CAPACITY ANALYSIS

6.1 INTRODUCTION

Upon completion of the DWF and WWF calibration, a capacity analysis of the modeled collection system was performed. The capacity analysis entailed identifying areas in the collection system where flow restrictions occur or where pipe capacity is insufficient to pass PWWF events. Pipes that do not have sufficient capacity to pass PWWF can produce backwater effects in the collection system and potentially cause undesirable SSOs. Typically, a design storm is used to quantify the PWWFs in the collection system and, coupled with design criteria, allows for an analysis of collection system capacities.

6.2 PLANNING AND DESIGN CRITERIA

The capacity of the City's sanitary sewer system was evaluated based on the analysis and design criteria defined in this chapter as well as the flows projected in Chapter 5. The developed criteria address the sewer system capacity, acceptable pipe slopes, acceptable depths of flow within pipes, minimum and maximum velocity of flow, and minimum pipe size.

6.2.1 Gravity Sewers

Capacity analysis of the gravity sewers was performed in accordance with the criteria established in this section.

6.2.1.1 <u>Pipe Capacities</u>

Sewer pipe capacities are dependent on many factors. These include roughness of the pipe, geometric configuration (cross-section and length), and slope. The Continuity equation and the Manning equation for steady-state flow can be used to calculate flow in a sewer pipe:

Continuity Equation: $$Q = V*A$$

Where:

Q = peak flow, cubic feet per second (cfs).

V = velocity, feet per second (fps).

A = cross-sectional area of pipe, square feet (sf).

$$\text{Manning Equation:} \qquad V = (1.486 * R^{2/3} * S^{1/2})/n$$

Where:

V = velocity, fps.

n = Manning's coefficient of friction.

R = hydraulic radius (area divided by wetted perimeter), feet.

S = slope of pipe, feet per foot.

6.2.1.2 Manning Coefficient (n)

The Manning coefficient 'n' is a friction coefficient and varies with respect to pipe material, size of pipe, depth of flow, smoothness of joints, root intrusion, and other factors. For sewer pipes, the Manning coefficient typically ranges between 0.011 and 0.017, with 0.013 being a representative value used for system master planning purposes.

6.2.1.3 Allowable Slopes and Velocity

To minimize the settlement of sewage solids, it is standard design practice to specify that a minimum velocity of 2 fps be met or exceeded at least once per day. At this velocity, the sewer flow will typically provide self-cleaning characteristics. Due to the hydraulic properties of a circular conduit, velocity of half-full flow in pipes approaches the velocity of the full flow condition in pipes. The minimum acceptable slopes, based on 2 fps velocity, for sewer pipe sizes are located in the City design guide and should be adjusted if flow characteristics are changed. The maximum velocity in a pipe should not exceed 10 fps, unless special provisions are made to mitigate odor, agitation, and loss of solid in the flow. In addition, public sewers should be designed so the minimum pipe size is 8 inches in diameter.

6.2.1.4 Flow Depth (d/D) and Surcharge Criteria

When designing sewer pipelines, it is common practice to adopt variable flow/depth criteria for various pipe sizes. This criterion is expressed as a maximum depth of flow to pipe diameter ratio (d/D). Design d/D ratios typically range from 0.5 to 1.0, with the lower values typically used for smaller pipes – which may experience flow peaks greater than planned or may experience blockages from debris, paper, or rags. A pipe is said to be “capacity deficient” when it is flowing greater than 75 percent full (i.e., d/D is greater than 0.75) under DWF conditions. A d/D ratio of 0.75 was used in analyzing the system under existing and build-out DWF conditions.

In determining deficient pipes under WWF conditions, a design storm I/I is routed through the collection system in the hydraulic model. The hydraulic model determines which pipelines in the collection system are unable to convey the PWWF caused by the design storm. The City has established PWWF criteria upon which to make improvements in the collection system. The PWWF criteria (or surcharge criteria), established by the City, allows the hydraulic grade line (HGL) to surcharge up to 3 feet below the manhole rim elevation. A pipeline is surcharged when the HGL rises above the crown elevation of the pipe. The rationale for adopting surcharge criteria provides the most effective compromise between creating a significant risk for excessive sanitary sewer overflows and creating a number of capital improvement projects necessary to reduce the HGL to a lower level.

This criterion was used to determine which pipelines in the modeled collection system are capacity deficient. Upon identifying these pipelines, the collection system hydraulic model is restructured to replace deficient sewers and provide additional capacity. Several sections of pipeline in the collection system will be full during the PWWFs of the design storm and yet will not require improvements because backwater effects do not elevate the HGL above the City's surcharge criteria.

6.3 DESIGN STORM

Design storms are synthetic rainfall events used to analyze the performance of a collection system under peak flows and volumes. Design storms have a specific recurrence interval and rainfall duration. The development of rainfall intensity, pattern, and total volume, are critical steps in developing a realistic design storm for the City.

6.3.1 Rainfall Analysis

The rainfall analysis used a multitude of sources in determining design storm volumes. The following sources were primarily used in the rainfall analysis.

- NOAA Atlas 2
- Historical precipitation records for Oroville Airport, East Oroville, and Oroville Dam rain gauges

Using the above sources, rainfall volumes and intensities for design storms of 5- and 10-year return periods were developed. The 24-hour duration design storm was considered since it is most representative of the rainfall in northern California that has a significant impact on sanitary sewer WWFs. A rainfall pattern similar to the large December 2005 storm was used to distribute the volume within the 24-hour duration. A 5-year design storm is expected to have a volume of 3.5 inches with a peak intensity of 0.56 inches per hour. A 10-year design storm is expected to have a volume of 4.0 inches and a peak intensity of 0.64 inches per hour. Both of these storms were evaluated using the hydraulic model for appropriateness.

6.3.2 Selection of Design Storm

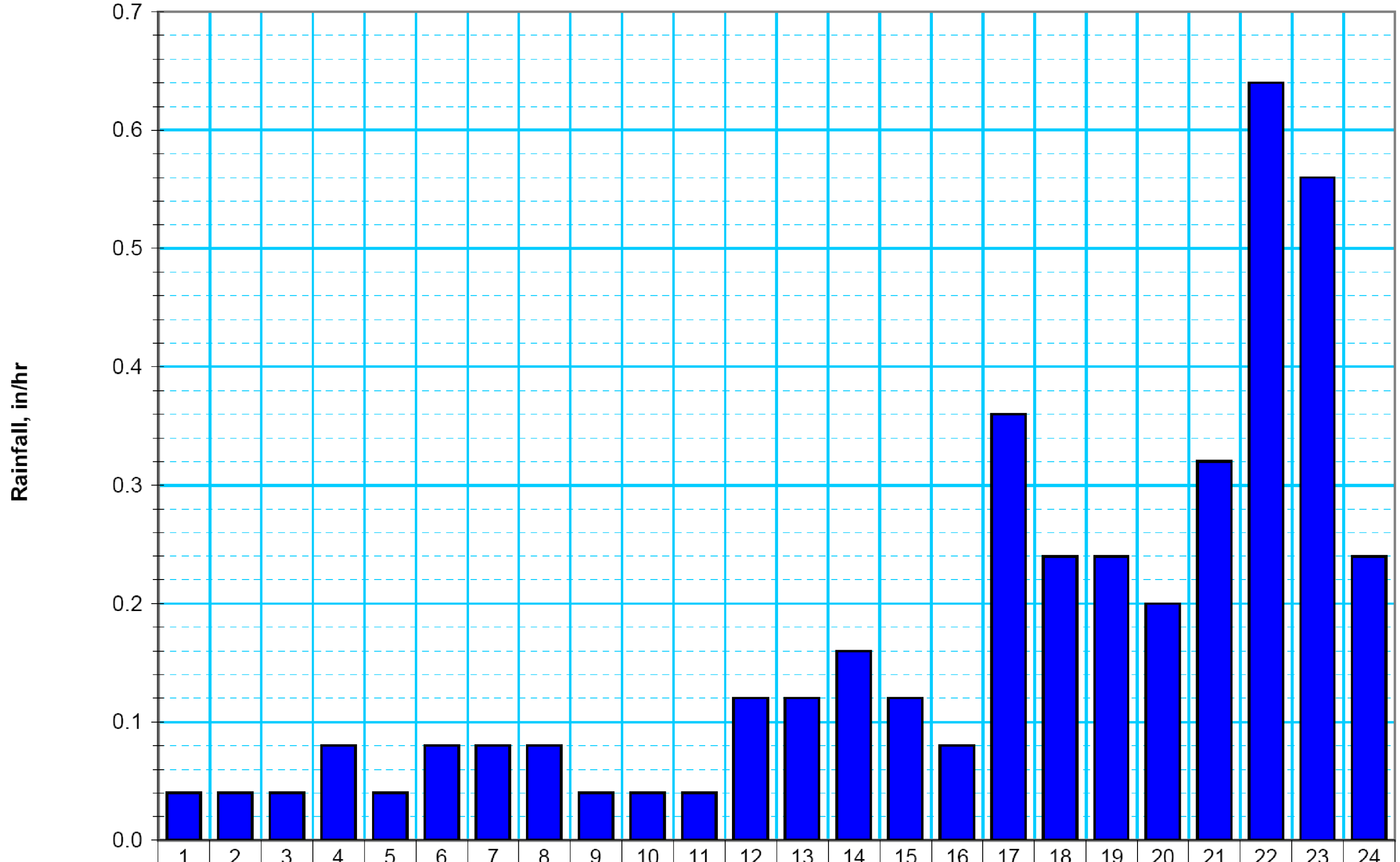
Following the completion of the rainfall analysis, a sensitivity analysis was performed to determine the appropriate design storm to use in determining system improvements. The collection system was evaluated under existing DWF conditions using the 5-year, 24-hour and 10-year, 24-hour design storms. The results of the sensitivity analysis are presented in Table 6.1. The 5-year design storm estimates 23 SSOs would occur. Under the 10-year design storm, 34 SSOs are projected. Due to the relatively small increase in projected SSOs, the 10-year, 24-hour design storm was selected to evaluate the collection system.

While the decision to use the 10-year, 24-hour design storm will result in a larger CIP, the 10-year storm recurrence interval provides the City a greater level of protection while maximizing funds spent. Figure 6.1 presents the rainfall distribution pattern of the 10-year, 24-hour design storm.

Table 6.1 Design Storm Comparison
Sanitary Sewer Master Plan
City of Oroville

	5-Year, 24-Hour Design Storm	10-Year, 24-Hour Design Storm
Flow Characteristics		
Dry Weather Flow	Existing	Existing
Storm Characteristics		
Volume (inches)	3.5	4.0
Peak Intensity (in/hr)	0.56	0.64
PWWF(1) (mgd)	10.01	11.27
Pipeline Capacity		
d/D(2) < 0.75	257	244
0.75 < d/D < 1	11	13
d/D = 1	98	109
Manhole Depth		
SSO(3)	23	34
< 1-foot below rim	6	6
1 - 3 feet below rim	22	17
3 - 5 feet below rim	44	48
> 5 feet below rim	269	259

Notes:
1. PWWF = Peak wet weather flow including TWSD flows for model continuity.
2. d/D = Depth to diameter flow ratio.
3. SSO = Sanitary sewer overflow. HGL greater than rim elevation.



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Rainfall	0.04	0.04	0.04	0.08	0.04	0.08	0.08	0.08	0.04	0.04	0.04	0.12	0.12	0.16	0.12	0.08	0.36	0.24	0.24	0.20	0.32	0.64	0.56	0.24

Hour

Figure 6.1
10-YEAR, 24-HOUR DESIGN STORM
SANITARY SEWER MASTER PLAN
CITY OF OROVILLE

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6.4 COLLECTION SYSTEM CAPACITY EVALUATION

The City's collection system was analyzed using the 10-year, 24-hour design storm to determine the system capacity deficiencies. The capacity analysis was performed for the following conditions:

- Existing Condition
  - PDWF
  - PWWF (10-Year, 24-Hour Design Storm)
- Build-Out Condition
  - PDWF
  - PWWF (10-Year, 24-Hour Design Storm)

The collection system response for the four conditions described above are provided below. The City anticipates growth in the future to occur in the western, eastern, and southern portions of the City. In addition, some growth will be infill development of vacant parcels and the annexation of parcels on the periphery of the City boundary. The collection system capacity analysis revealed moderate impact from the developments in the eastern area of the City. The greatest impacts to the system are development in the western and southern portions of the City. In the west, ADWF is anticipated to increase from 0.44 mgd to 2.24 mgd (including TWSD); in the south, ADWF is estimated to grow from <0.01 mgd to 2.31 mgd. Most of the deficiencies in the modeled collection system based on the capacity analysis are due to I/I from storm events. The design storm was simulated such that the generated peak hourly WWFs coincided with the peak hourly DWF. This results in an analysis under worst-case conditions. The deficiencies indicated below will be mitigated by improvements projected, which are identified in the CIP.

6.4.1 Existing PDWF Capacity Analysis

The collection system was analyzed in the hydraulic model under existing DWF (Year 2007) conditions to identify capacity deficiencies. There were three pipelines that did not meet the City's DWF capacity criterion of d/D = 0.75 (depth of flow to diameter ratio of 0.75); however, none of the pipelines are of significant concern. One pipeline segment is located in TWSD's East Interceptor and has an adverse slope. The remaining two pipes each are short in length (<65 feet) and do not create significant backwater effects upstream. Figure 6.2 illustrates the deficient pipes that did not meet the City's flow depth criteria.

The City's ADWF contribution to the WWTP is 1.71 mgd. The ADWF is based on existing land use zoning data. In addition, the minimum hourly DWF conveyed is 0.85 mgd with a peak hourly DWF flow of 2.36 mgd.

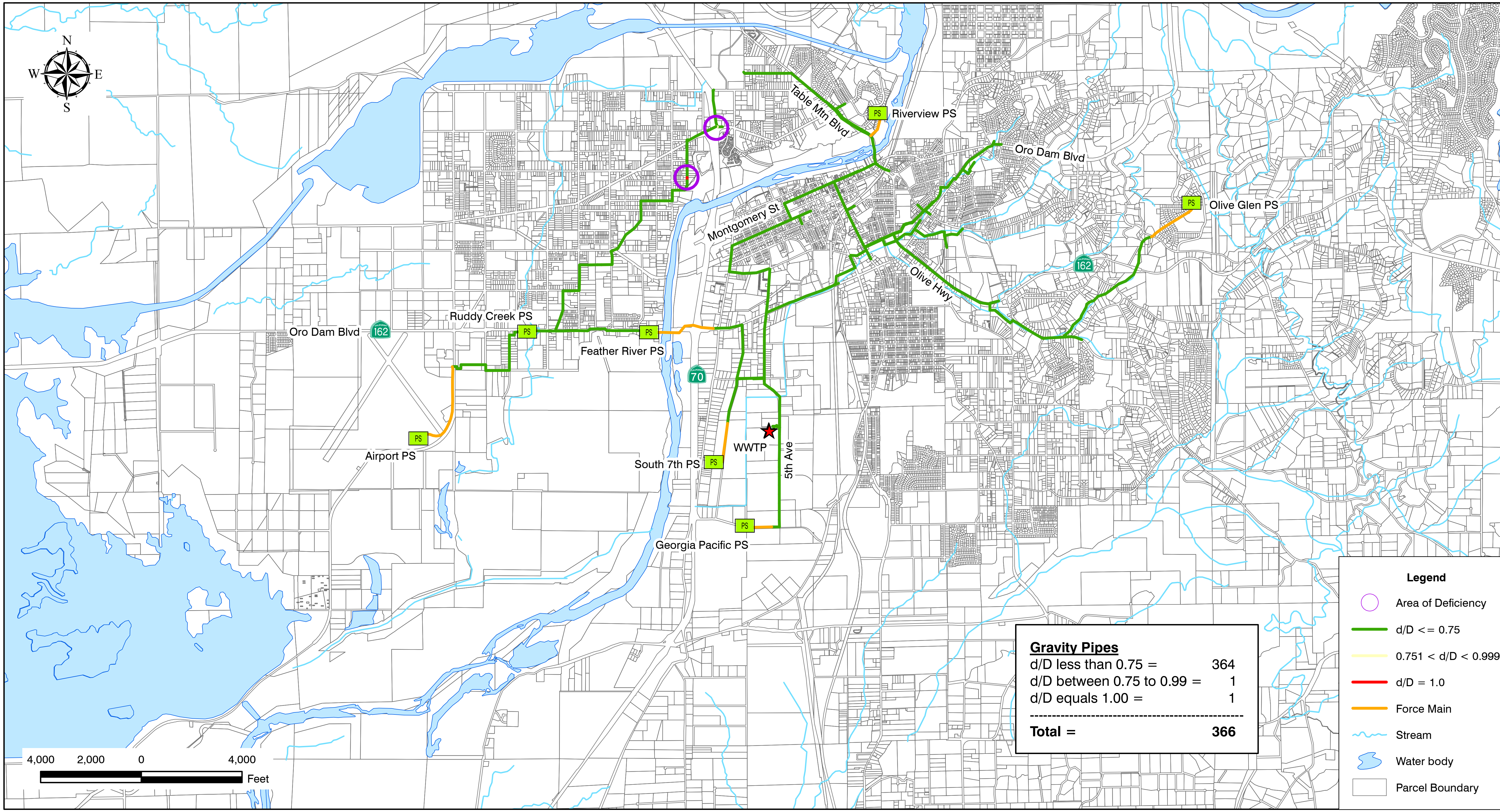

Figure 6.2
CAPACITY ANALYSIS
EXISTING DRY WEATHER FLOW
SANITARY SEWER MASTER PLAN
CITY OF OROVILLE

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6.4.2 Existing PWWF Capacity Analysis

The collection system was analyzed using the 10-year, 24-hour design storm added to the existing DWF (Year 2007). There were approximately 57 manholes and 75 pipes that did not meet the City's wet weather surcharge of 3 feet below rim elevation. A list of these pipelines is located in Appendix J. Figure 6.3 illustrates the deficient areas of the collection system. The areas of greatest concern in the collection system are along Mitchell Avenue (Basins 7 and 9) and along Table Mountain Boulevard north of Feather River (Basin 6). The City's PWWF at the WWTP during the 10-year, 24-hour design storm for the existing flow condition is estimated to be 11.27 mgd (excluding TWSD).

6.4.3 Future DWF Capacity Analysis

Under future DWF conditions, a total of 44 pipes (including the two from the existing flow condition) do not meet the City's DWF capacity criterion of d/D = 0.75 and are considered deficient. The future flow condition is increased 6.10 mgd from the existing flow condition. Most of the deficient pipes are located along 5th Avenue, south of the WWTP (Basin 1), where the most growth is expected. One deficient reach is SC-OR's West Interceptor pipes, which the City is not responsible to improve. Figure 6.4 illustrates the deficient pipes that did not meet the City's flow depth criterion.

Under future flow conditions, the City's ADWF contribution to the WWTP is 6.45 mgd. In addition, the minimum hourly DWF conveyed is 2.48 mgd with a peak hourly flow of 9.49 mgd.

6.4.4 Future PWWF Capacity Analysis

The collection system was analyzed using the 10-year, 24-hour design storm with future DWF. Modeled WWF intensities and volumes for existing areas were kept constant, and additional I/I contributions were added from future development areas. There were approximately 138 manholes that did not meet the City's wet weather surcharge criteria of 3 feet below rim elevation. Figure 6.5 illustrates the deficient areas of the collection system. In addition to the two major areas identified in the existing PWWF analysis, there are five additional areas of concern. They are located in the downstream portion of TWSD's East Interceptor, SC-OR's West Interceptor, along 7th Avenue (Basin 2), 5th Avenue south of the WWTP (Basin 1), and along Montgomery Street (Basin 4B). The City's PWWF at the WWTP during the 10-year, 24-hour design storm for the future flow condition is estimated to be 20.69 mgd. A summary of the modeled DWF and PWWF (10-year, 24-hour design storm) for the capacity analysis hydraulic modeling simulations are presented in Table 6.2.

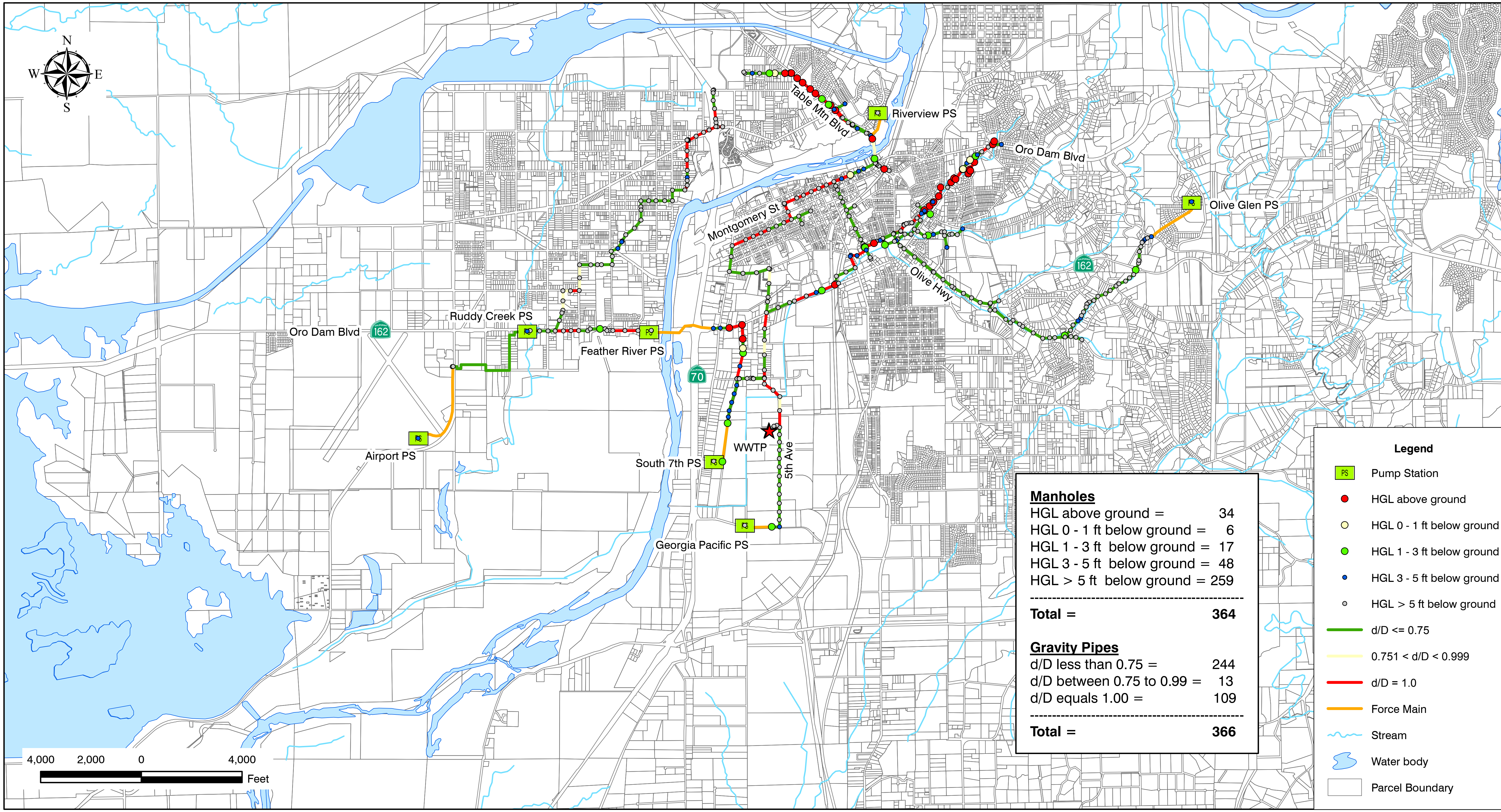

Figure 6.3
CAPACITY ANALYSIS
EXISTING 10-YEAR DESIGN STORM
SANITARY SEWER MASTER PLAN
CITY OF OROVILLE

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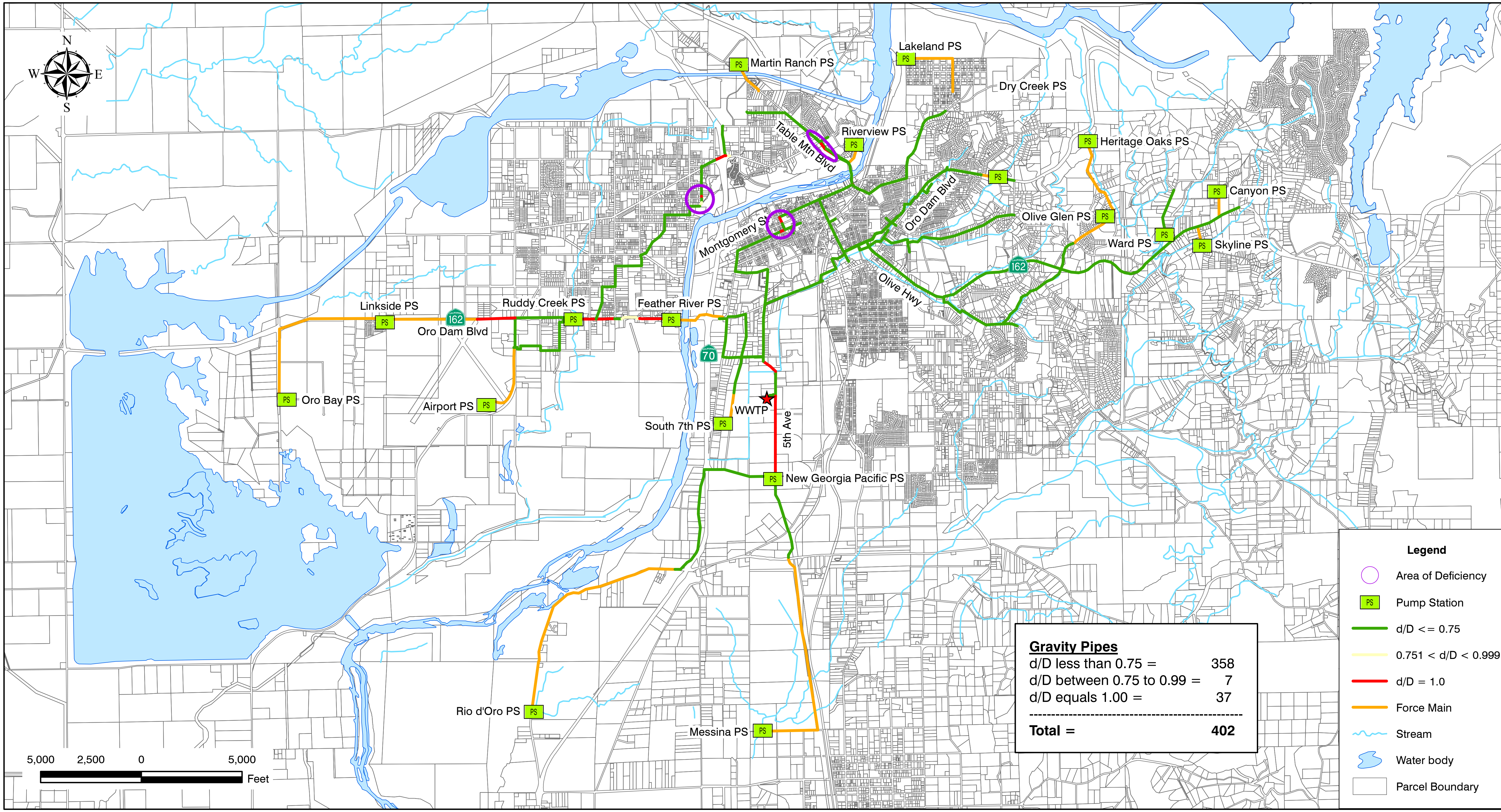

Figure 6.4
CAPACITY ANALYSIS
FUTURE DRY WEATHER FLOW
SANITARY SEWER MASTER PLAN
CITY OF OROVILLE

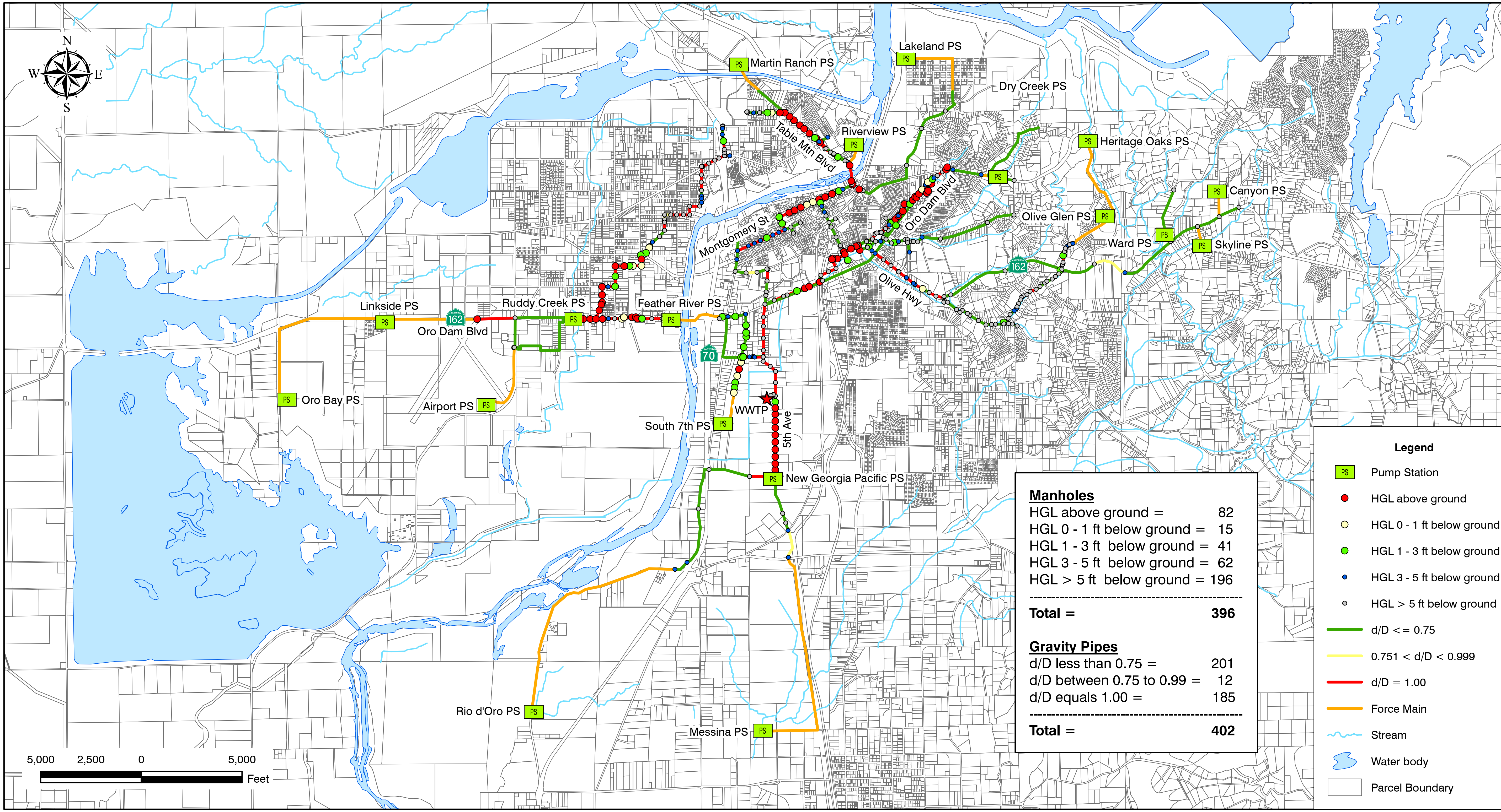

Figure 6.5
CAPACITY ANALYSIS
FUTURE 10-YEAR DESIGN STORM
SANITARY SEWER MASTER PLAN
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Table 6.2 Existing and Future Deficiencies
Sanitary Sewer Master Plan
City of Oroville

	Units	Existing Condition (Year 2007)	Future Condition (Year 2030)
Flow Type			
ADWF(1)	mgd(3)	1.71	6.45
Minimum DWF	mgd	0.85	2.48
PDWF(3)	mgd	2.36	9.49
PWWF(4)	mgd	11.27	20.69
DWF Pipeline Capacity			
d/D(6) < 0.75	---	364	356
0.75 < d/D < 1	---	1	7
d/D = 1	---	1	37
WWF Pipeline Capacity			
d/D(5) < 0.75	---	244	201
0.75 < d/D < 1	---	13	12
d/D = 1	---	109	185
WWF Manhole Depth			
SSO(6)	---	34	82
< 1 feet below rim	---	6	15
1 - 3 ft below rim	---	17	41
3 - 5 feet below rim	---	48	62
> 5 feet below rim	---	259	196

Notes:
1. ADWF = Average dry weather flow.
2. mgd = million gallons per day.
3. PDWF = Peak dry weather flow.
4. PWWF = Peak wet weather flow.
5. d/D = Depth to diameter flow ratio.
6. SSO = Sanitary sewer overflow (hydraulic grade exceeds rim elevation).

6.5 PUMP STATION CAPACITY ANALYSIS

A pump station capacity analysis was conducted to determine pump station deficiencies. Each pump station was analyzed using ADWF, PDWF, and PWWF flows for the existing and future conditions. PWWF includes wet weather from the 10-year, 24-hour design storm, and baseline I/I. Table 6.3 summarizes the pump station capacity analysis. Two SC-OR facilities were included in the analysis. The City is responsible for pump upgrades to the Ruddy Creek Pump Station, but is not responsible for improvements at FRPS. Under existing flow (Year 2007) conditions, one pump station is deficient.

- Feather River: PWWF is 2.74 mgd over firm capacity.

Under future flow conditions (Year 2030), four pump stations are deficient.

- Airport: PWWF is 0.43 mgd over firm capacity.
- Georgia Pacific: PWWF is 0.20 mgd over firm capacity.
- Ruddy Creek: PWWF is 4.84 mgd over firm capacity.
- Feather River: PWWF is 6.61 mgd over firm capacity.

6.6 SUMMARY

Overall, the City's collection system does not have adequate capacity to convey current WWFs in certain areas and has additional deficiencies relative to the future system needs. Under existing DWF conditions, the system performs well; however, under future DWF conditions, 11 percent of the pipes are capacity deficient. Capacity deficiencies under WWF conditions represent 16 percent (57 of 364 modeled manholes) in the existing system and 35 percent (138 of 397 modeled manholes) of the future collection system. The large number of deficiencies can be attributed to a significant increase in DWF over the planning period and I/I values within the existing system that are above industry standards.

6.7 ADDITIONAL ANALYSES

During the course of completing the master plan, several separate hydraulic analyses were performed at the request of City staff. The hydraulic analyses provided the City with preliminary modeling results that could be used to make informed decisions regarding other master plan tasks or outside studies. These analyses included the TWSD East Interceptor Capacity Analysis, the storage pond feasibility study, and the Oroville Dam Boulevard Relief Sewer routing alternative analysis.

Table 6.3 Pump Station Capacity Analysis
Sanitary Sewer Master Plan
City of Oroville

Name	Firm Capacity[1] (mgd)[2]	Total Capacity (mgd)	Existing Flow (Year 2007) ADWF[3] (mgd)	Existing Flow (Year 2007) PDWF[4] (mgd)	Existing Flow (Year 2007) PWWF[5] (mgd)	Existing Flow (Year 2007) Capacity Deficit[6] (mgd)	Future Flow (Year 2030) ADWF (mgd)	Future Flow (Year 2030) PDWF[4] (mgd)	Future Flow (Year 2030) PWWF[5] (mgd)	Future Flow (Year 2030) Capacity Deficit[6] (mgd)
Airport	0.47	0.94	0.014	0.022	0.084	None	0.38	0.59	0.90	0.43
South 7th	0.94	1.87	0.022	0.041	0.054	None	0.067	0.13	0.17	None
Georgia Pacific	0.36	0.72	<0.001	<0.001	0.011	None	0.22	0.42	0.56	0.20
Orangewood[7]	0.43	0.86	---	---	---	---	---	---	---	---
Butte Woods[7]	0.45	0.90	---	---	---	---	---	---	---	---
Riverview	0.50	1.00	0.028	0.047	0.22	None	0.048	0.081	0.26	None
Olive Glen	1.00	2.00	0.033	0.052	0.31	None	0.11	0.17	0.55	None
Ruddy Creek[8]	0.56	1.12	0.058	0.088	0.56	None	1.90	2.83	5.40	4.84
Feather River[9]	1.83	3.67	0.71	0.95	4.57	2.74	2.64	3.70	8.44	6.61

Notes:
1. Firm capacity assumes largest pump is out of service.
2. mgd = million gallons per day.
3. ADWF = Average dry weather flow.
4. PDWF = Peak dry weather flow (hourly).
5. Peak hourly wet weather flow (10-year 24-hour design storm) (I/I + DWF).
6. Capacity deficits are based on firm capacity and PWWF.
7. Pump station not modeled.
8. SC-OR facility. City responsible for upgrading pumps only.
9. SC-OR facility. City not responsible for improvements.

6.7.1 TWSD East Interceptor Capacity Analysis

The City and TWSD jointly requested an analysis investigating the capacity of TWSD's East Interceptor. By agreement, the City uses the East Interceptor to convey flows from Sub-basins 5th and C1 to the SC-OR Westside Interceptor. A study by Northstar Engineering (Northstar) indicated the interceptor was already at capacity. The hydraulic model constructed as part of this master plan was modified to include all pipe segments in the East Interceptor with data provided by Northstar. The model was then calibrated to DWF and WWF data from the 5th and Grand Meter as well as the FRPS. Based on the deficiency criteria outlined previously in this chapter, the East Interceptor was determined to have excess capacity at the present time.

Further comparison with the Northstar study revealed that WWF was estimated using a peaking factor plus a supplemental I/I rate, a method atypical in the industry. The Carollo analysis used an industry accepted approach of estimating WWF by routing a design storm through the calibrated hydraulic model. The details of the analysis are located in Appendix H.

6.7.2 Storage Pond Feasibility Study

The City owns four storage ponds on the east side of 5th Avenue across from the SC-OR WWTP. The ponds have a total volume of approximately 6.2 million gallons and were previously pretreatment lagoons for industrial users. Carollo evaluated the feasibility of using the ponds as wet weather storage to reduce peak flows to the WWTP. Preliminary modeling results indicate up to a peak flow of 10 mgd may be diverted to the ponds. The SC-OR Main Interceptor is lower than the ponds and flow would need to be pumped into the ponds. A second alternative would be to divert flow from LOAPUD's pipeline to the ponds. SC-OR's staff has indicated that infrastructure to divert flow to the ponds already exists and thus would be easier to implement. SC-OR staff has been briefed of this proposal and are supportive of any project that will reduce peak flows to the WWTP.

6.7.3 Oroville Dam Boulevard Relief Sewer Corridor Study

A corridor study was performed to explore the feasibility of a relief sewer along Oroville Dam Boulevard from Stanford Avenue to 5th Avenue. This alignment was investigated due to significant deficiencies in the Mitchell Avenue parallel sewers and the existing downstream of Oroville Dam Boulevard section.

This existing trunk sewer was constructed in narrow streets with multiple drainage crossings. The relief sewer would collect flows from Basin 9, Oroville Quincy Highway (Basin 7), and Olive Highway (Basin 8). From a master planning perspective, the relief sewer along Oroville Dam Boulevard has several advantages, including:

- A shorter length of pipeline
- Lower planning level cost
- Better constructability

However, the relief sewer will involve getting permits from Caltrans and construction in a busy roadway and may require sections to be constructed with trenchless installation methods. The City staff decided to proceed with presenting both alternatives in the master plan and basing the CIP on what appears at this time to be the most cost effective alternative.

Carollo conducted a detailed routing study to precede design and construction of the capital improvement. The costs, constructability, and permitting issues associated with the Oroville Dam Boulevard Relief Sewer alternative were explored, and the Oroville Dam Boulevard route continues to exhibit a lower overall cost and no fatal flaws that would preclude its implementation.