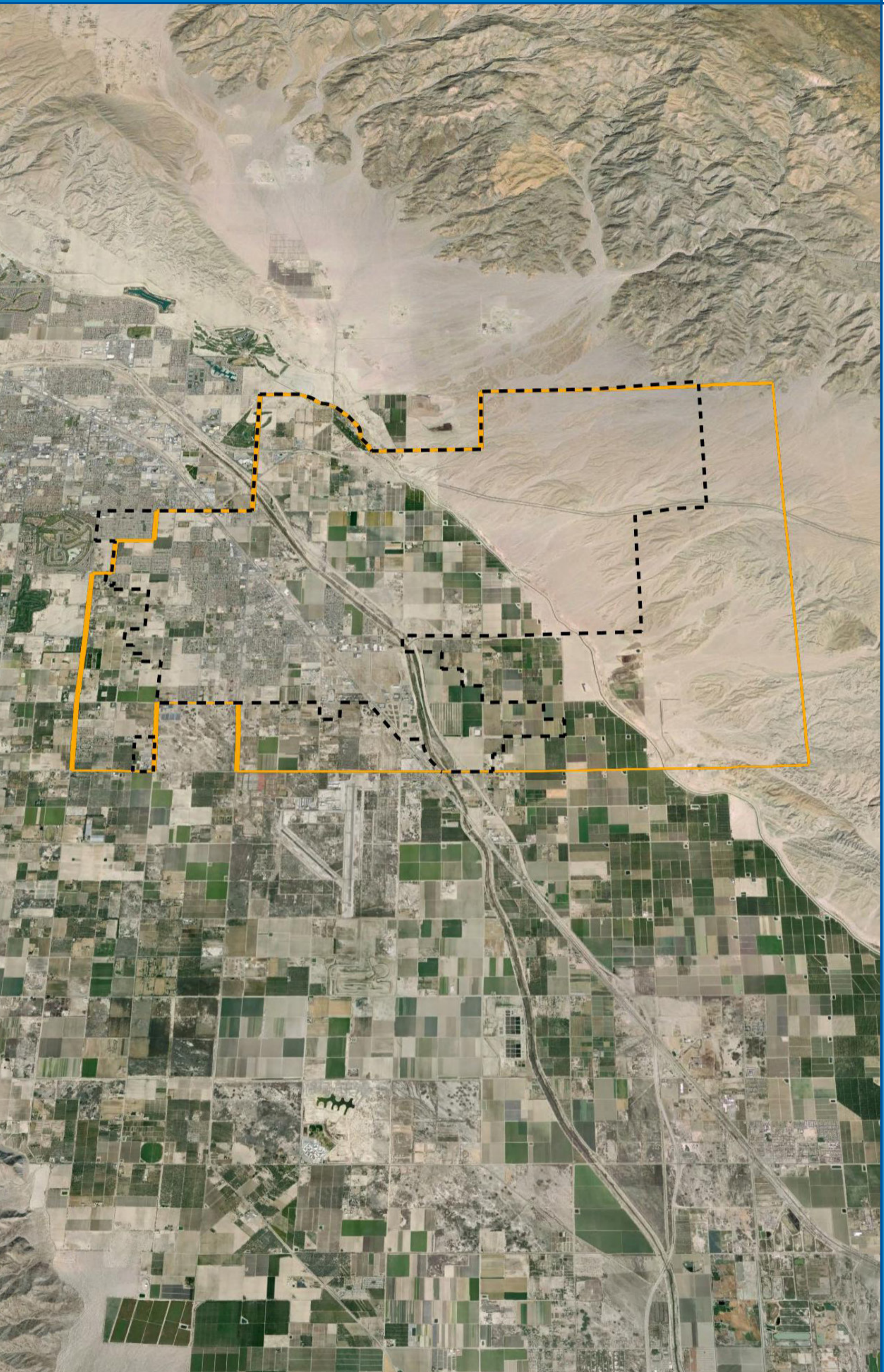
REPORT



# City of Coachella 2015 Sewer System Master Plan

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# Table of Contents

# Appendices

## List of Tables



## List of Figures

# Acronyms

| | |
|---|---|
| BWWF | Base Wastewater Flow |
| CIP | Capital Improvement Program |
| City | City of Coachella |
| CVHS | Coachella Valley High School |
| CVHS PS | Coachella Valley High School Pump Station |
| CVWD | Coachella Valley Water District |
| d/D | depth of flow in the pipe relative to the pipe diameter |
| ft/sec | feet per second |
| GIS | Geographic Information System |
| gpd | gallons per day |
| GWI | groundwater infiltration |
| HGL | hydraulic grade line |
| ID | identification number |
| mgd | million gallons per day |
| master plan | sewer system master plan |
| RDII | Rainfall-dependent infiltration/inflow |
| SR-86 | State Route 86 |
| WWTP | Waste Water Treatment Plant |

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# Section 1

# Background and Objectives

## 1.1 Introduction

The City of Coachella (City) is located in Riverside County, California; it is the easternmost city in the region collectively known as the Coachella Valley. Based on the survey of the existing manholes, the elevation of the existing service area ranged from 108 feet below sea level to 6 feet above sea level with an average elevation of 68 feet below sea level. The City's incorporated area is 30 square miles. The sanitary District's boundary includes 47 square miles. It is a growing community with a population of 43,633 (City data).

The overall goals of this Sewer System Master Plan (Master Plan) are to evaluate the existing system for capacity, identify existing and future deficiencies in the collection system as a result of future developments through year 2040 planning horizon, size pipes and pump stations to convey 2040 peak flow, and prioritize phased recommended improvements. This section provides the background of the Master Plan, objectives and summary of the organization of the Master Plan report.

## 1.2 Background

The City is developing with the construction of adopted or approved planned developments. According to the Development Services Department, it is possible that smaller properties may be developed using only a tentative tract map and design review process; there will also be master-planned developments using Specific Plans or Planned Development Overlay zones. The part of the City west of the State Route 86 (SR-86) is primarily comprised of residential development with moderate commercial and industrial mixed use. The currently vacant lots within the already developed areas will provide room for future growth by tentative tract maps, which will place increased loads on the existing sewer system. The eastern part of the City from the SR-86 to the east end of the Sanitary District's boundary is mostly agricultural or undeveloped land that will be developed by master plans as well as tentative tract maps. In addition, in parts of the City, some areas are currently served by septic systems, which have the potential to connect to the City's sewer system in the future.

To address the current and future increased loads on the existing system, the Master Plan includes hydraulic modeling of existing and future sewer infrastructure flow scenarios. The results of hydraulic modeling identified the existing system's deficiencies to convey the existing and future flows. The depth of flow compared to the pipe diameter (d/D) ratio as well as the hydraulic grade line (HGL) were used as criteria to identify system deficiencies. The identified deficiencies were examined by viewing pipe profile for each location to assess the severity of the hydraulic conditions. As a result, recommended improvements were determined and prioritized based on the proximity of the hydraulic grade line to the ground surface. The findings of this study will provide a basis for the City to develop a Capital Improvement Program (CIP) that will correct the system's deficiencies and provide direction to developers for the necessary infrastructure to accommodate future growth.

## 1.3 Project Goals and Objectives

The main goals and objectives of this Master Plan are:

- Perform a system-wide flow monitoring analysis to provide flow data for developing land use unit flow factors and calibration of the hydraulic model.
- Develop a system-wide sewer system hydraulic model calibrated for Existing conditions to be used as a tool to predict future system performance including Intermediate and 2040 conditions.
- Evaluate the collection system (10-inches in diameter and larger) for capacity as it currently exists and under future flow conditions, as well as identify system deficiencies.
- Develop a master plan for phased and prioritized system improvements to mitigate system deficiencies.
- Provide a hydraulic model database that, if the City purchases modeling software in the future, can be updated and used for evaluating sewer system capacity under different flow conditions.

## 1.4 Report Organization

The Master Plan is organized as follows:

### *Section 1 – Background and Objectives*

This section provides an introduction of the City's geographic location and recent development background. The resulting need for a sewer system Master Plan is described by the project goals and objectives. An outline of the report organization is provided at the end of this section.

### *Section 2 – Existing Sewer System and Model Development*

This section provides an overview of the City's existing sewer collection system including sewer mains, manholes, and pump stations. A description of model development introduces the methods and procedures of building a computerized hydraulic model from the City's information.

### *Section 3 – Land use*

Land use information provides the planning basis for the projection of wastewater flows. This section identifies the sources of land use information utilized for the analysis, and describes existing land uses and projected future land uses within the study area.

### *Section 4 – Model Calibration and Flow Projections*

This section provides a description of the system-wide flow monitoring program. The results were used to develop unit flow rates and diurnal curves for the major land use types based on the dry weather flow analysis. Wet weather flow analysis was not part of the scope of the Master Plan due to the very low amounts of rainfall that the city experiences (less than 5 inches per year). The methodologies used for model calibration and flow projections are also provided.

### *Section 5 – Hydraulic Criteria*

This Section describes the hydraulic criteria that were established to define the method by which gravity sewers, force mains, and pump stations are evaluated using the hydraulic model. The proposed hydraulic criteria were used to evaluate the sewer system's performance and identify the capacity deficiencies to meet existing and future needs

### *Section 6 – Sewer System Analysis*

This section presents the findings of the sewer system analysis to identify the need for system capacity under Existing, Intermediate and, 2040 planning horizon conditions.

### *Section 7 – Recommended Improvements*

This section presents the recommended sewer system improvements based on the hydraulic criteria and system analysis results. Improvements were prioritized based on the relative depth of pipe surcharge to the ground elevation. A planning level capital cost estimate is provided for the recommended improvements.

### *Appendix A – Dry Weather Flow Calibration Results*

This appendix provides comparison of the field measured dry weather flow data and simulated model results using the calibrated land use unit flow factors and diurnal patterns. A list of graphs of the metered verse modeled data at various flow monitoring locations is provided.

### *Appendix B – Existing, Intermediate and, 2040 Scenarios Maximum HGL Profiles*

This appendix provides the model hydraulic analysis results for Existing, intermediate, and 2040 Scenarios. The profiles show the maximum HGL for sewers that exceeded the hydraulic design criteria under peak dry weather flow conditions.

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# Section 2

# Existing Sewer System and Model Development

## 2.1 Introduction

This section describes the City's existing sewer system, and the development of a computerized hydraulic model to represent the system layout and configuration. The hydraulic model was used to analyze the existing sewer system under existing and future land use conditions, as discussed in later sections.

## 2.2 Description of Existing Facilities

The City of Coachella sewer system consists of sewers that collect local flows generated from the City's residential, commercial, and industrial areas and discharge to the City's Avenue 54 wastewater treatment plant (WWTP) with a capacity of 4.5 million gallon per day (MGD). The System collects flow by gravity from the majority of the area and discharges into the WWTP. There are two pump stations one located on the west side near the Coachella Valley High School and one located on Polk Street and Avenue 52.

The following sections describe the different components of the existing system: sewers, manholes, and lift stations.

### 2.2.1 Sewers

The City of Coachella's collection system includes about 90 miles of sanitary sewers ranging in size from 4-inches to 54-inches in diameter. About 35.6 miles, 39 percent of the total length, were modeled with diameters ranging in size from 6-inches to 54-inches, which are the backbone of the sewer system conveyance network. Modeling only sewers that are 10-inches in diameter and greater provides a good representation of the backbone of the sewer system conveyance network, which is in line with standard modeling practices and does not affect the outcome of a sewer master plan. Modeling an entire system is not needed and would be extremely costly and time consuming. The model has been developed so it can be expanded in the future as needed to model specific areas as needed to for capacity evaluation.

The hydraulic model included sewer lines provided by the City in an AutoCAD file, which was found to be schematic with diameter information only. The modeled sewer lines are the "trunk sewers" primarily 10 inches in diameter and larger. Some 6- and 8-inch diameter pipes were included in the model as needed to further define a specific area of interest where the size of the served area is too large or for the hydraulic connectivity purpose. This model's database was augmented by performing surveying of manholes' rim elevations and pipe inverts. The sewer lines not included in the hydraulic model are shown as non-modeled sewers in the model's GIS database. Table 2-1 summarizes the length of the sanitary sewers by diameter for the entire existing system and for the modeled system. Figure 2-1 shows the modeled and non-modeled sewer lines.

**Table 2-1 Inventory of Sanitary Sewers**

| Entire System | | | Modeled System | | |
|---|---|---|---|---|---|
| Diameter (in) | Length (ft) | % | Diameter (in) | Length (ft) | % |
| 4 | 344 | 0.1 | - | - | - |
| 6 | 10,236 | 2.1 | 6 | 3,222 | 1.72 |
| 8 | 289,933 | 60.7 | 8 | 22,038 | 11.73 |
| 10 | 24,572 | 5.1 | 10 | 24,572 | 13.08 |
| 12 | 42,288 | 8.9 | 12 | 33,244 | 17.70 |
| 15 | 48,401 | 10.1 | 15 | 48,401 | 25.77 |
| 18 | 23,132 | 4.8 | 18 | 18,797 | 10.01 |
| 21 | 8,894 | 1.9 | 21 | 8,894 | 4.74 |
| 24 | 20,978 | 4.4 | 24 | 20,511 | 10.92 |
| 27 | 955 | 0.2 | 27 | 955 | 0.51 |
| 42 | 6,015 | 1.3 | 42 | 6,015 | 3.20 |
| 54 | 1,178 | 0.2 | 54 | 1,178 | 0.63 |
| (blank) | 478 | 0.1 | | | |
| Grand Total | 477,403 | | | 187,827 | |

## 2.2.2 Manholes

The Coachella system has over 1,580 manholes as identified by manhole survey. As previously stated, modeling only sewers 10-inches in diameter and greater provides a good representation of the backbone of the sewer system conveyance network and is in line with standard modeling practices. As a result, 554 were included in the model. The naming convention of the manholes used within this report and the model corresponds with the nomenclature developed by this Master Plan project.

## 2.2.3 Pump Stations

The existing sewer system has two pump stations:

1- Coachella Valley High School Pump Station (CVHS PS): This pump station receives flows for the area along Airport Boulevard through an 8-inch line. The pump station has two pumps each at a rated capacity of 1.02 cfs. The flows are pumped through a 6-inch main to the north of Avenue 55 where flows are conveyed by gravity to the WWTP

2- Polk Street Pump Station: During the master plan study period, this pump station had to be taken out of service by the City for maintenance due to foaming problems in the wet well and equipment vandalism; the flow was temporarily bypassed by the City to the system west of ST-86 at Avenue 48. The pump station was assumed to be in operation for the analysis of the existing and future system conditions. The pump station has two pumps each at a rated capacity of 8.25 cfs. It receives flows from the areas north of Avenue 47 and east of the State Route 86 (ST-86) SPUR N through a 24-inch sewer line. The flows are pumped through a 24-inch force main to a manhole near Avenue 52 and La Hernandez Street and then conveyed by gravity to the WWTP.

The location of the pump stations are shown in Figure 2-1. Table 2-2 summarizes the pump station information and the hydraulic characteristics of the pumps.

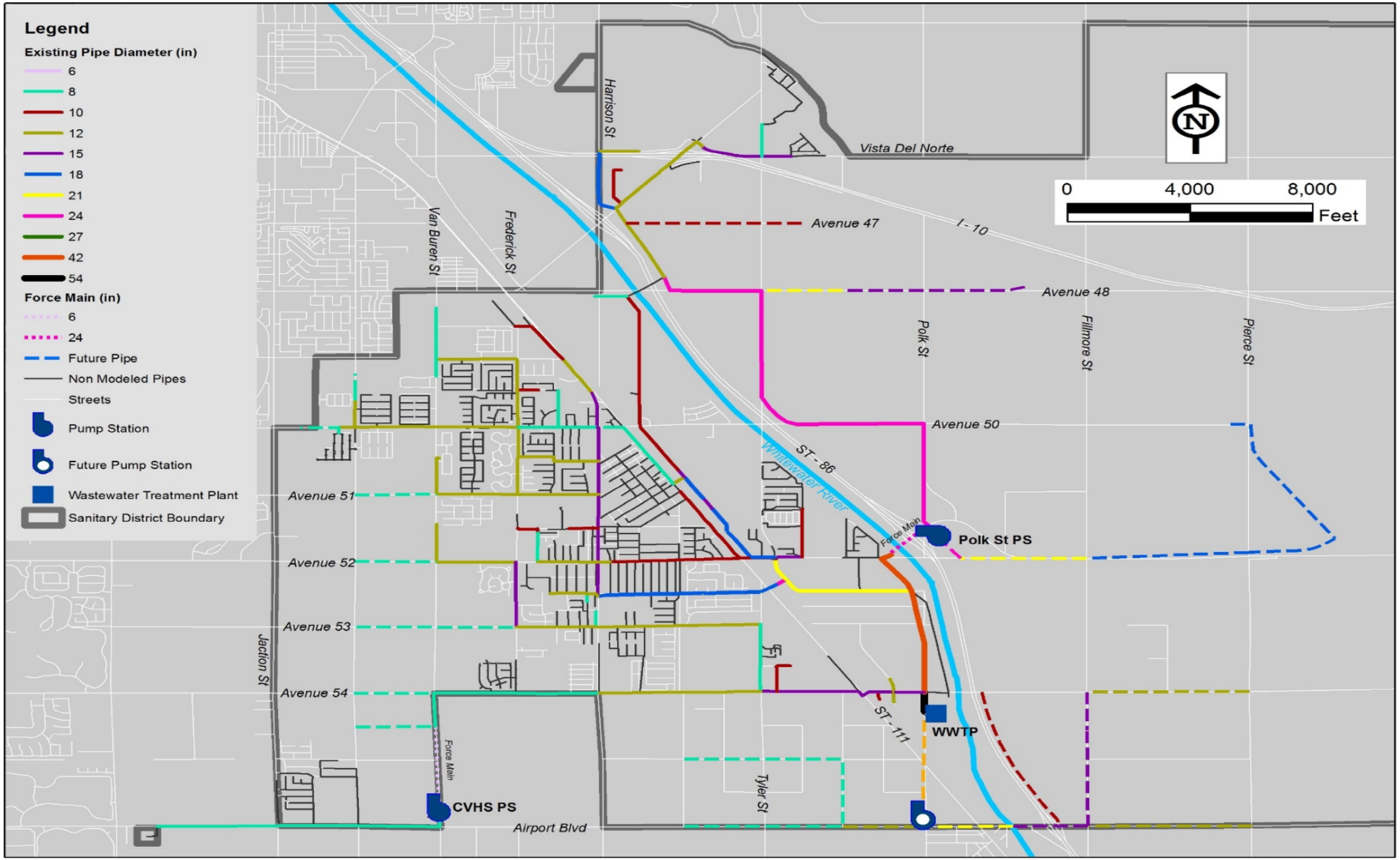


Figure 2-1
Modeled Existing and Proposed Future Pipelines

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**Table 2-2 Pump Station Hydraulic Characteristics**

| Station Name | Pump No. | Rated Flow (gpm) | Rated TDH (ft) |
|---|---|---|---|
| Polk Street | 1 | 3700 | 77 |
| | 2 | 3700 | 77 |
| Coachella Valley High School (CVHS) | 1 | 460 | 32 |
| | 2 | 460 | 32 |

## 2.3 Model Development

The hydraulic model of the City of Coachella wastewater collection system was developed using InfoSewer version 7.6 software developed by Innovyze®. InfoSewer is an ArcGIS-based computer program for use in the planning, design, analysis, and expansion of sanitary sewer collection systems. The program can effectively be used to model both dry and wet weather flows and determine cost-effective and reliable methods of wastewater collection. InfoSewer integrates advanced hydraulics and hydrology modeling functionality with the latest generation of ArcGIS and capitalizes on the intelligence and versatility of the geodatabase architecture to perform infrastructure management and planning. InfoSewer allows the user to create, edit, modify, run, map, analyze, design, and optimize sewer network models and then review, query, and display the simulation results within ArcGIS.

InfoSewer has comprehensive hydraulic and dynamic computational capabilities. The features of InfoSewer software are listed as follows:

- Steady-state analysis using various peaking factors
- Simulates complex flow (hydrograph) attenuation (peak flow damping effect) throughout the collection system using advanced Muskingum-Cunge explicit diffusion (dynamic) wave model
- Implements a dynamic flow routing model based on the well-established Muskingum-Cunge explicit diffusion wave algorithm (a simplified form of the full 1D Saint Venant equations neglecting inertial terms) to accurately track spatial and temporal variation of sewage flows throughout collection system
- Solves network flow dynamic equations using a highly robust and efficient explicit finite difference scheme with variable time step
- Tracks the percent of sewage flow from any given manhole reaching all other pipes and manholes over time
- Carries out accurate HGL calculations under surcharge conditions
- Calculates the age of sewage (time of concentration) throughout a network
- Satisfies conservation of mass during a surcharged extended period simulation (EPS) simulation

The hydraulic model was developed by inputting network components that were either surveyed and/or provided by the City. The manholes and pipes within the model area were placed directly into the model with their attributes. The manholes and pipes were then updated to reflect recent surveying efforts performed during this project.

Aerial photography and land use were displayed as background layers, allowing for confirmation of the sewer pipe network layout. Tributary sewer sheds for wastewater flow contribution were also delineated for modeling purposes. A description of these model elements follows.

### 2.3.1 Sewers

The sewers 10-inch in diameter and larger within the existing collection system were included in the model as the capacity issues of the trunk system are of greatest concern. Additional smaller diameter pipes were added as needed to keep the tributary areas to a reasonable size. The modeled system includes 35.6 miles of pipes. Model data for sewers included diameter, length, upstream and downstream manholes, and upstream and downstream invert elevation. The invert elevations of both ends of each modeled sewer were obtained from surveyed information, along with the length. The invert elevations determined the slopes of the sewers and enabled the model to track offsets between pipes entering or exiting the same manhole. A Manning's n value of 0.013 was used based on the typical roughness value for a vitrified clay pipe.

### 2.3.2 Manholes

Model data for manholes included manhole diameter, rim elevation, invert elevation, load, and loading pattern. The modeled manholes and their attributes were gathered from the field surveying efforts and supplemented with manholes with inverts calculated from record drawings.

The model IDs of majority existing manholes were developed by collaboration between the City and CDM Smith using the existing map provided by the City. The System Maps were setup using a grid map method. The grid map method consisted of 3,000' x 2,000' grids being applied over the City's sewer service area. Each grid was assigned a unique identification number (ID) which contained an alphabetical letter followed by numbers. Each grid was cut into one sheet of the System Maps, and the grid ID was used as the sheet numbers. Mainly 10-inch and larger sewer pipes were included in the System Maps, which are the pipes that will be hydraulically modeled, with some 8-inch and 6-inch diameter pipes added as needed to keep the tributary areas to a reasonable size.

As shown in Figure 2-2, the sheets that contain the said sewer pipes are encompassed by red borders and identified by red sheet numbers. Most manholes shown in these sheets were verified by field surveying to gather rim and invert elevations and were included in the hydraulic model in future tasks.

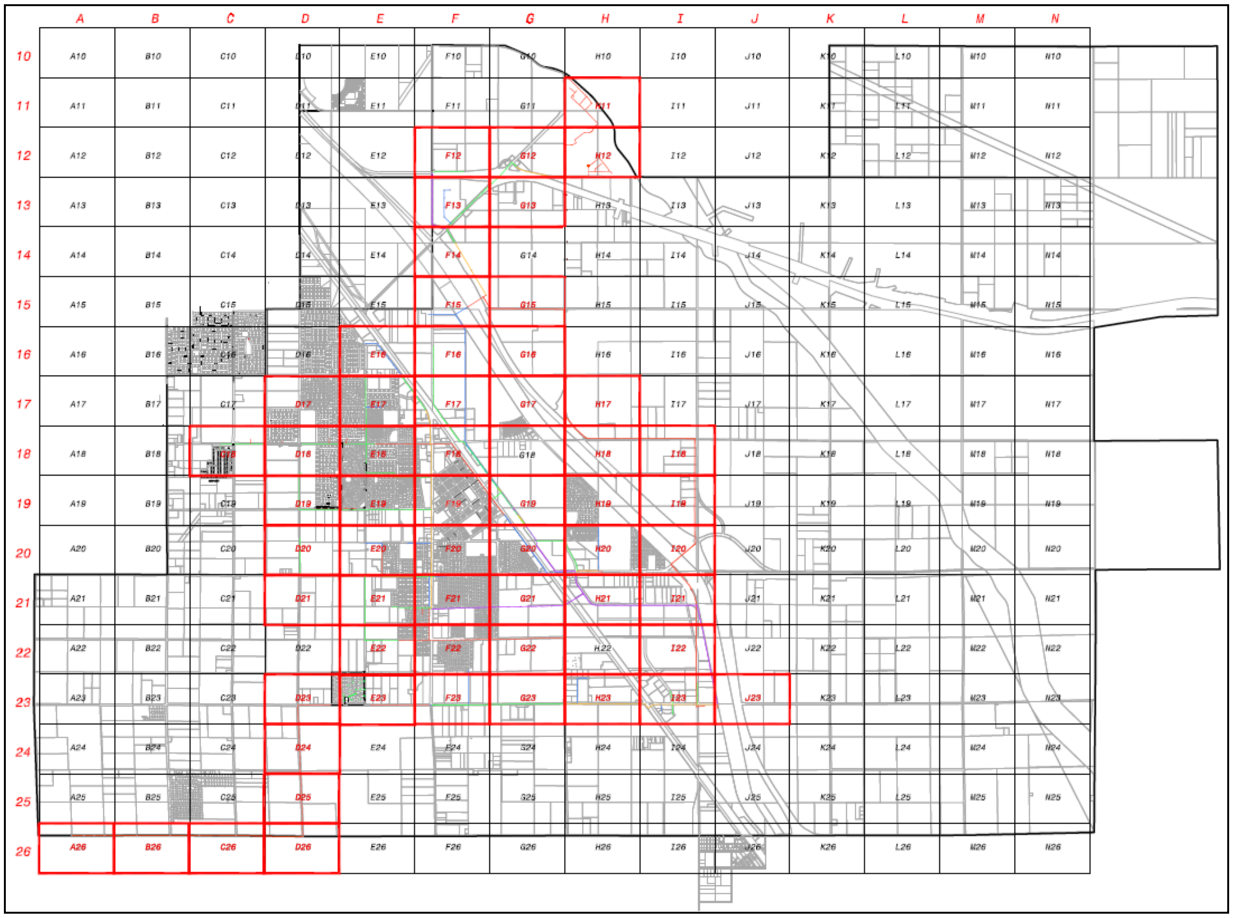


**Figure 2-2**
**Sewer System Manhole Numbering Grids**

The manholes on each sheet were identified by an ID using the sheet number and an assigned a numerical identification in the format of XXX-YYY. The first set of numbers, XXX, is the sheet number; the second set of numbers, YYY, is the MANHOLE number for manholes within that sheet, starting from 001 and ending to 999. The Manhole numbering sequence on each sheet can be assigned from west side to east side, from north side to south side, from upstream to downstream, from older sewer lines to newer sewer lines, or from larger diameter pipes to smaller diameter pipes, and not in any particular priority. This numbering method follows certain degree of logic and provides flexibility for including some of the existing 8-inch sewer pipes, missing manholes encountered during the survey and future system expansions.

Figure 2-3 shows a sample of the manhole numbering on Sheet F18. A manhole on the west end of Avenue 50 was assigned an ID of F18-001, and the manhole downstream along this 8-inch sewer was numbered from F18-002 through F18-010. Manhole IDs from F18-011 through F18-014 were assigned to the along the 15-inch sewer in Harrison Street from upstream to downstream as well as from north to south. The Manhole for the 12-inch sewer and the 10-inch sewer east of the Union Pacific Rail Road were numbered using the same methodology.

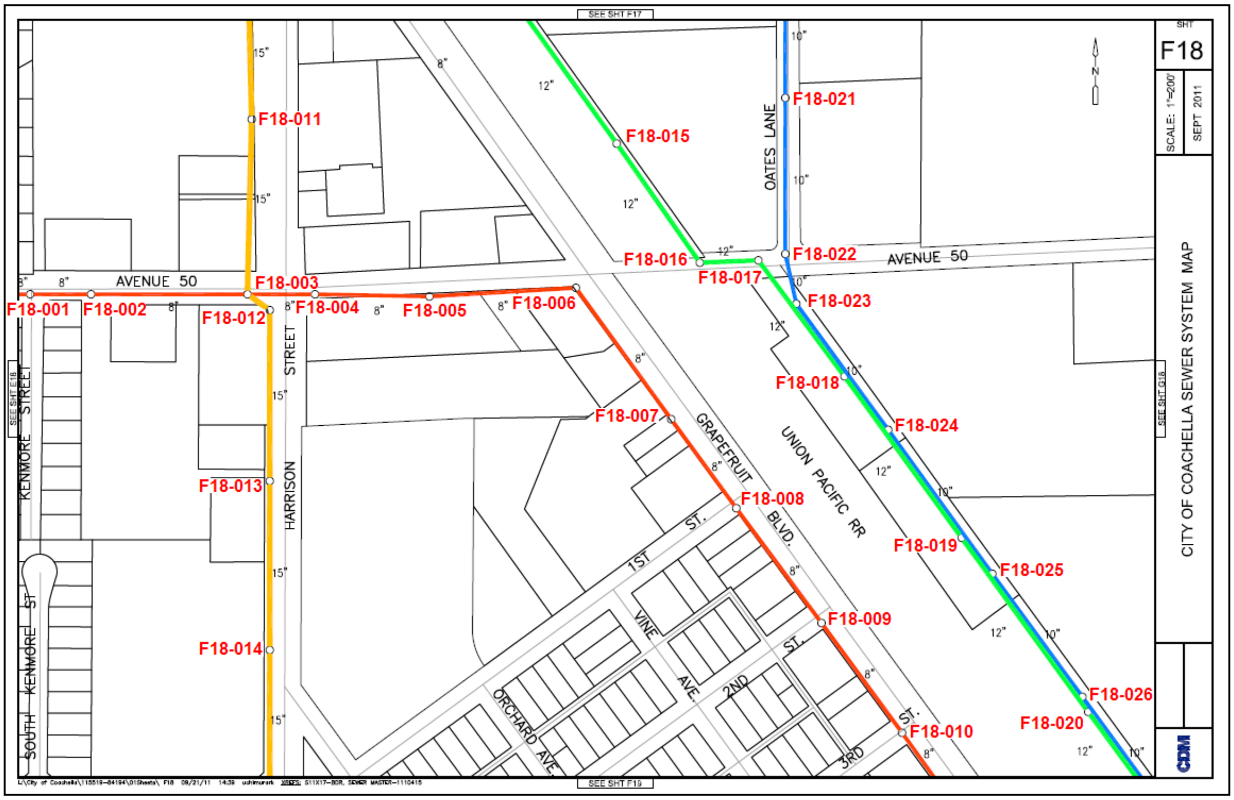


**Figure 2-3**
**Sample of Manhole Number Sequence**

There are additional existing sewers of diameter smaller than 10 inches in the area contained by Sheet F18, but they are not presently shown in this map. For example, there are 8-inch sewer lines on 1st Street and 2nd Street. If the City decides to include all the existing sewer pipes and manholes from the System Maps or other facility management records, manholes can be numbered after F18-026 shown on Sheet F18. If there is future development in this area and new pipes and manholes will be installed, they can also be numbered after the largest shown number on this Sheet. The above numbering system can be considered from larger pipes to smaller pipes or from older pipes to newer pipes. There are 999 different manhole numbers that can be used on each sheet, thus providing a lot of flexibility and room for expansion.

Once the manholes and pipes were imported into the model, an error-checking routine in the model was used to generate a list of facilities with missing information. As a result, a list of manholes with missing rim and invert elevations was developed. A portion of the data was later acquired with additional surveying and the rest was calculated from the city's record drawings.

The sewer system data for manholes can be stored under manhole IDs in the survey records, hydraulic model, as well as in other future facility management records kept by the City, such as a CCTV inspections, asset management, and GIS. The data that are important for the master plan include manhole location coordinates, rim elevation, invert elevations, and diameters of the pipes. Other data that could be added in the future by the City are diameter of the manhole, date of installation of the manhole, material and type of the manhole.

### 2.3.3 Pump Stations

There are two pump stations within the City of Coachella existing sewer system.

The operation of the pumps was based upon the depth of flow in the wet wells with lead and lag set points. Upon increasing level, when the level reaches the “Lead Pump On” set point, the lead pump will start. If the level in the wet well continues to rise and reaches the “Lag Pump On” set point then the lag pump will start. When the level in the wet well falls below the “Lag Pump Off” set point the lag pump will stop. If the level in the wet well falls below the “Lead Pump Off” set point then the lead pump will stop.

The model has the capability of modeling pumps in three different ways as: 1) Fixed Capacity; 2) Design Point Curve; and 3) Exponential 3-point curve. For this analysis, the exponential 3-point curve option is used based on the pump curve sheets provided by the City.

### 2.3.4 Sewershed Delineation

A sewershed is defined as a geographic and/or hydrologic region (tributary area) in which all wastewater flows are conveyed to a single point, or outlet, before being conveyed elsewhere. Topography usually governs the size and shape of the sewershed in a given collection system.

The study area was divided into 49 existing and 41 future sewersheds. Each sewershed is identified by the manhole to which the sewershed discharges its wastewater. These manholes, called “load points”, were selected based on topography and the locations of existing sewers to determine where flow from the sewershed would likely enter the modeled system. Figure 2-4 shows the sewersheds and their load points.

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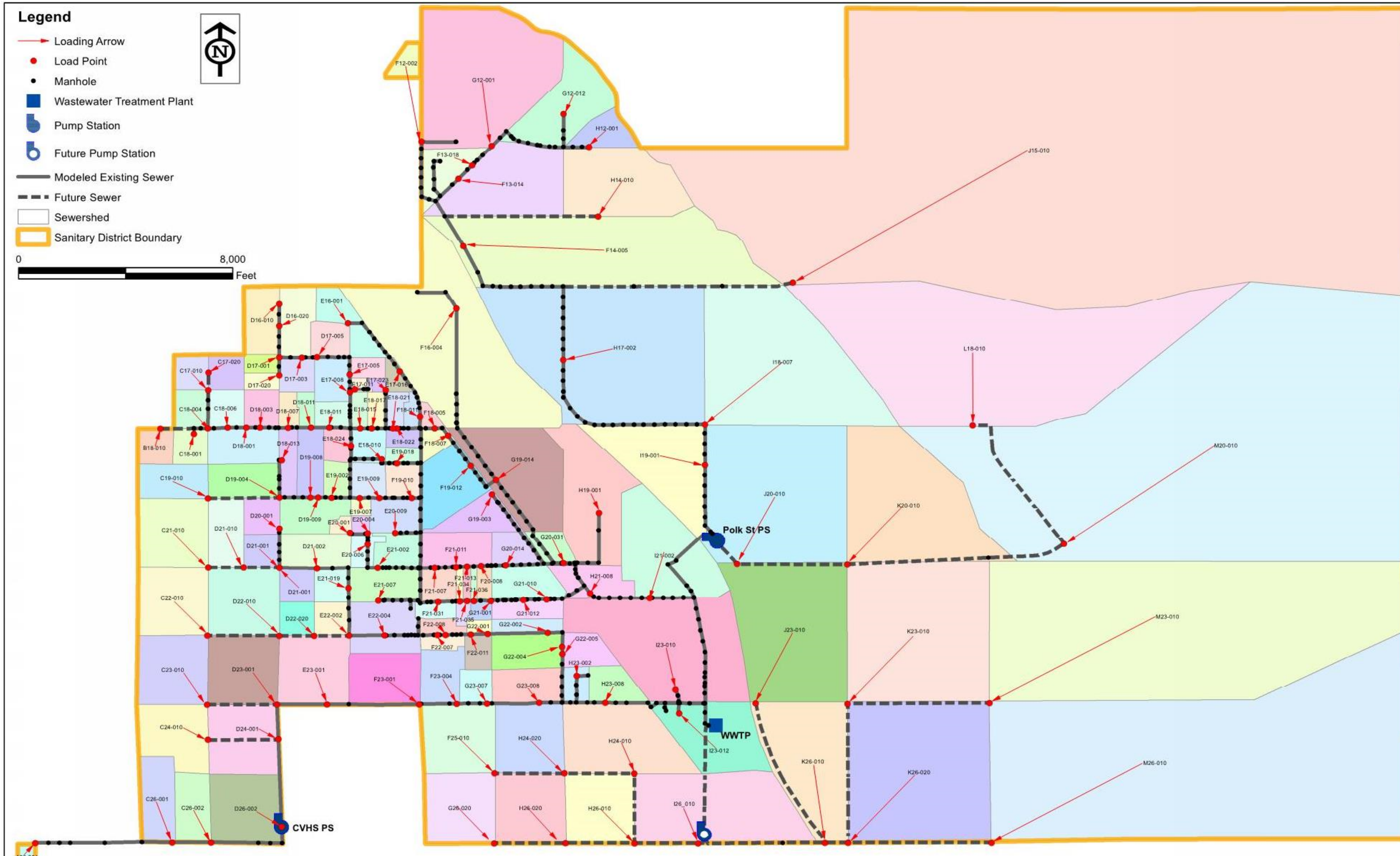


**Figure 2-4**
**Sewersheds and Load Points**

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# Section 3

# Land Use

## 3.1 Introduction

Land use information provides the planning basis for the projection of wastewater flows. Information on the number of acres of various land use types is used in conjunction with unit flow rates to generate wastewater flow projections. This section identifies the sources of land use information utilized by the analysis, and describes existing land uses and projected future land uses within the study area.

## 3.2 Existing Land Use

Land use data was provided by the City of Coachella as a land use/zoning layer in the Geographic Information System (GIS) format called "Land_use_Policy_Diagram". The existing land use that contributes wastewater flows was determined by excluding the vacant areas using the aerial photography map provided by the City. In Figure 3-2, the identified existing service areas are color coded in yellow. GIS tools were used to intersect the existing service area boundary with the land use zoning map. Table 3-1 summarizes the acreages by land use zoning category. Figure 3-1 shows the land use categories within the study area boundary.

**Table 3-1 Land Use Zoning Category**

| Land Use Zoning Category | Zoning Code | Area (acre) |
|---|---|---|
| Agriculture (1du/40ac) | AG | 7,950 |
| Entertainment Commercial | CE | 178 |
| General Commercial | CG | 432 |
| Heavy Industrial | IH | 187 |
| Light Industrial | IL | 494 |
| High Density Residential (0-20 Du/Ac) | RH | 54 |
| Low Density Residential (0-6 Du/Ac) | RL | 3,702 |
| Medium Density Residential (0-10 Du/Ac) | RM | 172 |
| Open Space | OS | 4,458 |
| Public Use | P | 234 |
| Very Low Density Residential (0-2 Du/Ac) | RVL | 64 |

## 3.3 Future Land Use

Future land use was determined by overlaying the City's zoning layer with the future sewer service areas. The City's zoning layer was developed by City's Development Services Department. The future actual detailed land use types may be fine-tuned or modified from the zoning layer; however, based on the knowledge of Development Services Department, the generated sewer flow will be within the anticipation by the zoning layer, and the zoning layer is sufficient for calculating sewer flow contribution by future service area for the master planning purpose.

The future sewer service areas and their development percentage and schedule were defined by information provided by City's Development Services Department. As shown in Figure 3-2 and summarized in Table 3-2, the 2040 and Intermediate service areas are determined as follows.

- Intermediate Scenario: In addition to the existing service area, the growth within the Intermediate Scenario was assumed based on the following three sources:
  - The areas defined about 40 tentative tract maps and specific plans provided by the City's Development Services Department will be 100 percent developed during the Intermediate Scenario. These areas are shown in dark blue in Figure 3-2.
  - As shown in dark blue in Figure 3-2, portion of the La Entrada development is assumed to happen in the Intermediate Scenario, based on the phasing map of the La Entrada master plan.
  - The rest of the vacant land or the developed land on septic system will be served by the City's sewer system in the Intermediate Scenario at a certain percentage determined by the City. As shown in Figure 3-2, 50 percent of the green areas and 25 percent of the light blue areas are anticipated to be served by the City's sewer system by Intermediate scenario.
- 2040 scenario: In addition to the existing and intermediate service area, the growth within the 2040 Scenario was assumed based on the following two sources:
  - As shown in purple in Figure 3-2, portion of the La Entrada development is assumed to happen in the 2040 Scenario, based on the phasing map of the La Entrada master plan.
  - The rest of the vacant land or the developed land on septic system will be served by the City's sewer system in the 2040 Scenario at a certain percentage determined by the City. As shown in Figure 3-2, 100 percent of the green areas and 50 percent of the light blue areas are anticipated to be served by the City's sewer system by 2040 scenario.

The acreage and percentage of development/service in each modeling scenario within the study area (total of 29,840 acre) is summarized in Table 3-2.

**Table 3-2 Sewer Service Area in Existing and Future Scenarios**

| Scenario | 100% Developed/Served (Acre) | 50% Developed/Served (Acre) | 25% Developed/Served (Acre) | Total Service Area (Acre) |
|---|---|---|---|---|
| Existing | 2,677 | | | 2,677 |
| Intermediate | 6,401 | 4,165 | 17,776 | 12,928 |
| 2040 | 12,061 | 17,776 | | 20,949 |

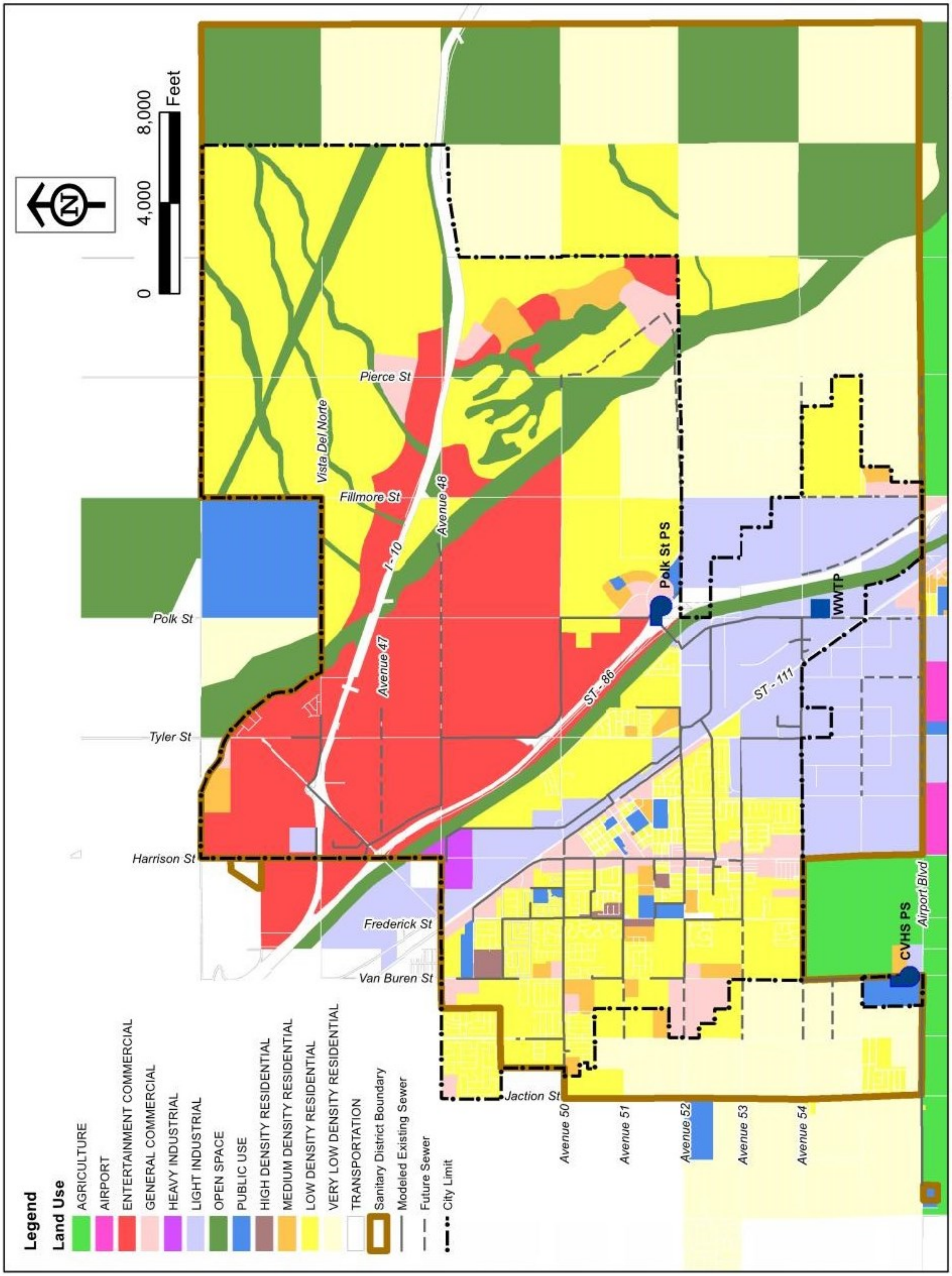


Figure 3-1
Land Use

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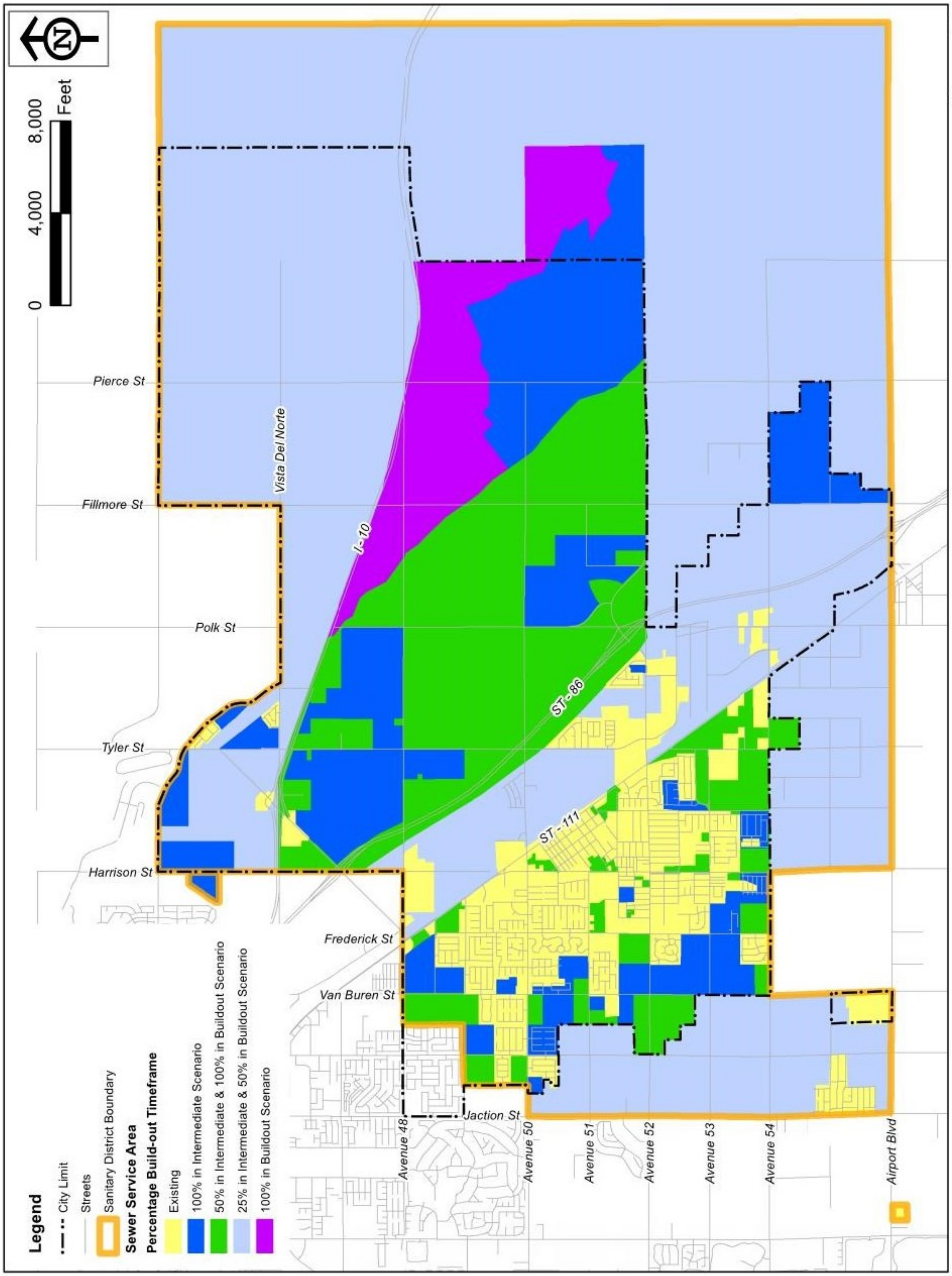


**Figure 3-2**
**Future Service Area**

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# Section 4
# Model Calibration and Flow Projections

## 4.1 Introduction

This section describes the wastewater flow components, including base wastewater flow, groundwater infiltration, and rainfall-dependent infiltration/inflow. It then discusses the methodology used in developing the base flow projections, the development of unit flow rates and diurnal curves for the major land use types, as well as the model calibration for dry weather conditions. The flow factors and parameters derived from these analyses were then used to generate the wastewater flow projections.

## 4.2 Wastewater Flow Components

Wastewater flow can be broken up into three main flow components as described below. Figure 4-1 is an illustration of typical wastewater flow component patterns.

*Base Wastewater Flow* (BWWF) is domestic wastewater from residential, commercial, and institutional (e.g., schools, churches, hospitals) and industrial sources. It is affected by the population and land uses in an area, and varies throughout the day in response to personal habits and business operations. Base wastewater flow is the primary component of dry weather flow.

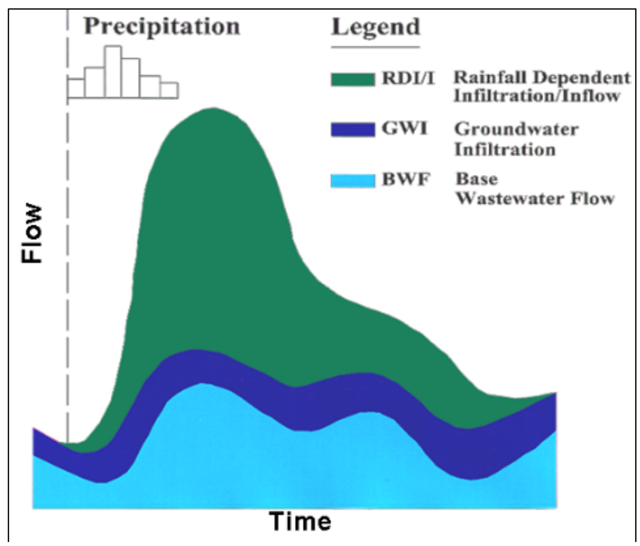


**Figure 4-1**
**Generic Schematic of Wastewater Flow Components**

*Groundwater Infiltration (GWI)* is defined as groundwater entering the collection system that is not related to a specific rain event. GWI occurs when groundwater is above the sewer pipe invert and infiltrates through defective pipes, pipe joints, and manhole walls. The magnitude of the groundwater infiltration depends on the depth of the groundwater table above the pipelines, the percentage of the system that is submerged and the physical condition of the system. GWI is seasonal and typically declines during dry weather periods as groundwater levels drop. Based on the USGS groundwater active well monitoring located northwest of Salton Sea, the groundwater table is more than 120 feet deep during the past 20 years. Current static water levels in production wells are as low as 38 feet; in addition, the region has experienced significant drought conditions resulting in low groundwater levels. Historic data indicates that static water levels may be as high as 10 feet below ground surface, mostly in the northern area of the City; also, the area includes an aquitard that creates a semi-perched aquifer. In addition, the Tom Levy Recharge is located southwest of the City. CVWD manages an aggressive recharge program that manages static water levels in the City. Therefore, levels are likely to rise during wet years. The GWI flow component was not considered in this master plan.

*Rainfall-Dependent Infiltration/Inflow (RDII)* is stormwater that enters the collection and trunk sewer system in direct response to the intensity and duration of individual rainfall events. RDII is comprised of storm water inflow and rainfall-dependent infiltration. Stormwater inflow reaches the collection system by direct connections rather than by first percolating through the soil. Stormwater inflow sources may include roof downspouts illegally connected to the sanitary sewers, yard and area drains, holes in manhole covers, or cross-connections with storm drains or catch basins. Rainfall-dependent infiltration includes all other rainfall-dependent flow that enters the system, including stormwater that enters defective pipes, pipe joints, and manhole walls after percolating through the soil. Since the City receives very low annual rainfall about average of 3 inches, the wet weather flow component was not included in this Master Plan.

## 4.3 Model Calibration

The purpose of model calibration is to confirm that the model accurately represents the flows and system hydraulics under existing conditions. The calibrated model can then be used to predict the hydraulic conditions under future land use scenarios using future flow projections. The hydraulic model, InfoSewer, has computational capabilities that account for flow variation during the entire cycle (i.e., it simulates flows and depths as they change over time); including high and low daily flows. Therefore, it is important to consider the shape of the diurnal flow hydrograph as well as the average flow produced by the unit flow factors.

A short-term flow monitoring program was conducted to develop the flow factors and parameters that define the wastewater flow characteristics in the City of Coachella's sewer system. To calibrate the hydraulic model, the appropriate unit flow factors and diurnal patterns for land use categories need to be determined. This process involved using the flow monitoring data and varying the unit flow factors within reasonable ranges to verify that the computed flows are consistent with the monitored flows, and developing diurnal patterns for land use categories.

### 4.3.1 Dry Weather Calibration

The appropriate unit flow rates to apply to the land use categories need to be determined through the model calibration process. This process involved varying the unit flow factors within reasonable ranges to verify the computed flows are consistent with the monitored flows and developing diurnal patterns for land use categories.

#### 4.3.1.1 Dry Weather Flow Monitoring

The purpose of the dry weather flow monitoring is to develop unit flow factors and diurnal patterns for residential and non-residential land uses, and to calibrate the model. CDM Smith used the existing land use information, aerial photography, and sewer system layout provided by the City to recommend seven (7) flow-monitoring locations across the City. V&A collected the data of flow, flow depth and velocity from June 6 to June 19, 2012. During the flow monitoring period, the lift station near 52nd Avenue and Polk Street was taken out of operation for maintenance and the flow to the lift station was diverted along Ave 50 to the east side of the system. As a result, Meter 7 at the lift station did not measure any flow and was excluded from this analysis. Flow monitoring locations are summarized in Table 4-1 and shown on Figure 4-2.

**Table 4-1 Dry Weather Flow Monitoring Locations**

| Flow Meter # | Installation Manhole ID | Location | Pipe Dia. (in) |
|---|---|---|---|
| 1 | E18-002 | Frederick Street, North of 50th Avenue | 12 |
| 2 | E18-013 | 50th Avenue and Mazatlan | 12 |
| 3 | F17-006 | Harrison Street, South of Park Lane | 15 |
| 4 | F18-026 | 85-711 Peter Rabbit Lane | 10 |
| 5 | I21-006 | Industrial Way, West of Polk Street | 18 |
| 6 | I23-020 | Treatment Plant | 54 |
| 7 | I20-002 | Polk Street | 24 |

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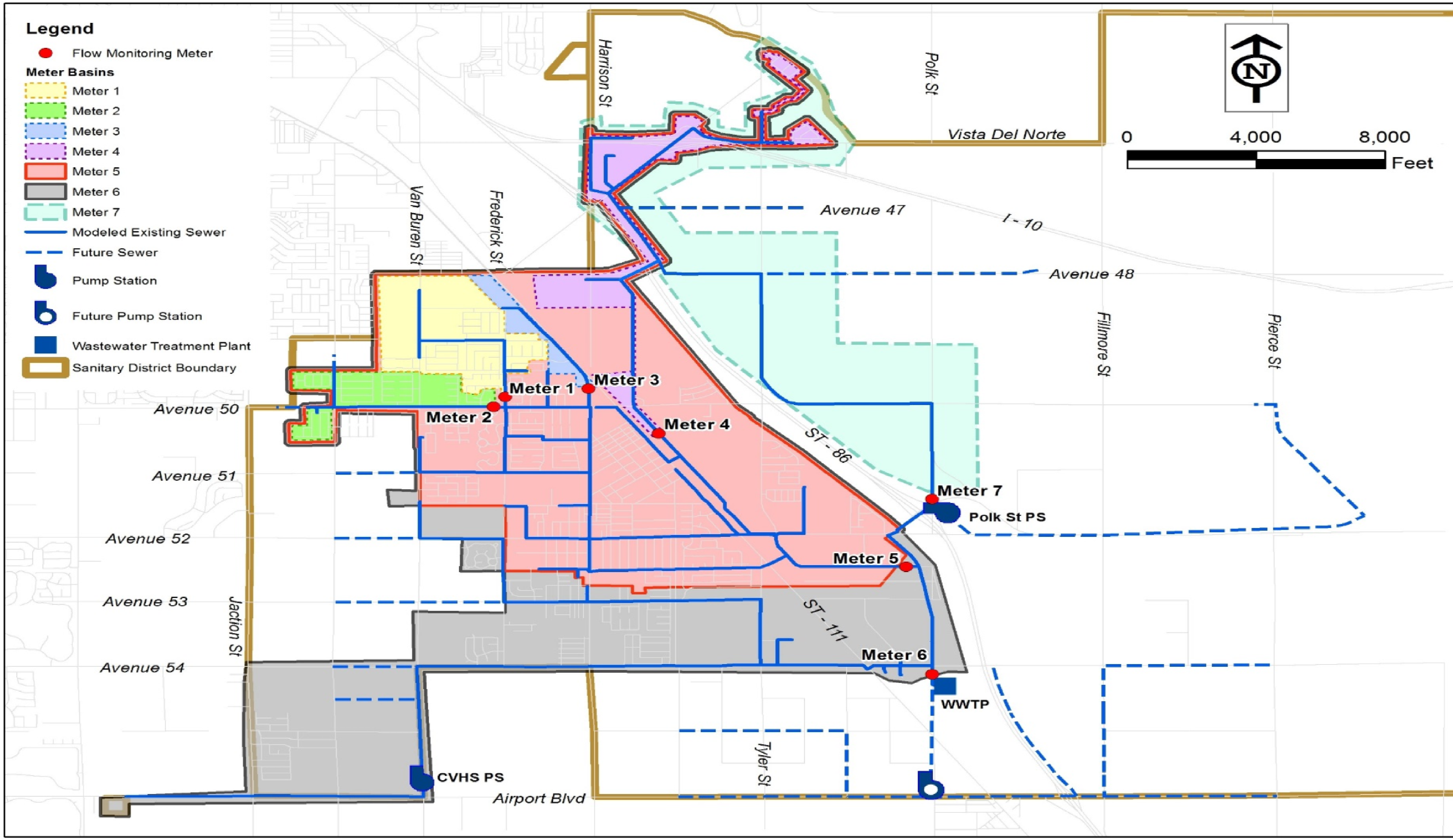


**Figure 4-2**
**Flow Monitoring Locations and Meter Basins**

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### 4.3.1.2 Flow Rate Development

A land use-based approach that is sensitive to land use categories was used to calculate sewer loads by sewershed and allocate to the model. This approach utilized the current land use categories as defined on the City's General Plan map, and developed average unit flow factors.

The unit flow rates were estimated for the various land use categories based on the flow monitoring data. The total acreage per land use was determined from the City's General Plan. The aerial photography and the sewer system layout provided by the City were used to identify the developed and sewered areas by excluding the vacant and un-sewered areas. As a result, the estimated existing developed acreage for each land use type was used to determine the unit flow rates shown in Table 4-2.

The sewer loads by sewershed were then calculated by multiplying the acreage of each land use type by the corresponding unit flow factor and summed up to estimate the total dry weather flow generated. The units of these factors are in gallons per day per acre (gpdac). The total flow for the sewershed was then loaded into the modeled trunk sewers.

**Table 4-2 – Average Dry Weather Flow Unit Flow Factors**

| Land Use Category | Unit Flow Rate (gpd/ac) | Metered Areas (acre) | | | | | |
|---|---|---|---|---|---|---|---|
| | | FM1 | FM2 | FM3 | FM4 | FM5 | FM6 |
| Agriculture (1du/40ac) | 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Entertainment Commercial | 600 | 0.00 | 0.00 | 0.00 | 65.34 | 56.11 | 56.20 |
| General Commercial | 600 | 0.43 | 7.83 | 35.94 | 0.00 | 175.51 | 211.97 |
| Heavy Industrial | 800 | 0.00 | 0.00 | 0.00 | 61.10 | 62.83 | 62.83 |
| High Density Residential (0-20 Du/Ac) | 2400 | 2.93 | 0.00 | 0.00 | 0.00 | 25.58 | 25.53 |
| Light Industrial | 400 | 0.00 | 0.00 | 0.00 | 34.93 | 164.46 | 294.42 |
| Low Density Residential (0-6 Du/Ac) | 1000 | 208.6 | 152.6 | 0.08 | 0.00 | 1449.9 | 1891.0 |
| Medium Density Residential (0-10 Du/Ac) | 1800 | 0.00 | 1.52 | 0.00 | 0.00 | 85.07 | 85.15 |
| Open Space | 0 | 0.00 | 0.00 | 0.00 | 0.18 | 0.93 | 0.36 |
| Public Use | 1000 | 0.00 | 0.00 | 0.00 | 0.00 | 84.29 | 149.6 |
| Very Low Density Residential (0-2 Du/Ac) | 300 | 38.6 | 0.77 | 0.00 | 0.00 | 0.76 | 23.95 |

### 4.3.1.3 Diurnal Curves

Sewer flow has a diurnal pattern that varies with land use categories throughout the day. Typically, a residential diurnal pattern has two peaks with the more pronounced peak occurring in the morning as people rise and take showers and the less pronounced peak in the evening. The low flows usually occur at night and in the early morning. Commercial developments typically do not have significant peaks and produce flow at a relatively constant rate throughout the business hours of the day. For the residential communities in City of Coachella, the flow monitoring data indicated that the higher peaking pattern is occurring late in the evening hours rather than the morning hours.

The diurnal flow patterns used in the City of Coachella model are based upon the flow monitoring data. Specific flow meters were located to measure flow from areas with primarily a single land use type. For these meters, all weekdays with complete data were averaged and normalized to produce a diurnal curve. This process was followed for the residential and commercial diurnal curves. Industrial areas are too small to measure separately and thus an exact determination of the industrial diurnal curve was not feasible. However, a diurnal pattern was estimated from Flow Meter 4, which captures most of the industrial areas.

For calibration purposes, only and since Meter 4 captures industrial, commercial and residential flows; a "Mixed" diurnal pattern was developed to represent the various flows. The pattern itself raises questions about the fluctuation of the flows every few hours as if some of the flows are pumped. However, this might be attributed to the entertainment commercial schedule of operation later in the day. Once developed, the initial diurnal curves were adjusted as part of the iterative calibration process to assure the model matched the current conditions as accurately as possible. The finalized diurnal curves are displayed in Figure 4-3.

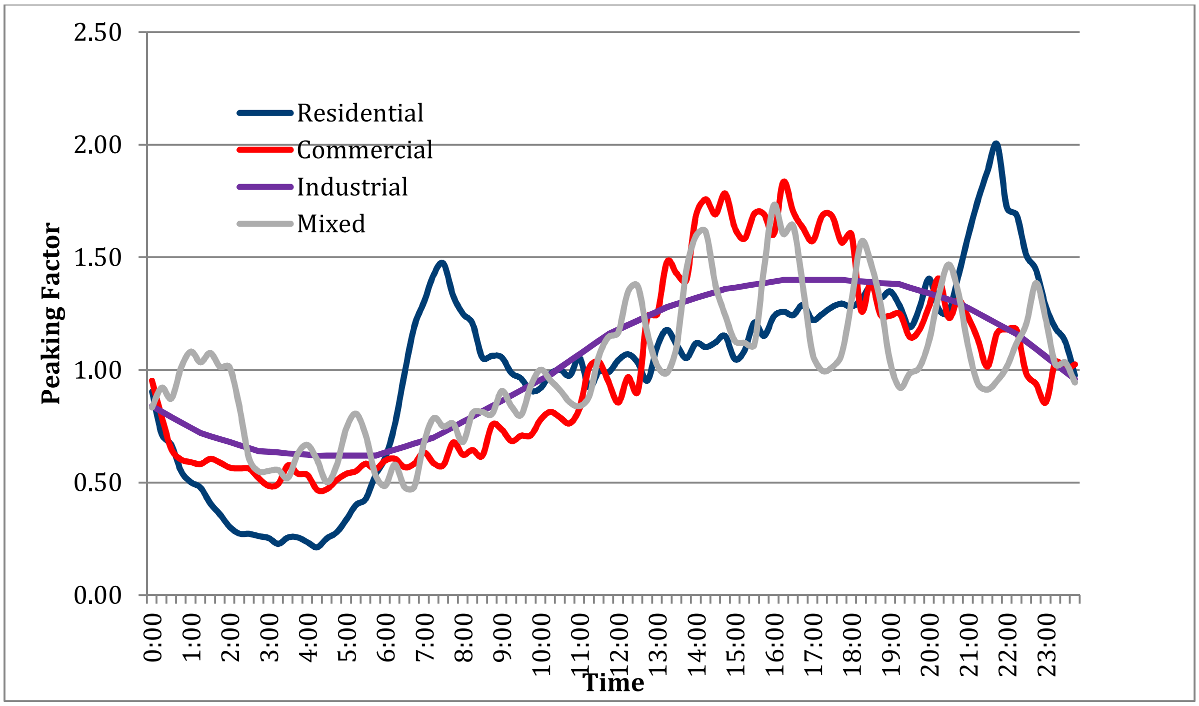


**Figure 4-3**
**Diurnal Curves**

### 4.3.1.4 Dry Weather Calibration Results

Using the unit flow rates and diurnal curves discussed above, Table 4-3 compares the model results with the flow monitoring data for average and for peak flow. Typically, model calibration efforts aim to be at least within 20 percent of the flow monitoring data and ideally within 10 percent or less.

Four out of six meters (Meter 7 was excluded, no flows) are within 10 percent of the flow data indicating a close and reasonable match with current conditions. Meter 1 seems to be in error. It measured average flow of 0.37 mgd from mainly 215 acres of residential areas. The calculated flow rate per acre is 1745 gpd/ac compared to 1000 gpd/ac for Meter 2 with similar land use characteristics. In addition, the northeast corner of Van Buren and Avenue 49, Las Flores Phase II, was never completed but the sewer lines were installed on the tract.

Flow Meter 6 near the wastewater treatment plant also seems to have low readings. Meter 6 measured similar flows as Meter 5. However, Meter 6 collects flows from a larger area including the southwest area along Airport Boulevard and 54th Avenue and the south central area through a 12-inch sewer line that collects flow from 53rd Avenue. In addition, it was noted that the field crew had some issues with the installation of the meter due to odors (high $H_2S$) and that may have affected the accuracy of the measurements.

Despite the fact that Meter 1 and Meter 6 presented some imprecision, Meter 5, which is near the wastewater treatment plant, captured the majority of the flow within the City and the simulated model results were within 2 percent of the measured flow. Meter 2 (residential), Meter 3 (commercial) and Meter 4 (Mixed land use) were all within 10 percent, which indicates that the unit flow factors are reasonably accurate and the model is calibrated and can reliably be used for the existing and future system analyses. Graphs of the dry-weather flow calibration results can be found in Appendix A.

**Table 4-3- Model Results Compared with Flow Monitoring Data**

| Flow Meter # | Average Flow | | | Peak Flow | | |
|---|---|---|---|---|---|---|
| | Metered Weekday Flow (mgd) | Modelled Flow (mgd) | % Difference | Metered Weekday Flow (mgd) | Modeled Flow (mgd) | % Difference |
| 1 | 0.366 | 0.148 | -60% | 0.635 | 0.272 | -57% |
| 2 | 0.155 | 0.160 | 3% | 0.283 | 0.287 | 1% |
| 3 | 0.022 | 0.023 | 5% | 0.044 | 0.040 | -9% |
| 4 | 0.094 | 0.097 | 3% | 0.163 | 0.158 | -3% |
| 5 | 1.980 | 1.934 | -2% | 2.886 | 2.858 | -1% |
| 6 | 2.000 | 2.590 | -30% | 2.820 | 3.790 | 34% |
| 7(1) | - | - | - | - | - | - |

1) Flow Meter 7 had no measurements due to diverting flows upstream for pump station maintenance.

Overall, the calibration results show that the model provides a good representation of the existing system.

### 4.3.2 Wet Weather Flows

Wet weather flow component usually is determined from analyzing field measured wet weather flow data collected during a rainy season. This wet weather flow analysis determines the magnitude of the RDII component which is in direct response to the rain and reflective of the physical sewer conditions. The analysis typically identifies which basins are contributing more infiltration and inflow to the system and determine wet weather basin-specific parameters to estimate the peak wet weather flow for evaluating system capacity. The 2012-2013 and 2013-2014 rainy seasons were very dry and thus no wet weather flow monitoring could be performed. For this analysis and based on the hydraulic performance criteria, it was assumed that the maximum capacity for 12-inch diameter pipes and smaller is d/D=0.5 and that for 15-inch diameter pipes and greater is d/D =0.75, the remaining capacity reserved for wet weather flows.

## 4.4 Existing and Future Flows

This section describes the development of the existing and future flows for use in evaluating the existing system and assessing the necessary improvements for future systems. The land-use-based unit flow factors were applied to the land use types delineated by the City's zoning maps. The service areas identified for Existing, Intermediate, and 2040 Scenarios were overlaid on the zoning layer to allocate base wastewater flows to each load point. To keep track of the flows by land use type, the flows were loaded to nine load fields in the model. Table 4-4 below shows the flow contributing land use zoning code, Land use type, unit flow factor, and load field in the model.

**Table 4-4 Land Use Type with Model Flow Load Field**

| Land Use Code | Land Use Description | Unit Flow Rate (gpd/acre) | Load Field |
|---|---|---|---|
| RVL | Very Low Density Residential (0-2 du/ac) | 300 | Load1 |
| RL | Low Density Residential (0-6 du/ac) | 1,000 | Load2 |
| RM | Medium Density Residential (0-10 du/ac) | 1,800 | Load3 |
| RH | High Density Residential (0-20 du/ac) | 2,400 | Load4 |
| CG | General Commercial | 600 | Load5 |
| CE | Entertainment Commercial | 600 | Load6 |
| IL | Light Industrial | 400 | Load7 |
| IH | Heavy Industrial | 1,000 | Load8 |
| P | Public Use | 1,600 | Load9 |

The Load Allocator module provided in the modeling software InfoSewer software provides an efficient tool to determine accurate and representative manhole loads and the spatial distribution of loads throughout the model network. It calculates dry weather loads by "unit flow rate X acreage", which was achieved by spatially intersecting multiple polygon layers:

- Land use polygon (Figure 3-1 Land Use) determines which unit flow rate should be applied based on land use types.
- Sewershed polygon (Figure 2-4 Sewersheds and Load Points) spatially correlates sewersheds to flow loading manholes.
- Service areas (Figure 3-2 Future Service Areas) provide sewer service areas of each scenario.

The resulting manhole loads were checked against the aerial photo to verify the land use types, dry weather flow load fields, and vacant area that do not contribute any existing flows. Table 4-5 summarizes the existing and future system flows.

**Table 4-5 Summary of Flow by Timeframe by Land Use Type**

| Land Use Type | Existing (cfs) | Intermediate (cfs) | 2040 (cfs) |
|---|---|---|---|
| Very Low Density Residential | 0.01 | 0.76 | 1.51 |
| Low Density Residential | 2.86 | 7.60 | 11.03 |
| Medium Density Residential | 0.21 | 1.07 | 1.24 |
| High Density Residential | 0.08 | 0.23 | 0.23 |
| General Commercial | 0.19 | 0.55 | 0.77 |
| Entertainment Commercial | 0.06 | 2.15 | 3.32 |
| Light Industrial | 0.18 | 0.69 | 1.19 |
| Heavy Industrial | 0.08 | 0.09 | 0.09 |
| Public Use | 0.23 | 0.30 | 0.32 |
| **Total** | 3.91 | 13.43 | 19.71 |

# Section 5

# Hydraulic Criteria

## 5.1 Introduction

This section discusses the City of Coachella's sewer evaluation and design criteria, which will be used to evaluate the system performance under the Existing, Intermediate, and 2040 flow conditions.

## 5.2 Hydraulic Criteria

The hydraulic criteria were established to define the method by which gravity sewers, force mains, and pump stations are evaluated using the hydraulic model. Based on City's review and input, the proposed hydraulic criteria were used for the sewer system analyses to ensure that the system has adequate capacity to accommodate projected flows and will operate without problems under anticipated flow conditions. Table 5-1 summarizes the hydraulic criteria for the analyses of the City's sewer system. In Section 6, the hydraulic model results are compared to these criteria to identify capacity deficiencies and improvements.

**Table 5-1 Summary of Hydraulic Criteria**

| Elements | Recommended Value |
|---|---|
| Manning's 'n' Factor | 0.013 for all pipe materials |
| Minimum Pipe Size | 8 -inches |
| Design Criteria for Gravity Sewer<br>Maximum Flow Depth | Under peak design dry weather flow conditions:<br>d/D = 0.50 for pipes 12-inches in diameter and smaller<br>d/D = 0.75 for pipes larger than 12-inches in diameter<br>The remaining capacity is assumed to be available for wet weather flows<br><br>For evaluating and prioritizing whether existing pipes require improvement, surcharge is allowed as long as the hydraulic grade line (HGL) remains at least 5 feet below the rim elevation under peak design flow conditions. This criterion is only for evaluating whether existing pipes require replacement or relief.<br><br>All new pipe improvements and replacement projects are sized to convey the peak 2040 design flow at full pipe capacity (d/D=1) without any surcharge. |
| Velocity Criteria for Gravity Lines | Minimum: 2.5 ft/sec at half-full pipe.<br>Maximum: 8 ft/sec |
| Minimum Slopes for Gravity Lines | 8-inch diameter: 0.0040    15-inch diameter: 0.0017<br>10-inch diameter: 0.0030    18-inch diameter: 0.0014<br>12-inch diameter: 0.0024    21-inch diameter: 0.0011<br><br>For 24-inch diameter lines and larger other than minimum cleansing velocity (such as construction tolerances and potential ground subsidence) may govern the minimum slope selection. |
| Force Main Hydraulic Criteria | Maximum velocity: 6 ft/sec for new pipes, 8 ft/sec for existing pipes<br>Minimum velocity : 3.5 ft/sec<br>Hazen-Williams Head Loss Coefficient: C=100 -120 depending on pipe size, material, and age. |
| Pump Station Capacity | Firm Capacity, with largest pump as a standby unit for peak design flows. |

Below is a discussion of each element including relevant standards, typical criteria used by other agencies, and recommended values. The criteria also consider generally accepted industry standards, based on experience with similar projects.

### 5.2.1 Manning's "n" Factor

Manning's roughness coefficient 'n' is the friction factor utilized in the Manning's Equation for gravity flow to describe the roughness of a particular pipe material or condition. Manning's 'n' value generally ranges from 0.009 for plastic pipe to 0.016 for unlined concrete pipe with vitrified clay pipe between the two values.

For the hydraulic model and master plan, and for design purposes, it is recommended that a Manning's 'n' value of 0.013 be used for all pipe materials. This design value is widely accepted in the industry and is a reasonably conservative value for planning purposes, since it accounts for aging and buildup of material inside pipes over time. Although the City is now using plastic pipe for its sewer system, there will be buildup of material inside the plastic pipes that will increase the Manning's 'n' coefficient over time.

### 5.2.2 Minimum Pipe Diameter for Gravity Sewers

Some agencies allow new 6-inch gravity sewers, and many agencies, including the City of Coachella have existing 6-inch pipes. However, a minimum sanitary sewer pipe size of 8-inches is generally accepted as the industry standard for maintenance purposes, and the City of Coachella has proposed an 8-inch minimum as its new design standard.

### 5.2.3 Maximum Allowable Flow Depth

The depth of flow in the pipe (d) relative to the pipe diameter (D) is a parameter typically used for evaluating capacity needs. For the master plan analysis, the following criteria are used:

- Under peak dry weather conditions, a d/D of 0.50 for pipes 12 inches in diameter and smaller, and d/D of 0.75 for pipes larger than 12 inches in diameter is recommended. The remaining available capacity is assumed adequate to convey peak wet weather flows. This lower (d/D) ratio is conservative and is used to prevent flow blockages in smaller pipes due to debris and avoid potential backup into connected service laterals. During more detailed evaluation and design of specific improvements, it should be confirmed that the anticipated peak wet weather can be conveyed with maximum d/D=1.0 (full pipe flow).
- In order to save costs, some agencies allow surcharging of large diameter gravity flow sewers under peak flows associated with infrequent (long return period) storm events.
- For evaluating and prioritizing whether existing pipes require improvement, surcharge is allowed as long as the HGL remains at least 5 feet below the rim elevation under peak design flow conditions. Maintaining at least 5 feet of freeboard to the ground above the modeled hydraulic gradient is deemed sufficient so that wastewater would not overflow onto the ground or backup into any home or business. Under surcharged conditions, the pipe flows at greater than full pipe flow and the HGL is above the top of the pipe (pressurized flow). This criterion is only for evaluating whether existing pipes require replacement. All new pipe improvements and replacement projects are sized to convey the 2040peak design flow without any surcharge.

### 5.2.4 Minimum Velocity/ Slope

A minimum velocity of 2.5 feet per second (ft/sec) is recommended for the master plan, which is consistent with the previous master plan; in certain cases, City staff can approve to use 2 ft/sec. For municipal wastewater with its associated grit and solids content, 2 ft/sec has been commonly used as the minimum design velocity at full or half-full pipe flow conditions. When the sewers are less than half-full, velocities will drop below 2 ft/sec, and some deposition of solids will occur. Re-suspension of solids occurs when the depth of sewage is greater than half-full, and the velocity increases above 2 ft/sec until a maximum velocity is reached at approximately 94 percent of full pipe depth. From 94 percent depth to full pipe, the velocity decreases back to 2 ft/sec.

Minimum design slopes must provide a minimum velocity of 2 ft/sec (2.5 ft/sec is preferred) for sewers between 8 and 18 inches in diameter and a minimum velocity between 2.5 and 3 ft/sec for sewers greater than 18 in diameters. The velocities are calculated with Manning's 'n' =0.013 and full pipe conditions.

### 5.2.5 Maximum Velocity

Maximum velocity is an issue often related to topography where steep slopes have to be accommodated in the design. Excessive velocities may cause abrasion of the pipe material and have a hydraulic impact on the receiving system. Typically, the maximum velocity criterion used by various agencies generally ranges from 8 to 15 ft/sec. For this master plan, it is recommended that a maximum velocity of 8 ft/sec be used for gravity sewers.

### 5.2.6 Force Mains – Hydraulic Criteria

Various agencies use different design criteria for minimum and maximum velocities in force mains. The maximum velocity in a force main is usually determined by balancing a number of factors. These factors include cost of the pipeline; cost of power usage (higher velocity results in higher head loss), and cost of pumps, motors, electrical equipment, and surge protection facilities.

For sewer force mains, the design flow rate is typically the peak wet weather flow at 2040, which occurs infrequently. Therefore, it is generally cost effective to set the maximum velocity under peak design conditions at a relatively high velocity, since the daily peak flow rate is typically much lower and typical velocities will be less. For this master plan, a maximum force main velocity of 6 ft/sec under peak design flow conditions is recommended for new pipes and a maximum velocity of 8 ft/sec for evaluating existing pipes.

Usually, small pump stations operate intermittently and the solids in the force mains can settle out during low flow periods as the wet well fills. To re-suspend the solids a minimum velocity of 3.5 ft/sec is recommended.

The Hazen-Williams formula will be used for calculating the friction head loss of force mains. The Hazen-Williams roughness coefficient, C, varies with pipe material, velocity, size, and age. For this master plan, a roughness coefficient of C = 100 - 130 is proposed depending on pipe sizes and materials.

### 5.2.7 Pump Station Capacity

Pump stations should have firm capacity that matches or exceeds the peak hourly flows for current and future conditions. Firm capacity is defined as the capacity with the largest pump as a standby unit.

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# Section 6

# Sewer System Analysis

## 6.1 Introduction

This section summarizes the hydraulic model results under dry weather conditions for Existing, Intermediate, and 2040 scenarios. The model results are compared with the hydraulic criteria presented in Section 5 to evaluate the sewer system's performance and identify the capacity deficiencies to meet existing and future needs.

## 6.2 Hydraulic Model Results and Capacity Evaluation

Based upon the hydraulic criteria and modeling protocol, the collection system was evaluated under Existing, Intermediate, and 2040 Scenarios for the capacities of gravity sewers, force mains, and pump stations. Under each scenario, the collection system was analyzed for peak dry weather conditions using the d/D criteria that allows for wet weather flows using the remaining available capacity.

### 6.2.1 Existing Gravity Sewer Capacity

Under Existing Scenario, the model identified a total of 3.6 miles of gravity sewers that exceeded the design criteria of d/D ratio. The locations of these pipes are shown on Figure 6-1. The hydraulic model evaluates the computed flow against the design capacity criteria of d/D. However, when the pipe is full the reported flow depth in the pipe is 1.0. In order to assess how close the hydraulic grade line is to the surface, the model results were examined at each location where the d/D criteria were exceeded by creating and viewing a hydraulic grade line profile. The hydraulic grade line profile for the existing conditions are shown in Appendix B-I

Based on the model results, the HGL and the level of surcharge were used as criteria to determine the severity of the hydraulic conditions taking into account the depth of sewer pipes and available "freeboard". "Freeboard" is the depth of the HGL relative the ground surface. The higher the HGL to the ground surface, the higher the priority.

Below are two HGL profile examples; one shows significant surcharge where the maximum HGL is about 4 feet above the crown of the pipe and one where the HGL is within the pipe, but the d/D criterion was exceeded.

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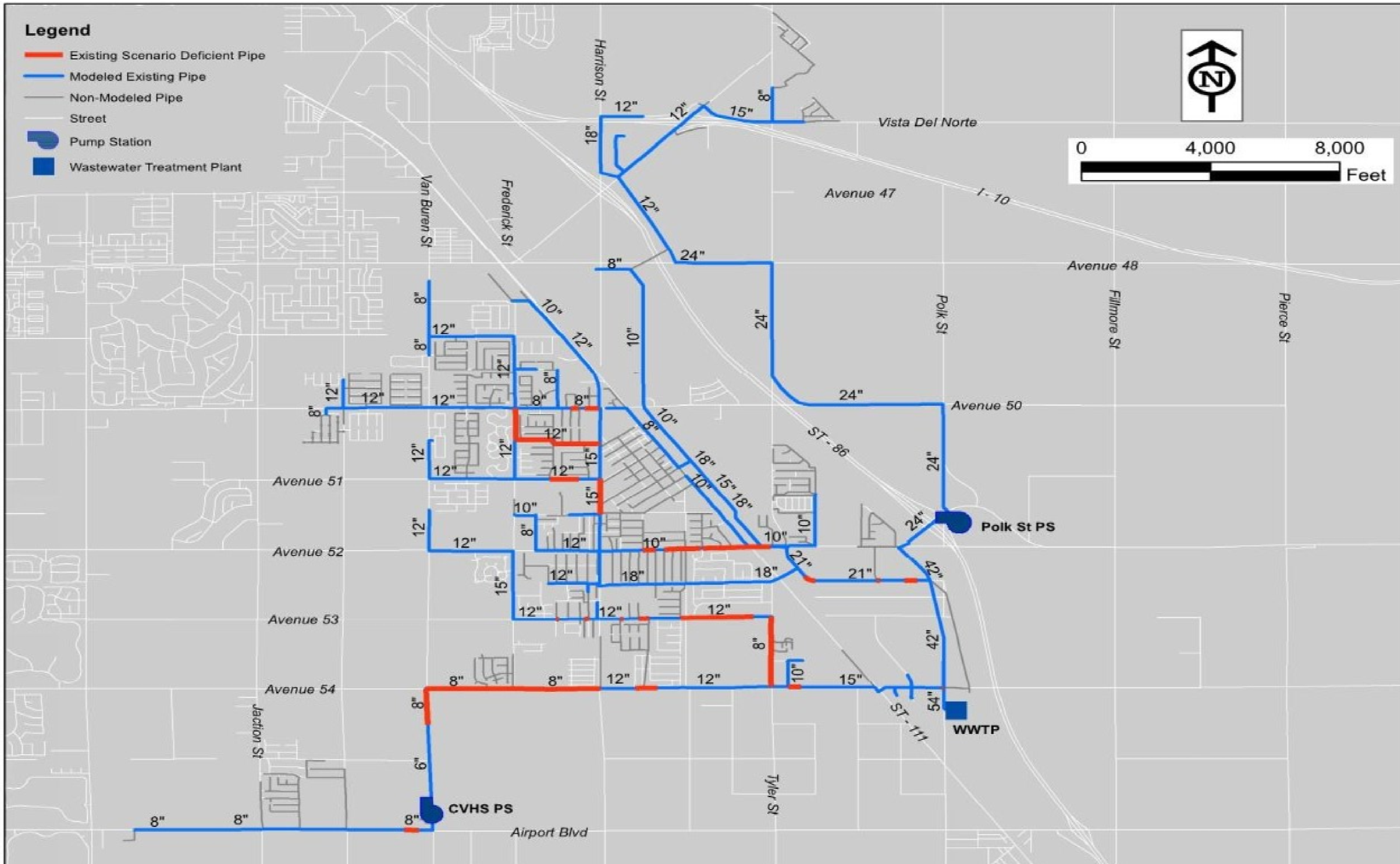


Figure 6-1
Existing Scenario Deficient Pipes

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### 6.2.1.1 Example of Significant Surcharge

The HGL profile in Figure 6-2 shows significant surcharge as predicted by the hydraulic modeling under peak dry weather flow conditions. These pipe segments are 8-inches diameters, located in Tyler Street, are surcharging due to lack of capacity where the HGL slope is steeper than the physical pipe slope. These pipe segments are ranked as a high priority for improvements.

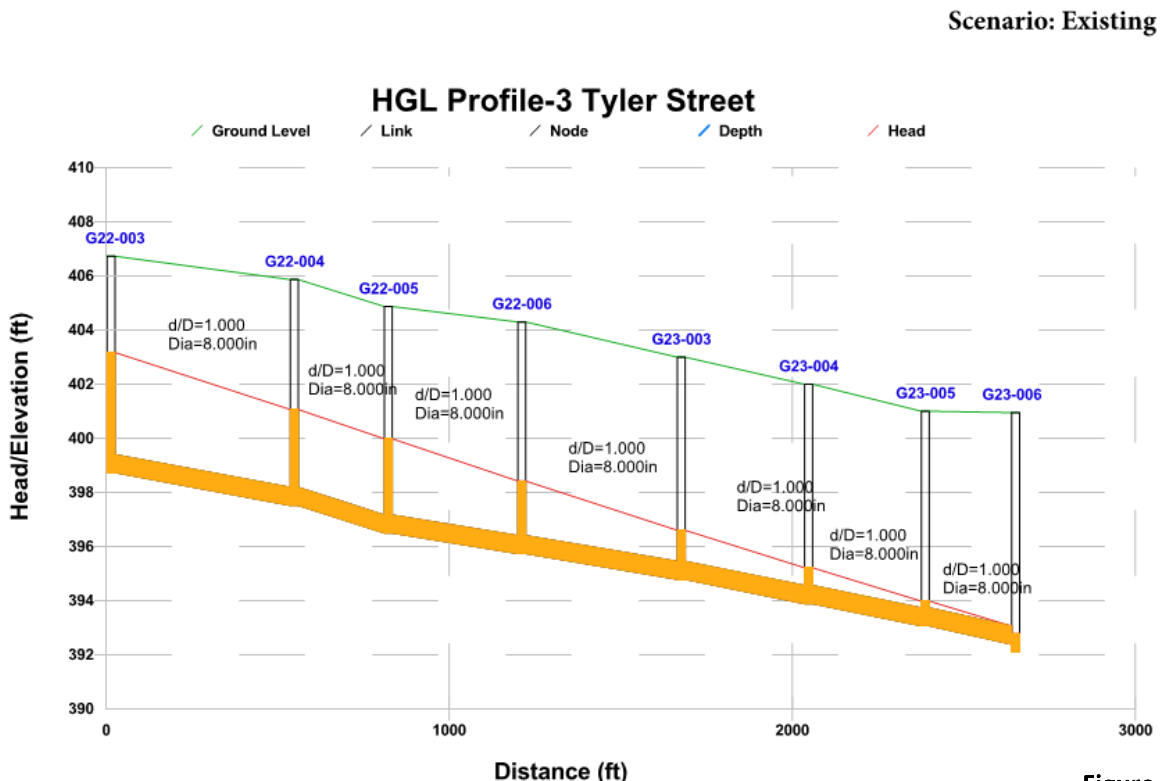


**Figure 6-2**
**Existing Scenario HGL Significant Surcharge**

### 6.2.1.2 Example of d/D Criteria Exceeded Within the Pipe

The HGL profile in Figure 6-3 shows d/D criteria is being exceeded as predicted by the hydraulic model under peak dry weather flow conditions where the hydraulic grade line is below the crown of the pipe. These pipe segments are 15-inches diameters, located in Harrison Street. The pipe segments between manhole F20-003 and manhole F19-007 show a d/D ration above 0.75, which is the maximum for 15-inch diameters and larger. These pipes will be ranked as a lower priority for improvements.

Scenario: Existing

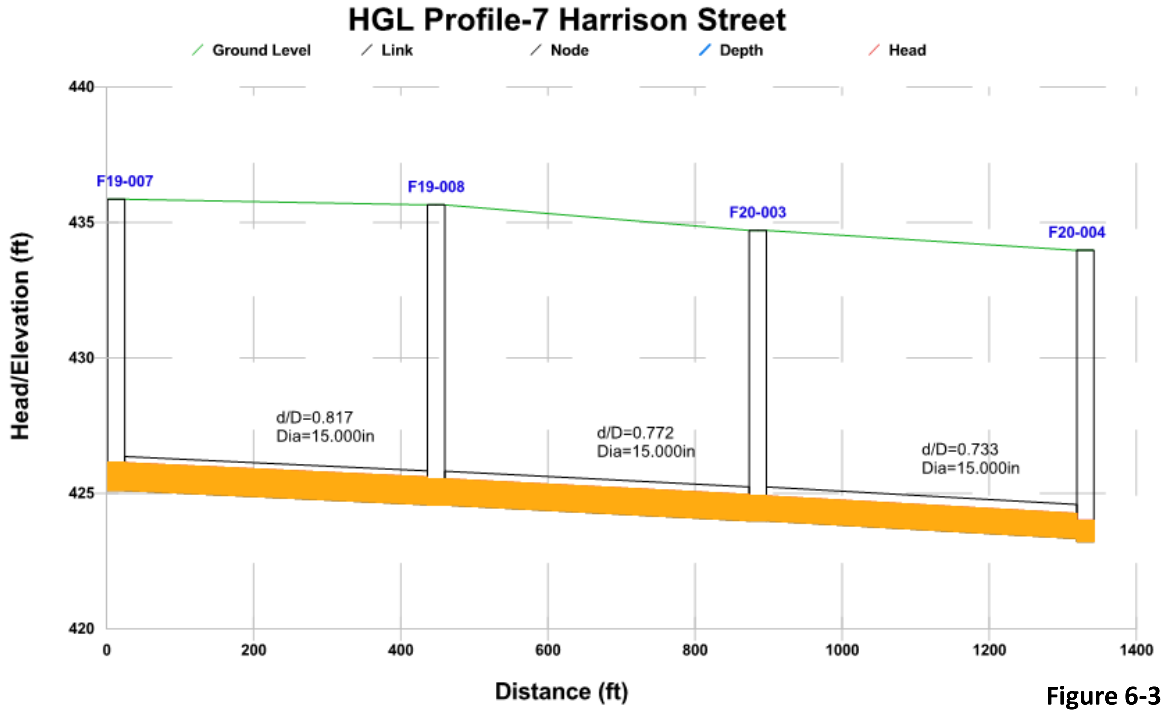


**Figure 6-3**
**Existing Scenario HGL within the Pipe**

Figure 6-4 shows the deficient lengths of pipes for the existing system as a percentage of the modeled system as well as the entire system.

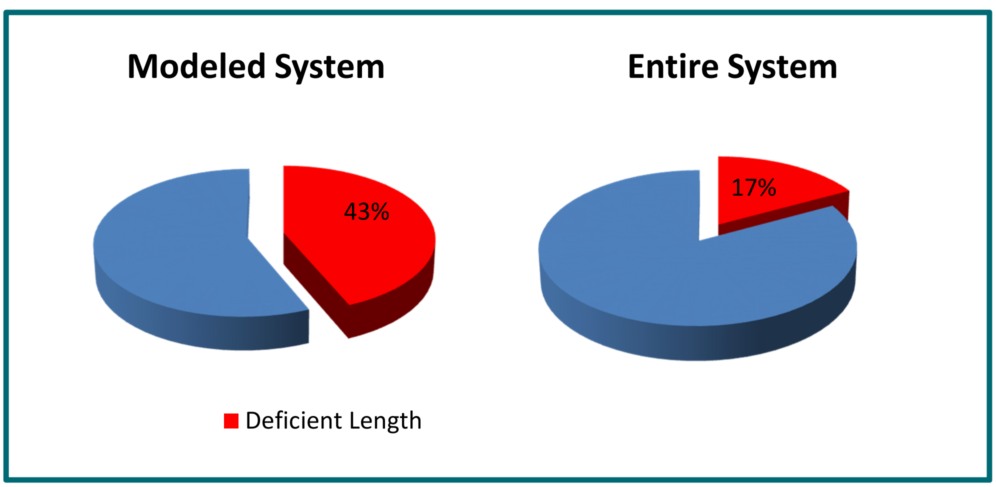


**Figure 6-4**
**Percentage of Modelled Deficient Gravity Sewer in Existing Scenario**

## 6.2.2 Intermediate Gravity Sewer Capacity

The analysis of the Intermediate Scenario was performed with the improvements identified under existing conditions are in place. The model results showed that 7.8 miles of gravity sewers exceeded the design criteria of d/D ratio under peak dry weather conditions. The locations of these pipes are shown on Figure 6-5. The hydraulic grade line profile for the intermediate conditions are shown in Appendix B-II.

### 6.2.2.1 Example of Significant Surcharge

The HGL profile in Figure 6-5 shows significant surcharge as predicted by the hydraulic modeling under Intermediate peak dry weather flow conditions. These pipe segments ranging from 12- to 15-inch in diameter, located in Avenue 54, are surcharging due to lack of capacity where the HGL slope is steeper than the physical pipe slope. These pipe segments are ranked as a high priority for improvements.

**Scenario: Intermediate**

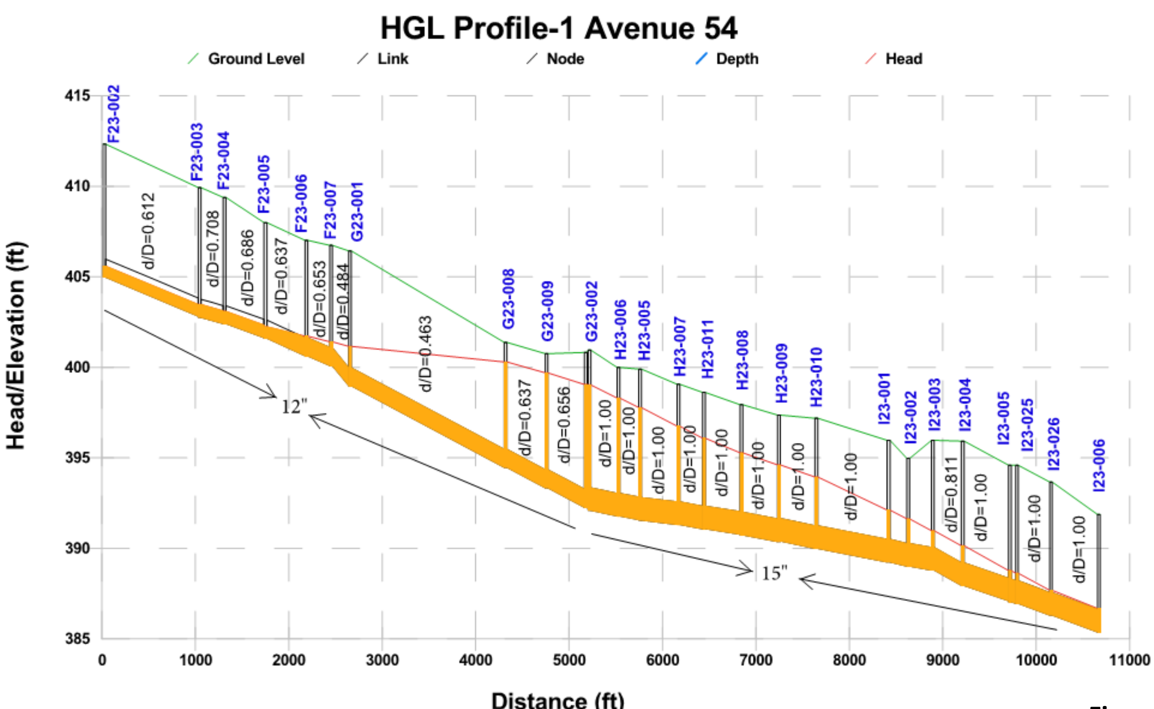


**Figure 6-5**
**Intermediate Scenario HGL Significant Surcharge**

### 6.2.2.2 Example of d/D Criteria Exceeded Within the Pipe

The HGL profile in Figure 6-7 shows d/D criteria is being exceeded as predicted by the hydraulic model under peak dry weather flow conditions where the hydraulic grade line is below the crown of the pipe. These pipe segments are 15 inches diameters and located in Harrison Street. The pipe segments between MANHOLE F20-003 and MANHOLE F19-007 show a d/D ration above 0.75, which is the maximum for 15-inch diameters and larger. These pipes will be ranked as a lower priority for improvements.

**Scenario: Intermediate**

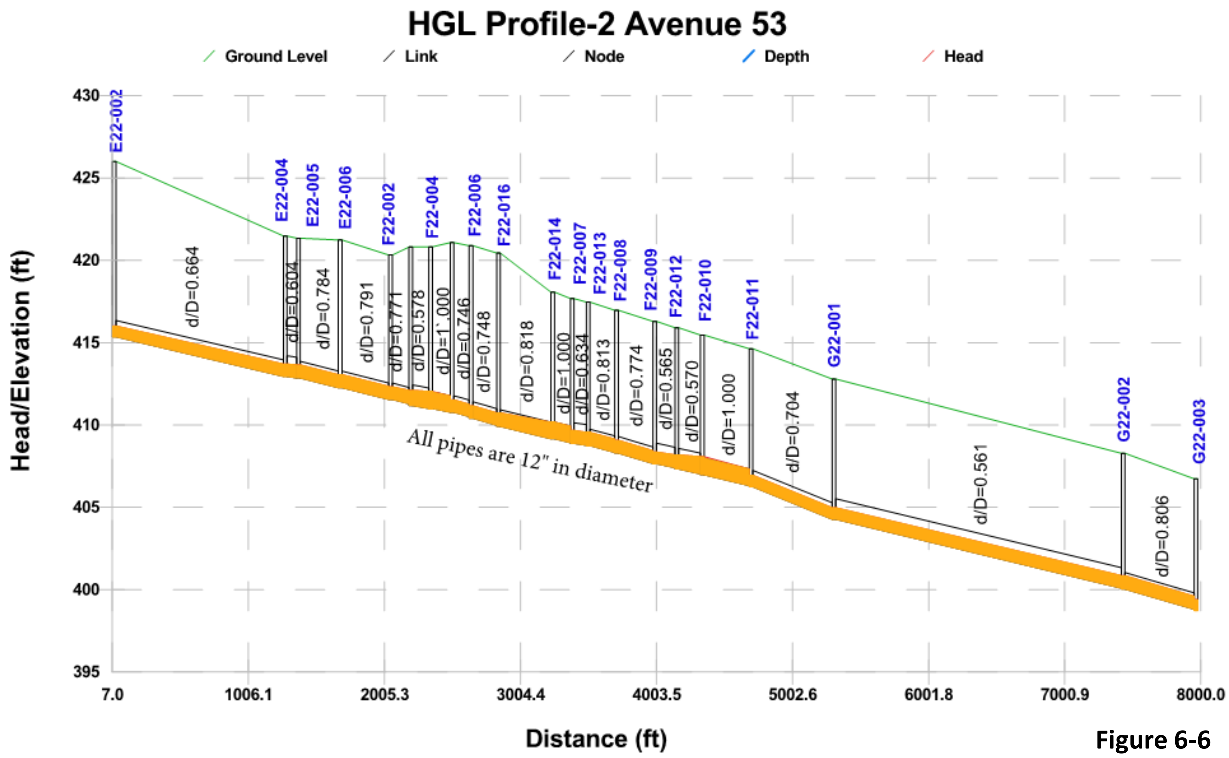


**Figure 6-6**
**Intermediate Scenario HGL within Pipe**

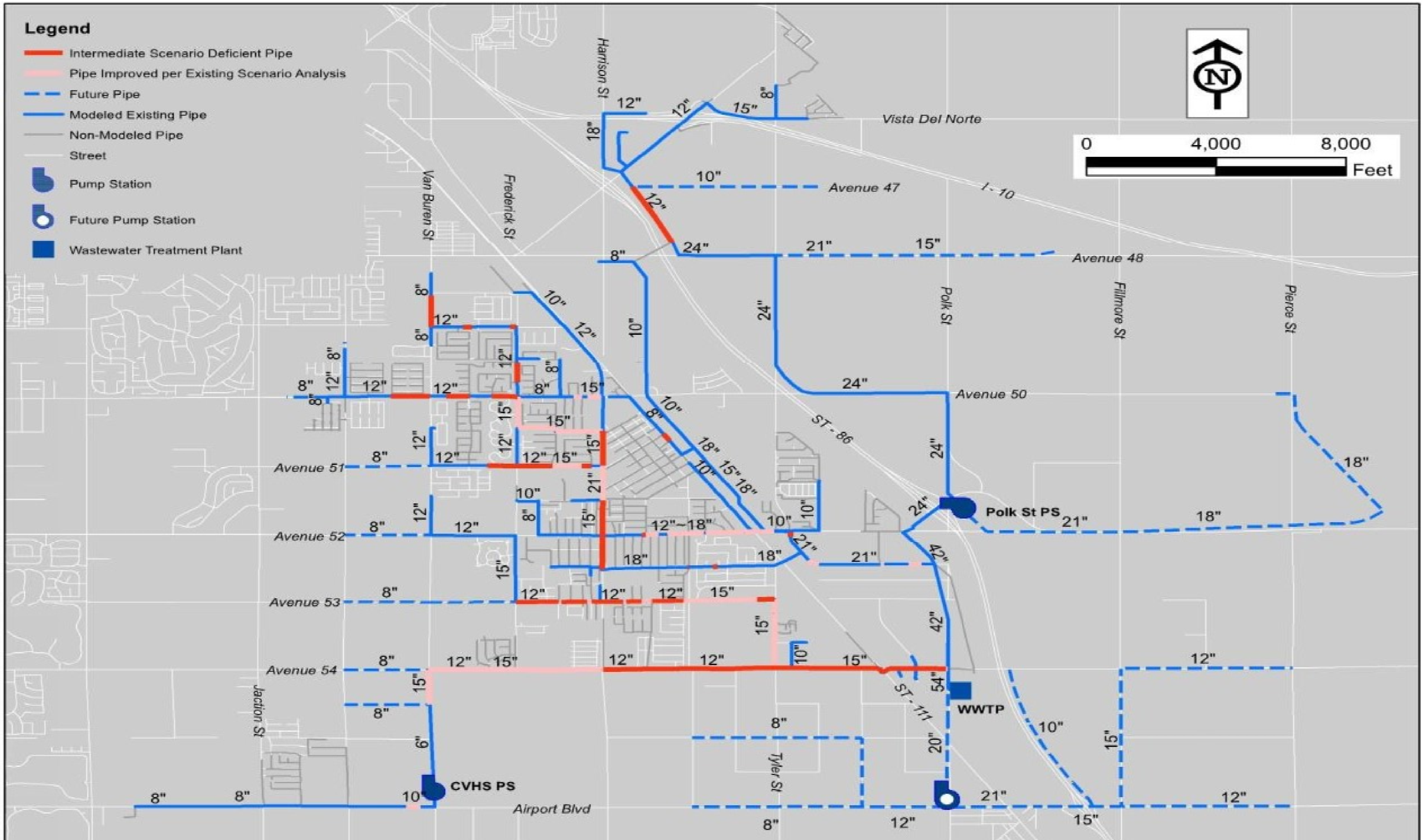


Figure 6-5
Intermediate Scenario Deficient Pipes

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### 6.2.3 2040 Gravity Sewer Capacity

The analysis of the 2040 Scenario was performed with the improvements identified under Existing and Intermediate conditions as being in place. The model results showed that 3.7 miles of gravity sewers exceeded the design criteria of d/D ratio under peak dry weather conditions. The locations of these pipes are shown on Figure 6-8. The hydraulic grade line profile for the 2040 conditions are shown in Appendix B-III

#### 6.2.3.1 Example of Significant Surcharge

The hydraulic model results for the 2040 Scenario showed that there are no pipe segments with significant surcharge. However, there are pipes reaching full capacity with d/D = 1.0

#### 6.2.3.2 Example of d/D Criteria Exceeded Within the Pipe

The HGL profile in Figure 6-9 shows d/D criteria is being exceeded as predicted by the hydraulic model under peak dry weather flow conditions where the hydraulic grade line is either reaching or below the crown of the pipe. The pipe segments range from 21- to 27-inch in diameters and are located in Industrial Way. Mainly, the pipe segments with 21-inch in diameter show depth exceeded the d/D=075 criteria. These pipes will be ranked as a priority for improvements.

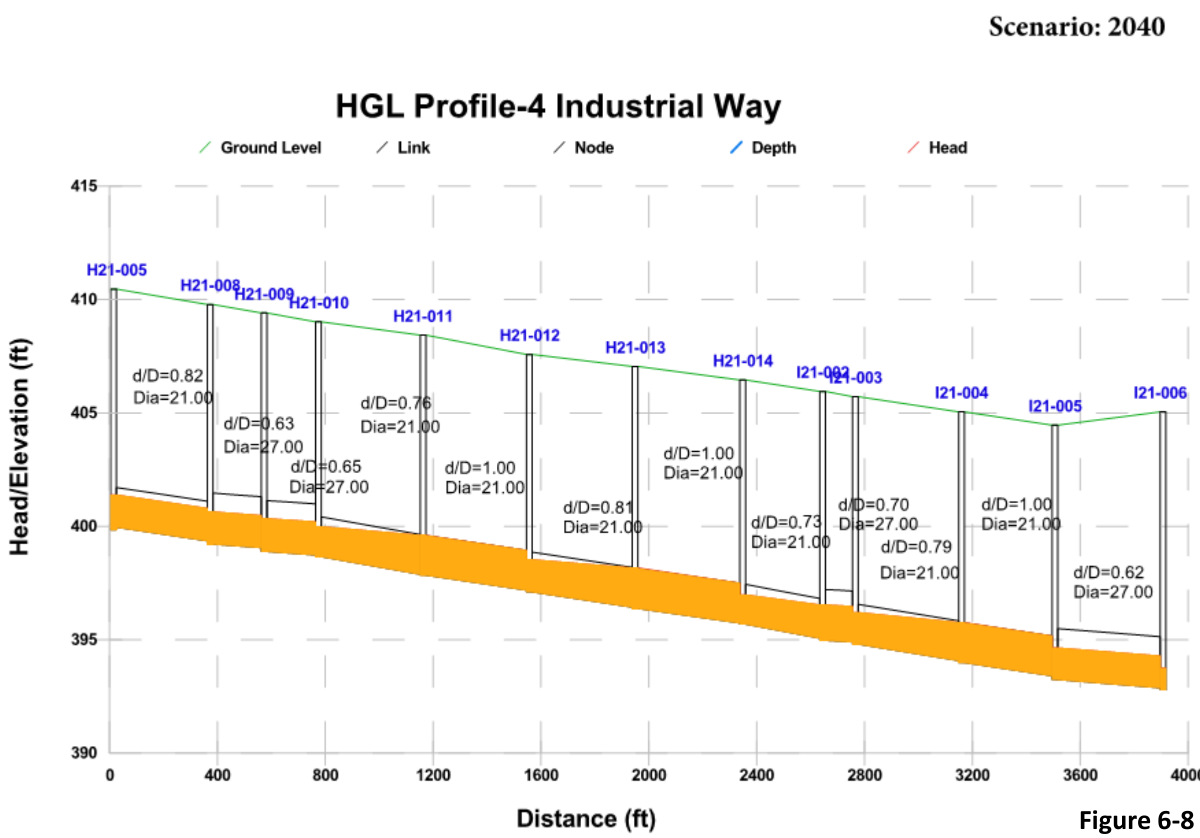


**Figure 6-8**
**2040 Scenario HGL within Pipe**

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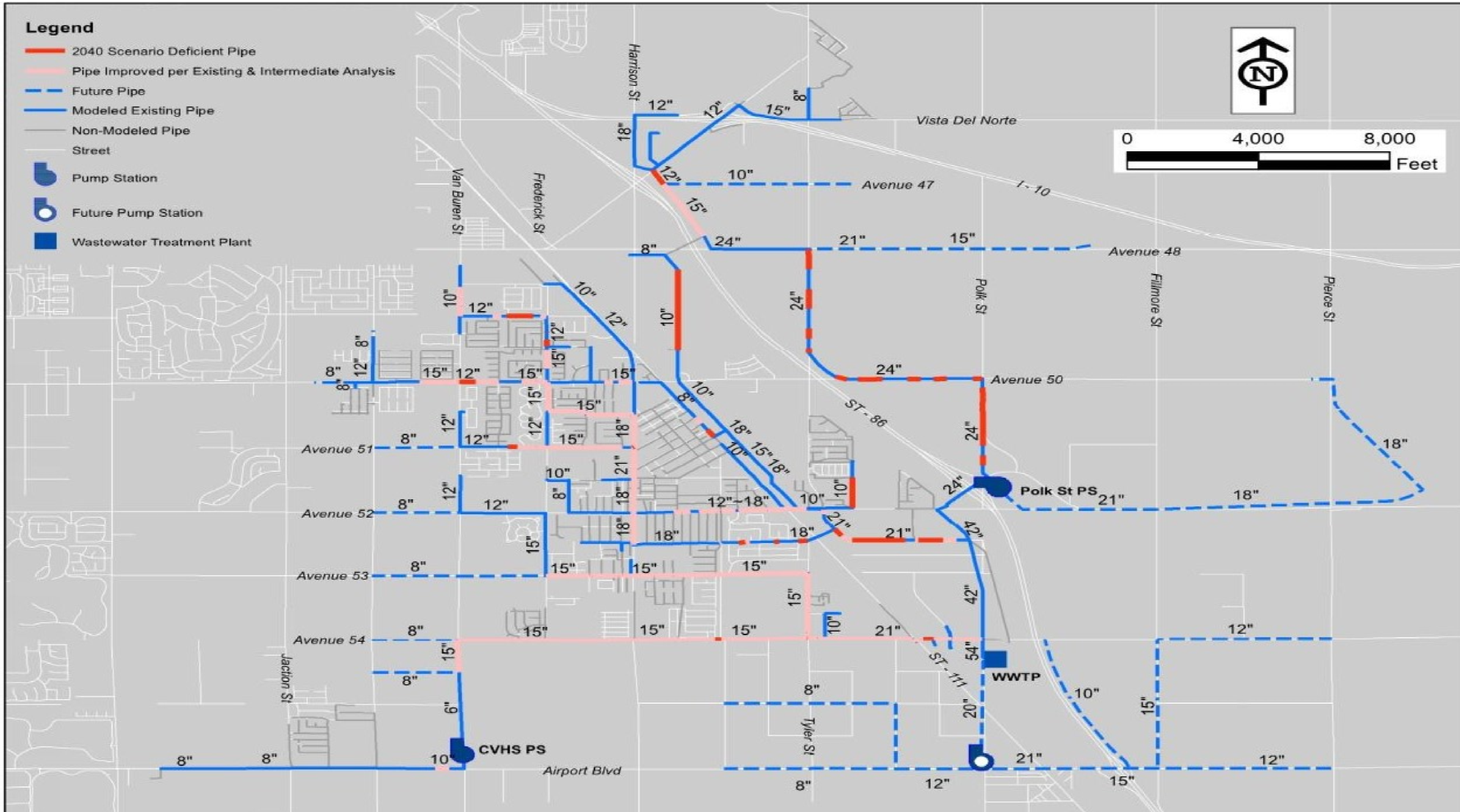


Figure 6-8
2040 Scenario Deficient Pipes

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### 6.2.4 Force Mains Capacity Evaluation

The force main deficiency criteria of this master plan is 6 ft/sec maximum force main velocity under peak design flow conditions for new pipes and 8 ft/sec maximum velocity for existing pipes. Under all the modeled scenarios and flow conditions, no deficiency has been identified for force mains during the planning period of this master plan. Table 6-1 shows the maximum velocity of force mains under 2040 Scenario, which represents the worst-case scenario.

**Table 6-1 Force Mains Maximum Velocity**

| Pump Station Name | Force Main Diameter (in) | Force Main Velocity (ft/s) |
|---|---|---|
| Coachella Valley High School (CVHS) | 6 | 5.2 |
| Polk Street at Avenue 52 | 24 | 5.3 |
| Airport Blvd at Polk Street | 12 | 4.0 |

The pump curve for CVHS shows that the design point is at 460 gpm (1.02 cfs) and a TDH of 32 feet. The pump curve was confirmed by the City with the pump manufacturer. If the pump is to operate at the design point that will make the 8-inch gravity line in Avenue 54 deficient and that is not consistent with the system operator observation. In order to verify the flows, the City conducted field measurements on October 20, 2014 and the measured flow was only 0.3 cfs. There are couple of explanations as to why the measured flow is much lower than the design point:

1- Possible hydraulic constraints at the discharge side of the pump station

2- Since the force main was constructed in the early 1970's it is possible that the interior lining of the pipe has exhibited buildup and tuberculation conditions that might have resulted in reducing the capacity of the force main.

As a sensitivity check, a hydraulic model simulation was performed with a friction coefficient of 90 and a reduced diameter of 5- inch in order to match the measured flow.

It is recommended that in the future the City conduct a field condition assessment of the force main in order to investigate the possible reasons for the reduced capacity of the pump.

### 6.2.5 Pumping Capacity Evaluation

The deficiency criterion for pump stations is that capacity must be equal to or greater than the maximum expected inflow with at least one of the largest units as a standby unit. Table 6-2 compares the maximum dry weather inflows to the pump stations for the modeled scenarios with the firm capacity of each pump station. Firm capacity is defined as the maximum pumping capacity with the largest pump is out of service. Table 6-2 also summarizes the existing firm capacity of each pump station and the maximum peak dry flows by modeled scenario. The hydraulic analysis showed that:

- The existing Coachella Valley High School Pump Station has adequate capacity to meet 2040 planning horizon peak flow conditions.
- The existing pump station at Polk Street and Avenue 52 appears to be oversized in anticipation of future growth. Under existing conditions, the maximum flow is only 0.12 cfs compared to the 8.25 cfs firm capacity. Under Intermediate conditions the peak dry weather flows exceed the rated capacity of the existing pump station, However, this deficit is marginal and could be

mitigated by managing flow levels in the wet well under peak flow conditions. Under 2040 conditions a new pump of similar capacity to the existing pump (8.25 cfs) is needed to meet the 2040 peak dry weather flows. The Polk Street pump station is needed and could not be eliminated. It is recommended that, as part of the La Entrada project, the developer looks at the potential of re-routing the flows from the Polk Street LS and using the lift station until the flows are sufficient enough that the LS is needed to operate.

- A new pump station is needed near the intersection of Airport Boulevard and Polk Street to convey the flows from future developments to the WWTP. A firm capacity of 2.1 cfs is needed to meet the Intermediate Scenario peak dry weather flows. Another pump with similar capacity is needed to meet the 2040 peak dry weather flows.

**Table 6-2 Pump Station Capacity**

| Pump Station Name | Pump Station Total/Firm Capacity (cfs) | Existing Scenario Maximum Flow (cfs) | Intermediate Scenario Maximum Flow (cfs) | 2040 Scenario Maximum Flow (cfs) | Comments |
|---|---|---|---|---|---|
| Coachella Valley High School (CVHS) | 1.02 | 0.26 | 0.29 | 0.32 | Existing pump station |
| Polk Street at Avenue 52 | 8.25 | 0.12 | 9.29 | 16.20 | Existing pump station |
| Airport Blvd at Polk Street | 0 | 0 | 2.10 | 3.20 | New Future pump station |

Usually pump stations are designed to meet peak flows, which include dry weather and wet weather flow components. For this Master Plan, there was no wet weather flow data available to determine the wet weather flow component. This flow component is the amount of rain that reaches directly or indirectly the sewer lines during rainfall events and is an indicator of the sewer conditions within a specific sewershed.

Even though the annual average rainfall for the City of Coachella is low ranges from 2 to 4 inches, it is recommended to include an allowance of wet weather flow to the peak design flow of a pump station. Additionally, it is recommended that the City operates and maintains its pump stations in accordance with the Operations and Maintenance of Wastewater Collection Systems manuals developed by the California Water Environment Association and the Office of Water Programs of California State University.

### 6.2.6 Wastewater Treatment Plant Capacity Evaluation

The City of Coachella and surrounding areas, receives its sanitary wastewater treatment services through the Coachella Sanitary District. The Coachella Sanitary District was established in 1936 and maintains approximately 90 miles of wastewater conveyance pipeline, two pump stations and a 4.5 MGD wastewater treatment plant.

The Avenue 54 Wastewater Treatment Plant consists of a headwork's facility with screenings and grit removal equipment. There are two separate secondary treatment process trains including contact stabilization tanks, (sludge removed from the CST digester is conveyed by pump to drying beds located just south of the oxidation ditches) and oxidation ditches. As the solids concentration builds in the OD process, a portion of the settled sludge is removed from the bottom of the clarifier via sludge wasting pumps, which pump the sludge to drying beds on the south end of the plant. There are also effluent disinfection facilities. Secondary effluent from the plant, following disinfection with sodium hypochlorite and dechlorination with sodium bisulfite is discharged through an outfall into the Coachella Valley Storm water Channel. A layout and Process Flow Diagram of the facility are shown on Figures 6-9 and 6-10 respectively.

The existing capacity of Avenue 54 WWTP is 4.5 MGD after the completion of its Phase 2 expansion in 2012. The WWTP provides adequate treatment capacity for the Existing scenario flow. However, depending on the rate of growth of future developments, the WWTP may need to be considered for expansion to accommodate the future flows and is recommended that the City perform a future evaluation of the WWTP capacity.

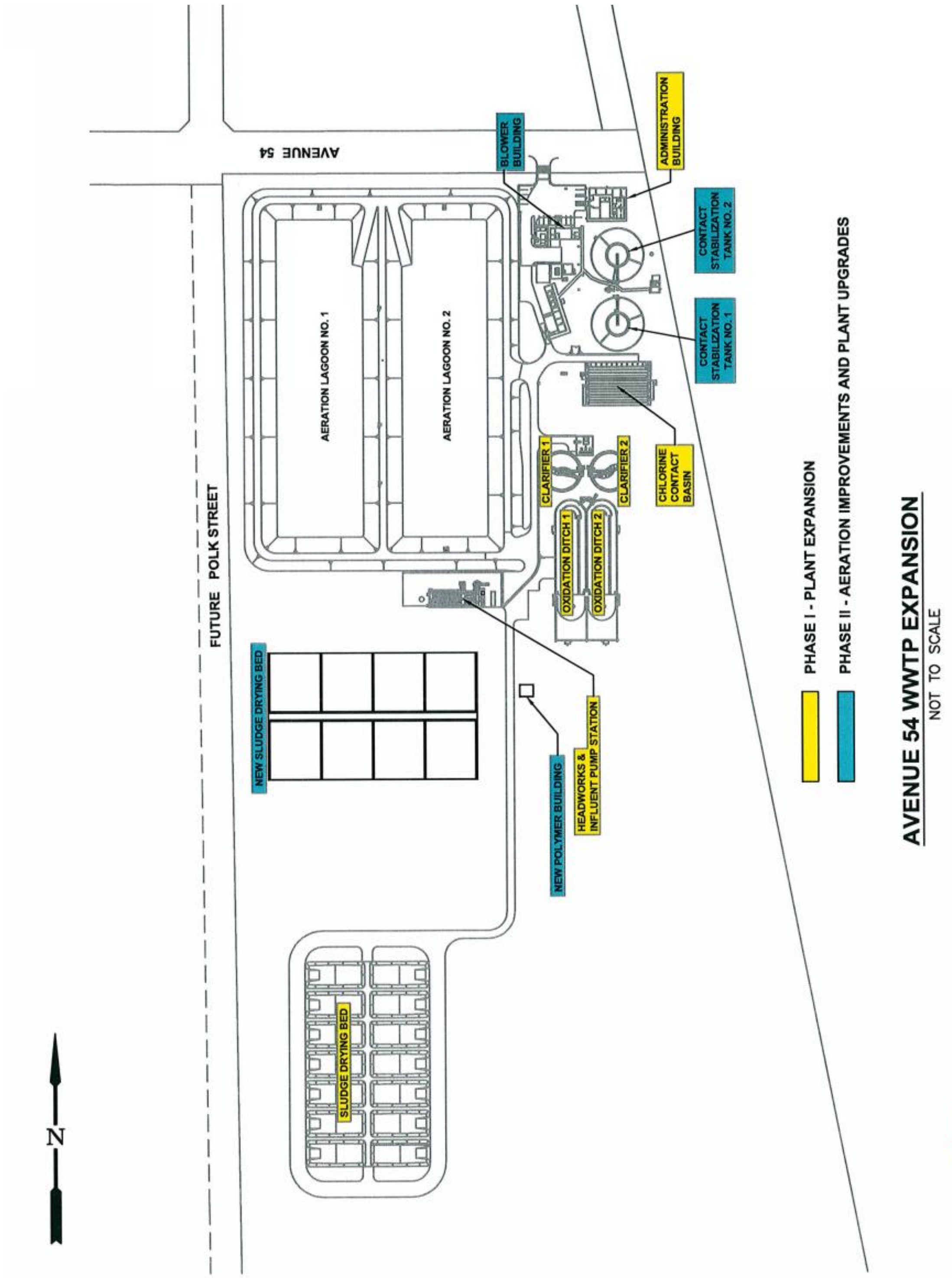


**Figure 6-10**
**Existing WWTP Layout**

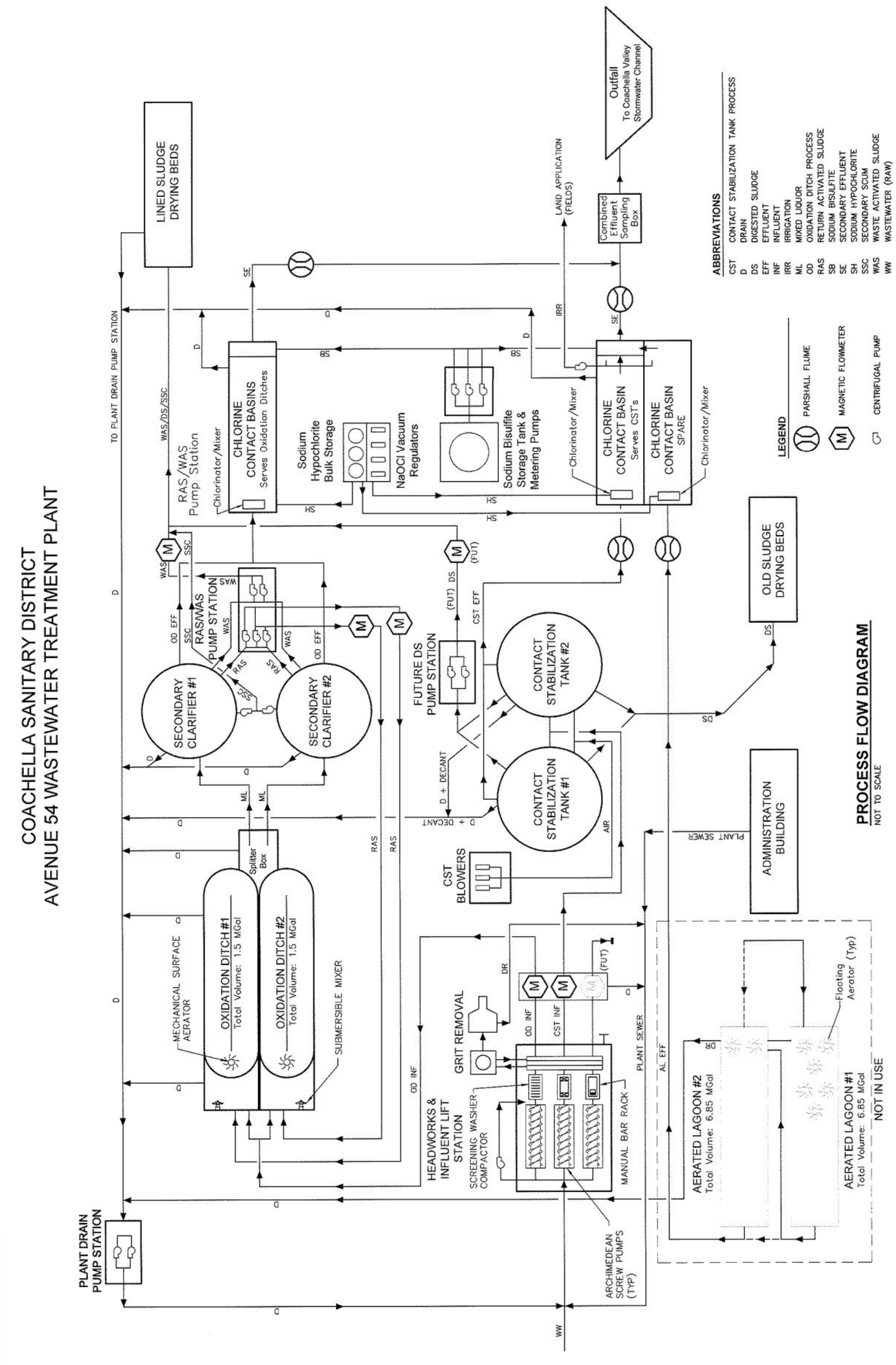


**Figure 6-11**
**Existing WWTP Process Flow Diagram**

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# Section 7
# Recommended Capital Improvement Projects

## 7.1 Introduction

This section presents the recommended sewer system capacity improvements based on the hydraulic criteria provided in Section 5 and sewer system analysis in Section 6 of this Master Plan. These improvements are intended to correct the system deficiencies to provide sanitary sewer system capacity to convey projected 2040 planning horizon flows. Figure 7-1 shows the location of the recommended sewer system capital improvement projects (CIP). An engineer's opinion of the anticipated planning level capital costs is provided for the recommended improvements.

## 7.2 Types of Sewer Pipeline Improvements

The sewer system capacity deficiencies can be eliminated by implementing either replacement or relief sewers.

Relief sewers may be constructed parallel to an existing trunk sewer, or along an independent route designed to bypass areas that are hydraulically limited. Relief sewers may be designed as on-line or off-line systems. On-line relief sewers must ensure that minimum hour dry weather flow velocities are maintained above 1.0 to 1.5 feet per second to prevent solids deposition and resultant odor and maintenance problems. Off-line relief sewers can be controlled hydraulically via a fixed weir or junction box, or mechanically using a power-operated gate, valve, or other control device. Relief sewers are often advantageous in that, in addition to providing necessary wet weather conveyance capacity, they increase sewer maintenance flexibility by allowing one line to be removed from service (without bypass pumping).

Replacement sewers may be preferable to relief sewer construction if the existing trunk sewer is in poor condition, or if construction easement limitations and/or land acquisition requirements preclude cost-effective relief sewer construction. However, replacement sewer material costs are typically higher than relief sewer costs, since the replacement sewers need to be sized larger to offer equivalent capacity as parallel sewers (existing and relief). In addition, the need to maintain sewer flow during replacement sewer construction necessitates special construction procedures (e.g., bypass pumping) and can significantly increase costs.

## 7-3 Recommended Sewer Pipeline Capacity Improvements

The recommended sewer CIP projects were identified based on the capacity evaluation discussed in Section 6. During the capacity evaluation, deficient sewers that did not meet the hydraulic capacity criteria were identified under peak dry weather conditions for the Existing, Intermediate, and 2040 planning horizon timeframes.

After identifying the deficient sewers, a more detailed analysis was conducted and the results were examined by reviewing the hydraulic grade line profile for each or consecutive pipe segments. The pipe segments that exceeded the d/D criteria were prioritized based on the water depth within the pipe or level of surcharge relative to the ground surface. The closer the hydraulic grade line is to the surface, the higher the priority. Recommended improvements were sized to convey projected maximum 2040 planning horizon flows.

### 7.3.1 Recommended Capital Improvement Projects

Deficient pipes were grouped into projects and listed in the order of priority with the highest priority first. Some projects included small segments of pipes that are not deficient, located between deficient pipes, in order to maintain the size of pipes to be larger as flow travels downstream. Figure 7-1 shows a summary of all the recommended capital improvement projects, including Existing (E1 - E6), Intermediate (I1 - I7), Build-out or 2040 (B1 - B4), and Future (F1 - F16) pipelines. These projects are also listed in Table 7-1 for Existing Scenario, Table 7-2 for Intermediate Scenario, Table 7-3 for Build-out (or 2040) scenario, and Table 7-7 for Future pipelines.

New sewers that are needed to relieve capacity deficiencies were sized to accommodate peak dry weather flows for the 2040 planning horizon using at a d/D = 0.50 for pipes equal to or less than 12-inch in diameter and d/D = 0.75 for pipes equal to or greater than 15-inch. Deficient pipes can be either replaced with larger diameter pipes, or relieved by paralleling the existing pipes. Due to space constraints, the City prefers that the existing lines be replaced. Therefore, for this master plan, it was assumed that sewer line improvements will be replaced along the same existing alignment. It is recommended that the City consider the use of trenchless technologies such as pipe bursting for the replacement of these lines, as it has been proven that these technologies have much less social and environmental impacts than open trench replacement of pipelines.

Table 7-1 through Table 7-3 present the CIPs for Existing, Intermediate, and 2040 scenarios. The capital planning level costs listed in the tables include construction cost plus 50 percent additional costs including administrative (10 percent), design (10 percent), contingency (30 percent), etc.

It is important to note that if the improvements shown under the existing scenario are not carried out by the City in the short term, potential impacts can occur to the system, including increases in grease build-up in the sewers, odors ($H_2S$), and even potential sewer backup into homes. All of these issues can lead to higher costs in maintenance and additional liability to the City.

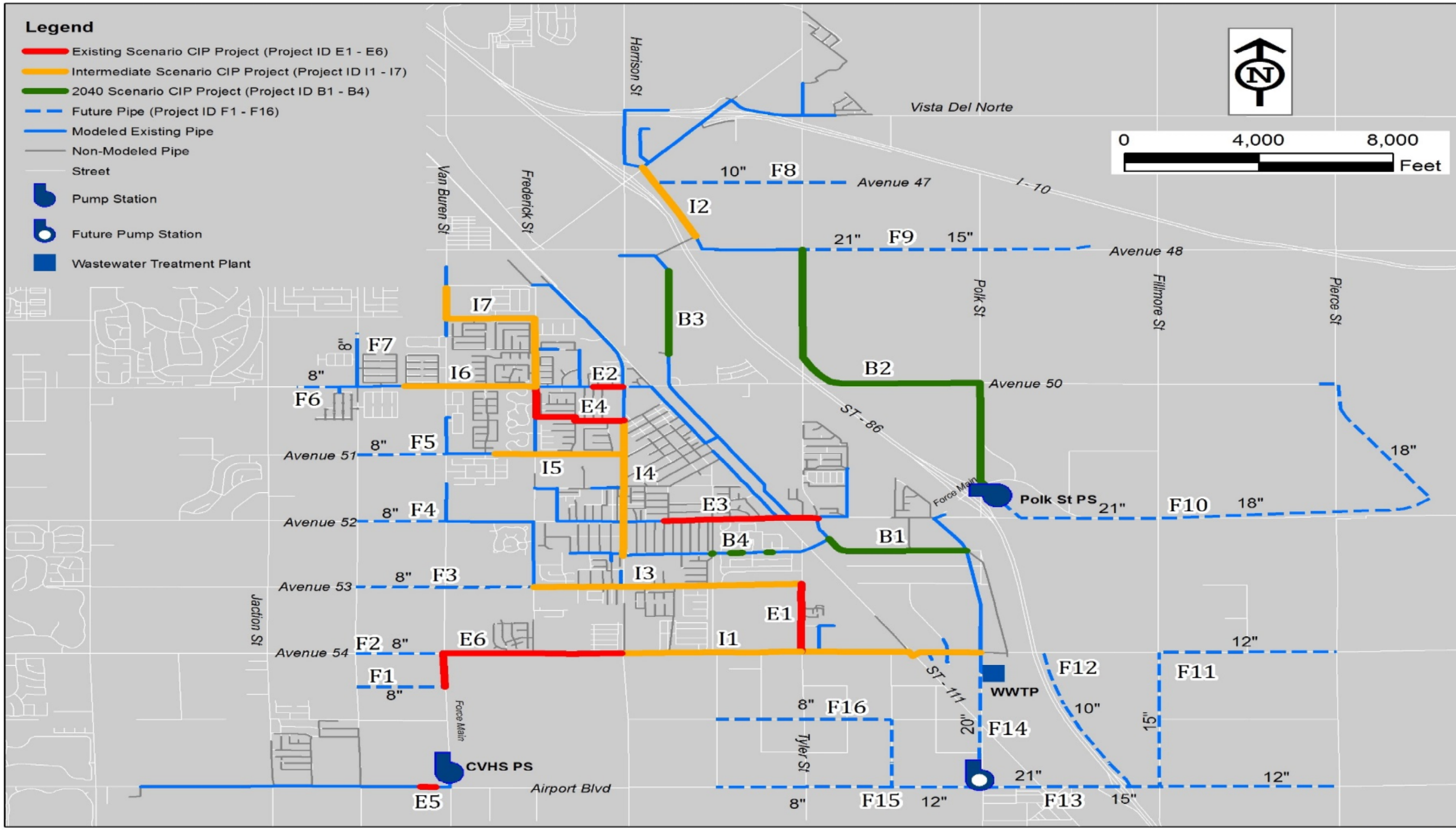


Figure 7-1
CIP Projects

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**Table 7-1 Recommended Improvement Projects and Planning Level Costs for Existing Scenario**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| E1 | G22-003 | G22-004 | 398.73 | 397.52 | 8 | 550 | 0.0022 | 15 | 0.215 | 1. Tyler St.<br>2. Appendix B-I, Profile # 3<br>3. All pipe sections show moderate surcharge with freeboard of 4' to ground surface around MANHOLE G22-003 |
| | G22-004 | G22-005 | 397.52 | 396.50 | 8 | 270 | 0.0038 | 15 | 0.105 | |
| | G22-005 | G22-006 | 396.50 | 395.74 | 8 | 394 | 0.0019 | 15 | 0.154 | |
| | G22-006 | G23-003 | 395.74 | 394.79 | 8 | 475 | 0.0020 | 15 | 0.185 | |
| | G23-003 | G23-004 | 394.79 | 393.89 | 8 | 374 | 0.0024 | 15 | 0.146 | |
| | G23-004 | G23-005 | 393.89 | 393.07 | 8 | 341 | 0.0024 | 15 | 0.133 | |
| | G23-005 | G23-006 | 393.07 | 392.39 | 8 | 259 | 0.0026 | 15 | 0.101 | |
| | **Total** | | | | | **2,663** | | | **1.039** | |
| E2 | E18-022 | E18-023 | 434.64 | 434.47 | 8 | 135 | 0.0013 | 10 | 0.035 | 1. Avenue 50<br>2. Appendix B-I, Profile # 10<br>3. The upstream end marginally exceeds d/D criteria of 0.5, and the downstream end with flat slope shows slight surcharge |
| | E18-023 | F18-001 | 434.46 | 434.32 | 8 | 131 | 0.0011 | 10 | 0.034 | |
| | F18-001 | F18-002 | 434.30 | 433.92 | 8 | 163 | 0.0023 | 10 | 0.042 | |
| | F18-002 | F18-012A | 433.91 | 433.90 | 8 | 433 | 0.0001 | 15 | 0.169 | |
| | **Total** | | | | | **862** | | | **0.281** | |
| E3 | F21-010 | F21-011 | 414.40 | 414.27 | 10 | 116 | 0.0011 | 12 | 0.036 | 1. Avenue 52<br>2. Appendix B-I, Profile # 6<br>3. All sections exceed d/D criteria of 0.5, some sections with minimum surcharge |
| | F21-011 | F21-012 | 414.26 | 414.10 | 10 | 147 | 0.0011 | 12 | 0.046 | |
| | F21-012 | F21-013 | 414.06 | 413.67 | 10 | 260 | 0.0015 | 12 | 0.081 | |
| | F21-013 | F20-007 | 413.63 | 412.79 | 10 | 260 | 0.0032 | 12 | 0.081 | |
| | F20-007 | F20-008 | 412.76 | 412.27 | 10 | 265 | 0.0019 | 12 | 0.083 | |
| | F20-008 | G20-009 | 412.23 | 412.20 | 10 | 261 | 0.0001 | 12 | 0.081 | |
| | G20-009 | G20-010 | 412.20 | 411.91 | 10 | 123 | 0.0024 | 12 | 0.038 | |
| | G20-010 | G20-011 | 411.95 | 411.58 | 10 | 11 | 0.0327 | 12 | 0.004 | |
| | G20-011 | G20-012 | 411.87 | 411.80 | 10 | 166 | 0.0004 | 15 | 0.065 | |
| | G20-012 | G20-013 | 411.80 | 411.40 | 10 | 300 | 0.0013 | 15 | 0.117 | |
| | G20-013 | G20-014 | 411.44 | 411.23 | 10 | 62 | 0.0034 | 15 | 0.024 | |
| | G20-014 | G20-015 | 411.28 | 410.79 | 10 | 267 | 0.0018 | 15 | 0.104 | |
| | G20-015 | G20-016 | 410.78 | 410.32 | 10 | 306 | 0.0015 | 15 | 0.120 | |
| | G20-016 | G20-017 | 410.28 | 12.00 | 10 | 311 | 0.0014 | 15 | 0.121 | |
| | G20-017 | G20-018 | 409.83 | 409.48 | 10 | 310 | 0.0011 | 15 | 0.121 | |
| | G20-018 | G20-008 | 409.48 | 409.39 | 10 | 241 | 0.0004 | 15 | 0.094 | |
| | G20-008 | G20-019 | 408.87 | 407.96 | 10 | 317 | 0.0029 | 15 | 0.123 | |
| | G20-019 | G20-020 | 407.96 | 406.83 | 10 | 352 | 0.0032 | 15 | 0.137 | |
| | G20-020 | H20-006 | 406.84 | 403.60 | 10 | 308 | 0.0105 | 15 | 0.120 | |
| | H20-006 | H20-007 | 403.60 | 402.04 | 10 | 199 | 0.0078 | 15 | 0.078 | |
| | **Total** | | | | | **4,582** | | | **1.674** | |

**Table 7-1 Recommended Improvement Projects and Planning Level Costs for Existing Scenario - Continued**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| E4 | E18-025 | E18-004 | 435.00 | 434.14 | 12 | 396 | 0.0022 | 15 | 0.154 | 1. Frederick St.->Julia Dr.->Westfield Way<br>2. Appendix B-I, Profile # 9<br>3. All sections exceed d/D criteria of 0.5, flow within the pipes with one section reaches full flow |
| | E18-004 | E18-024 | 434.06 | 433.62 | 12 | 279 | 0.0016 | 15 | 0.109 | |
| | E18-024 | E18-005 | 433.62 | 433.45 | 12 | 109 | 0.0016 | 15 | 0.043 | |
| | E18-005 | E18-006 | 433.44 | 432.84 | 12 | 390 | 0.0015 | 15 | 0.152 | |
| | E18-006 | E18-007 | 432.80 | 432.70 | 12 | 163 | 0.0006 | 15 | 0.064 | |
| | E18-007 | E18-008 | 432.60 | 432.19 | 12 | 329 | 0.0012 | 15 | 0.128 | |
| | E18-008 | E18-009 | 432.17 | 431.73 | 12 | 329 | 0.0013 | 15 | 0.128 | |
| | E18-009 | E18-010 | 431.72 | 431.19 | 12 | 330 | 0.0016 | 15 | 0.129 | |
| | E18-010 | E19-013 | 431.13 | 430.80 | 12 | 159 | 0.0021 | 15 | 0.062 | |
| | E19-013 | E19-014 | 430.63 | 430.60 | 12 | 19 | 0.0016 | 15 | 0.007 | |
| | E19-014 | E19-016 | 430.57 | 430.20 | 12 | 267 | 0.0014 | 15 | 0.104 | |
| | E19-016 | E19-017 | 430.07 | 429.83 | 12 | 202 | 0.0012 | 15 | 0.079 | |
| | E19-017 | E19-018 | 429.78 | 429.59 | 12 | 72 | 0.0027 | 15 | 0.028 | |
| | E19-018 | F19-001 | 429.54 | 429.10 | 12 | 255 | 0.0017 | 15 | 0.099 | |
| | F19-001 | F19-002 | 429.05 | 428.71 | 12 | 154 | 0.0022 | 15 | 0.060 | |
| | F19-002 | F19-003 | 428.69 | 427.99 | 12 | 391 | 0.0018 | 15 | 0.152 | |
| | F19-003 | F19-004 | 427.93 | 427.62 | 12 | 103 | 0.0030 | 15 | 0.040 | |
| | **Total** | | | | | **3,947** | | | **1.539** | |
| **E5** | D26-004 | D26-003 | 415.12 | 414.72 | 8 | **443** | 0.0009 | 10 | **0.115** | 1. Airport Blvd<br>2. Appendix B-I, Profile # 12<br>3. HGL d/D =0.6, caused by sudden flat slope change |
| **E6** | D24-001 | D23-001 | 421.40 | 418.18 | 8 | 1322 | 0.0024 | 15 | 0.516 | 1. Downstream of CVHS pump station force main Van Buren St->Avenue 54<br>2. Appendix B-I, Profile # 2<br>3. All sections may experience full flow or even slight surcharge. The deficiency is caused by CVHS PS pumping. The model assumes flow in equals flow out, but in reality the flows are attenuated in the force main if it is not full when the pump is on and in the receiving manhole at the end of the force main. |
| | D23-001 | E23-001 | 418.08 | 413.60 | 8 | 1879 | 0.0024 | 15 | 0.733 | |
| | E23-001 | E23-002 | 413.52 | 411.75 | 8 | 823 | 0.0022 | 15 | 0.321 | |
| | E23-002 | F23-001 | 411.73 | 405.49 | 8 | 2621 | 0.0024 | 15 | 1.022 | |
| | F23-001 | F23-002 | 405.19 | 405.04 | 8 | 81 | 0.0019 | 15 | 0.031 | |
| | **Total** | | | | | 6,726 | | | **2.623** | |
| **Existing Grand Total** | | | | | | **19,222** | | | **7.271** | |

**Table 7-2 Recommended Improvement Projects and Planning Level Costs for Intermediate Scenario**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| I1 | F23-002 | F23-003 | 404.98 | 402.83 | 12 | 1068 | 0.0020 | 15 | 0.417 | 1. Avenue 54<br>2. Appendix B-II, Profile # 1<br>3. Significant surcharge shown in most sections, with only 1' of freeboard to ground surface around MANHOLEG23-008 |
| | F23-003 | F23-004 | 402.76 | 402.41 | 12 | 256 | 0.0014 | 15 | 0.100 | |
| | F23-004 | F23-005 | 402.41 | 401.65 | 12 | 440 | 0.0017 | 15 | 0.172 | |
| | F23-005 | F23-006 | 401.61 | 400.64 | 12 | 440 | 0.0022 | 15 | 0.172 | |
| | F23-006 | G23-007 | 400.63 | 400.10 | 12 | 253 | 0.0021 | 15 | 0.099 | |
| | G23-007 | G23-001 | 400.09 | 398.98 | 12 | 188 | 0.0059 | 15 | 0.073 | |
| | G23-001 | G23-008 | 398.98 | 394.47 | 12 | 1761 | 0.0026 | 15 | 0.687 | |
| | G23-008 | G23-009 | 394.47 | 393.35 | 12 | 440 | 0.0025 | 15 | 0.172 | |
| | G23-009 | G23-002 | 393.35 | 392.27 | 12 | 421 | 0.0026 | 15 | 0.164 | |
| | G23-002 | G23-006 | 392.27 | 392.18 | 12 | 13 | 0.0071 | 15 | 0.005 | |
| | G23-006 | H23-006 | 392.10 | 391.79 | 15 | 303 | 0.0010 | 21 | 0.165 | |
| | H23-006 | H23-005 | 391.79 | 391.59 | 15 | 219 | 0.0009 | 21 | 0.120 | |
| | H23-005 | H23-007 | 391.56 | 391.32 | 15 | 409 | 0.0006 | 21 | 0.223 | |
| | H23-007 | H23-011 | 391.32 | 391.08 | 15 | 265 | 0.0009 | 21 | 0.144 | |
| | H23-011 | H23-008 | 391.09 | 390.78 | 15 | 401 | 0.0008 | 21 | 0.219 | |
| | H23-008 | H23-009 | 390.75 | 390.38 | 15 | 400 | 0.0009 | 21 | 0.219 | |
| | H23-009 | H23-010 | 390.41 | 390.00 | 15 | 400 | 0.0010 | 21 | 0.218 | |
| | H23-010 | I23-001 | 390.00 | 389.25 | 15 | 801 | 0.0009 | 21 | 0.438 | |
| | I23-001 | I23-002 | 389.25 | 389.02 | 15 | 196 | 0.0012 | 21 | 0.107 | |
| | I23-002 | I23-003 | 389.02 | 388.80 | 15 | 253 | 0.0009 | 21 | 0.138 | |
| | I23-003 | I23-004 | 388.80 | 387.96 | 15 | 313 | 0.0027 | 21 | 0.171 | |
| | I23-004 | I23-005 | 387.95 | 387.10 | 15 | 512 | 0.0017 | 21 | 0.279 | |
| | I23-005 | I23-025 | 387.05 | 386.96 | 15 | 53 | 0.0017 | 21 | 0.029 | |
| | I23-025 | I23-026 | 386.96 | 386.31 | 15 | 360 | 0.0018 | 21 | 0.197 | |
| | I23-026 | I23-006 | 386.31 | 385.38 | 15 | 521 | 0.0018 | 21 | 0.284 | |
| | I23-006 | I23-007 | 385.37 | 385.36 | 15 | 51 | 0.0002 | 27 | 0.039 | |
| | **Total** | | | | | **10,736** | | | **5.050** | |
| I2 | F14-001 | F14-002 | 449.20 | 445.33 | 12 | 681 | 0.0057 | 15 | 0.265 | 1. SPUR of ST-86<br>2. Appendix B-II, Profile # 11<br>3. All sections exceed d/D criteria of 0.5, downstream end shown full flow |
| | F14-002 | F14-005 | 445.35 | 442.55 | 12 | 1312 | 0.0021 | 15 | 0.512 | |
| | F14-005 | F15-001 | 442.55 | 440.04 | 12 | 1149 | 0.0022 | 15 | 0.448 | |
| | **Total** | | | | | **3,141** | | | **1.225** | |

**Table 7-2 Recommended Improvement Projects and Planning Level Costs for Intermediate Scenario (continued)**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| I3 | E22-002 | E22-004 | 415.35 | 412.95 | 12 | 1323 | 0.0018 | 15 | 0.516 | 1. Avenue 53<br>2. Appendix B-II, Profile # 2<br>3. All sections exceed the d/D criteria of 0.5, with small segments of full flow |
| | E22-004 | E22-005 | 412.95 | 412.86 | 12 | 77 | 0.0012 | 15 | 0.030 | |
| | E22-005 | E22-006 | 412.86 | 412.27 | 12 | 300 | 0.0020 | 15 | 0.117 | |
| | E22-006 | F22-002 | 412.27 | 411.55 | 12 | 369 | 0.0019 | 15 | 0.144 | |
| | F22-002 | F22-003 | 411.55 | 411.28 | 12 | 131 | 0.0021 | 15 | 0.051 | |
| | F22-003 | F22-004 | 411.18 | 411.00 | 12 | 130 | 0.0014 | 15 | 0.051 | |
| | F22-004 | F22-005 | 411.00 | 410.78 | 12 | 140 | 0.0016 | 15 | 0.055 | |
| | F22-005 | F22-006 | 410.78 | 410.51 | 12 | 121 | 0.0022 | 15 | 0.047 | |
| | F22-006 | F22-016 | 410.41 | 409.99 | 12 | 189 | 0.0022 | 15 | 0.074 | |
| | F22-016 | F22-014 | 409.92 | 409.17 | 12 | 400 | 0.0019 | 15 | 0.156 | |
| | F22-014 | F22-007 | 409.17 | 408.93 | 12 | 126 | 0.0019 | 15 | 0.049 | |
| | F22-007 | F22-013 | 408.88 | 408.77 | 12 | 97 | 0.0011 | 15 | 0.038 | |
| | F22-013 | F22-008 | 408.74 | 408.34 | 12 | 196 | 0.0020 | 15 | 0.076 | |
| | F22-008 | F22-009 | 408.30 | 407.66 | 12 | 273 | 0.0023 | 15 | 0.106 | |
| | F22-009 | F22-012 | 407.65 | 407.40 | 12 | 146 | 0.0017 | 15 | 0.057 | |
| | F22-012 | F22-010 | 407.33 | 407.04 | 12 | 173 | 0.0017 | 15 | 0.068 | |
| | F22-010 | F22-011 | 406.98 | 406.32 | 12 | 357 | 0.0019 | 15 | 0.139 | |
| | F22-011 | G22-001 | 406.25 | 404.28 | 12 | 626 | 0.0031 | 15 | 0.244 | |
| | G22-001 | G22-002 | 404.28 | 400.09 | 12 | 2257 | 0.0019 | 15 | 0.880 | |
| | G22-002 | G22-003 | 400.04 | 398.81 | 12 | 547 | 0.0022 | 15 | 0.213 | |
| | **Total** | | | | | **7,978** | | | **3.111** | |
| I4 | F19-004 | F19-005 | 427.21 | 426.58 | 15 | 433 | 0.0015 | 18 | 0.203 | 1. Harrison St<br>2. Appendix B-II, Profile # 6<br>3. Most of the sections exceed d/D criteria of 0.75, with some full flow sections |
| | F19-005 | F19-006 | 426.55 | 426.02 | 15 | 440 | 0.0012 | 18 | 0.206 | |
| | F19-006 | F19-007 | 425.76 | 425.17 | 15 | 442 | 0.0013 | 18 | 0.207 | |
| | F19-007 | F19-008 | 425.11 | 424.58 | 15 | 443 | 0.0012 | 21 | 0.242 | |
| | F19-008 | F20-003 | 424.57 | 423.99 | 15 | 445 | 0.0013 | 21 | 0.243 | |
| | F20-003 | F20-004 | 423.99 | 423.34 | 15 | 454 | 0.0014 | 21 | 0.248 | |
| | F20-004 | F20-005 | 423.22 | 422.18 | 15 | 438 | 0.0024 | 21 | 0.239 | |
| | F20-005 | F20-006 | 422.17 | 420.98 | 15 | 444 | 0.0027 | 21 | 0.242 | |
| | F20-006 | F21-003 | 420.93 | 419.64 | 15 | 458 | 0.0028 | 21 | 0.250 | |
| | F21-003 | F21-027 | 419.64 | 418.15 | 15 | 429 | 0.0035 | 18 | 0.201 | |
| | F21-027 | F21-028 | 418.10 | 416.48 | 15 | 446 | 0.0036 | 18 | 0.209 | |
| | F21-028 | F21-029 | 416.39 | 415.00 | 15 | 476 | 0.0029 | 18 | 0.223 | |
| | **Total** | | | | | **5,347** | | | **2.712** | |

**Table 7-2 Recommended Improvement Projects and Planning Level Costs for Intermediate Scenario (continued)**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| I5 | D19-009 | E19-001 | 434.99 | 434.34 | 12 | 281 | 0.0023 | 15 | 0.110 | 1. Avenue 51<br>2. Appendix B-II, Profile # 7<br>3. Most of the sections exceed d/D criteria of 0.5 but flow stay within the pipes |
| | E19-001 | E19-002 | 434.33 | 433.92 | 12 | 223 | 0.0018 | 15 | 0.087 | |
| | E19-002 | E19-003 | 433.92 | 433.63 | 12 | 160 | 0.0018 | 15 | 0.062 | |
| | E19-003 | E19-004 | 433.63 | 433.28 | 12 | 189 | 0.0019 | 15 | 0.074 | |
| | E19-004 | E19-006 | 433.28 | 432.55 | 12 | 347 | 0.0021 | 15 | 0.135 | |
| | E19-006 | E19-007 | 432.50 | 431.73 | 12 | 352 | 0.0022 | 15 | 0.137 | |
| | E19-007 | E19-008 | 431.73 | 431.16 | 12 | 342 | 0.0017 | 15 | 0.133 | |
| | E19-008 | E19-009 | 431.00 | 430.36 | 12 | 394 | 0.0016 | 15 | 0.154 | |
| | E19-009 | E19-010 | 430.30 | 429.93 | 12 | 243 | 0.0015 | 15 | 0.095 | |
| | E19-010 | E19-011 | 429.88 | 429.44 | 12 | 267 | 0.0016 | 15 | 0.104 | |
| | E19-011 | E19-012 | 429.44 | 429.36 | 12 | 61 | 0.0013 | 15 | 0.024 | |
| | E19-012 | F19-009 | 429.34 | 428.71 | 12 | 326 | 0.0019 | 15 | 0.127 | |
| | F19-009 | F19-015 | 428.68 | 428.30 | 12 | 151 | 0.0025 | 15 | 0.059 | |
| | F19-015 | F19-010 | 428.30 | 427.91 | 12 | 159 | 0.0025 | 15 | 0.062 | |
| | F19-010 | F19-011 | 427.84 | 425.60 | 12 | 294 | 0.0076 | 15 | 0.115 | |
| | F19-011 | F19-007 | 425.60 | 425.31 | 12 | 52 | 0.0056 | 15 | 0.020 | |
| | **Total** | | | | | **3,841** | | | **1.498** | |
| I6 | D18-001 | D18-002 | 447.09 | 446.57 | 12 | 350 | 0.0015 | 15 | 0.136 | 1. Avenue 50<br>2. Appendix B-II, Profile # 9<br>3. Upstream end marginally exceeds d/D criteria of 0.5, downstream pipe d/D reaches 0.8 |
| | D18-002 | D18-003 | 446.53 | 446.24 | 12 | 163 | 0.0018 | 15 | 0.064 | |
| | D18-003 | D18-004 | 446.24 | 445.56 | 12 | 346 | 0.0020 | 15 | 0.135 | |
| | D18-004 | D18-005 | 445.51 | 445.12 | 12 | 246 | 0.0016 | 15 | 0.096 | |
| | D18-005 | D18-006 | 445.12 | 444.96 | 12 | 105 | 0.0015 | 15 | 0.041 | |
| | D18-006 | D18-007 | 444.84 | 443.59 | 12 | 350 | 0.0036 | 15 | 0.136 | |
| | D18-007 | D18-008 | 443.53 | 443.04 | 12 | 152 | 0.0032 | 15 | 0.059 | |
| | D18-008 | D18-009 | 443.02 | 442.63 | 12 | 198 | 0.0020 | 15 | 0.077 | |
| | D18-009 | D18-011 | 442.59 | 442.00 | 12 | 515 | 0.0011 | 15 | 0.201 | |
| | D18-011 | E18-011 | 441.45 | 436.21 | 12 | 683 | 0.0077 | 12 | 0.213 | |
| | E18-011 | E18-012 | 436.21 | 436.00 | 12 | 88 | 0.0024 | 15 | 0.034 | |
| | E18-012 | E18-013 | 435.90 | 435.47 | 12 | 344 | 0.0012 | 15 | 0.134 | |
| | E18-013 | E18-025 | 435.37 | 435.00 | 12 | 356 | 0.0010 | 15 | 0.139 | |
| | **Total** | | | | | **3,895** | | | **1.466** | |

**Table 7-2 Recommended Improvement Projects and Planning Level Costs for Intermediate Scenario (continued)**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| I7 | D16-020 | D16-024 | 451.06 | 449.75 | 8 | 401 | 0.0033 | 10 | 0.104 | 1. Van Buren St. -> Avenue 49 -> Frederick St.<br>2. Appendix B-II, Profile # 10<br>3. Some sections slightly exceed d/D criteria of 0.5 |
| | D16-024 | D16-028 | 449.65 | 448.34 | 8 | 400 | 0.0033 | 10 | 0.104 | |
| | D16-028 | D17-001 | 448.24 | 446.93 | 8 | 400 | 0.0033 | 10 | 0.104 | |
| | D17-001 | D17-002 | 446.80 | 445.89 | 12 | 435 | 0.0021 | 12 | 0.136 | |
| | D17-002 | D17-003 | 445.89 | 444.92 | 12 | 410 | 0.0024 | 12 | 0.128 | |
| | D17-003 | D17-008 | 444.91 | 444.65 | 12 | 149 | 0.0017 | 12 | 0.046 | |
| | D17-008 | D17-004 | 444.59 | 444.41 | 12 | 272 | 0.0007 | 15 | 0.106 | |
| | D17-004 | D17-005 | 444.40 | 444.10 | 12 | 150 | 0.0020 | 15 | 0.058 | |
| | D17-005 | E17-001 | 444.05 | 443.40 | 12 | 420 | 0.0015 | 15 | 0.164 | |
| | E17-001 | E17-002 | 443.30 | 442.74 | 12 | 400 | 0.0014 | 15 | 0.156 | |
| | E17-002 | E17-019 | 442.64 | 442.10 | 12 | 210 | 0.0026 | 15 | 0.082 | |
| | E17-019 | E17-003 | 441.98 | 441.81 | 12 | 189 | 0.0009 | 15 | 0.074 | |
| | E17-003 | E17-021 | 441.80 | 440.96 | 12 | 314 | 0.0027 | 15 | 0.122 | |
| | E17-021 | E17-004 | 440.89 | 440.29 | 12 | 316 | 0.0019 | 15 | 0.123 | |
| | E17-005 | E17-006 | 440.26 | 439.57 | 12 | 291 | 0.0024 | 15 | 0.113 | |
| | E17-004 | E17-005 | 440.24 | 440.20 | 12 | 28 | 0.0014 | 15 | 0.011 | |
| | E17-006 | E17-007 | 439.55 | 439.12 | 12 | 277 | 0.0016 | 15 | 0.108 | |
| | E17-007 | E17-008 | 439.10 | 438.83 | 12 | 105 | 0.0026 | 15 | 0.041 | |
| | E17-008 | E17-009 | 438.83 | 438.65 | 12 | 29 | 0.0062 | 15 | 0.011 | |
| | E17-009 | E17-010 | 438.64 | 437.69 | 12 | 483 | 0.0020 | 15 | 0.188 | |
| | E17-010 | E18-001 | 437.63 | 437.33 | 12 | 201 | 0.0015 | 15 | 0.078 | |
| | E18-001 | E18-026 | 437.33 | 437.14 | 12 | 125 | 0.0015 | 15 | 0.049 | |
| | E18-026 | E18-002 | 437.08 | 436.63 | 12 | 112 | 0.0040 | 15 | 0.044 | |
| | E18-002 | E18-025 | 436.63 | 435.00 | 12 | 429 | 0.0038 | 15 | 0.167 | |
| | **Total** | | | | | **6,545** | | | **2.319** | |
| **Intermediate Grand Total** | | | | | | **41,483** | | | **17.381** | |

**Table 7-3 Recommended Improvement Projects and Planning Level Costs for 2040 Scenario**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| B1 | H21-005 | H21-008 | 399.96 | 399.36 | 21 | 363 | 0.0017 | 24 | 0.226 | 1. Industrial Way<br>2. Appendix B-III, Profile # 4<br>3. All sections exceed d/D criteria of 0.75, with some segments of full flow |
| | H21-008 | H21-009 | 399.22 | 399.06 | 21 | 193 | 0.0008 | 27 | 0.150 | |
| | H21-009 | H21-010 | 398.89 | 398.74 | 21 | 195 | 0.0008 | 27 | 0.152 | |
| | H21-010 | H21-011 | 398.66 | 397.89 | 21 | 395 | 0.0019 | 27 | 0.308 | |
| | H21-011 | H21-012 | 397.84 | 397.19 | 21 | 401 | 0.0016 | 27 | 0.313 | |
| | H21-012 | H21-013 | 397.11 | 396.44 | 21 | 399 | 0.0017 | 27 | 0.311 | |
| | H21-013 | H21-014 | 396.38 | 395.74 | 21 | 407 | 0.0016 | 27 | 0.317 | |
| | H21-014 | I21-002 | 395.69 | 395.07 | 21 | 296 | 0.0021 | 27 | 0.231 | |
| | I21-002 | I21-003 | 394.97 | 394.90 | 21 | 108 | 0.0006 | 27 | 0.084 | |
| | I21-003 | I21-004 | 394.81 | 394.08 | 21 | 401 | 0.0018 | 27 | 0.313 | |
| | I21-004 | I21-005 | 393.99 | 393.41 | 21 | 350 | 0.0017 | 27 | 0.273 | |
| | I21-005 | I21-006 | 393.24 | 392.88 | 21 | 409 | 0.0009 | 27 | 0.319 | |
| | I21-006 | I21-011 | 392.79 | 390.80 | 21 | 384 | 0.0052 | 27 | 0.299 | |
| | **Total** | | | | | **4,302** | | | **3.299** | |
| B2 | G15-009 | G15-010 | 426.96 | 426.47 | 24 | 410 | 0.0012 | 27 | 0.320 | 1. Tyler St. -> Avenue 50 -> Polk St.<br>2. Appendix B-III, Profile # 11<br>3. Some sections exceed d/D criteria of 0.75, with small segments of full flow |
| | G15-010 | G16-001 | 426.35 | 425.84 | 24 | 400 | 0.0013 | 27 | 0.312 | |
| | G16-001 | G16-002 | 425.76 | 425.00 | 24 | 401 | 0.0019 | 27 | 0.312 | |
| | G16-002 | G16-003 | 425.00 | 424.29 | 24 | 399 | 0.0018 | 27 | 0.311 | |
| | G16-003 | G16-004 | 424.25 | 423.98 | 24 | 400 | 0.0007 | 27 | 0.312 | |
| | G16-004 | H17-001 | 423.84 | 423.40 | 24 | 400 | 0.0011 | 27 | 0.312 | |
| | H17-001 | H17-002 | 423.30 | 422.60 | 24 | 400 | 0.0017 | 27 | 0.312 | |
| | H17-002 | H17-003 | 422.57 | 421.88 | 24 | 400 | 0.0017 | 27 | 0.312 | |
| | H17-003 | H17-004 | 421.81 | 421.30 | 24 | 400 | 0.0013 | 27 | 0.312 | |
| | H17-004 | H17-005 | 421.28 | 420.63 | 24 | 400 | 0.0016 | 27 | 0.312 | |
| | H17-005 | H17-006 | 420.53 | 420.23 | 24 | 226 | 0.0013 | 27 | 0.176 | |
| | H17-006 | H18-001 | 420.18 | 419.14 | 24 | 500 | 0.0021 | 27 | 0.390 | |
| | H18-001 | H18-010 | 419.11 | 418.45 | 24 | 401 | 0.0016 | 27 | 0.312 | |
| | H18-010 | H18-011 | 418.43 | 417.79 | 24 | 372 | 0.0017 | 27 | 0.290 | |
| | H18-011 | H18-002 | 417.76 | 417.30 | 24 | 363 | 0.0013 | 27 | 0.283 | |
| | H18-002 | H18-003 | 417.19 | 415.86 | 24 | 336 | 0.0040 | 27 | 0.262 | |

**Table 7-3 Recommended Improvement Projects and Planning Level Costs for 2040 Scenario (continued)**

| Project ID | From MANHOLE | To MANHOLE | From Invert (ft) | To Invert (ft) | Existing Diameter (in) | Length (ft) | Slope (ft/ft) | Replacement Diameter (in) | Replacement Capital Cost ($Million) | Location & Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| B2 | H18-003 | H18-004 | 415.18 | 414.65 | 24 | 350 | 0.0015 | 27 | 0.273 | 1. Tyler St./Avenue 50/Polk St.<br>2. Appendix B-III, Profile # 11<br>3. Some sections marginally exceed d/D criteria of 0.75, with small segments of full flow |
| | H18-004 | H18-005 | 414.59 | 414.04 | 24 | 400 | 0.0014 | 27 | 0.312 | |
| | H18-005 | H18-006 | 413.98 | 411.56 | 24 | 399 | 0.0061 | 27 | 0.311 | |
| | H18-006 | H18-007 | 410.35 | 409.64 | 24 | 352 | 0.0020 | 27 | 0.274 | |
| | H18-007 | I18-002 | 409.54 | 408.93 | 24 | 399 | 0.0015 | 27 | 0.311 | |
| | I18-002 | I18-003 | 408.90 | 408.26 | 24 | 399 | 0.0016 | 27 | 0.311 | |
| | I18-003 | I18-004 | 408.16 | 407.73 | 24 | 401 | 0.0011 | 27 | 0.313 | |
| | I18-004 | I18-005 | 407.65 | 406.79 | 24 | 400 | 0.0021 | 27 | 0.312 | |
| | I18-005 | I18-006 | 406.69 | 402.83 | 24 | 400 | 0.0097 | 27 | 0.312 | |
| | I18-006 | I18-007 | 402.72 | 402.45 | 24 | 285 | 0.0009 | 27 | 0.222 | |
| | I18-007 | I18-008 | 402.10 | 401.23 | 24 | 353 | 0.0025 | 27 | 0.275 | |
| | I18-008 | I18-009 | 401.08 | 400.43 | 24 | 399 | 0.0016 | 27 | 0.311 | |
| | I18-009 | I18-010 | 400.40 | 399.75 | 24 | 389 | 0.0017 | 27 | 0.304 | |
| | I18-010 | I19-001 | 399.72 | 399.40 | 24 | 400 | 0.0008 | 27 | 0.312 | |
| | I19-001 | I19-002 | 399.25 | 398.62 | 24 | 400 | 0.0016 | 27 | 0.312 | |
| | I19-002 | I19-003 | 398.45 | 397.89 | 24 | 400 | 0.0014 | 27 | 0.312 | |
| | I19-003 | I19-004 | 397.87 | 397.50 | 24 | 400 | 0.0009 | 27 | 0.312 | |
| | I19-004 | I19-005 | 397.31 | 396.54 | 24 | 401 | 0.0019 | 27 | 0.313 | |
| | I19-005 | I20-001 | 396.49 | 396.14 | 24 | 400 | 0.0009 | 27 | 0.312 | |
| | I20-001 | I20-002 | 396.05 | 395.24 | 24 | 331 | 0.0024 | 27 | 0.258 | |
| | I20-002 | I20-006 | 395.14 | 390.97 | 24 | 316 | 0.0132 | 27 | 0.247 | |
| | I20-006 | I20-007 | 390.74 | 390.03 | 24 | 102 | 0.0069 | 27 | 0.080 | |
| | I20-007 | POLK_ST_WW | 389.88 | 382.00 | 24 | 382 | 0.0206 | 27 | 0.298 | |
| | **Total** | | | | | **14,666** | | | **11.439** | |
| **B3** | F16-004 | F17-013 | 434.76 | 430.39 | 10 | **3269** | 0.0013 | 12 | **1.020** | 1. Oates Ln<br>2. Appendix B-III, Profile # 10<br>3. Pipe marginally exceeds d/D criteria of 0.5 |
| **B4** | G21-001 | G21-002 | 408.30 | 408.24 | 18 | 49 | 0.0012 | 21 | 0.027 | 1. Valley Rd -> Avenida Allenah<br>2. Appendix B-III, Profile # 5<br>3. Sections marginally exceed d/D criteria of 0.75 |
| | G21-015 | G21-014 | 406.75 | 406.40 | 18 | 206 | 0.0017 | 21 | 0.112 | |
| | G21-014 | G21-004 | 406.40 | 406.19 | 18 | 141 | 0.0015 | 21 | 0.077 | |
| | G21-011 | G21-006 | 404.27 | 404.00 | 18 | 144 | 0.0019 | 21 | 0.078 | |
| | **Total** | | | | | **540** | | | **0.295** | |
| **2040 Grand Total** | | | | | | **22,776** | | | **16.053** | |

Table 7-4 summarizes by timeframe the total length of deficient pipes and estimated capital cost for replacing or paralleling pipes.

**Table 7-4 Summary of Capital Improvement Projects**

| Scenario | Length (ft) | Replacement Cost $Million | Parallel Cost $Million |
|---|---|---|---|
| Existing | 19,222 | 7.27 | 6.00 |
| Intermediate | 41,483 | 17.38 | 12.57 |
| 2040 | 22,776 | 16.05 | 8.33 |
| Total | 83,481 | 40.70 | 26.89 |

The planning level cost estimates for the recommended improvements are based on the current cost information. The unit costs per linear foot of replaced pipe construction are estimated based upon the similar projects conducted in the adjacent areas of City of Coachella.

## 7.3.2 Basis for Sewer Planning Level Construction Cost

An engineer's opinion of the anticipated planning level capital costs is provided to assist the City of Coachella in the future development of its capital improvement program projects. During the actual development of infrastructure improvements, it is recommended that the City re-evaluate and update these planning level costs as a check against current cost data.

The capital cost in Table 7-1 through Table 7-4 are based on the unit cost required to either replace or relief the deficient sewers for their 2040 planning horizon flows depending on the length and diameter of the proposed sewers, and plus a 40 percent estimated costs for engineering (10 percent), administration (10 percent), contingency (20 percent), etc. The pipe capital unit costs for sewers up to 15 feet deep are shown in Table 7-5; these costs include manholes and other sewer appurtenances (cleanouts, laterals, etc.), shoring, traffic control, pavement restoration and other incidentals that are required in typical construction of sewer pipelines within the street right-of-way.

**Table 7-5 Planning Level Costs for Sewer Replacement**

| Diameter (in) | Replacement |
|---|---|
| | Unit Capital Planning Level Cost ($/lf) |
| 8 | 210 |
| 10 | 260 |
| 12 | 310 |
| 15 | 390 |
| 18 | 470 |
| 21 | 550 |
| 24 | 780 |
| 27 | 620 |
| 36 | 940 |

## 7.4 Pump Station Capital Improvement Project

Based on the hydraulic criteria provided in Section 5 and sewer system analysis in Section 6, the Polk Street Pump Station is anticipated to need a future new pump that is similar to the existing pumps in the 2040 Scenario. Table 7-6 provides a summary of the proposed pump and estimated capital cost.

**Table 7-6 Summary of Pump Station Capital Improvement**

| Number of Future Pump | Pump Capacity (cfs) | TDH (ft) | Capital Cost ($Million) |
|---|---|---|---|
| 1 | 8.25 | 77 | 0.13 |

## 7.5 Future Pipeline Projects

Figure 7-1 shows the conceptual location of the future pipes that were model to serve projected future growth. The future pipes are grouped into future pipeline projects based on their locations. The project IDs are labeled in Figure 7-1 and are listed in Table 7-7, in which the diameter, length, and street names are also provided. Future pipes are usually funded by developers and should not be part of the CIP.

**Table 7-7 Summary of Future Sewer Projects**

| Project ID | Pipe Id | From MANHOLE | To MANHOLE | From Invert | To Invert | Diameter (in) | Length (ft) | Location |
|---|---|---|---|---|---|---|---|---|
| **F1** | 2022 | C24-010 | D24-001 | 433.0 | 421.5 | 8 | ***2,634*** | Between Avenue 55 and Avenue 54 |
| **F2** | 1865 | C23-010 | D23-001 | 436.0 | 418.3 | 8 | ***2,605*** | Avenue 54 |
| **F3** | 1991 | D22-010 | D22-020 | 428.0 | 422.1 | 8 | 1,306 | Avenue 53 |
| | 1993 | D22-020 | E22-002 | 422.0 | 415.5 | 8 | 1,313 | |
| | 1995 | C22-010 | D22-010 | 440.0 | 428.1 | 8 | 2,683 | |
| | ***Total*** | | | | | | ***5,302*** | |
| **F4** | 1987 | C21-010 | D21-010 | 441.5 | 436.2 | 8 | 1,324 | Avenue 52 |
| | 1989 | D21-010 | D21-001 | 436.1 | 430.8 | 8 | 1,338 | |
| | ***Total*** | | | | | | ***2,662*** | |
| **F5** | 1985 | C19-010 | D19-004 | 454.0 | 437.8 | 8 | ***2,672*** | Avenue 51 |
| **F6** | 1983 | B18-010 | C18-002 | 465.0 | 451.1 | 8 | ***1,255*** | Avenue 50 |
| **F7** | 1975 | C17-010 | C17-002 | 457.6 | 455.6 | 8 | 392 | Calhoun St. |
| | 1981 | C17-020 | C17-010 | 465.0 | 457.7 | 8 | 670 | |
| | ***Total*** | | | | | | ***1,062*** | |
| **F8** | 2001 | H14-010 | F14-002 | 462.0 | 445.4 | 10 | ***5,732*** | Avenue 47 |
| **F9** | 1965 | I15-051 | H15-060 | 469.0 | 434.1 | 15 | 3,308 | Avenue 48 |
| | 2003 | J15-010 | I15-051 | 535.0 | 469.1 | 15 | 2,649 | |
| | 1957 | H15-060 | G15-009 | 434.0 | 427.8 | 21 | 2,645 | |
| | ***Total*** | | | | | | ***8,602*** | |

**Table 7-7 Summary of Future Sewer Projects (continued)**

| Project ID | Pipe Id | From MANHOLE | To MANHOLE | From Invert | To Invert | Diameter (in) | Length (ft) | Location |
|---|---|---|---|---|---|---|---|---|
| **F10** | 2007 | L18-010 | M20-010 | 617.0 | 609.1 | 18 | 6,080 | Avenue 52/ East of Canal |
| | 2009 | M20-010 | K20-020 | 609.0 | 503.1 | 18 | 2,930 | |
| | 2023 | K20-020 | K20-010 | 503.0 | 399 | 18 | 5,295 | |
| | 2025 | K20-010 | J20-010 | 398.9 | 390.8 | 21 | 4,121 | |
| | 2027 | J20-010 | POLK_ST_WW | 390.7 | 388.5 | 24 | 1,082 | |
| | ***Total*** | | | | | | ***19,508*** | |
| **F11** | 2011 | M23-010 | K23-010 | 449.0 | 387 | 12 | 5,304 | Fillmore St. |
| | 1973 | K23-010 | K26-020 | 386.9 | 375.4 | 15 | 5,286 | Avenue 54 |
| | ***Total*** | | | | | | ***10,590*** | |
| **F12** | 2013 | J23-010 | K26-010 | 391.0 | 375 | 10 | ***5,963*** | SPUR South |
| **F13** | 2017 | M26-010 | K26-020 | 413.0 | 376 | 12 | 5,358 | Airport Blvd. East of Future PS |
| | 2019 | K26-020 | K26-010 | 375.3 | 373.5 | 15 | 871 | |
| | 2021 | K26-010 | J26-020 | 373.4 | 366 | 15 | 1,563 | |
| | 2995 | J26-020 | 9004 | 365.9 | 360 | 21 | 3,053 | |
| | ***Total*** | | | | | | ***10,845*** | |
| **F14** | 2993 | 58 | I23-021 | 376.0 | 381.0 | 12 | ***4,089*** | Future PS to WWTP Easement |
| **F15** | 1967 | G26-020 | H26-020 | 391.0 | 382.0 | 8 | 2,664 | Airport Blvd. West of Future PS |
| | 1969 | H26-020 | H26-010 | 381.9 | 373.5 | 8 | 2,573 | |
| | 1971 | H26-010 | I26-010 | 372.9 | 365.8 | 12 | 2,378 | |
| | 2991 | I26-010 | 9004 | 365.7 | 364.6 | 12 | 366 | |
| | ***Total*** | | | | | | ***7,981*** | |
| **F16** | 1959 | F25-010 | H24-020 | 397.0 | 389.0 | 8 | 2,608 | Avenue 55 /Easement |
| | 1961 | H24-020 | H24-010 | 388.9 | 381.0 | 8 | 2,617 | |
| | 1963 | H24-010 | H26-010 | 380.9 | 373.0 | 8 | 2,659 | |
| | ***Total*** | | | | | | ***7,884*** | |

# 7.6 Funding for Capital Improvement Projects

It is important that the City of Coachella start programming projects identified for the existing conditions in Table 7-1, E1 through E6, for implementation in the 2015-2020 Capital Improvement Program. Projects for the intermediate and build-out scenarios are expected to be funded under a combination of development impact fees and rate increases. It is recommended that the City conducts a local limit study, updates the development impact fees and performs a study to see if sewer rates need to be increased.

# 7.7 Recommendations for Condition Assessment

This sewer collection system master plan project evaluated capacity of the system and its deficiencies under existing, intermediate and build-out scenarios. It is recommended that, as the next phase of having a full understanding of the system, the City embark in a condition assessment program that would entail performing video (CCTV) inspections of the system using the Pipeline Assessment Certification Program (PACP ®) coding standard developed by the National Association of Sewer Service Companies (NASSCO).

The inspections can be performed by licensed CCTV contractors, or the City may also opt for providing PACP training to their existing field staff so they can perform the inspections. Using the condition assessment, the City can then develop a rehabilitation and replacement program and CIP that goes hand-in-hand with the capacity enhancement projects presented in this Master Plan, in order to comply with the State Water Resources Control Board Statewide General Waste Discharge Requirements for Sanitary Sewer Systems.

# Appendices

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# Appendix A

# Dry Weather Flow Model Calibration Results

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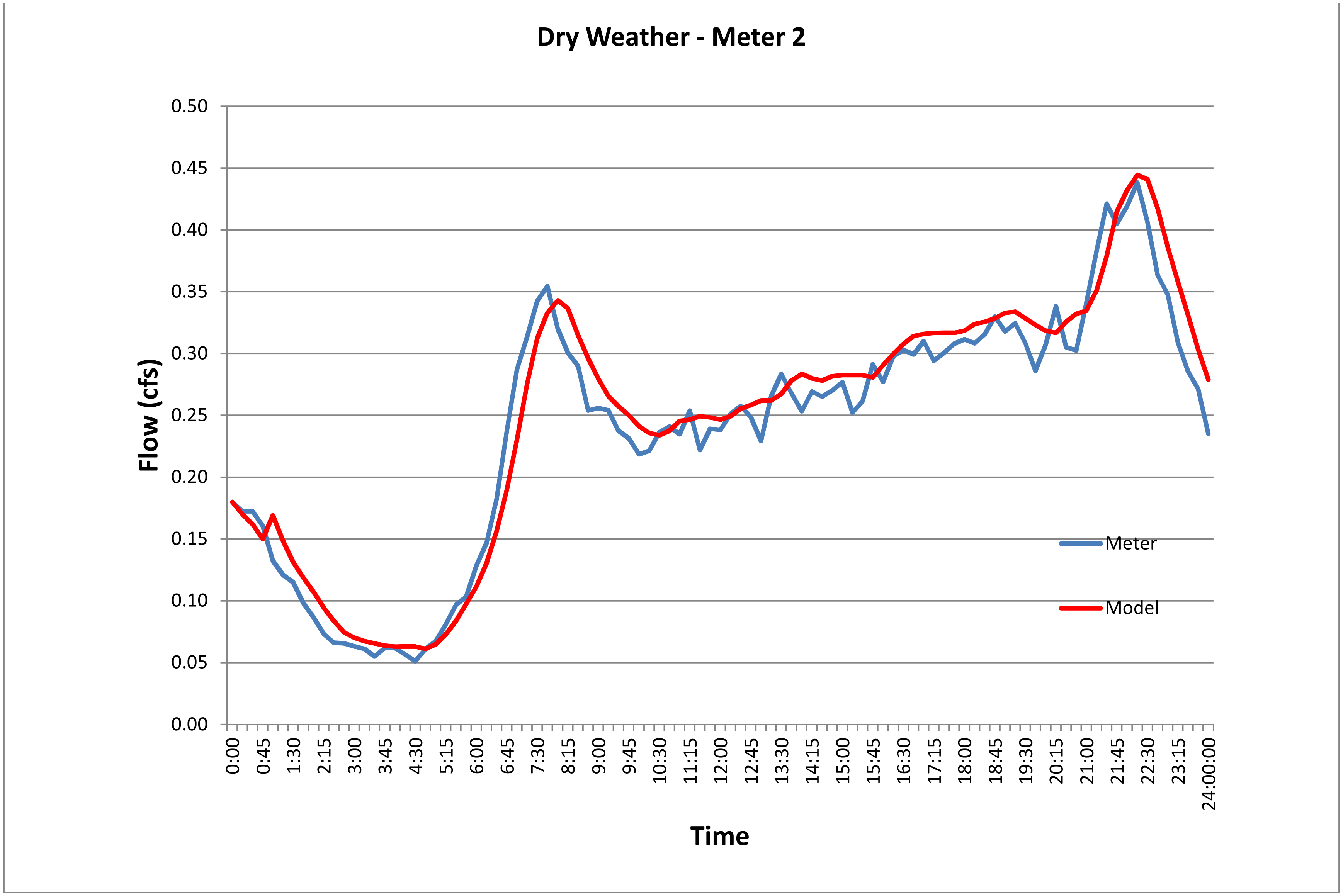
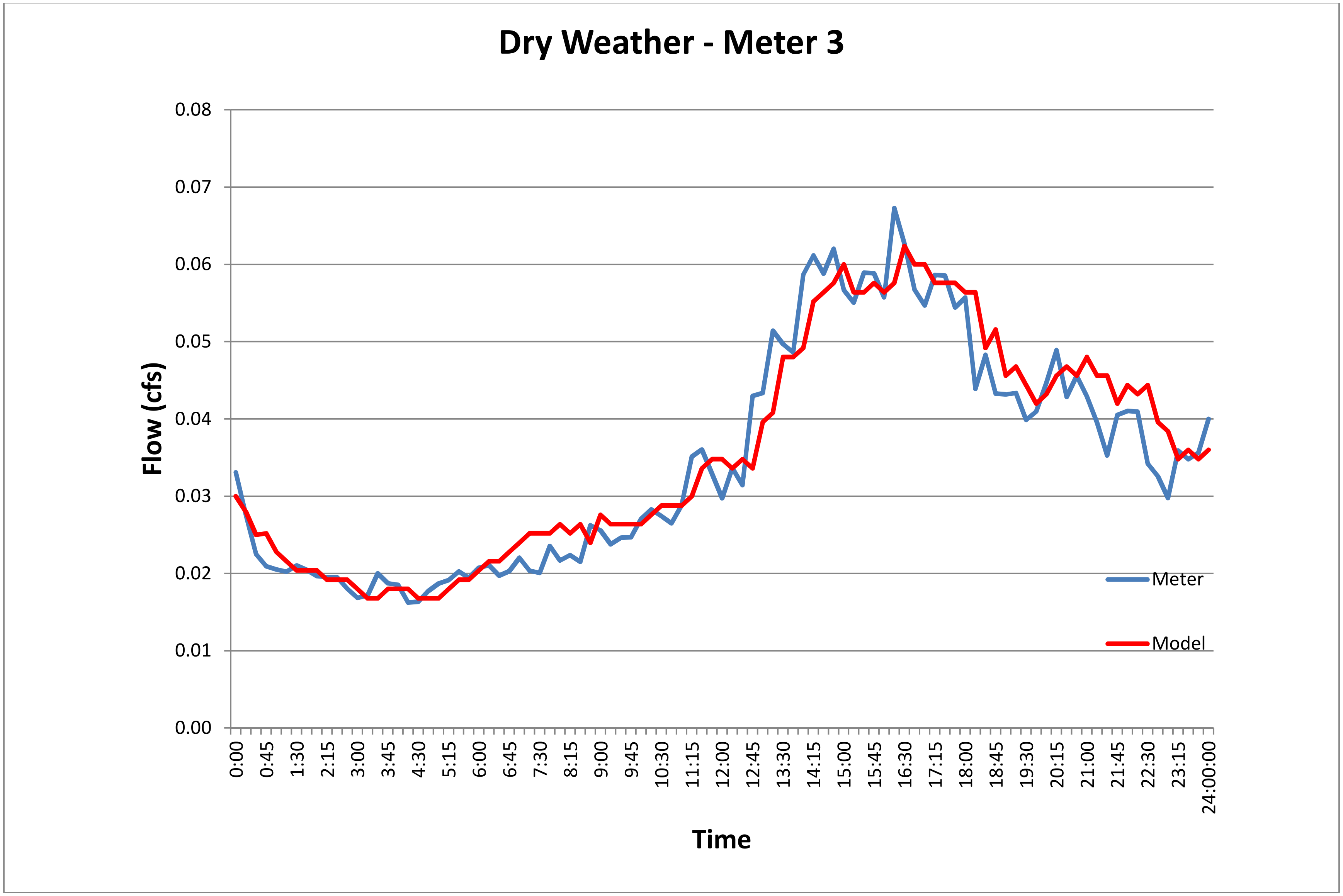
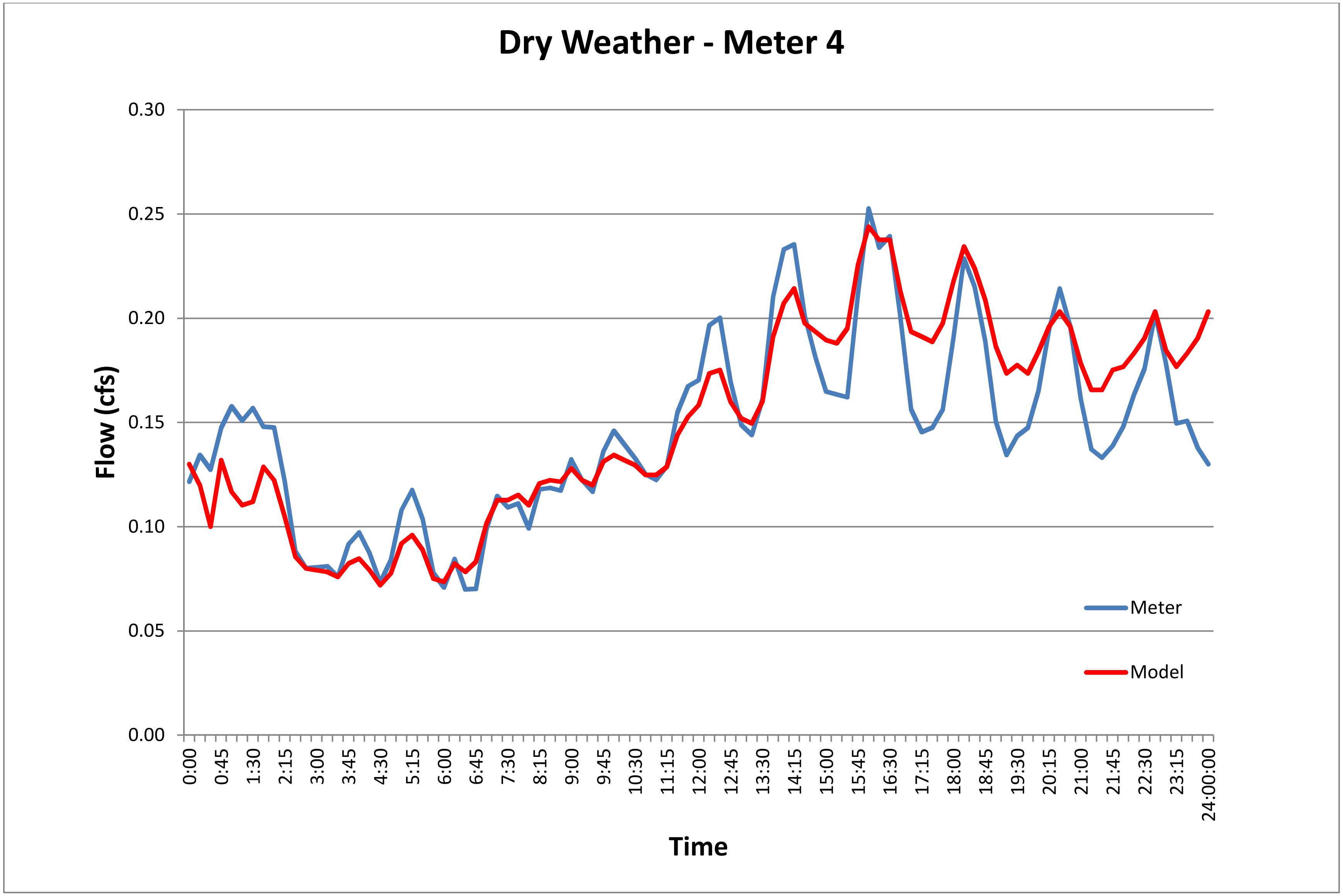

Dry Weather - Meter 2
Flow (cfs)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0:00
0:45
1:30
2:15
3:00
3:45
4:30
5:15
6:00
6:45
7:30
8:15
9:00
9:45
10:30
11:15
12:00
12:45
13:30
14:15
15:00
15:45
16:30
17:15
18:00
18:45
19:30
20:15
21:00
21:45
22:30
23:15
24:00:00
Time
Meter
Model



Dry Weather - Meter 3
Flow (cfs)
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
0:00
0:45
1:30
2:15
3:00
3:45
4:30
5:15
6:00
6:45
7:30
8:15
9:00
9:45
10:30
11:15
12:00
12:45
13:30
14:15
15:00
15:45
16:30
17:15
18:00
18:45
19:30
20:15
21:00
21:45
22:30
23:15
24:00:00
Time
Meter
Model



Dry Weather - Meter 4
Flow (cfs)
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0:00
0:45
1:30
2:15
3:00
3:45
4:30
5:15
6:00
6:45
7:30
8:15
9:00
9:45
10:30
11:15
12:00
12:45
13:30
14:15
15:00
15:45
16:30
17:15
18:00
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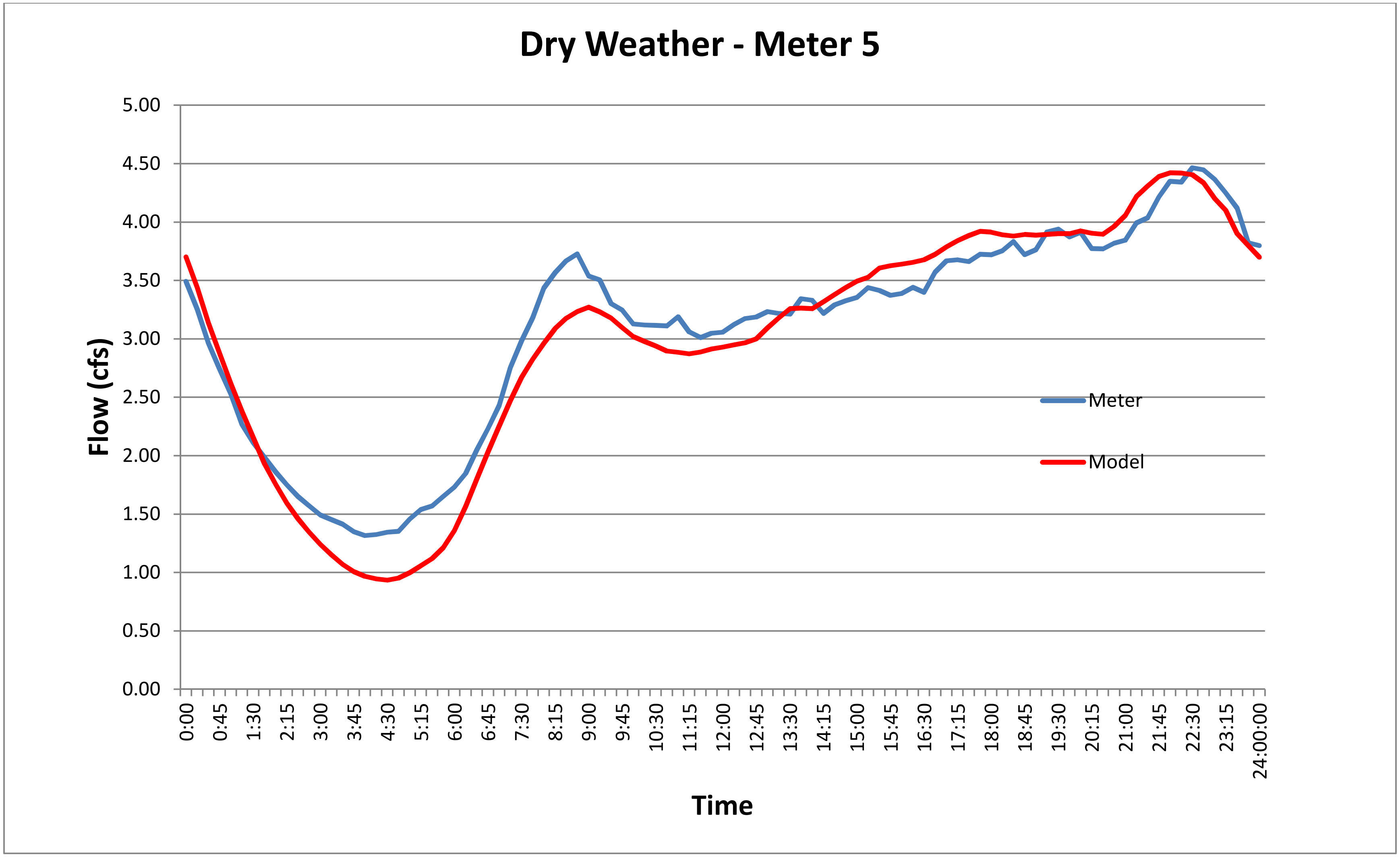
19:30
20:15
21:00
21:45
22:30
23:15
24:00:00
Time
Meter
Model


Dry Weather - Meter 5
Flow (cfs)
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0:00
0:45
1:30
2:15
3:00
3:45
4:30
5:15
6:00
6:45
7:30
8:15
9:00
9:45
10:30
11:15
12:00
12:45
13:30
14:15
15:00
15:45
16:30
17:15
18:00
18:45
19:30
20:15
21:00
21:45
22:30
23:15
24:00:00
Time
Meter
Model

# Appendix B-I

# Existing Results d/D

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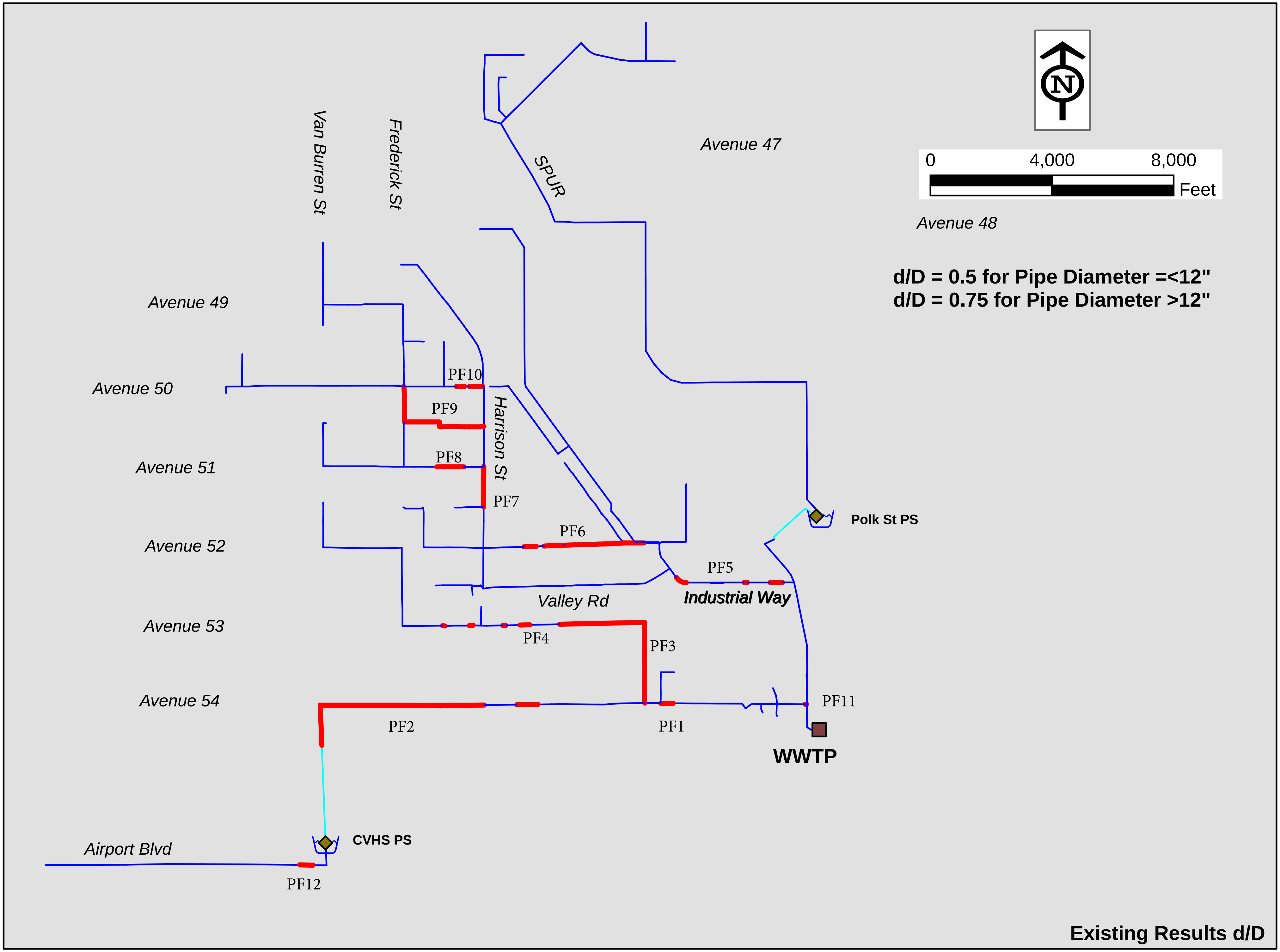
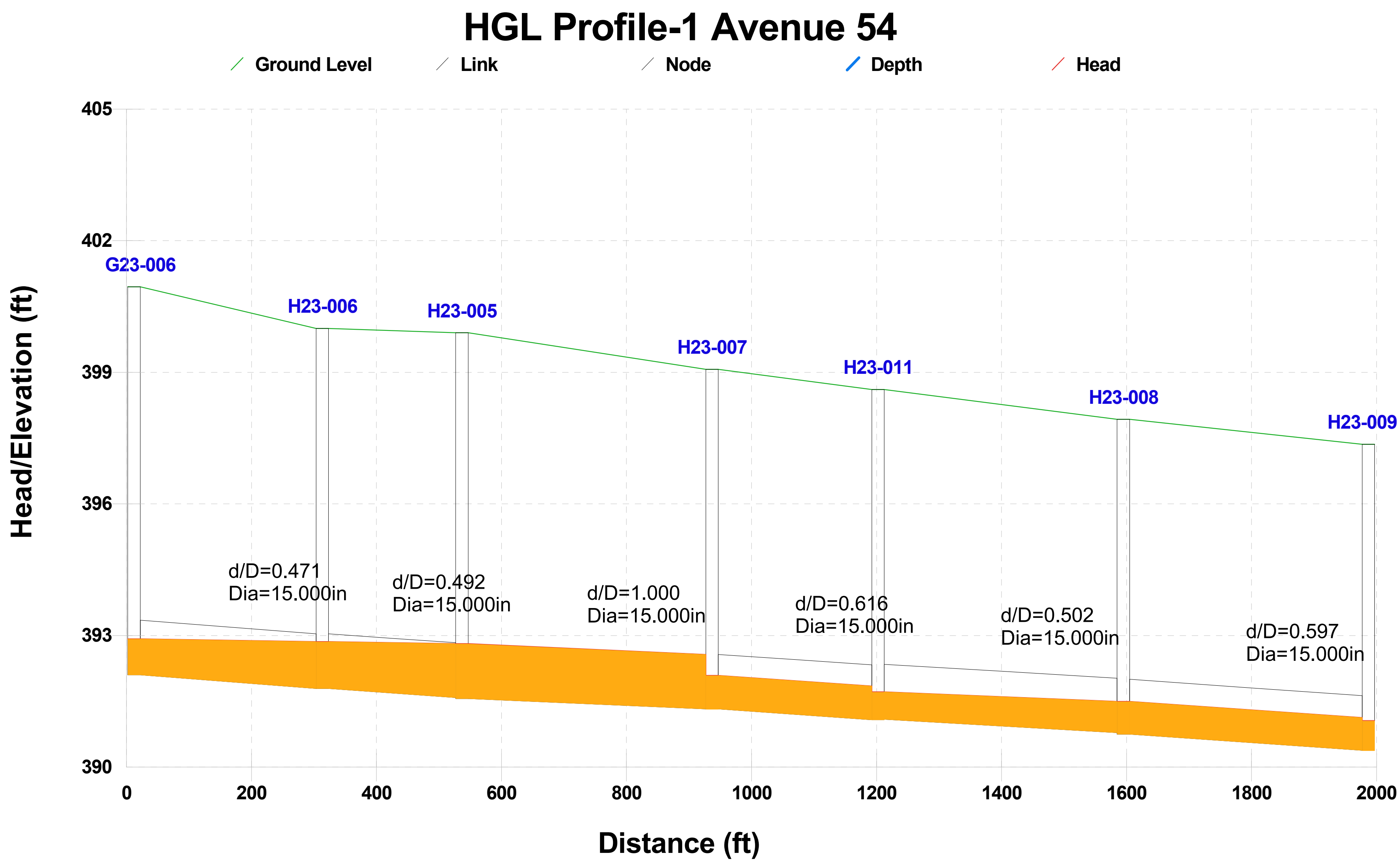
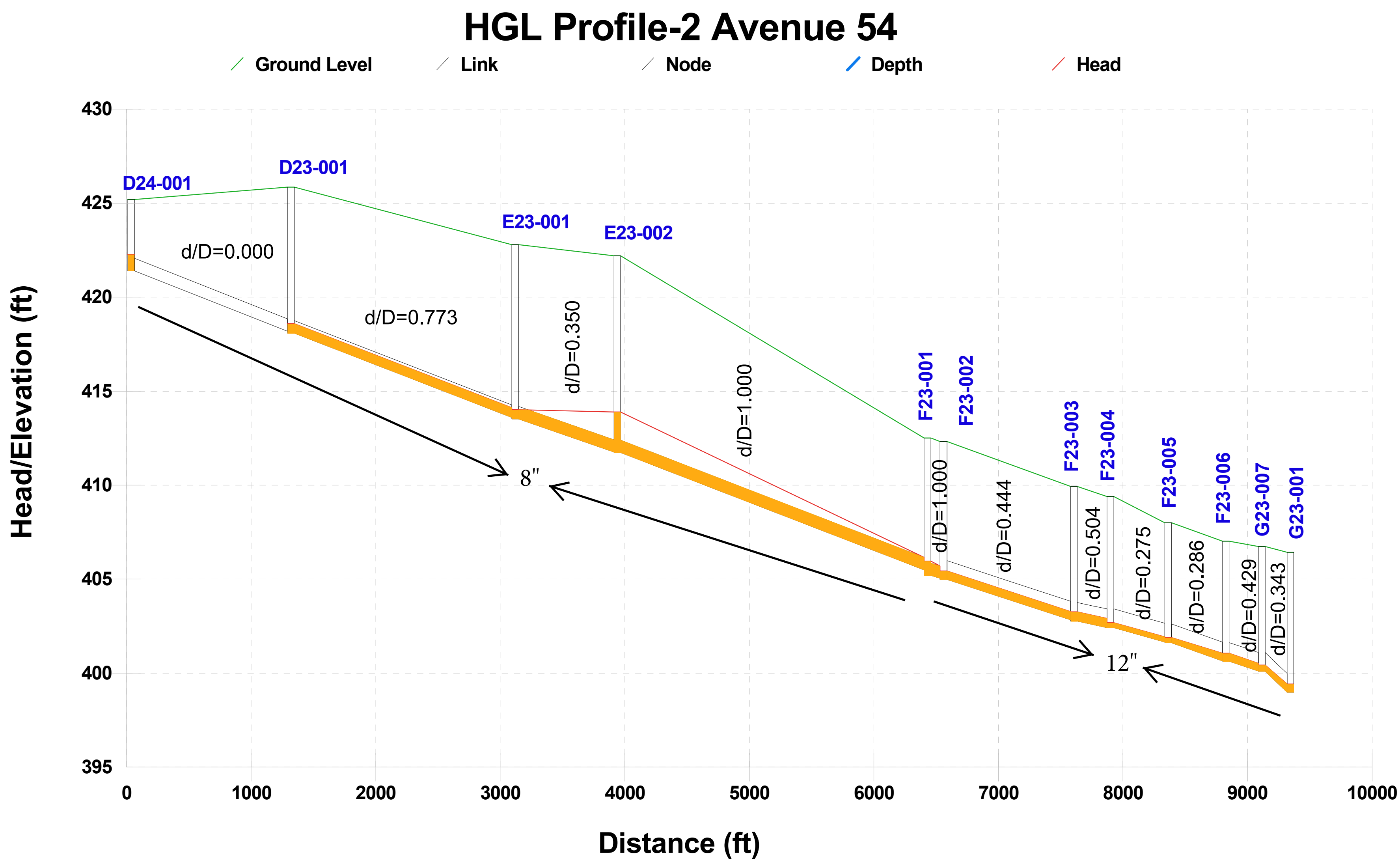


N
0
4,000
8,000
Feet
d/D = 0.5 for Pipe Diameter =<12"
d/D = 0.75 for Pipe Diameter >12"
Van Burren St
Frederick St
SPUR
Avenue 47
Avenue 48
Avenue 49
Avenue 50
Avenue 51
Avenue 52
Avenue 53
Avenue 54
Harrison St
Valley Rd
Industrial Way
Airport Blvd
Polk St PS
CVHS PS
WWTP
PF1
PF2
PF3
PF4
PF5
PF6
PF7
PF8
PF9
PF10
PF11
PF12
Existing Results d/D

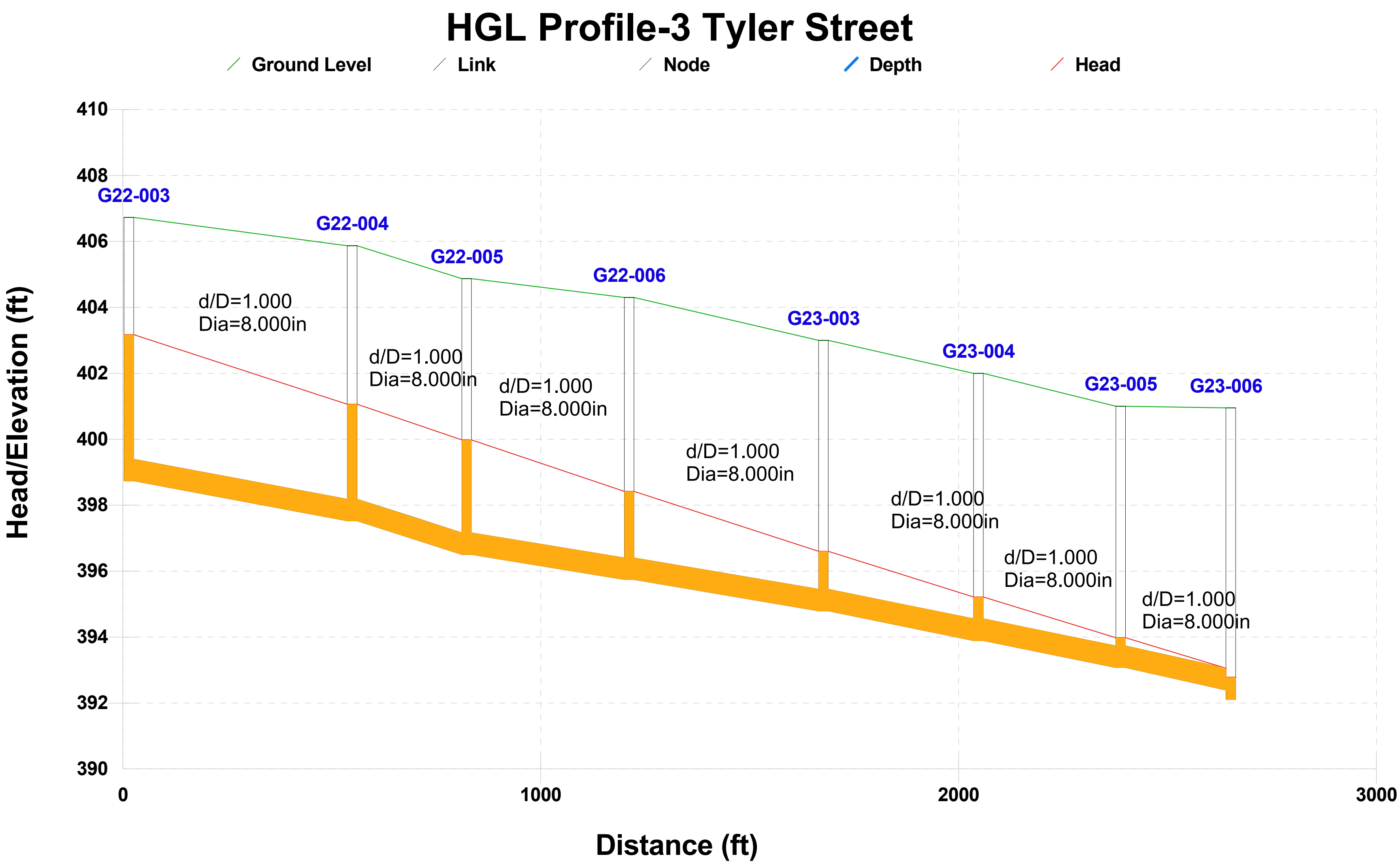
Scenario: Existing



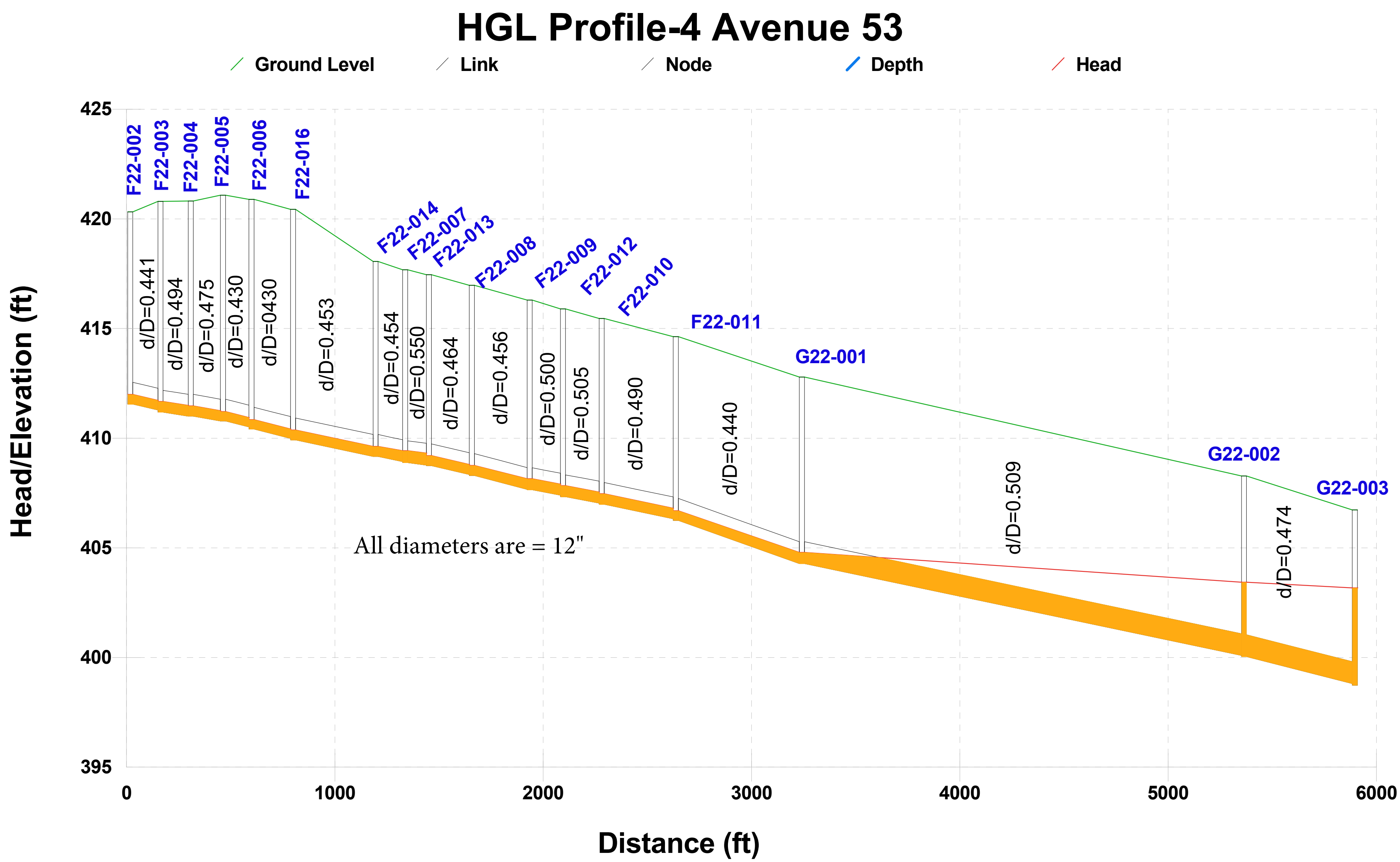
Scenario: Existing



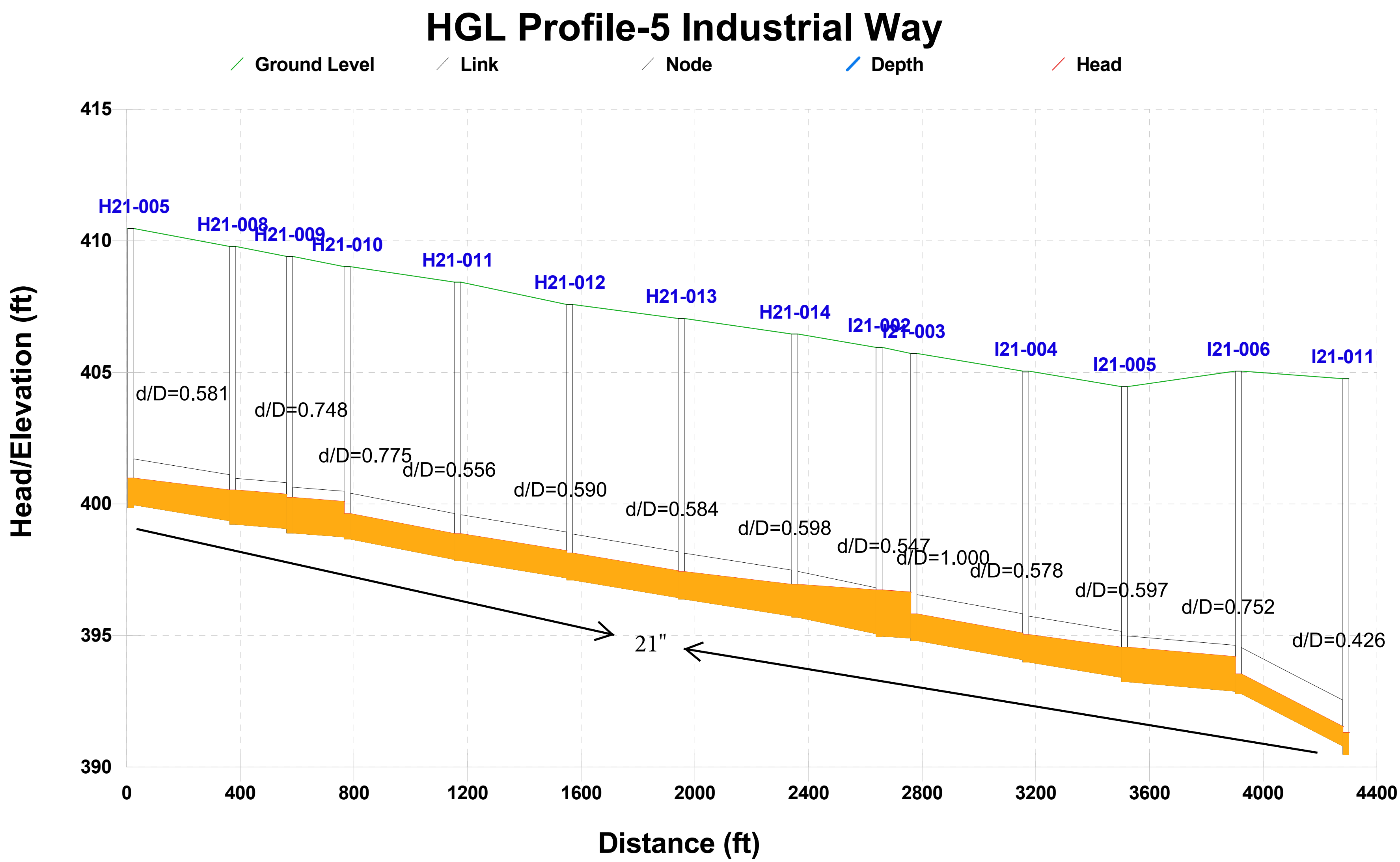
Scenario: Existing



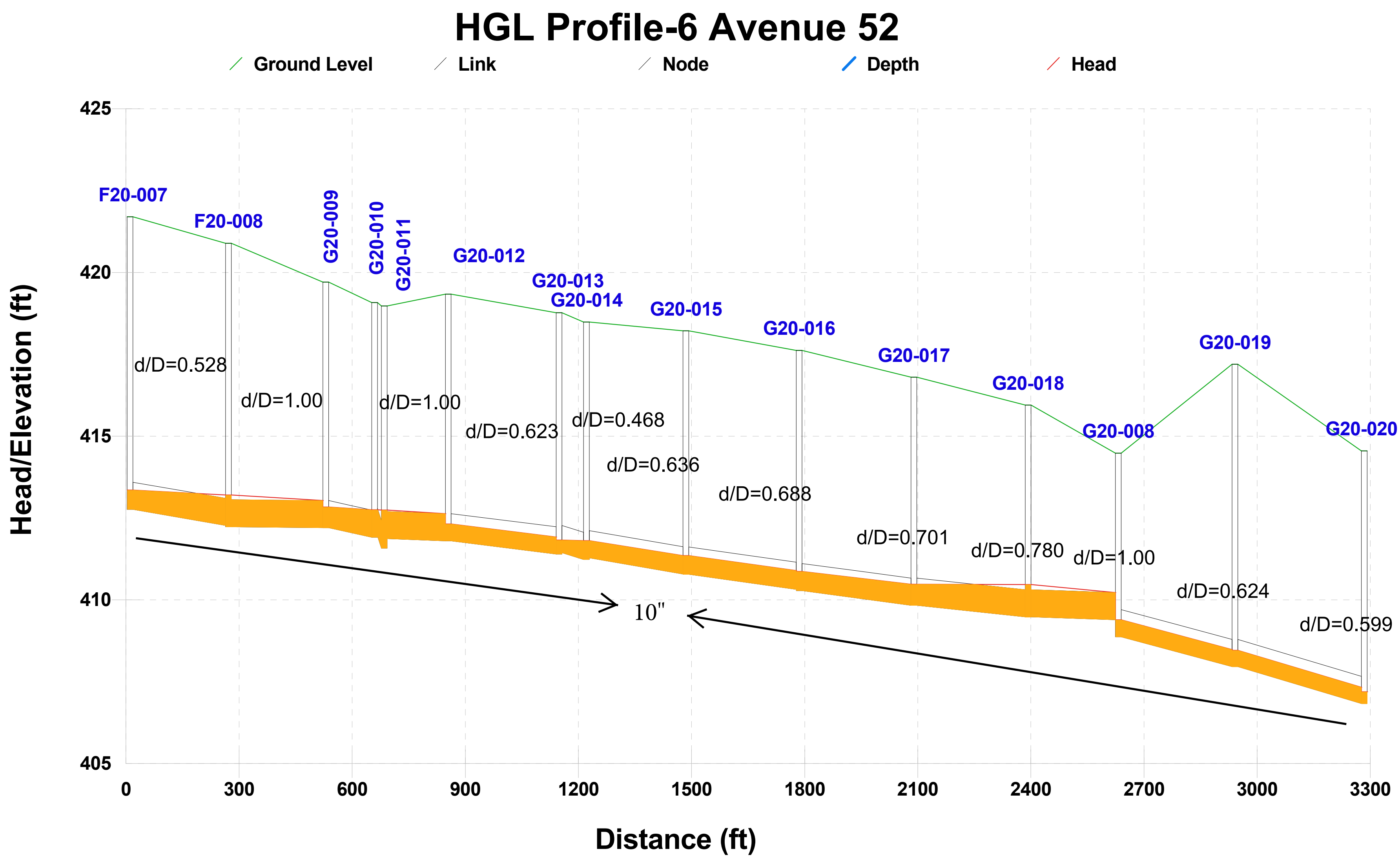
Scenario: Existing



**Scenario: Existing**

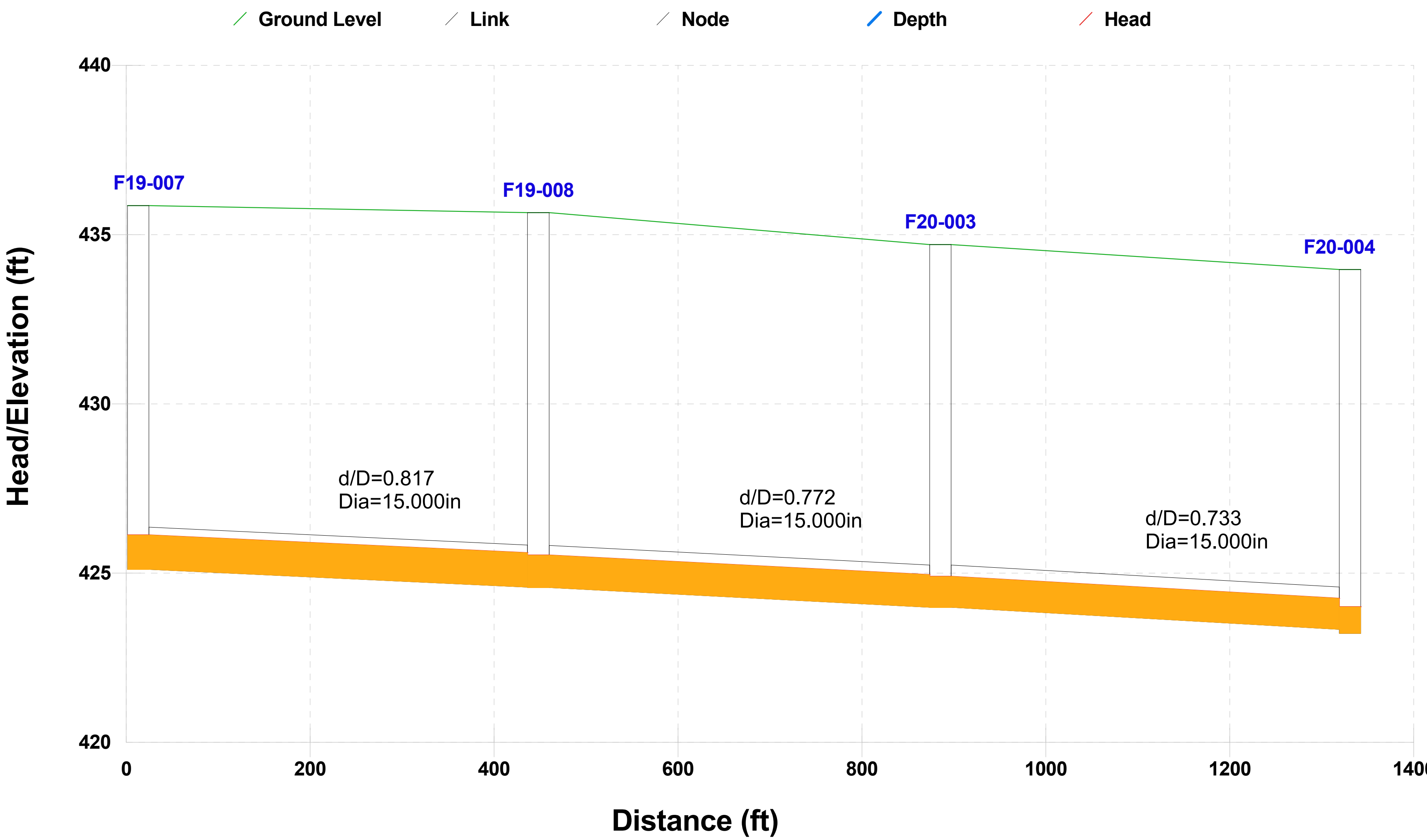


Scenario: Existing

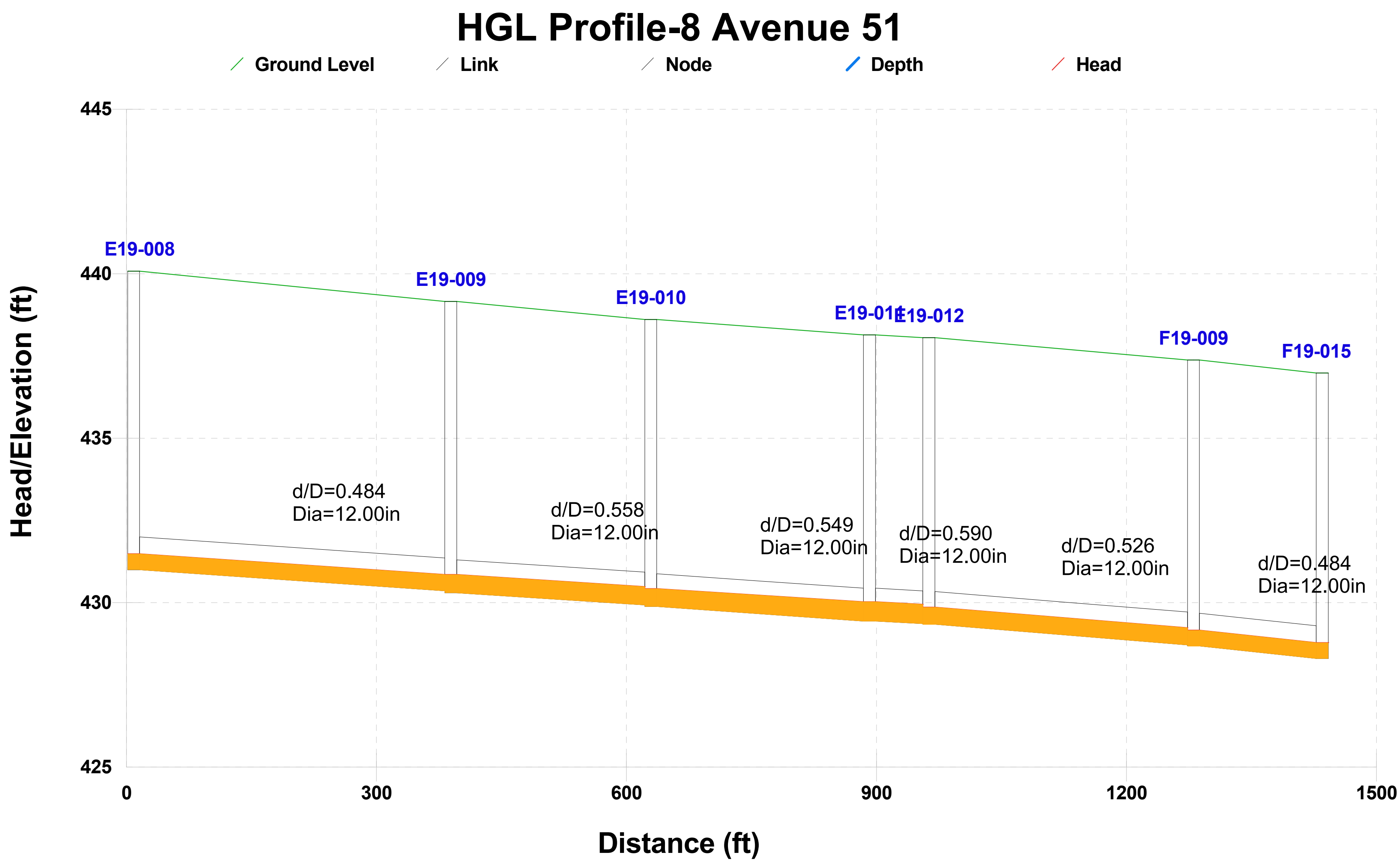


Scenario: Existing

# HGL Profile-7 Harrison Street

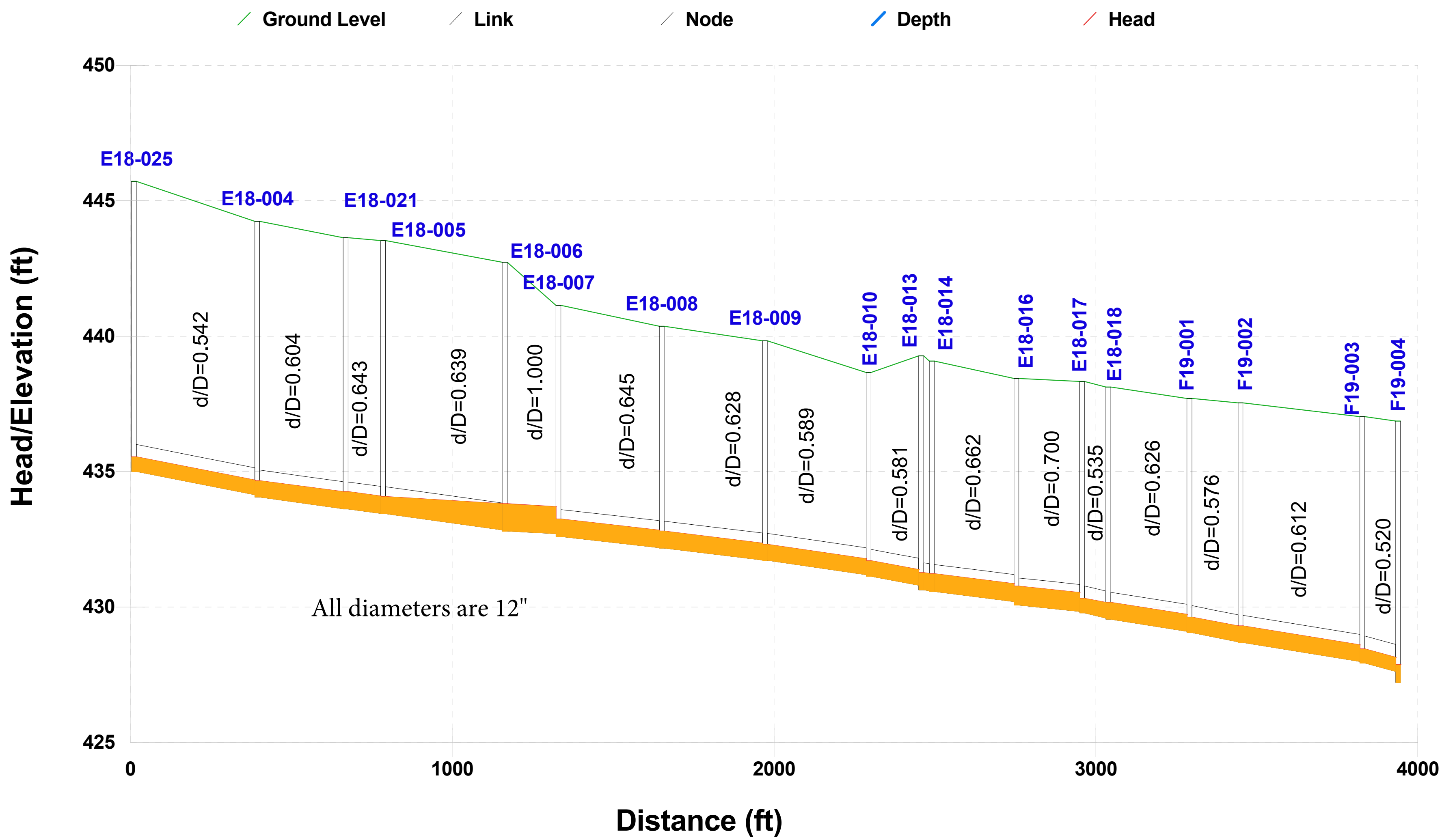


Scenario: Existing

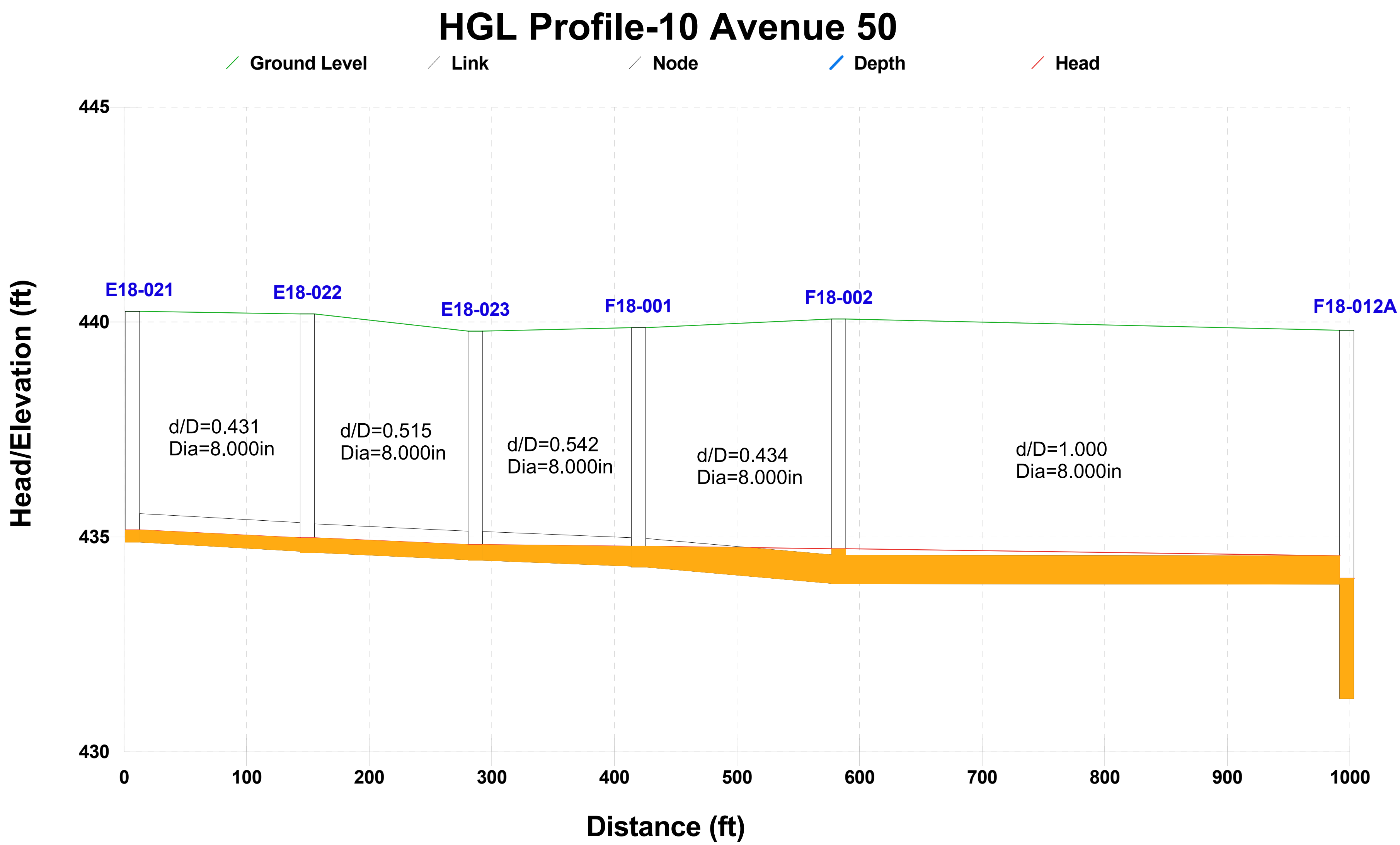


Scenario: Existing

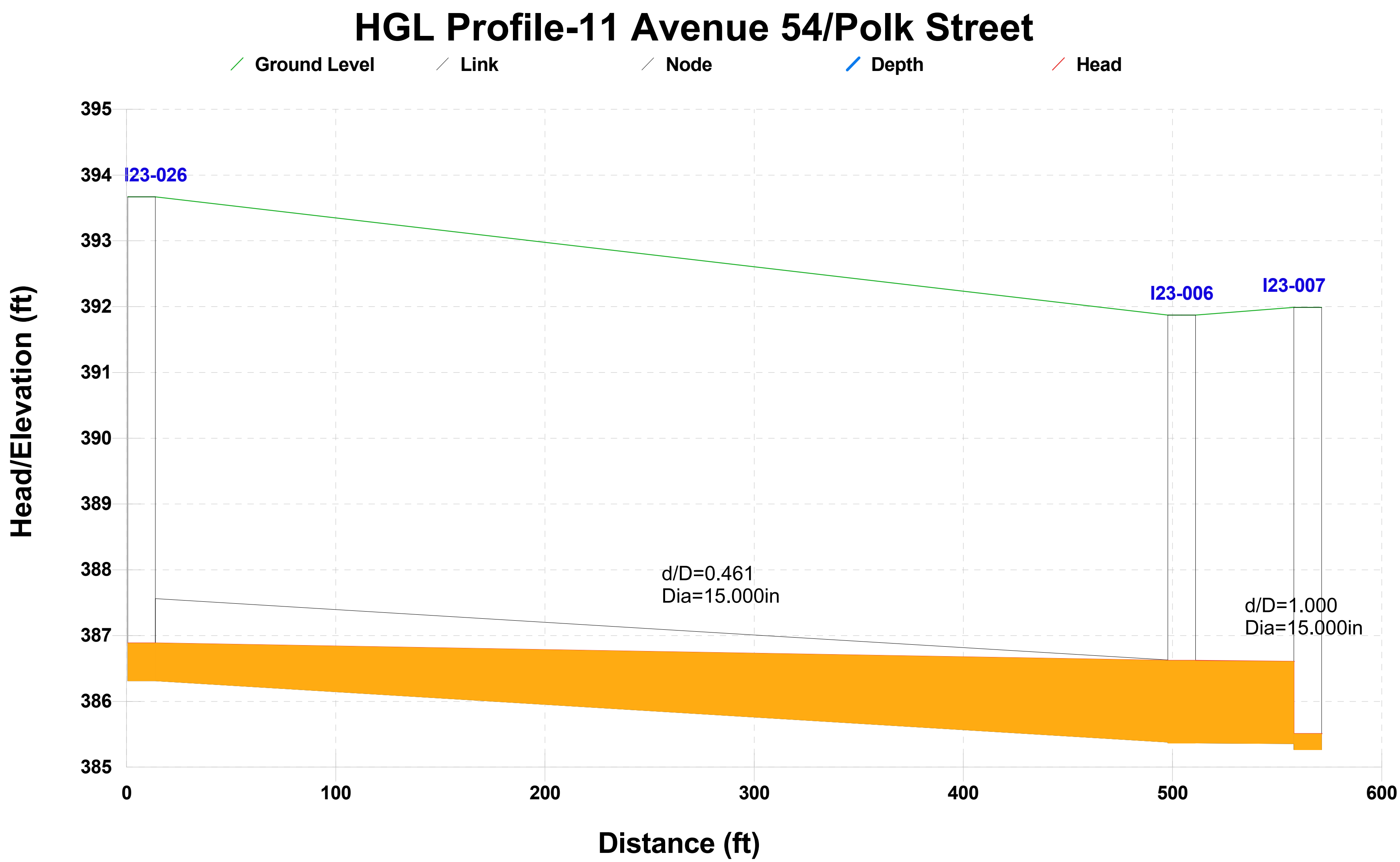
# HGL Profile-9 Westfield Way/Julia Dr./Frederick St.



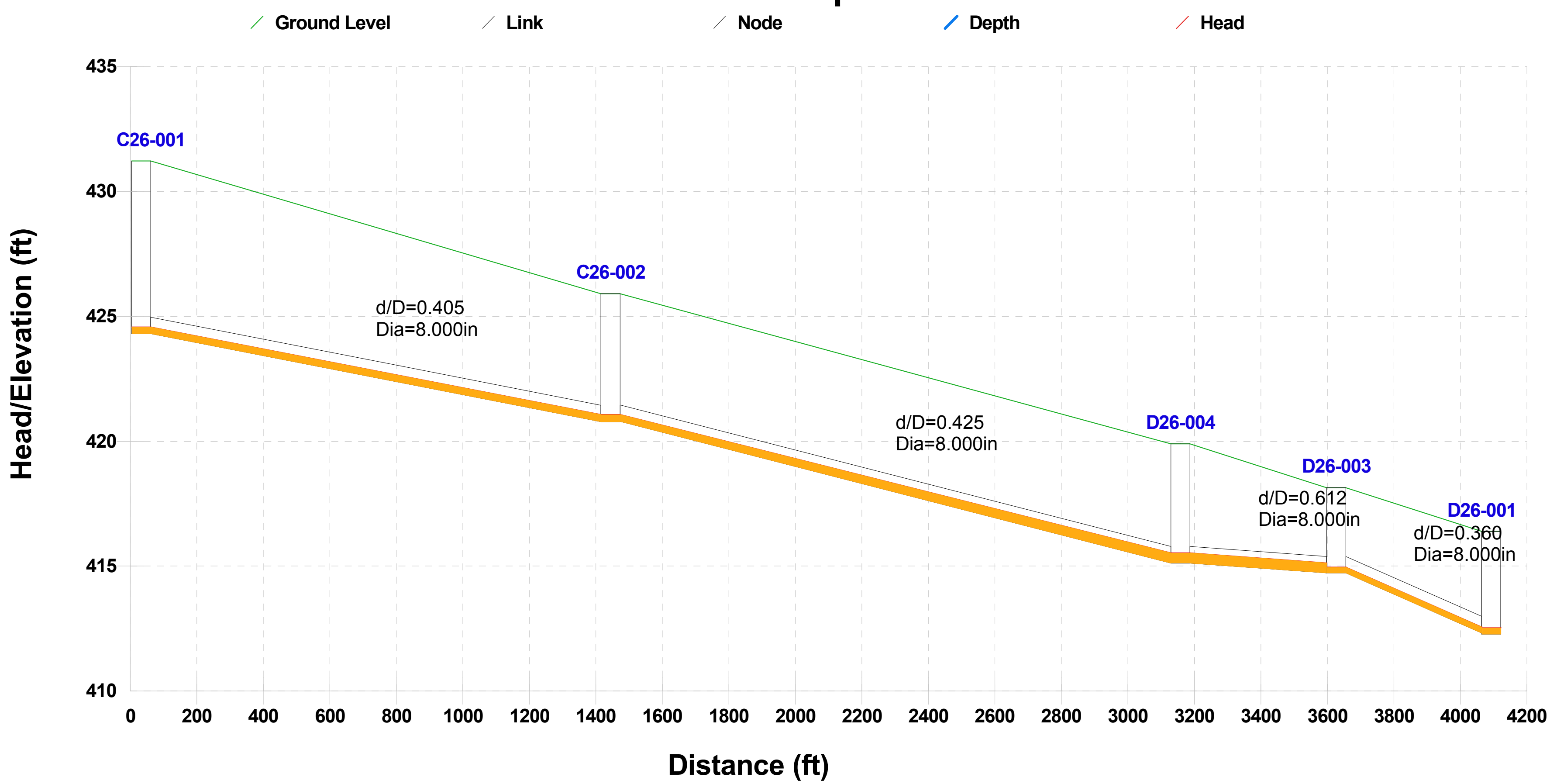
Scenario: Existing



Scenario: Existing



# HGL Profile-12 Airport Blvd



# Appendix B-II

# Intermediate Results d/D

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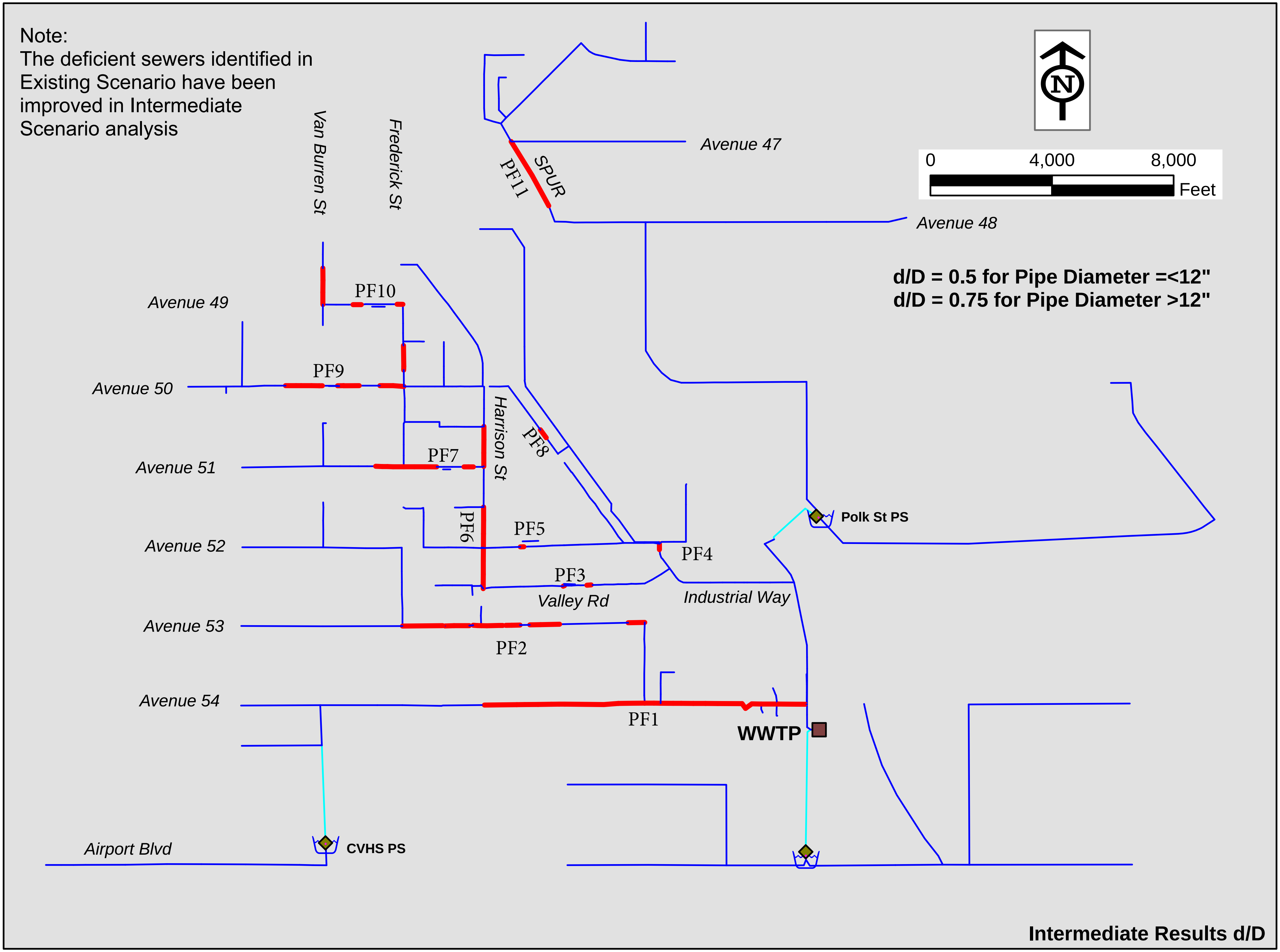
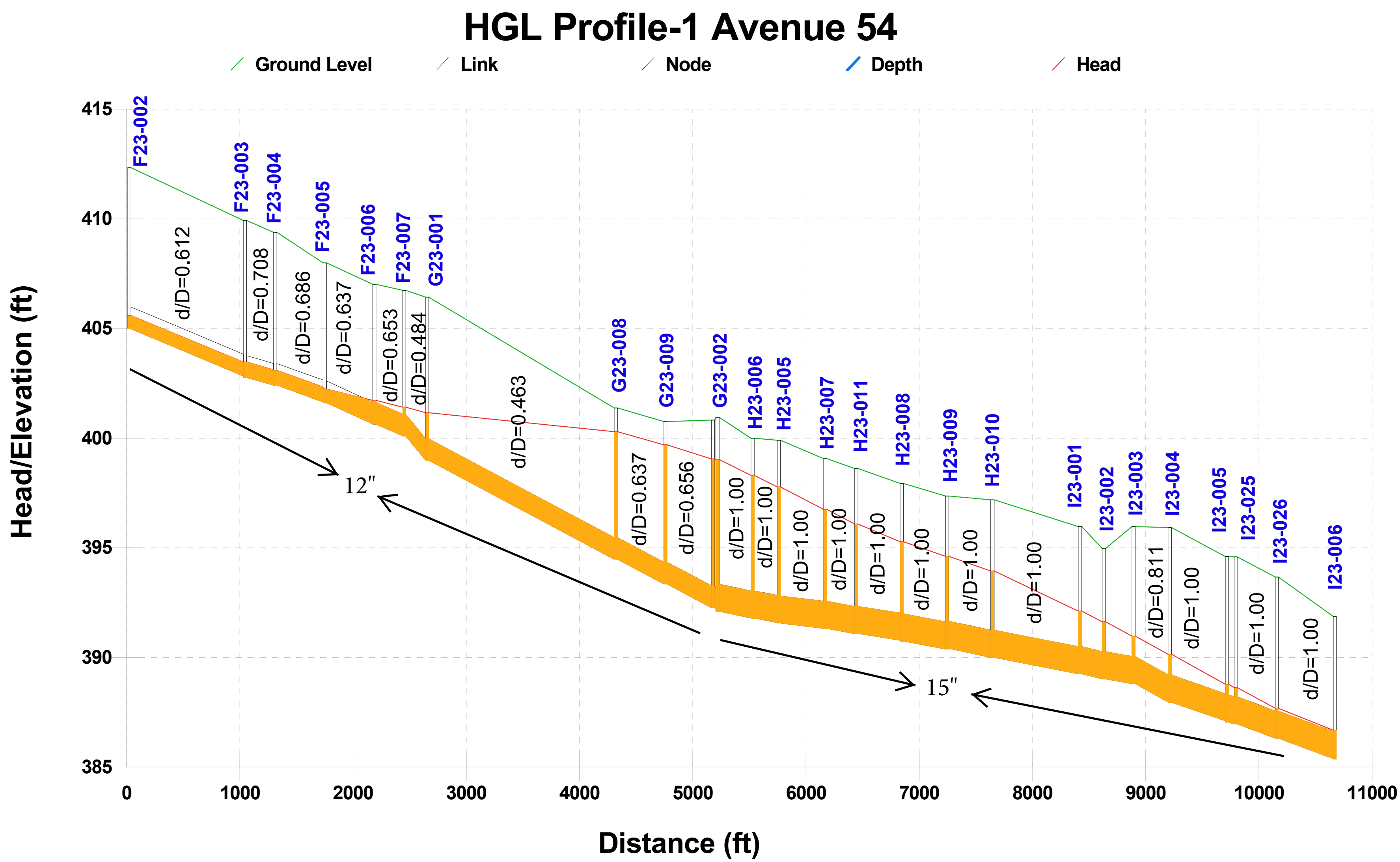
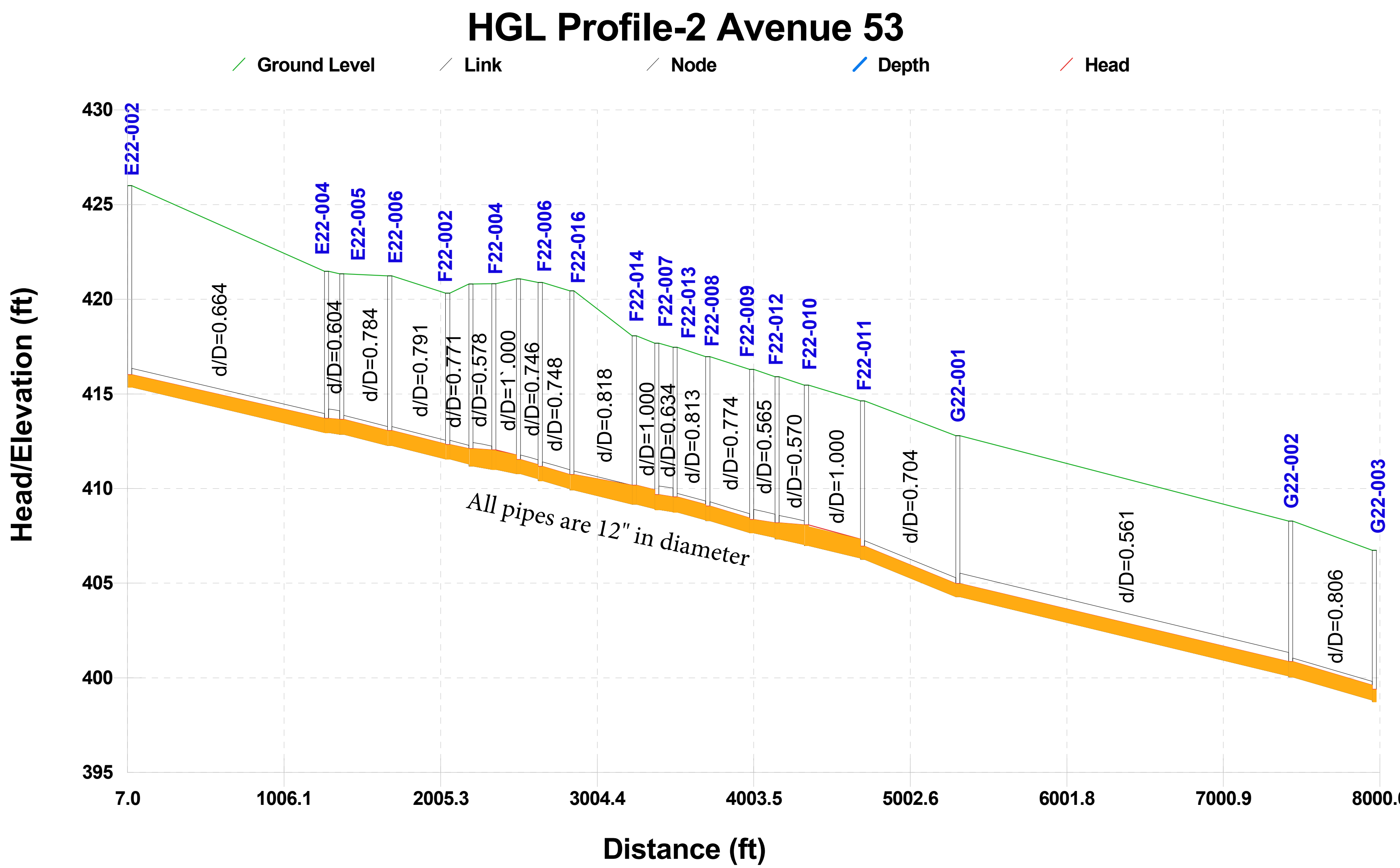

Note:
The deficient sewers identified in Existing Scenario have been improved in Intermediate Scenario analysis
N
0
4,000
8,000
Feet
d/D = 0.5 for Pipe Diameter =<12"
d/D = 0.75 for Pipe Diameter >12"
Van Burren St
Frederick St
Harrison St
Avenue 47
Avenue 48
Avenue 49
Avenue 50
Avenue 51
Avenue 52
Avenue 53
Avenue 54
Airport Blvd
Valley Rd
Industrial Way
SPUR
PF11
PF10
PF9
PF8
PF7
PF6
PF5
PF4
PF3
PF2
PF1
Polk St PS
WWTP
CVHS PS
Intermediate Results d/D

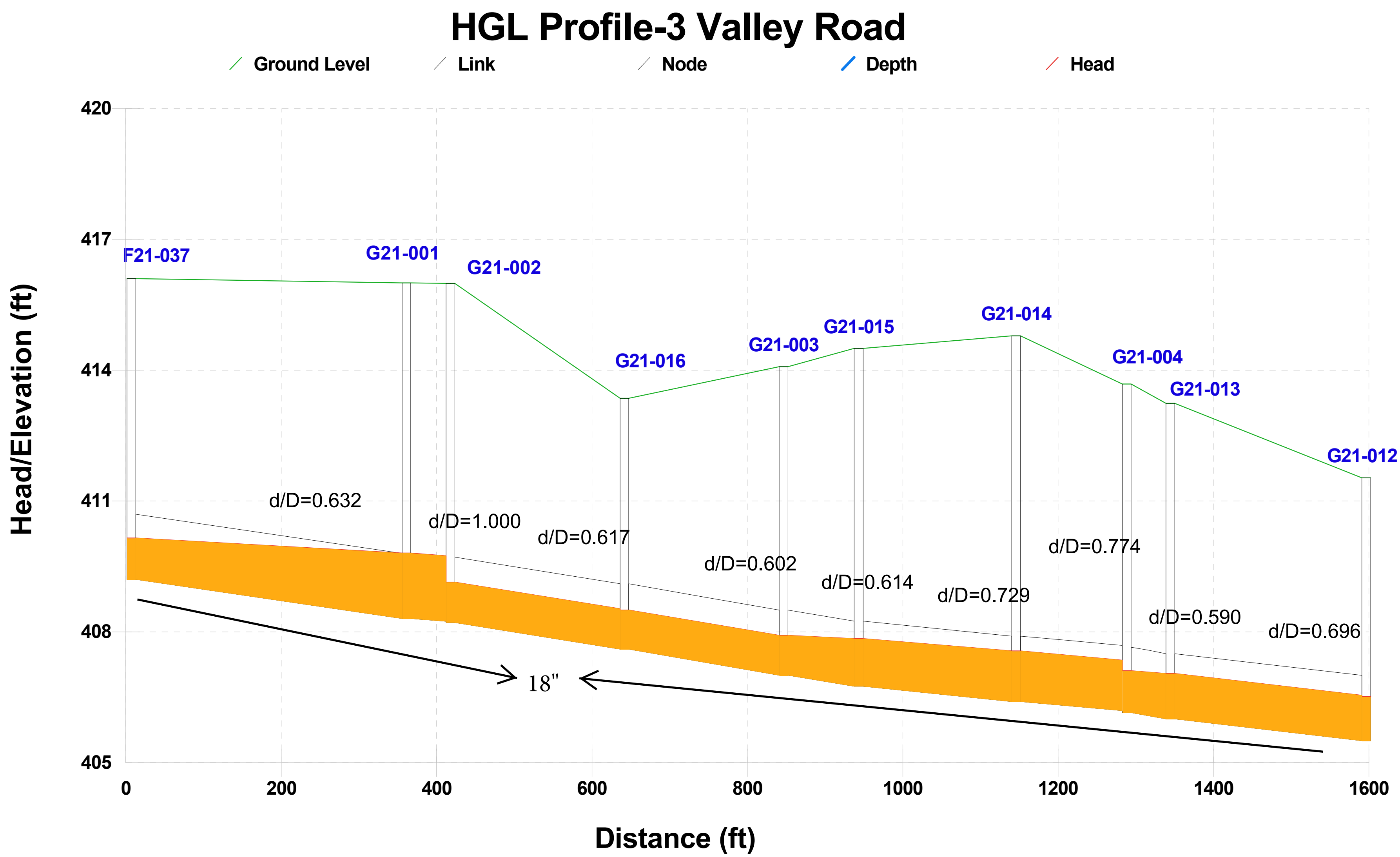
Scenario: Intermediate



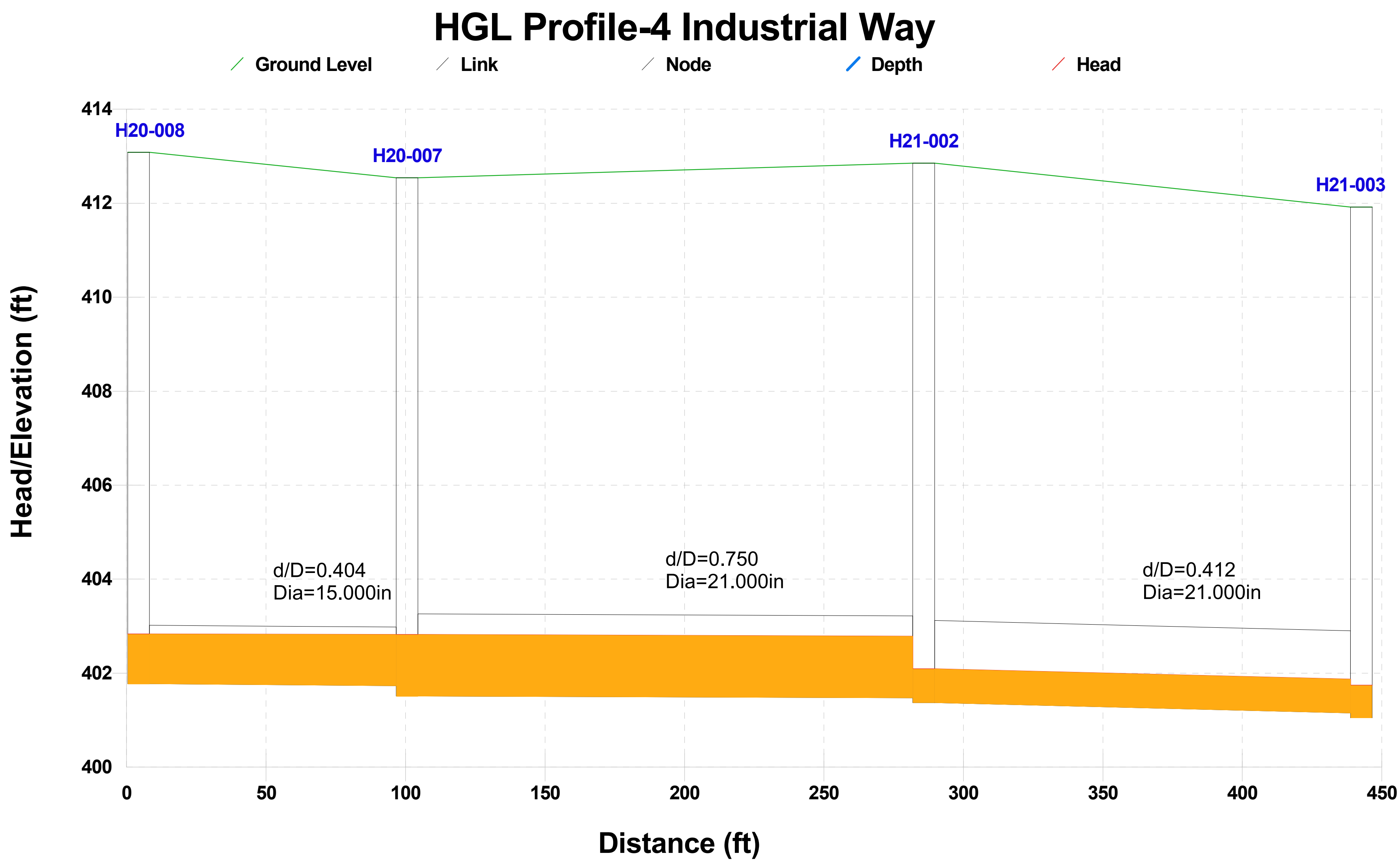
**Scenario: Intermediate**



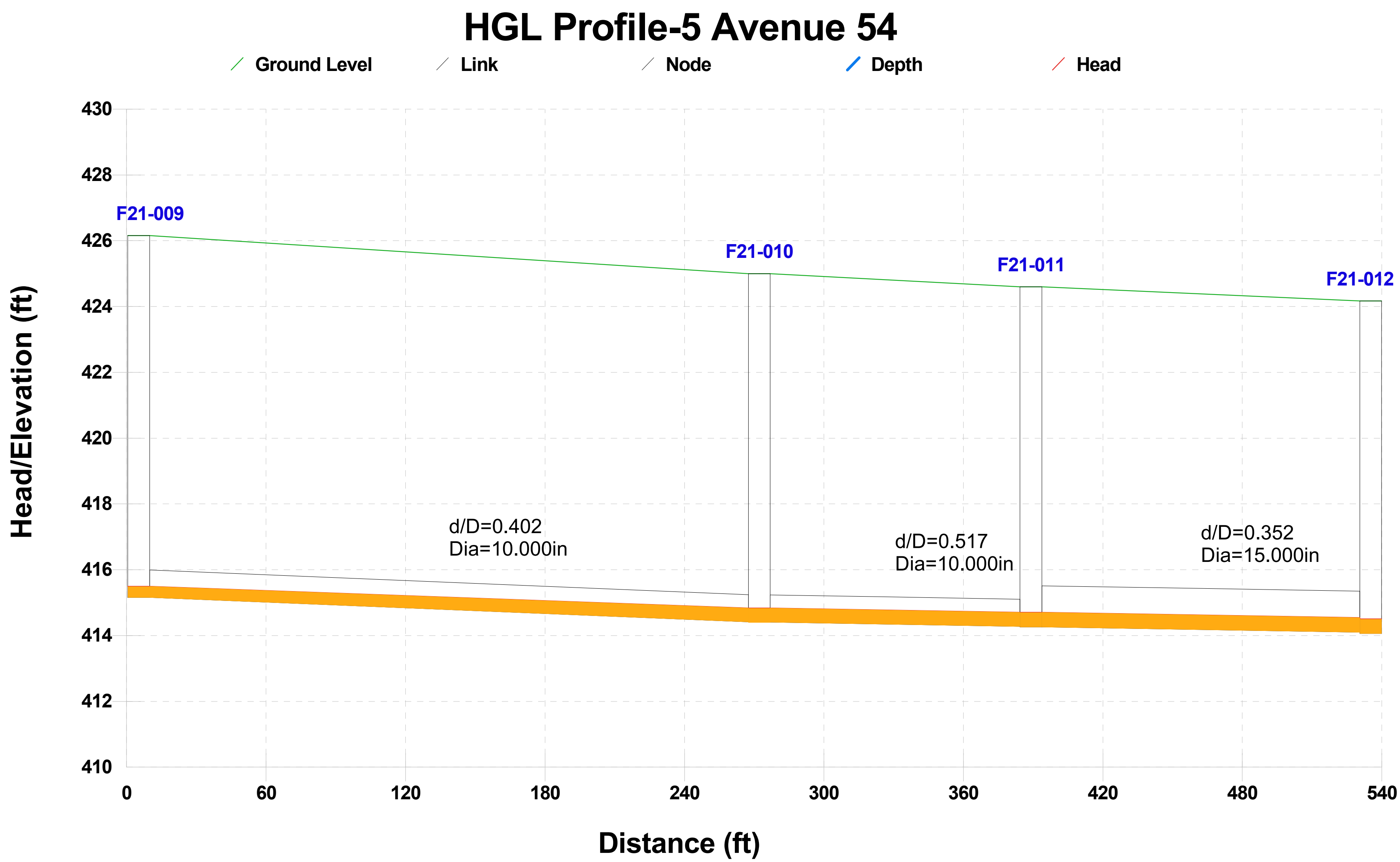
Scenario: Intermediate



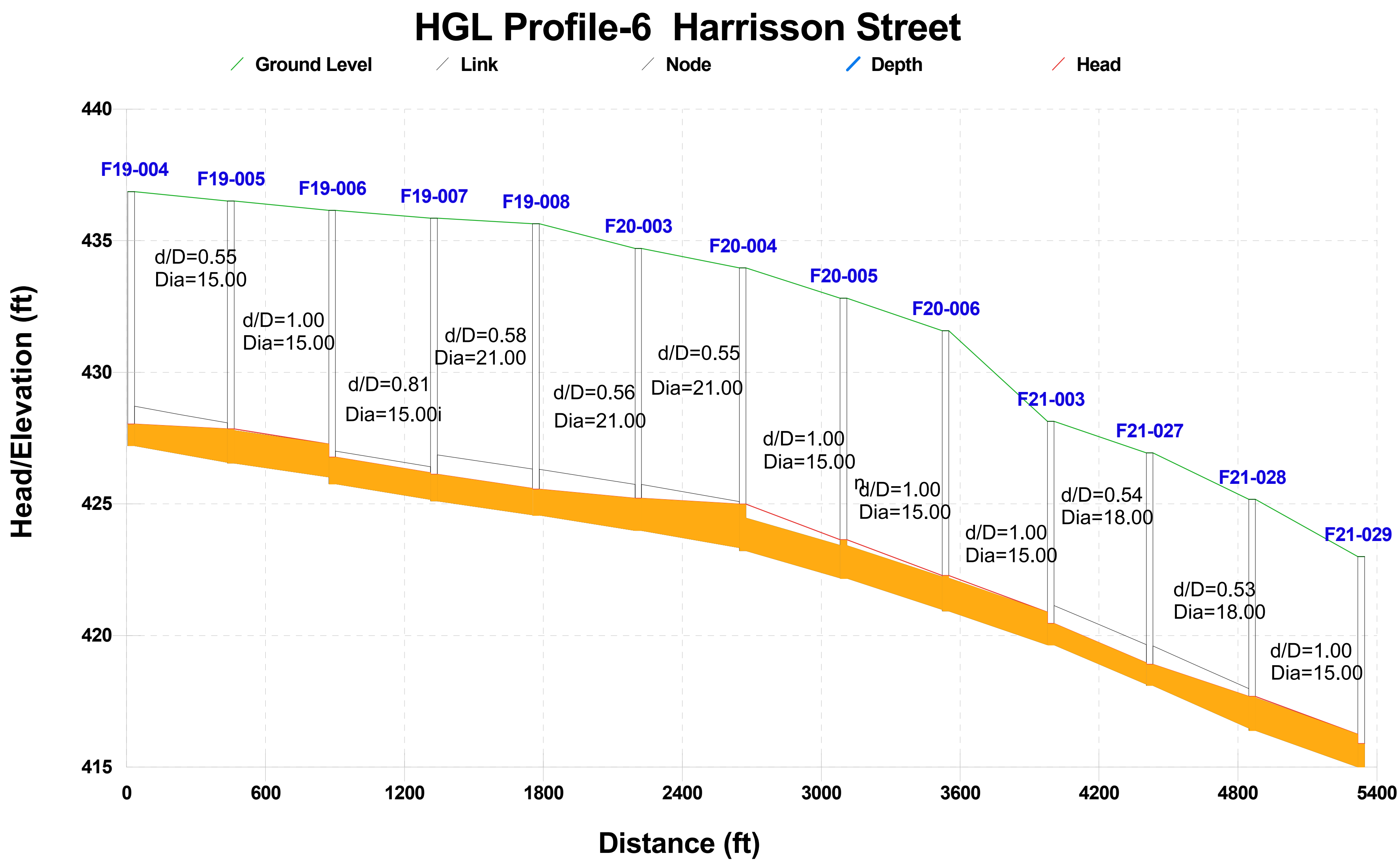
Scenario: Intermediate



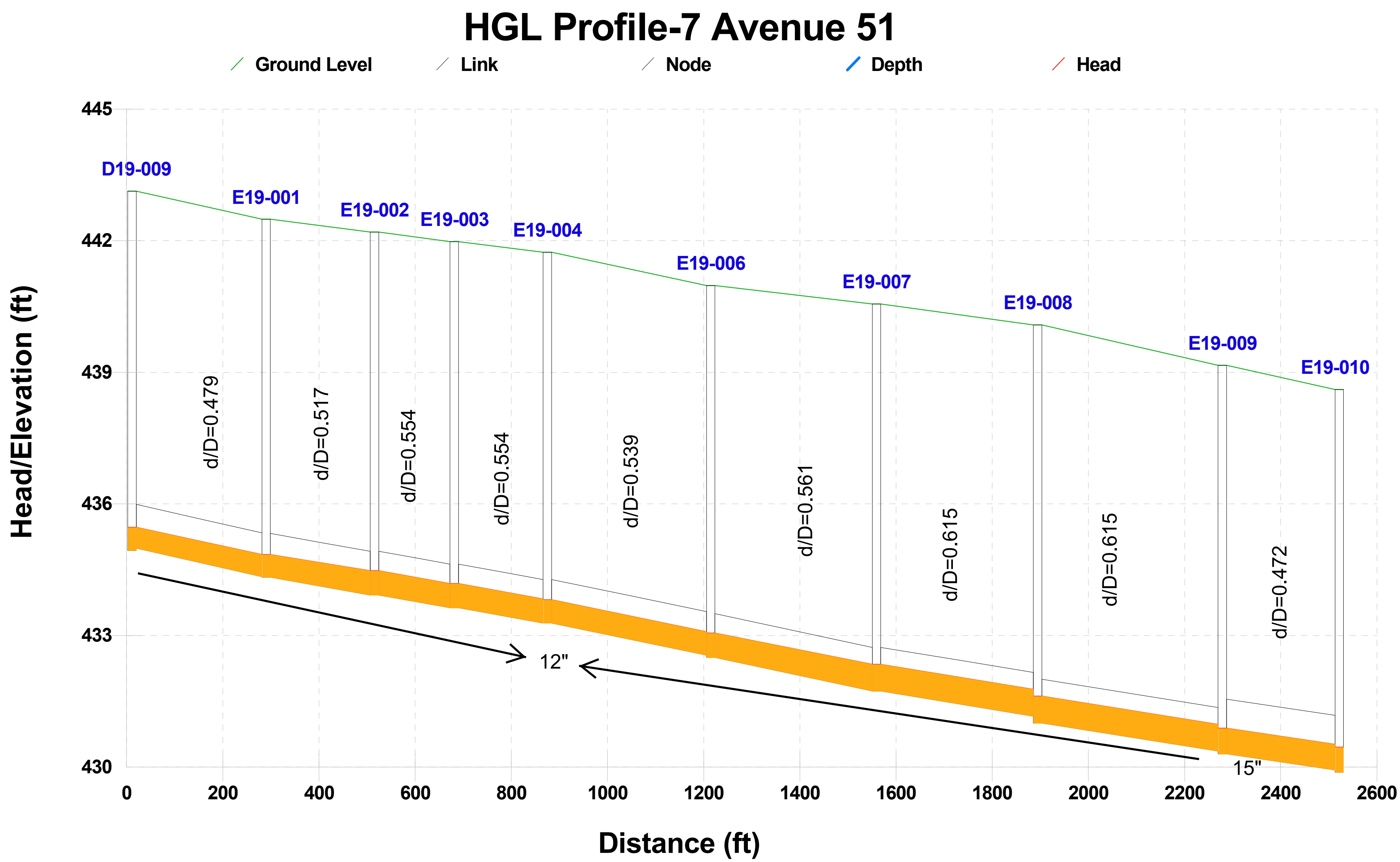
Scenario: Intermediate



**Scenario: Intermediate**

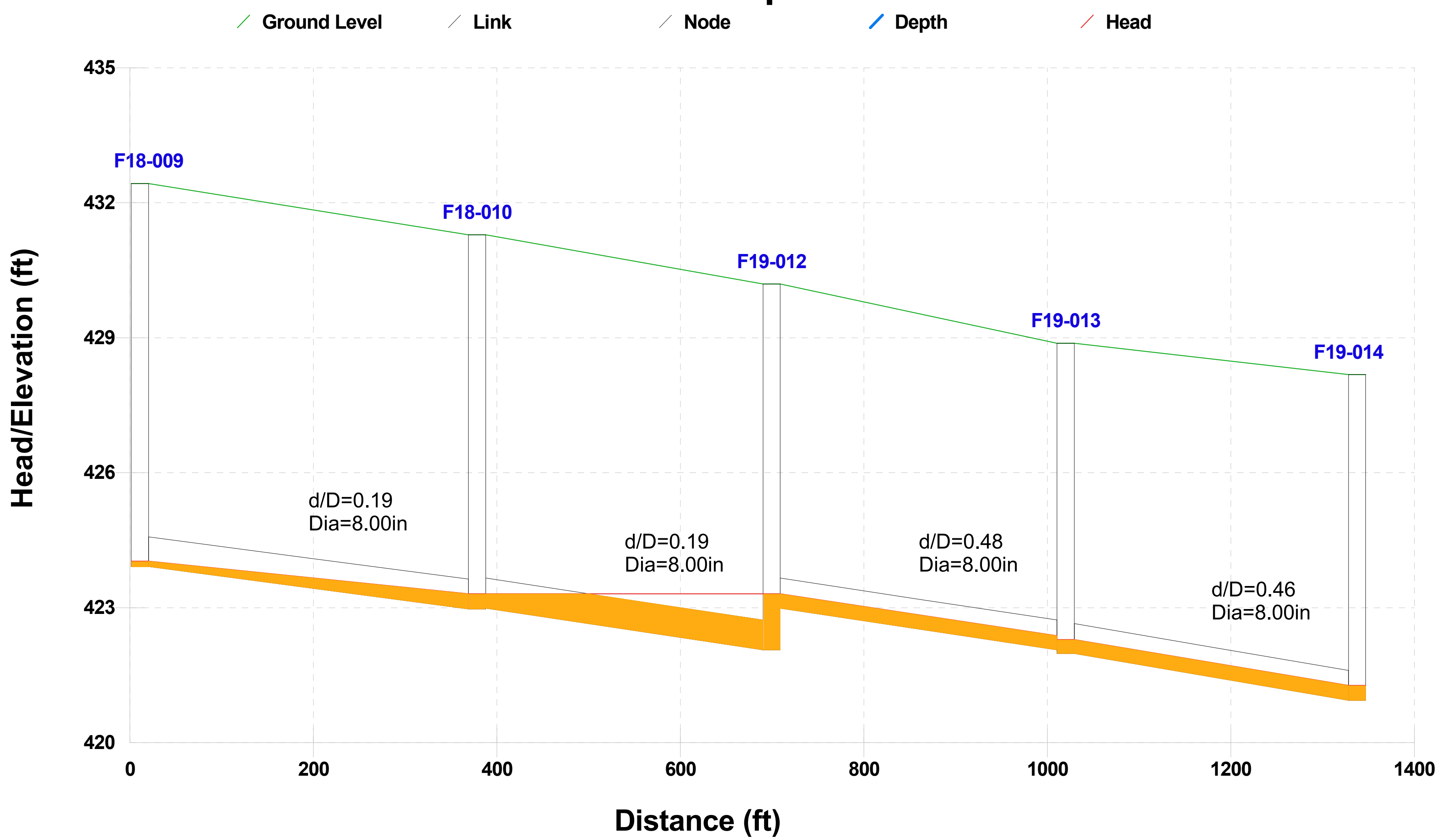


Scenario: Intermediate

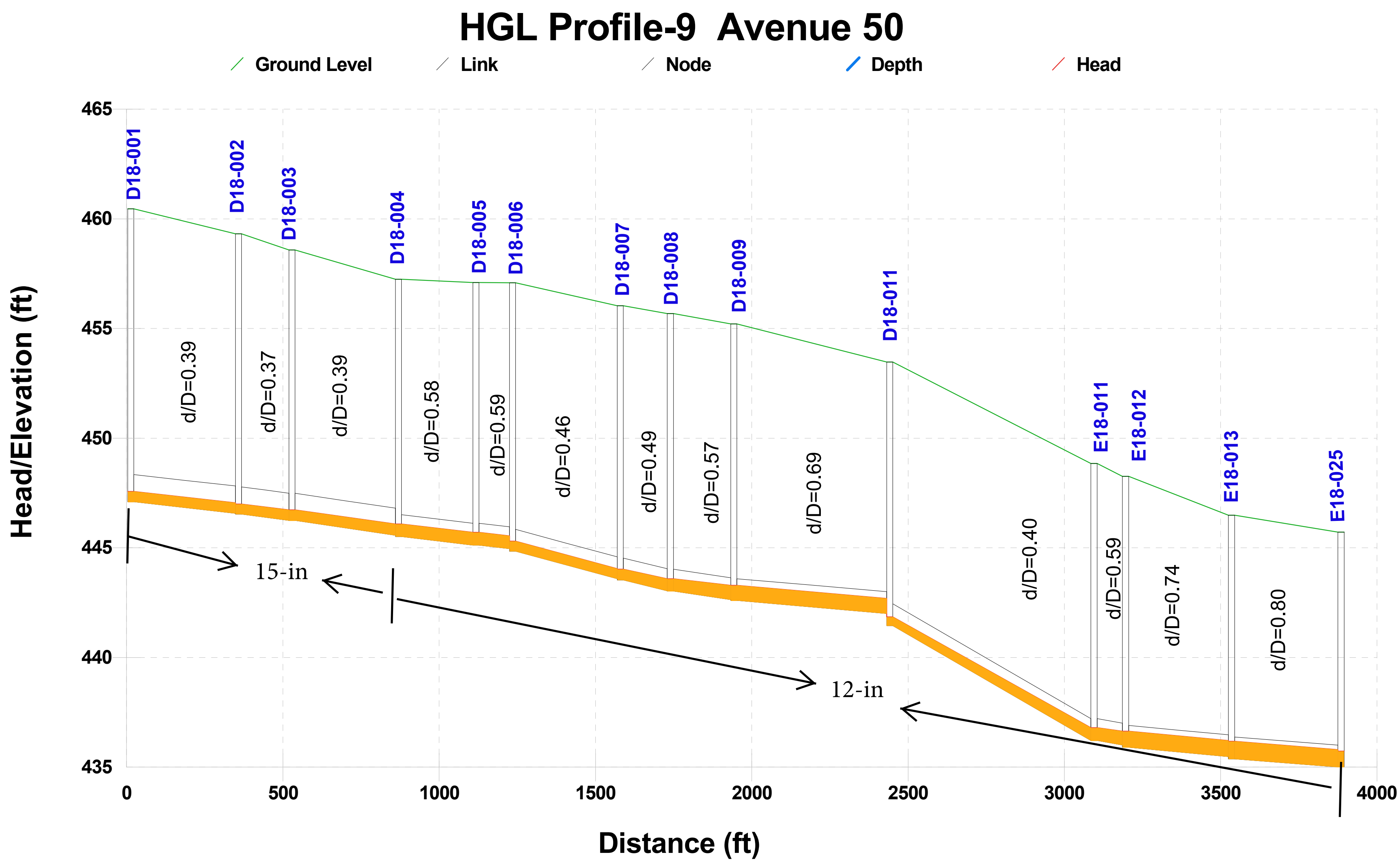


**Scenario: Intermediate**

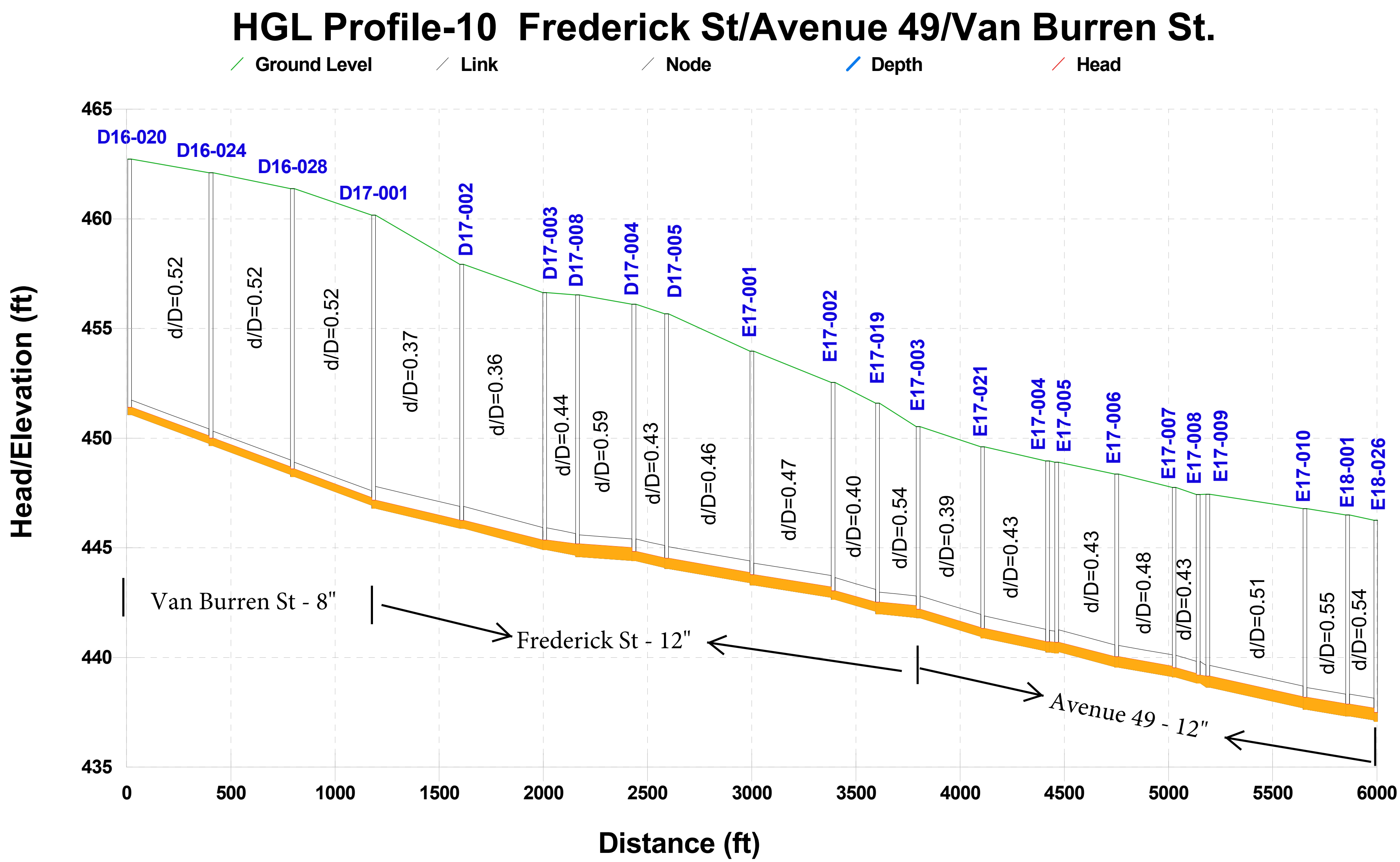
## HGL Profile-8 Grapefruit Blvd



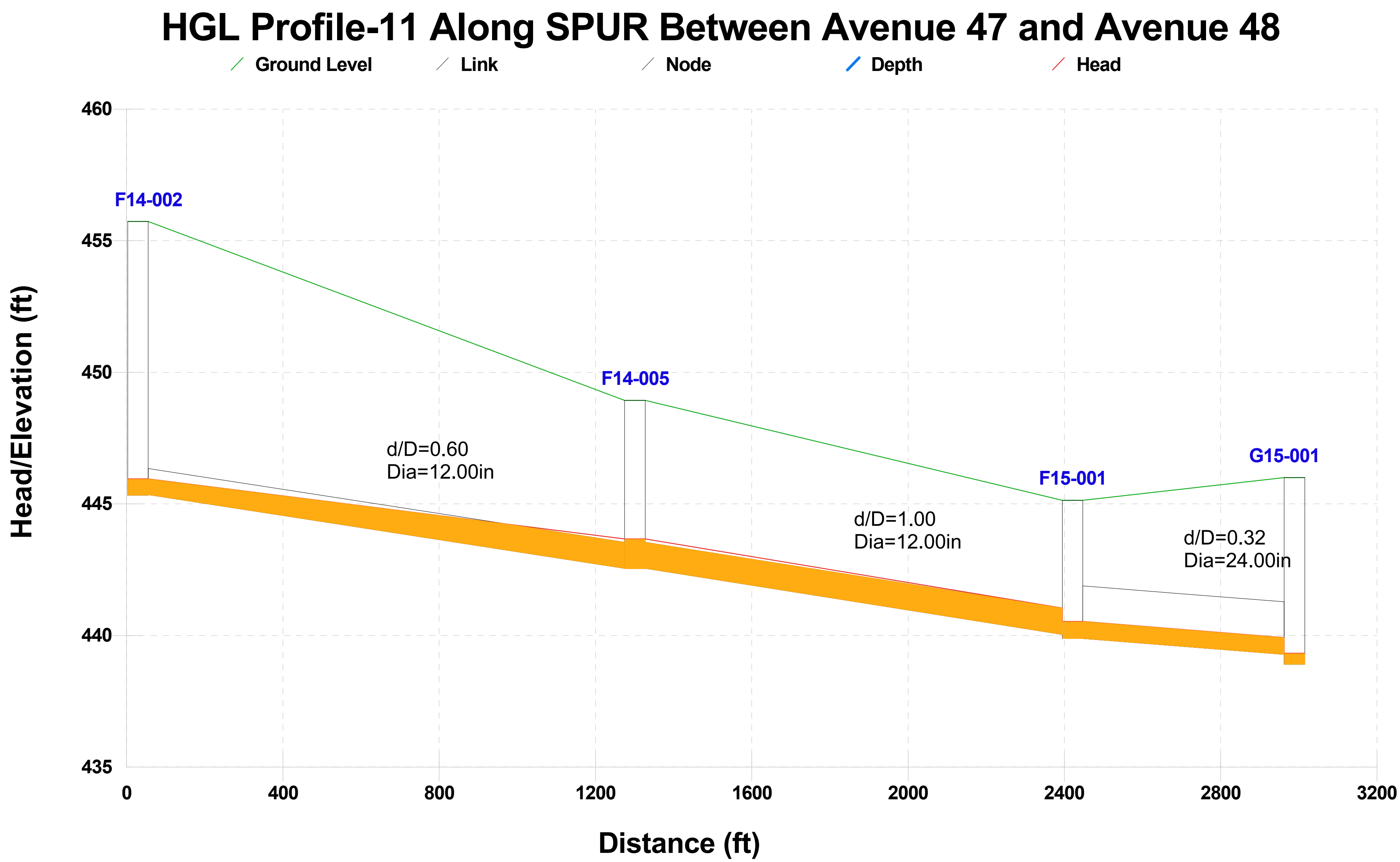
Scenario: Intermediate



Scenario: Intermediate



Scenario: Intermediate



# Appendix B-III

# 2040 Results d/D

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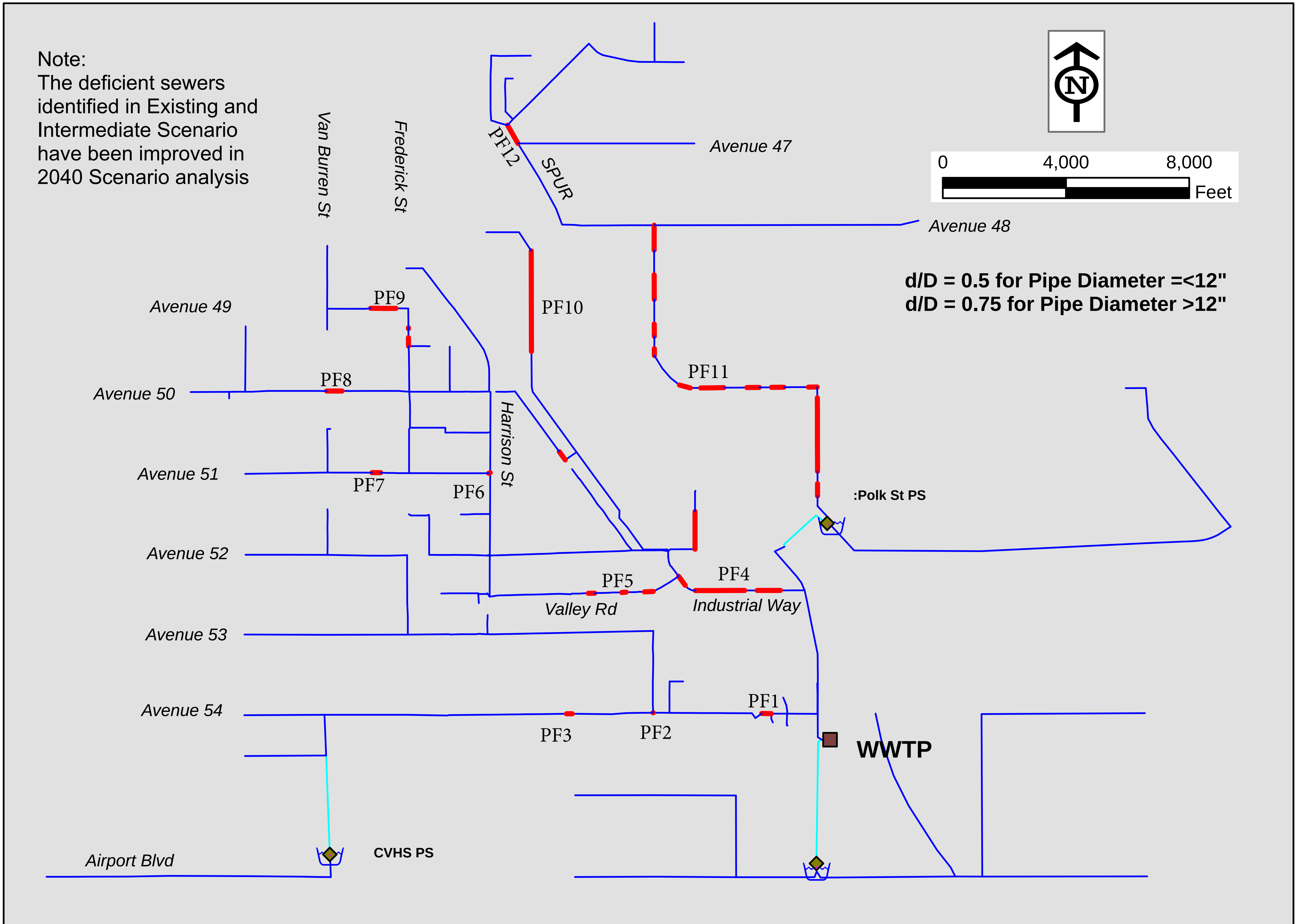
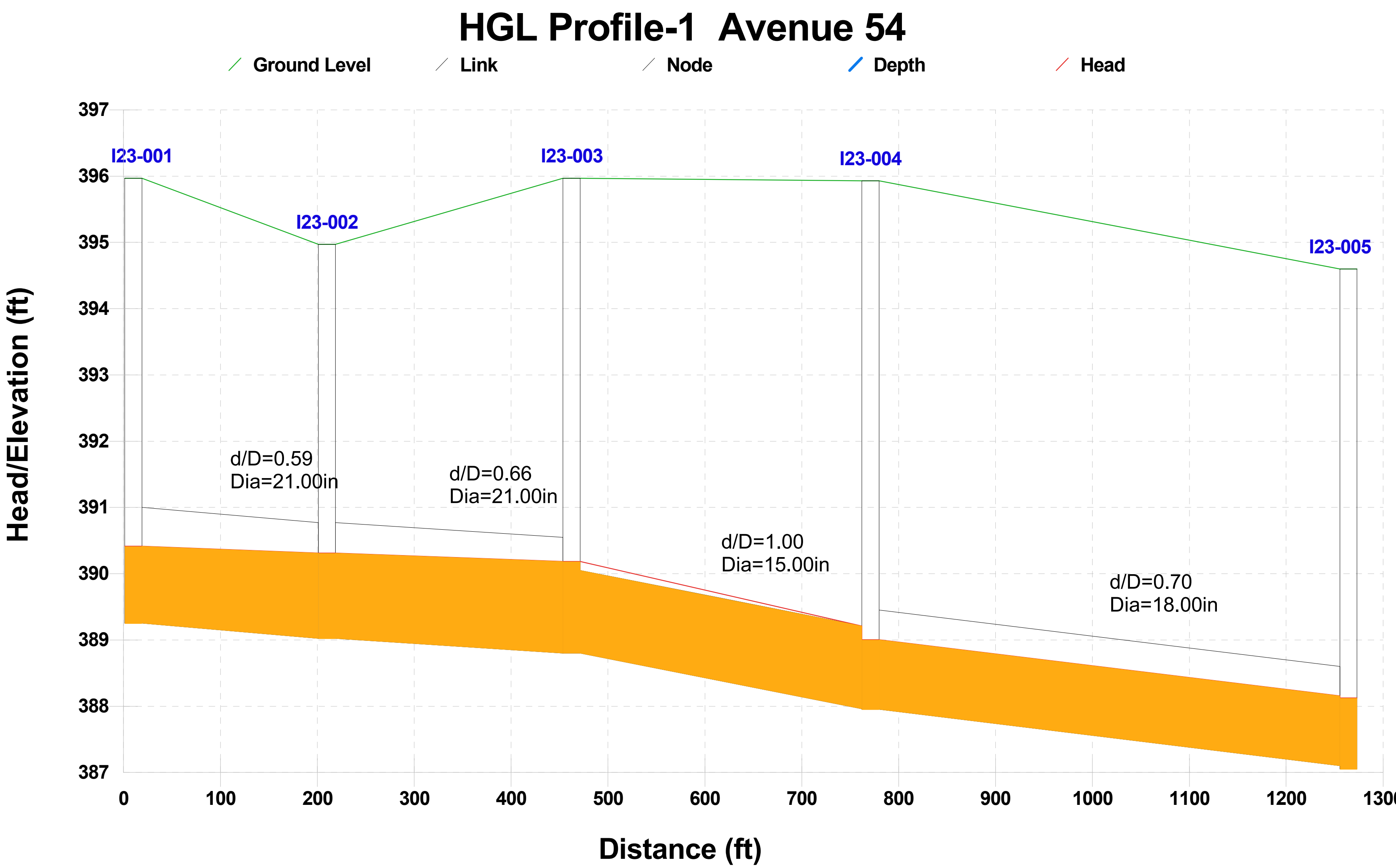
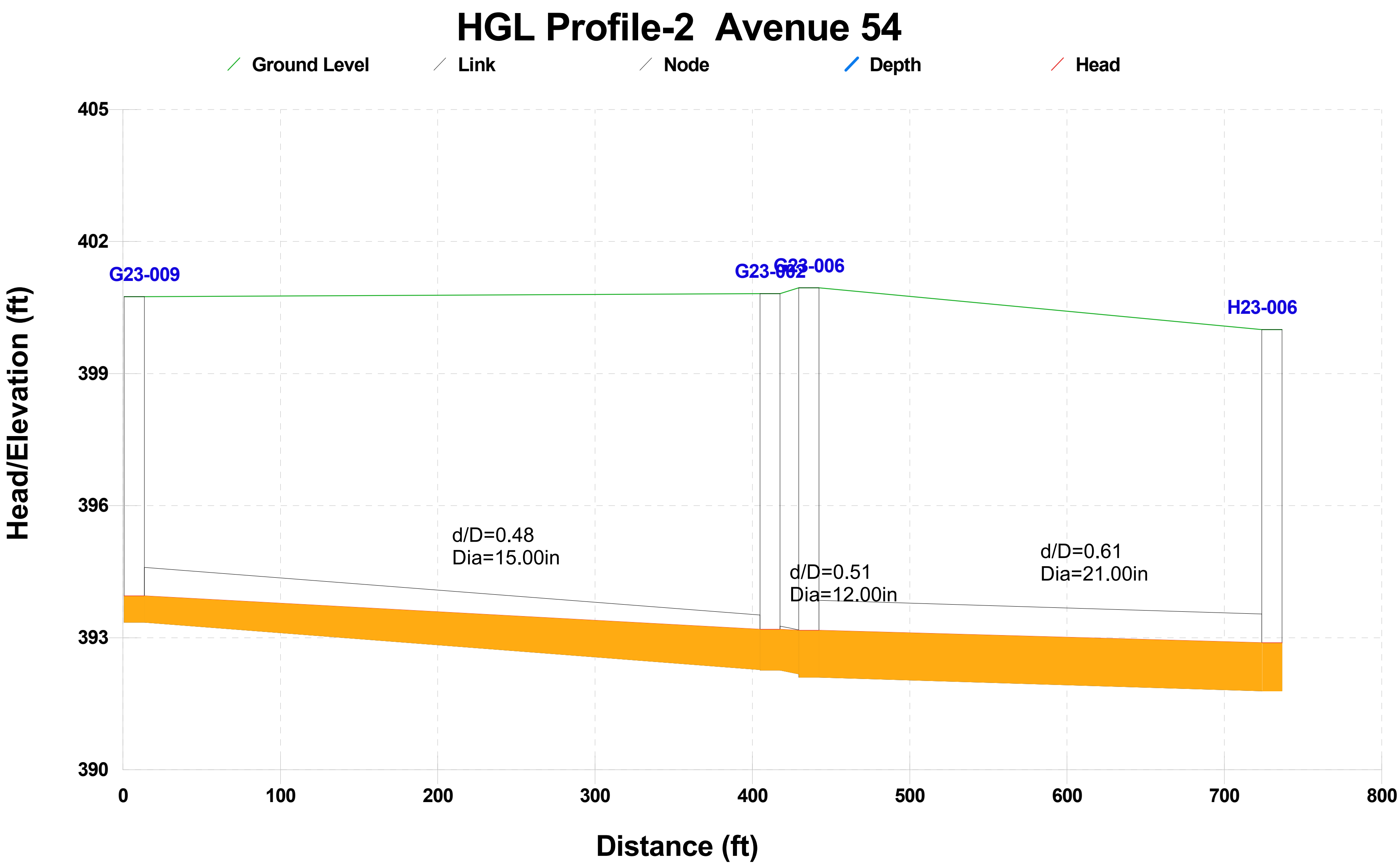



2040 Results d/D

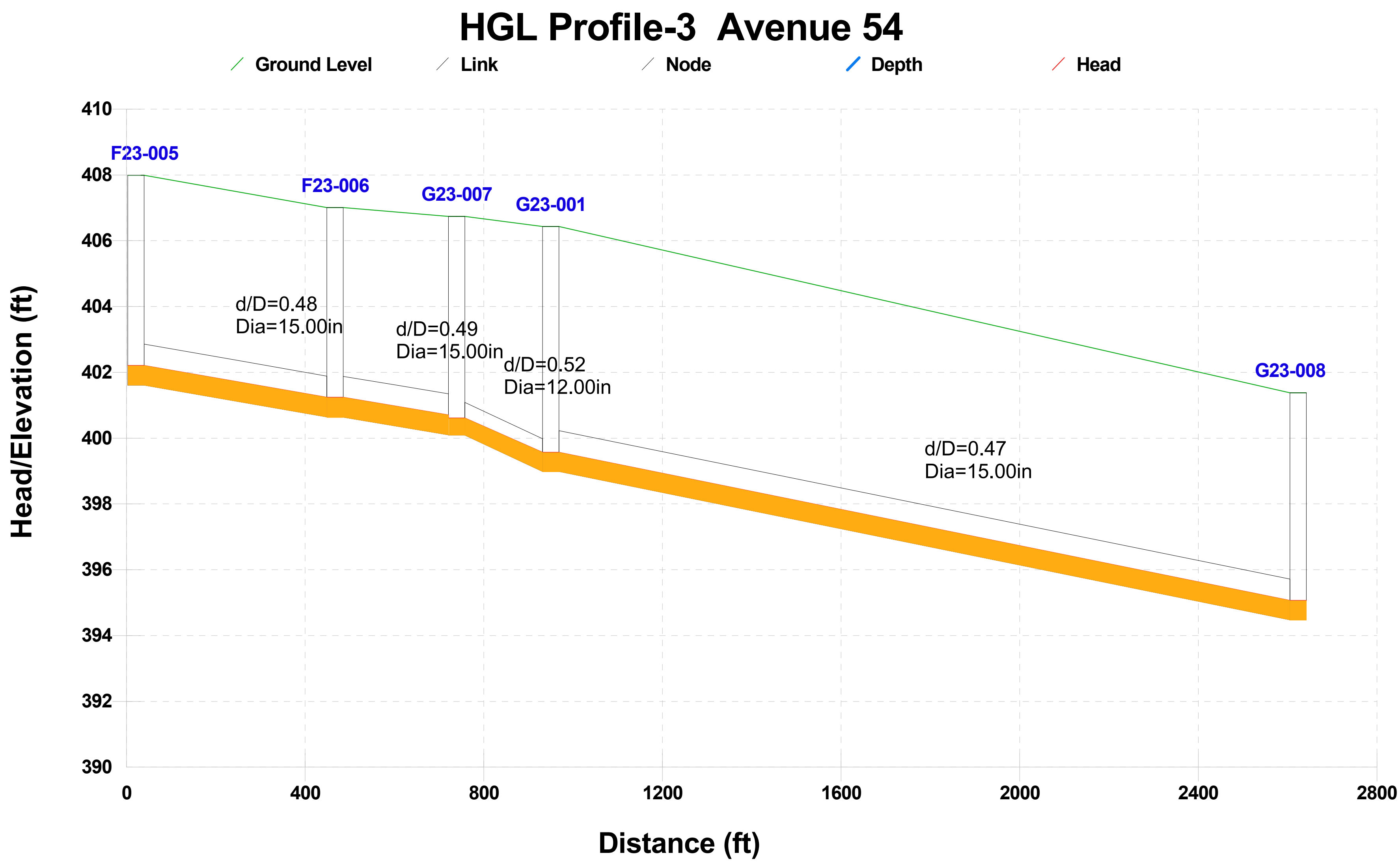
Scenario: 2040



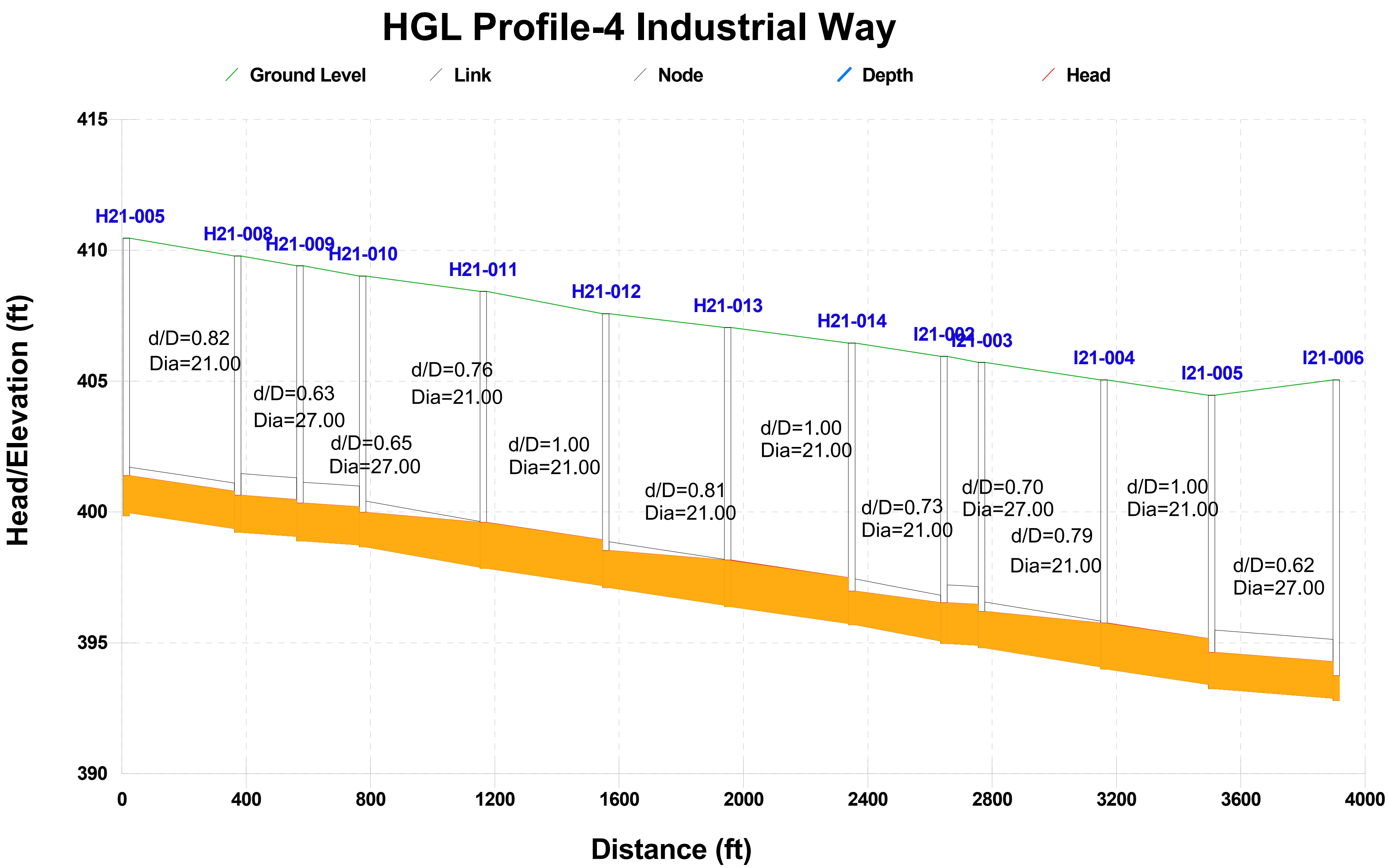
Scenario: 2040



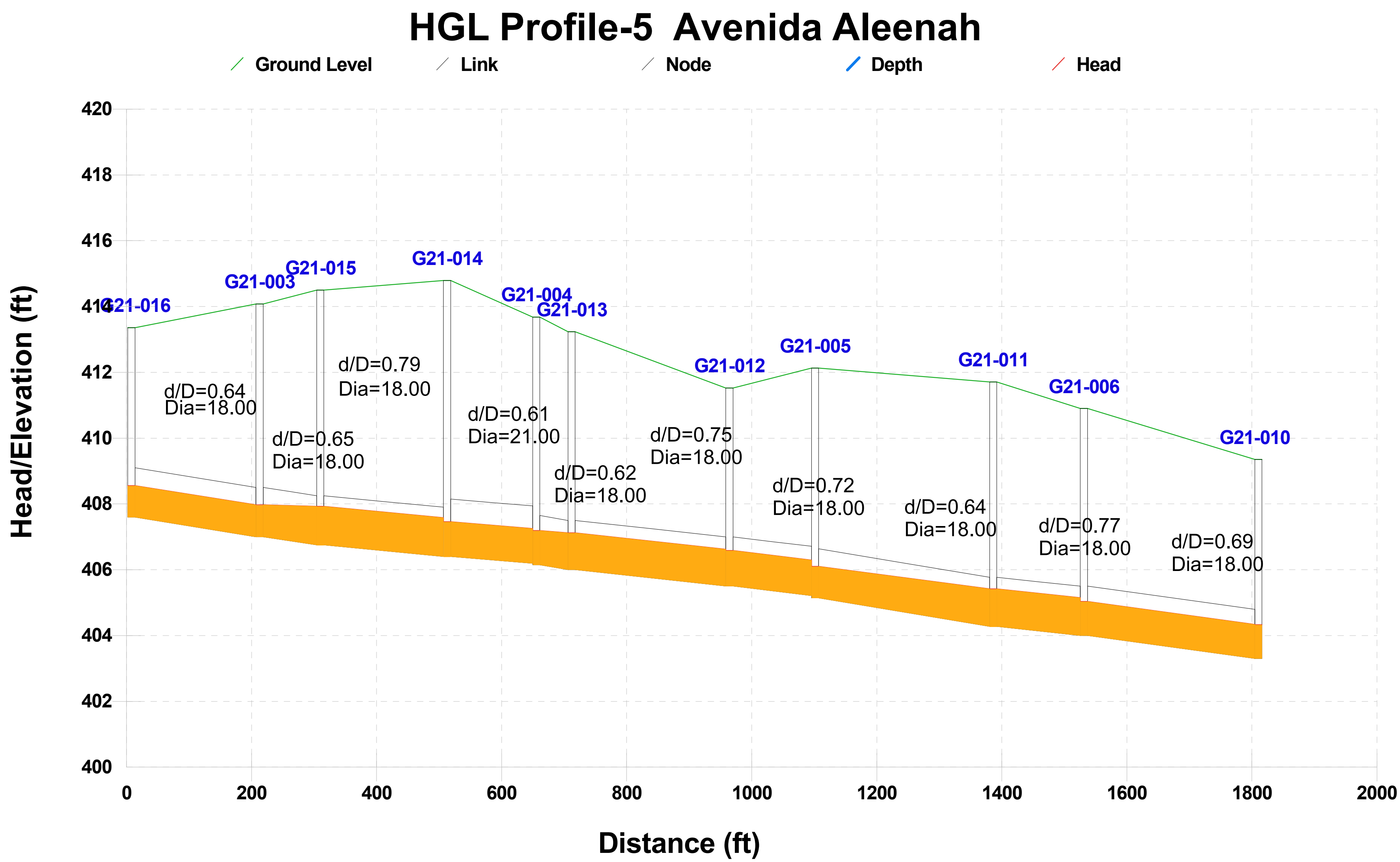
Scenario: 2040



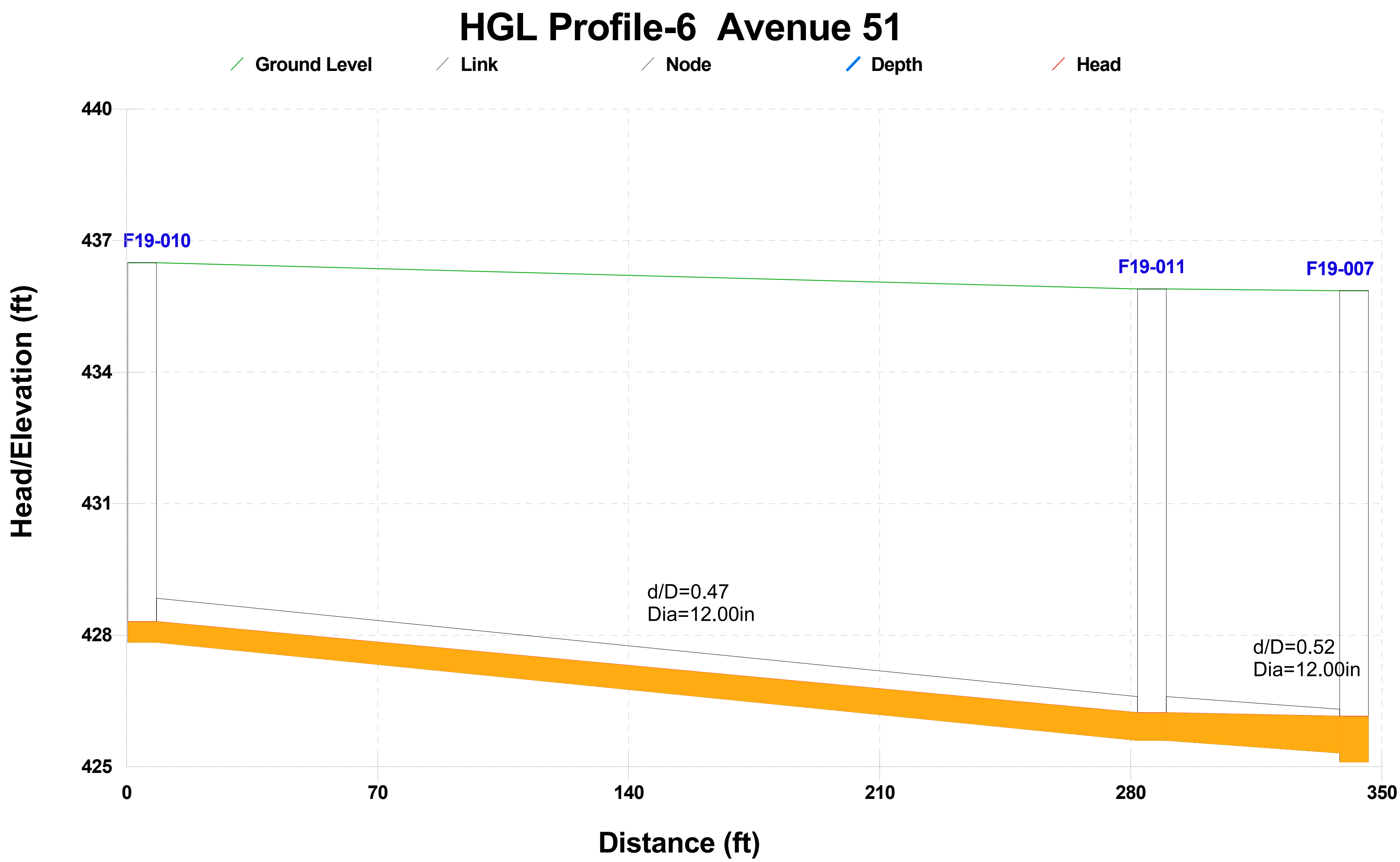
Scenario: 2040



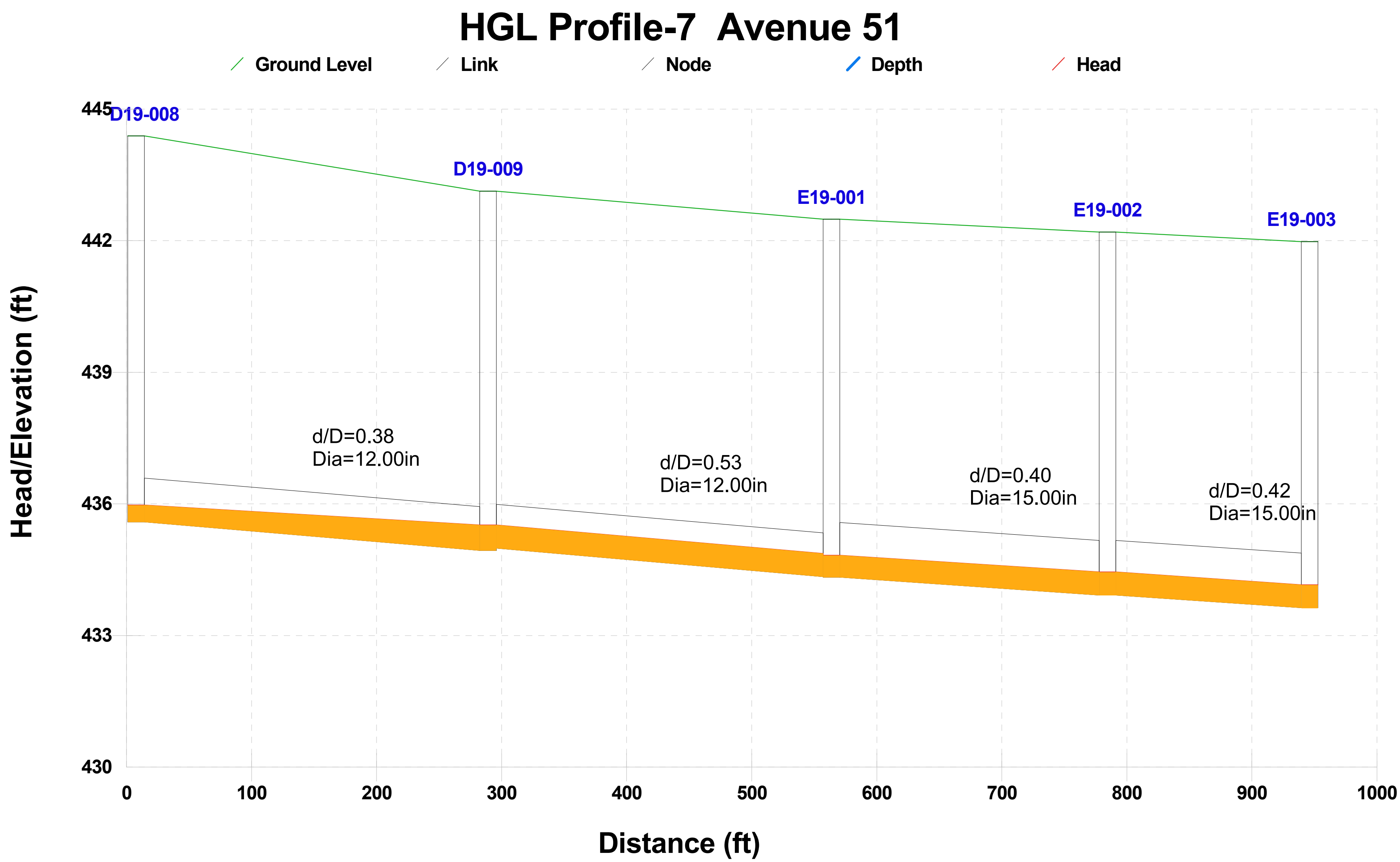
Scenario: 2040



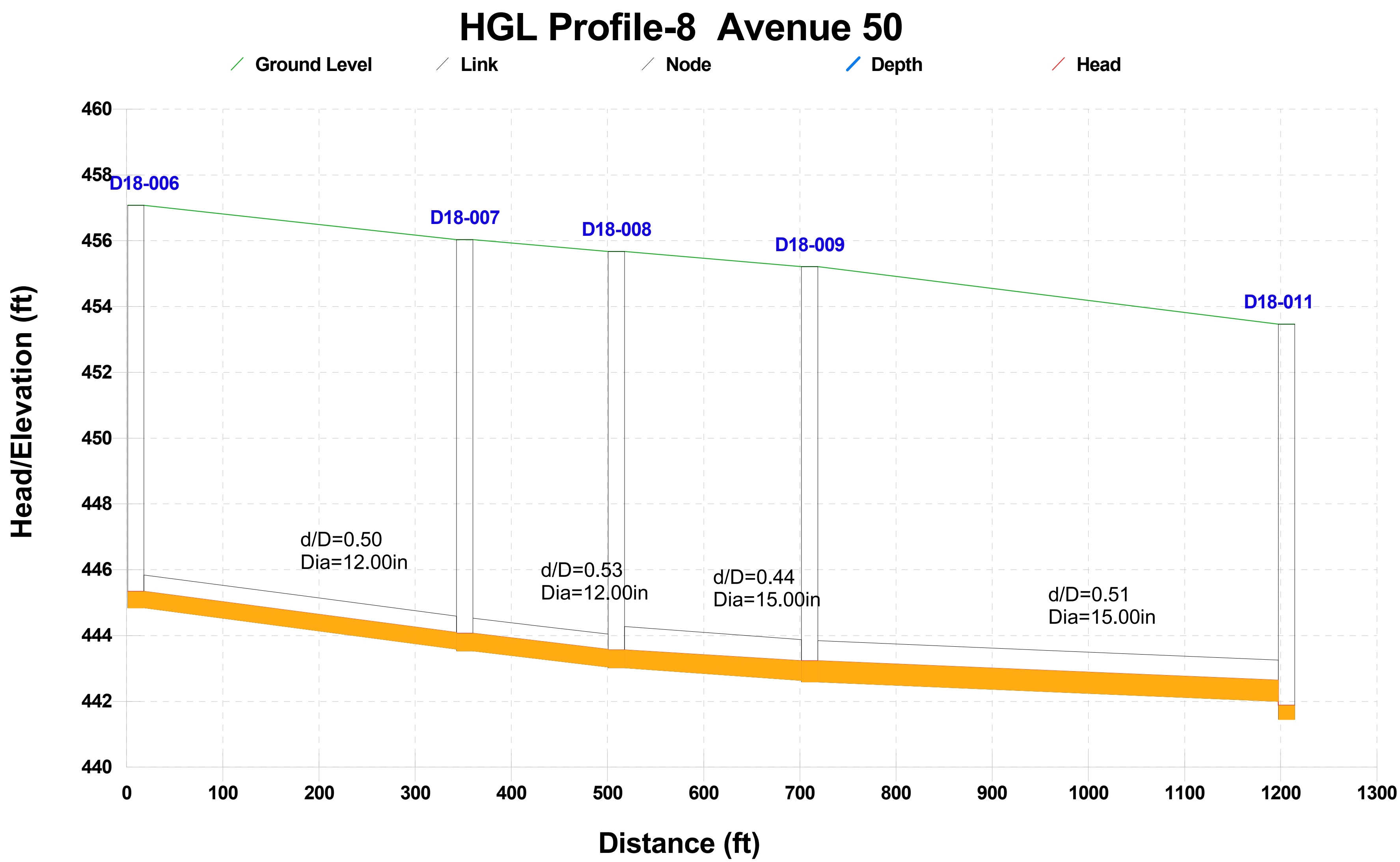
Scenario: 2040



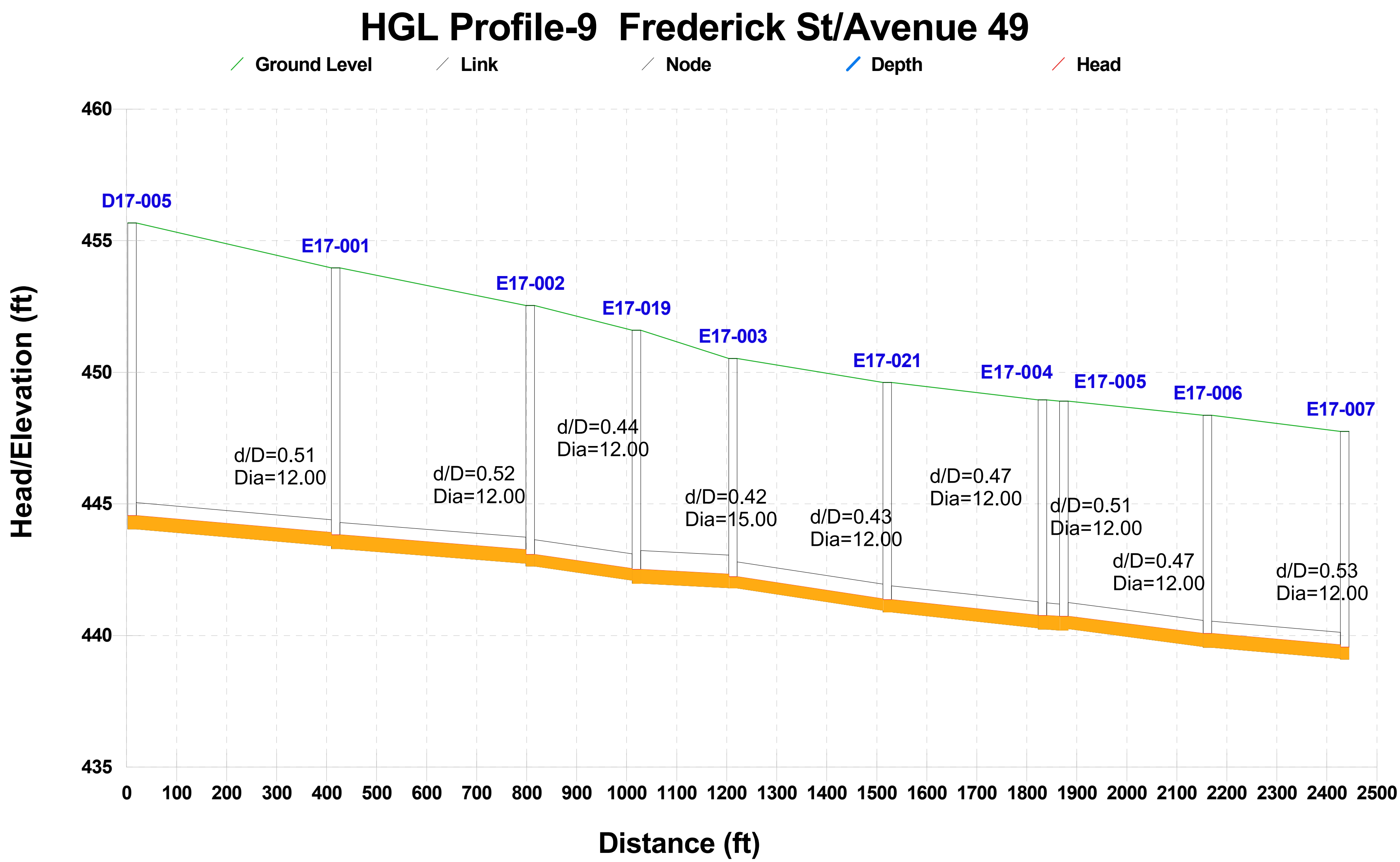
Scenario: 2040



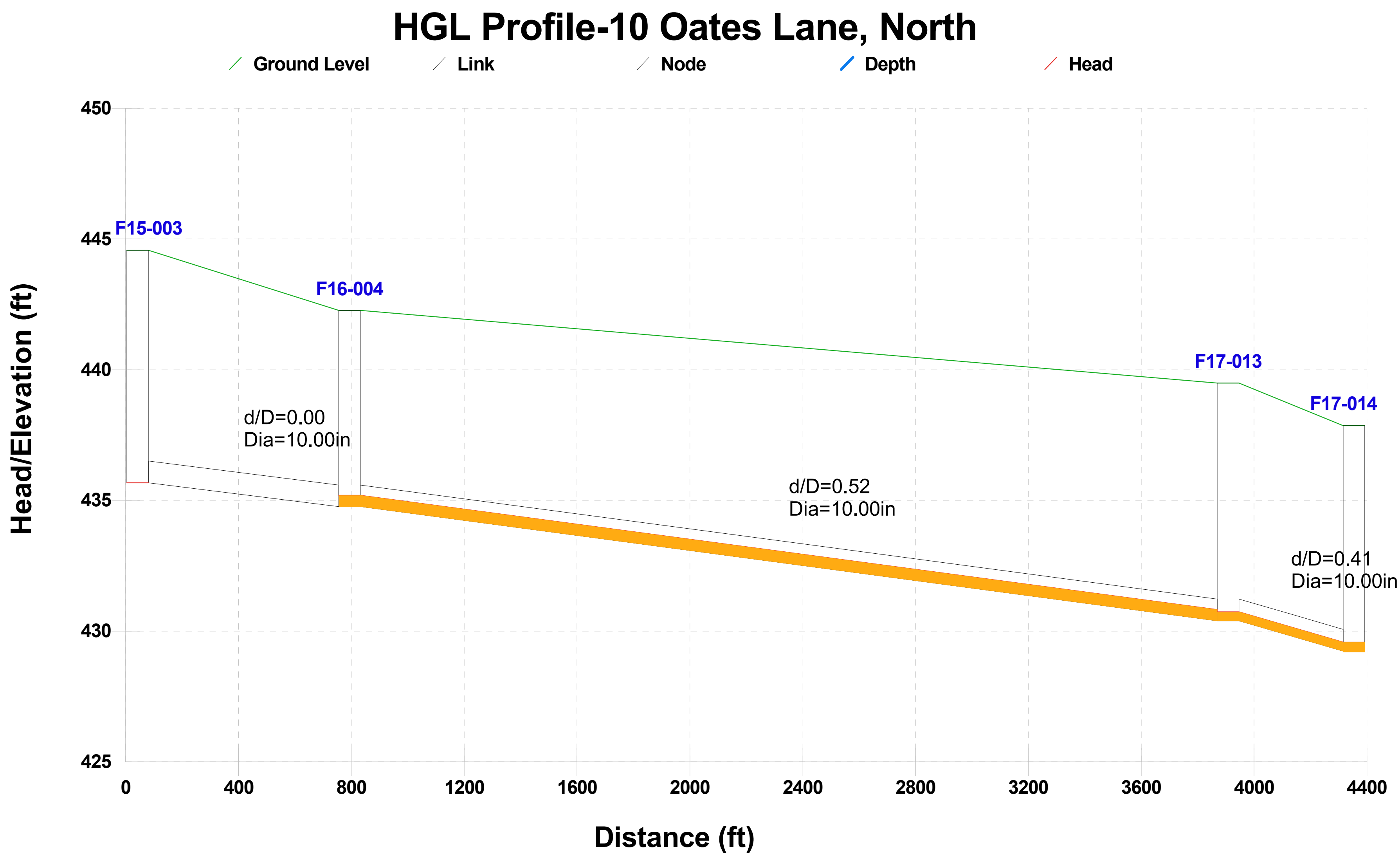
Scenario: 2040



Scenario: 2040

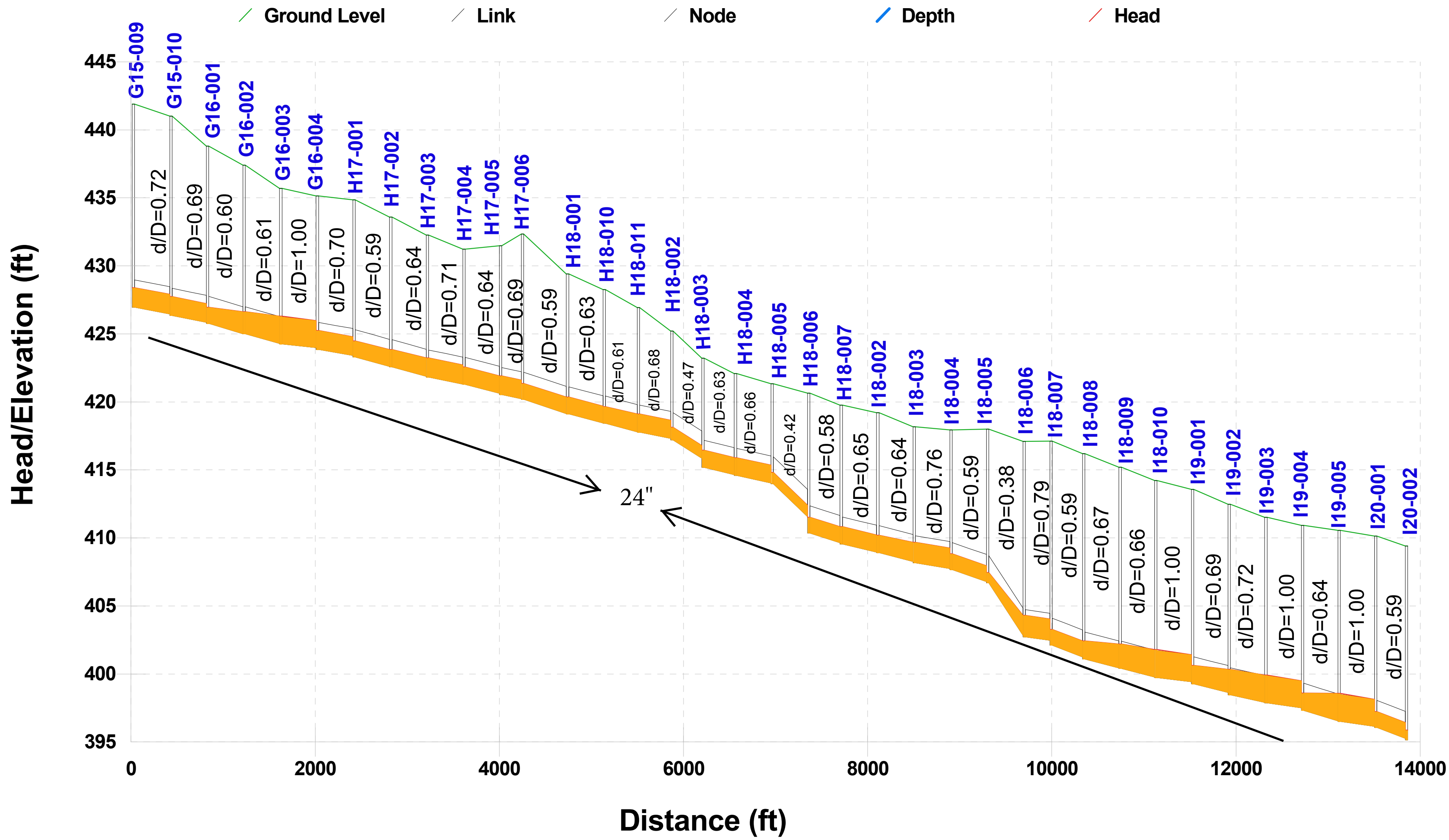


**Scenario: 2040**



Scenario: 2040

## HGL Profile-11 Tyler St./Avenue 50/ Polk St.



Scenario: 2040