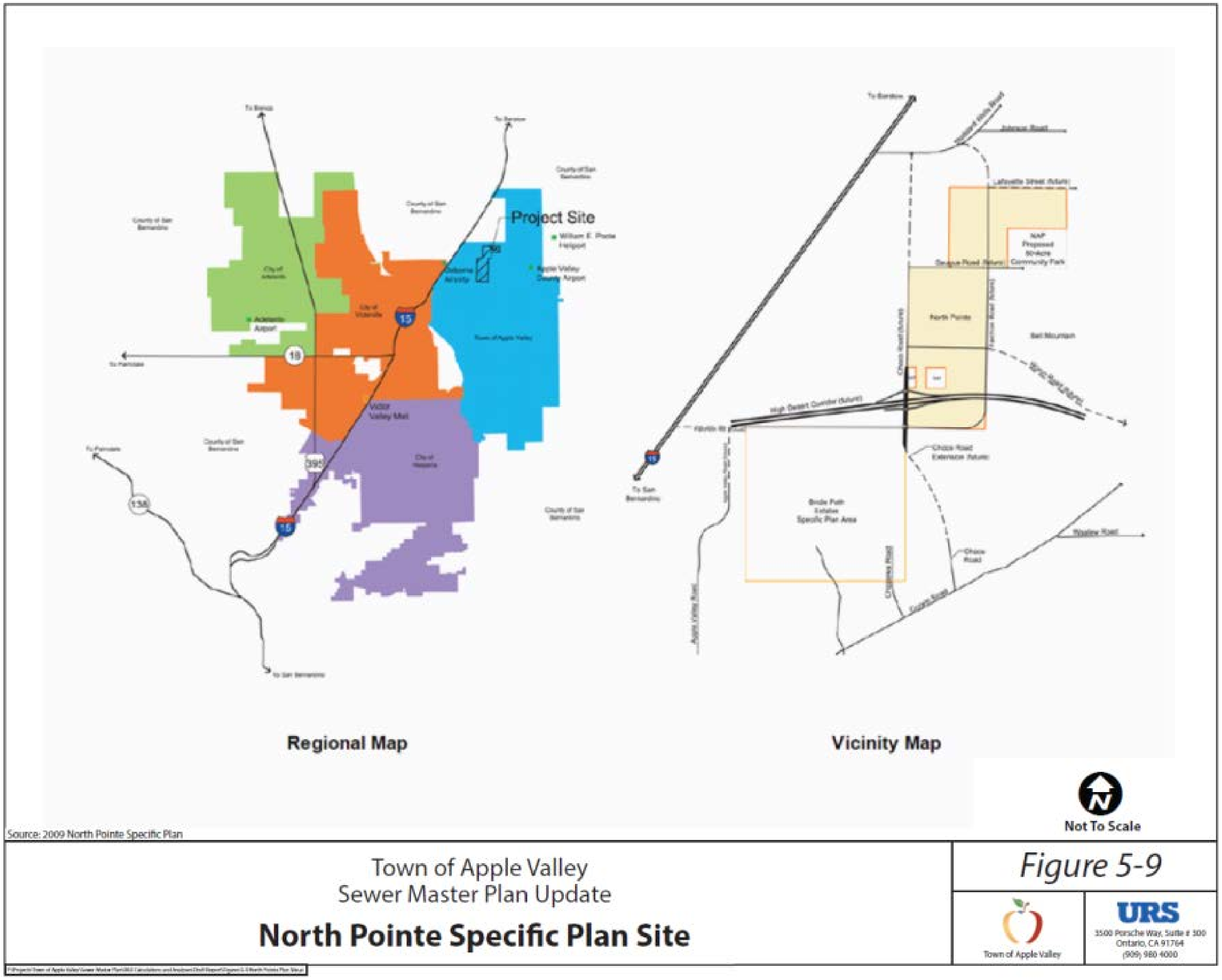
5.4.3.7 North Pointe Specific Plan

The North Pointe Specific Plan encompasses 485 acres of residential development, mixed use, commercial and retail land uses, open space, and multi-use trail system. The Specific Plan area is located west of Bell Mountain, west of Falchion Road (future road), mostly north of the High Desert Corridor (future road), and south of Johnson and Stoddard Wells Road, as shown in Figure 5-9. Table 5-11 includes the Specific Plan land use acreage summary based on land use categories.

Table 5-11: Land Use Acreage Summary for North Pointe Specific Plan

Land Use Designation	Approximate Gross Acreage	Approximate Dwelling Units	Approximate Square Feet
Single-Family Residential (R-SF) - half-acre minimum (18,000 square feet)	327	428	-
Single-Family Residential (R-SF) - from 21,000 to 45,000 square feet	-	90	-
General Commercial (C-G)	46	-	350,000
Mixed-Used (MU)	74	290	195,000
Open Space - Conservation (OS-C)	5	-	-
High Desert Corridor Right-of-Way	25	-	-
Retention Basins (Demonstration Gardens) (OS-C)	5	-	-
Open Space Trail	3	-	-
Approximate Total Gross Acreage of North Pointe Specific Plan Area	**485**	**808**	**545,000**
Not-A-Part (NAP) Properties	15	-	-
Approximate Overall Total Gross Acreage	**500**	**-**	**-**

Figure 5-9: North Pointe Specific Plan Site



5.4.3.8 Golden Triangle (Annexation 2008-001) Area

There are two annexation areas which fall within the Town's Sphere of Influence, as shown in Figure 5-2. The areas are Annexation 2008-001, also known as "Golden Triangle," and Annexation 2008-002. Annexation 2008-002 has been incorporated into the North Apple Valley Industrial Specific Plan. According to the Town, the Golden Triangle area will not be annexed, but the sewer flow generated in this area will still be delivered to the Northern Apple Valley Interceptor; therefore, the expected build out flows were also considered as part of this evaluation. The expected land use designations and total areas were obtained from the General Plan, and are summarized in Table 5-12.

Table 5-12: Golden Triangle's Statistical Summary of Land Uses Designation

Land Use Designation	Developed Acres	Vacant Acres	Total Acres	EDU
Residential Land Uses				
Estate Residential (1 du / 1-2.5 ac.)	55.7	722.3	778	1.48
Medium Density (4 -20 du / ac.)	41.5	177.3	218.8	10.42
Mixed Use (4 - 30 du / ac.)	-	94.8	94.8	9.03
Total	**97.2**	**994.4**	**1,091.6**	**20.93**
Commercial Land Uses				
Mixed Use	55.7	94.9	150.6	645.43
General Commercial	41.5	50.5	92	394.29
Regional Commercial	-	435.7	435.7	1,867.29
Office Professional	-	183.1	183.1	784.71
Total	**97.2**	**764.2**	**861.4**	**3,692**

5.4.4 Buildout Projections for the Specific Plan Sites

Buildout assessments and flow projections were made by considering the developments established on each of the Specific Plans. Table 5-13 contains a summary of land use, dwelling units (EDU) and total flow expected for all areas designated as a Specific Plan in Figure 2-2. The buildout flows were incorporated into the InfoWorks model as part of the buildout hydraulic evaluation.

Table 5-13: Build Out Summary of EDUs for Areas Designated as Specific Plan

Land Designation	Bridle Path Estates	Deep Creek Estates	Jess Ranch	LCER	Meadowbrook	North Apple Valley Industrial	North Pointe	Golden Triangle
Low Density Residential	-	-	25	-	-	-	-	-
Medium Density Residential	-	-	-	-	-	-	-	10.4
Estate Residential	-	-	-	-	-	-	-	1.5
Equestrian Residential	194	81	-	-	-	-	-	-
Single Family Residential	904	55	-	-	92	-	518	-
Commercial	-	-	344	-	24	334	46	3,692
Industrial	-	-	-	-	-	9,864	-	-
Public Facilities/School	-	-	-	86	1	-	-	-
Senior Living	-	-	1,966	-	250	-	-	-
Mixed Used	-	-	-	-	-	-	290	9
Total EDUs	**1,098**	**136**	**2,335**	**86**	**367**	**10,198**	**854**	**3,713**
Total Flow (gpd)	**230,580**	**28,560**	**254,379**	**18,000**	**47,472**	**2,141,531**	**179,322**	**779,655**

6.0 FLOW MONITORING

Flow and rainfall monitoring was not specifically performed for this project. However, VVWRA provided flow data for the period of September 8 through September 24, 2012 that was used to determine flows in the system, and rainfall dependent infiltration and inflow analysis (I/I).

6.1 INFLOW AND INFILTRATION

Infiltration occurs when groundwater flow enters the sanitary sewer system through pipe defects or leaky joints. Inflow occurs when storm water flows enter a sewer system through manholes or other surface components. Infiltration can be present during dry weather or wet weather, but inflow is only present during a storm event.

Because the groundwater table in Apple Valley is deep relative to the sewer system, groundwater infiltration does not occur during dry weather periods.

- *Average Dry Weather Flow (ADWF)* is defined as the wastewater normally transported in the sewer system exclusive of inflow and infiltration (I/I). The instantaneous wastewater production flow rate varies throughout each day, with the highest normally between 7:00 a.m. and 11:00 a.m., depending on the location within the sewer system. The ratio of peak flow to the ADWF is defined as the diurnal flow peaking factor.
- *Peak Dry Weather Flow (PDWF)* is defined as the peak flow during an average day.
- *Infiltration* is part of groundwater which enters the sanitary sewer system through defective pipes, pipe joins, and manhole structures below the manhole cone. Infiltration rate depends on the depth of groundwater above the defects, the size of defects, and the percentage of the collection system submerged. Variation in groundwater levels and the associated infiltration are seasonal and weather-dependent. Base infiltration occurs during dry weather/low groundwater conditions. High groundwater (total) infiltration occurs during high groundwater conditions following rain events.
- *Inflow* is rainfall-related water which enters the sanitary sewer system from sources such as private sewer laterals, downspouts, foundation drains, yard and area drains, stormwater sump pumps, manholes, defective piping, and cross-connections from storm drains. The quantity of inflow is directly influenced by the intensity and duration of a storm event. Therefore inflow is not a fixed quantity.

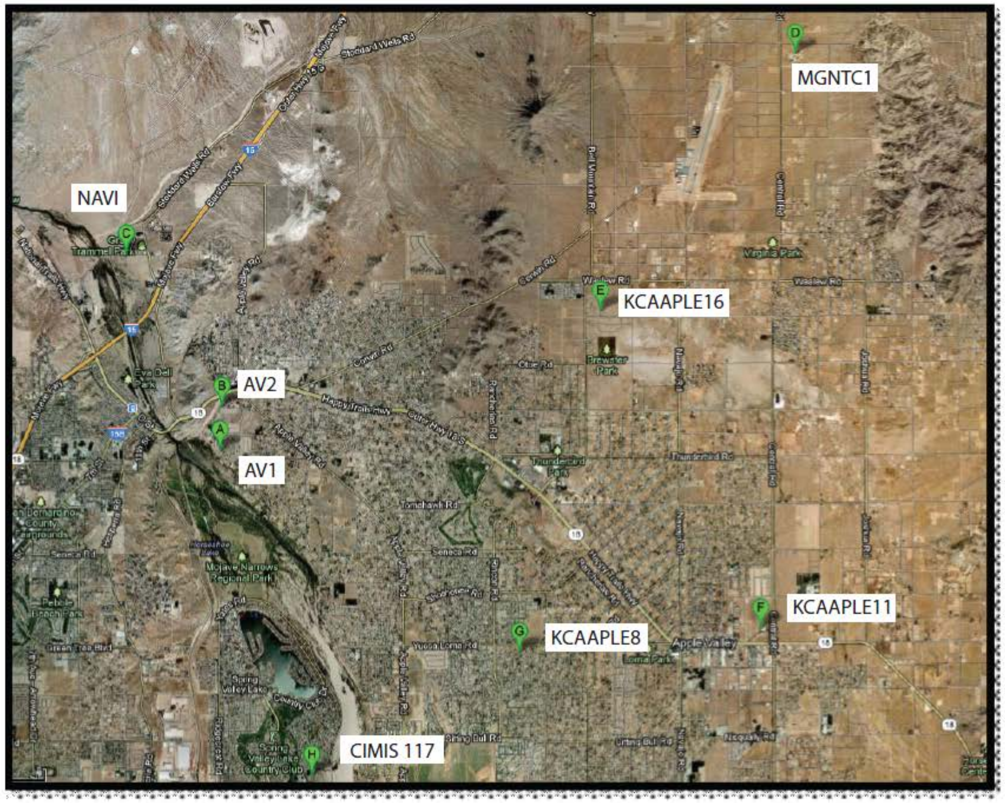
6.2 Flow Monitoring Locations

Rainfall and flow monitoring locations used in this Master Plan are shown in Figure 6-1. Table 6-1 provides a summary of the characteristics of the flow monitoring locations.

Table 6-1: Flow Monitoring Locations

Site Number	Site Name	Measured Diameter (in)
1	AV No. 1 (MH8)	14
2	AV No. 2	18
3	AV North (NAVI)	23

Figure 6-1: Flow and Rainfall Monitoring Site



Not To Scale

Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update

Flow and Rainfall Monitoring Site

Figure 6-1

Town of Apple Valley

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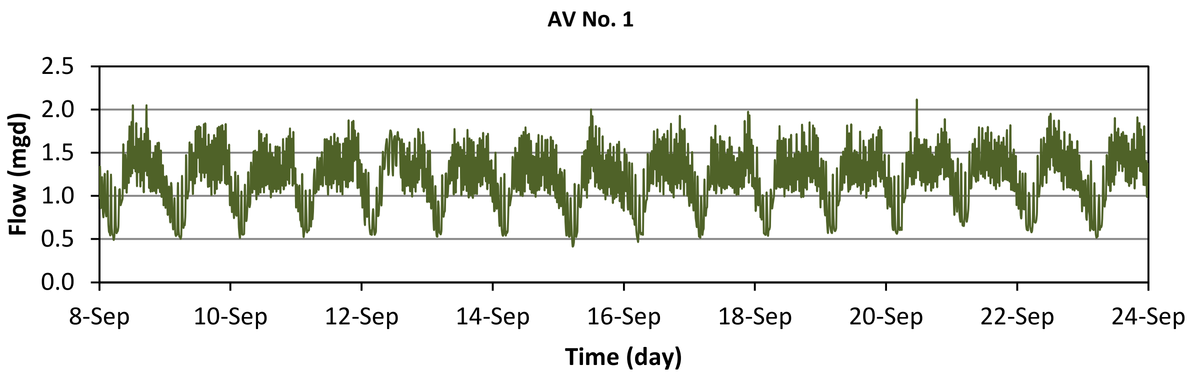
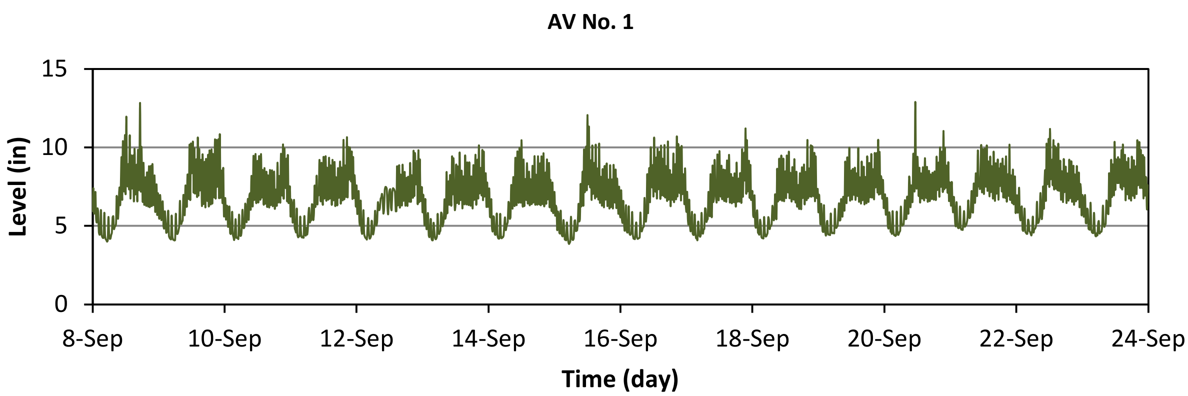
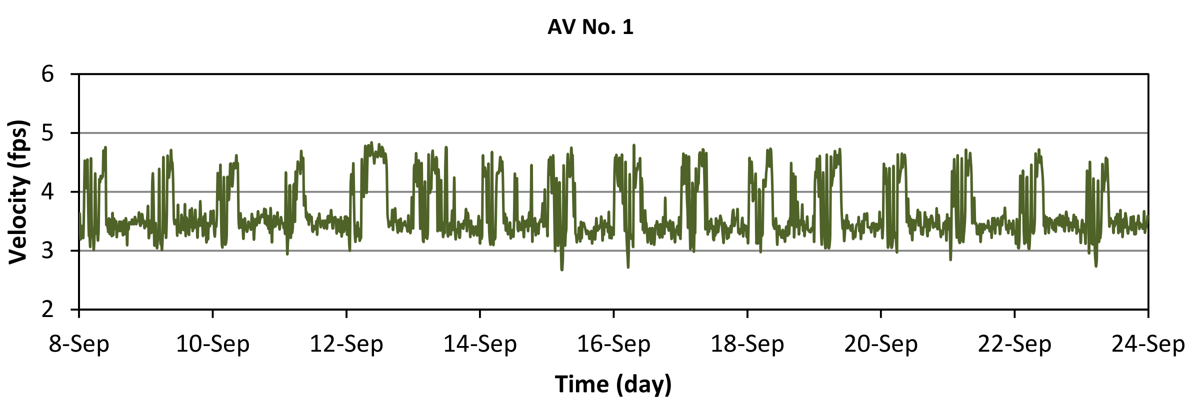
6.2.1 Flow Monitor Site No. 1

Monitoring Site No. 1, identified as Apple Valley (AV) No. 1, was located on a 14 inch diameter pipe. Figure 6-2 shows the flow, level, and velocity data recorded between September 8 and September 24, 2012. The average flow is 1.20 mgd, peak flow is 2.12 mgd, and peaking factor is 1.8.

Figure 6-2: Flow, Depth and Velocity at Monitoring Site No.1 (AV-01)



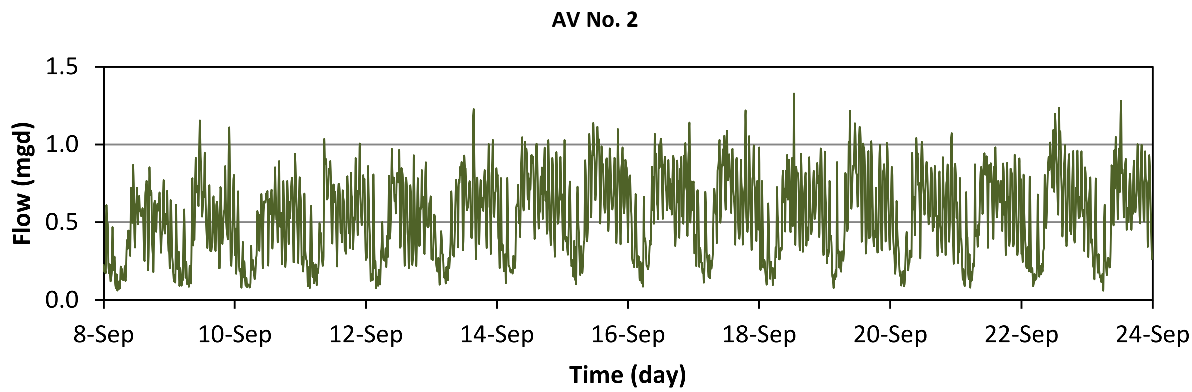


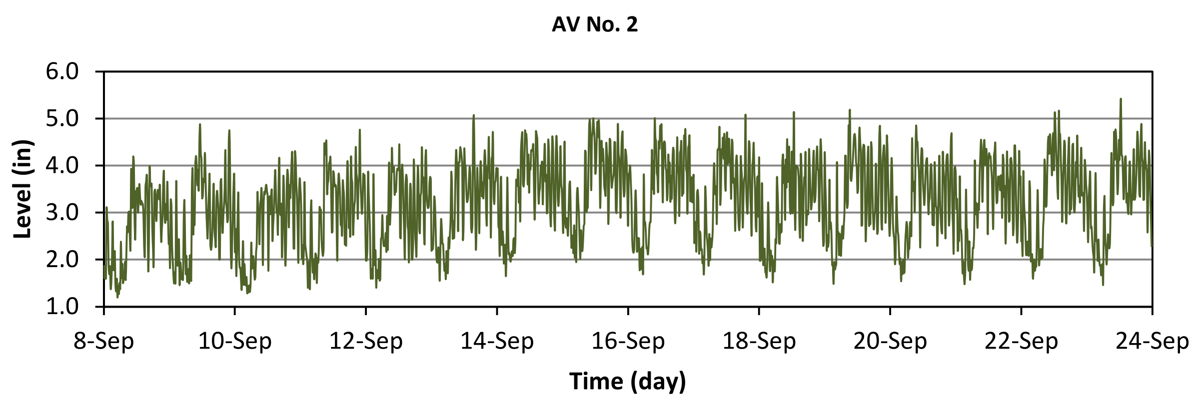


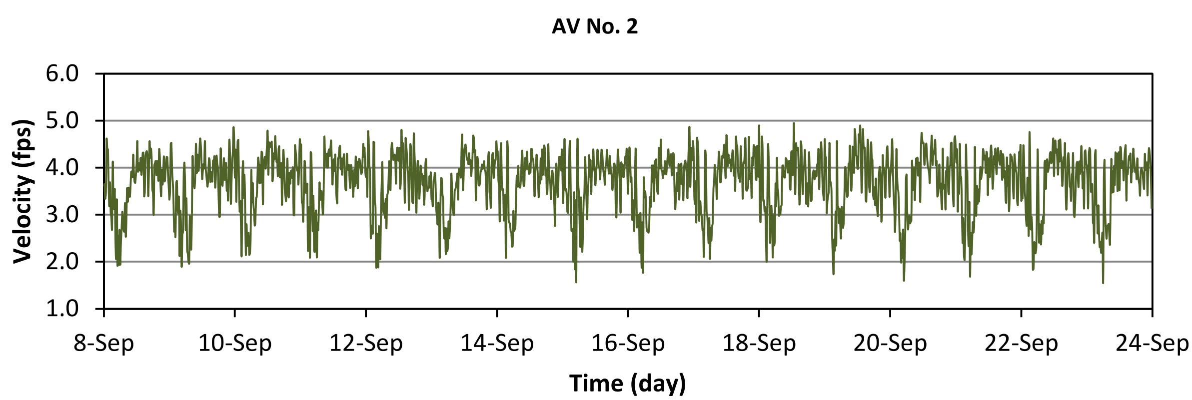
6.2.2 Flow Monitor Site No. 2

Monitoring Site No. 2, identified as AV No. 2, was located on an 18 inch diameter pipe. Figure 6-3 shows the flow, level, and velocity data recorded between September 8 and September 24, 2012. The average flow is 0.54 mgd, peak flow is 1.33 mgd, and peaking factor is 2.4. The level sensor for this site appears to have drifted from September 12 through September 20.

Figure 6-3: Flow, Depth and Velocity at Monitoring Site No.2 (AV-02)



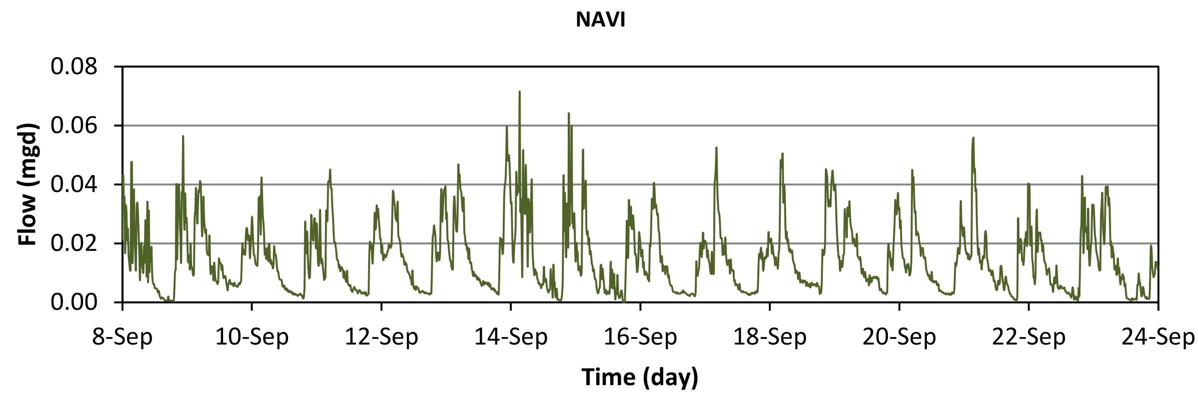


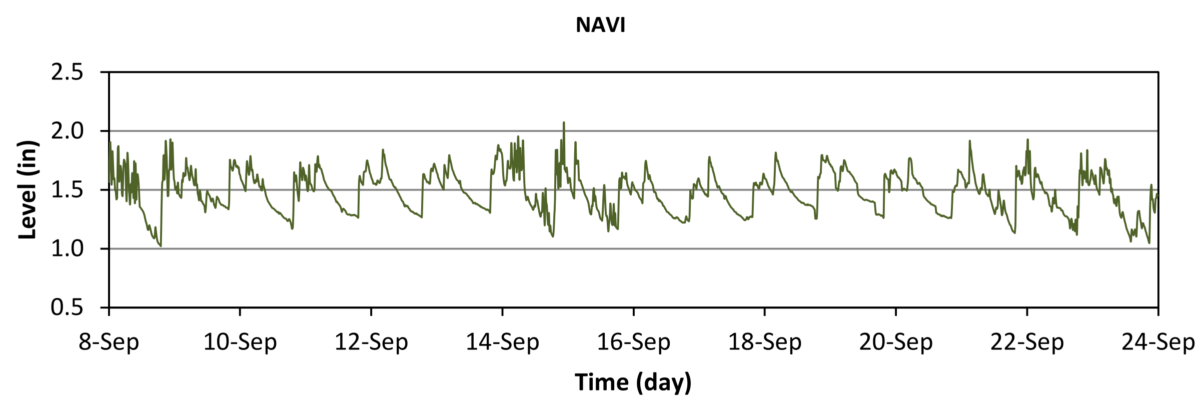


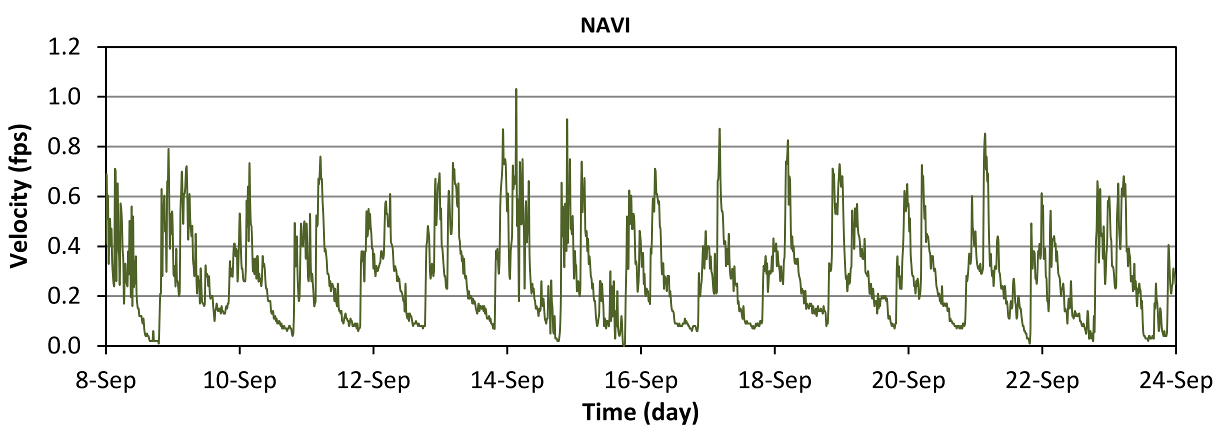
6.2.3 Flow Monitor Site No. 3

Monitoring Site No. 3, identified as NAVI, was located on a 23 inch diameter pipe in the City of Victorville. Figure 6-4 shows the flow, level, and velocity data recorded between September 8 and September 24, 2012. The average flow is 0.02 mgd, peak flow is 0.07 mgd, and peaking factor is 4.8.

Figure 6-4: Flow, Depth and Velocity at Monitoring Site No. 3 (NAVI)





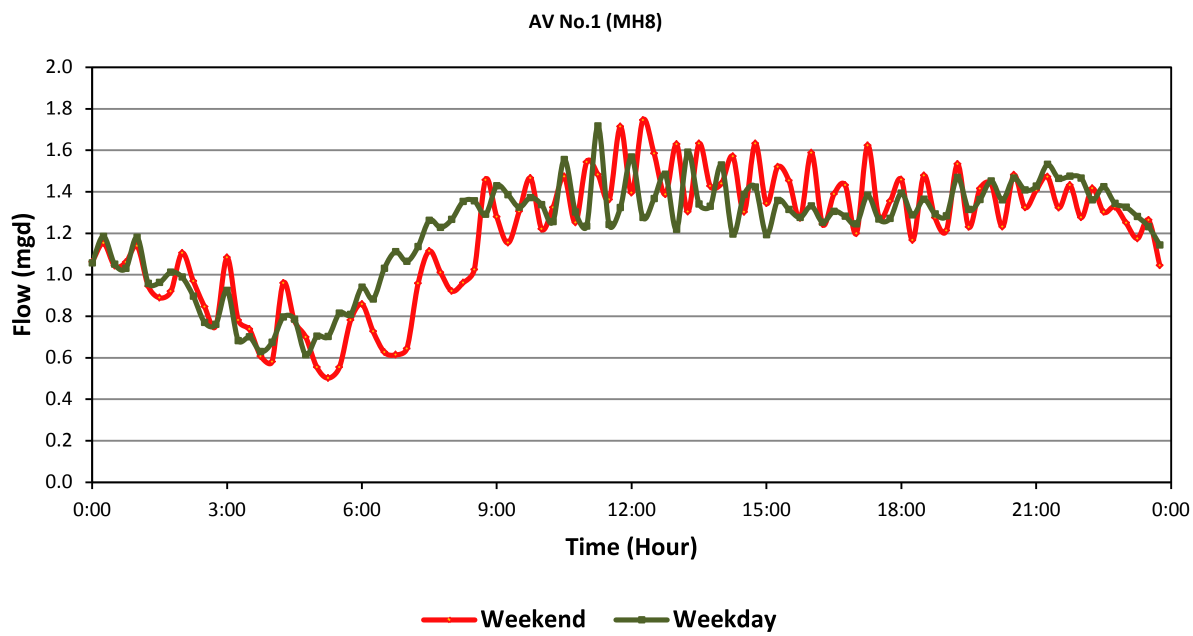


6.3 AVERAGE DRY WEATHER FLOW

Average Dry Weather Flow (ADWF) is the baseline, or average daily flow. ADWF occurs during dry weather condition, or when there is no rainfall event. This flow includes sanitary wastewater generated from residential, commercial, and industrial users, plus any applicable baseline groundwater flow.

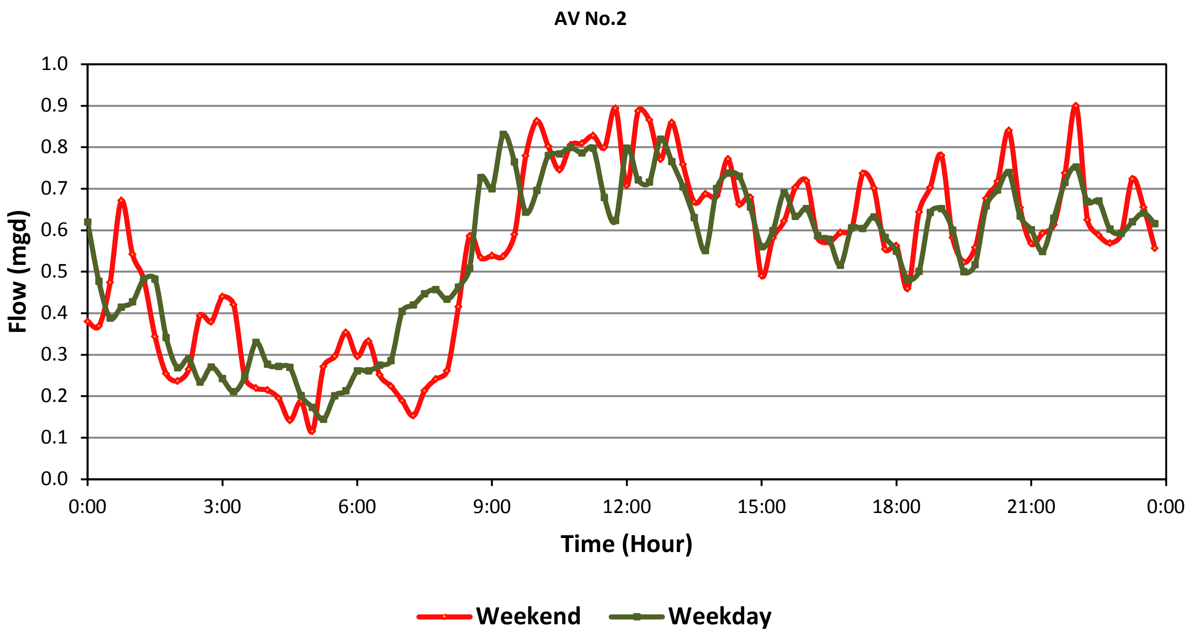
The ADWF curves shown in Figures 6-5, 6-6, and 6-7 were produced from the flow readings in a 15 minute time interval throughout the day. From this data, average flow value for each time period was then calculated for both weekday and weekend.

Figure 6-5: Average Dry Weather Flow at Monitoring Site No.1



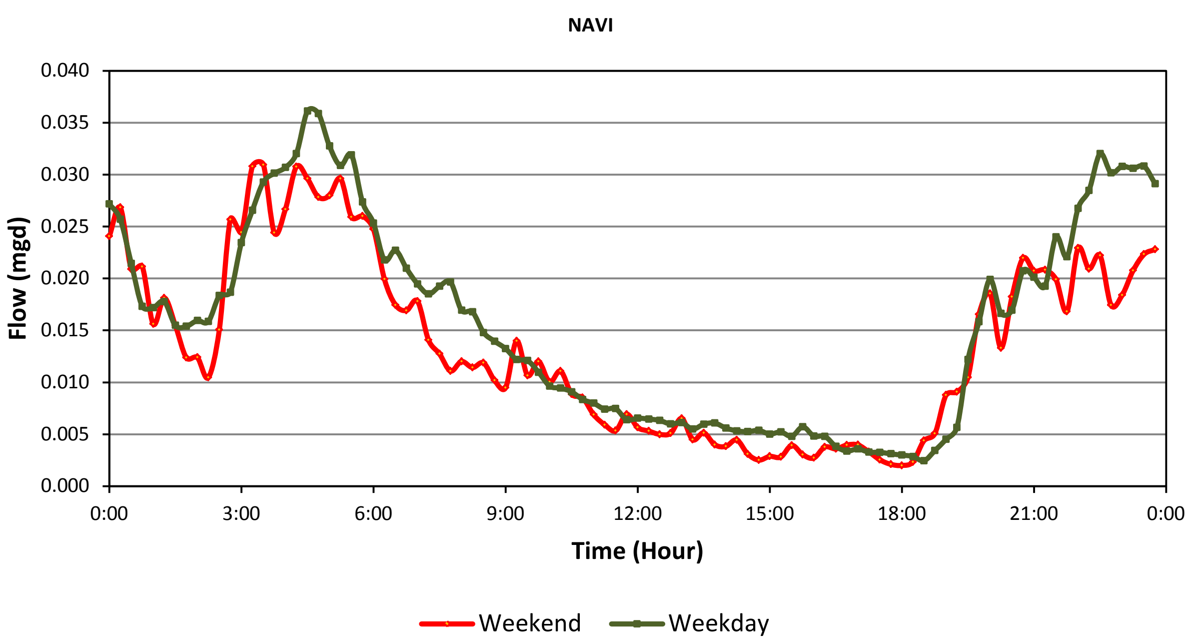
In Figure 6-5, ADWF for both weekday and weekend are shown for Monitoring Site No.1. The weekday flow shows an average flow of 1.21 mgd and a peak dry weather flow of 1.72 mgd. The weekend flow shows an average flow of 1.19 mgd and a peak dry weather flow of 1.75 mgd.

Figure 6-6: Average Dry Weather Flow at Monitoring Site No. 2



In Figure 6-6, ADWF for both weekday and weekend are shown for Monitoring Site No.2. The weekday flow shows an average flow of 0.54 mgd and a peak dry weather flow of 0.83 mgd. The weekend flow shows an average flow of 0.55 mgd and a peak dry weather flow of 0.90 mgd.

Figure 6-7: Average Dry Weather Flow at Monitoring Site No. 3



In Figure 6-7, ADF for both weekday and weekend are shown for Monitoring Site No.3. The weekday flow shows an average flow of 0.02 mgd and a peak flow of 0.04 mgd. The weekend flow shows an average flow of 0.01 mgd and a peak flow of 0.03 mgd.

6.4 PEAK WET WEATHER FLOW

Peak Wet Weather Flow (PWWF) is the result of inflow from rainfall events and an increase in infiltration. The PWF is used to assist engineers in designing sanitary sewer collection systems, lift stations, and trunk sewer systems.

There was no considerable amount of rainfall in Apple Valley during the monitoring period. Multiple weather stations were analyzed surrounding the monitoring sites and only one station located at the northern portion of the Town registered rainfall amounts, station MGNTC1. Figure 6-1 shows an overview of the monitoring and rainfall station locations.

Wastewater flows were monitored at three monitoring sites: AV No. 1, AV No. 2, and NAVI. The monitored data provided by VVWRA was compared against rainfall data that occurred during the period of September 8 through September 24, 2012. Rainfall data, listed in Table 6-2, was obtained for the period of September 8, 2012 through September 11, 2012 and compared to the flow of each of the monitored site to determine if there were any deviations from the expected average flows.

Table 6-2: Monitoring Daily Rainfall for MGNTC1 Station

Date	Daily Rainfall (in)
9/8/2012	0.06
9/9/2012	0.06
9/10/2012	1.02
9/11/2012	0.13
Total	**1.27**

Rainfall station MGNTC1 is located approximately 10 miles northeast of Apple Valley No. 1, Apple Valley No. 2, and the NAVI monitoring sites.

Figure 6-8: Comparison between Flow, Level, and Velocity Data vs Rainfall data at AV1

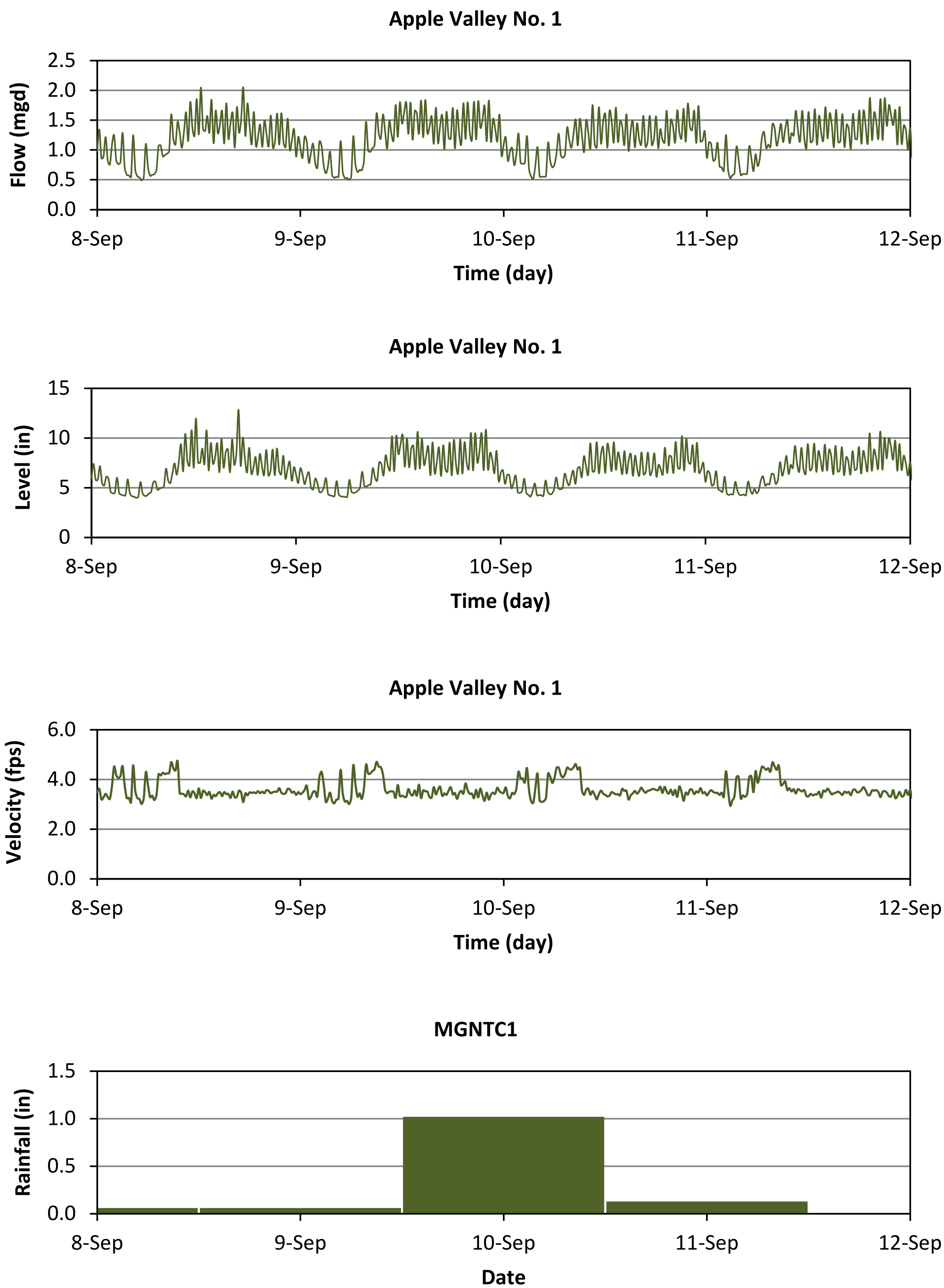
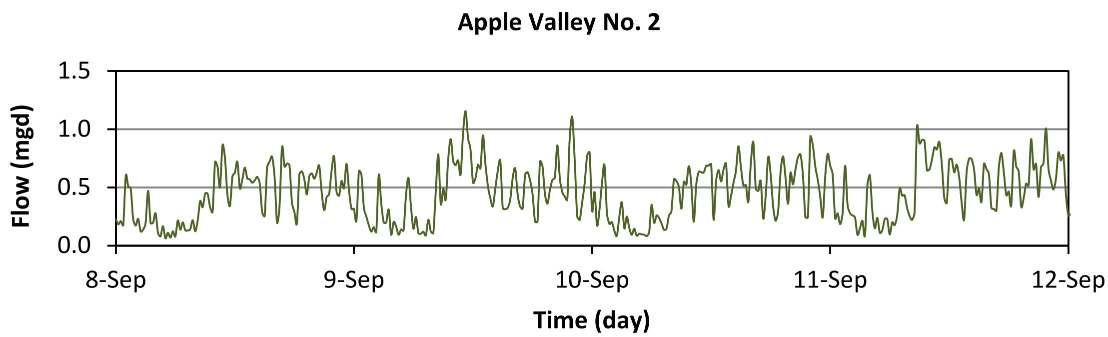
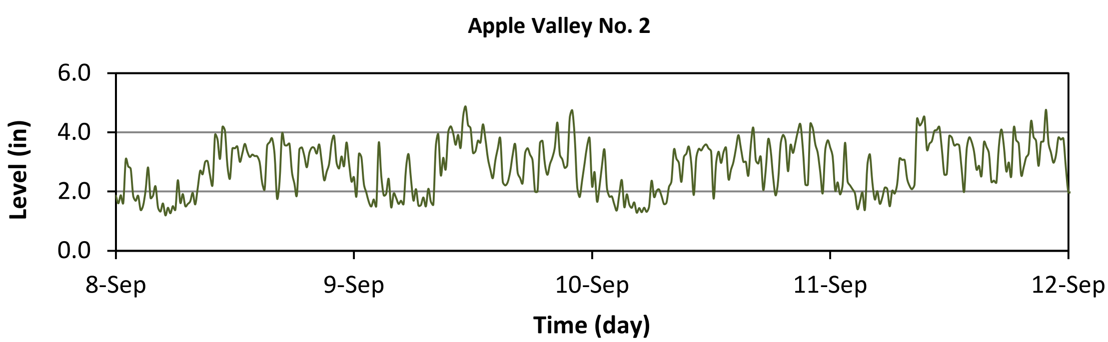
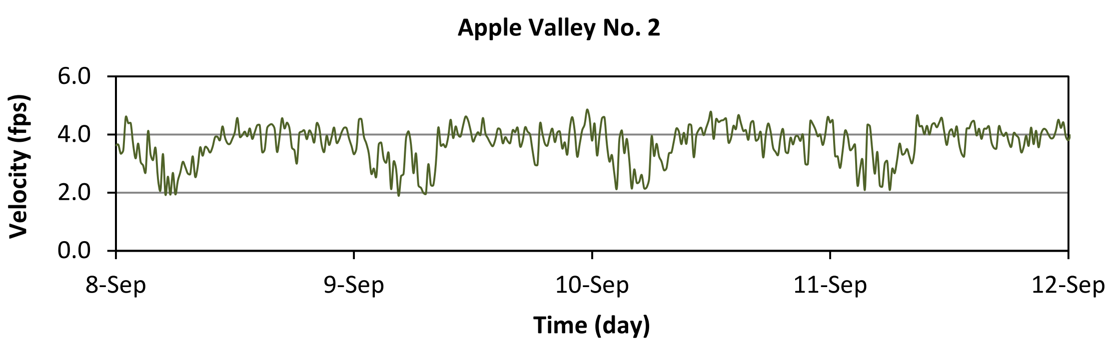


Figure 6-9: Comparison between Flow, Level, and Velocity Data vs Rainfall data at AV2







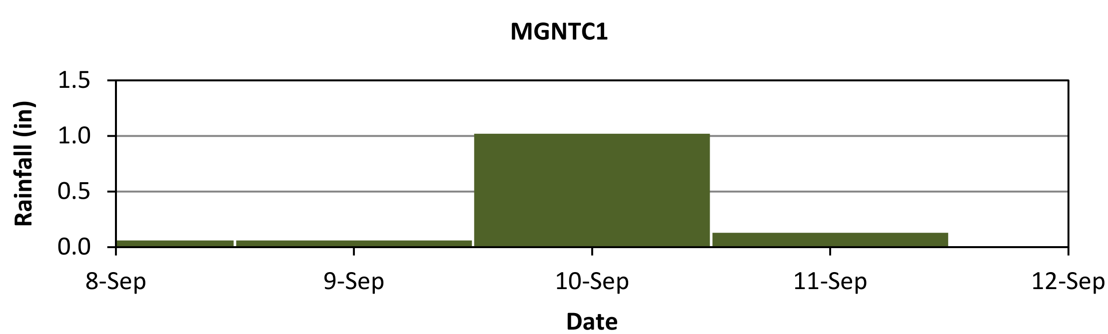
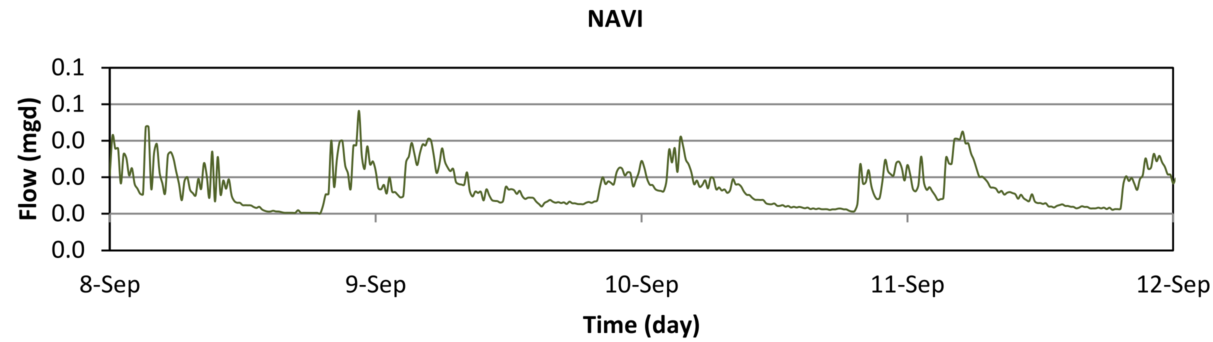
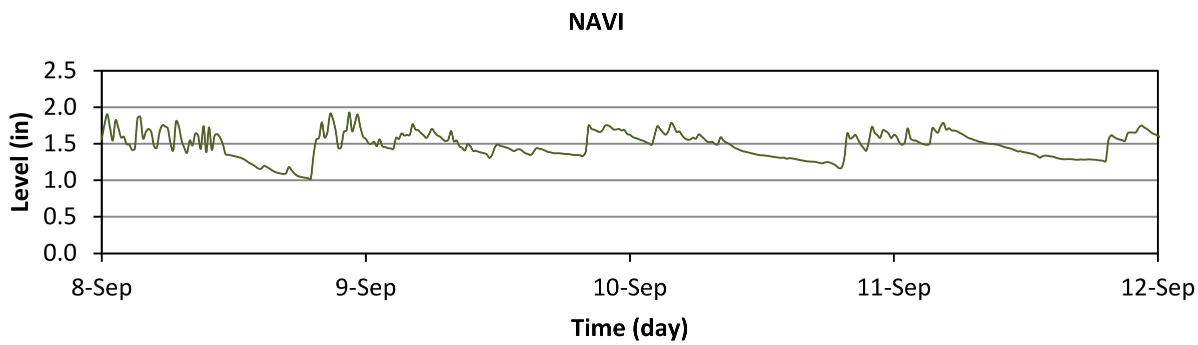
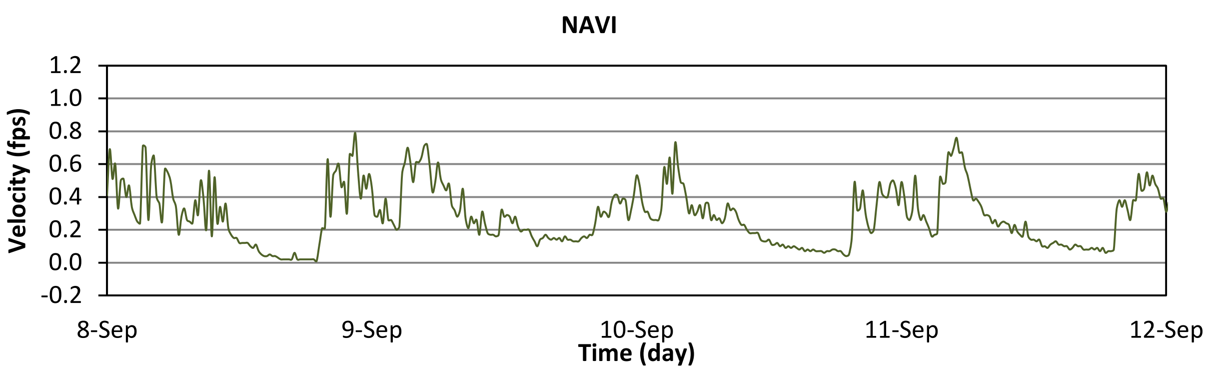


Figure 6-10: Comparison between Flow, Level, and Velocity Data vs Rainfall data at NAVI







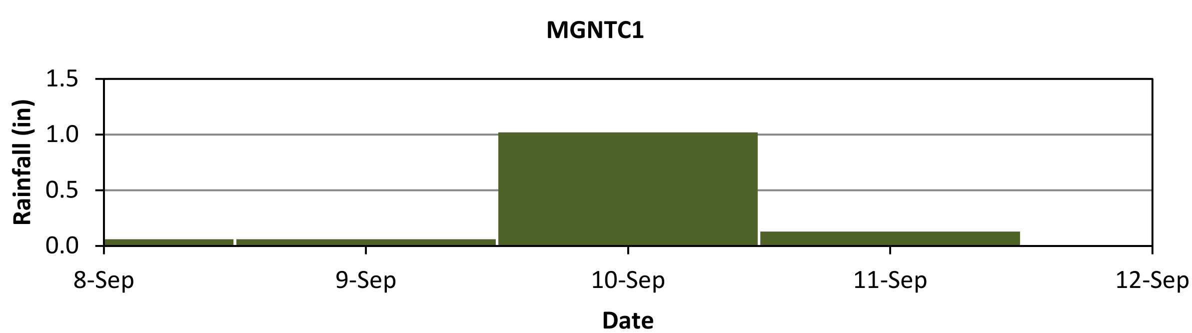


Figure 6-8 through 6-10 show wastewater flows in the sewer system versus rainfall for the period of September 8 through September 11, 2012. The total rainfall for this period was 1.27 inches. As seen from the figures, there seems to be no significant deviations from the average sewer flow suggesting that there is no inflow at the monitoring sites. Due to the Town's dry weather climate, sewer system evaluations will be based on dry weather flows.

6.5 DIURNAL CURVE

The ratio of the average daily flow to total average daily flow is defined by the wastewater production diurnal curve. The diurnal curves were produced to display how the system flow varies throughout the day. Figures 6-11 through 6-13 display the diurnal curves for the three monitoring sites.

Figure 6-11: Diurnal Curve for Monitoring Site No. 1

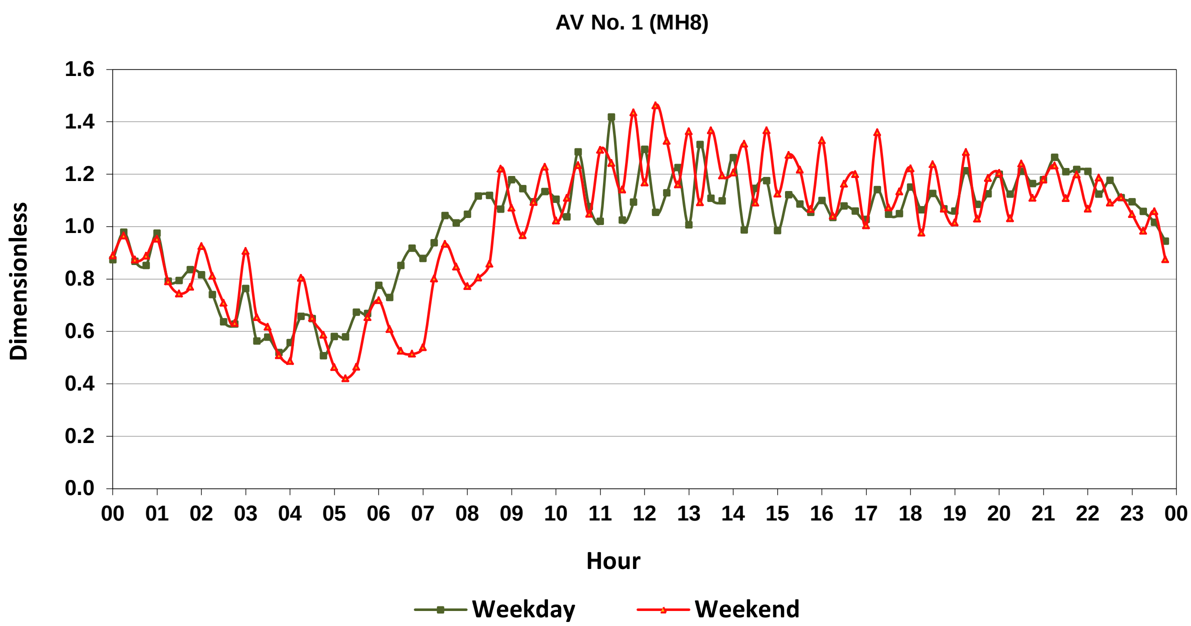


Figure 6-11 shows the diurnal curve for Monitoring Site No. 1. Overall, the peaking factors for the weekday and weekend are 1.42 and 1.46, respectively.

Figure 6-12: Diurnal Curve for Monitoring Site No. 2

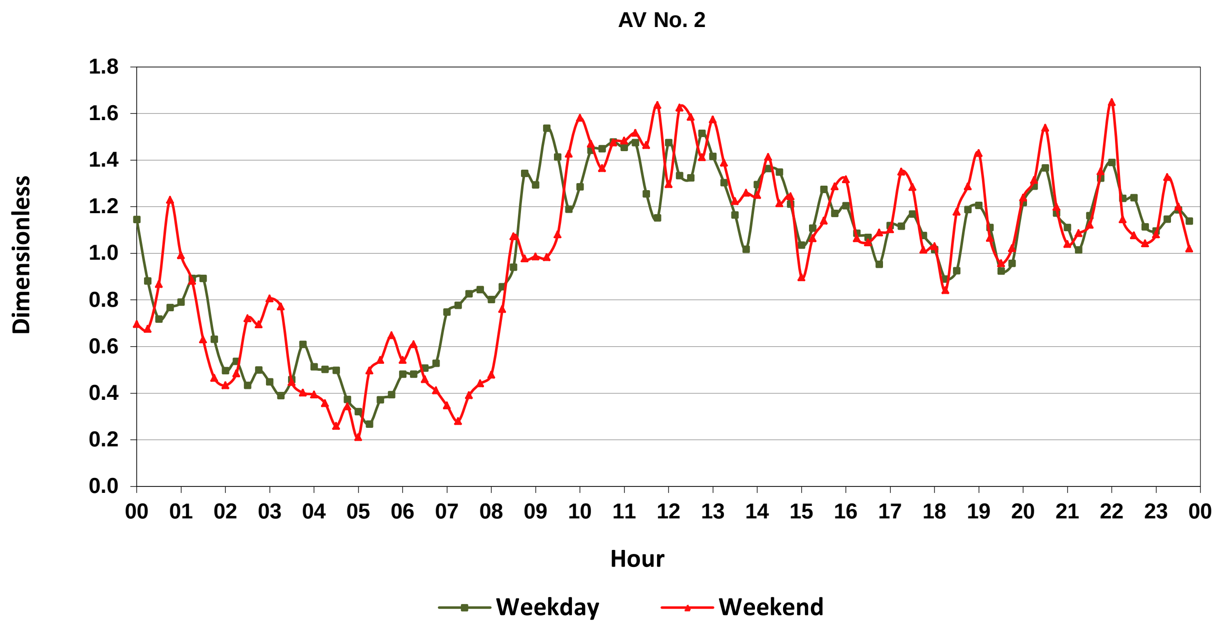


Figure 6-12 shows the diurnal curve for Monitoring Site No. 2. Overall, the peaking factors for the weekday and weekend are 1.54 and 1.65, respectively.

Figure 6-13: Diurnal Curve for Monitoring Site No. 3

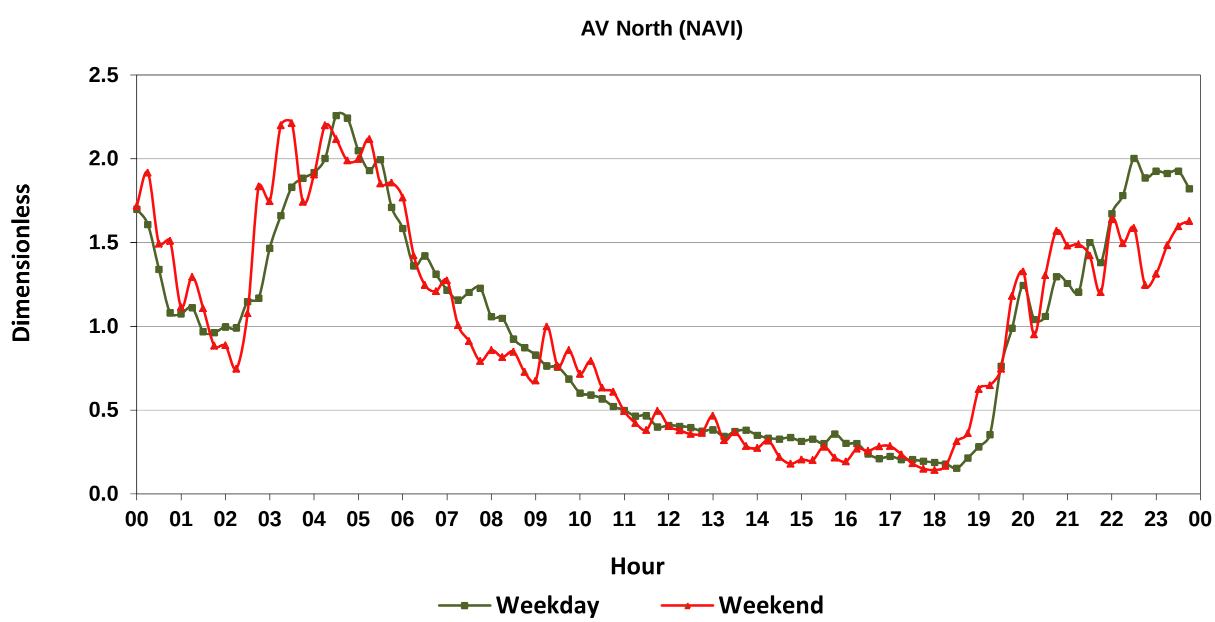


Figure 6-13 shows the diurnal curve for NAVI. Overall, the peaking factors for the weekday and weekend are 2.26 and 2.21, respectively.

6.6 Summary of Flows at Monitoring Sites

Table 6-3 shows the summary of the average day and peak flows for the September 9 through September 24, 2012 flow monitoring period.

Table 6-3: Flow Monitoring Data for September 9 through September 24, 2012

Site	Average Day Flow (mgd)	Peak Flow (mgd)
Site 1—AV No. 1	1.21	1.72
Site 2—AV No. 2	0.54	0.83
Site 3—NAVI	0.02	0.04
Total	**1.77**	**2.59**

6.7 VVWRA Billing Flows

Table 6-4 shows the average day flows per VVWRA billing.

Table 6-4: Average Day Flows per VVWRA Billing

VVWRA Interceptor (mgd)	NAVI (mgd)	Total (mgd)
1.73	0.006	1.736

7.0 EXISTING AND FUTURE FLOW DEVELOPMENT

Flow analysis is critical to modeling the existing and proposed sewer systems for any master plan. Flow analysis utilizes data from the existing land use and population, the General Plan, trends in water usage, and data for proposed facilities.

The first step in flow analysis is the establishment of a base flow. This base flow is an average wastewater generated from a household. Once this base line was created, the peaking factors were determined from the diurnal curves.

7.1 EXISTING SYSTEM

To determine flows for the existing system, URS used the base parcel maps. Also included on these maps were the existing sewer system layout that URS created from the Town's as-built plans. URS utilized Bing map to determine how many houses were tributary to each manhole in the Town's sewer system. The same approach was used to determine the commercial and industrial developments contributory to each manhole. The approach for estimating the residential, commercial and industrial flows are discussed below.

7.1.1 Residential Flows

URS evaluated flows for the Town of Apple Valley by assigning a base flow for each sewered house tributary to the Town's sewer system. To quantify the base flows, URS used the principle of a base flow standard unit, called an equivalent dwelling unit, or EDU. One EDU is equal to a single family residence. Per previous master plan, "an equivalent dwelling unit is generally defined as having 20 plumbing fixture units. A typical 3-bedroom, 2-bathroom house is considered as one EDU. Average day and peak wastewater flow are related by peaking factors as identified in the County of San Bernardino Special Districts Department *Standards for Sanitary Sewers*."

URS determined the number of properties (residential, commercial, and industrial) that exist in each polygon by using the Bing aerial map. Based on the aerial view maps, URS was able to count the existing homes in each polygon and calculate the total EDUs. For those units that were determined to be high-density residential, the flows were determined by using the land use designations listed in the Town's Land Use Map. The vacant lots were excluded from the model. Once the total number of households was found, the total EDUs were calculated and entered into the GIS database, and exported to the InfoWorks model.

7.1.2 Commercial Flows

Flows for the commercial areas were calculated as acreage of commercial building space, and then converted to EDU's. The Town's commercial criterion for flows is 1,200 gallons per day per acre. To convert commercial building space acreage into EDU's, the ratio of 1,200 gallons per day per acre was divided by 210 gallons per day per EDU, or 5.71 EDU per acre. Average daily flows used in the hydraulic model were calculated by multiplying cumulative tributary EDU flows by 210 gallons per day per EDU, and converted to gallons per minute.

7.1.3 Industrial Flows

All flows for the industrial areas were calculated as acreage of building space, and then converted to EDU's. The Town's industrial criterion for flows is 1,500 gallons per day per acre. To convert industrial building space acreage into EDU's, the ratio of 1,500 gallons per day per acre was divided by 210 gallons per day per EDU, or 7.14 EDU per acre. Average daily flows used in the hydraulic model were calculated by multiplying cumulative tributary EDU flows by 210 gallons per day per EDU, and converted to gallons per minute.

7.1.4 Largest Dischargers

The Town of Apple Valley provided a list of the largest wastewater dischargers, as shown in Table 7-1, based on establishments having 400 or more fixtures. Discharges for these users were converted to EDU's using 20 Plumbing Fixture Units (PFU) per EDU.

Table 7-1: Largest Wastewater Dischargers

School Name	Address	PFU'S	EDU'S	GPD
Rio Vista Elementary School	13590 Havasu Rd	445	22.25	4,673
Mesquite Elementary School	12951 Mesquite Rd	460	23	4,830
Sitting Bull Elementary School	19355 Sitting Bull Rd	554	27.7	5,817
Apple Valley Early Education Center	18415 Nakash	637	31.85	6,689
High Desert Premier (Former Apple Valley Middle School)	12555 Navajo Rd	688	34.4	7,224
Rancho Verde Elementary School	14334 Pioneer Rd	864.4	43.22	9,076
Yucca Loma Elementary School	21351 Yucca Loma Rd	1,050	52.5	11,025
Sitting Bull Middle School	19445 Sitting Bull Rd	1,098	54.9	11,529
Granite Hills High School	22900 Esaws Rd	2,072	103.6	21,756
Lewis Center	17500 Mana Rd	784	39.2	8,232
Total			**432.62**	**90,850**
Mobile Home Park Name	**Address**	**PFU'S**	**EDU'S**	**GPD**
Apple Valley Mobile Home Park	21923 Ottawa Rd	760	38	7,980
Apple Valley Ranchos Mobile Home Park	21922 Ottawa Rd	820	41	8,610
Pioneer Mobile Home Park	13892 Pioneer Rd	840	42	8,820
Mountain View Villas	21621 Sandia Rd	1,000	50	10,500
Santiago Apple Valley Estates	22020 Nisqually Rd	1,540	77	16,170
Apple Valley Mobile Home Lodge	22325 Highway 18	2,120	106	22,260
Vista Del Rosa	22241 Nisqually Rd	2,900	145	30,450
Total			**499**	**104,790**
Business Name	**Address**	**PFU'S**	**EDU'S**	**GPD**
Stater Bros. Markets	12253 Apple Valley Rd	404	20.2	4,242[7]
Ultra Star	22311 Bear Valley Rd	428	21.4	4,494
Cinemark	18935 Bear Valley Rd	430	21.5	4,515
Winco Foods	19047 Bear Valley Rd	434	21.7	4,557[7]
Animal Shelter	22131 Powhatan	434	21.7	4,557[7]
Victor Valley College Regional Public Safety Training Facility	19190 Navajo Rd	440	22	4,620
Albertson's	20261 Highway 18	476	23.8	4998[7]

Corwin Medical Center	18523 Corwin Rd	509	25.45	5,345
Apple Valley Retirement Care Center	11959 Apple Valley Rd	608	30.4	6,384
Super Target	20288 Highway 18	668	33.4	7,014[7]
Walmart Distribution Center	21101 Johnson Rd	688	34.4	7,224[7]
Merrill Gardens	11825 Apple Valley Rd	1,094	54.7	11,487
Apple Valley Retirement Center	20594 Bear Valley Rd	1,401	70.05	14,711[7]
Juvenile Detention Center	21101 Dale Evans Pkwy	2,013	100.65	21,137
St. Mary's Hospital	18300 Highway 18	4,284	214.2	44,982
Total			**715.55**	**150,266**

[7]EDU's not given, but ratio of 20 PFU per EDU was used for consistency.

7.1.5 Existing System Peak Flows

Table 7-2 shows the model peak flows under existing conditions.

Table 7-2: Modeled Peak Flow Rate Under Existing Conditions

VVWRA Interceptor Peak Flow (mgd)	NAVI Peak Flow (mgd)	Total Peak Flow (mgd)
2.55	0.03	2.58

7.2 BUILDOUT SYSTEM

URS analyzed anticipated flow patterns from projected land use and population for buildout. The proposed buildout system is a combination of the existing system and anticipated CIPs outlined in this Report.

7.2.1 Residential

Future buildout residential land use classifications were obtained from the 2009 General Plan. Using similar approach for developing flows from the existing system, future residential flows were estimated by delineating polygons for contributing areas referred to as subcatchment. Per the Town's design guidelines, URS used 210 gallons/EDU to estimate flows from the future residential areas.

7.2.2 Commercial

As described in Section 7.1.2, URS estimated future buildout flows from commercial areas using the General Plan land use and Specific Plan maps. Subcatchments were defined by delineating contributory polygons for the commercial areas and determining the area within each commercial polygon. To be conservative, unlike the existing commercial area where the area of the buildings were measured, for future buildout conditions, the total area was used to estimate flows using 1,200 gallons per acre per day. The delineated subcatchments were then added to the future buildout InfoWorks model. This approach was used in order to arrive at more conservative estimated flows.

7.2.3 Industrial

The approach used for estimating flows for the Industrial area is similar to the commercial flow assessment described above. A unit flow of 1,500 gallons per acre per day was applied to the total estimated acreage for the industrial polygon to arrive at an estimated buildout flow from the industrial areas. It should be noted that the difference between the existing and the buildout flow estimation is that for existing conditions, only the buildout footprint were used to estimate the existing flows, whereas for buildout conditions, the total acreage from the industrial area was based on land use maps from either the General Plan or Specific Plans. This approach was used in order to arrive at more conservative flow estimate.

7.2.4 Future Buildout Peak Flows

Table 7-3 shows the model peak flows for future buildout.

Table 7-3: Modeled Peak Flow Rate for Future Buildout

VVWRA Interceptor Peak Flow (mgd)	NAVI Peak Flow (mgd)	Total Peak Flow (mgd)
18.01	12.10	30.11

8.0 MODEL DEVELOPMENT

In order to develop the hydraulic model, URS used information from the previously developed GIS database. The database consisted of system attribute data. Prior to the development of the hydraulic model, URS reviewed the existing pertinent data including previous models from the 1993 Master Plan. It should be noted that there was no electronic data available from the 1993 Master Plan that could be used for this project. As a result, URS started from scratch to develop the GIS database that was ultimately used for creation of the hydraulic model.

To create the model, the Town provided URS the following items:

- As-built drawings of the sewer system, 1993 Sewer Master Plan, videotape and log sheet information, Operations and Maintenance (O&M) records, and any other pertinent information. The as-built drawings provided information on the location of sewer pipes, manholes, and lift stations. The O&M records provided insight into the locations of reported problem areas.
- Existing CIP schedules from Town staff.
- Existing and proposed land use for the project area. This land use information was utilized in estimating existing and proposed wastewater flow projections.

8.1 SOFTWARE SELECTION

URS assisted the Town in selecting the appropriate hydraulic modeling software for performing the hydraulic analysis of the system.

Based on discussions between URS and Apple Valley staff, it was decided that InfoWorks was the most suitable software to model the Town's system. The decision was reached based on:

1. The ability of the software to model the Town of Apple Valley sewer system in a comprehensive manner.
2. The cost of purchasing the software.
3. The annual maintenance costs of the software.

URS purchased this software on behalf of the Town of Apple Valley, used it to perform the modeling, and then delivered the software and the modeling files to the Town staff following completion of the project.

URS installed the software on the Town's computers, documented the installation procedures, and ensured compatibility with Town's computer network. URS worked closely with the software vendor to estimate the computer storage for housing the final sewer system model. The final storage and software size was determined by the completion of the modeling process, since the size required depends on the number of nodes in the model, as well as the system complexity, system surcharge, the modeling time step, etc. URS provided the Town with one copy of the software and vendor documentation, which are both available online and are part of the installation. There is an annual maintenance and support subscription associated with each license of approximately $4,500. The main features that are provided by the annual subscription fee include accessibility to software updates, upgrades, product line extensions and access to unlimited software support.

8.2 PREPARATION OF COMPUTER MODEL

Information required for developing the hydraulic model was obtained from the Town's as-built drawings. The model consists of pipes of 8 inches and larger in diameter amounting to over 90 percent of the system. By modeling over 90 percent of the Town's system, the URS model properly account for system storage. The Town provided as-built drawings representing Town's current sewer system of approximately 3,500 manholes and 175 miles of sewers. The Town also provided data on recent wastewater discharge by large industries, data for the 9 lift stations including number of pumps, pump curve and pump control operating conditions.

Following review of the information gathered, URS identified the gaps of the additional data that would be needed to complete the project. For some of the missing data, it was as simple as interpolating the data. Also, as-built data was available for most parts of the system except for the Jess Ranch area. This was discussed with the Town of Apple Valley and it was decided that this portion of the system be excluded from the hydraulic model as it consists of only 6-inch diameter sewers.

URS utilized the pertinent information provided by the Town such as as-built drawings of the pipe network and lift stations, pump curves, etc., to construct the InfoWorks hydraulic model. Key activities in this task included:

- Extract elevation, sewer pipeline and facility information from as-built maps including pipe diameter, rim and invert elevations, and installation year to infer pipe roughness, etc.
- Add information related to lift stations, previous pumping tests, pump and efficiency curves, design documentation and operation data, wet-well size, etc., to develop the hydraulic model.
- Review land use data provided by the Town to determine the adequacy of the data for use in the hydraulic modeling efforts.
- Prepare a gap analysis to determine the additional data needed to complete the analysis such as missing pipe rim and invert level information, pump curve data, etc. URS worked with Town staff to resolve any issues regarding the missing data needed to construct the hydraulic model.
- Develop appropriate boundary conditions for the hydraulic model and review the boundary data with the Town.
- Develop unit per capita flow for use in assigning sewer flows to the model. Use Town unit flow data to estimate future development and redevelopment flow projections for both existing and future conditions.

- Review and analyze flow monitoring results and compare the results to the model output.

8.2.1 Flow Input into Hydraulic Model

URS conducted hydraulic modeling simulations for two different modeling scenarios based on the Town's existing and projected populations. As mentioned in Section 7, existing and projected households (EDUs) were established by land use areas. The Town uses 210 gallons per household per day for residential, 1,200 gallons per developed acre of building structure for commercial, and 1,500 gallons per developed acre of building structure for industrial areas to estimate flows.

To model the flow delivered to the sewer network, URS divided the Town into subcatchment areas from which a manhole collects wastewater. Subcatchments were created over the land use map, enclosing one or more manholes (nodes). Non-overlapping subcatchments were drawn to cover the total area of study. The properties within each contributing area were determined and entered in the hydraulic model to account for the flows delivered to the upstream manhole. The following is a description of how the polygons were generated for residential, commercial, and industrial categories.

8.2.1.1 Existing Subcatchment

Subcatchments for the existing scenario were established for existing developments. The flows generated from this assessment were used to establish and calibrate the base model. Flows generated by properties on septic system areas were excluded from the assessment.

8.2.1.2 Future Subcatchment

Future subcatchments were produced using the Town's Land Use Map. The buildout model consisted of adding new subcatchment to the existing scenario model. The land use information was used to define the study areas projected to grow. The buildout model was completed by defining new proposed sewer lines in order to connect the projected future flows to the existing sewer infrastructure.

9.0 MODEL CALIBRATION

Model calibration is an iterative process where the model is run a number of times, and the parameters such as pipe roughness, other pipe attribute data (based on field verification of data anomalies), and land use information are adjusted until simulation results match observed flow data within industry tolerances. Where wet weather has an impact on the system flow response, percent impermeability is considered in the calibration. In the case of Apple Valley, the system did not show any response to rainfall, therefore the model was calibrated to dry weather conditions only with roughness, diurnal curve, and flow input being the main parameters that were adjusted to arrive at a calibrated model. Simulation results were compared to observed flow data.

9.1 DRY WEATHER

For dry weather calibration, URS ran the model during a full dry week period. The predicted flows, depths, and velocities were compared to flow monitoring data to achieve the following calibration criteria.

- The difference between observed and simulated peak flow rates, as well as the difference between observed and simulated peak depths in ± 10% range.
- The difference between observed and simulated volumes of flow in ± 10% range (This criterion was checked only over the periods for which the observed flows were expected to be accurate).

9.2 WET WEATHER

Wet weather scenario model is normally calibrated using a range of rainfall events recorded during the flow monitoring period. Ideally, to calibrate the wet weather model, as many as possible storm events are considered, from small events to storms large enough to cause overflows in the system. In the case of the Apple Valley Sewer Master Plan Update, no wet-weather calibrations were carried out because the system does not appear to respond to rainfall events. However, this assumption was reached based on limited concurrent flow and rainfall data. URS recommends further evaluation using concurrent flow and rainfall data conducted during the wet-weather season in Apple Valley.

9.3 CALIBRATION ANALYSIS AND RESULTS

As mentioned in Section 6.2, VVWRA provided URS the results of a sanitary sewer flow monitoring performed by V&A from September 8-24, 2012. The results from three monitoring sites within the Town's sewer infrastructure were utilized to calibrate the InfoWorks Model.

The following sections present the dry weather calibration analysis for each of the three sites.

9.3.1 Apple Valley No. 1

The percentage fit for flow, velocity, and depth at location AV No. 1 are listed in Table 9-1. The results show that there is a good comparison between the model and the measured data pattern (Figure 9-1, 9-2, and 9-3). The percentage fit on peak flow and velocity are within acceptable limits. The model under predicts the measured peak depth by 41 percent. Overall, the model is within the average measurements of the data. The analysis appears to be an acceptable fit for flow at location AV No. 1.

Table 9-1: Peak Data Comparison between Measured and Modeled Data at AV No. 1

AV No. 1			
	Measured	**Modeled**	**% Difference**
Flow (mgd)	2.05	1.62	21%
Depth (in)	12.84	7.63	41%
Velocity (fps)	4.84	3.98	18%

Figure 9-1: Model vs Measured Flow Data (AV No. 1)

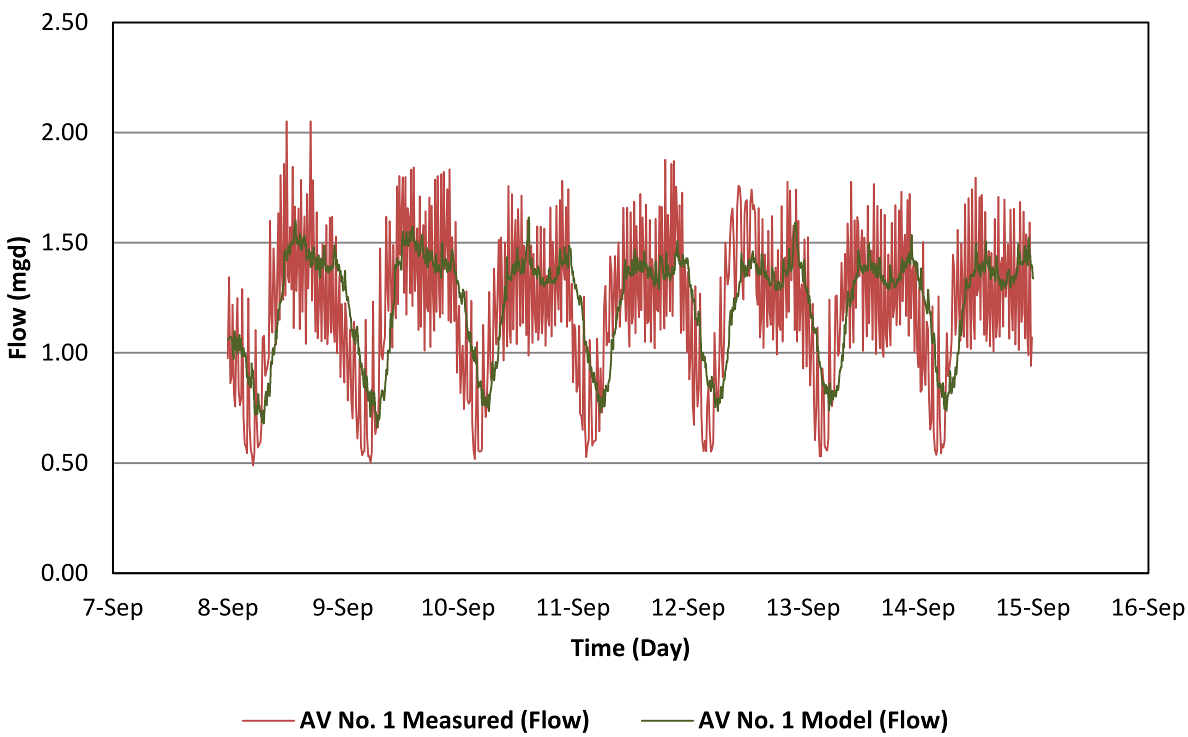


Figure 9-2: Model vs Measured Depth Data (AV No. 1)

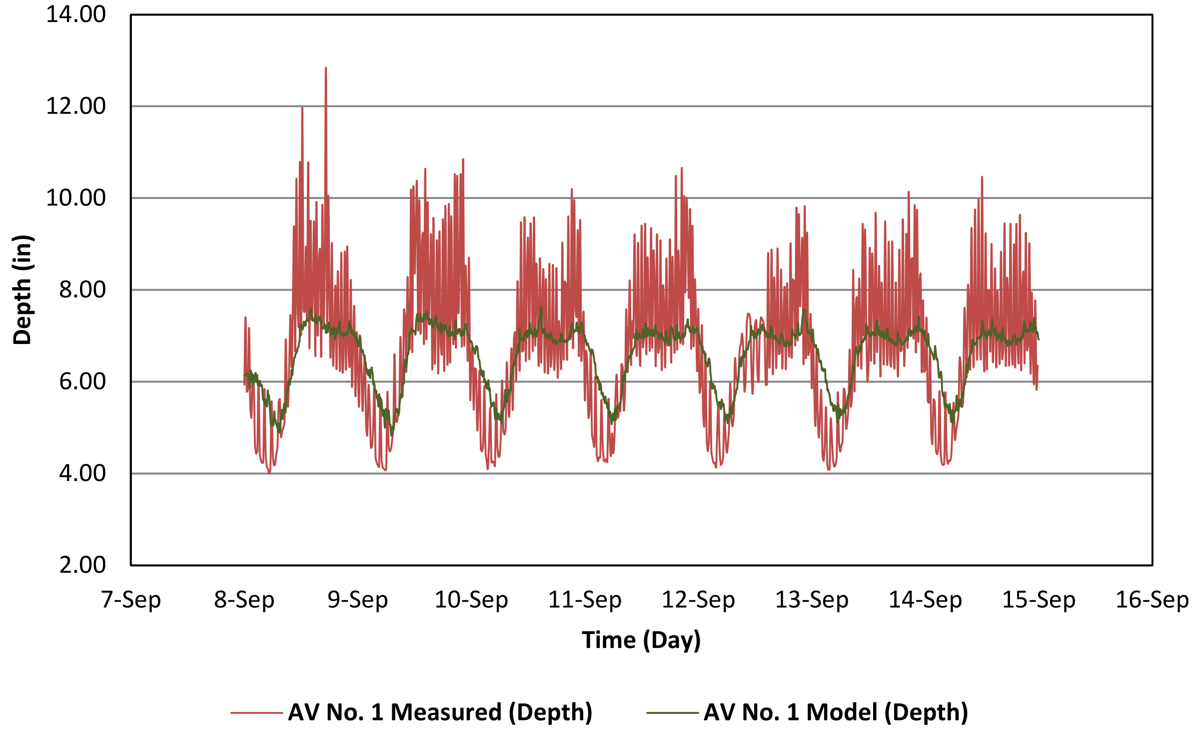
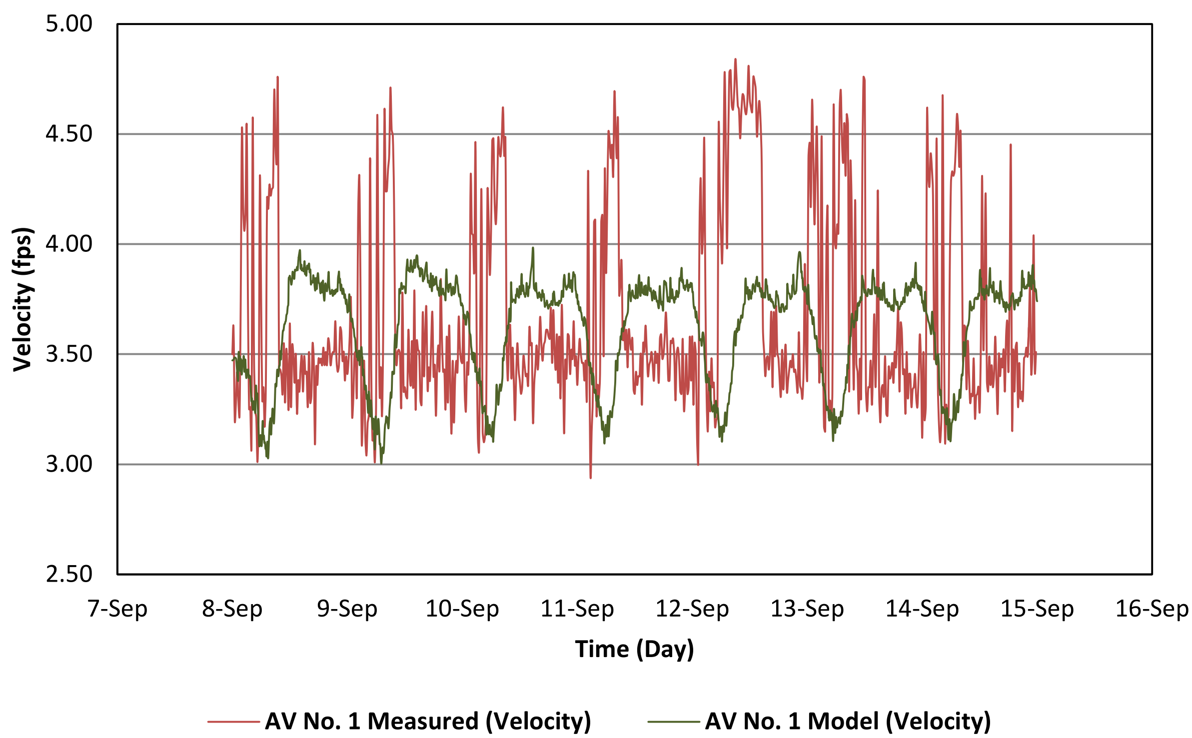


Figure 9-3: Model vs Measured Velocity Data (AV No. 1)



9.3.2 Apple Valley No. 2

The percentage fit for flow, velocity, and depth at location AV No. 2 are listed in Table 9-2. The model results and the flow monitoring data, as shown in Figures 9-4, 9-5, and 9-6, closely match each. Predicted peak flow is 19 percent less than recorded flow. The model predicted velocities are within 7 percent of the flow monitored data. Similarly, the model predicted depths are within 13 percent difference. Overall, the model accurately follows the pattern from the measured data and is within acceptable limits of peak measurements. In conclusion, AV No. 2 site is considered a good fit for flow, velocity, and depth.

Table 9-2: Peak Data Comparison between Measured and Modeled Data at AV No. 2

AV No. 2			
	Measured	**Modeled**	**% Difference**
Flow (mgd)	1.23	0.99	19%
Depth (in)	5.07	4.43	13%
Velocity (fps)	4.86	4.54	7%

Figure 9-4: Model vs Measured Flow Data (AV No. 2)

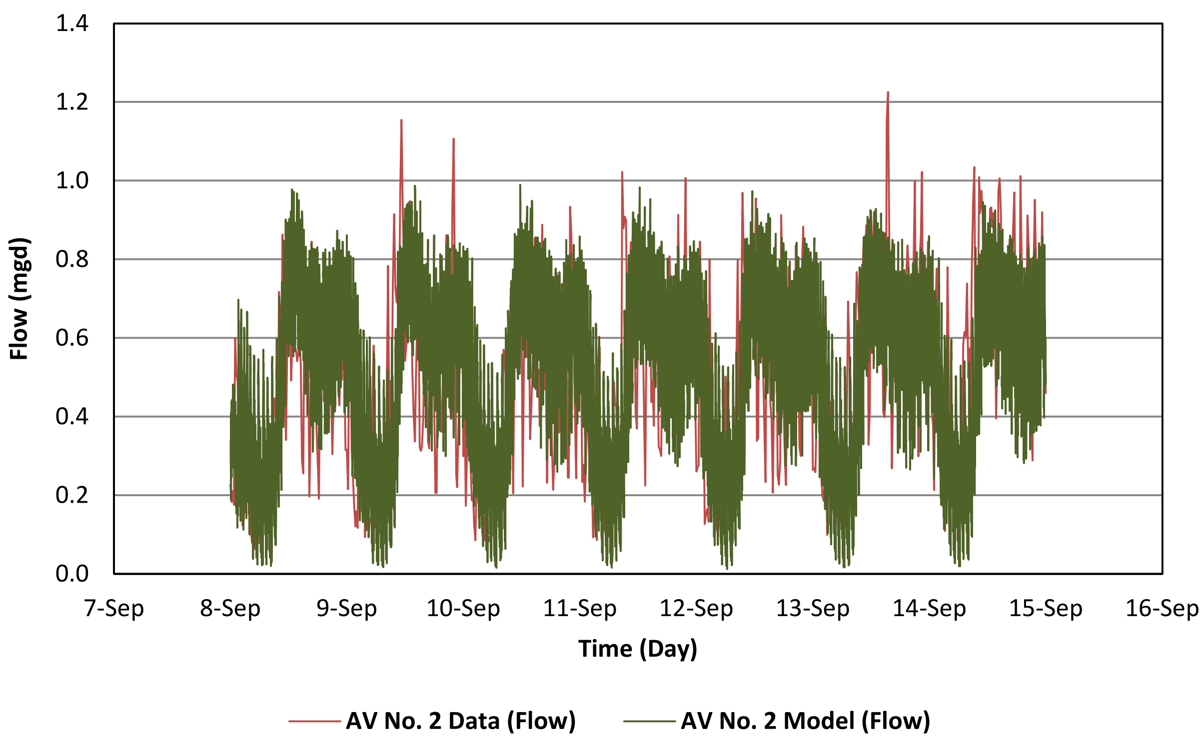


Figure 9-5: Model vs Measured Depth Data (AV No. 2)

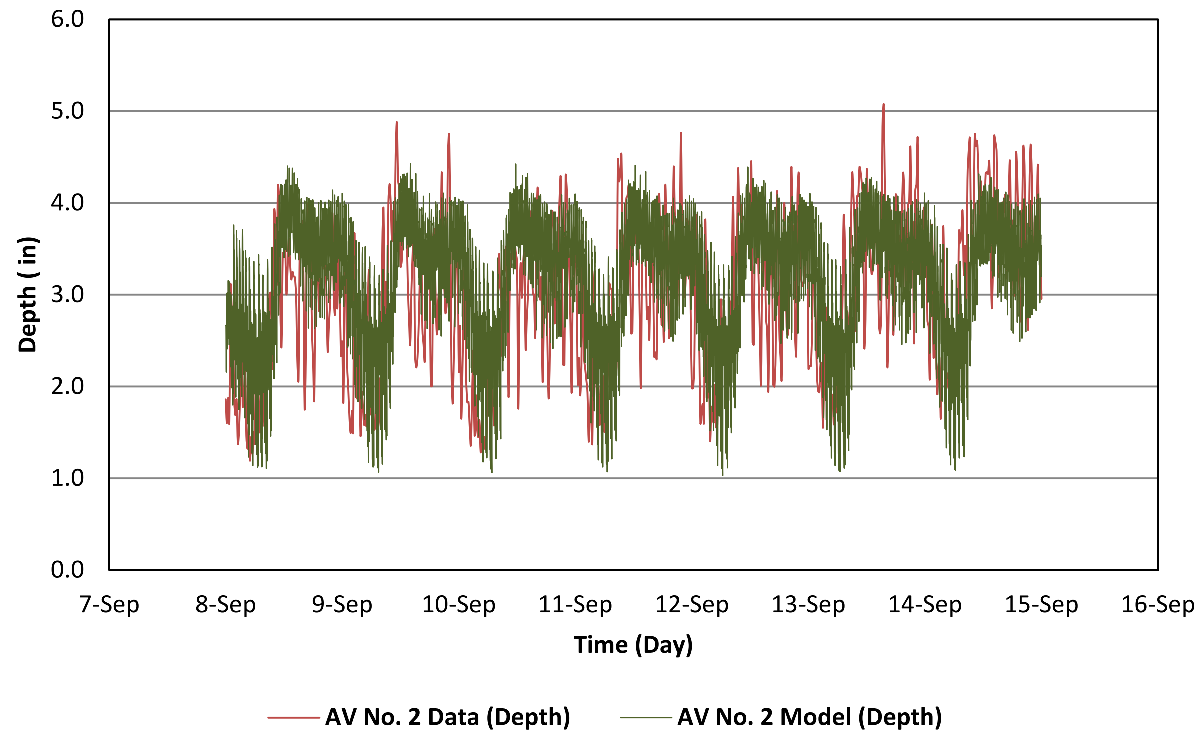
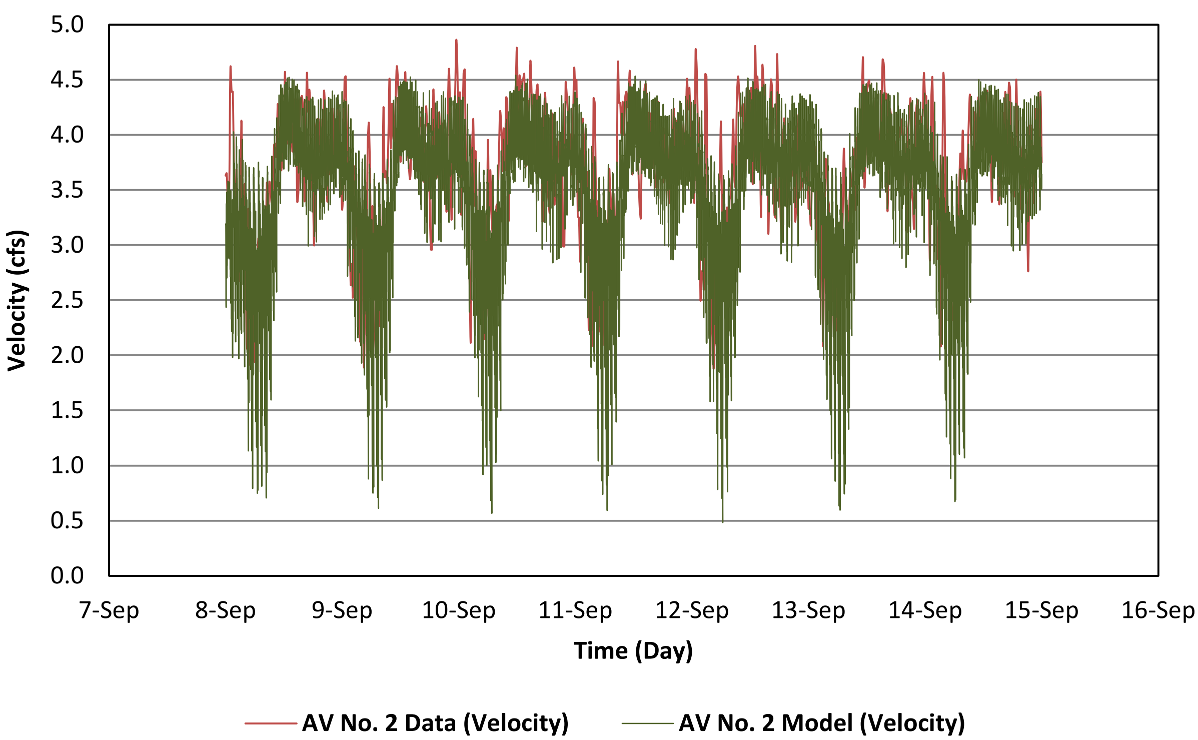


Figure 9-6: Model vs Measured Velocity Data (AV No. 2)



9.3.3 NAVI

Currently, flows from the NAVI are extremely low because it only receives discharge from the Juvenile Detention Center located in the upper north part of the Town. Overall, the model follows the shape patterns of the measured data, as shown in Figures 9-7, 9-8, and 9-9. The model appears to have a lag phase which can be attributed to missing data. Some of the pipeline within this segment lies outside of the Town's boundary; therefore, data was not available to be incorporated into the model. Overall, the percentage fit on peak velocities and peak depths are within acceptable limits. The percentage fit for flow, velocity, and depth at location NAVI are listed in Table 9-3.

Table 9-3: Peak Data Comparison between Measured and Modeled Data at the NAVI

NAVI			
	Measured	**Modeled**	**% Difference**
Flow (mgd)	0.07	0.03	52%
Depth (in)	2.07	1.48	28%
Velocity (fps)	1.03	0.66	36%

Figure 9-7: Model vs Measured Flow Data (NAVI)

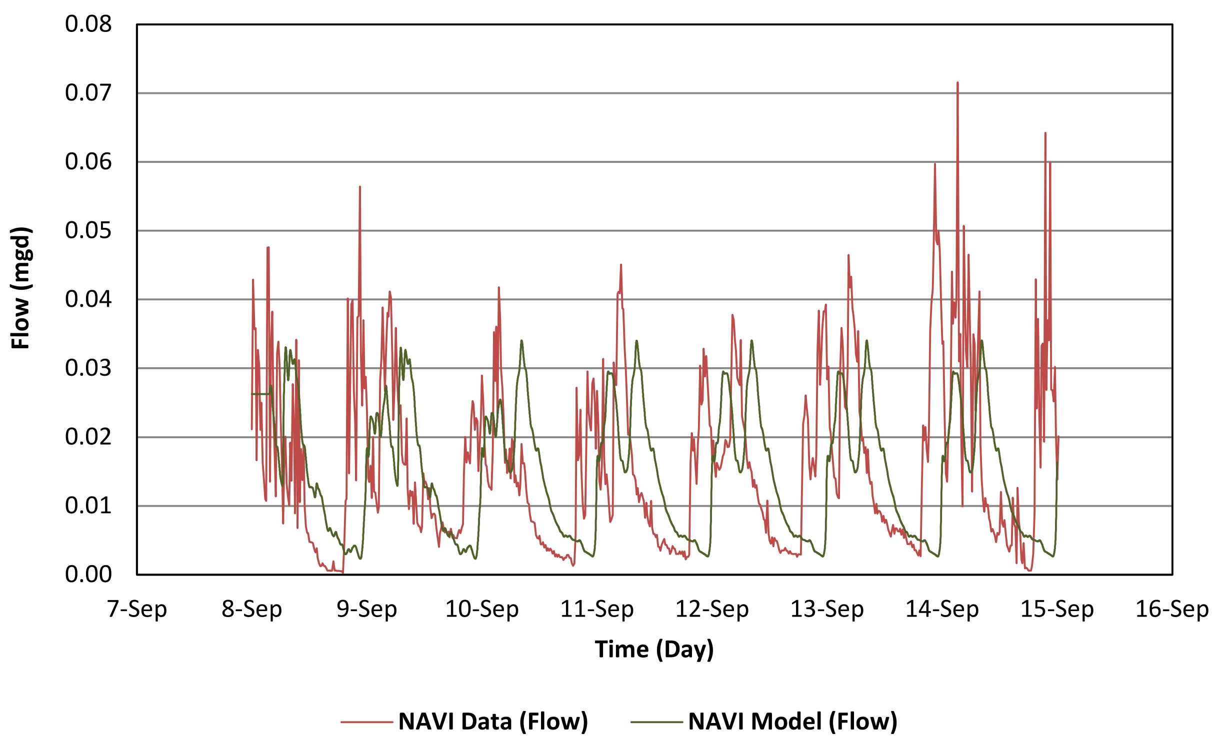


Figure 9-8: Model vs Measured Depth Data (NAVI)

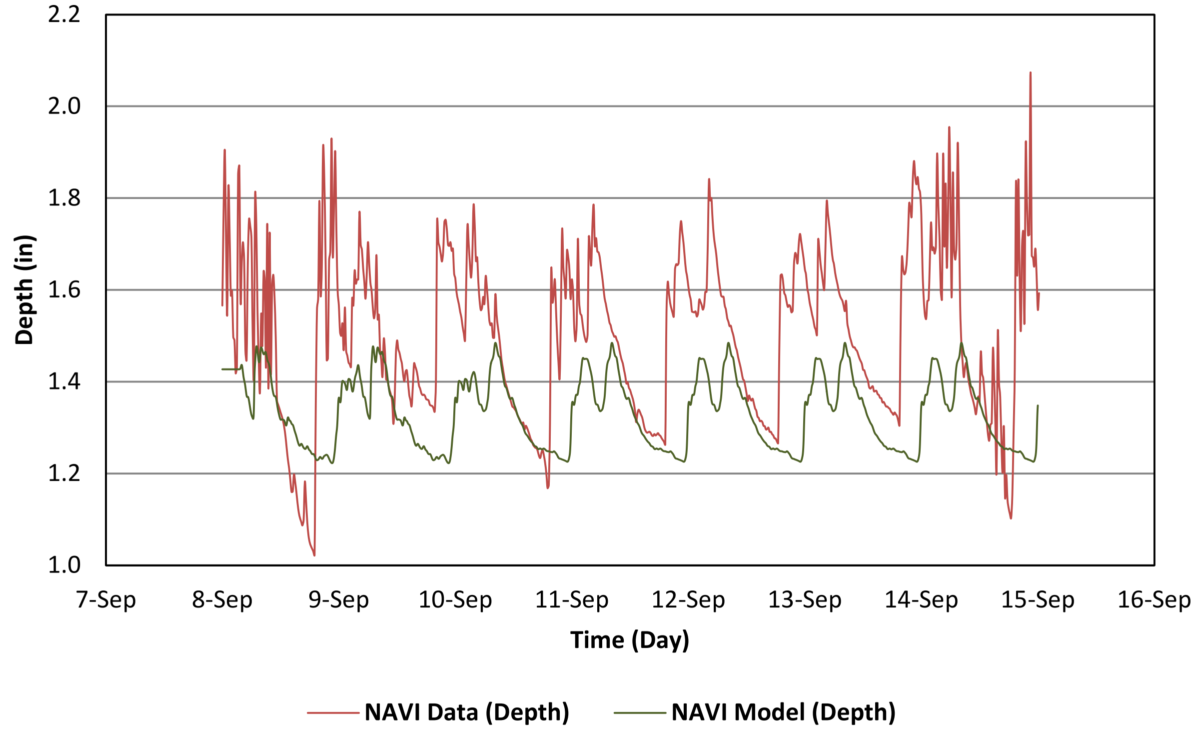
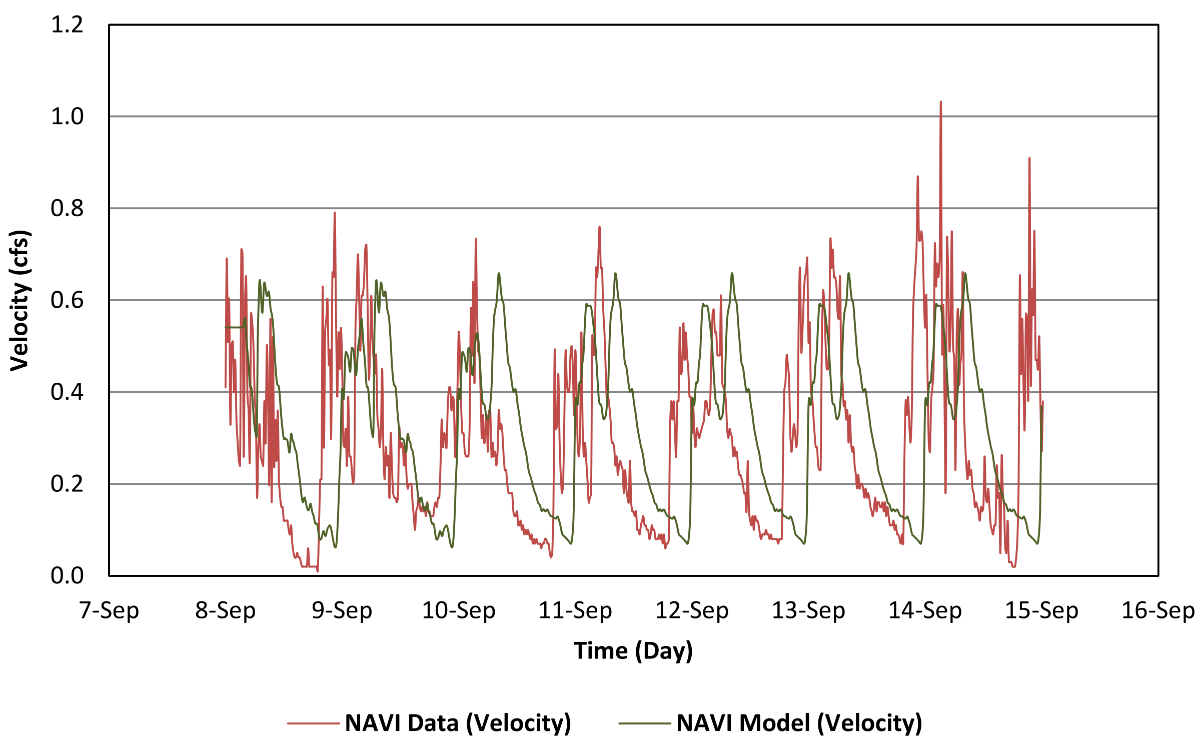


Figure 9-9: Model vs Measured Velocity Data (NAVI)



After calibration, the baseline scenario models were ready to be used to evaluate system capacity, identify hydraulic deficiencies and to evaluate alternatives for providing hydraulic capacity for both existing and future development scenarios.

10.0 EVALUATION OF HYDRAULIC DEFICIENCIES

Following the model development and calibration to establish confidence in the model, the model was used to evaluate the hydraulic deficiencies in both the existing and future systems.

10.1 EXISTING SYSTEM ANALYSIS

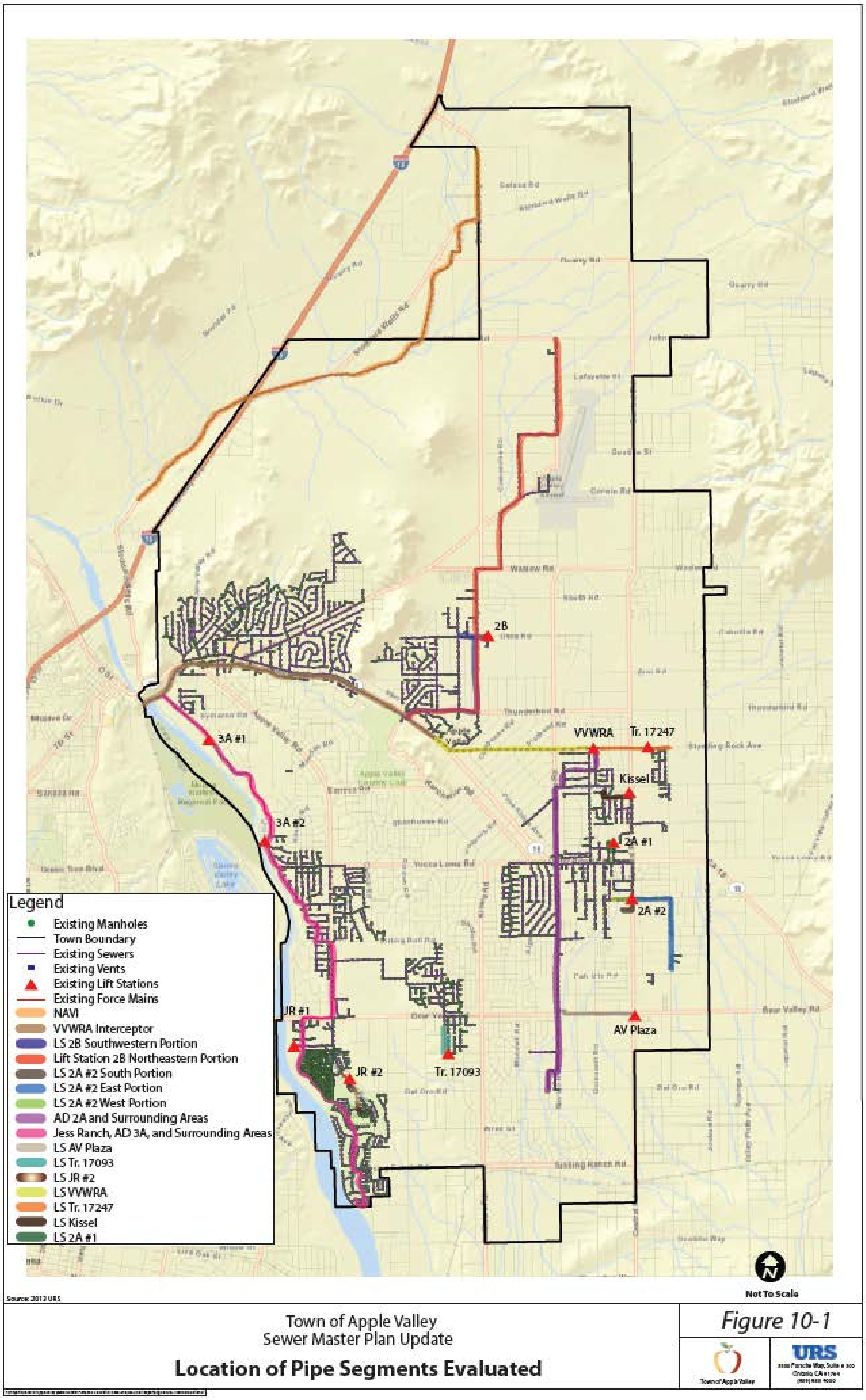
URS ran existing system model to identify system hydraulic deficiencies and develop recommended solutions to address the deficiencies identified. The results of the analysis are a list of existing CIPs (Appendix A) for the Town to implement to address the hydraulic deficiencies identified.

10.1.1 Evaluation of Existing System Conditions

A hydraulic evaluation was carried out to assess the capacity and performance of the existing sewer system. Figure 10-1 shows the locations of the pipeline segments evaluated.

Existing system hydraulic model were run to simulate flows under Dry Weather Conditions. The following subsections presents the results of the computer model runs for existing conditions. The ground level elevation, hydraulic grade line, pipe invert and sewer pipeline, and manholes are shown on each of the hydraulic profiles displayed.

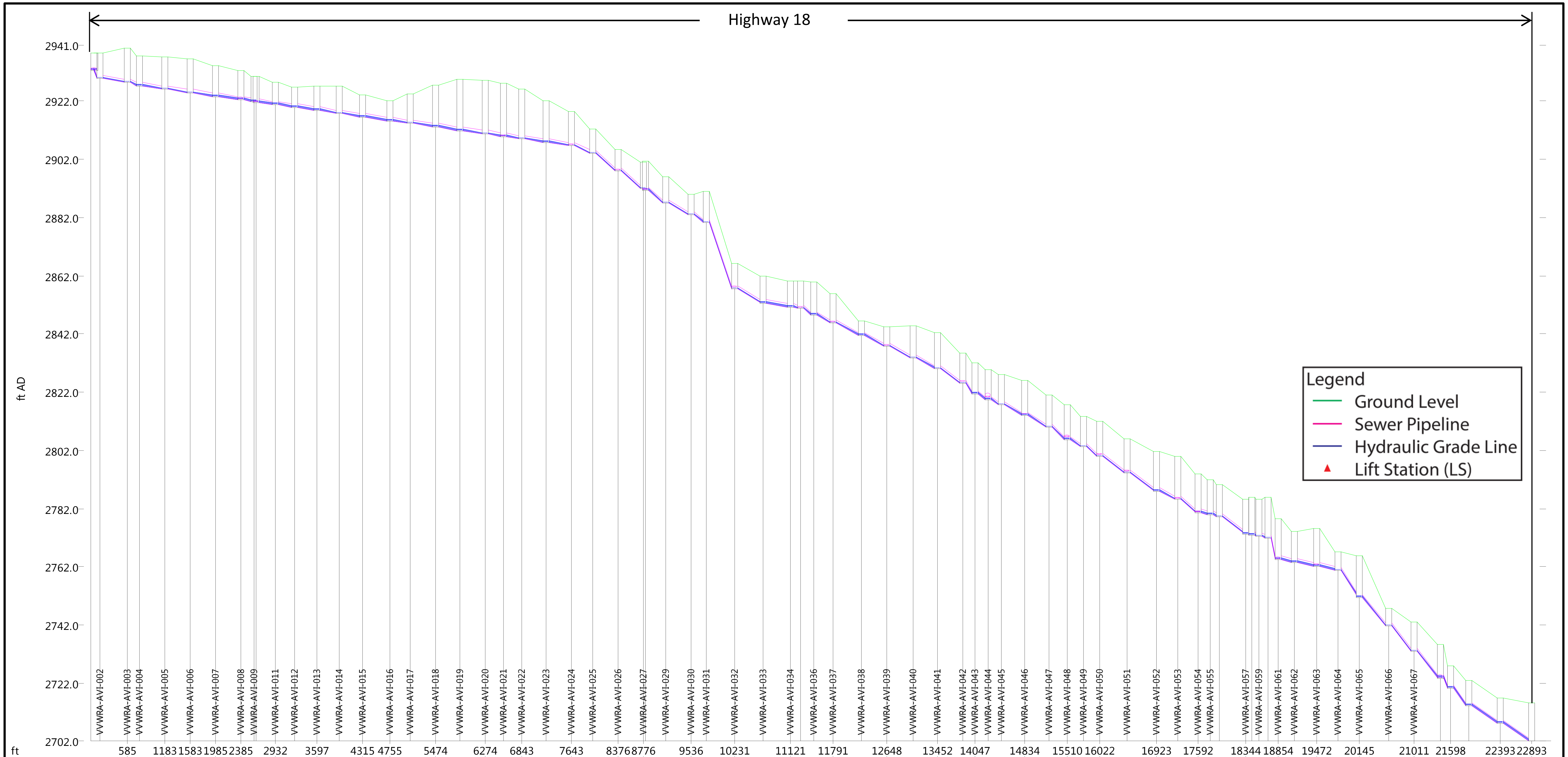
Figure 10-1: Location of Pipe Segments Evaluated



10.1.1.1 Evaluation of Existing Flows for VVWRA Interceptor

The results of the simulation for the VVWRA interceptor are shown in Figure 10-2. The results for this segment show that there is no surcharge in the VVWRA interceptor, indicating that the pipeline has adequate capacity to covey flows under existing dry weather conditions.

Figure 10-2: Modeled Results of Existing Flows for VVWRA Interceptor Pipeline Segment



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Existing Flows for VVWRA Interceptor Pipeline Segment

Figure 10-2

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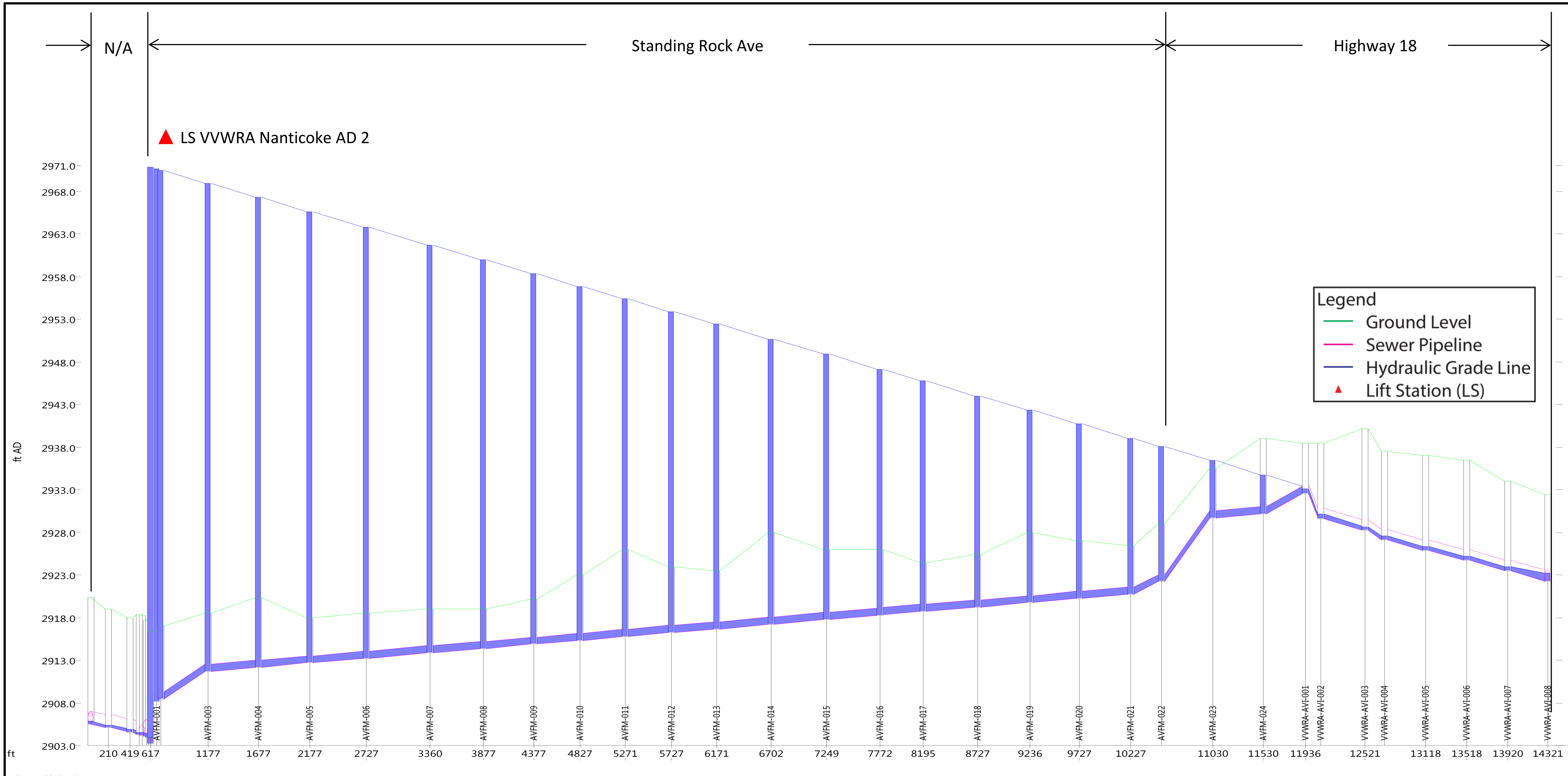
10.1.1.1.1 Evaluation of Existing Flows for Lift Station VVWRA Nanticoke AD 2

Figure 10-3 shows the pipeline segment upstream of Lift Station VVWRA Nanticoke AD 2 along Nanticoke Road. The model results indicate no backing up of flows. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station. It should be noted operations and maintenance of this lift station is the responsibility of VVWRA.

Figure 10-3: Modeled Results of Existing Flows for Lift Station VVWRA Nanticoke AD 2



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Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station VVWRA Nanticoke AD 2

Figure 10-3

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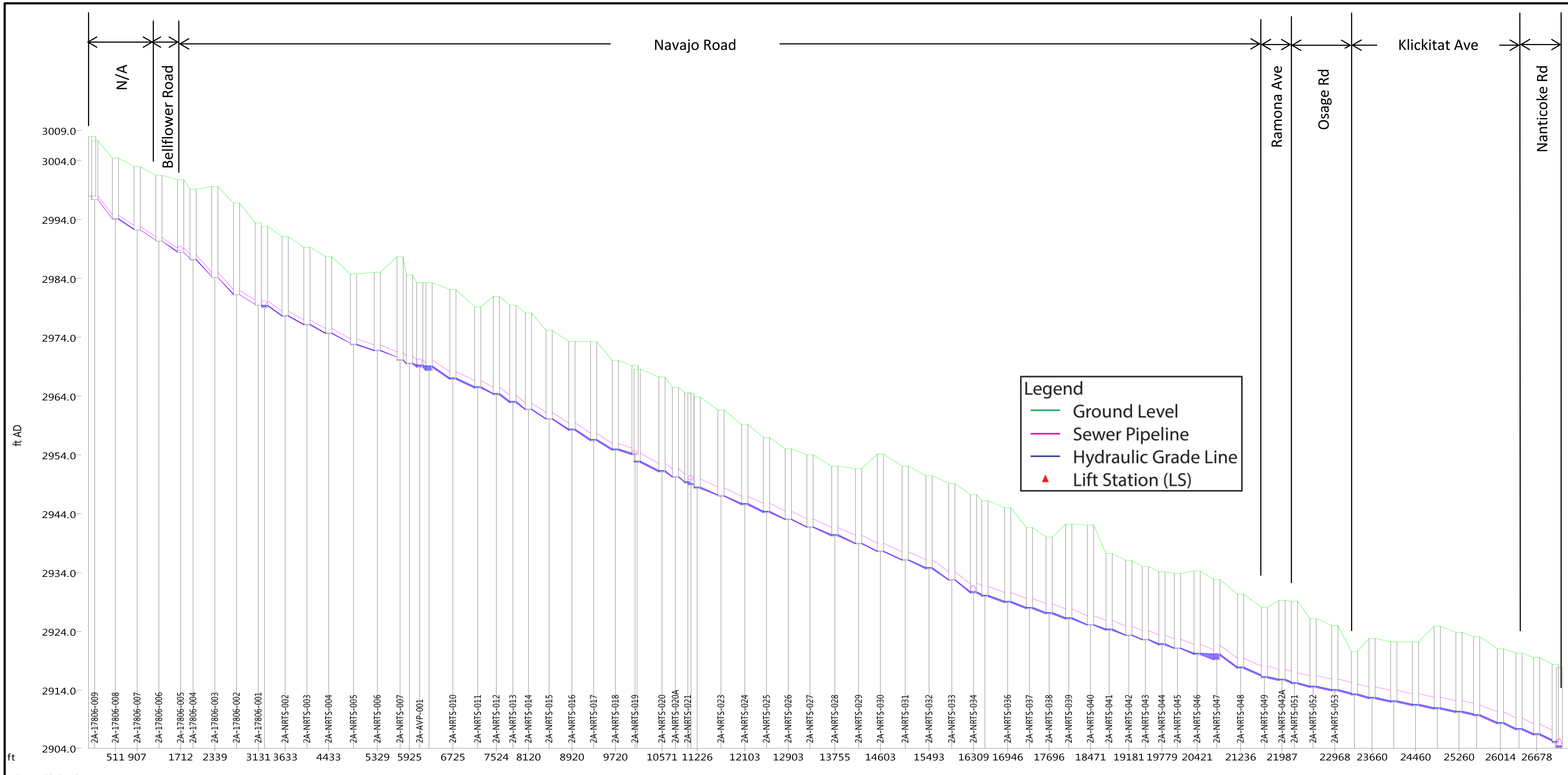
10.1.1.2 Evaluation of Existing Flows for AD 2A and Surrounding Areas

The model results show that the pipe segment conveying flows from AD 2A and surrounding areas to the Lift Station VVWRA Nanticoke AD 2 along Navajo Road (Figure 10-4) has adequate capacity to convey flows during dry weather conditions. The model results for this segment show no surcharging in the pipeline segment.

Figure 10-4: Modeled Results of Existing Flows for AD 2A and Surrounding Areas Pipeline Segment



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Sewer Master Plan Update
Modeled Results of Existing Flows for AD 2A and Surrounding Areas Pipeline Segment

Figure 10-4

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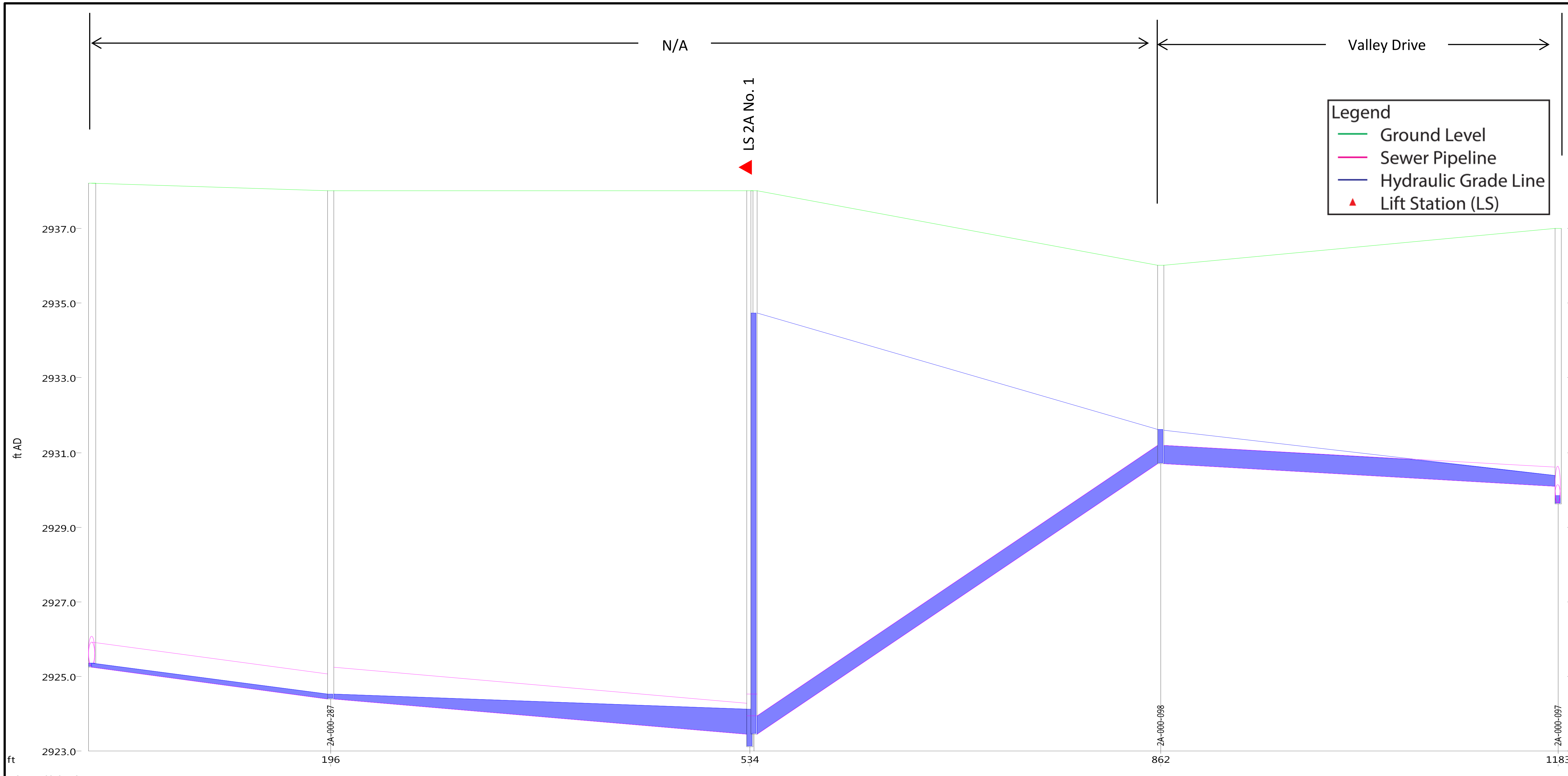
10.1.1.2.1 Evaluation of Existing Flows for Lift Station 2A No. 1

The model results shown on Figure 10-5 indicate that the Lift Station 2A No.1 has adequate capacity to convey flows under existing dry weather conditions. No backing up of flows is predicted upstream of Lift Station 2A No 1.

Figure 10-5: Modeled Results of Existing Flows for Lift Station 2A No. 1



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station 2A No. 1

Figure 10-5

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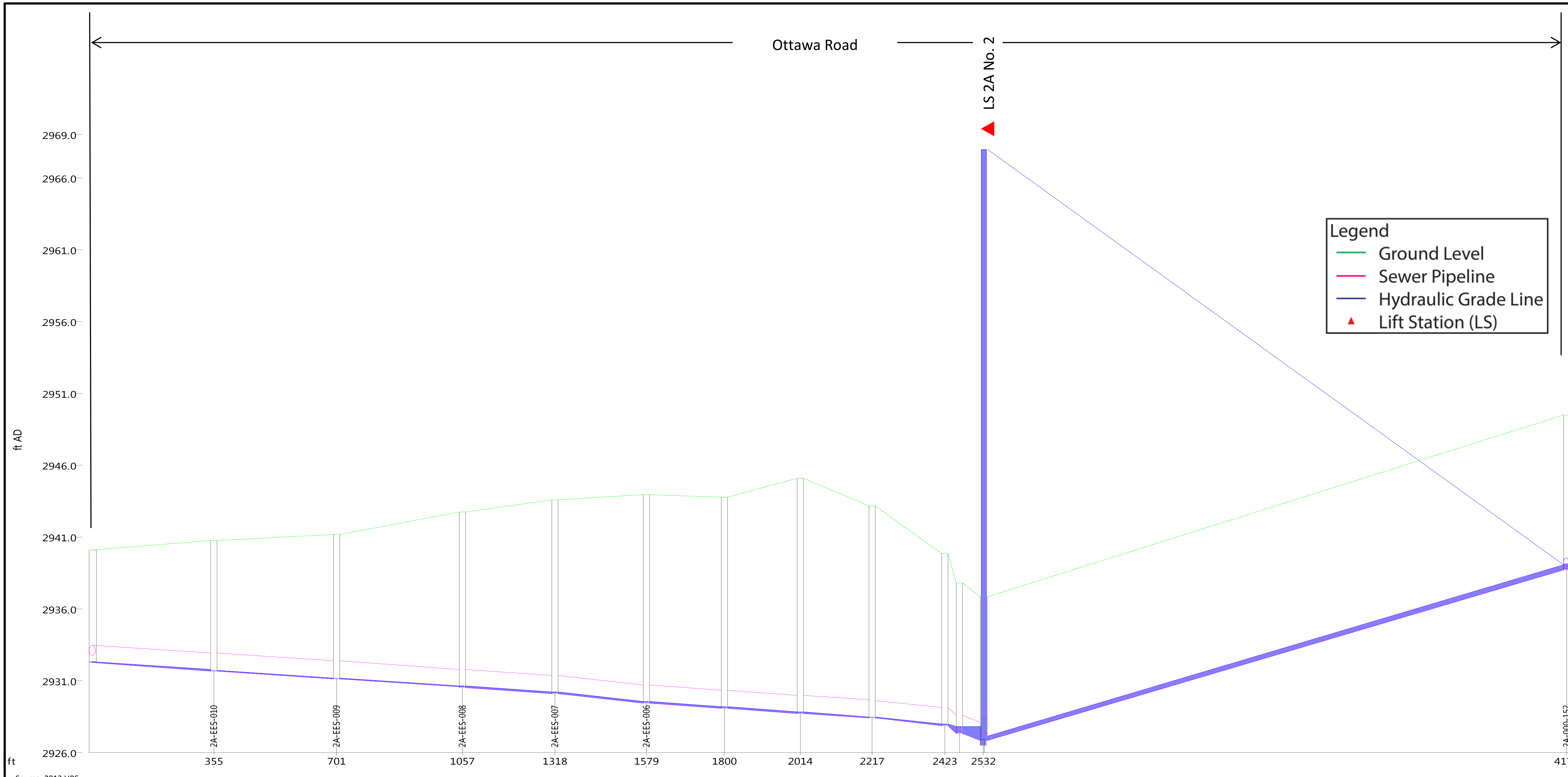
10.1.1.2.2 Evaluation of Existing Flows for Lift Station 2A No. 2

There are three pipeline segments discharging into Lift Station 2A No. 2.

The first segment is the pipeline segment that travels north on Mesquite Road, between Nisqually Road and Ottawa Road and heads west on Ottawa Road until reaching Lift Station 2A No. 2. Modeled results (Figure 10-6) show that there is no backing up of flows to the pipeline immediately upstream of this lift station under existing conditions.

Figure 10-6: Modeled Results of Existing Flows for Lift Station 2A No. 2 (East of LS 2A No. 2)



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Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station 2A No. 2 (East of LS 2A No. 2)

Figure 10-6

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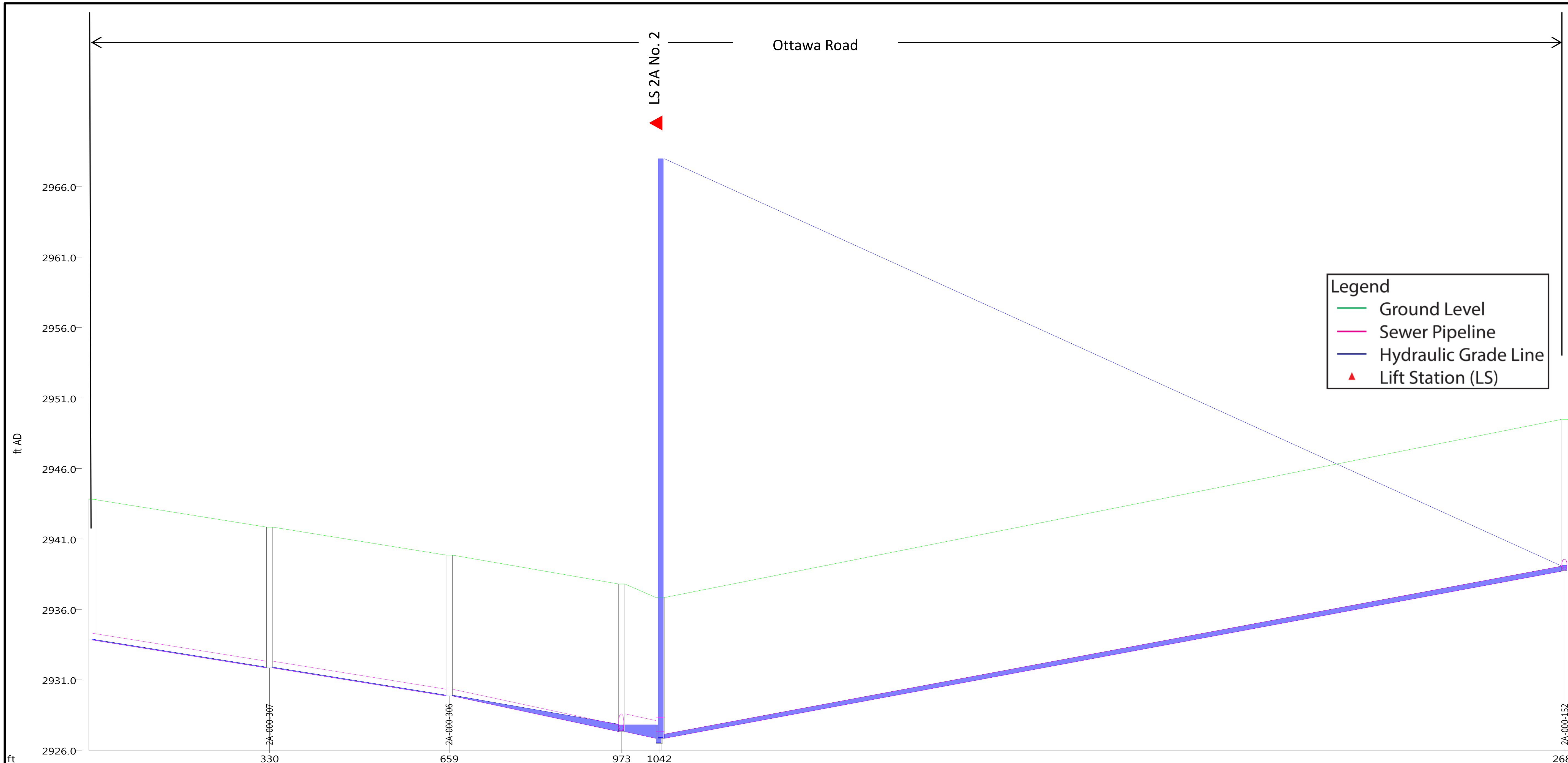
10.1.1.2.3 Evaluation of Existing Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)

The second segment travels east along Ottawa Road, between Malaki Road and Central Road until reaching Lift Station 2A No. 2. Modeled results (Figure 10-7) show no surcharging along this pipeline segment.

Figure 10-7: Modeled Results of Existing Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)



Source: 2013 URS

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Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)

Figure 10-7

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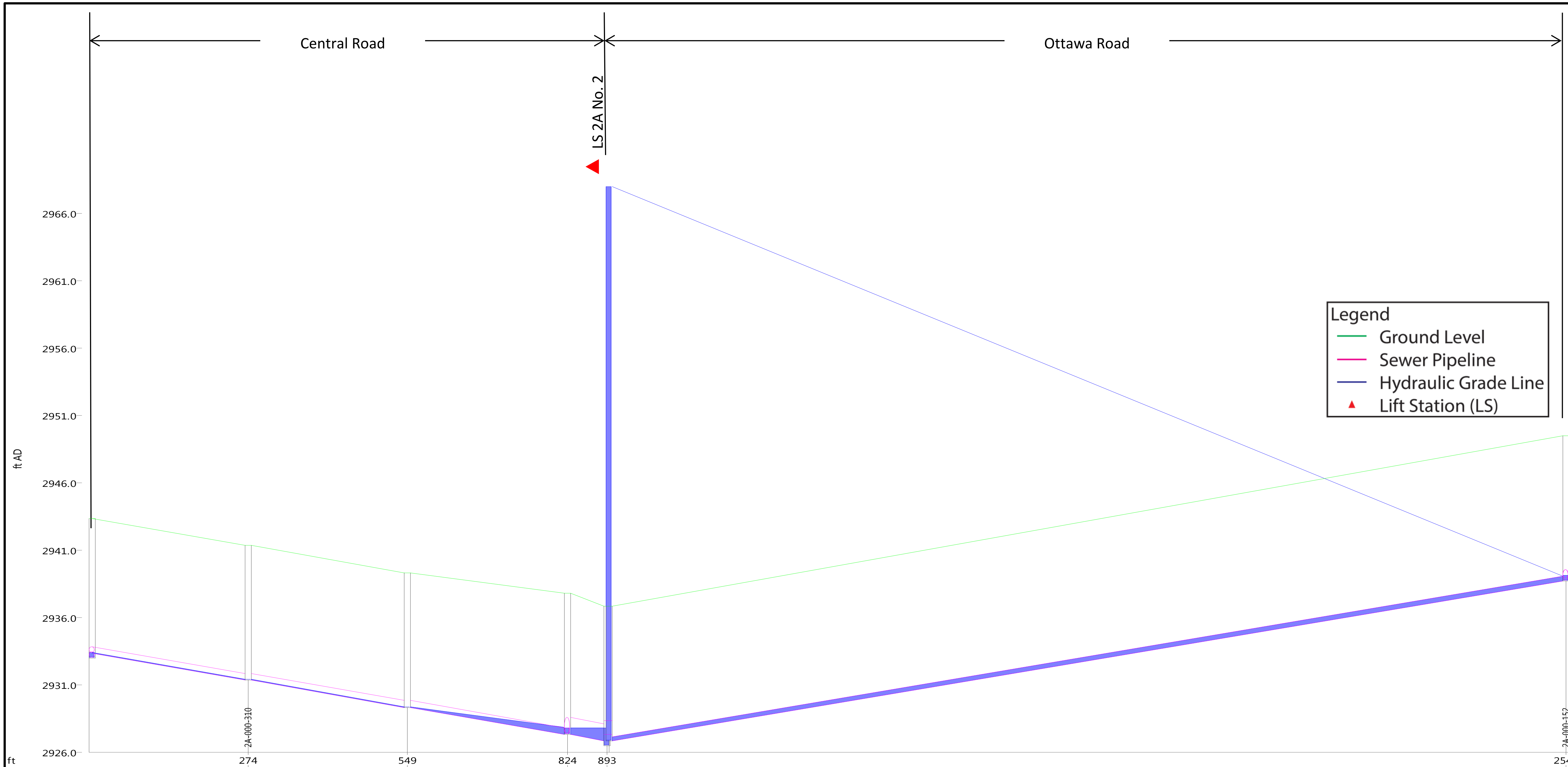
10.1.1.2.4 Evaluation of Existing Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)

The third segment includes flow South of Lift Station 2A No. 2 along Central Road. Modeled results (Figure 10-8), show no backing of flows in this pipeline segment during existing dry weather conditions.

Figure 10-8: Modeled Results of Existing Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)



Source: 2013 URS

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Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)

Figure 10-8

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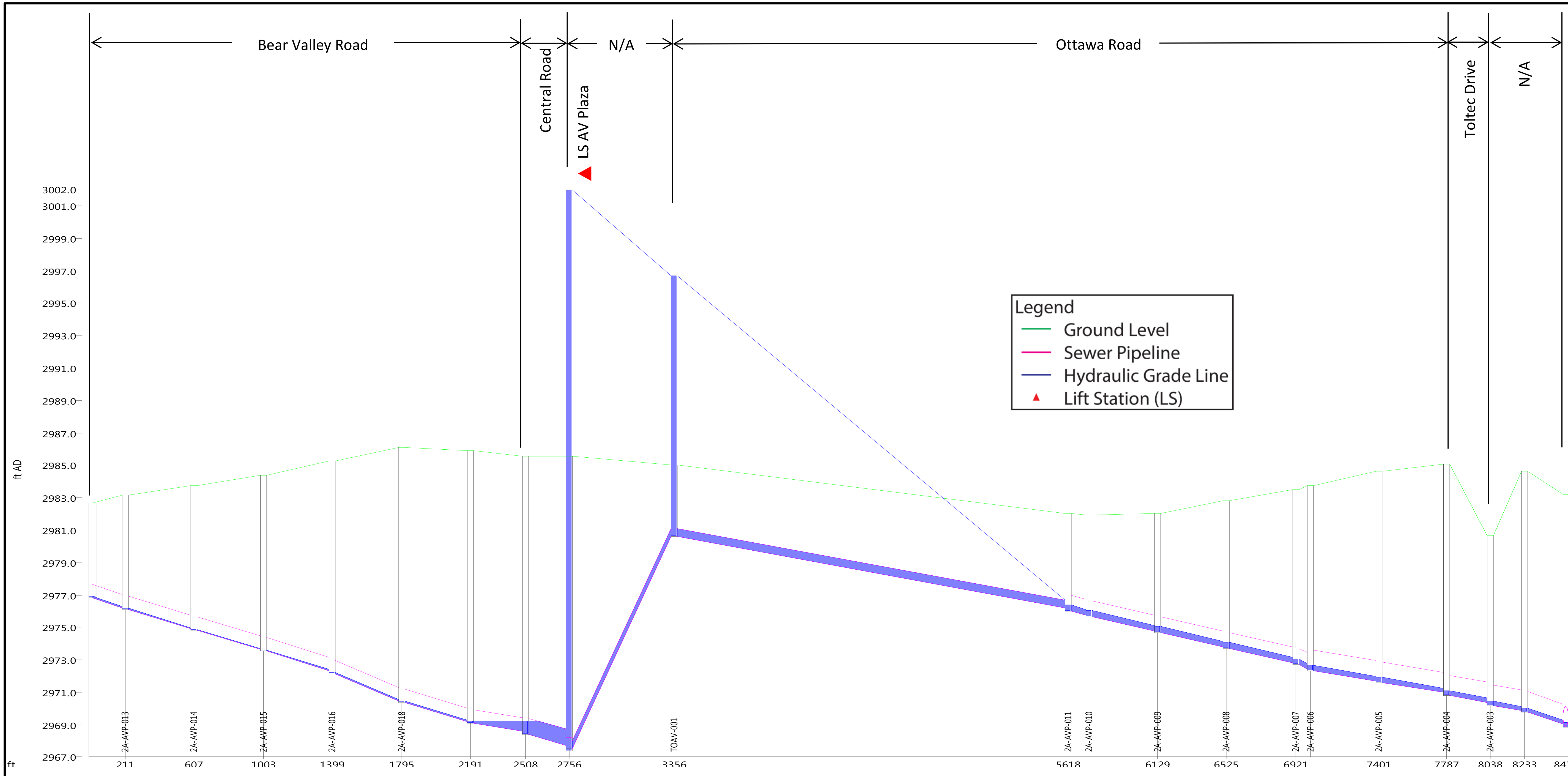
10.1.1.2.5   Evaluation of Existing Flows for Lift Station AV Plaza

Figure 10-9 shows the pipeline segment upstream of Lift Station AV Plaza along Central Road. The model results indicate a slight backup in the last pipeline segment leading to this lift station. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station under existing dry weather conditions.

Figure 10-9: Modeled Results of Existing Flows for Lift Station AV Plaza



Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station AV Plaza

Figure 10-9

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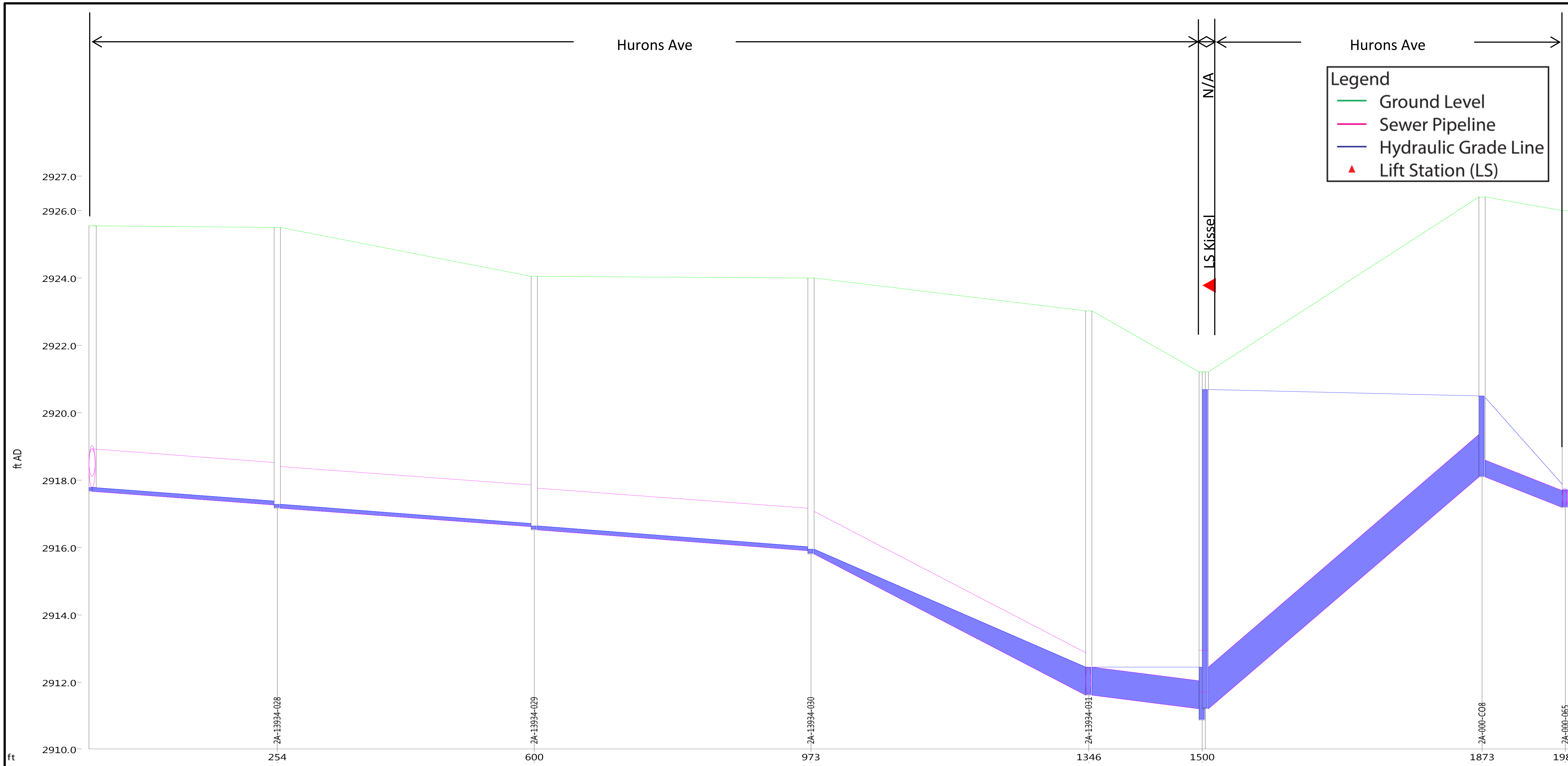
10.1.1.2.6 Evaluation of Existing Flows for Lift Station Kissel

Figure 10-10 shows the pipeline segment along Hurons Avenue upstream of Lift Station Kissel. The model results indicate a slight backup in the last pipeline segment leading to this lift station. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station under existing dry weather conditions.

Figure 10-10: Modeled Results of Existing Flows for Lift Station Kissel



Source: 2013 URS

Town of Apple Valley
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Modeled Results of Existing Flows for Lift Station Kissel

Figure 10-10

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10.1.1.3 **Evaluation of Existing Flows for AD 2B and Surrounding Areas**

There are two pipe segments that discharge to Lift Station 2B.

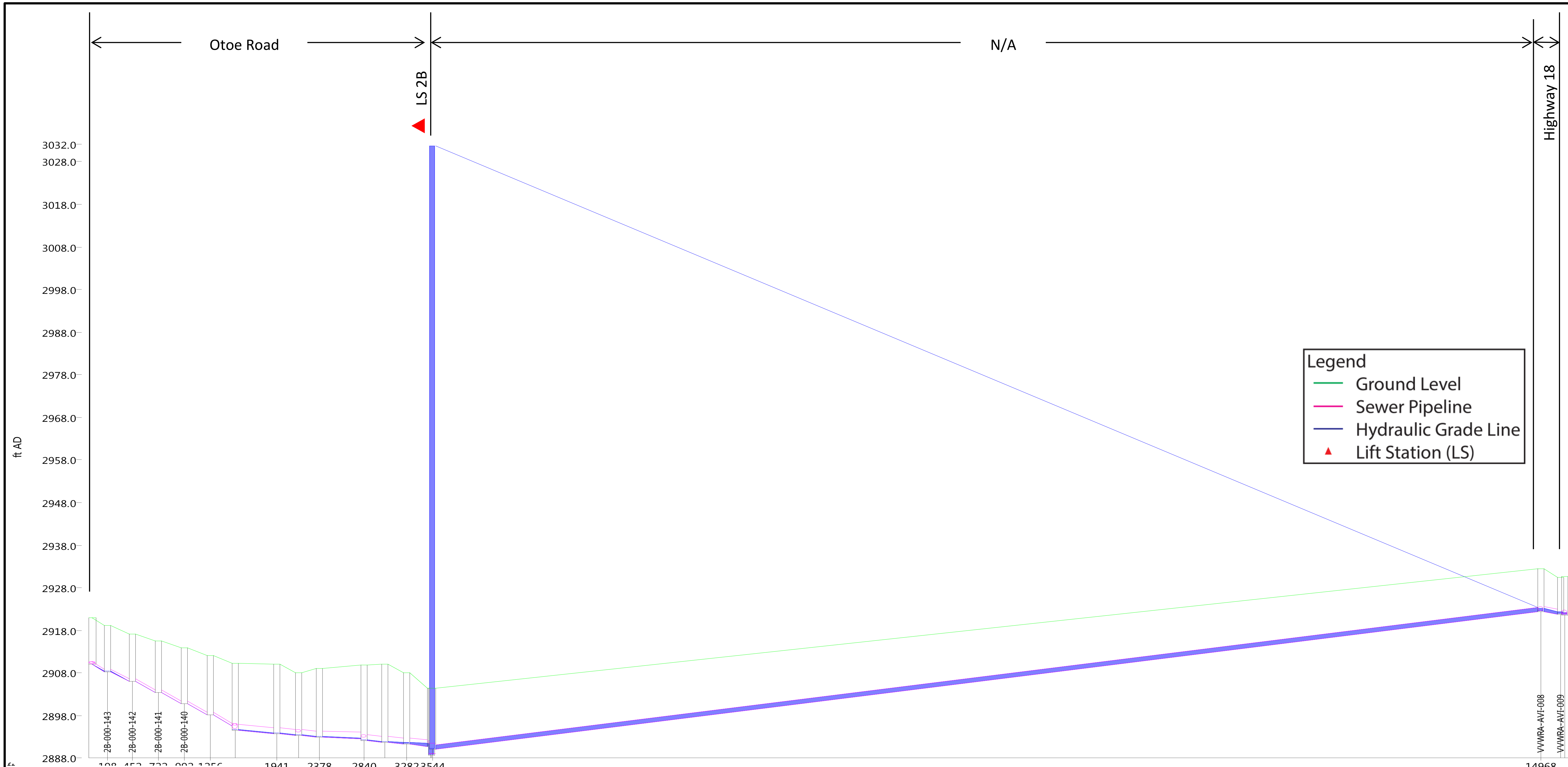
The southwestern portion travels along Otoe Road and Wichita Road.

The northeastern portion travels along Otoe Road, Dale Evans Parkway, Waalew Road, Comanche Road, Corwin Road and Navajo Road.

10.1.1.3.1 Evaluation of Existing Flows for Lift Station 2B (Southwestern Portion)

Figure 10-11 shows the pipeline segment upstream of Lift Station 2B along Otoe Road, and Wichita Road, referred to as the Southwest portion. The model results indicate no backing up of flows. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station.

Figure 10-11: Modeled Results of Existing Flows for Lift Station 2B (Southwestern Portion)



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Existing Flows for Lift Station 2B (Southwestern Portion)

Figure 10-11

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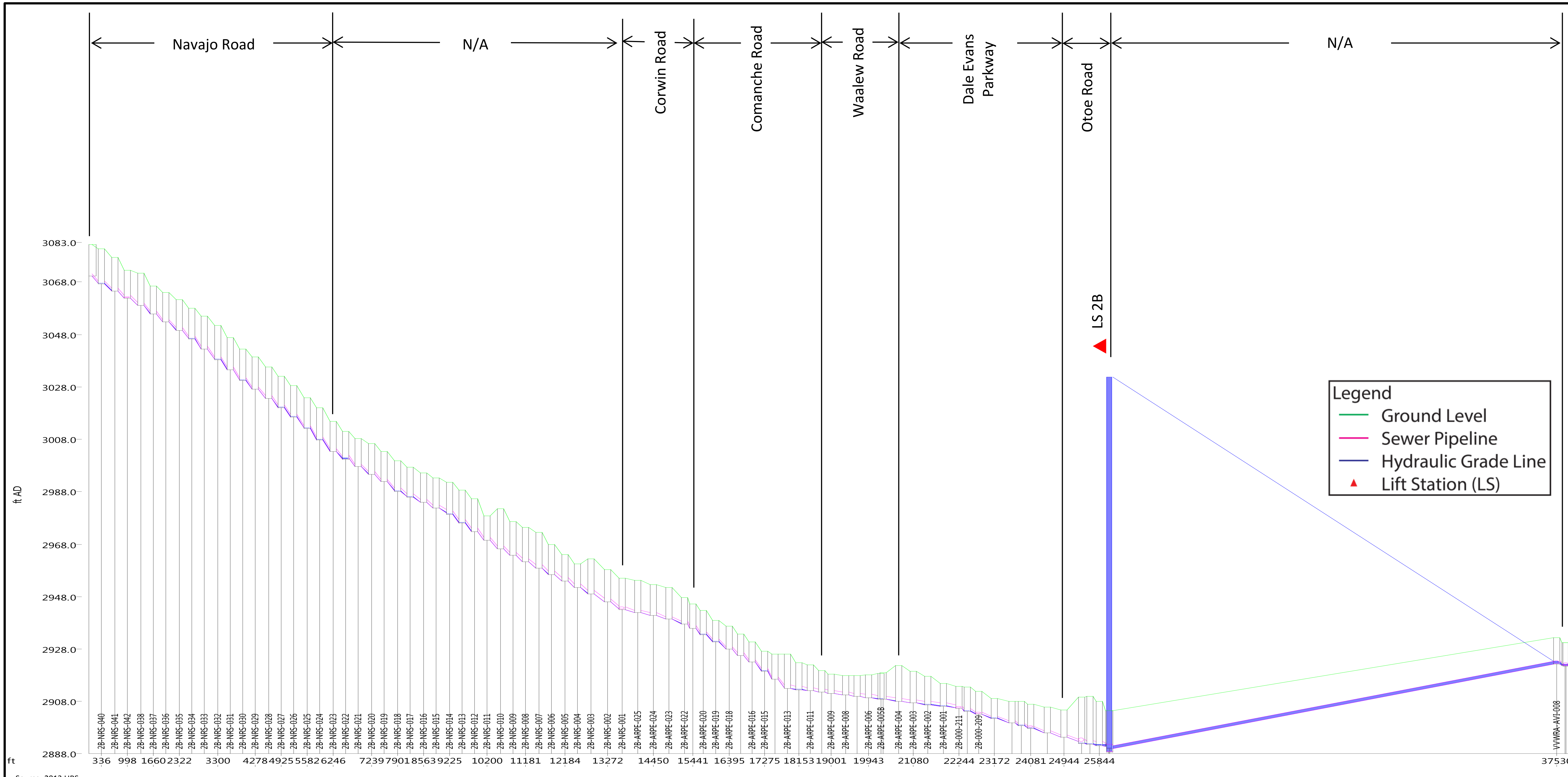
10.1.1.3.2 Evaluation of Existing Flows for Lift Station 2B (Northeastern Portion)

Figure 10-12 shows the pipeline segment upstream of Lift Station 2B along Otoe Road, and Dale Evans Parkway. This segment also travels along Waalew Road, Comanche Road, Corwin, and Navajo Road. The model results indicate no backing up of flows. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station, under existing dry weather conditions.

Figure 10-12: Modeled Results of Existing Flows for Lift Station 2B (Northeastern Portion)



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Modeled Results of Existing Flows for Lift Station 2B (Northeastern Portion)

Figure 10-12

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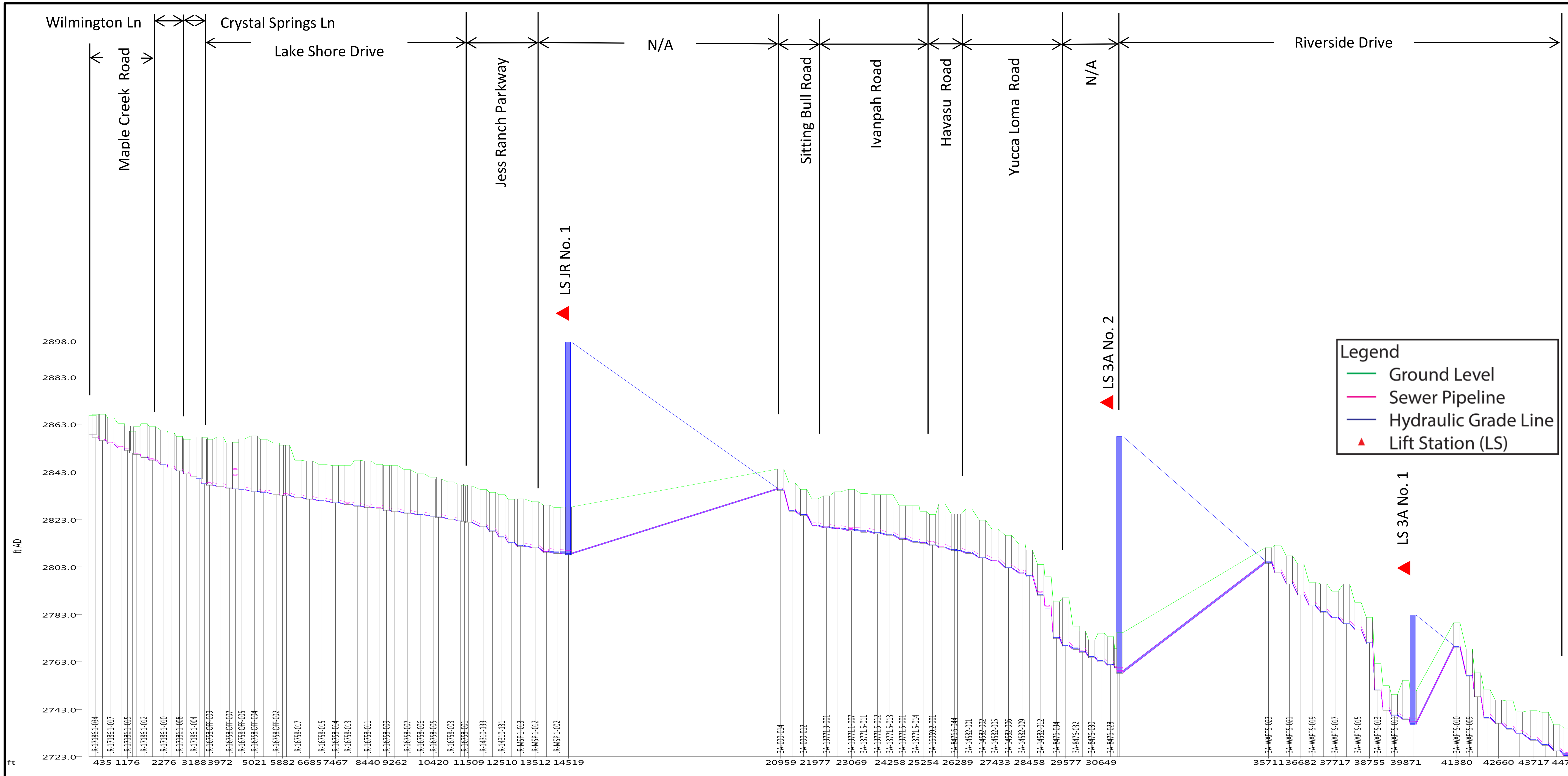
10.1.1.4 **Evaluation of Existing Flows for Jess Ranch, AD 3A, and Surrounding Areas**

The results of the simulation for Jess Ranch, AD 3A, and surrounding areas are shown in Figure 10-13. The pipeline segment through Jess Ranch and surrounding areas has adequate capacity to convey flows under existing conditions. The model simulation results show no surcharging of the pipeline segment under dry weather conditions.

Figure 10-13: Modeled Results of Existing Flows for Jess Ranch, AD 3A, and Surrounding Areas Pipeline Segment



Source: 2013 URS

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Modeled Results of Existing Flows for Jess Ranch, AD 3A, and Surrounding Areas Pipeline Segment

Figure 10-13

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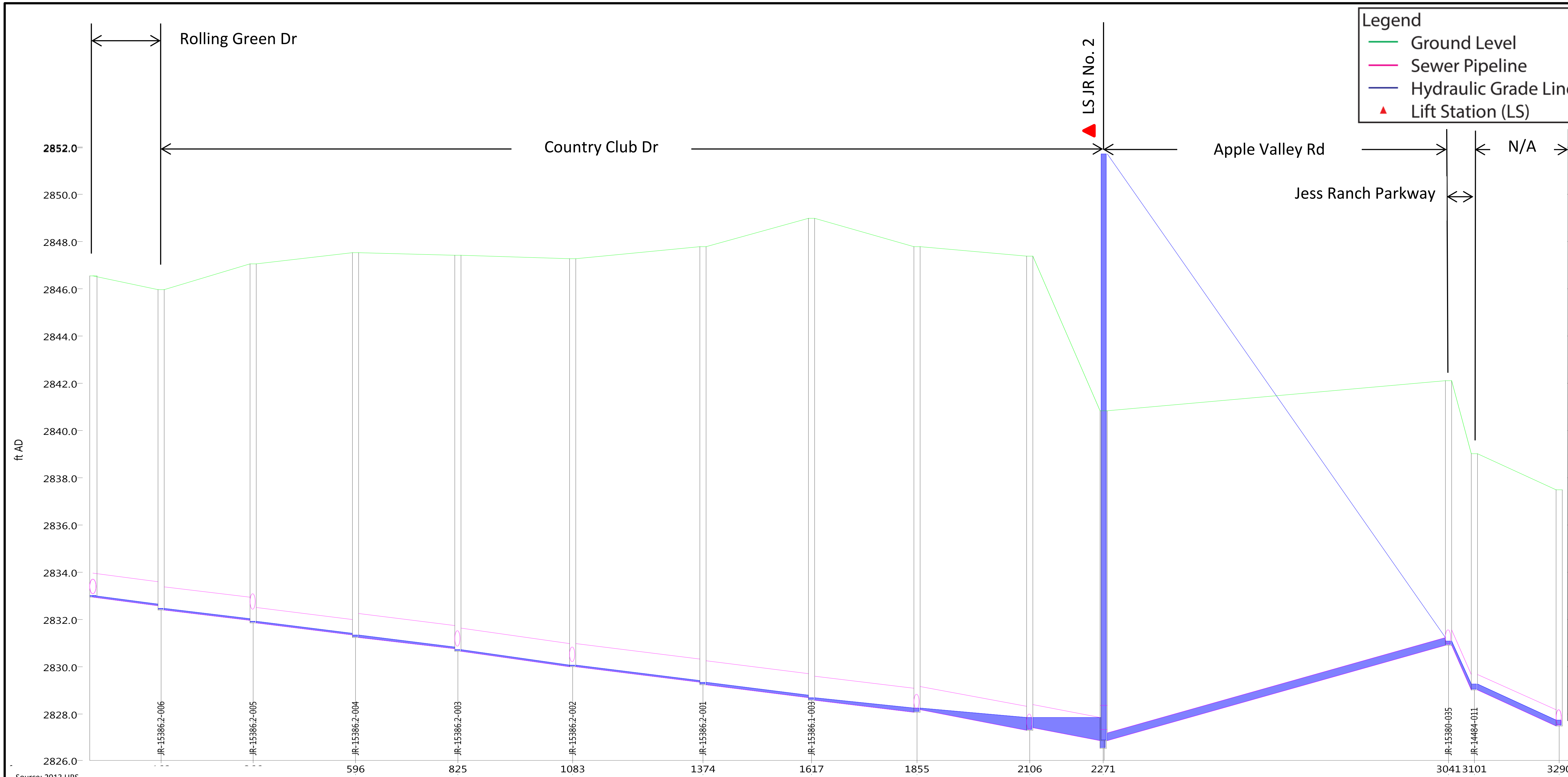
10.1.1.4.1 Evaluation of Existing Flows for Lift Station JR No. 2

The results of the model simulation for Lift Station JR No. 2 is shown on Figure 10-14. The model simulation results show no surcharging of the pipeline segment under existing dry weather conditions. The lift station has adequate capacity and no upgrades are necessary.

Figure 10-14: Modeled Results of Existing Flows for Lift Station JR No. 2



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Modeled Results of Existing Flows for Lift Station JR No. 2

Figure 10-14

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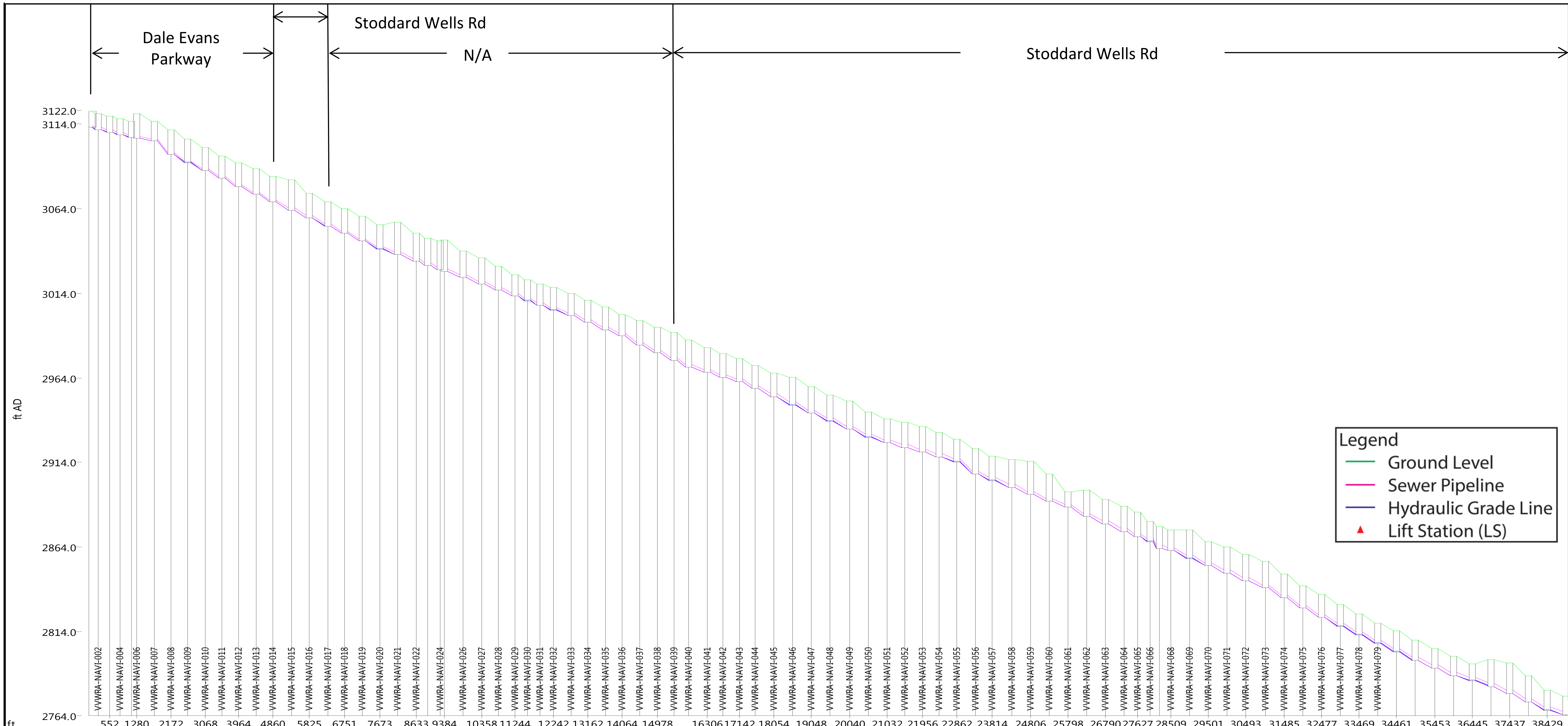
10.1.1.5 **Evaluation of Existing Flows for NAVI**

The model simulation result for the NAVI under dry weather conditions is shown in Figure 10-15. This pipeline segment has adequate capacity to covey flows during dry weather conditions. It should be noted that this pipeline currently receives flows mainly the Juvenile Detention Center. The pipeline has lots of room to handle additional flows from the proposed developments planned for the northern part of Apple Valley near the airport. The model results for this segment show that there is no surcharge in the NAVI during dry weather conditions.

Figure 10-15: Modeled Results of Existing Flows for NAVI Pipeline Segment



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Existing Flows for NAVI Pipeline Segment

Figure 10-15

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10.2 Future System Analysis

This analysis is similar to the existing system except the analysis was performed under future buildout conditions.

Land use data from the 2009 General Plan and Specific Plans were used to generate the future flows that were imported into the InfoWorks hydraulic model to simulate the future buildout conditions. No analysis were performed for intermediate years such as 2020 and 2030 due to unavailability of data about how much the system would have developed during these intermediate years. URS started to use SCAG population projection to make an assessment of the intermediate 2020 and 2030 years but found discrepancies with the data.

The InfoWorks model with the future buildout flows were simulated to assess the improvements needed to accommodate future buildout flows. Based on the modeling results, hydraulic deficiencies for the projected growth were identified. Further, the system capacity expansion was analyzed and the need for new pipes required to support growth projections were identified.

10.2.1 Evaluation of Future System Conditions

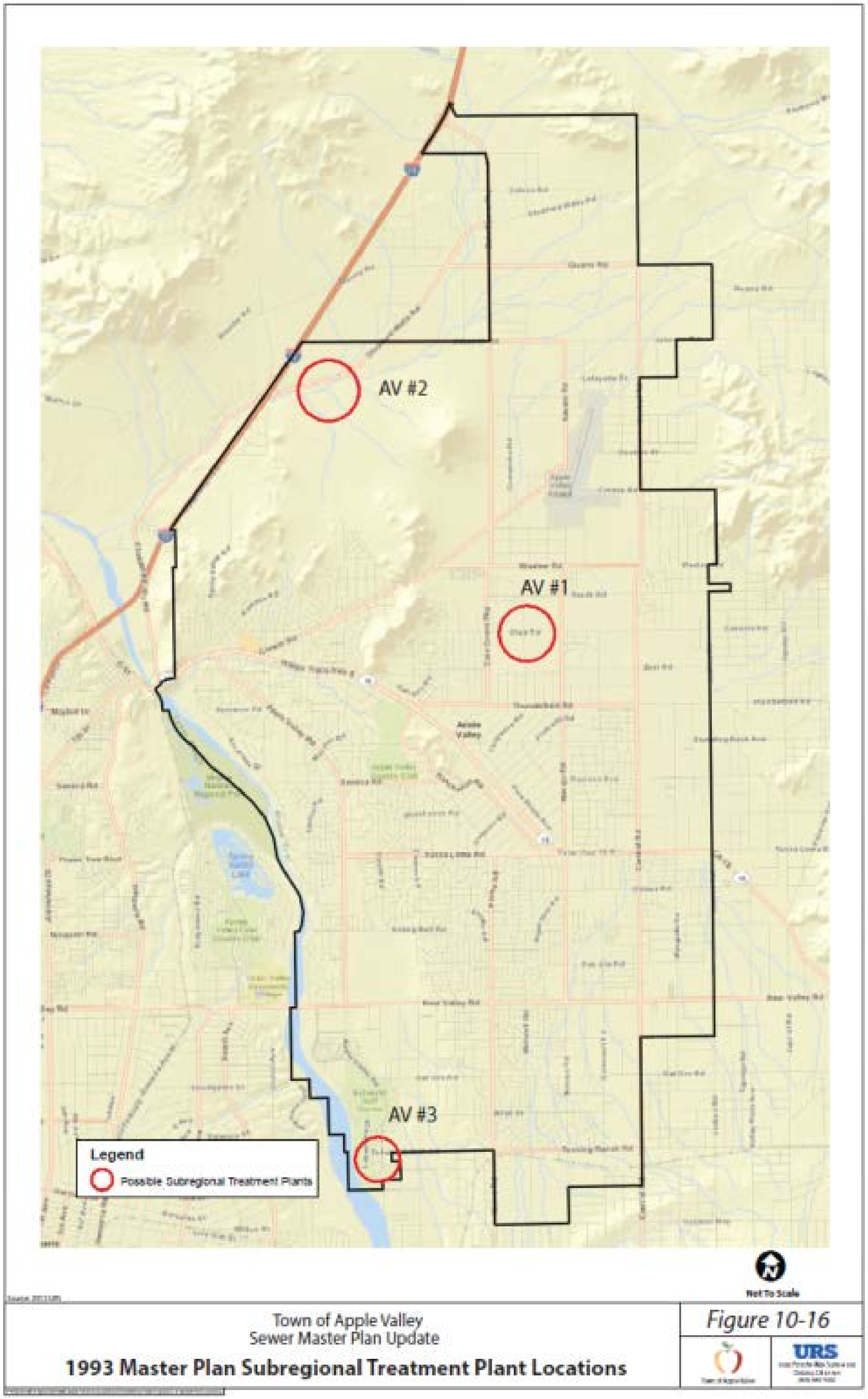
The objective of this section was to evaluate the impact of buildout flows on the existing system and to model the CIPs needed to accommodate the projected buildout flows.

To serve future development, the 1993 Master Plan's proposed sewer system improvements included directing projected future flow to three (3) proposed sub-regional wastewater treatment plants, as shown in Figure 10-16. As of the creation of this Sewer Master Plan Update, these plants have yet to be constructed. The assessment of the existing sewer system includes determining if the sewer infrastructure is adequate to serve future developments, not considering the future sub-regional wastewater treatment facilities, and developing options to upgrade the capacity of the sewer system where capacity is not met.

Currently, the VVWRA interceptor receives most of the wastewater flows generated from the Town's sewered developments. For the future scenario, some of the projected future flows in the northwest area of the Town were directed to the NAVI interceptor due to the interceptor's proximity and to relieve the overloading on the VVWRA pipeline. The proposed future flows were added in the InfoWorks model, and the model was run to simulate the flows generated under dry weather conditions.

The hydraulic profiles of the future system model runs (Figures 10-17 through 10-32) display the ground level elevation (green line), hydraulic grade line (blue line), and sewer pipeline (pink line). The following subsections present the results of the computer model runs for buildout system conditions.

Figure 10-16: 1993 Master Plan Subregional Treatment Plant Locations



10.2.1.1 **Evaluation of Buildout Flows for VVWRA Interceptor**

The simulation results for the buildout scenario for the VVWRA interceptor is shown in Figure 10-17 on the following page 10-35. The future flow projections incorporated into the hydraulic model show overflow problems within the VVWRA line.

As previously mentioned, the 1993 Master Plan identified locations of wastewater facilities needed to serve future developments. The flows in AD 2B were designed to flow to Lift Station 2B, located near Dale Evans Parkway and Otoe Road, with the intention of being compatible with a future sub-regional wastewater treatment plant. If the sub-regional wastewater treatment plant materializes, this will lessen the burden on the VVWRA interceptor sewer.

Based on the buildout allocation of flows to the model, the entire VVWRA pipeline segment is predicted to have inadequate capacity to receive the proposed flows from the buildout scenario. The future buildout model results show severe surcharging in the VVWRA interceptor. The model predicted SSOs at a number of locations as shown on Figure 10-17.

To develop the upgrades necessary to alleviate the overloading, URS used the InfoWorks model to simulate upgrading the VVWRA pipeline to a 30 inch diameter pipeline from its existing size. The surcharging in the VVWRA interceptor was reduced with this upgrade. Since this pipeline segment belongs to VVWRA, URS recommends the Town of Apple opening a discussion with VVWRA regarding this potential overloading during buildout. An agreement should be reached between the Town and VVWRA regarding whether this upgrade should be carried out or whether the new sub-regional interceptor should be implemented.

Figure 10-17: Modeled Results of Buildout Flows for VVWRA Interceptor Pipeline Segment

Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update

Modeled Results of Buildout Flows for VVWRA Interceptor Pipeline Segment

Figure 10-17

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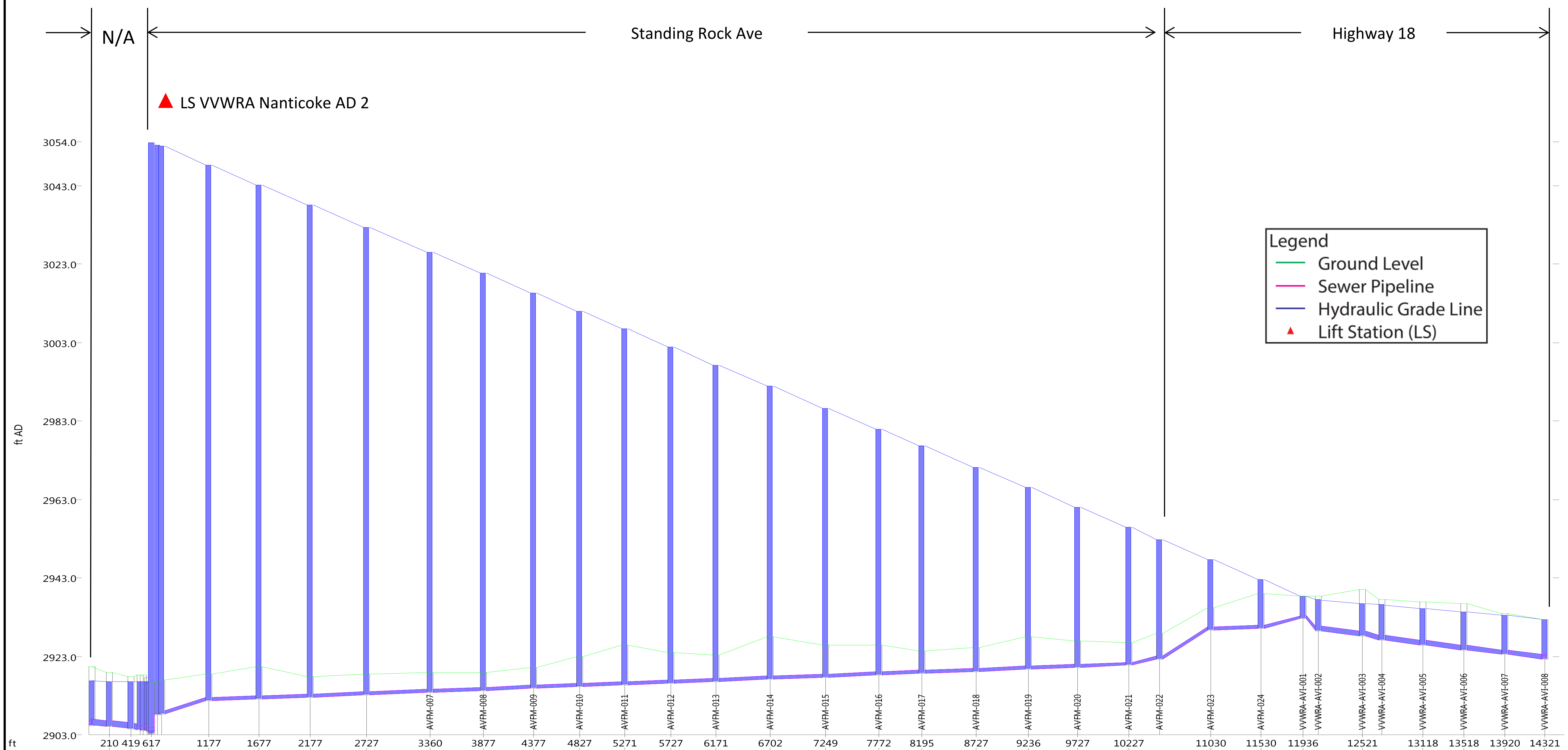
10.2.1.1.1 Evaluation of Buildout Flows for Lift Station VVWRA Nanticoke AD 2

There is backing up of flows to the pipeline immediately upstream of the Lift Station VVWRA Nanticoke AD 2 resulting in surcharging of this pipe segment (Figure 10-18). The backup is due to the lift station having inadequate capacity. To alleviate this problem, URS added two additional pumps using the larger of the existing pumps at the station. The forcemain diameter was increased from 10 inches to 16 inches in diameter.

This lift station and forcemain is the responsibility of VVWRA. URS provided the recommended upgrades in this report to enable full evaluation of the system improvements. URS assumes that VVWRA will be responsible for conducting further analysis to determine the upgrades needed at this station and its downstream pipeline segments on Highway 18, owned and operated by VVWRA.

Figure 10-18: Modeled Results of Buildout Flows for Lift Station VVWRA Nanticoke AD 2



Source: 2013 URS

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Modeled Results of Buildout Flows for Lift Station VVWRA Nanticoke AD 2

Figure 10-18

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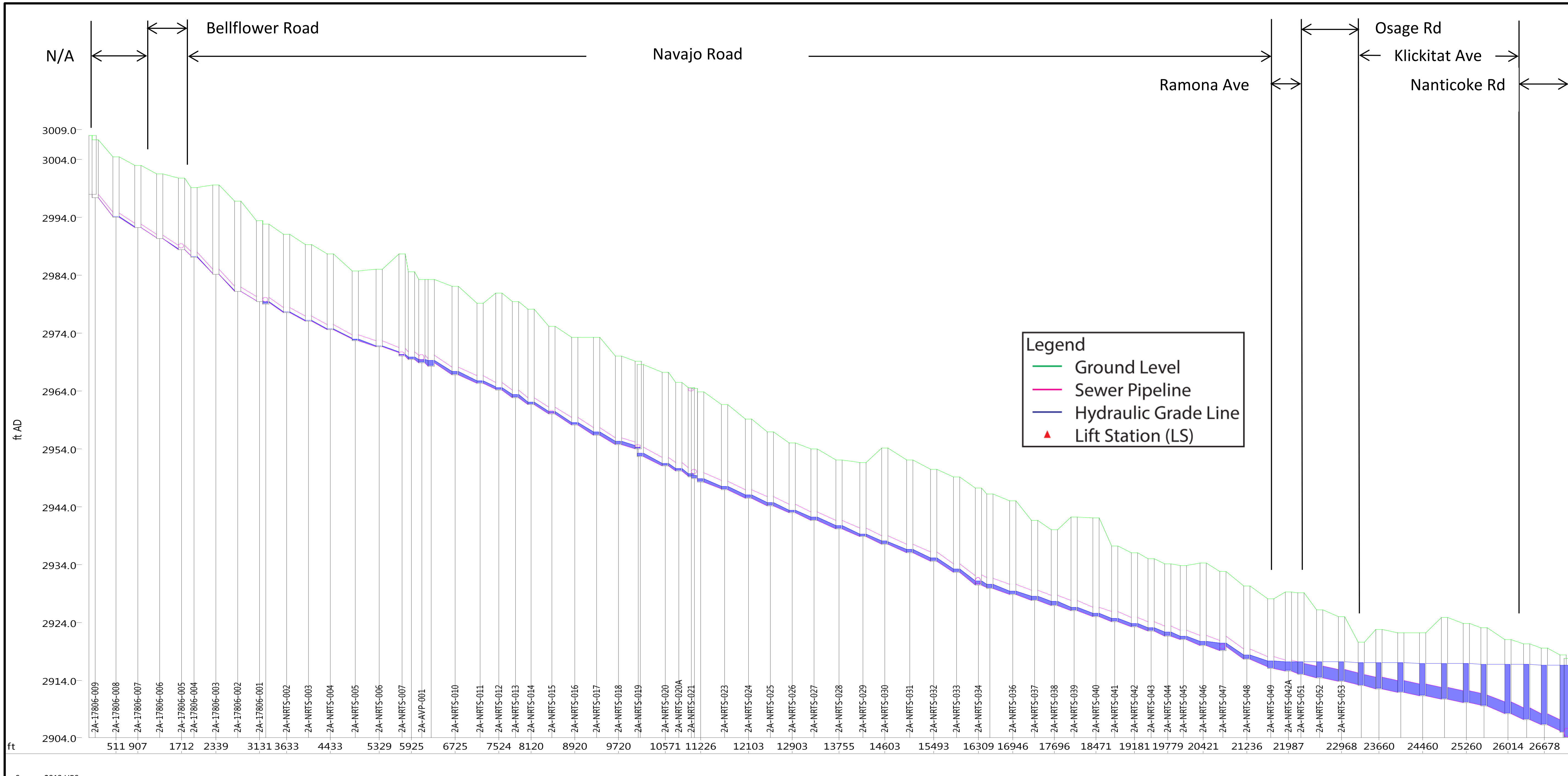
10.2.1.2 Evaluation of Buildout Flows for AD 2A and Surrounding Areas

The model results shows that the pipe segment conveying flows from AD 2A and surrounding areas to Lift Station VVWRA Nanticoke AD 2 along Navajo Road (Figure 10-19) has inadequate capacity to convey flows during dry weather conditions at the downstream end near the Lift Station VVWRA Nanticoke AD 2. The surcharging is due to inadequate capacity of the Lift Station VVWRA Nanticoke AD 2.

As can be seen on Figure 10-18, most of the pipeline segment has adequate capacity except the downstream end from Ramona Avenue to the Lift Station VVWRA Nanticoke AD 2. This surcharging will be alleviated by implementation of VVWRA interceptor and the Lift Station VVWRA Nanticoke AD 2 improvements.

Figure 10-19: Modeled Results of Buildout Flows for AD 2A and Surrounding Areas Pipeline Segment



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Modeled Results of Buildout Flows for AD 2A and Surrounding Areas Pipeline Segment

Figure 10-19

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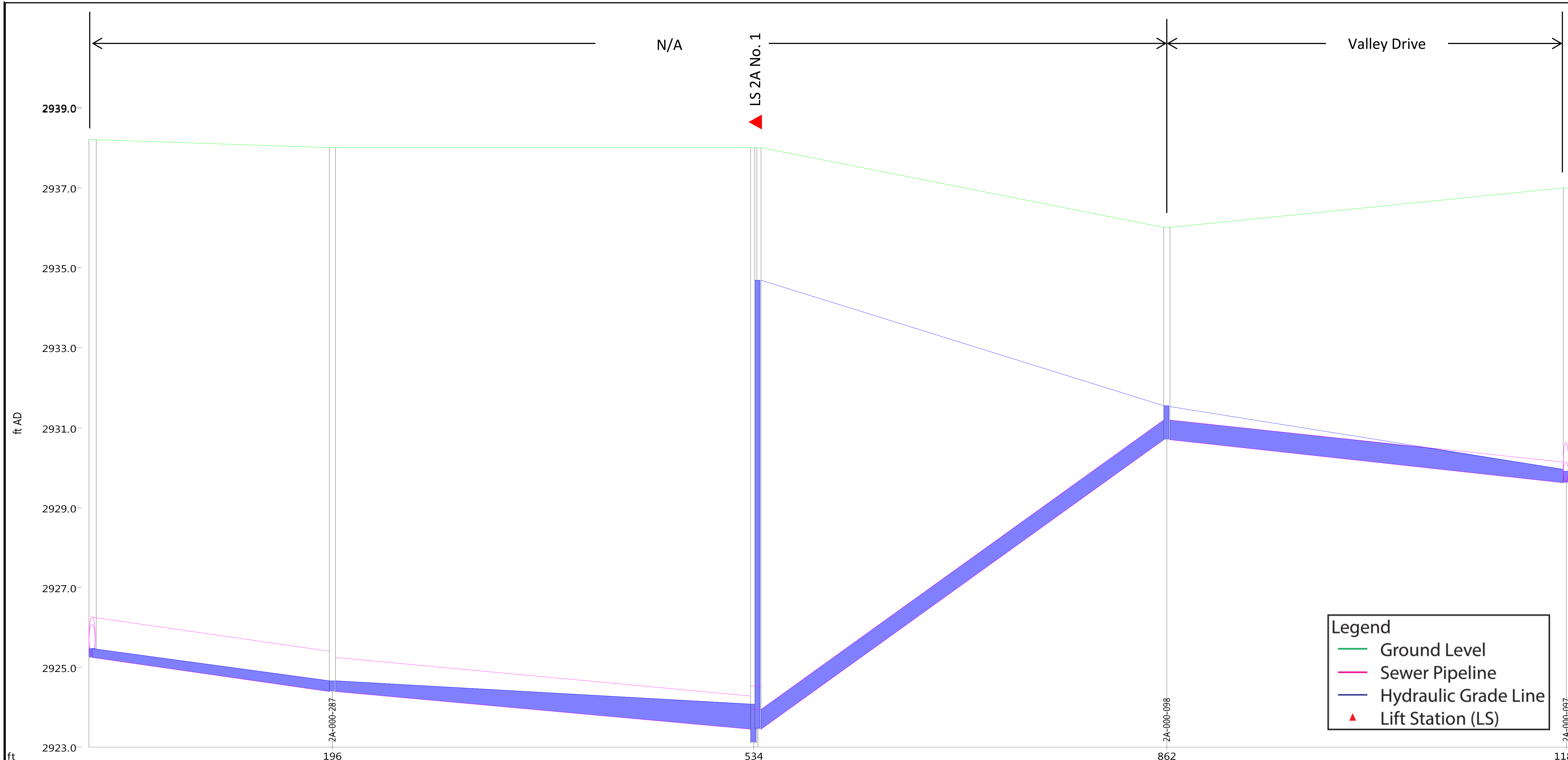
10.2.1.2.1 Evaluation of Buildout Flows for Lift Station 2A No. 1

The model results shown on Figure 10-20 indicate that the Lift Station 2A No. 1 has adequate capacity to convey flows under future buildout dry weather conditions. No backing up of flows is predicted upstream of Lift Station 2A No. 1, and no upgrades are required for this Lift Station at future buildout.

Figure 10-20: Modeled Results of Buildout Flows for Lift Station 2A No. 1



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station 2A No. 1

Figure 10-20

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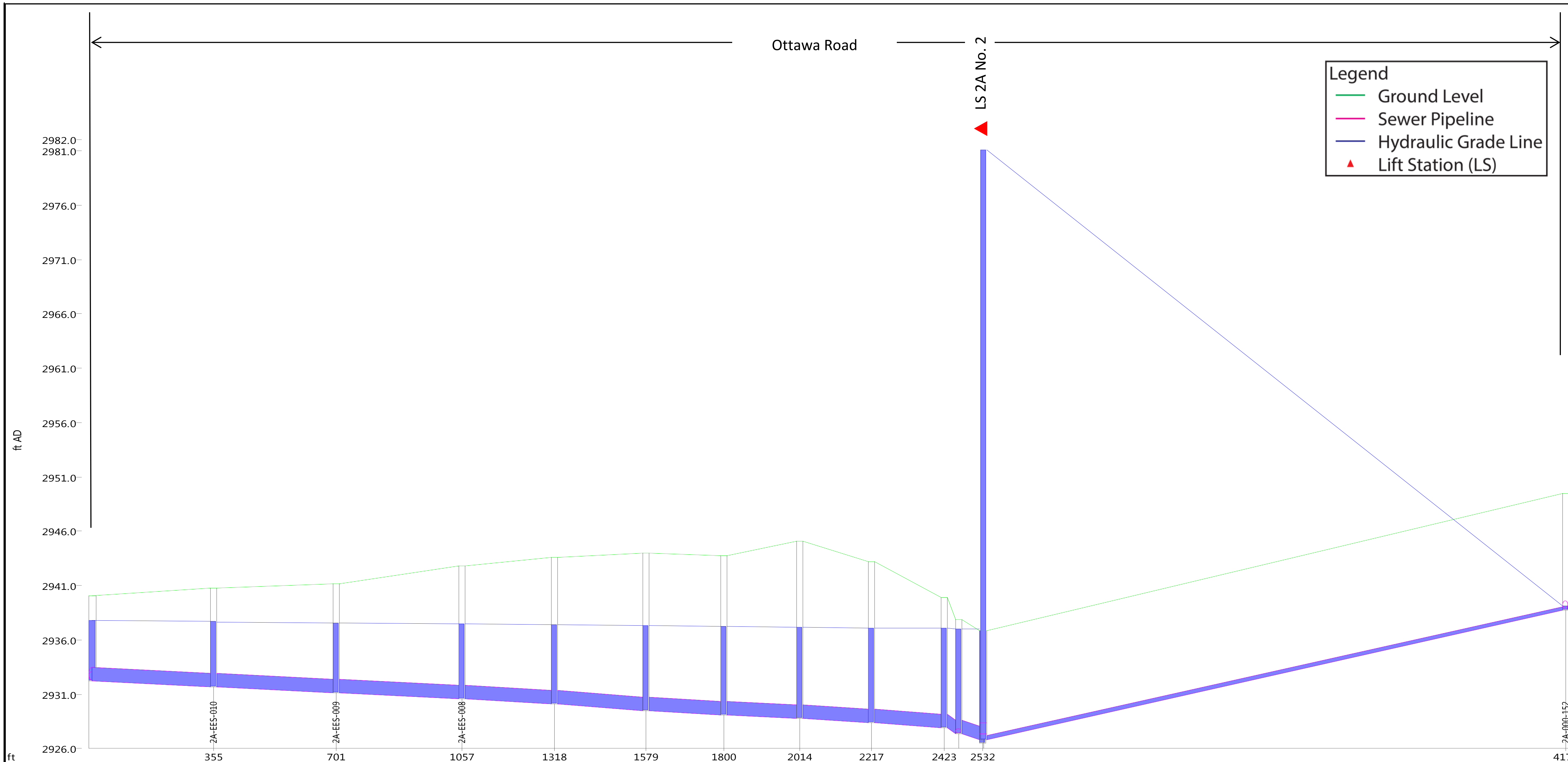
10.2.1.2.2 Evaluation of Buildout Flows for Lift Station 2A No. 2 (East of LS 2A No. 2)

Figure 10-21 shows there is backing up of flows to the pipeline immediately upstream of the lift station LS 2A No. 2 resulting in surcharging of the surrounding pipe segments. The backup is due to the lift station having inadequate capacity. To alleviate this problem, URS modeled a pump with a similar pump curve characteristics as the existing pump and upgraded the forcemain. The original forcemain is 4 inches in diameter and is inadequate to covey the lift station discharge at buildout, therefore URS increased the forcemain diameter to 12 inches. The problem predicted by the model buildout was alleviated by these changes.

Figure 10-21: Modeled Results of Buildout Flows for Lift Station 2A No. 2 (East of LS 2A No. 2)



Source: 2013 URS

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Modeled Results of Buildout Flows for Lift Station 2A No. 2 (East of LS 2A No. 2)

Figure 10-21

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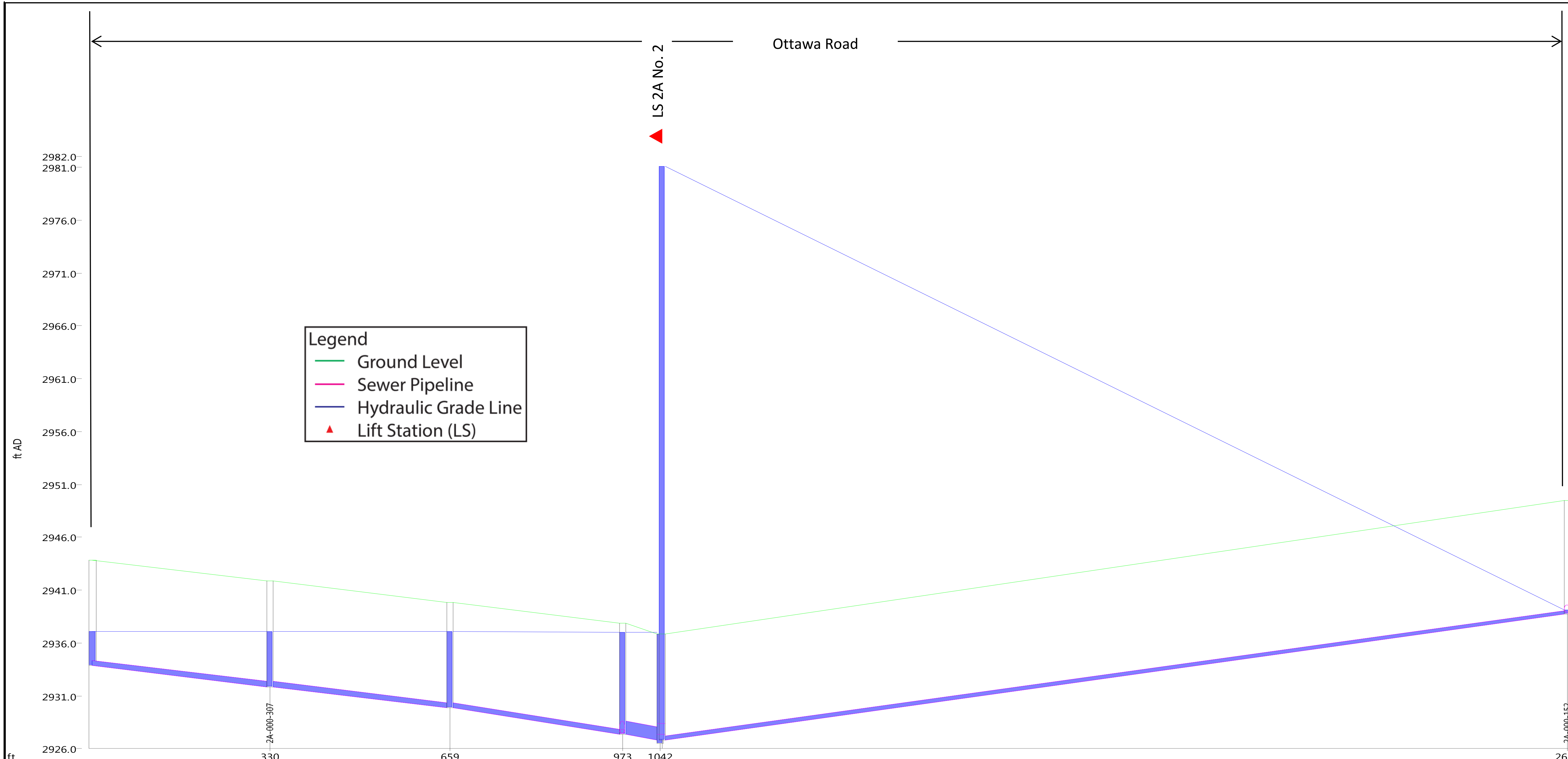
10.2.1.2.3 Evaluation of Buildout Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)

The second segment travels east along Ottawa Road, between Malaki Road and Central Road until reaching Lift Station 2A No. 2. Modeled results (Figure 10-22) show surcharging in the manholes along the entire segment.

Upgrading this lift station per discussions in previous section will alleviate the hydraulic deficiency identified with Lift Station 2A No. 2.

Figure 10-22: Modeled Results of Buildout Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)



Source: 2013 URS

Town of Apple Valley
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Modeled Results of Buildout Flows for Lift Station 2A No. 2 (West of LS 2A No. 2)

Figure 10-22

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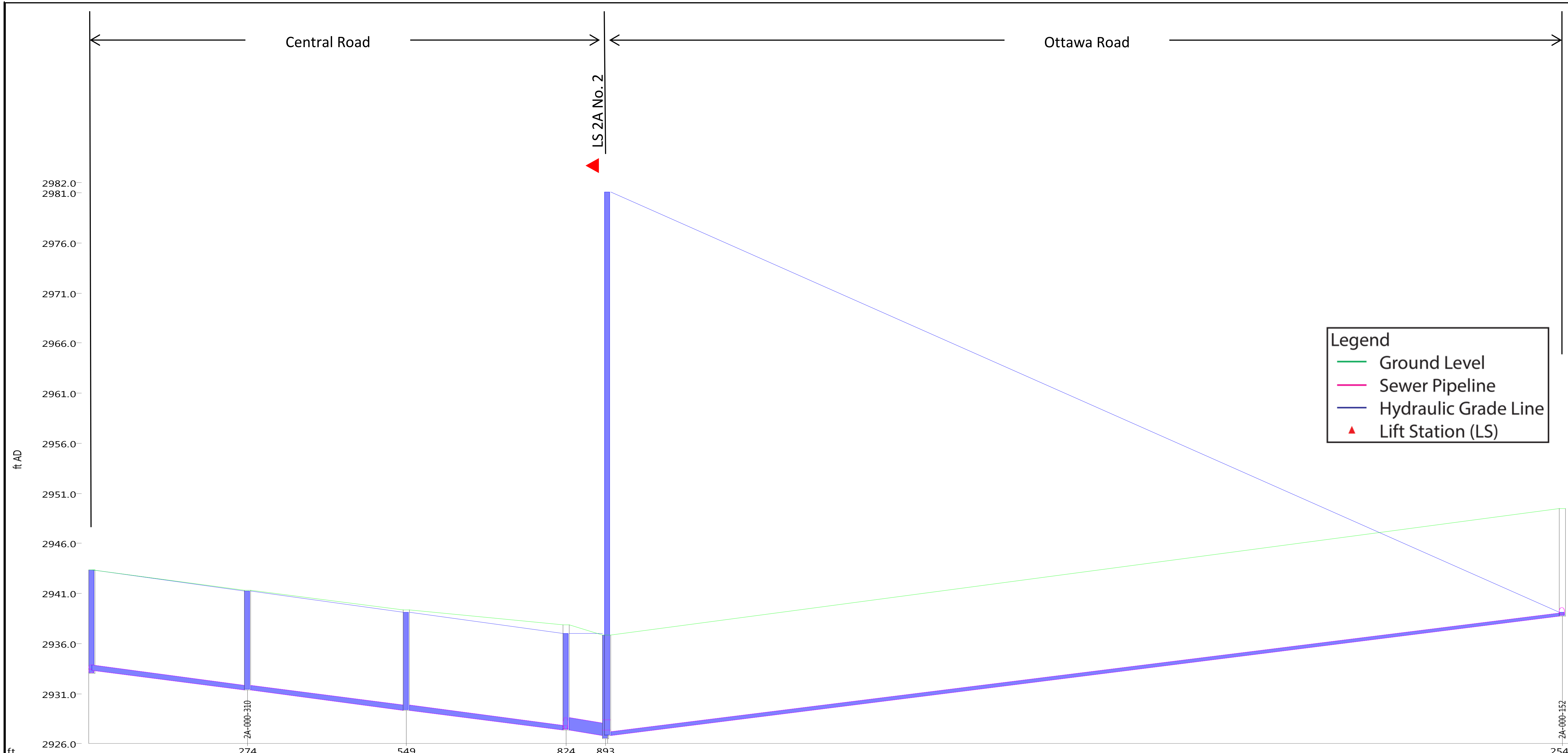
10.2.1.2.4 Evaluation of Buildout Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)

The third segment includes flow south of Lift Station 2A No. 2. Modeled results (Figure 10-23) show backup of flows with SSOs predicted.

Upgrading this lift station per discussions in previous section will alleviate the hydraulic deficiency identified with Lift Station 2A No. 2.

Figure 10-23: Modeled Results of Buildout Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)



Source: 2013 URS

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Modeled Results of Buildout Flows for Lift Station 2A No. 2 (South of LS 2A No. 2)

Figure 10-23

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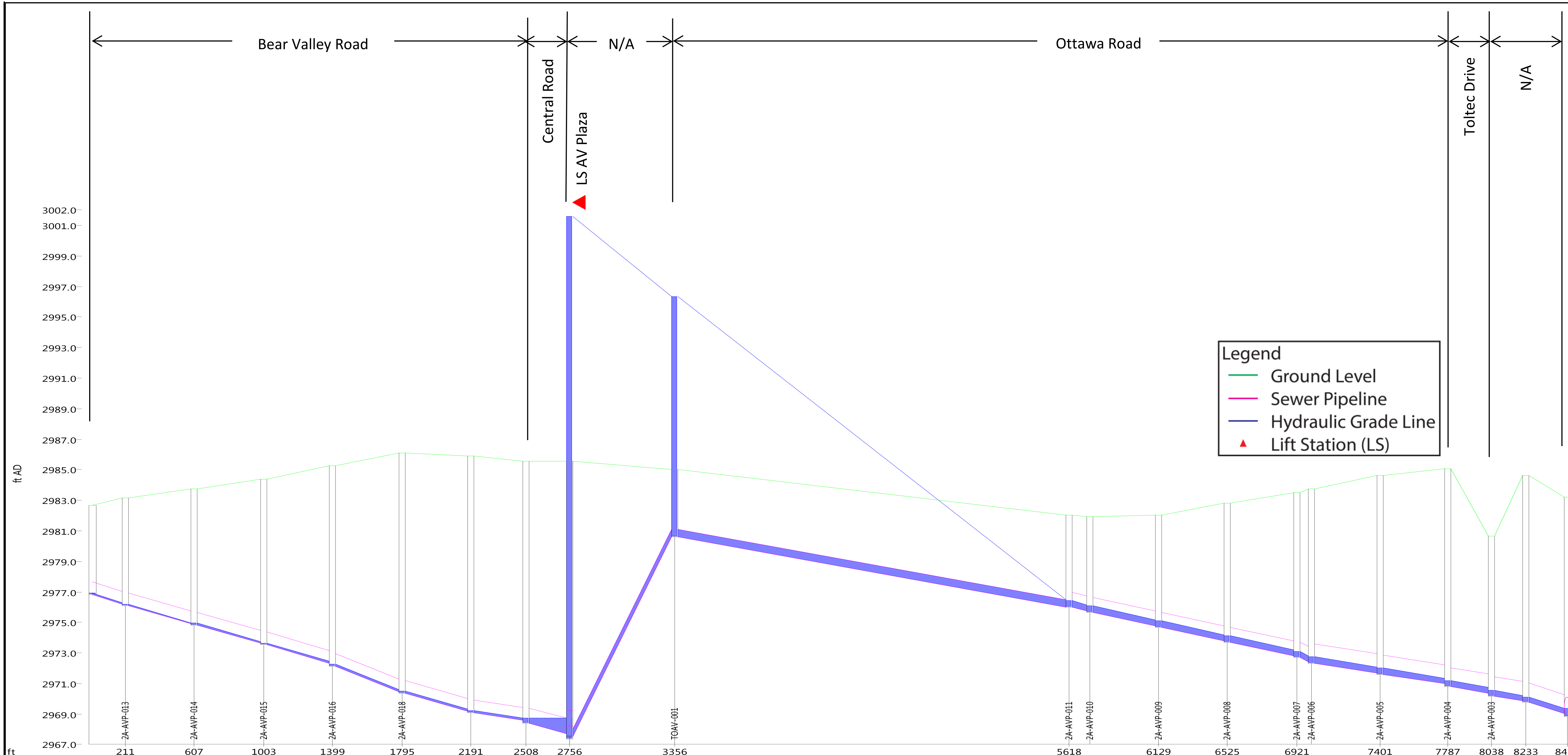
10.2.1.2.5 Evaluation of Buildout Flows for Lift Station AV Plaza

Figure 10-24 shows the pipeline segment upstream of Lift Station AV Plaza along Central Road. The model results indicate a slight backup in the last pipeline segment leading to this lift station. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station under future buildout conditions.

Figure 10-24: Modeled Results of Buildout Flows for Lift Station AV Plaza



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station AV Plaza

Figure 10-24

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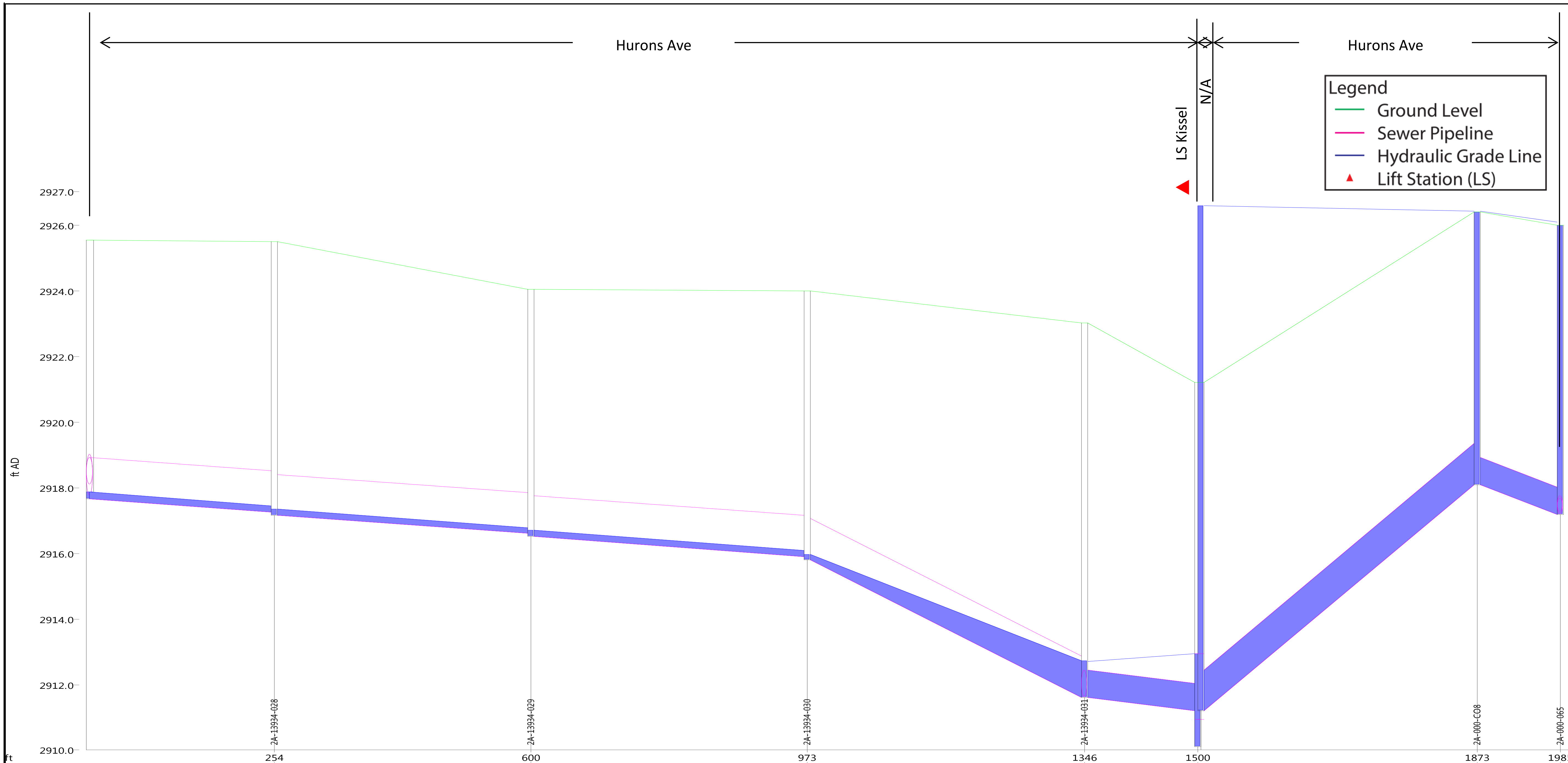
10.2.1.2.6 Evaluation of Buildout Flows for Lift Station Kissel

Figure 10-25 shows the pipeline segment upstream of Lift Station Kissel along Hurons Avenue. The model results indicate a slight backup in the last pipeline segment leading to this lift station. The lift station has adequate pumping capacity, and no upgrades are needed for this lift station under future buildout conditions.

Figure 10-25: Modeled Results of Buildout Flows for Lift Station Kissel



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station Kissel

Figure 10-25

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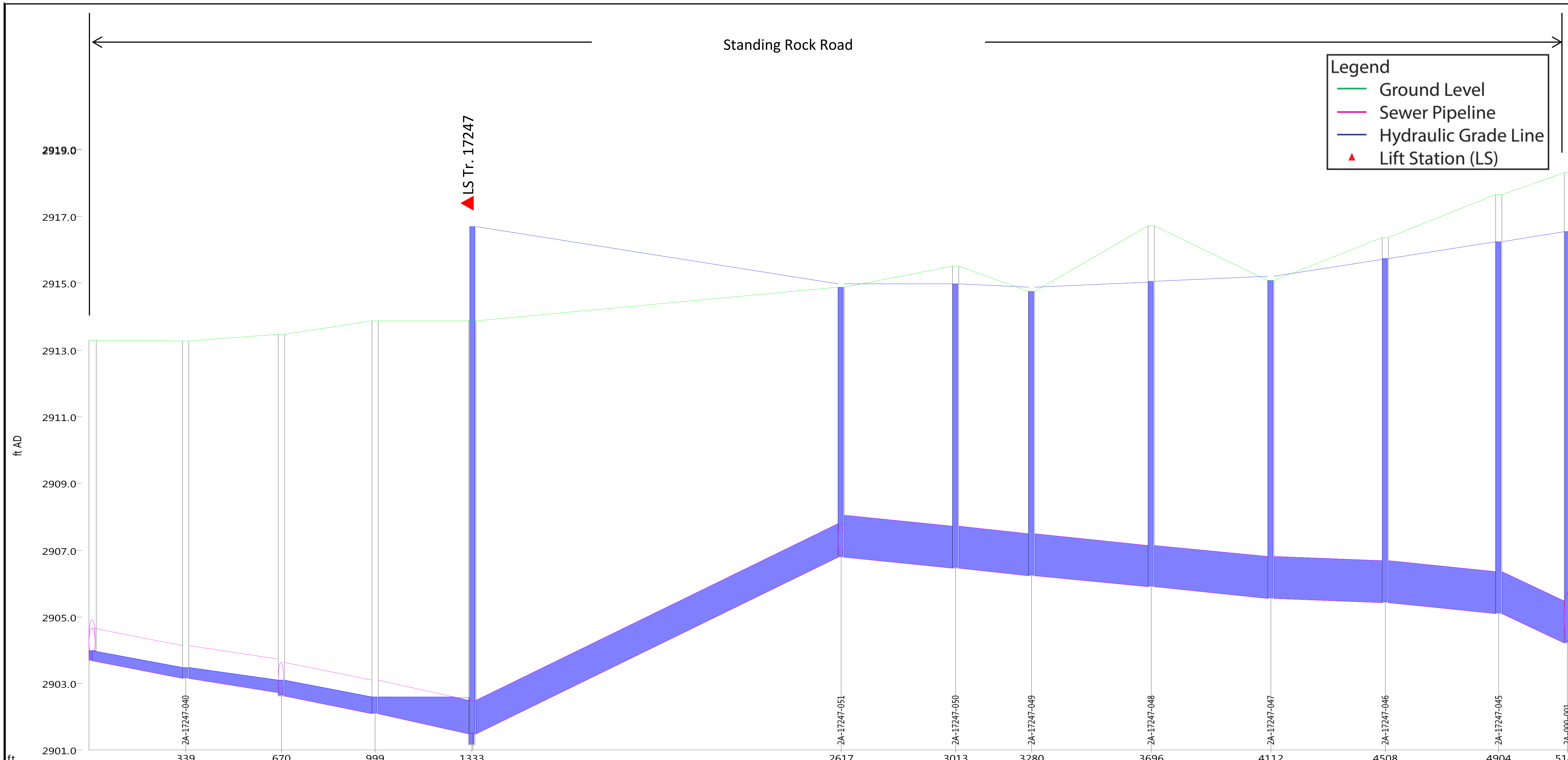
10.2.1.2.7 Evaluation of Buildout Flows for Lift Station Tr. 17247

There are currently no existing pumps at this lift station location under existing conditions. The sewer leading to the Lift Station VVWRA Nanticoke AD 2 in Standing Rock Avenue experiences backing and surcharging (Figure 10-26). This affects the ability of the Lift Station TR 17247 to discharge freely into the pipeline leading to the Lift Station VVWRA Nanticoke AD 2. To alleviate the hydraulic bottleneck experienced at this location, there is a need to upgrade the downstream Lift Station VVWRA Nanticoke AD 2 and its forcemain. This solution enables the TR 17247 pump to freely discharge to the Lift Station VVWRA Nanticoke AD 2. The recommended upgrades include installation of two pumps each 150 gpm capacity.

Figure 10-26: Model Results of Buildout Flows for Lift Station Tr. 17247



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station Tr. 17247

Figure 10-26

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10.2.1.3 **Evaluation of Buildout Flows for AD 2B and Surrounding Areas**

There are two pipe segments that discharge to Lift Station 2B.

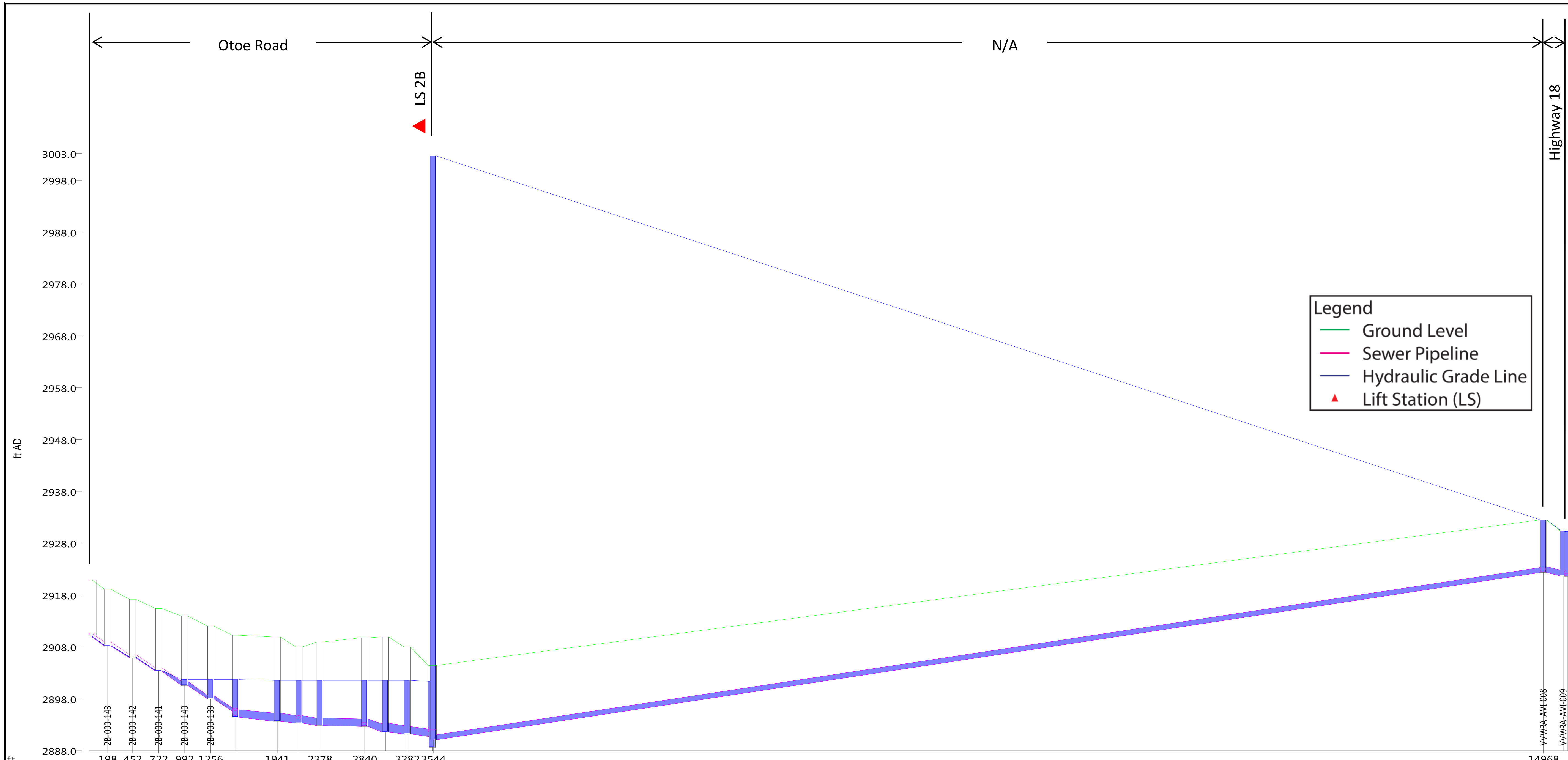
The southeastern portion travels along Otoe Road and Wichita Road.

The northeastern portion travels along Otoe Road, Dale Evans Parkway, Waalew Road, Comanche Road, Corwin Road and Navajo Road.

10.2.1.3.1 Evaluation of Buildout Flows for Lift Station 2B (Southwestern Portion)

Figure 10-27 shows the pipeline segment upstream of Lift Station 2B along Otoe Road, and Wichita Road. The model results indicate backing up of flows. The lift station has inadequate capacity to convey flows. The problem is also exercabated by the surcharging in the VVWRA interceptor at buildout. The recommended solution is to upgrade the VVWRA interceptor and also upgrade the lift station. Upgrading of the Lift Station VVWRA Nanticoke AD 2 involves adding two additional pumps at the station. The forcemain size was also increased from 12 inch diameter to 18 inch diameter. The upstream pipe segments to Lift Station 2B was increased from 18 inch to 30 inch for the segment in Otoe Road.

Figure 10-27: Modeled Results of Buildout Flows for Lift Station 2B (Southwestern Portion)



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station 2B (Southwestern Portion)

Figure 10-27

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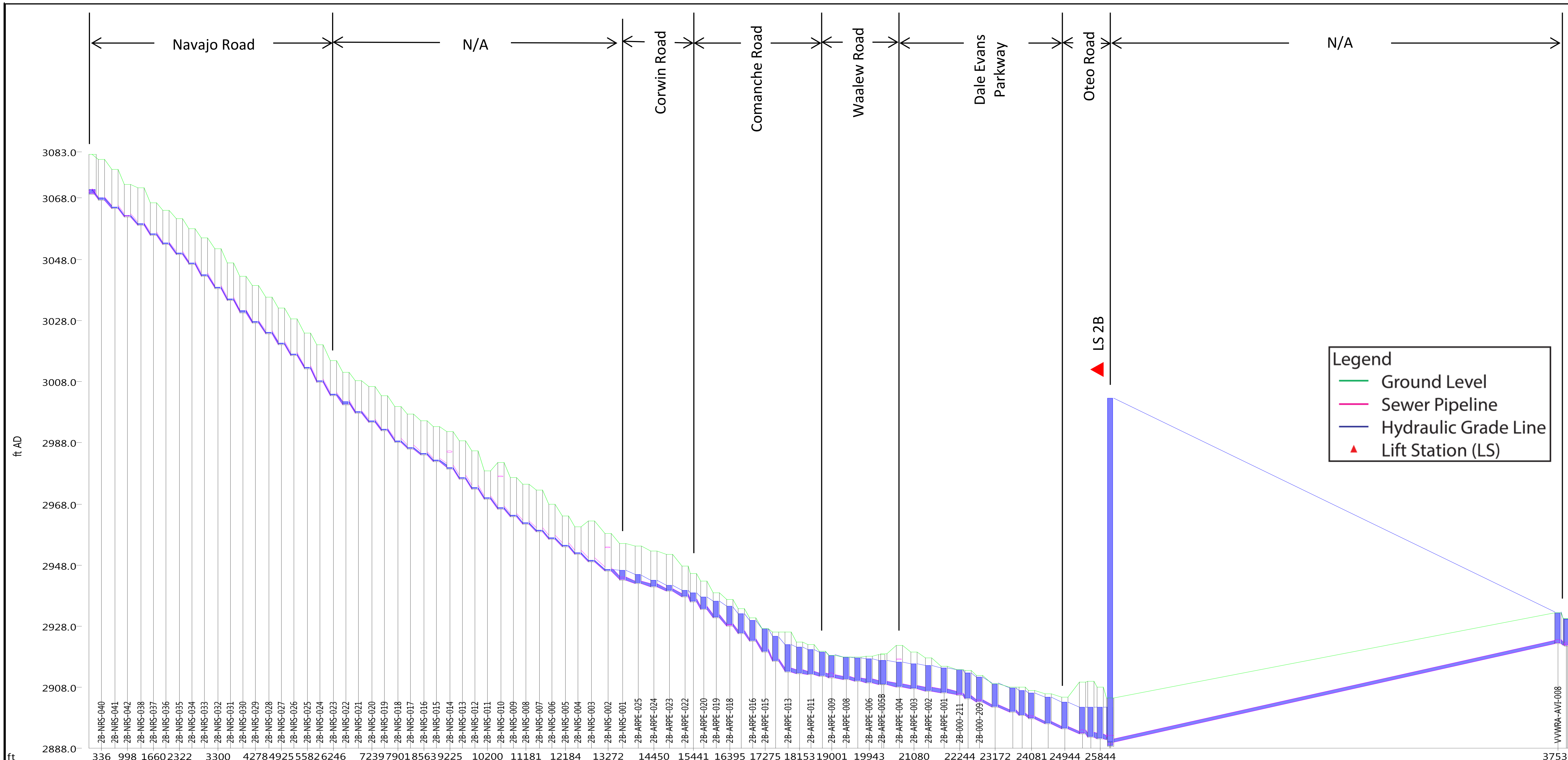
10.2.1.3.2 Evaluation of Buildout Flows for Lift Station 2B (Northeastern Portion)

Figure 10-28 shows the pipeline segment upstream of Lift Station 2B along Otoe Road, and Dale Evans Parkway. The model results indicate backing up of flows. The lift station has inadequate capacity to convey flows. The problem is also exercabated by the surcharging in the VVWRA interceptor at buildout. Similar to the southwestern portion of the segment to this lift station, the recommended solution is to upgrade the VVWRA interceptor and also upgrade the lift station. Upgrading of the lift station involves adding two additional pumps at the station. The forcemain size was also increased from 12 inch diameter to 18 inch diameter. The upstream pipe segments to Lift Station 2B was increased from 18 inch to 30 inch for the segment in Otoe Road, from 15 inch to 21 inch in Dale Evans Parkway, and from 15 inch to 21 inch in Waalew Road.

Figure 10-28: Modeled Results of Buildout Flows for Lift Station 2B (Northeastern Portion)



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station 2B (Northeastern Portion)

Figure 10-28

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10.2.1.4 Evaluation of Buildout Flows for Jess Ranch, AD 3A, and Surrounding Areas

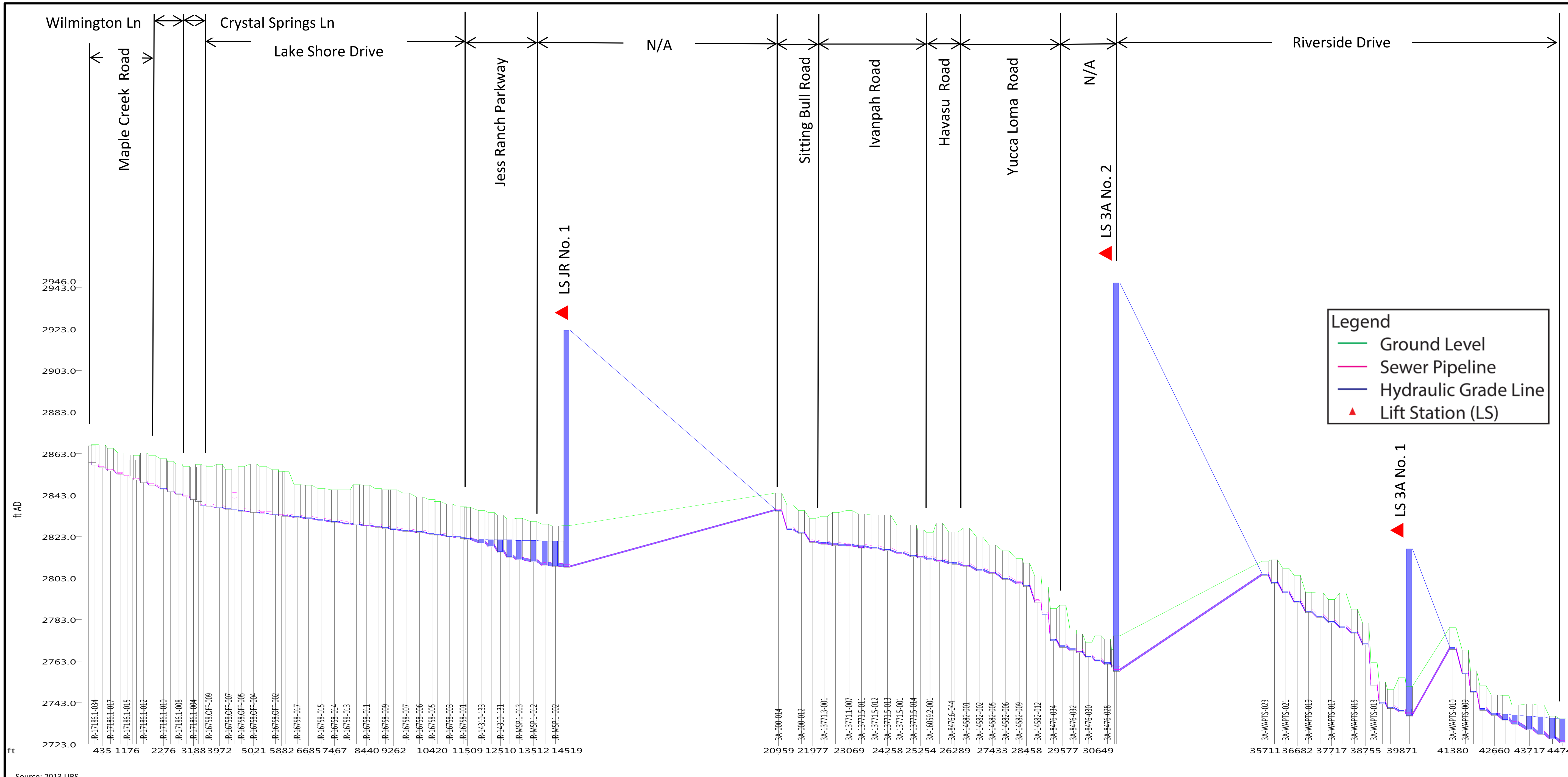
Figure 10-29 shows pipeline segment in AD 3A and surrounding areas. The model indicates surcharging of the pipe segment upstream of Lift Station JR No. 1 along Jess Ranch Parkway.

The backing up predicted by the model in Jess Ranch Parkway upstream of Lift Station JR No. 1 is due to inadequate force main capacity at Lift Station JR No. 1.

Due to overloading in the VVWRA interceptor, the pipe segment in Riverside Drive is predicted to experience surcharging. This surcharging will be alleviated by upsizing of the VVWRA interceptor outfall pipes.

Lift Station 3A No. 2 has adequate capacity to convey flows at buildout.

Figure 10-29: Modeled Results of Buildout Flows for Jess Ranch, AD 3A, and Surrounding Areas Pipeline Segment



Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Jess Ranch, AD 3A, and Surrounding Areas Pipeline Segment

Figure 10-29

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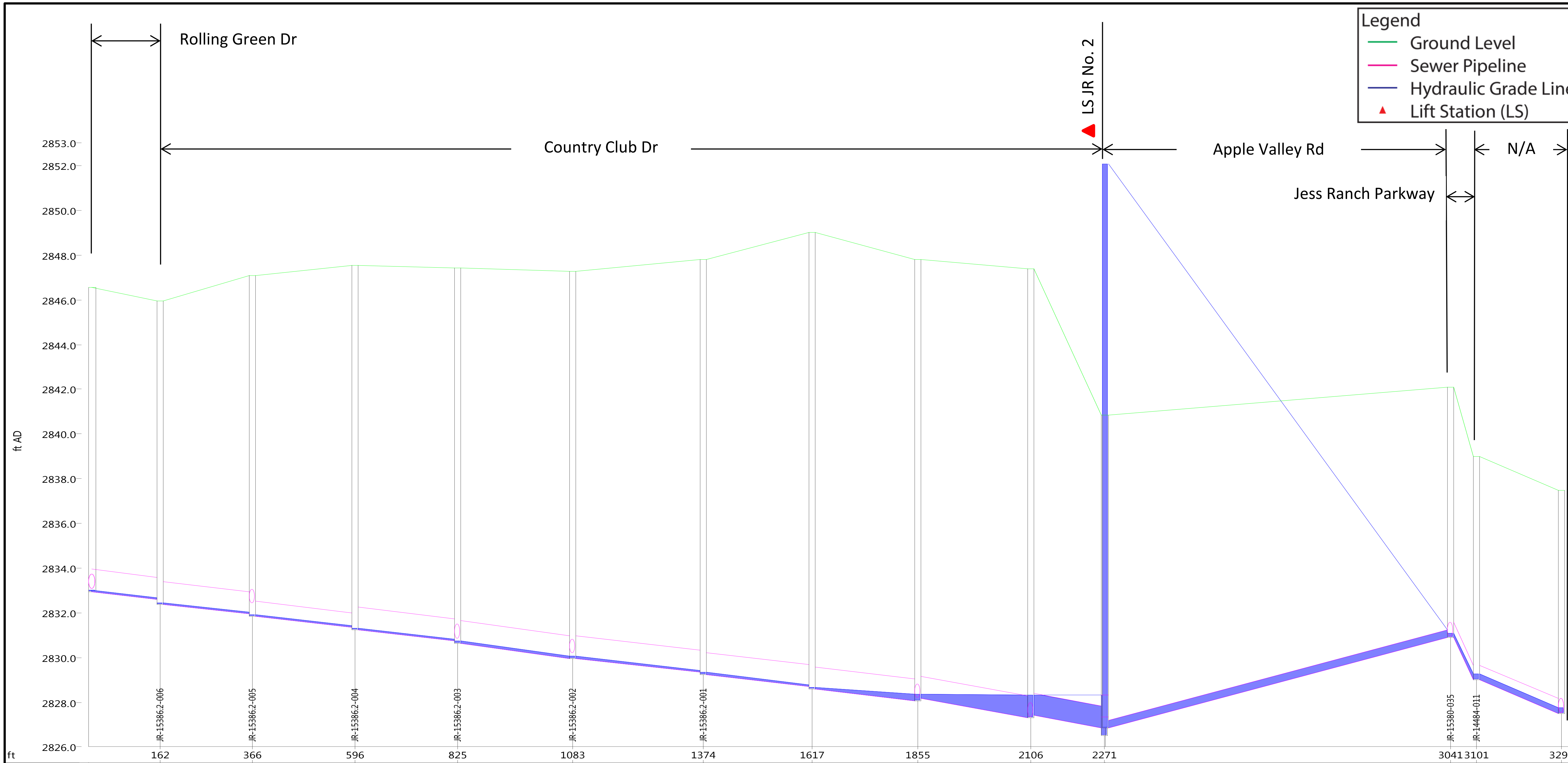
10.2.1.4.1 Evaluation of Buildout Flows for Lift Station JR No. 2

The model predicts a negligible surcharging of the pipe segment upstream of Lift Station JR No. 2 (Figure 10-30). The lift station has adequate capacity to convey flows at future buildout conditions.

Figure 10-30: Modeled Results of Buildout Flows for Lift Station JR No. 2



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station JR No. 2

Figure 10-30

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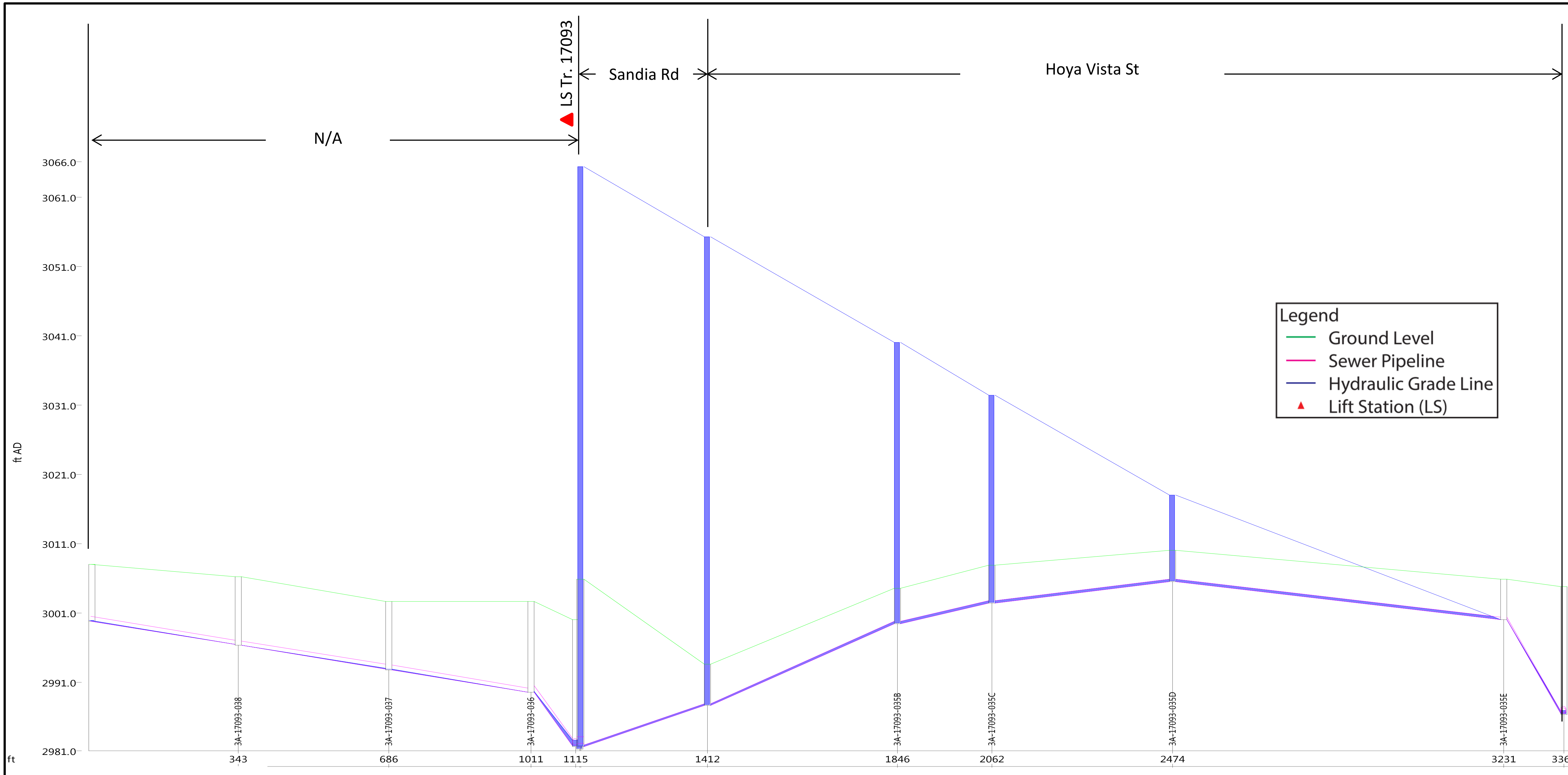
10.2.1.4.2 Evaluation of Buildout Flows for Lift Station Tr. 17093

Figure 10-31 shows there is no backing up of flows in the pipeline located immediately upstream of Lift Station 17093. The lift station has adequate pump capacity.

Figure 10-31: Model Results of Buildout Flows for Lift Station Tr. 17093



Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for Lift Station Tr. 17093

Figure 10-31

Town of Apple Valley

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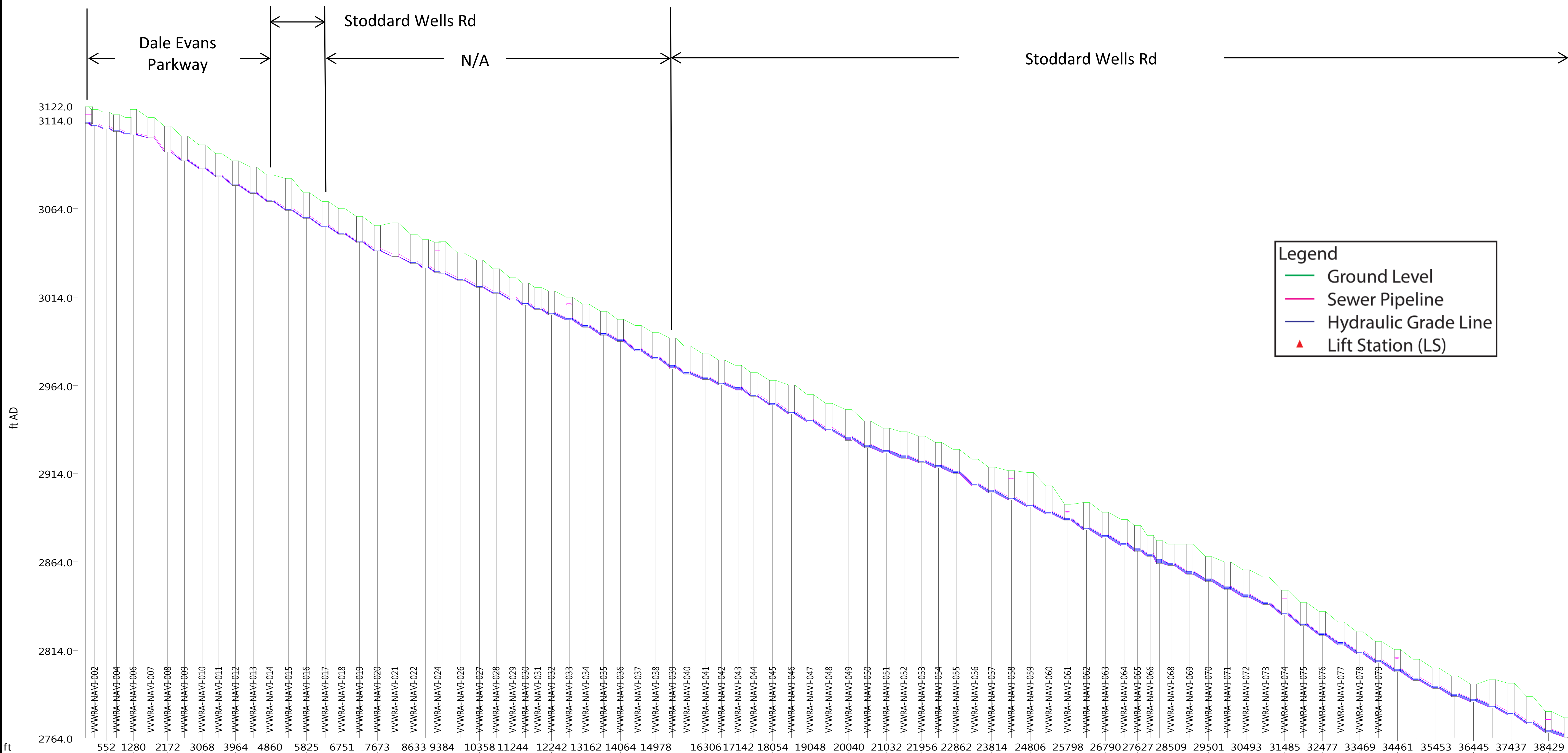
10.2.1.5 **Evaluation of Buildout Flows for NAVI**

The future buildout model results shown in Figure 10-32 indicates that the NAVI has adequate capacity to receive the projected flows from the development around the airport and the Golden Triangle without a need for upgrading the sewer pipe diameter. The NAVI has room to receive additional flows during future buildout dry weather conditions.

Figure 10-32: Modeled Results of Buildout Flows for NAVI Pipeline Segment



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update
Modeled Results of Buildout Flows for NAVI Pipeline Segment

Figure 10-32

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11.0 CONDITION ASSESSMENT

In September 2009, the Town of Apple Valley began conducting closed circuit television (CCTV) inspections of the sewer collection system in-house to properly manage, inspect, and maintain the sewer pipelines. Prior to this date, the inspections were directed to outside contractors and required for all new pipeline installations. The CCTV surveillance is part of the Town's Sanitary Sewer Management Plan which provides guidelines to properly manage, operate, inspect, and maintain all components of the Town's sewer collection system.

The pipeline segments are surveyed on a five (5) year cycle, latest cycle beginning January 2010. The total footage of video inspected by Town staff since September 2009 is 310,379 linear feet. The total footage of video inspected by contractors prior to September 2009 is 166,286 linear feet. Approximately 35 percent of the Town's sewer system has been inspected by the staff since the latest inspection cycle. The inspections are completed in a systematic approach; all Tracts within an Assessment District are surveyed before moving on to the following Assessment District. Any issues (i.e., roots, heavy grease, descaling, etc.) found are handled immediately by the Town staff using an ENZ bulldog.

The Town provided URS the available CCTV information conducted in September 2012. URS used the information provided by the Town staff to perform the condition assessment of the sewer collection system, using Pipeline Assessment and Certification Program (PACP) scoring system of the National Association of Sewer Service Companies (NASSCO).

The Town staff maintains record of the historic problem areas which was provided to URS as shown in Table 11-1.

Table 11-1: Town of Apple Valley Preventive Maintenance Locations Log

AD	Nearest Intersection	Frequency	Issue	Comments
Interceptor	Hwy 18 & Apple Valley Rd.	Monthly	Siphon	North side dirt road next to the County storm drainage channel.
Jess Ranch	Palo Verde Dr. & Ash St.	Bi-Monthly	Diapers (adult)	In condo parking lot.
Jess Ranch	Town Center Dr. & Apple Valley Rd.	Bi-Monthly	Sag	Old wet well in grass south of block wall.
Jess Ranch	Town Center Dr. & Apple Valley Rd.	Bi-Monthly	Sag	Downstream MH northeast of above MH in parking lot.
1A	Wika Rd. & Muni Rd.	Monthly	Grease	Visual in road. Due to Turbelance & Surchage.
1B	Corwin Rd. & Wintun Rd.	Quarterly	Grit	Intersection in road.
1C	Hwy 18 North & Tao Rd.	Bi-Monthly	Grit	Intersection in road.
1C	Hwy 18 North & Kasota Rd.	Night Bi-Monthly	Grit	Intersection in road.
7972	Siskiyou Rd. & Siskiyou Ct.	Quarterly	Grit	Visual northeast corner behind curb.
2A	Rancherias Rd. & Ottawa Rd.	Quarterly	Roots	Check for surcharge MH#3 (MH north of Ottawa Rd.).
1B	Hwy 18 North at Hospital	Monthly	Grease w/	Manhole west of storm drain.

AD	Nearest Intersection	Frequency	Issue	Comments
			sag to East	
Jess Ranch	Jess Ranch Pkwy. & Palo Verde Dr.	Bi-Annual	Sag	Manhole intersection.

11.1 CONDITION ASSESSMENT

URS reviewed approximately 2,676 feet of CCTV data to identify defects and possible remediation procedures within the sewer pipelines. A total of 10 pipeline segments, shown in Figure 11-1, were selected by the Town of Apple Valley based on historic maintenance issues.

The pipeline segments were reviewed and graded based on the observations made from the CCTV recordings. The general assignment of pipe condition grades were based on NASSCO PACP defect coding listed in Table 11-2.

Table 11-2: Grading System of Pipe Condition

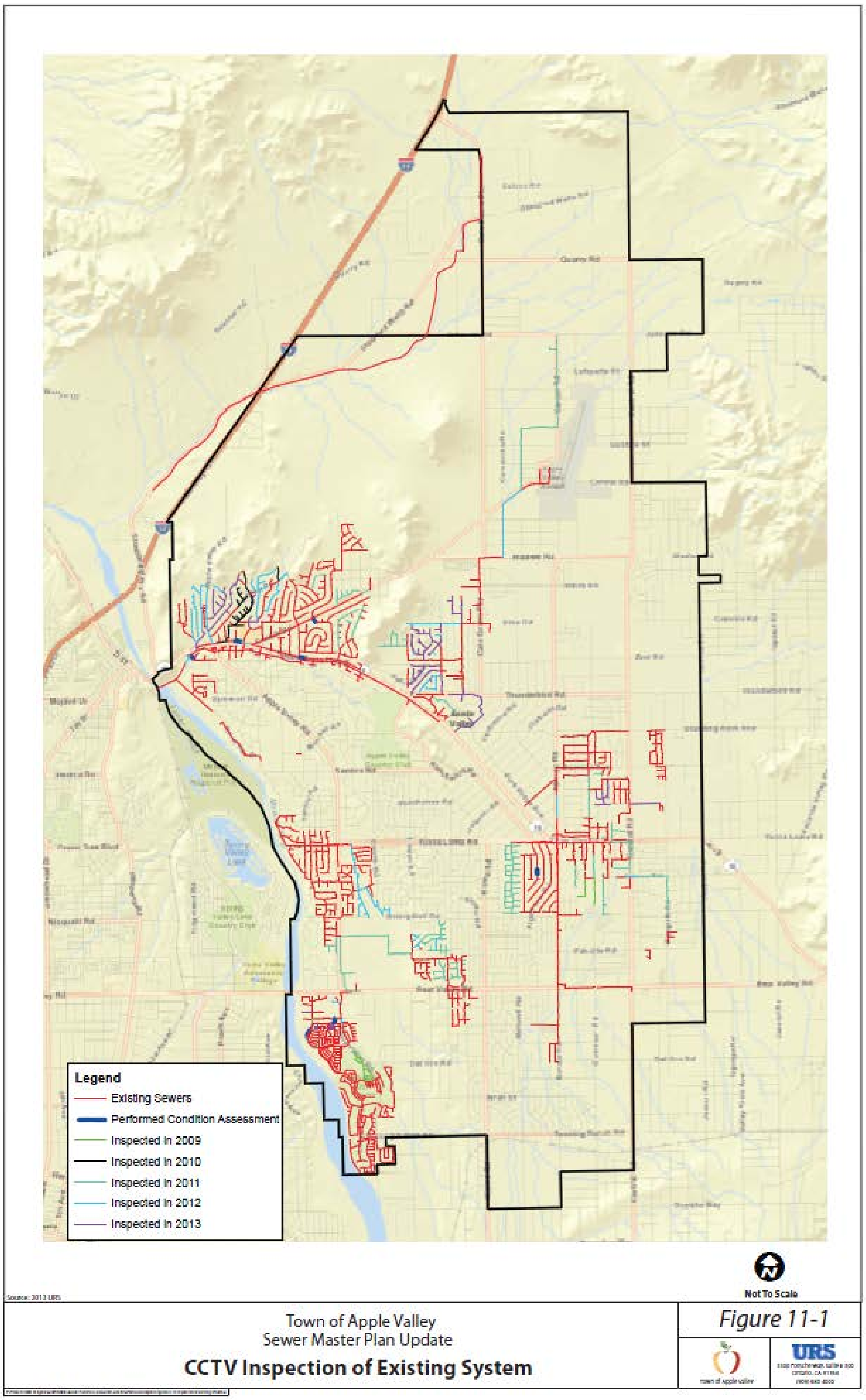
Grade	Pipe Condition
Grade 5	Collapsed or collapse imminent
Grade 4	Collapse likely in foreseeable future
Grade 3	Collapse unlikely in near future
Grade 2	Minimal collapse risk
Grade 1	Acceptable structural condition

11.2 CCTV DATA AND CONDITION ASSESSMENT RESULTS

CCTV data was collected to assess the condition of sewer pipelines, observing for signs of stress, structural defects, breaks, leaks or cracks. Sewer condition and deterioration assessments conducted are summarized in the following subsections.

Figure 11-1 shows the location of pipeline segments inspected by CCTV to date.

Figure 11-1: CCTV Inspection of Existing System



11.2.1 Location 1: St. Mary's Hospital / 18 North

Mainline ID

Upstream: 1B-000-003
Downstream: 1B-000-002

Summary and Findings

Location 1 was surveyed a total distance of 342.2 feet, from upstream manhole 1B-000-003 to downstream manhole 1B-000-002. There are multiple locations where the video equipment was submerged (Figure 11-2) which resulted in a very poor image quality.

There were two structural defects, sags, observed within the pipe segment. The first sag, located at the beginning of the upstream manhole, spanned 270 feet. The second sag, located at 300.3 feet, spanned 41.9 feet.

Blisters within the lining of the pipe were found 296.7 feet from the upstream manhole (Figure 11-3). It is inconclusive whether or not blisters exist prior to this location due to the poor quality of the video. The condition grade assigned to this segment is Grade 3 due to the unlikelihood of collapse in the near future of this segment.

Figure 11-2: Manhole Segment 1B-000-0003 to 1B-000-002

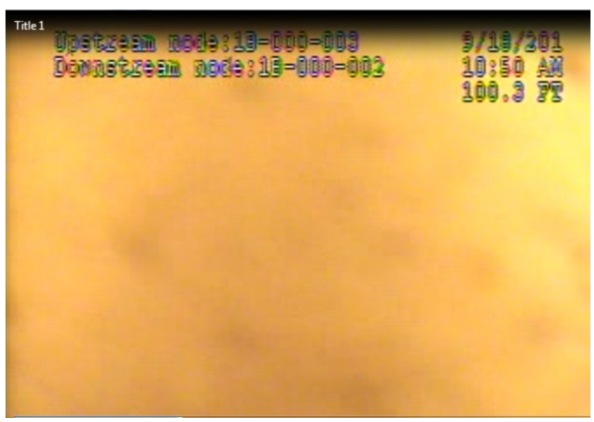


Figure 11-3: Manhole Segment 1B-000-003 to 1B-000-002

11.2.2 Location 2: St. Mary's Hospital / 18 North

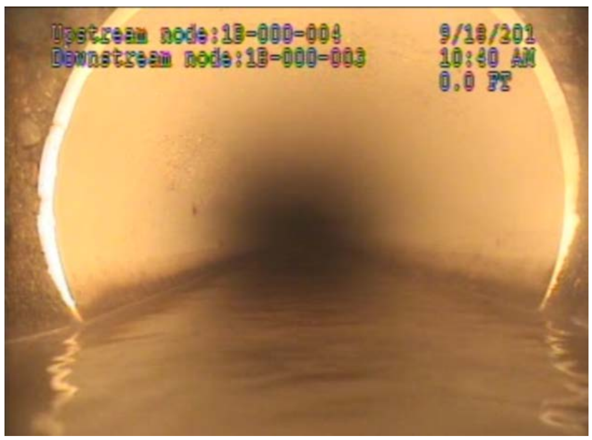
Mainline ID

Upstream: 1B-000-004
Downstream: 1B-000-003

Summary and Findings

The section surveyed for Location 2 began at upstream manhole 1B-000-004 and ended 156.5 feet downstream, 203.5 feet before reaching manhole 1B-000-003. The quality of the video recording was clear. From a structural standpoint, there was sag observed in the pipe, starting at the upstream manhole (Figure 11-4). The sag spanned throughout the distance surveyed. No other defects were found. The condition grade assigned to this segment is Grade 3.

Figure 11-4: Manhole Segment 1B-000-004 to 1B-000-003



11.2.3 Location 3: 18 North / Tao

Mainline ID

Upstream: 1C-3382-009
Downstream: 1C-3382-008

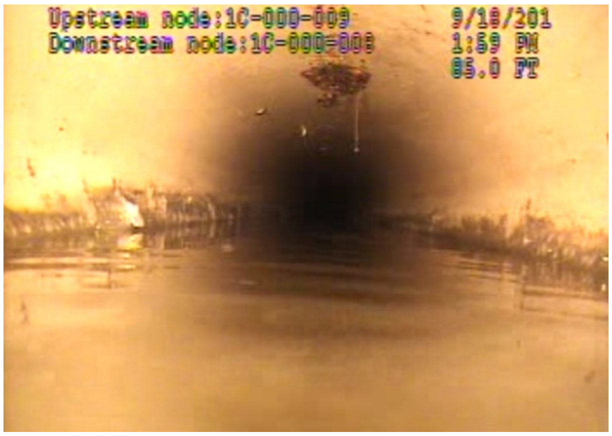
Summary and Findings

The pipe segment surveyed for Location 3 began 8 feet from upstream manhole 1C-3382-009 and continued for 121.2 feet downstream, about 303.8 feet before reaching manhole 1B-3382-008. Video quality was clear and recorded a sag in the pipe at 40 feet from the starting point, sag span of 81.2 feet. A crack was observed 110.8 feet from the starting point (Figure 11-5). Based on the general assessment of the crack, the condition grade assigned to this segment is Grade 4. This classification suggests a possibility of collapse at this location in the near future.

Surface spalling was recorded at a top corner of the pipe segment located 85 feet from the upstream manhole (Figure 11-6). For the exception of the crack section discussed above, the overall condition of this pipe segment is Grade 3 due to unlikelihood of collapse in the near future.

Figure 11-5: Manhole Segment 1C-3382-009 to 1C-3382-008

Figure 11-6: Manhole Segment 1C-3382-009 to 1C-3382-008



11.2.4 Location 4: Wintun / Corwin

Mainline ID

Upstream: 1C-5770-056
Downstream: 1B-000-022

Summary and Findings

Location 4 was surveyed 406.3 feet, covering the pipe segment from upstream manhole 1C-5770-056 to downstream manhole 1B-000-022. The quality of the video recording was clear (Figure 11-7) and displayed three recordings of sag. The first sag, located 100.9 feet from starting point, spanned 27.9 feet. The second sag, located 177.8 feet downstream, spanned 24 feet. The last sag was located 312.9 feet from starting point, spanning a total of 66.4 feet. No other defects were found within the entire stretch of the pipe segment. A Grade 3 was assigned to this segment due to unlikelihood of collapse in the near future.

Figure 11-7: Manhole Segment 1C-5770-056 to 1B-000-022

11.2.5 Location *5:* Rancherias / Ottawa

Mainline ID

Upstream: 2A-000-205
Downstream: 2A-000-204

Summary and Findings

The evaluation of Location 5 included 348.5 feet of pipeline. The video quality was clear and resulted in the identification of three structural sags. The first sag was located 227.2 feet from 2A-000-205, spanning 4.7 feet. The second sag was located at 291.2 feet, spanning 6.7 feet. The third sag was found 297.9 feet from starting point, spanning 27.9 feet.

A deformation was found at one of the lateral connections, 296.3 feet from 2A-000-205 (Figure 11-8). This section was assigned a Grade 4 due to the possibility of collapse in the near future. Root intrusion was present at one of the lateral connection 218.7 feet from the starting point, as shown in Figure 11-9. Although this defect is minor, it could potentially pose a problem in the future. Segment was assigned a Grade 3 rating.

Figure 11-8: Manhole Segment 2A-000-205 to 2A-000-204

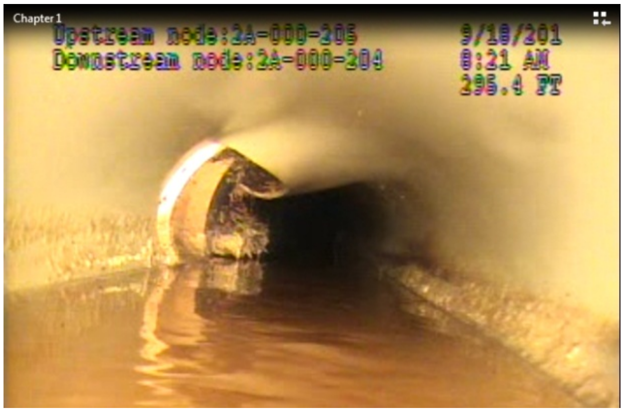
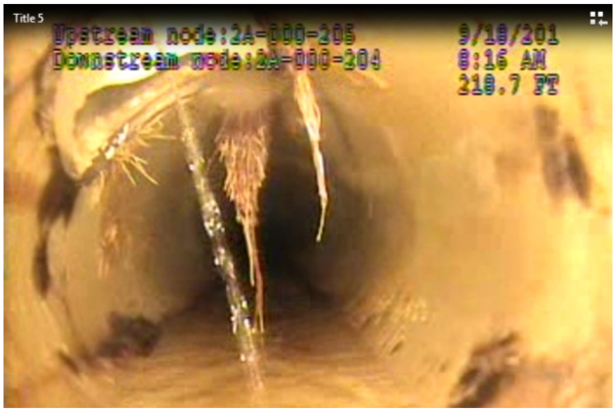


Figure 11-9: Manhole Segment 2A-000-205 to 2A-000-204



11.2.6 Location 6: Patomac / Quantico

Mainline ID

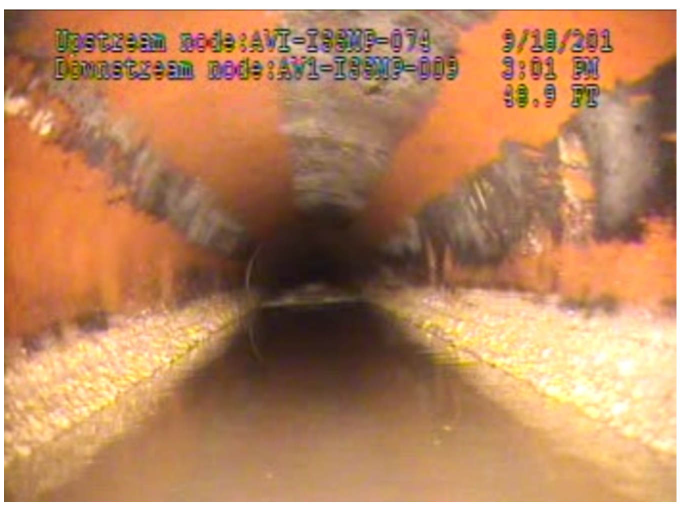
Upstream: TOAV-AVI-4494-001
Downstream: VVWRA-AVI-063

Summary and Findings

The video inspection from Location 6 began at upstream manhole TOAVI-4494-001 and continued 337.4 feet downstream, 40.6 feet before reaching manhole VVWRA-AVI-063. The quality of the video recording was clear. From a structural standpoint, two recordings of sag were noted. The first sag and second sag were located 8.1 feet and 42.9 feet from the starting point, respectively. The first sag spanned 9.7 feet and the second one spanned 287.4 feet.

There was surface spalling noticed at 35 feet which ended 53 feet from the starting point (Figure 11-10). This section was assigned a Grade 3 due to unlikelihood of collapse in the near future. However, the data provided was insufficient since the recording device submerged under water 55 feet from the upstream manhole to downstream at 337.4 feet. No further defects were identified.

Figure 11-10: Manhole Segment TOAV-AVI-4494-001 to VVWRA-AVI-063



11.2.7 Location 7: American Security Bank

Mainline ID

Upstream: JR-14310-359
Downstream: JR-MSP.1-007

Summary and Findings

The evaluation of Location 7 included a total of 239.5 feet of pipeline, beginning from the upstream manhole JR-14310-359 to the downstream manhole JR-MSP.1-007. The quality of the video recording was clear. There were two recordings of sag noted. The first sag, located 45.7 feet from starting point, spanned 20.7 feet, and the second sag, located at 99.9 feet, spanned 24.1 feet.

There were three joint defects located at 140.4, 160.5 and 200.8 feet, respectively. All three deformed joints are located at the top corner of the pipe as shown in Figure 11-11. Based on general assessments, all three joints are at Grade 3 due to the unlikelihood of collapse in the near future.

Figure 11-11: Manhole Segment JR-14310-359 to JR-MSP.1-007

11.2.8 Location 8: Jess Ranch Parkway / Jess Ranch Place

Mainline ID

Upstream: JR-14310-130
Downstream: JR-MSP.1-013

Summary and Findings

Location 8 was surveyed 346.2 feet from upstream manhole JR-14310-131 to downstream manhole JR-MSP.1-013. The quality of the video recording was clear. One recording of sag was observed, located 26.4 feet from starting point. The sag spanned 267 feet with the other end experiencing severe sagging.

Collection of debris was observed 33.6 feet from the upstream manhole (Figure 11-12). This section was assigned a Grade 3 due to the unlikelihood of collapse in the near future. The recording device submerged under water 53 feet from starting point and resurfaced at 242 feet. Due to this, a complete assessment of the segment was not possible.

Figure 11-12: Manhole Segment JR-14310-130 to JR-MSP.1-013

11.2.9 Location 9: American Security Bank / Jess Ranch

Mainline ID

Upstream: JR-14310-359
Downstream: JR-14310-359A

Summary and Findings

Location 9 was surveyed from upstream manhole JR-14310-359 to 84.9 feet downstream. An assessment of this location was not possible because the recording device submerged under water during the footage.

11.2.10 Location 10: Jess Ranch Parkway / Jess Ranch Place

Mainline ID

Upstream: JR-14310-131
Downstream: JR-14310-130

Summary and Findings

The CCTV inspection covered a total distance of 293.2 feet, beginning from upstream manhole JR-14310-131 to downstream manhole JR-14310-130. From a structural standpoint, only one recording of sag was noted. The sag was found 83.8 feet from the starting point, spanning 120.9 feet (Figure 11-13). There were no other defects found along the stretch of the segment. The segment was assigned a Grade 3 due to unlikelihood of collapse in the near future.

Figure 11-13: Manhole Segment JR-14310-131 to JR-14310-130

11.2.11 Summary

Table 11-3 summarizes the results from each location surveyed. Approximately 96 percent of the surveyed segments were classified as a Grade 3. Overall, the general assessment of the conditions of the pipes are in good condition. Location 3 was found to contain a crack at 110.8 feet from the upstream manhole IC-3382-009 and Location 5 was found to contain deformation at one of the lateral connections, 296.3 feet from the upstream manhole 2A-000-205. These two segments may need immediate consideration due to the severity of the conditions. Location 9 was not assessed since the video device was submerged under water during the duration of the recording.

Table 11-3: Summary of Estimated Condition of Sewer Pipes

Location	Grade 1	Grade 2	Grade 3	Grade 4	Grade 5	Unknown
1	-	-	342.2	-	-	-
2	-	-	156.5	-	-	-
3	-	-	110.8	10.4	-	-
4	-	-	406.3	-	-	-
5	-	-	343.4	5.1	-	-
6	-	-	337.4	-	-	-
7	-	-	239.5	-	-	-
8	-	-	346.2	-	-	-
9	-	-	-	-	-	84.9
10	-	-	293.2	-	-	-
Total (ft)	**-**	**-**	**2575.5**	**15.5**	**-**	**84.9**

Table 11-4 lists the Structural and Operation and Maintenance (O&M) findings for the ten locations surveyed.

Table 11-4: Summary of Pipeline Structural Defects and O&M Results

Location	Structural	O&M
1	Sags (2) and blisters	-
2	Sag	-
3	Sag , crack, and surface spalling	Debris
4	Sags (3)	-
5	Sags (3), deformation and root intrusion	Deformation
6	Sags (2) and surface spalling	-
7	Sags (2) and joint deformations (3)	-
8	Sag	Debris
9	Unknown	-
10	Sag	-

The condition assessment performed on the CCTV data provided by the Town staff revealed the following as summarized in Table 11-5.

Table 11-5: CCTV Pipe Segment Evaluation

No.	Location	Segment	Defect Summary
1	St. Mary's Hospital / 18 North	Upstream: 1B-000-003 Downstream: 1B-000-002	• Structure defects • Sags • Blisters within the pipe lining • Grade 3
2	St. Mary's Hospital / 18 North	Upstream: 1B-000-004 Downstream: 1B-000-003	• Sag • Grade 3
3	18 North / Tao Road	Upstream: 1C-3382-009 Downstream: 1C-3382-008	• Sag • Crack • Grade 4, due to possibility of collapse
4	Wintun / Corwin	Upstream: 1C-5770-056 Downstream: 1C-008-022	• Sag • Grade 3
5	Rancherias / Ottawa Road	Upstream: 2A-000-205 Downstream: 2A-000-204	• Sag • Root Intrusion • Grade 4, due to possibility of collapse
6	Potomac Road / Quantico Road	Upstream: TOAV-AVI-4494-001 Downstream: VVWRA-AVI-063	• Sag • Spalling • Grade 3
7	American Security Bank	Upstream: JR-14310-130 Downstream: JR-MSP1-007	• Sag • Joint defects • Grade 3
8	Jess Ranch Parkway / Jess Ranch Place	Upstream: JR-14310-359 Downstream: JR-MSP1-013	• Severe Sag • Collection of debris • Grade 3
9	American Security Bank / Jess Ranch	Upstream: JR-14310-359 Downstream: JR-14310-359A	• Video submerged therefore no assessment available
10	Jess Ranch Parkway / Jess Ranch Place	Upstream: JR-14310-131 Downstream: 14310-130	• Sag • Grade 3

11.3 Long-Term Routine Maintenance Program

In addition to the specific capital improvements projects described in this report, continued maintenance and rehabilitation are necessary for a wastewater collection system to ensure proper operation and to avoid pipeline failures. Routine maintenance, testing, and inspection, and related data management functions should be included in the Town's current maintenance program.

11.3.1 Routine Maintenance

A routine maintenance program provides for systematic cleaning and root control within the collection system. Performed on a continued basis, these activities should keep the system operating properly. Emergency repairs can be minimized by these types of activities as well as through implementation of routine testing and inspection. Many communities utilize a 2 to 5 year routine cleaning cycle, including root control performed as necessary. This means that every pipeline in the sewer system is cleaned once every 2 to 5 years. URS understands the Town is on a 5 year inspection cycle. A cleaning production rate of 2,000 to 3,000 feet per day can be achieved with a full-time crew. This rate assumes that no major problems are encountered.

Root control techniques consist of auguring the line, followed by a chemical herbicide treatment. Chemical treatment is considered a root inhibitor. This technique is usually performed every 2 to 3 years, or more frequently, depending on the area. Areas with a history of frequent problems should be scheduled for cleaning as often as appropriate. If roots continue to be a problem in a given location, pipe lining should be considered. Root intrusion can cause stoppages in the pipeline, decrease the capacity, and allow entry of I/I.

Routine maintenance activity should continue to include emergency repair of pipelines and access structures when severe defects are identified during maintenance activity. If the pipe or structure is found to be damaged but not in danger of failure in the near-term, then it could be referred to the Town's Public Works Department for inclusion in the capital improvement program.

Lift station maintenance should consist of routine inspection, repair/replacement of equipment items, and documentation of the maintenance activities performed at the lift station and to the equipment. Documentation of the maintenance activity will identify equipment items in need of future repair/replacement. These items can be scheduled for work in the future. When the maintenance cost to an item exceeds the capital cost, or when maintenance appears excessive, consideration should be given to replacing the equipment item.

11.3.2 Testing and Inspection

Testing and inspection of the sewer system on a routine basis should be used to identify areas of the system in deteriorated structural condition and any areas for potential I/I. It should be noted that I/I problems can cause significant problems to the collection system and accelerate deterioration of the system. The three major testing and inspection procedures are manhole inspection, CCTV inspection, and smoke testing. Priority should be given to the older portions of the system.

Manhole inspection is used to observe physical characteristics of the manhole and, as necessary, the inlet and outlet sewers. Information including dimensions, construction materials, structural conditions, and presence of infiltration is noted. This technique is useful in conjunction with the CCTV inspection as a complete evaluation of the collection system. Manhole inspections can be as general as noting the structural integrity as "Good," "Fair," and "Poor," or as specific as the detailed condition of each individual part (i.e., frame, riser, cone, barrel, shelf). The rehabilitation technique applied to the manhole depends on the defects identified; therefore, it is important to obtain as much detailed information as possible. Documentation should include sources of I/I, corrosion, and structural condition.

It is estimated that approximately 30-40 manholes can be inspected from the surface per day. If manhole entry is required, approximately 10-20 manholes can be inspected per day. This estimate is based on a full-time manhole inspection crew with 2-persons per crew per day. This estimate is based on a full-time manhole crew with 2-persons per crew for above ground inspection and 3-persons for manhole entry. This procedure should be applied to the entire collection system at least once every 10 years.

CCTV inspection is used to view the internal condition of the pipeline and to detect infiltration into the sewer through defects such as broken pipes and joints. It is important to clearly document every defect within the pipeline. The results of CCTV inspection are used to determine the applicable rehabilitation technique. Documentation should include lateral types and location, joint condition, structural problems, corrosion problems, infiltration, grease, roots, debris, etc. CCTV production rate of 1,500 to 2,000 feet per day can be achieved if conducted on a full-time basis (i.e., a complete day) with no major problems encountered. This includes cleaning of the sewer line in preparation for CCTV inspection.

CCTV inspection should also be required as part of the post-construction acceptance program for installation of new or rehabilitated pipelines. Prior to the expiration of the warranty period (usually 1 year), the pipeline should be televised to examine the internal condition and to note any signs of infiltration, even if air pressure testing was conducted at the time of construction. This record forms the basis of comparison in the future should defects occur in the system at a later date. For example, some minor offsets or pipe flaws in the new pipe may develop into defects at a later date. With the video record, the rate of deterioration may be determined. Any costs associated with this work should be required by the specifications to be borne by the Contractor.

Many communities perform smoke testing every 10 years to identify problems in the sewer lines that could contribute to I/I problems. Defects such as storm drain and area drain connections, broken pipes, connected downspouts, etc., can be located.

The results of the sewer system testing and inspection program would dictate the type and extent of repair or rehabilitation needed for manholes, trunk sewers, and laterals. An effective and successful maintenance program requires that accurate records of each activity be stored for use in analyzing costs and conditions of the existing collection system. This data can then be used as a planning tool to determine annual maintenance budgets, equipment requirements, and personnel needs as well as provide the data for subsequent capital improvement projects.

12.0 DEVELOPMENT OF CAPITAL IMPROVEMENT PROJECTS

Following the analysis of the results of the computer modeling, a Capital Improvement Project (CIP) was developed for the modification of the existing sewers and the addition of the new sewers.

URS defined the alternatives to be evaluated, which included one or more of the following elements:

- Parallel versus replacement sewer
- Rerouting of flows
- Gravity sewer service versus pumping station service for growth areas

Each defined alternative was evaluated as to the feasibility of its implementation. The alternatives were evaluated in conjunction with the sanitary sewer collection system expansion and included consideration for buildout development.

Our recommendations were based on both economic and non-economic factors. Planning level cost estimates were developed for each alternative and the benefits and drawback of each were defined.

A meeting with the Town staff was held to review present worth values developed to compare life-cycle costs for each alternative. Advantages and disadvantages of various alternatives and the basis for our recommendation were discussed with the Town staff.

Alternatives considered for implementation underwent further analysis using the hydraulic model. The impact and distribution of growth were considered during the evaluation. Based on our discussion with the Town staff and in consideration of potential growth scenarios, URS selected the alternative(s) that offered the most flexibility.

The selected alternative(s) were itemized into specific CIPs. The following section provides a description of the development of CIPs.

12.1 EXISTING COLLECTION SYSTEM EVALUATION

Proposed sewer system improvements from the 1993 Sewer Master Plan for the Town of Apple Valley included construction of three sub-regional wastewater treatment reclamation plants and future sewer systems based on land use patterns and topographic data from USGS contour maps. None of the three proposed subregional wastewater treatment plants has been built to date. Most of the sewer systems developed within the past 20 years occurred mainly due to developments concentrated in the southern portion of the Town of Apple Valley.

The computer hydraulic model simulations results identified a system which is adequate, with no observed deficiencies.

12.2 FUTURE COLLECTION SYSTEM EVALUATION

The future sewer system evaluation was based on the assumption that the Town will grow/develop as defined by the Town's 2009 General Plan and Specific Plans. A model was created to represent the buildout scenario. The projected sewer system at buildout was determined by identifying all residential, commercial, and industrial vacant/undeveloped lots and applying flow factors based on land use designation.

12.2.1 Future Proposed Sewered Areas

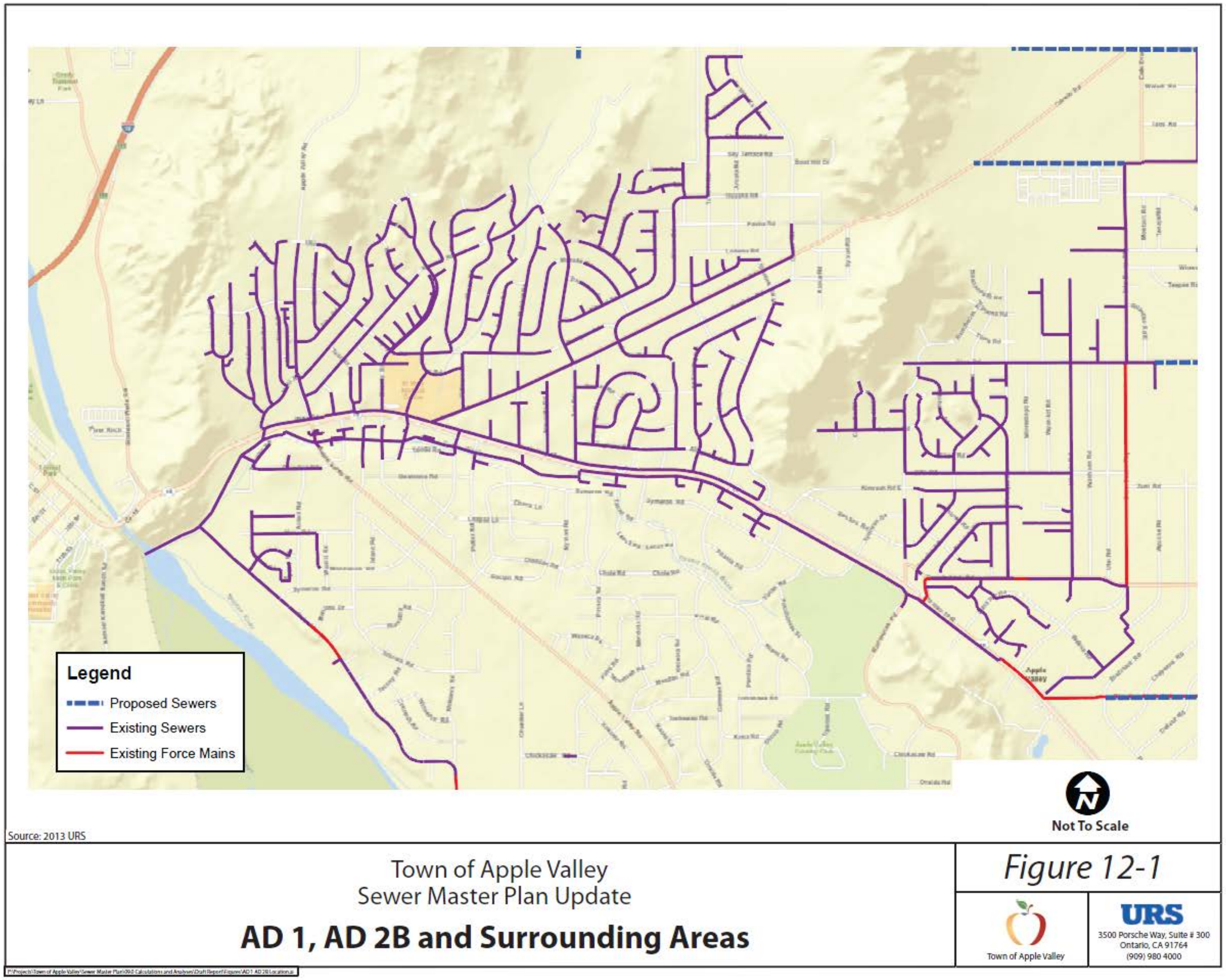
According to the Town of Apple Valley's adopted 2006 Sewer Connection Policy found in Appendix D, all new single-family lots created by subdivisions, located within one-half (½) mile away from existing sewer infrastructure and with total gross lot size of less than one acre are required to connect to the Town's sewer system. Based on the 2009 General Plan, all new developments will ultimately be connected to the public sewer system. Using these criteria, URS developed a list of proposed sewer system upgrades and future sewer collection system. The sewer lines were laid out using USGS contour maps to determine the elevations. The pipes were laid out to follow the natural slope of the area to eliminate the use of new lift stations. The proposed sewer system map is shown in Figure 12-5.The following subsections provide a brief description of system upgrades and additions for each part of Town.

12.2.1.1 Proposed System Improvements to AD 1, AD 2B, and Surrounding Areas

This service area includes AD 1, AD 2B, and surrounding areas, as shown in Figure 12-1. All residential and commercial areas within AD 1 are connected to the sewer infrastructure. Proposed improvements for this area consist of replacing an approximate total pipe length of 1,300 feet. The surrounding areas located south of Happy Trails Highway (Highway 18) are on septic system, and thereby are not required to connect to the sewer system. New sewers recommended for the area are listed in Appendix A.

The ground level in AD 2B slopes downward in the northeast direction. AD 2B's existing sewer pipelines direct all of the sewer flow north to LS 2B located east of Dale Evans Parkway and Otoe Road. Based on the 1993 Master Plan, the lift station and force main were designed to be adapted and used with a proposed sub-regional wastewater treatment reclamation facility. Per the 1993 Sewer Master Plan, the force main can be adjusted as a gravity sewer and the lift station as a low head lift station that can provide flows to the headworks of the treatment facility. The proposed improvements near AD 2B consist of replacing an approximate total pipe length of 19,000 feet and constructing an approximate total length of 110,000 feet of proposed pipeline. The lift station and force main require upgrade to convey the flows from LS 2B to the VVWRA interceptor. If the future subregional wastewater treatment plant does not go through, capital improvements projects in AD 2B can include a pipe along Wichita Road and redirecting the flows to travel south to connect to the VVWRA interceptor.

Figure 12-1: AD 1, AD 2B, and Surrounding Areas



12.2.1.2 Proposed System Improvements to AD 3A, Jess Ranch and Surrounding Areas

The service area includes AD 3A, Jess Ranch and surrounding areas. The surrounding area consists of low density residential homes, single family residential, and Deep Creek Estates Specific Plan. The majority of the surrounding area is undeveloped. Currently, this service area has four lift stations (JR No. 1, JR No. 2, 3A No. 1, and 3A No. 2) that pump the sewer flow up north to the VVWRA interceptor. Proposed improvements for this area consist of replacing an approximate total pipe length of 12,000 feet. Also, four future collector sewer pipes need to be constructed to connect existing and future development to the sewer infrastructure (Figure 12-2). The approximate total length of the proposed pipelines is 24,000 feet. Newly proposed sewer running along Tussing Ranch Road will connect to MH JR-16758.OFF-013. The flow from this area will travel north to JR No. 1. New proposed sewers along Grande Vista Street and Del Oro Road are to connect to JR-16840-002 and JR-16840-012, respectively, where they will travel by gravity to LS JR No. 1. Final proposed sewer line will run along Bear Valley Road, where it will head west to MH 3A-AVCBV-001. This flow will travel by gravity to LS 3A No. 2.

Lift Station Tr. 17093, which is currently non-existent, is required under future buildout. The lift station will require two pumps, each with a capacity 100 gpm.

Figure 12-2: Jess Ranch, AD 3A, and Surrounding Areas

Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update

Jess Ranch, AD 3 and Surrounding Areas

Figure 12-2

Town of Apple Valley

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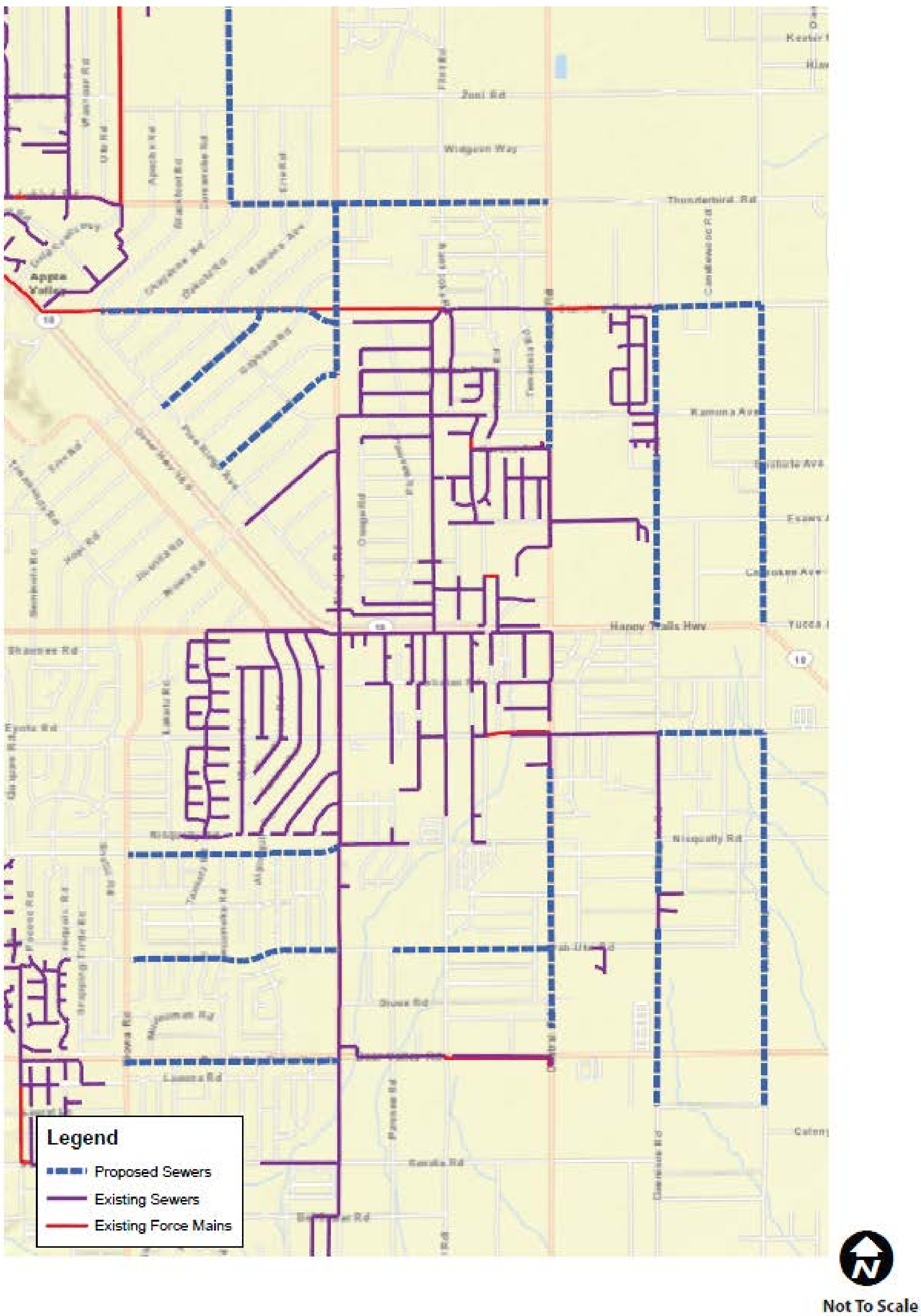
Ontario, CA 91764
(909) 980 4000

12.2.1.3 Proposed System Improvements to AD 2A and Surrounding Areas

There are five existing lift stations that direct the sewer flow to the VVWRA interceptor. The southern portion bounded by Sandia Road to the north, Kiowa Road to the west, Ocotillo Way and Poppy Road to the south, and Central Road to the east are not expected to be connected to the existing sewer system, and will remain on septic system. There is an approximate total length of 10,500 feet of existing sewer system pipeline that needs to be upgraded in order to handle the projected buildout flows. The proposed improvements for this portion of the Town consist of adding approximately 79,500 feet of proposed sewer pipeline to the existing sewer infrastructure (Figure 12-3). Proposed CIP improvements are listed in Appendix A and B.

Lift Station Tr. 17247 is proposed under buildout and consists of two pumps, each rated at 150 gpm. Under existing conditions, this lift station does not exist but will be required at future buildout.

Figure 12-3: AD 2A and Surrounding Areas



Source: 2013 URS

Town of Apple Valley
Sewer Master Plan Update

AD 2A and Surrounding Areas

Figure 12-3

Town of Apple Valley

URS
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Ontario, CA 91764
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12.2.1.4 **Proposed System Improvements to Northern Service Area**

The northern portion of Apple Valley, as shown in Figure 12-4, is mainly undeveloped. The existing sewer system consists of a small area located directly above AD 1B, sewer lines running along the west side of the airport, and the NAVI. At this time the NAVI only receives a very small portion of its total capacity flow from the Juvenile Detention Center. In order to distribute the flows in an evenly manner, the projected flows from the north and west portion of this area have been laid out to connect directly to the NAVI.

The southeast portion flows from the northern service area will connect to the existing sewer lines that run along the west side of the airport. These flows will travel south to the lift station located east of Otoe Road and Dale Evans Parkway. Flows will then be directed to the VVWRA Interceptor through the Lift Station 2B and its force main. The proposed improvements for this portion of the Town consist of constructing approximately 159,000 feet of proposed sewer pipeline.

Figure 12-4: Northern Apple Valley

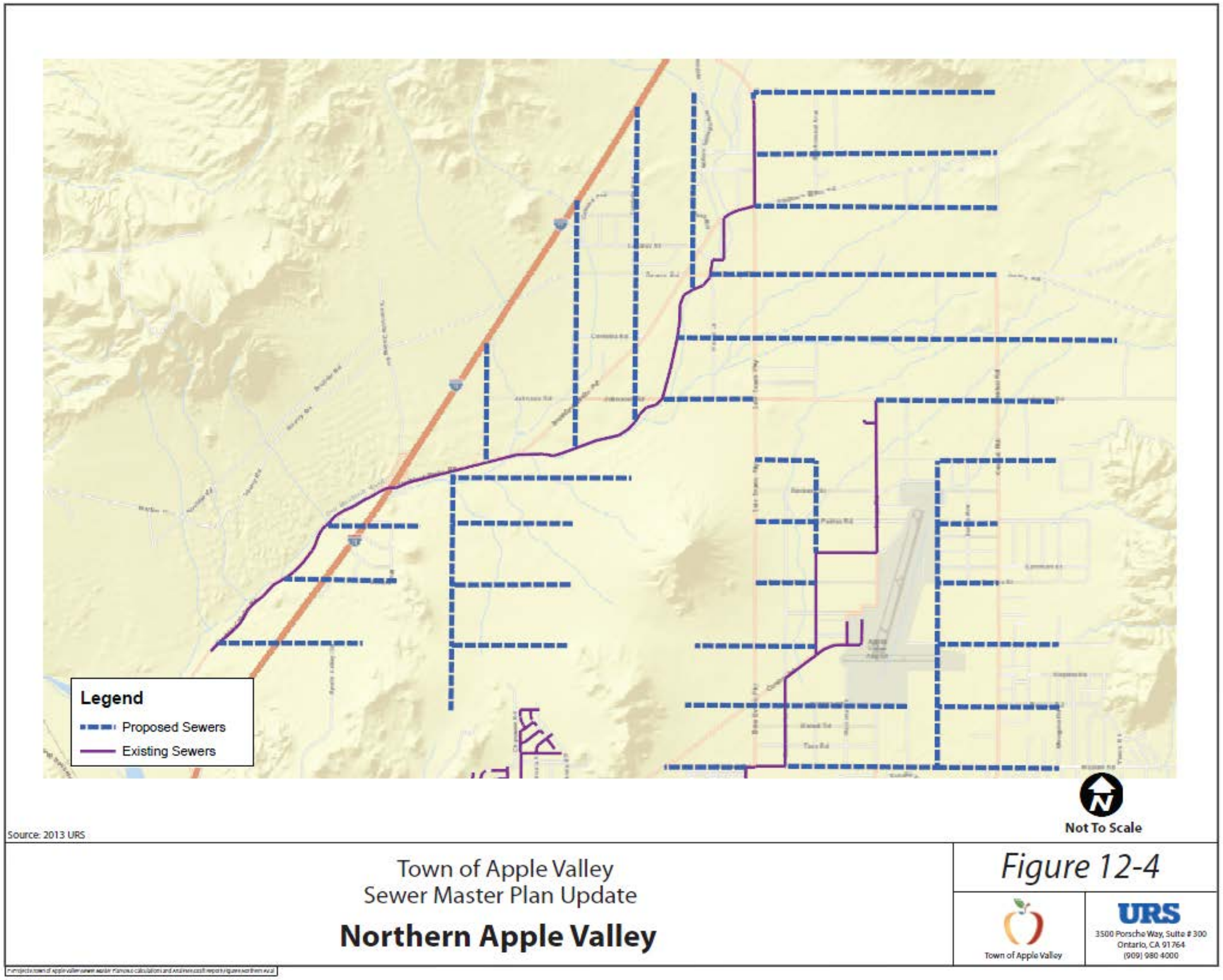
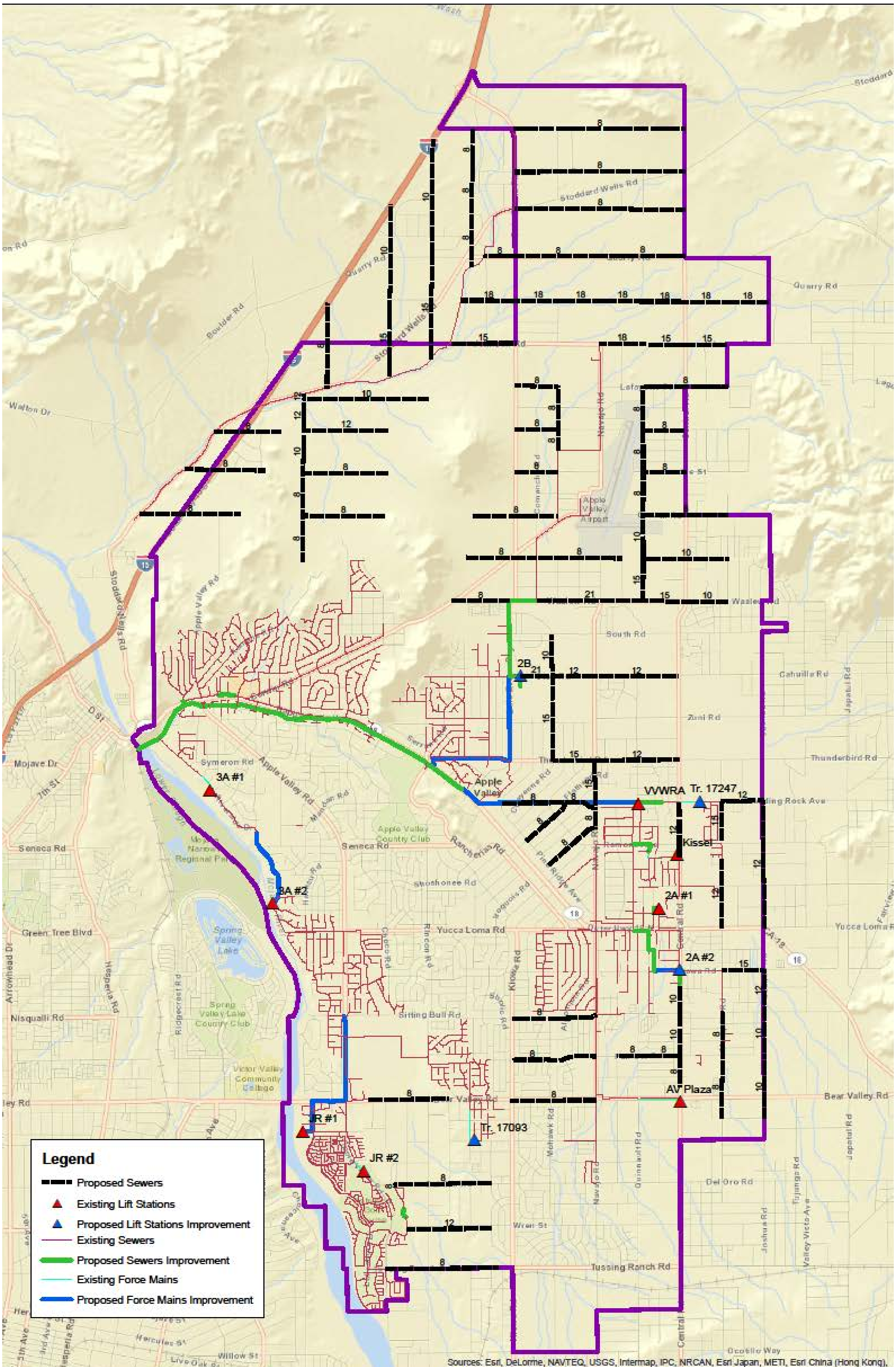


Figure 12-6: Proposed Sewer System Recommended CIPs



Town of Apple Valley Sewer Master Plan Update **Proposed Sewer System Recommended CIPs**	Figure 12-5
	Town of Apple Valley — URS, 3500 Porsche Way, Suite #300, Ontario, CA 91764, (909) 980-4000

12.3 Recommended Capital Improvement Projects

This section summarizes the capital improvement projects planned for the Town's sewer collection system based on the findings from the InfoWorks model. Improvements to existing sewers and proposed sewer lines were developed to relieve existing capacity deficiencies and to keep pace with the expansion of growth areas, respectively.

The cost estimates presented in this report are based on the unit construction costs listed in Table 12-1. All unit costs were assumed to include material and installation. Costs for engineering, legal, administration, construction management, and contingency are not of the unit construction cost.

Table 12-1: Unit Construction Cost

Description			
Gravity Pipeline Diameter (in)	**Unit Cost[6] ($/LF)**	**Force Main Pipeline Diameter (in)**	**Unit Cost[8] ($/LF)**
6	110	6	60
8	140	8	70
10	145	10	80
12	155	12	90
15	160	16	115
18	180	18	130
20	200	20	140
24	220	24	170
30	275	30	210
36	335		

[8] *Unit Construction Costs do not include contingency or other mark-ups.*

The proposed improvements that are needed to resolve existing sewer infrastructure deficiencies projected under the future buildout scenario and recommended CIPs that are needed to provide sewage service to areas projected to grow are shown on Figure 12-5 and included in Appendix A and B. Tables 12-2 and 12-3 show a summary of the projected CIP cost. Total cost includes 35 percent contingency allowance for total construction cost, engineering administration, legal and construction management costs. The costs are based on 2013 cost estimates.

The overall CIPs can be broken down into two categories. The first category deals with improvements to upgrade the existing sewer infrastructure. The estimated cost of these improvements is $11.6 million.

Table 12-2: Estimated Cost of Recommended CIPs - Town of Apple Valley Responsibility

Description	Construction Cost ($)	Contingencies ($)	Total ($)
Upgrading Existing Sewer System to Accommodate Buildout Flows	$5,855,800	$2,049,500	$7,905,300
Lift Stations	$1,297,800	$454,230	$1,752,030
Total			**$9,657,330**

Category 2 is considered to be the responsibility of the developers and it consists of construction of new pipeline segments to serve new growth areas at an estimated cost of $74.2 million.

Table 12-3: Estimated Cost of Recommended CIPs - Developer Responsibility

Description	Construction Cost ($)	Contingencies ($)	Total ($)
Proposed New Sewer System Pipeline to Serve Growth Areas	$54,968,246	$19,238,886	$74,207,132
Total			**$ 74,207,132**

VVWRA pipeline which is not the responsibility of the Town is also projected to require upgrades. The cost of upgrades has not been included in this Report.

13.0 CONCLUSIONS AND RECOMMENDATIONS

The calibrated hydraulic model was used to evaluate the hydraulic deficiencies of the existing and the future systems. The existing system model was updated with future land use information and buildout used to evaluate the system under future buildout conditions. The system appears to suffer from considerable hydraulic deficiency during the buildout conditions and warrants a need for a massive Capital Improvement Project to accommodate the anticipated land use classification outlined in the 2009 General Plan.

13.1 CONCLUSIONS

The existing sewer system appears to have adequate capacity to covey flows during dry weather conditions. Currently, only about 30 percent of the Town's development is connected to the sewer system. Sewer flows are projected to greatly increase during buildout conditions, thereby increasing sewer collection infrastructure requirements. The increase in flows is projected to cause tremendous stress on the existing pipe network, resulting in system hydraulic deficiencies. The hydraulic model results showed that the most critical deficiencies occur in the VVWRA interceptor. Under the future buildout scenario, surcharging is predicted in the majority of the interceptor sewers including the VVWRA interceptor and Assessment Districts, with the exception of the North Apple Valley interceptor. On the VVWRA interceptor, SSOs are predicted during future buildout conditions. Recommended CIPs were developed using the hydraulic model to simulate the replacement of pipelines needed to accommodate the future buildout flows. Although, the upgrades were simulated as replacement, URS believes that it would be best if parallel pipelines are constructed in order to continue to provide service when the construction is in progress. The parallel lines will provide a way to maintain sewer service while improvements are made. Proposed CIPs for future needs were also established to expand the sewer collection system to areas that are currently not covered, but are expected to require sewer service. Based on pipeline replacements and construction, the overall cost of CIPs presented in this Master Plan is approximately $83.9 million.

13.1.1 Existing System

The following statements summarize the findings of the hydraulic evaluation of the existing system:

- The existing system appears to have adequate capacity to convey flows under average dry weather conditions.
- The existing lift stations appear to have adequate pumping capacity.
- The system does not appear to have been heavily influenced by ingress of infiltration and inflow into the sewer system during rain events.

13.1.2 Future Buildout System

The future buildout system appears to have inadequate capacity to convey flows. The deficiencies identified are the result of the new development that is projected to connect to the existing sewer collection system. The 1993 Sewer Master Plan directed flows to three (3) proposed sub-regional plants to be located in the Southwest Apple Valley Area, near AD 2A, and in the Northwest Apple Valley Area. The Updated Master Plan prepared by URS has directed some flows away from the water reclamation plants given that these may not materialize. The direction of flows to the VVWRA sewer has resulted in overloading of the VVWRA interceptor during buildout. This result points to the fact that the VVWRA interceptor may require upgrading or one of the proposed Water Reclamation Plant is necessary to alleviate the overloading of the VVWRA interceptor. The proposed plant located near Dale Evans Parkway and Otoe Road is probably the most logical choice to be built to alleviate the hydraulic deficiencies identified in the VVWRA at future buildout. The analysis also shows that the Jess Ranch sewer becomes overloaded with the proposed developments to the south of Jess Ranch.

The following provides a summary of the deficiencies within each of the locations during future buildout conditions.

- ***VVWRA Interceptor—***The VVWRA pipeline is predicted to surcharge during future buildout conditions, and SSOs are predicted along this pipeline segment. This pipeline should be upsized to accommodate the future buildout flows. The proposed upgraded pipe sizes are listed in Appendix A. In lieu of the upgrading the VVWRA pipeline segment, the proposed water reclamation plant near Dale Evans Parkway and Otoe Road should be considered. Also, the VVWRA Nanticoke AD 2 Lift Station requires upgrading at future buildout. The VVWRA pipeline segment and its Nanticoke AD 2 Lift Station are operated and maintained by VVWRA. The responsibility to upgrade this pipeline segment and its lift station is that of VVWRA, not the Town of Apple Valley.
- ***Jess Ranch, AD 3A, and Surrounding Areas—***At buildout, Lift Station JR No. 1 has inadequate force main capacity resulting in backing up flows. This force main requires upgrading at buildout to alleviate the surcharging predicted in Jess Ranch Parkway.
- ***AD 2B and Surrounding Areas—***At buildout Lift Station 2B has inadequate capacity resulting in backing up of flows to its upstream pipe segments. This lift station will require upgrade at buildout.
- ***AD 2A and Surrounding Areas —***All three pipeline segments leading to the Lift Station 2A No. 2 are predicted to surcharge due to the lift station's inadequate capacity during future buildout. To alleviate the surcharging, Lift Station 2A should be upgraded at buildout.

13.2 RECOMMENDATIONS

Based on the analysis presented in this report, URS recommends the following:

- ***Existing Sewer System—***Based on the hydraulic evaluation performed, the existing sewer system has adequate hydraulic capacity to convey flows under existing conditions. However, the Town is advised to consider the following:

- *Inflow Control*— The Town has some manhole covers that are perforated. These covers allow inflow to enter the sewer system during heavy rainfall events when ponding occurs over these manholes. Although, inflow was not observed as a problem during the analysis, URS recommends the replacement of these manhole covers. The recommended repairs involve providing water resistant lids and frames or raising the frames. The lack of evidence of inflow into the system during the analysis is attributable to the lack of adequate rainfall data concurrent with the flow monitoring data to conclusively confirm that inflow and infiltration as not being a problem in the system.

- *CCTV Inspections*— The Town has been diligent in performing CCTV inspections of historic problem areas. However, consideration should be given to performing additional CCTV inspections on the remainder of the system to determine the condition of the older sewer systems. Also, it appears the Town has gathered considerable CCTV data over the years and the data should be analyzed to determine the condition of the pipes and develop a proactive pipeline renewal and replacement plan. The Town has inspected approximately 35 percent of the sewer system since September 2009.

- *Flow and Rainfall Monitoring*—The flow and rainfall monitoring used for the analysis was provided by VVWRA and was performed during the dry weather season. The typical approach to performing a sewer master plan is to monitor flows and rainfall during both dry and wet weather conditions to ascertain the system response to rainfall. URS performed a limited assessment of the system response to rainfall using rain gauges provided by agencies within the Town. However, a more comprehensive infiltration and inflow assessment is recommended to be performed by the Town using a flow and rainfall monitoring data performed during the wet-weather season.

- ***Future Buildout System—***The hydraulic evaluation predicts a massive deficiency in the Town's sewer network to accommodate flows during future buildout conditions. Based on the General Plan, all new developments are projected to connect to the sewage collection system. In order to carry out the additions of growth areas to the sewer system, URS has developed a list of recommended CIPs to provide services to areas that are expected to grow (Appendix A and B). Also to address the deficiencies identified in the existing interceptor sewers, URS modeled the pipeline infrastructure required. The total cost of the infrastructure required to accommodate future buildout flows including the upgrading of existing infrastructure and the provision of new pipe segments to serve new development is estimated at approximately $83.9 million. Planning and design criteria for the recommended capital improvements projects in this Master Plan were developed at a preliminary basis. URS recommends that the Town conduct site specific assessments and refine budget-level costs as needed as more refined data becomes available.

- ***Hydraulic Modeling Updates—***The hydraulic modeling was conducted to assess the sewer system requirements for existing and future buildout conditions. URS recommends that the Town maintain the existing sewer network database with up-to-date information (e.g., addition of new pipelines, manholes, force mains, lift stations, etc.). The General Plan and the Specific Plans will be updated periodically as new and refined data becomes available. URS recommends that the Town updates the Sewer Master Plan and the InfoWater hydraulic model at approximately five (5) year intervals or as a result of a major proposed development which will have significant impact on the system hydraulics. Also, should the proposed water reclamation plant near Dale Evans Parkway and Otoe Road be considered for construction, the hydraulic evaluation presented in this report will need to re-evaluated. The current assumption in this Master Plan Update is that the water reclamation plant will not be constructed and therefore flows were directed to the North Apple Valley interceptor and to the VVWRA interceptor respectively. Should the Town desire to construct the water reclamation plant, an immediate re-evaluation of the recommendations presented in this report regarding sewer infrastructure to accommodate future buildout should be performed.

14.0 REFERENCES

1. Apple Valley's 2009 General Plan, Terra Nova Planning & Research, Inc.

2. Bridle Path Estates at Apple Valley Specific Plan, October 2006.

3. Deep Creek Estates at Apple Valley Specific Plan, September 2006. Revised June 2008.

4. Jess Ranch Planned Unit Development Amendment No. Three, January 1998.

5. LCER Specific Plan, October 2010.

6. Meadowbrook Planned Unit Development, January 1991.

7. North Apple Valley Industrial Specific Plan, Adopted October 2006.

8. North Pointe Specific Plan, June 2007.

9. Southern California Association of Governments. Regional Transportation Plan 2012-2035 Sustainable Communities Strategy-Towards a Sustainable Future. Growth Forecast Appendix, April 2012.

10. Synopsis of Apple Valley Water District Sewer Master Plan, So & Associates, November 1992.

11. The Town of Apple Valley, Sewer Master Plan, So & Associates, August 1993.

12. Town of Apple Valley Sewer Connection Policy, January 2006.

APPENDIX A
RECOMMENDED CIPS FOR EXISTING SEWER AREAS

14-1: Recommended CIPs for Existing Sewer Pipe Segments

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
1	Wika Road	Pipe	1A-DK1-DK1	1A-7802-001	73.4	8	12	155	$11,377
2	N/A	Pipe	1A-DK1-DK1	VVWRA-AVI-061	172.8	12	18	180	$31,104
3	Outer Hwy 18 N	Pipe	1B-000-001	1B-000-120	160	8	10	145	$23,200
4	Outer Hwy 18 N	Pipe	1B-000-002	1B-000-001	331	8	10	145	$47,995
5	Outer Hwy 18 N	Pipe	1B-000-003	1B-000-002	301	8	10	145	$43,645
6	Outer Hwy 18 N	Pipe	1B-000-004	1B-000-003	100	8	10	145	$14,500
7	Outer Hwy 18 N	Pipe	1B-000-120	1A-DK1-010	160	8	10	145	$23,200
8	Nanticoke Road	Pipe	2A-17247-043	2A-17247-043X	13	18	21	200	$2,600
9	Nanticoke Road	Force Main	2A-17247-043Y	AVFM-001	59.8	10	16	115	$6,877
10	Nanticoke Road	Pipe	2A-17247-044	2A-17247-043	26.5	18	21	200	$5,300
11	Standing Rock Ave	Pipe	2A-17247-045	2A-000-001	237.8	15	18	180	$42,804
12	Standing Rock Ave	Pipe	2A-17247-046	2A-17247-045	396	15	18	180	$71,280
13	Standing Rock Ave	Pipe	2A-17247-047	2A-17247-046	396	15	18	180	$71,280
14	Standing Rock Ave	Pipe	2A-17247-048	2A-17247-047	416	15	18	180	$74,880
15	Ramona Ave	Pipe	2A-000-061	2A-000-036	379.6	8	10	145	$55,042
16	Ramona Ave	Pipe	2A-000-062	2A-000-061	374.6	8	10	145	$54,317
17	Pioneer Road	Pipe	2A-000-064	2A-000-345	240.6	6	10	145	$34,887
18	Pioneer Road	Pipe	2A-000-065	2A-000-064	234.6	6	10	145	$34,017
19	Valley Dr	Pipe	2A-000-098	2A-000-097	321	6	8	140	$44,940
20	Quinnault Rd	Pipe	2A-000-120	2A-000-119	273.4	10	15	160	$43,744
21	Highway 18	Pipe	2A-000-142	2A-000-120	325.5	10	15	160	$52,080
22	Highway 18	Pipe	2A-000-143	2A-000-142	325.3	10	15	160	$52,048
23	Nomwaket Ln	Pipe	2A-000-144	2A-000-143	234.9	10	15	160	$37,584
24	Nomwaket Ln	Pipe	2A-000-145	2A-000-144	379.1	10	15	160	$60,656
25	Nomwaket Ln	Pipe	2A-000-146	2A-000-145	307	10	15	160	$49,120
26	Nomwaket Ln	Pipe	2A-000-147	2A-000-146	307	10	15	160	$49,120
27	Powhatan Rd	Pipe	2A-000-148	2A-000-147	326	10	15	160	$52,160
28	N/A	Pipe	2A-000-149	2A-000-148	326	10	15	160	$52,160
29	N/A	Pipe	2A-000-150	2A-000-149	326	10	15	160	$52,160
30	N/A	Pipe	2A-000-151	2A-000-150	326	10	15	160	$52,160
31	N/A	Pipe	2A-000-152	2A-000-151	326	10	15	160	$52,160

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
32	Valley Dr	Force Main	2A-000-287Y	2A-000-098	328	6	8	70	$22,960
33	Central Rd	Force Main	2A-000-305Y	2A-000-152	1647.5	4	12	90	$148,275
34	Central Rd	Pipe	2A-000-309	2A-000-305	275	6	10	145	$39,875
35	Central Rd	Pipe	2A-000-310	2A-000-309	275	6	10	145	$39,875
36	Central Rd	Pipe	2A-000-311	2A-000-310	274	6	10	145	$39,730
37	Central Rd	Pipe	2A-000-312	2A-000-311	176	6	10	145	$25,520
38	Ramona Ave	Pipe	2A-000-345	2A-000-062	373.6	8	10	145	$54,172
39	Hurons Ave	Pipe	2A-13934-031	2A-13934-031X	153.6	10	15	160	$24,576
40	Dale Evans Pwy	Pipe	2B-ARPE-001	2B-000-211	397	15	21	200	$79,400
41	Dale Evans Pwy	Pipe	2B-ARPE-002	2B-ARPE-001	392.4	15	21	200	$78,480
42	Dale Evans Pwy	Pipe	2B-ARPE-003	2B-ARPE-002	374.6	15	21	200	$74,920
43	Dale Evans Pwy	Pipe	2B-ARPE-004	2B-ARPE-003	353.9	15	21	200	$70,780
44	Waalew Rd	Pipe	2B-ARPE-005	2B-ARPE-004	400.5	15	21	200	$80,100
45	Waalew Rd	Pipe	2B-ARPE-005B	2B-ARPE-005	59.4	15	21	200	$11,880
46	Waalew Rd	Pipe	2B-ARPE-006	2B-ARPE-005B	322.8	15	21	200	$64,560
47	Waalew Rd	Pipe	2B-ARPE-007	2B-ARPE-006	281	15	21	200	$56,200
48	Waalew Rd	Pipe	2B-ARPE-008	2B-ARPE-007	310.1	15	21	200	$62,020
49	Waalew Rd	Pipe	2B-ARPE-009	2B-ARPE-008	351.6	15	21	200	$70,320
50	Otoe Rd	Pipe	2B-AO-001	2B-000-232	296	8	12	155	$45,880
51	Otoe Rd	Pipe	2B-AO-002	2B-AO-001	331	8	10	145	$47,995
52	Otoe Rd	Pipe	2B-000-001	2B-000-234	215.1	18	21	200	$43,020
53	Otoe Rd	Pipe	2B-000-233	2B-000-232	249.1	18	30	275	$68,503
54	N/A	Force Main	2B-000-232Y	VVWRA-AVI-008	11424	12	18	130	$1,485,120
55	Otoe Rd	Pipe	2B-000-232	2B-000-232X	12.7	18	30	275	$3,493
56	Dale Evans Pwy	Pipe	2B-000-204	2B-000-001	457.9	10	21	200	$91,580
57	Dale Evans Pwy	Pipe	2B-000-205	2B-000-204	430	10	21	200	$86,000
58	Dale Evans Pwy	Pipe	2B-000-206	2B-000-205	433.2	10	21	200	$86,640
59	Dale Evans Pwy	Pipe	2B-000-207	2B-000-207A	238.2	10	21	200	$47,640
60	Dale Evans Pwy	Pipe	2B-000-207A	2B-000-206	237.8	10	21	200	$47,560
61	Dale Evans Pwy	Pipe	2B-000-208	2B-000-207	433.2	10	21	200	$86,640
62	Dale Evans Pwy	Pipe	2B-000-209	2B-000-208	413	10	21	200	$82,600
63	Dale Evans Pwy	Pipe	2B-000-210	2B-000-209	282.8	10	21	200	$56,560
64	Dale Evans Pwy	Pipe	2B-000-211	2B-000-210	232	10	21	200	$46,400
65	Riverside Dr	Force Main	3A-8476.8-027Y	3A-WAPTS-023	4499.3	12	16	115	$517,420

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
66	Dandelion Ln	Pipe	JR-16840-005	JR-16840-006	292.6	8	10	145	$42,427
67	Dandelion Ln	Pipe	JR-16840-006	JR-16840-007	335.8	8	10	145	$48,691
68	N/A	Force Main	JR-MSP.1-002Y	3A-000-014	6440	8	12	90	$579,600
69	Highway 18	Pipe	VVWRA-AVI-001	VVWRA-AVI-002	150.3	12	30	NA	NA
70	Highway 18	Pipe	VVWRA-AVI-002	VVWRA-AVI-003	434.6	15	30	NA	NA
71	Highway 18	Pipe	VVWRA-AVI-003	VVWRA-AVI-004	198.1	15	30	NA	NA
72	Highway 18	Pipe	VVWRA-AVI-004	VVWRA-AVI-005	399.8	15	30	NA	NA
73	Highway 18	Pipe	VVWRA-AVI-005	VVWRA-AVI-006	400	15	30	NA	NA
74	Highway 18	Pipe	VVWRA-AVI-006	VVWRA-AVI-007	401.8	15	30	NA	NA
75	Highway 18	Pipe	VVWRA-AVI-007	VVWRA-AVI-008	400.2	15	30	NA	NA
76	Highway 18	Pipe	VVWRA-AVI-008	VVWRA-AVI-009	209.1	15	30	NA	NA
77	Highway 18	Pipe	VVWRA-AVI-009	VVWRA-AVI-010	44.1	15	30	NA	NA
78	Highway 18	Pipe	VVWRA-AVI-010	VVWRA-AVI-011	294.3	15	30	NA	NA
79	Highway 18	Pipe	VVWRA-AVI-011	VVWRA-AVI-012	304.6	15	30	NA	NA
80	Highway 18	Pipe	VVWRA-AVI-012	VVWRA-AVI-013	360.2	15	30	NA	NA
81	Highway 18	Pipe	VVWRA-AVI-013	VVWRA-AVI-014	357.9	15	30	NA	NA
82	Highway 18	Pipe	VVWRA-AVI-014	VVWRA-AVI-015	360.2	15	30	NA	NA
83	Highway 18	Pipe	VVWRA-AVI-015	VVWRA-AVI-016	439.7	15	30	NA	NA
84	Highway 18	Pipe	VVWRA-AVI-016	VVWRA-AVI-017	318.2	15	30	NA	NA
85	Highway 18	Pipe	VVWRA-AVI-017	VVWRA-AVI-018	401.3	15	30	NA	NA
86	Highway 18	Pipe	VVWRA-AVI-018	VVWRA-AVI-019	400	15	30	NA	NA
87	Highway 18	Pipe	VVWRA-AVI-019	VVWRA-AVI-020	399.5	15	30	NA	NA
88	Highway 18	Pipe	VVWRA-AVI-020	VVWRA-AVI-021	294.5	15	30	NA	NA
89	Highway 18	Pipe	VVWRA-AVI-021	VVWRA-AVI-022	274.4	15	30	NA	NA
90	Highway 18	Pipe	VVWRA-AVI-022	VVWRA-AVI-023	395.9	15	30	NA	NA
91	Highway 18	Pipe	VVWRA-AVI-023	VVWRA-AVI-024	404.4	15	30	NA	NA
92	Highway 18	Pipe	VVWRA-AVI-	VVWRA-AVI-	336.7	15	30	NA	NA

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
			024	025					
93	Highway 18	Pipe	VVWRA-AVI-025	VVWRA-AVI-026	396.2	12	30	NA	NA
94	Highway 18	Pipe	VVWRA-AVI-026	VVWRA-AVI-027	400.1	12	30	NA	NA
95	Highway 18	Pipe	VVWRA-AVI-027	VVWRA-AVI-028	40	12	30	NA	NA
96	Highway 18	Pipe	VVWRA-AVI-028	VVWRA-AVI-029	319	12	30	NA	NA
97	Highway 18	Pipe	VVWRA-AVI-029	VVWRA-AVI-030	401.2	12	30	NA	NA
98	Highway 18	Pipe	VVWRA-AVI-030	VVWRA-AVI-031	250.2	12	30	NA	NA
99	Highway 18	Pipe	VVWRA-AVI-031	VVWRA-AVI-032	444.2	12	30	NA	NA
100	Highway 18	Pipe	VVWRA-AVI-032	VVWRA-AVI-033	450.9	12	30	NA	NA
101	Highway 18	Pipe	VVWRA-AVI-033	VVWRA-AVI-034	439.9	15	30	NA	NA
102	Highway 18	Pipe	VVWRA-AVI-034	VVWRA-AVI-035	154.7	15	30	NA	NA
103	Highway 18	Pipe	VVWRA-AVI-035	VVWRA-AVI-036	215	12	30	NA	NA
104	Highway 18	Pipe	VVWRA-AVI-036	VVWRA-AVI-037	300	12	30	NA	NA
105	Highway 18	Pipe	VVWRA-AVI-037	VVWRA-AVI-038	448.6	12	30	NA	NA
106	Highway 18	Pipe	VVWRA-AVI-038	VVWRA-AVI-039	408.2	12	30	NA	NA
107	Highway 18	Pipe	VVWRA-AVI-039	VVWRA-AVI-040	417	12	30	NA	NA
108	Highway 18	Pipe	VVWRA-AVI-040	VVWRA-AVI-041	387	12	30	NA	NA
109	Highway 18	Pipe	VVWRA-AVI-041	VVWRA-AVI-042	400.7	12	30	NA	NA
110	Highway 18	Pipe	VVWRA-AVI-042	VVWRA-AVI-043	194.7	12	30	NA	NA
111	Highway 18	Pipe	VVWRA-AVI-043	VVWRA-AVI-044	208.4	12	30	NA	NA
112	Highway 18	Pipe	VVWRA-AVI-044	VVWRA-AVI-045	205	12	30	NA	NA
113	Highway 18	Pipe	VVWRA-AVI-045	VVWRA-AVI-046	373.2	12	30	NA	NA
114	Highway 18	Pipe	VVWRA-AVI-046	VVWRA-AVI-047	382	12	30	NA	NA
115	Highway 18	Pipe	VVWRA-AVI-047	VVWRA-AVI-048	294	12	30	NA	NA
116	Highway 18	Pipe	VVWRA-AVI-048	VVWRA-AVI-049	254.7	12	30	NA	NA
117	Highway 18	Pipe	VVWRA-AVI-049	VVWRA-AVI-050	257.9	12	30	NA	NA

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
118	Highway 18	Pipe	VVWRA-AVI-050	VVWRA-AVI-051	445.5	12	30	NA	NA
119	Highway 18	Pipe	VVWRA-AVI-051	VVWRA-AVI-052	455.3	12	30	NA	NA
120	Highway 18	Pipe	VVWRA-AVI-052	VVWRA-AVI-053	342.8	15	30	NA	NA
121	Highway 18	Pipe	VVWRA-AVI-053	VVWRA-AVI-054	326.2	12	30	NA	NA
122	Highway 18	Pipe	VVWRA-AVI-054	VVWRA-AVI-055	181.8	15	30	NA	NA
123	Highway 18	Pipe	VVWRA-AVI-055	VVWRA-AVI-056	149.9	15	30	NA	NA
124	Highway 18	Pipe	VVWRA-AVI-056	VVWRA-AVI-057	420.3	12	30	NA	NA
125	Highway 18	Pipe	VVWRA-AVI-057	VVWRA-AVI-058	90.8	15	30	NA	NA
126	Highway 18	Pipe	VVWRA-AVI-058	VVWRA-AVI-059	117	15	30	NA	NA
127	Highway 18	Pipe	VVWRA-AVI-059	VVWRA-AVI-060	146	15	30	NA	NA
128	Highway 18	Pipe	VVWRA-AVI-060	VVWRA-AVI-061	155.9	12	30	NA	NA
129	Highway 18	Pipe	VVWRA-AVI-061	VVWRA-AVI-062	259.9	15	30	NA	NA
130	Highway 18	Pipe	VVWRA-AVI-062	VVWRA-AVI-063	358.1	15	30	NA	NA
131	Highway 18	Pipe	VVWRA-AVI-063	VVWRA-AVI-064	338	15	30	NA	NA
132	Highway 18	Pipe	VVWRA-AVI-064	VVWRA-AVI-065	335	12	30	NA	NA
133	Highway 18	Pipe	VVWRA-AVI-065	VVWRA-AVI-066	466.3	12	30	NA	NA
134	Highway 18	Pipe	VVWRA-AVI-066	VVWRA-AVI-067	400	12	30	NA	NA
135	Highway 18	Pipe	VVWRA-AVI-067	VVWRA-AVI-068	423	12	30	NA	NA
136	Highway 18	Pipe	VVWRA-AVI-068	VVWRA-AVI-069	163.6	12	30	NA	NA
137	Highway 18	Pipe	VVWRA-AVI-069	VVWRA-AVI-070	281.9	12	30	NA	NA
138	Highway 18	Pipe	VVWRA-AVI-070	VVWRA-AVI-071	513.1	12	30	NA	NA
139	Highway 18	Pipe	VVWRA-AVI-071	VVWRA-AVI-072	500	12	30	NA	NA
140	Nanticoke Road	Force Main	2A-17247-043Y	AVFM-001	59.8	10	18	NA	NA
141	Elkalo Rd	Force Main	AVFM-001	AVFM-002	35	10	18	NA	NA
142	Elkalo Rd	Force Main	AVFM-002	AVFM-003	465	10	18	NA	NA
143	Elkalo Rd	Force Main	AVFM-003	AVFM-004	500	10	18	NA	NA

No.	Location	Project Category	Upstream Node ID	Downstream Node ID	Length (ft)	Existing Diameter (in)	Parallel Pipe (in)	Unit Cost ($/LF)	Cost ($)
144	Elkalo Rd	Force Main	AVFM-004	AVFM-005	500	10	18	NA	NA
145	Elkalo Rd	Force Main	AVFM-005	AVFM-006	550	10	18	NA	NA
146	Elkalo Rd	Force Main	AVFM-006	AVFM-007	633	10	18	NA	NA
147	N/A	Force Main	AVFM-007	AVFM-008	517	10	18	NA	NA
148	N/A	Force Main	AVFM-008	AVFM-009	500	10	18	NA	NA
149	Standing Rock Ave	Force Main	AVFM-009	AVFM-010	450	10	18	NA	NA
150	Standing Rock Ave	Force Main	AVFM-010	AVFM-011	444	10	18	NA	NA
151	Standing Rock Ave	Force Main	AVFM-011	AVFM-012	456	10	18	NA	NA
152	Standing Rock Ave	Force Main	AVFM-012	AVFM-013	444	10	18	NA	NA
153	Standing Rock Ave	Force Main	AVFM-013	AVFM-014	531	10	18	NA	NA
154	Standing Rock Ave	Force Main	AVFM-014	AVFM-015	547.3	10	18	NA	NA
155	Standing Rock Ave	Force Main	AVFM-015	AVFM-016	522.7	10	18	NA	NA
156	Standing Rock Ave	Force Main	AVFM-016	AVFM-017	422.8	10	18	NA	NA
157	Standing Rock Ave	Force Main	AVFM-017	AVFM-018	532.2	10	18	NA	NA
158	Standing Rock Ave	Force Main	AVFM-018	AVFM-019	509	10	18	NA	NA
159	Standing Rock Ave	Force Main	AVFM-019	AVFM-020	491	10	18	NA	NA
160	Standing Rock Ave	Force Main	AVFM-020	AVFM-021	500	10	18	NA	NA
161	Standing Rock Ave	Force Main	AVFM-021	AVFM-022	301	10	18	NA	NA
162	Highway 18	Force Main	AVFM-022	AVFM-023	502	10	18	NA	NA
163	Highway 18	Force Main	AVFM-023	AVFM-024	500	10	18	NA	NA
164	Highway 18	Force Main	AVFM-024	VVWRA-AVI-001	405.7	10	18	NA	NA
							Construction Cost		$5,855,808
							Contingencies (35%)		$2,049,533
							Total Cost		$7,905,340

NA=Not Applicable. It is the responsibility of the Victor Valley Wastewater Reclamation Authority to provide upgrades required for the VVWRA interceptor.

APPENDIX B
RECOMMENDED CIPS FOR FUTURE SEWER AREAS

14-2: Recommended CIPs for Future Sewer Areas

No.	Area	Project Category	Upstream MH	Downstream MH	Pipe Diameter (in)	Pipe Length (ft)	Unit Cost ($/LF)	Total Cost ($)	Location	Description
1	2A	Pipe	B-OUT-051	B-OUT-076	15	2,941	160	$470,496	Navajo Road	From Standing Rock Ave to Thunderbird Rd
2	2A	Pipe	B-OUT-052	B-OUT-051	8	1,298	140	$181,664	Navajo Road	From Iroquois Road to Standing Rock Avenue
3	2A	Pipe	B-OUT-053	B-OUT-052	8	3,741	140	$523,783	Iroquois Rd	From Pine Ridge Ave to Navajo Road
4	2A	Pipe	B-OUT-054	B-OUT-051	8	1,988	140	$278,271	Standing Rock Ave	From Flathead Rd to Navajo Rd
5	2A	Pipe	B-OUT-055	B-OUT-054	8	3,950	140	$553,037	Standing Rock Ave	From Pine Ridge Avenue to Flathead Rd
6	2A	Pipe	B-OUT-056	B-OUT-054	8	3,490	140	$488,621	Flathead Rd	From Pine Ridge Avenue to Standing Rock Ave (AVFM-011).
7	2A	Pipe	B-OUT-057	2A-17247-041	12	2,638	155	$408,911	Standing Rock Ave	From Joshua Road to Mesquite Road (2A-17247-041)
8	2A	Pipe	B-OUT-058	B-OUT-057	12	7,957	155	$1,233,390	Joshua Rd	From Yucca Loma Rd to Standing Rock Ave
9	2A	Pipe	B-OUT-059	2A-16810-006	12	4,289	155	$664,732	Ocotilla Rd	From Happy Trails Hwy to MH 2A-16810-006
10	2A	Pipe	B-OUT-060	2A-EES-011	15	2,623	160	$419,754	Ottawa Rd	From Joshua Road to Mesquite Road (MH 2A-EES-011)
11	2A	Pipe	B-OUT-061	B-OUT-091	10	3,886	145	$563,431	Joshua Rd	From Colony Rd to Pah-Ute Rd
12	2A	Pipe	B-OUT-062	B-OUT-090	8	3,890	140	$544,548	Dennison Rd	From Colony Rd to Pah-Ute Rd
13	2A	Pipe	B-OUT-063	B-OUT-096	10	2,634	145	$381,951	Pah-Ute Rd	From Central Rd to Quinnault Road
14	2A	Pipe	B-OUT-064	B-OUT-097	8	1,892	140	$264,848	Pah-Ute Rd	From Pawnee Road to Quinnault Road
15	2A	Pipe	B-OUT-065	B-OUT-063	8	1,972	140	$276,037	Central Rd	From Sioux Rd to Pah-Ute Rd
16	2A	Pipe	B-OUT-066	2A-NRTS-021	8	5,273	140	$738,213	Sitting Bull Rd	From Kiowa Road to Navajo Road
17	2A	Pipe	B-OUT-067	2A-NRTS-015	8	5,250	140	$734,995	Pah-Ute Rd	From Kiowa Road to Navajo Road (2A-NRTS-015)
18	2A	Pipe	B-OUT-068	2A-NRTS-007	8	5,330	140	$746,213	Bear Valley Rd	From Kiowall Rd to Navajo Rd (2A-NRTS-007)
19	2A	Pipe	B-OUT-090	2A-EES-019	8	2,757	140	$385,931	Mesquite Road	From Colony Rd to Pah-Ute Rd

No.	Area	Project Category	Upstream MH	Downstream MH	Pipe Diameter (in)	Pipe Length (ft)	Unit Cost ($/LF)	Total Cost ($)	Location	Description
20	2A	Pipe	B-OUT-091	B-OUT-092	10	2,801	145	$406,093	Joshua Rd	From Pah-Ute Rd to Nisqually Rd
21	2A	Pipe	B-OUT-092	B-OUT-060	12	2,556	155	$396,241	Joshua Rd	From Nisqually Rd to Ottawa Rd
22	2A	Pipe	B-OUT-093	2A-17247-041	15	2,368	160	$378,853	Central Rd	From Ramon Avenue to Standing Rock Avenue (2A-17247-051).
23	2A	Pipe	B-OUT-096	2A-000-312	10	1,833	145	$265,791	Central Rd	From Nisquality Rd to Lucilla Rd (2A-000-312)
24	2A	Pipe	B-OUT-097	B-OUT-063	8	2,052	140	$287,251	Pah-Ute Rd	From Quinnault Road to Central Road
25	2B	Pipe	B-OUT-012	B-OUT-095	15	2,482	160	$397,165	Johnson Rd	N/A
26	2B	Pipe	B-OUT-013	2B-NRS-014	8	1,345	140	$188,324	Dakota Rd	From Los Padres Road to Lafayette St.
27	2B	Pipe	B-OUT-014	B-OUT-013	8	2,670	140	$373,758	N/A	N/A
28	2B	Pipe	B-OUT-015	B-OUT-014	8	2,689	140	$376,508	N/A	N/A
29	2B	Pipe	B-OUT-016	B-OUT-013	8	2,703	140	$378,431	N/A	N/A
30	2B	Pipe	B-OUT-017	2B-NRS-010	8	2,697	140	$377,637	N/A	N/A
31	2B	Pipe	B-OUT-018	2B-NRS-002	8	5,326	140	$745,668	N/A	N/A
32	2B	Pipe	B-OUT-019	2B-ARPE-018	8	4,408	140	$617,184	N/A	N/A
33	2B	Pipe	B-OUT-020	2B-ARPE-018	8	5,351	140	$749,141	N/A	N/A
34	2B	Pipe	B-OUT-021	2B-ARPE-004	8	3,579	140	$501,113	N/A	N/A
35	2B	Pipe	B-OUT-022	B-OUT-035	10	2,531	145	$366,947	N/A	N/A
36	2B	Pipe	B-OUT-023	2B-ARPE-009	21	6,681	200	$1,336,247	N/A	N/A
37	2B	Pipe	B-OUT-024	B-OUT-023	15	2,645	160	$423,144	N/A	N/A
38	2B	Pipe	B-OUT-025	B-OUT-024	10	2,728	145	$395,498	N/A	N/A
39	2B	Pipe	B-OUT-026	B-OUT-025	8	2,674	140	$374,412	N/A	N/A
40	2B	Pipe	B-OUT-027	B-OUT-026	8	2,585	140	$361,840	N/A	N/A
41	2B	Pipe	B-OUT-028	B-OUT-027	8	2,754	140	$385,579	N/A	N/A
42	2B	Pipe	B-OUT-029	B-OUT-028	8	5,282	140	$739,486	N/A	N/A
43	2B	Pipe	B-OUT-030	B-OUT-027	8	2,646	140	$370,491	N/A	N/A

No.	Area	Project Category	Upstream MH	Downstream MH	Pipe Diameter (in)	Pipe Length (ft)	Unit Cost ($/LF)	Total Cost ($)	Location	Description
44	2B	Pipe	B-OUT-031	B-OUT-026	8	2,673	140	$374,286	N/A	N/A
45	2B	Pipe	B-OUT-032	B-OUT-025	8	5,337	140	$747,118	N/A	N/A
46	2B	Pipe	B-OUT-033	B-OUT-024	10	5,343	145	$774,800	N/A	N/A
47	2B	Pipe	B-OUT-034	B-OUT-085	10	2,669	145	$387,032	N/A	N/A
48	2B	Pipe	B-OUT-035	2B-000-232	21	2,001	200	$400,261	N/A	N/A
49	2B	Pipe	B-OUT-036	B-OUT-098	12	5,267	155	$816,391	N/A	N/A
50	2B	Pipe	B-OUT-037	B-OUT-035	15	5,277	160	$844,336	N/A	N/A
51	2B	Pipe	B-OUT-038	B-OUT-076	12	5,284	155	$819,067	N/A	N/A
52	2B	Pipe	B-OUT-076	B-OUT-037	15	2,614	160	$418,249	Thunderbird Rd	From Navajo Rd to Cheyenne Rd
53	2B	Pipe	B-OUT-094	2B-NRS-039	18	2,758	180	$496,429	Johnson Rd	From Rialto Ave to Navajo Rd (2B-NRS-039)
54	2B	Pipe	B-OUT-095	B-OUT-094	15	2,724	160	$435,760	Johnson Rd	From Central Rd to Rialto Ave
55	2B	Pipe	B-OUT-098	B-OUT-035	12	2,646	155	$410,067	Otoe Road	From Navajo Rd to Cheyenne Rd
56	JR	Pipe	B-OUT-070	3A-AVCBV-001	8	5,002	140	$700,273	Bear Valley Rd	From Savage Lane to 3A-AVCBV-001.
57	JR	Pipe	B-OUT-073	B-OUT-072	8	6,060	140	$848,406	Del Oro Rd	From Merino Ave to Dandelion Lane to Connect with JR-16840-012.
58	JR	Pipe	B-OUT-074	JR-16840-001	12	5,329	155	$826,017	Grande Vista St	From Caribou Ave to JR-16840-001
59	JR	Pipe	B-OUT-075	JR-16758.OFF-013	8	7,069	140	$989,688	Tussing Ranch Rd	From Merino Ave to JR-16758.OFF-013
60	JR	Pipe	B-OUT-072	JR-16840-012	8	213	140	$29,820	Dandelion Ln	From Del Oro Rd to JR-16840-012
61	Northern AV	Pipe	B-OUT-001	B-OUT-086	8	2,710	140	$379,402	N/A	N/A
62	Northern AV	Pipe	B-OUT-002	VVWRA-NAVI-001	8	326	140	$45,583	Dale Evans Pky	From Morro Rd to VVWRA-NAVI-001
63	Northern AV	Pipe	B-OUT-003	B-OUT-002	8	10,685	140	$1,495,888	Morro Rd	From Central Road to Bell Mountain Rd
64	Northern AV	Pipe	B-OUT-004	VVWRA-NAVI-009	8	10,654	140	$1,491,508	Colusa Rd	N/A

No.	Area	Project Category	Upstream MH	Downstream MH	Pipe Diameter (in)	Pipe Length (ft)	Unit Cost ($/LF)	Total Cost ($)	Location	Description
65	Northern AV	Pipe	B-OUT-005	VVWRA-NAVI-014	8	10,647	140	$1,490,549	N/A	N/A
66	Northern AV	Pipe	B-OUT-006	B-OUT-084	8	5,235	140	$732,853	Quarry Rd	From Central Road to Harris Ln
67	Northern AV	Pipe	B-OUT-007	B-OUT-088	10	7,309	145	$1,059,796	Fairfield Ave	From I-15 to Harris Rd
68	Northern AV	Pipe	B-OUT-008	B-OUT-089	10	5,843	145	$847,247	N/A	N/A
69	Northern AV	Pipe	B-OUT-009	VVWRA-NAVI-058	8	5,273	140	$738,167	N/A	N/A
70	Northern AV	Pipe	B-OUT-010	B-OUT-082	18	2,669	180	$480,385	Cordova	N/A
71	Northern AV	Pipe	B-OUT-011	VVWRA-NAVI-039	15	4,085	160	$653,608	Johnson Rd	From Dale Evans Road to Harris Rd
72	Northern AV	Pipe	B-OUT-039	VVWRA-NAVI-061	12	294	155	$45,513	N/A	N/A
73	Northern AV	Pipe	B-OUT-040	B-OUT-039	10	7,844	145	$1,137,386	N/A	N/A
74	Northern AV	Pipe	B-OUT-041	B-OUT-039	12	1,977	155	$306,494	N/A	N/A
75	Northern AV	Pipe	B-OUT-042	B-OUT-041	12	5,258	155	$815,033	N/A	N/A
76	Northern AV	Pipe	B-OUT-043	B-OUT-041	10	2,694	145	$390,665	N/A	N/A
77	Northern AV	Pipe	B-OUT-044	B-OUT-043	8	5,294	140	$741,170	N/A	N/A
78	Northern AV	Pipe	B-OUT-045	B-OUT-043	8	2,629	140	$368,027	N/A	N/A
79	Northern AV	Pipe	B-OUT-046	B-OUT-045	8	5,083	140	$711,676	N/A	N/A
80	Northern AV	Pipe	B-OUT-047	B-OUT-045	8	2,861	140	$400,515	N/A	N/A
81	Northern AV	Pipe	B-OUT-048	VVWRA-NAVI-074	8	4,161	140	$582,481	Isaugus Road	From Pauma to VVWRA-NAVI-074.
82	Northern AV	Pipe	B-OUT-049	VVWRA-NAVI-080	8	4,948	140	$692,725	Solano Rd	From Solano Road to VVWRA-NAVI-080.
83	Northern AV	Pipe	B-OUT-050	VVWRA-NAVI-088	8	6,320	140	$884,773	Iroquois Rd	From Pine Ridge Ave to Navajo Road
84	Northern AV	Pipe	B-OUT-077	B-OUT-079	18	2,573	180	$463,195	Cardova Rd	N/A
85	Northern AV	Pipe	B-OUT-078	VVWRA-NAVI-033	18	3,449	180	$620,845	Cardova Rd	From Dale Evans Road to VVWRA-NAVI-033
86	Northern AV	Pipe	B-OUT-079	B-OUT-078	18	2,767	180	$498,043	Cardova Rd	From Dachshund Ave to Dale Evans Pky

No.	Area	Project Category	Upstream MH	Downstream MH	Pipe Diameter (in)	Pipe Length (ft)	Unit Cost ($/LF)	Total Cost ($)	Location	Description
87	Northern AV	Pipe	B-OUT-080	B-OUT-077	18	2,691	180	$484,435	Cardova Rd	N/A
88	Northern AV	Pipe	B-OUT-081	B-OUT-080	18	2,592	180	$466,579	Cardova Rd	N/A
89	Northern AV	Pipe	B-OUT-082	B-OUT-081	18	2,553	180	$459,505	Cardova Rd	N/A
90	Northern AV	Pipe	B-OUT-083	VVWRA-NAVI-024	8	1,963	140	$274,794	Quarry Rd	From Bell Mountain Rd to Harris Ln (VVWRA-NAVI-024)
91	Northern AV	Pipe	B-OUT-084	B-OUT-083	8	5,362	140	$750,667	Quarry Rd	From Central Rd to Bell Mountain Rd
92	Northern AV	Pipe	B-OUT-085	B-OUT-023	15	2,679	160	$428,615	Waalew Rd	From Central Road to Camel Ln
93	Northern AV	Pipe	B-OUT-086	B-OUT-087	8	2,245	140	$314,272	Wild Wash Rd	From Morro Road to Wild Wash Rd
94	Northern AV	Pipe	B-OUT-087	VVWRA-NAVI-027	8	3,671	140	$513,889	Wild Wash Rd	From Wild Wash Road to VVWRA-NAVI-027.
95	Northern AV	Pipe	B-OUT-088	VVWRA-NAVI-043	15	6,463	160	$1,034,146	Fairfield Ave	From Kimshew St to VVWRA-NAVI-043.
96	Northern AV	Pipe	B-OUT-089	VVWRA-NAVI-049	15	4,989	160	$798,192	Cordova Rd	From Cordova Rd to VVWRA-NAVI-055.
						Construction Cost		**$54,968,246**		
						Contingencies (35%)		**$19,238,886**		
						Total Cost		**$74,207,132**		

APPENDIX C
RECOMMENDED CIPS FOR LIFT STATIONS

No.	Lift Station	Assessment District	Upgrade Description	Cost ($)
1	2A #2	AD 2A	Add one additional pump at 150 gpm	$75,600
2	Tr. 17247		Add two pumps at 150 gpm	$151,200
3	2B	AD 2B	Add two pumps at 725 and 1,200 gpm	$970,200
4	Tr. 17093	AD 3A	Add two pumps at 100 gpm	$100,800
5	VVWRA Nanticoke AD 2	AD 3A	N/A*	N/A*
			Construction Cost	**$1,297,800**
			Contingencies (35%)	**$454,230**
			Total Cost	**$1,752,030**

*This pump station requires upgrade but the cost for upgrade is not included in this report as the operation and maintenance of this station is the responsibility of VVWRA.

APPENDIX D
TOWN OF APPLE VALLEY SEWER CONNECTION POLICY

Town of Apple Valley Sewer Connection Policy

Effective January 11, 2006, all new single-family lots created by subdivision, whether by Tentative Tract Map, Tentative Parcel Map, or Lot Split, where the newly created lots will have a total gross lot size of less than one acre, shall be required to connect to the Town Sewer System.

When the existing sewer infrastructure is more than one-half (½)-mile away, the developer shall be required to install a "dry sewer," consisting of onsite main collector sewers and laterals to each new lot. Sewer service laterals shall be extended from the main collector sewer, to within five (5) feet of the septic system on all newly created lots. The collector sewer and laterals shall be installed in conformance with Town Standards prior to final map. All plan check and construction inspection fees shall be paid at time of plan submittal. Sewer connection fees shall be paid upon final inspection of construction to accommodate sewer connection by the homeowner at a later date.

At the discretion of the developer, and as an alternative to a "dry sewer", the developer may choose to install an interim "Holding Tank System" as approved by the Town. The holding tank system will create a working collector sewer system within the development to serve the newly created lots. The developer will be required to set aside sufficient land from the development project to accommodate proper maintenance and operation of the holding tank system installation. A homeowners association or other mechanism approved by the Town, will be required to provide required maintenance and pumping of the holding tank system. Sewer connection and use fees shall be paid upon final inspection of construction. Sewer connection fees shall accommodate connection to the sewer system by the homeowner at a later date, and sewer use fees shall be collected to fund the cost of treatment of the wastewater flows through an approved location on the Town's existing sewer system.

If approved by the Regional Water Quality Control Board and the Town, the developer may install an interim "Community Septic System" for disposal of wastewater flows from the development. The community septic system will create a working collector sewer system within the development to serve the newly created lots. The developer will be required to set aside sufficient land from the development project to accommodate proper maintenance and operation of the system, including disposal expansion area. A homeowners association will be required to provide required maintenance and pumping of the community septic system. Sewer connection fees shall be paid upon final inspection of construction to accommodate sewer connection by the homeowner at a later date. Sewer use fees will be required only upon permanent connection to the Town sewer system infrastructure.

Upon connection and activation of the collector sewers to the Town's existing sewer infrastructure, all lots within the tract, parcel map, or lot split subdivision shall be required to abandon interim holding tank/septic tank/community septic system use, and make permanent connection to the Town sewer system as required. In the case of connection of lots utilizing dry sewers and interim septic systems, the cost for connection of the lateral and abandonment of the interim septic system shall be the responsibility of the homeowner to pay at the time connection is made.

Adopted per Town Council Minute Action on January 10, 2006.