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City of Oxnard

Public Works Integrated Master Plan

STORMWATER

PROJECT MEMORANDUM 5.2
INFRASTRUCTURE MODELING AND ALTERNATIVES

REVISED FINAL DRAFT
September 2017

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PREFACE

The analysis and evaluations contained in these Project Memorandum (PM) are based on data and information available at the time of the original date of publication, December 2015. After development of the December 2015 Final Draft PMs, the City continued to move forward on two concurrent aspects: 1) advancing the facilities planning for the water, wastewater, recycled water, and stormwater facilities; and 2) developing Updated Cost of Service (COS) Studies (Carollo, 2017) for the wastewater/collection system and the water/distribution system. The updated 2017 COS studies contain the most recent near-term Capital Improvement Projects (CIP). **The complete updated CIP based on the near-term and long-term projects is contained in the Brief History and Overview of the City of Oxnard Public Works Department's Integrated Planning Efforts: May 2014 – August 2017 section.**

At the time of this Revised PWIMP, minor edits were also incorporated into the PMs. Minor edits included items such as table title changes and updating reports that were completed after the December 2015 original publication date.

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STORMWATER

PROJECT MEMORANDUM 5.2
INFRASTRUCTURE MODELING AND ALTERNATIVES

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INFRASTRUCTURE MODELING AND ALTERNATIVES

1.0 INTRODUCTION

The City of Oxnard's (City's) stormwater system serves the City with the surrounding lands that drain into Oxnard, approximately 35 square miles in area. The capacity of the City's stormwater drainage system was evaluated based on the planning criteria defined in the following sections. This Project Memorandum (PM) describes the development of the City's storm drainage hydrologic and hydraulic model. The model was used for identifying existing system deficiencies, identifying infrastructure needs for future growth, and developing capital improvements to mitigate deficiencies and meet the City's planning criteria.

1.1 PMs Used for Reference

The stormwater modeling alternatives outlined in this PM are made in concert with recommendations and analyses from other related PMs:

- PM 1.4 - Overall – Basis of Costs.
- PM 5.1 - Stormwater System - Background Summary.
- PM 5.3 - Stormwater System - Condition Assessment.
- PM 5.4 - Stormwater System - Treatment Alternatives.

2.0 PLANNING CRITERIA AND ASSUMPTIONS

2.1 Introduction

The capacity of the City's stormwater drainage system was evaluated based on the analysis and design criteria defined in this section. Criteria and assumptions were developed based on the City's standards and Carollo's storm drain planning experience.

2.2 Hydraulic Criteria

2.2.1 Gravity Pipes

Conveyance facilities owned by the City consist mainly of enclosed gravity storm drainage pipelines. Capacity analysis was performed on pipelines 24-inches in diameter and larger, as well as other critical facilities of all sizes. Rainfall data were used to generate the basis for stormwater evaluations. More details on design storm events can be found in Section 2.3.2.

2.2.2 Surcharge Depth and Street Flooding

Storm drains are designed to surcharge under normal operation. It is common engineering practice in drainage to allow curb and gutters along streets to act as storage and conveyance, similar to overland flow, for a given rainfall intensity and duration in order to protect adjacent properties from flooding. When evaluating the adequacy of the exiting conveyance facilities serving existing developments for the 10-year storm, City's drains should have enough capacity to convey peak runoffs. Additionally, the storm drain system allows street flooding not above the building finish floor levels for the 100-year storm.

2.3 Hydrologic Criteria

This section describes the hydrological characteristics of the City and the design storms that were used to estimate existing and future storm flows.

2.3.1 Design Storm Characteristics

The capacity of storm drainage facilities depends on the selection of a level of protection provided by those facilities. The level of protection is often expressed in terms of the frequency, or return period, of the storm for which the facilities are to prevent damage or for which the facilities will safely pass the stormwater flows. This storm is referred to as the design storm and is an idealized representation of a typical storm with a specified return period.

Selection of the design storm can have a significant impact on the size and cost of required drainage facilities. There are three elements of a design storm: precipitation depth, duration, and frequency.

Precipitation depth is the amount of precipitation occurring during a specified storm duration. The depths of rainfall are statistical depths obtained by studying historical precipitation data to find the depth for each duration and for a particular frequency. Precipitation depth is usually expressed in inches.

Duration is the specified length of storm time considered. Duration of a design storm event should be at least four times the response time of the basin. The response time is the time required for the peak flow to reach the point of interest, such as a structure, outlet, or spillway. When the design of storage facilities is involved, the duration should be sufficiently long so that the runoff and storage volumes return to near their level at the beginning of the simulation. Duration may be expressed in any time unit such as minutes, hours, or days.

Frequency is the number of occurrences of events with the specified precipitation depth and duration. It is expressed in terms of return period. In order to provide a reasonable level of flood protection, the statistical concept of return period or recurrence interval is utilized, which aids in assigning a probabilistic meaning to a precipitation event.

2.3.2 Development of the Design Storms

The design storms for the City were developed using U.S. Department of Agriculture and Natural Resources Conservation Service (NRCS) standardized 24-hour distribution curves with historical precipitation data. The NRCS developed normalized rainfall hyetograph distribution curves based on the storm's geographical location. The distribution curves are applied to total storm event volumes (design storm depth) in order to develop hourly storm event hyetographs. There are four types of rainfall distributions used to represent various regions throughout the United States (Type I, IA, II, and III). The City lies geographically within the Type IA boundary; therefore, the Type IA distribution was used.

The synthetic design storms were based on long-term, historical rainfall depth-duration-frequency (DDF) data from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, published for California in 2011.[1] The NOAA Atlas 14 serves as an industry standard for determining total rainfall depth at specified frequencies and durations in Central and Northern California.

Based on the NOAA data, a 10-year, 24-hour design storm for the City would create a total rainfall of 4 inches. This design storm has a ten percent chance (1/10) that 4 inches of rain will fall within any 24-hour period in a given year. Similarly, the 50-year, 24-hour storm event for Oxnard would create a total rainfall of 5.5 inches and the 100-year, 24-hour storm event would create a total rainfall of 6.4 inches. Design storms for the City are illustrated in Figure 1 with summary data provided in Table 1.

2.3.3 Soil Characteristics

Soil characteristics are another factor affecting the volume and rate of runoff in a study area. The absorptive capacity of soils, including infiltration and percolation characteristics, influences the amount of runoff as well as subsurface flows. Infiltrated water can also return to a discharge point (channel or stream) or be effectively lost to groundwater. Soil characteristics can also affect how near surface groundwater contributes infiltration to drain pipes or streams.

There are several ways to estimate the volume and/or the rate of infiltration of water into a soil. Three common estimation methods are Green-Ampt, Soil Conservation Service (SCS) method, and Horton's method. All of these equations provide a relatively accurate assessment of the infiltration characteristics of the soil in question. The Horton equation is an empirical formula that states that infiltration starts at a given rate and decreases exponentially with time. After a period of time when the soil saturation level reaches a certain value, the rate of infiltration will become constant. Parameters for the Horton equation can be reasonably estimated from literature and USDA soil data. Therefore, for the Study Area, infiltration into the soil in pervious areas was estimated for each subbasin in the model using the Horton equation.

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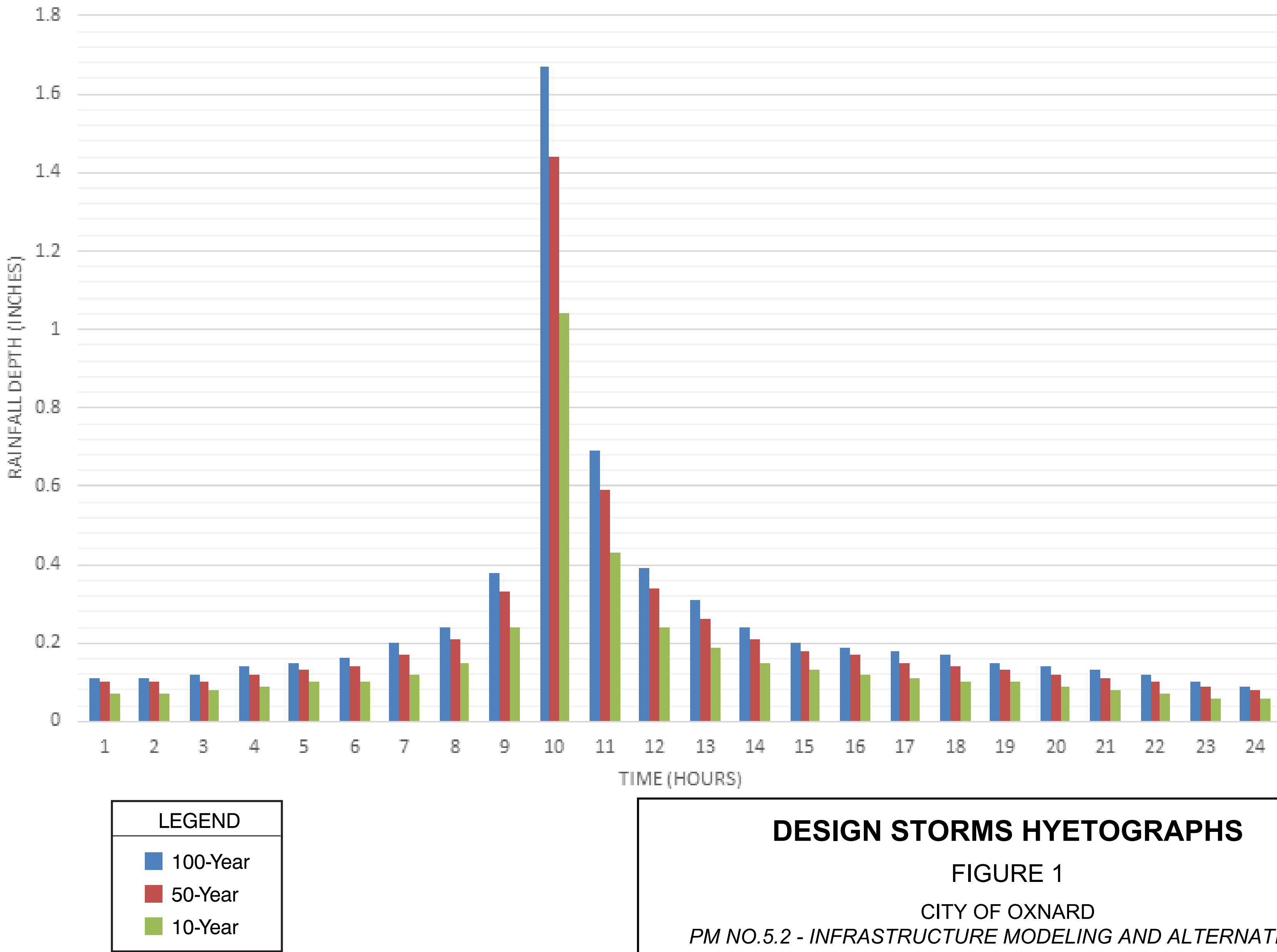
[1] NOAA Precipitation Frequency estimates (DDF data) for the City of Oxnard can be found at http://hdsc.nws.noaa.gov/hdsc/pfds/index.html.



DESIGN STORMS HYETOGRAPHS

FIGURE 1

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Table 1	**Design Storms Public Works Integrated Master Plan City of Oxnard**					
	10-year		**50-year**		**100-year**	
Hour	**Rainfall (inch/hr)**	**Cumulative (inch)**	**Rainfall (inch/hr)**	**Cumulative (inch)**	**Rainfall (inch/hr)**	**Cumulative (inch)**
0	0.07	0.1	0.1	0.1	0.11	0.1
1	0.07	0.1	0.1	0.2	0.11	0.2
2	0.08	0.2	0.1	0.3	0.12	0.3
3	0.09	0.3	0.12	0.4	0.14	0.5
4	0.1	0.4	0.13	0.6	0.15	0.6
5	0.1	0.5	0.14	0.7	0.16	0.8
6	0.12	0.6	0.17	0.9	0.2	1.0
7	0.15	0.8	0.21	1.1	0.24	1.2
8	0.24	1.0	0.33	1.4	0.38	1.6
9	1.04	2.1	1.44	2.8	1.67	3.3
10	0.43	2.5	0.59	3.4	0.69	4.0
11	0.24	2.7	0.34	3.8	0.39	4.4
12	0.19	2.9	0.26	4.0	0.31	4.7
13	0.15	3.1	0.21	4.2	0.24	4.9
14	0.13	3.2	0.18	4.4	0.2	5.1
15	0.12	3.3	0.17	4.6	0.19	5.3
16	0.11	3.4	0.15	4.7	0.18	5.5
17	0.1	3.5	0.14	4.9	0.17	5.7
18	0.1	3.6	0.13	5.0	0.15	5.8
19	0.09	3.7	0.12	5.1	0.14	5.9
20	0.08	3.8	0.11	5.2	0.13	6.1
21	0.07	3.9	0.1	5.3	0.12	6.2
22	0.06	3.9	0.09	5.4	0.1	6.3
23	0.06	4.0	0.08	5.5	0.09	6.4

In order to determine infiltration parameters for use in the Horton equation, the soils within the study area were mapped based on Hydrologic Soil Group. The Horton equation uses four hydrologic soil groups. The soils are classified by water intake at the end of long duration storms after prior wetting and an opportunity for swelling and without the proactive effects of vegetation. The hydrologic soil groups, as defined by SCS, are:

A. Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes the downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with high swelling potential, soils with permanent high water table, soils with claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Each soil group is associated with the typical infiltration soil properties as listed in Table 2. By determining the percentages of each hydrologic soil group within a subcatchment, maximum and minimum infiltration rates can be calculated. The constant decay rate for Horton infiltration analysis was set to 0.0015 per second. Weighted average soil properties were determined for each hydraulic model subcatchment based on the amount of each hydrologic soil group in the subcatchment, and typical soil properties for each group.

The majority of the study area has an NRCS Type C classification, followed by Type B and some Type A near the coast. Soil Types B and C tend to have moderate to slow infiltration and percolation rates when thoroughly wetted. Figure 2 illustrates the soil hydrological classifications.

Table 2 Infiltration Rates for NRCS Hydrologic Soil Groups
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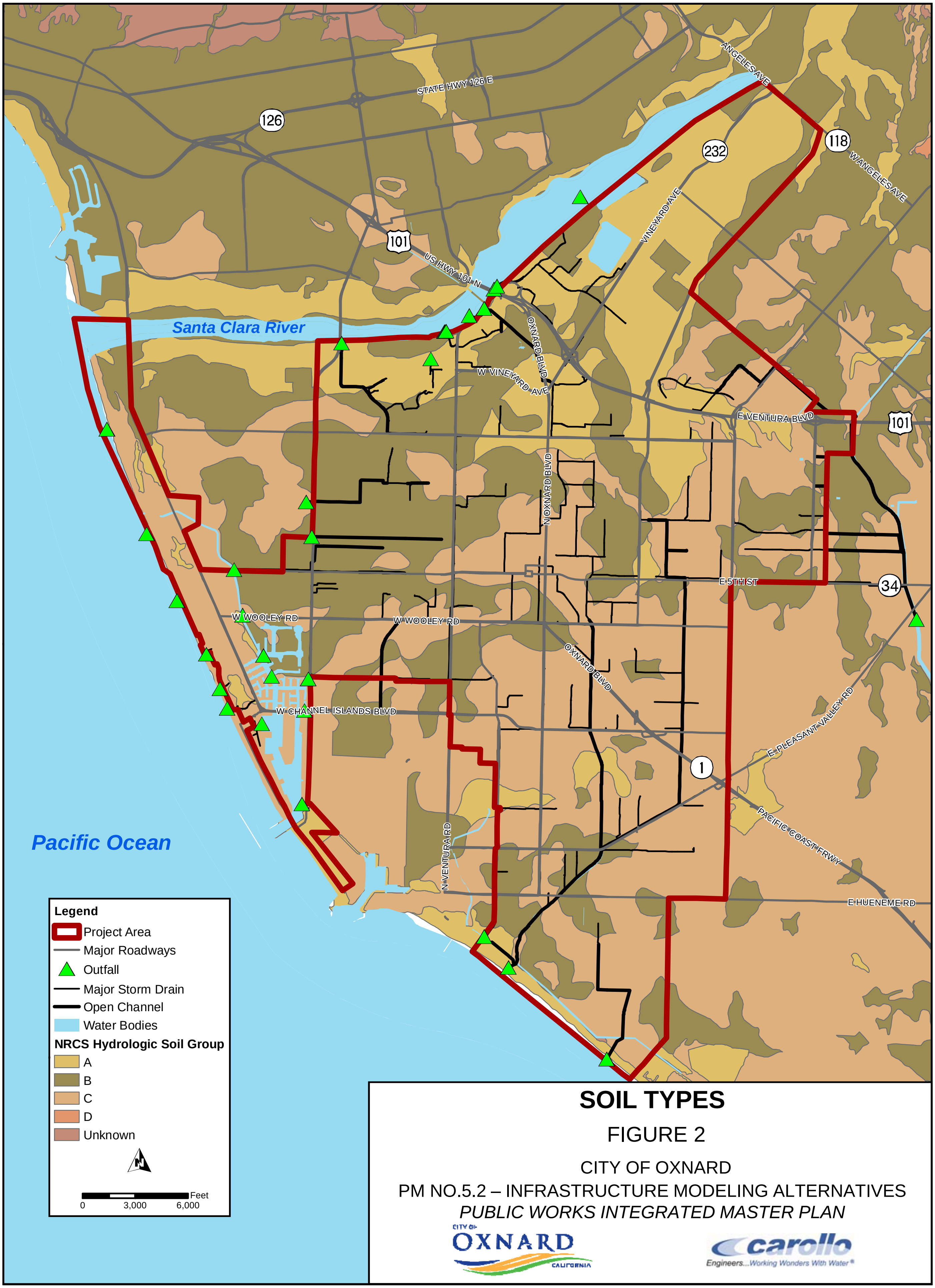
City of Oxnard

Soil Group	Maximum Infiltration Rate (in/hr)	Minimum Infiltration Rate (in/hr)
A	2.0	0.065
B	1.5	0.050
C	1.0	0.035
D	0.5	0.020



SOIL TYPES

FIGURE 2

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2.3.4 Impervious Land Areas and Basis for Runoff Estimates

High-resolution satellite imagery and aerial photography were used to determine existing land use imperviousness throughout the Project Area. Multispectral imagery was used to identify vegetation, water bodies, and man-made features. Vegetation appears as shades of red, water as shades of blue or black, and urban areas as shades of blue-gray. Impervious and pervious surfaces were classified from the satellite imagery bands in a raster format. The National Land Cover Database 2006 (2011 Edition) was used to get existing percent developed imperviousness[2].

The future total percent impervious area values were estimated by developing impervious estimates based on land use type. Existing representative sample areas were selected for each land use category presented in PM 5.1 and existing percent impervious were calculated for each land use type. Figure 3 illustrates the existing estimated percent imperviousness for each subcatchment within the project area. Table 3 summarizes the assumed values calculated for use with future land use estimates. These values are applied to future land use coverages to estimate the future percent imperviousness, illustrated in Figure 4.

Table 3 Percent Impervious by Land Uses
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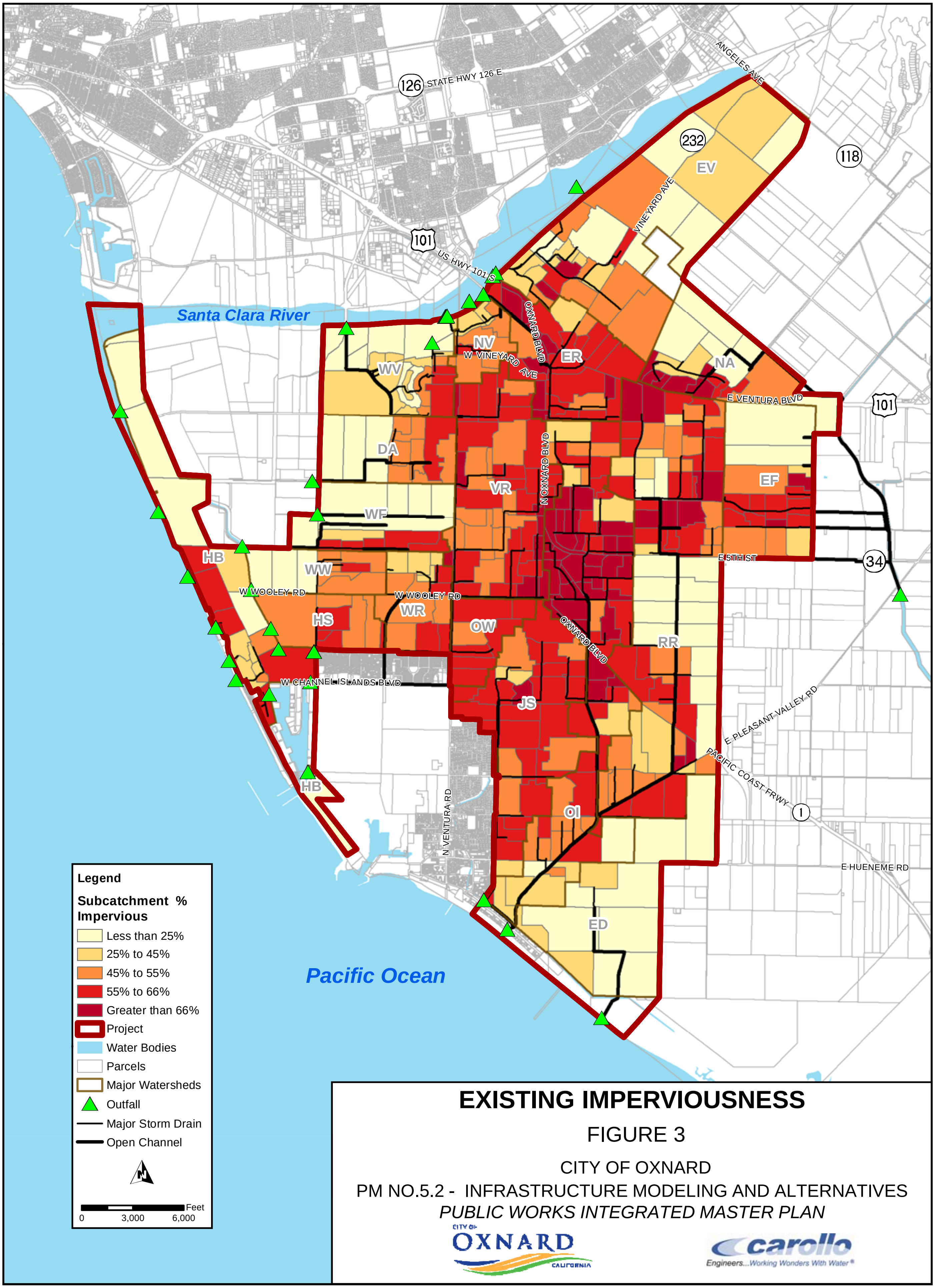
City of Oxnard

Land Use Category	Imperviousness (%)
Agriculture	3.2
Commercial	80.5
Industrial	66.8
Parks/Open Space/Resource Protection	17.5
Public	39.4
Residential	57.8

2.3.5 Ground Slope

Ground slopes were determined using the City's elevation data and ArcGIS. An average overland flow path slope was required for each subcatchment. This value was determined through intersection of subcatchments areas with the Digital Elevation Model (DEM) derived from the City elevation data points and survey data. The elevation grid was intersected with the subcatchments and the slope of each grid cell within the subcatchments was calculated. Using the number of cells within each subcatchments, the average subcatchments slope was calculated. To verify this procedure, slopes for selected subcatchments were manually estimated using available ground contour elevations and following guidelines provided by

[2] Percent developed imperviousness values are available for the City of Oxnard from the National Land Cover Database (NLCD) 2011 Edition - http://www.mrlc.gov/nlcd06_data.php.



EXISTING IMPERVIOUSNESS

FIGURE 3

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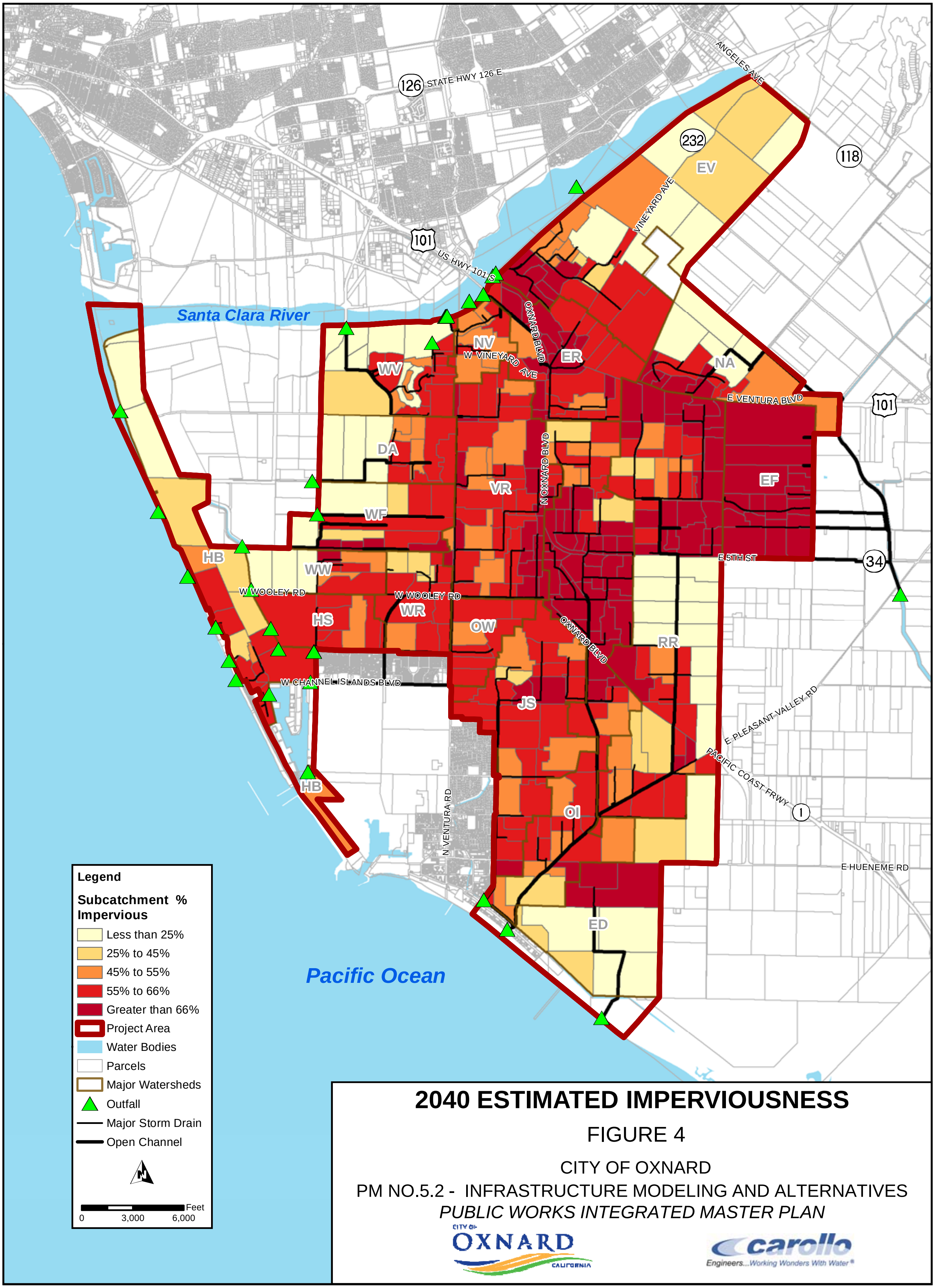

2040 ESTIMATED IMPERVIOUSNESS

FIGURE 4

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the hydraulic model manufacturer. Figure 2 located in PM 5.1 illustrates the project area topography.

2.3.6 <u>Manning's n-Value</u>

The overland flow travel time is affected by the types of surface covers. For each subcatchment, a roughness coefficient was input into the model for both pervious and impervious surfaces. Impervious areas were assigned a value of 0.024 and pervious areas were assigned a value of 0.35.

3.0 HYDRAULIC MODEL DEVELOPMENT

A storm drainage system model is a simplified representation of the City's actual storm system. The storm system model is used to assess the conveyance capacity for the drainage system. In addition, storm drainage system models can perform "what if" scenarios to assess the impacts of future developments and land use changes. The City's storm drainage system hydraulic model was constructed using a multi-step process utilizing data from a variety of sources. This section summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled collection system, the hydraulic model elements, and the model creation process.

3.1 Selected Hydraulic Modeling Software

SewerGems, by Bentley, was selected by the City for the sewer collection system model. Therefore, it was logical to use the same software for the storm drainage system model. SewerGems is a fully dynamic, geospatial wastewater and stormwater modeling and management software application. The hydraulic modeling engine for the SewerGems software package uses the Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM), which is widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems.

3.2 Modeled Stormwater Drainage System

Skeletonization is the process by which storm drainage systems are stripped of pipelines not considered essential for the intended purpose of analysis. The purpose of skeletonizing a system is to develop a model that accurately simulates the hydraulics of a drainage system while reducing the complexity of a large model.

It is common practice in stormwater system master planning to exclude small diameter pipelines when developing a hydraulic computer model. The City's hydraulic model includes pipelines that are 24-inches in diameter and larger. Some smaller diameter pipelines (less than 24-inches in diameter) were included in the City's hydraulic model if needed for connectivity, or if the pipelines serve a significant drainage purpose. Otherwise, pipelines 24-inches in diameter and smaller were excluded from the model.

The modeled stormwater system consists of approximately 59 miles (approximately 42 percent of the total system piping) of circular pipelines ranging from 8-inches to 96-inches in diameter, rectangular drains up to 62 feet by 12 feet, and other shaped conduits, and 34 miles of open channels. Some of the Ventura County channels were included in the hydraulic model. None of the pump stations were included as they service smaller areas that were not included in the skeletonization process. Figure 5 presents the City's modeled stormwater system.

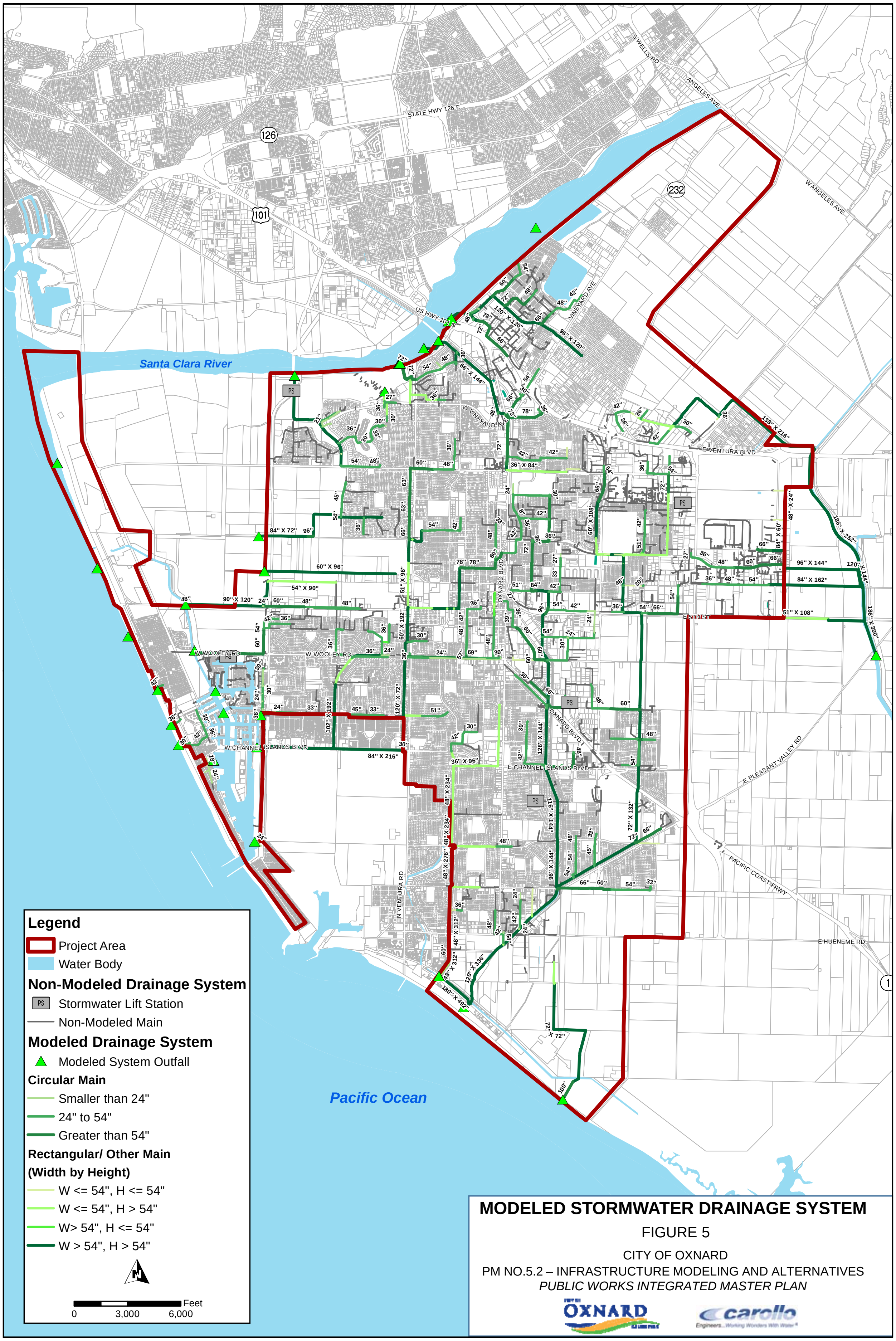
The 18 drainage basins presented in PM 5.1 were further divided into 418 drainage subbasins (or subcatchments) to provide detailed drainage areas that feed flows into the hydraulic elements of the system. These subcatchments are illustrated in Figure 6.

3.3 Elements of the Hydrologic/Hydraulic Model

The following provides an overview of the elements of the hydrologic/hydraulic model.

- **Junctions:** Storm manholes, catch basins, drainage inlets, as well as other locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. Required inputs for junctions include rim elevation, invert elevation, and surcharge depth (used to represent pressurized systems). Junctions are also used to represent locations where flows are split or diverted between two or more downstream links.
- **Pipes:** Gravity pipes and force mains are represented as pipes in the hydraulic model. Input parameters for pipes include length, friction factor (e.g., Manning’s n for gravity mains, Hazen Williams C for force mains), invert elevations, diameter, and whether or not the pipe is a force main.
- **Outfalls:** Outfalls represent areas where flow leaves the system. For storm system modeling, an outfall typically represents outfalls to canals or other waterways.
- **Rain Gauges:** Rain gauges are input into the hydraulic model to simulate historical or theoretical hourly rainfall events.
- **Subcatchments:** Subcatchments represent the hydrologic units of land area whose topography and drainage characteristics direct surface runoff to a single discharge point in the storm drainage system. Subcatchment parameters ultimately determine how much stormwater inflow enters the drainage system.

Analysis of the City’s storm drainage system was performed using the SWMM computational engine in SewerGEMS. The hydrologic portion of this engine is designed to simulate the surface water runoff response of a drainage basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components used to simulate the quantity of storm water runoff that flows overland in each subcatchment during a particular storm event.



MODELED STORMWATER DRAINAGE SYSTEM

FIGURE 5

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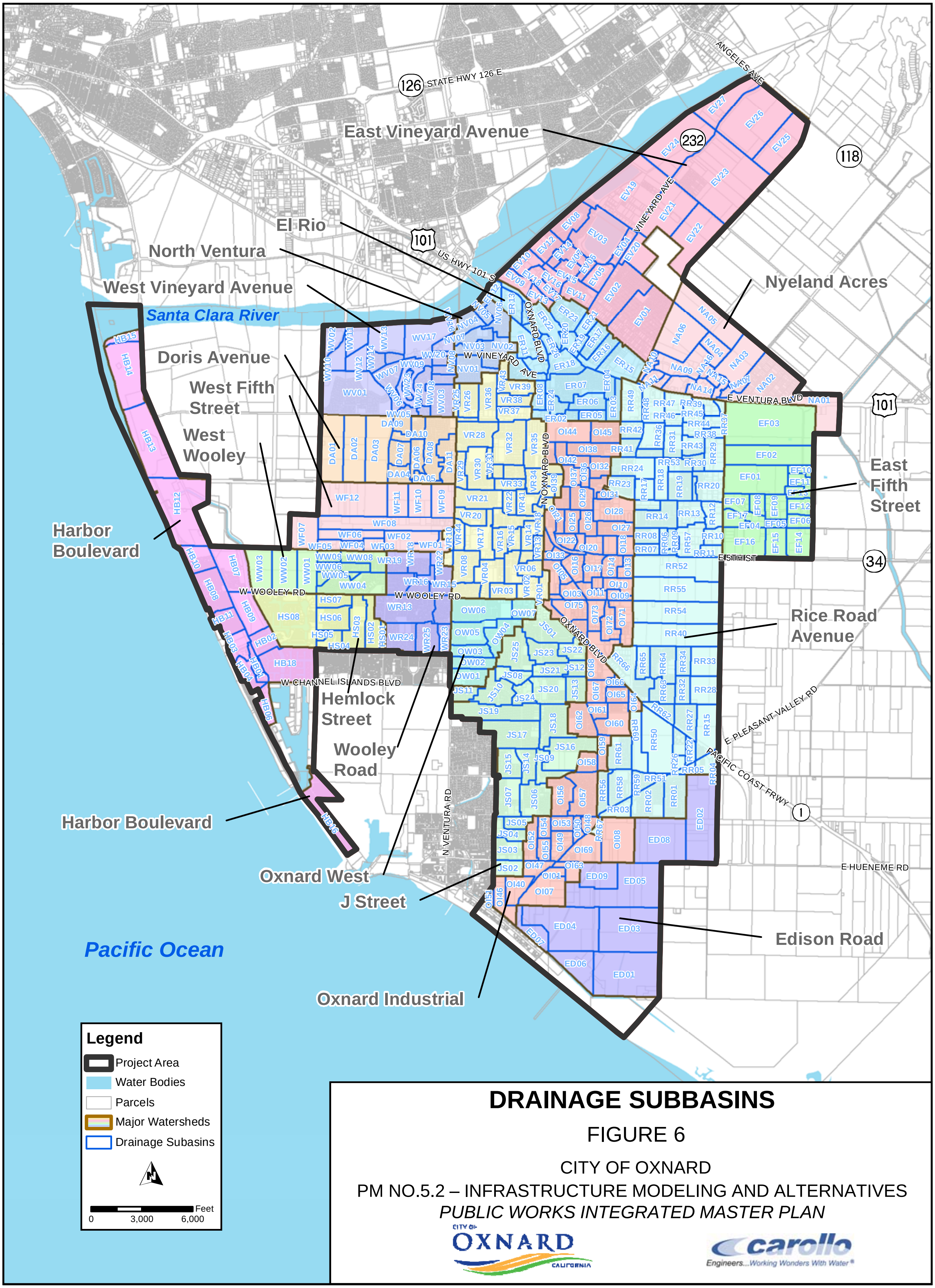

DRAINAGE SUBBASINS

FIGURE 6

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The input parameters for the hydrologic portion of the model are detailed below. The input parameters describe the various components of the model, including land use, soils, vegetation, drainage channels, and topography.

- **Design Rainfall.** Design hyetographs were determined for the 10-year, 50-year, and 100-year, 24-hour, design storms as detailed above.

- **Subcatchment Area.** The City 22,709 acres within the service area was divided up into 418 individual subcatchments and appropriate outlet points (i.e. drainage inlets and catch basins in City Streets, or nearby manholes) were defined. The resulting subcatchments range 1.7 acres to 374.9 acres and average approximately 54.3 acres. The larger subcatchments are outside the dense developed City area. The area and boundary of each subcatchment was determined with the use of development plans, available topographic data, and field observations to determine the drainage path. Determining the appropriate size of subcatchment is important in developing the modeled hydrologic characteristics of the City, because the size of the subcatchment (among other parameters) affects the peak and volume of water experienced at a single inlet point in the system. Subcatchments that are too large can create peak inflows that uncharacteristically overload portions of the storm drainage system, while subcatchments that are too small can be underestimates of peak flows experienced at different locations in the system.

- **Subcatchment Width.** The hydrologic model uses the width of each subcatchment to estimate the flow from the furthest point in the drainage area to the subcatchment outlet. This width parameter is the inverse of the typical time-of-concentration. Determination of this physical width of overland flow is difficult because it depends on storage and shape effects of the subbasin. The width is commonly used as a model calibration parameter to account for the impact of varying drainage characteristics within each subcatchment on flow travel time. However, due to inadequate data for calibrating the runoff from each subbasin, subbasin width was not considered as a calibration parameter in this analysis. Instead, the width was estimated first by determining the maximum length of overland flow within each subcatchment and dividing the area by this length. This method is recommended in the SWMM User's Manual.

- **Subcatchment Percent Imperviousness.** The model uses the percent imperviousness of each subcatchment to estimate runoff flows. Development of subcatchment imperviousness for both existing and future conditions is detailed above in section 2.3.4.

3.4 Hydraulic Model Components

The hydraulic model was used to simulate the hydraulic conditions in the City's storm drainage system, analyze the storm drainage system, identify deficiencies, and propose system improvements:

- **Flow Routing.** Flow routing within a conduit link is governed by the conservation of mass and momentum equations for gradually varied unsteady flow (i.e., the St. Venant equations). The user has a choice on the level of sophistication used to solve these equations:
  - Steady Flow.
  - Kinematic Wave Routing.
  - Dynamic Wave Routing.

The City's hydraulic model used Dynamic wave routing to analyze the storm drainage system. Dynamic wave routing solves the complete St. Venant flow equations and therefore produces the most accurate results. These equations consist of the continuity and momentum equations for conduits and a flow continuity equation at nodes.

Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples the solution for both water levels at nodes and flow in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions or flow regulation via weirs and orifices.

3.5 Model Construction

The hydraulic model performs calculations to solve a series of mathematical equations to simulate runoff from subcatchments and flows in pipes.

The model construction process consisted of six steps:

- **Step 1** - The City's geographic information system (GIS) shapefiles for the stormwater drainage system were obtained.
- **Step 2** - The GIS data were reviewed and formatted to allow easy import into the modeling platform. The City's GIS did include information on pipeline inverts. These data were reviewed and adjusted when necessary in order to be in the same datum. The areas with connectivity questions were sent to the City for review and several areas were checked with as-built drawings.
- **Step 3** - The City's GIS data were skeletonized to exclude pipelines less than 24-inches in diameter (except where needed for connectivity).

- **Step 4** - The drainage system pipeline, junctions and outfalls. Pipelines and junctions with invert discrepancies were reviewed and manually input or modified based on City records, field reconnaissance (survey), and engineering judgment.

  Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the data was entered correctly and that the flow direction and size of the modeled pipelines were logical.

- **Step 5:** Parameters describing the runoff characteristics of the model subcatchments were entered into the hydraulic model, including tributary drainage area, percent imperviousness, width, slope, and Manning's n factors (development of these values is discussed in Section 2.3). Assigned outlets of system subcatchments were confirmed.

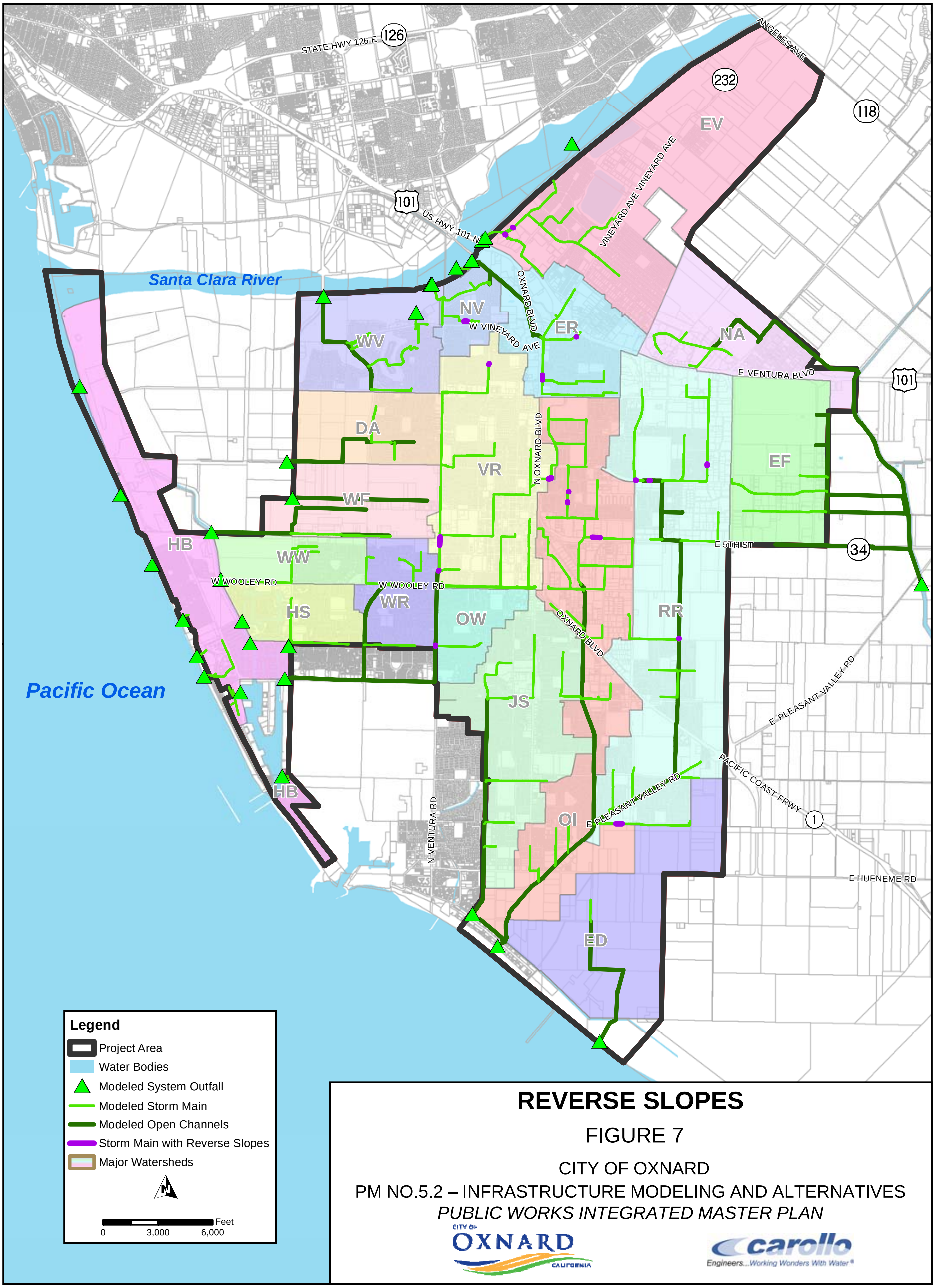
- **Step 6** - The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting parameters, output units, and flow routing method.

3.6 Reverse Slopes and Sump Conditions

There are multiple locations in the City's storm drain network where pipes are sloped adverse to the direction of flow, based on the survey and GIS information compiled by the City. There are approximately 20 pipe segments in the system with a negative elevation drop greater than 0.1 foot. This could be due to improper construction, settlement, or earthquake damage. Or this information may be wrong or outdated. Therefore, these locations should be further inspected by the City and the model updated accordingly. These pipes are illustrated in Figure 7.

In all cases, the negative elevation drop is less than the pipe diameter, meaning that stormwater could still flow by gravity once the depth of water exceeds the negative drop in elevation. However, the negative slope can cause sediment and debris to build up in pipes and manholes, further limiting hydraulic capacity. It is recommended to inspect and maintain these pipes on a regular basis. These pipes all surcharge during the 10-year, 24-hour design storm, however, no improvements are recommended for these areas as surcharging is only due to the reverse pipe slope.

During the model build process, numerous locations with sump conditions were found throughout the City's storm drainage network, where the upstream pipe invert is lower than the downstream invert. Most of these locations are City's owned pipes connecting to the Ventura channels. Some were due to datum difference in the GIS database and were corrected in the hydraulic model, however, the system still has several locations with this type of configuration. Some of these locations could experience severe flooding and should be monitored by the City. Surveying might be necessary to check elevations and confirm uncertainty in datums. Updates should be included in the GIS as well as the model.



REVERSE SLOPES

FIGURE 7

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3.7 Model Verification

The reasonableness of the model results and the hydraulic grade line (HGL) profiles were evaluated during the initial model runs. This was accomplished by comparing areas of flooding predicted by the model with observations offered by the City. Areas around the City that experience flooding were confirmed by the model results. Following the verification process, the model was used for the existing and future storm drainage system analysis.

4.0 SYSTEM CAPACITY EVALUATION

4.1 Level of Service

The City drainage criteria require that storm drains be designed to convey a 10-year, 24-hour design storm and building finish floor levels be above a 100-year, 24-hour flood level. This provides an acceptable level of protection when combined with the requirement that sumps be designed for a 50-year, 24-hour storm, and an emergency escape path be provided. A 10-year level of protection is a common standard for urban areas and it provides a reasonable level of protection while maintaining capital improvements within reasonable limits. The recommended planning criteria for the storm drainage portion of this PWIMP are summarized in Table 4.

Table 4 Level of Service Criteria
Public Works Integrated Master Plan
City of Oxnard

Design Storm	Facilities to be Evaluated	Maximum HGL Depth/Flooding Depth Criteria
10-year, 24-hour	Storm Conveyance Facilities and Basins	Surcharging allowed, but no flooding above surface elevation
100-year, 24-hour	Combined Capacity of Streets, Basins, and Pipes	Flooding allowed not higher than the building finish floor levels

4.2 Basin Runoff Results

The peak runoff flows at each outfall from the 10-year design storm were compared between existing and year 2040 conditions. Table 5 shows the peak runoff for the two scenarios. Outfalls locations are shown on Figure 5 presented above.

Table 5 Peak Outfall Runoff
Public Works Integrated Master Plan
City of Oxnard

Model Outfall ID	Peak Outfall Runoff (cfs)[1]		
	Existing	Year 2040	Increase (%)
DA	214	230	6.8%
ED	187	407	54.1%
EF	651	1,377	52.7%
ER01	209	209	0.0%
ER02	384	395	2.7%
ER03	21	21	0.0%
EV	528	633	16.6%
EV-Future	455	455	0.0%
HB01	34	34	0.0%
HB02	30	40	24.0%
HB03	14	14	0.7%
HB04	59	59	0.0%
HB05	88	127	30.9%
HB06	25	66	61.1%
HB08	69	71	2.5%
HB12	41	65	36.6%
HS01	95	95	0.2%
HS02	88	98	9.4%
HS03	43	89	51.5%
Tsumas Creek	697	713	2.2%
Kiddie Beach	11	55	80.5%
NV01	54	58	6.4%
NV02	54	58	6.4%
NV03	117	131	10.4%
OLW	2,409	2,577	6.5%
WF01	23	82	71.7%
WF02	144	200	28.2%
WR & OW	1,147	1,181	2.9%
WV01	51	74	31.0%
WV02	218	282	22.7%
WW	143	149	4.6%

Note:
(1) cfs - cubic feet per second.

4.3 Storm Drain Surcharge

4.3.1 Surcharge Due To Capacity Deficiencies

For a gravity storm drain (not including inverted siphons flowing full), a surcharged pipeline occurs when the hydraulic grade line rises above the crown elevation of the pipe, or when d/D exceeds 1.0 (flowing over 100 percent full), under design storm conditions. A surcharged condition for gravity pipes is caused by the existing sewer being unable to convey the peak flow due to insufficient diameter. Surcharging is an indicator of pipes at risk of overflow due to insufficient hydraulic capacity. However, pipe surcharging does not necessarily translate to overflow above manhole rims. Capacity improvements to address deficiencies may be implemented for the surcharging pipe by upsizing the pipe or by improving downstream pipelines that limit capacity due to backwater effects.

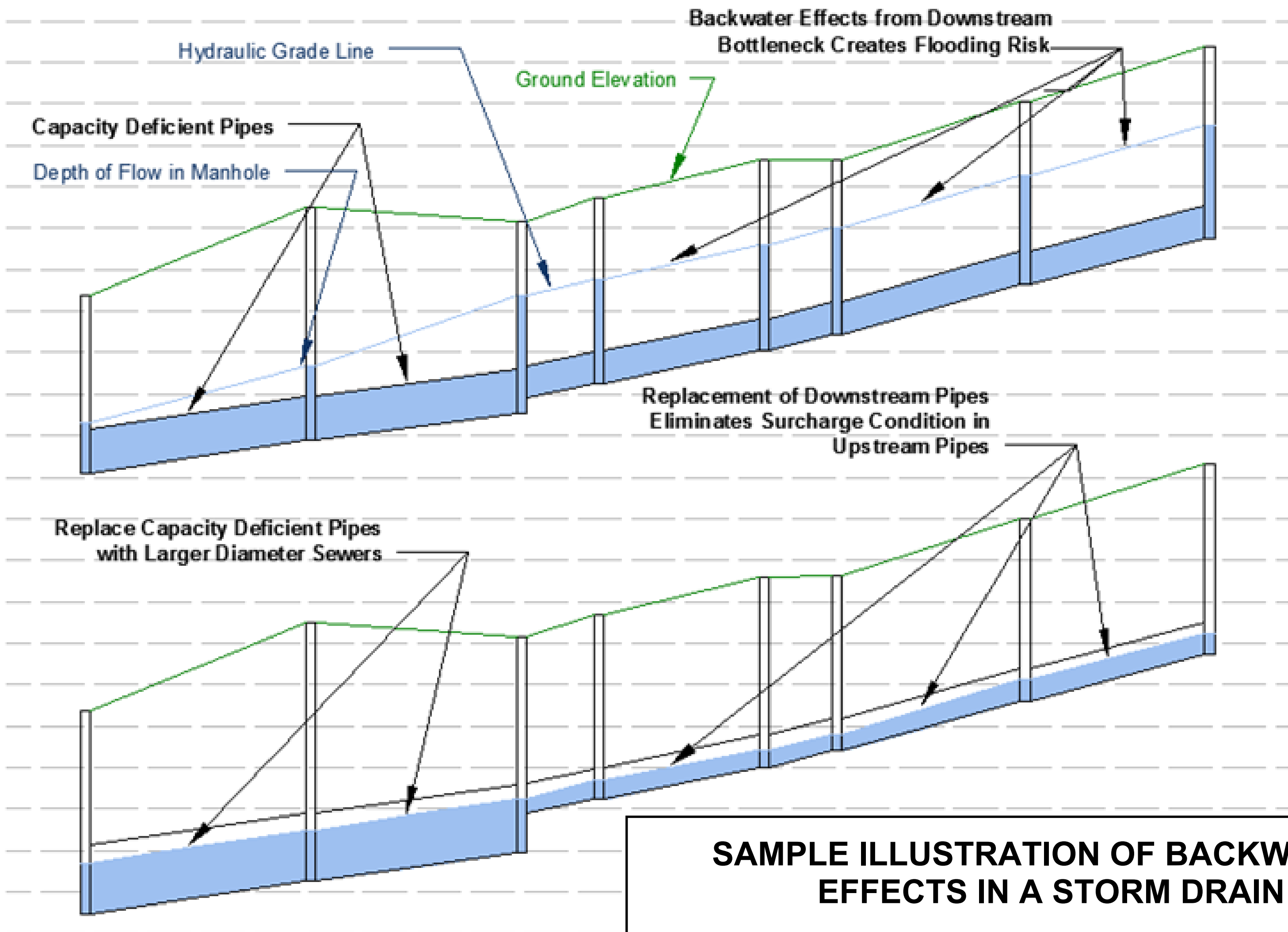
4.3.2 Surcharge Due To Backwater

Surcharging of a pipe can occur due to backwater effects of a downstream pipe, not due to insufficient capacity of the pipe itself. If the downstream pipe is capacity deficient, it can cause backup - and even reversal of flow in the upstream pipe - resulting in surcharge of the upstream pipe that otherwise is not capacity limited. If the downstream pipe capacity is increased, then the upstream pipe may no longer require capacity improvements, as illustrated in Figure 8. Therefore, it is recommended that the City address downstream issues to limit the overall cost of the Plan, rather than backwater improvements.

An analysis was performed on model results to determine if indicated surcharging was caused by inadequate hydraulic capacity of that pipe, or by backwater effects from pipes downstream. The analysis was conducted by comparison of the peak wet weather flow conveyed by the pipe in the model to the pipe’s calculated hydraulic capacity and through the model hydraulic profiles.

4.3.3 Backwater Effects from Ventura Channels

During the 10-year, 24-hour design storm, the HGL in the Ventura channels is elevated and results in significant surcharging in the City's storm pipes draining to the channels due to backwater effects. No improvements to the City's drainage pipes due to this type of surcharging is proposed because it's not a capacity deficiency in City pipes, but in the conveyance capacity of the Ventura channels. It is rather recommended to improve the Ventura channel conveyance to lower the HGL in these channels and allow more stormwater to drain to these canals without being held upstream in the City's system. Figure 9 illustrates the locations where these hydraulic conditions are present. Therefore, the City should work closely with Ventura County to remediate these situations. If the Ventura channels are designed to accept all flows from the City without causing backwater conditions, then Ventura County will need to improve the channels to meet this hydraulic



SAMPLE ILLUSTRATION OF BACKWATER EFFECTS IN A STORM DRAIN

FIGURE 8

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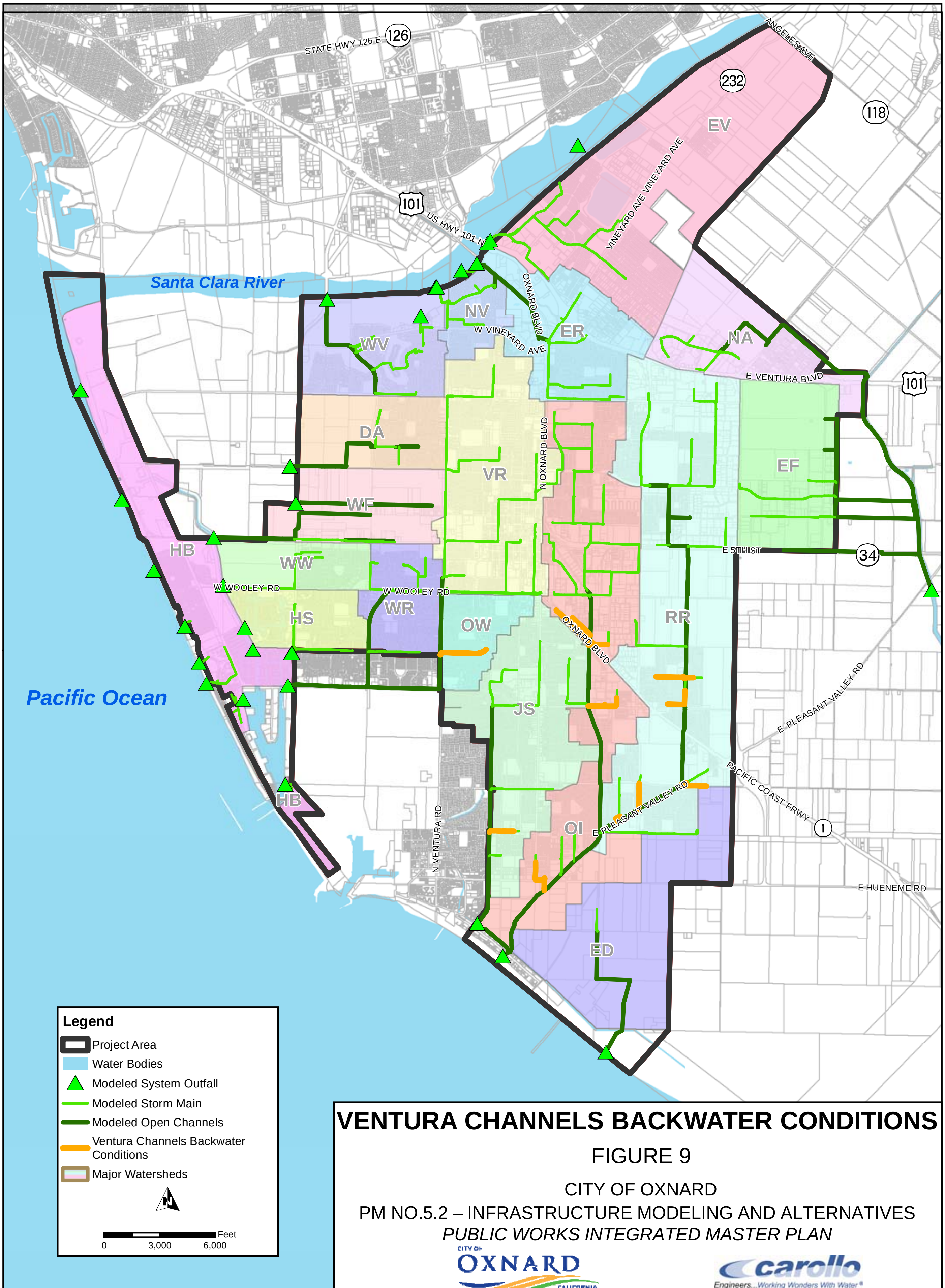

VENTURA CHANNELS BACKWATER CONDITIONS

FIGURE 9

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criteria. If this is not possible, the County could work with the City to potentially provide storage facilities to attenuate the flows in upstream areas and thus reduce the peak flows in the Ventura channels.

4.4 Current Conditions

Evaluation of the capacity of the City' storm drainage system involved identifying areas in the system where surcharging and flooding occurred under the 10-year. Pipelines and channels that lacked sufficient capacity to convey runoff generated from the design storm could produce backwater effects in the drainage system and potentially cause excessive flooding. This section discusses the locations of existing deficiencies.

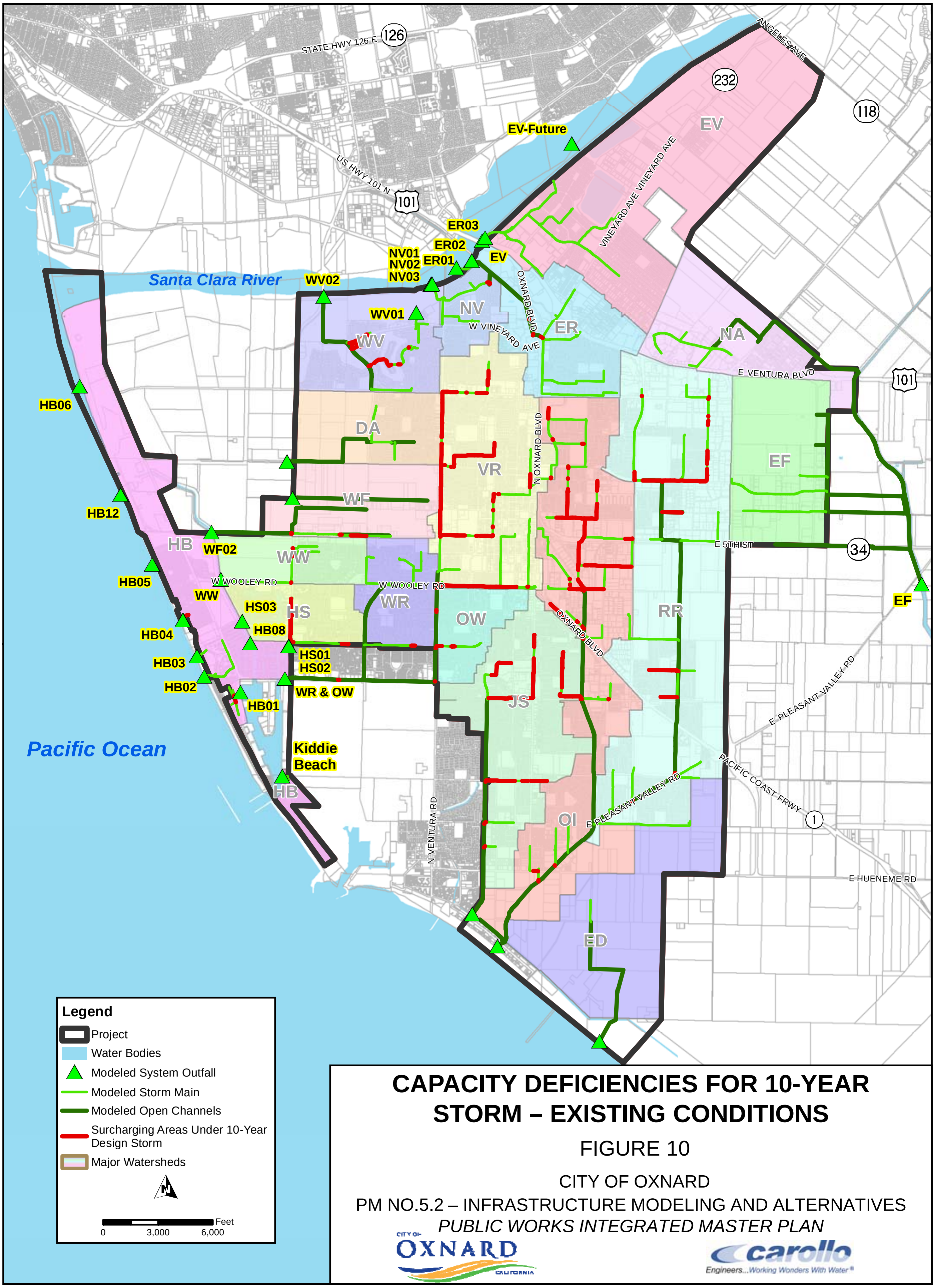
Figure 10 provides the locations of the existing problem areas identified during modeling. The majority of the surcharging and flooding problems under the 10-year design storm are located in Ventura Road, "J" Street, Oxnard Industrial, and north of Rice Road Avenue watersheds, which correspond to the downtown core of the City. It is in similar locations that the existing storm drain system lacks sufficient capacity to convey the 100-year design runoff while meeting the flooding criteria. Areas with existing deficiencies are dispersed throughout the City, but area generally limited to several locations where larger interceptors are required to convey flows collected from large tributary areas. Figure 11 shows the location of the areas flooding during a 100-year storm.

4.5 2040 Conditions

As the City develops beyond the currently developed areas, the storm drainage system will grow. Growth of the City by 2040 will add residential, commercial, and industrial areas. As new development is added, the percent imperviousness of the catchment is increased and thus the volume and peak flow will increase, potentially causing additional flooding. Therefore, existing storm drain infrastructure many have to be upgraded to accommodate this additional flow due to development. The impact of growth on the existing drainage system is illustrated in Figure 12.

5.0 GREEN ALLEYS PLAN

The City recently completed a Green Alleys Plan, the goal of which is to determine which alleys in the City are good candidates for green alley projects and to provide a framework to guide the future design and implementation of green alley projects. The report focuses on the locations of potential alleys' improvements. The plan conducted an analysis in order to prioritize identified green alleys neighborhoods mainly through GIS analysis. One of the four models developed in this plan was an environmental impact analysis model and neighborhoods were prioritized based on impermeable cover, tree coverage, and alley area. As illustrated in Figure 8, highest priority areas for environmental green alley improvements are located throughout the heart of the City.



CAPACITY DEFICIENCIES FOR 10-YEAR STORM – EXISTING CONDITIONS

FIGURE 10

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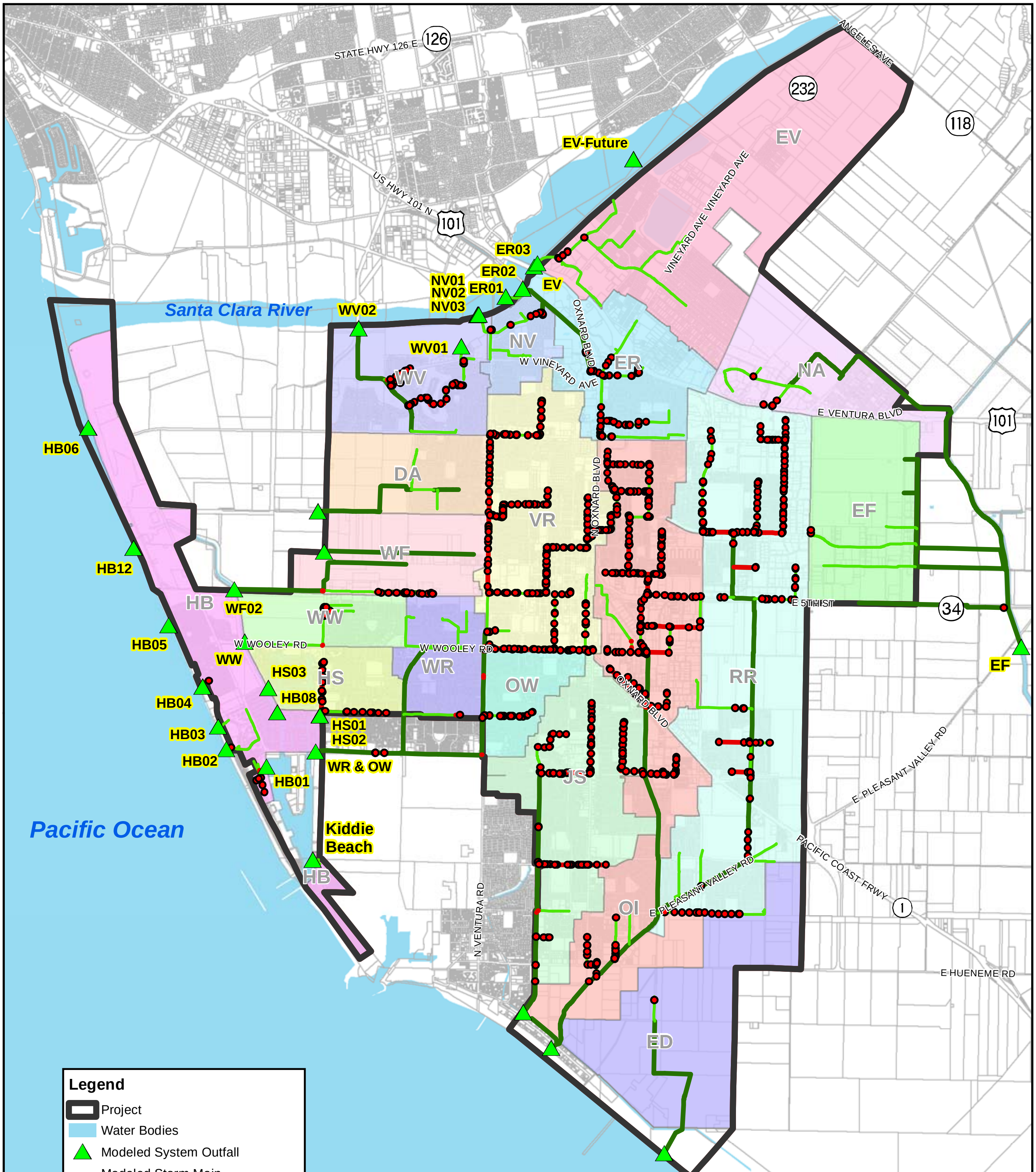

FLOODING AREAS DURING 100-YEAR, 24 HOUR DESIGN STORM – EXISTING CONDITIONS

FIGURE 11

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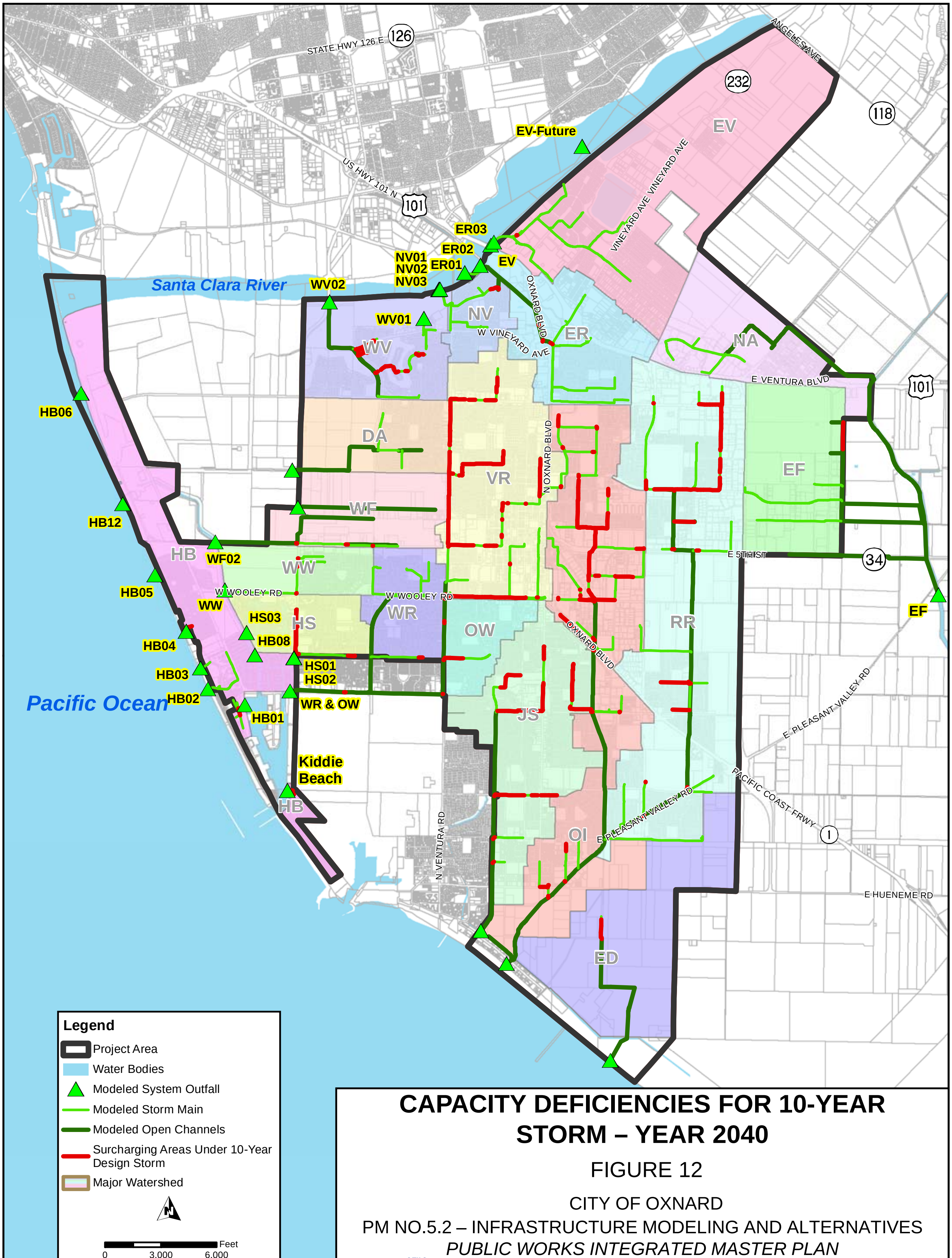

CAPACITY DEFICIENCIES FOR 10-YEAR STORM – YEAR 2040

FIGURE 12

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Hydraulic modeling analysis determined areas of flooding in the current City's stormwater drainage system, as illustrated in Section 4.0. After comparison with the environmental prioritization results performed in the Green Alley program, it was noted that some of the high priority public alleys overlap with the observed areas of flooding. It is recommended that the City incorporate bioswales, permeable paving, or rain barrels (for community gardens) when appropriate to help decrease flooding in these locations. Figure 13 shows the environmental prioritization analysis results, along with existing flooding areas.

6.0 PROPOSED SYSTEM IMPROVEMENTS

Selecting the most appropriate capital improvement projects for a specific problem area is heavily influenced by the development conditions of the watershed in which the target problem area is located. There are four basic approaches to stormwater management applicable to the City. These are:

- No action.
- Upgrade the existing system.
- Storage/detention.
- Green stormwater infrastructure (GSI) such as infiltration systems.

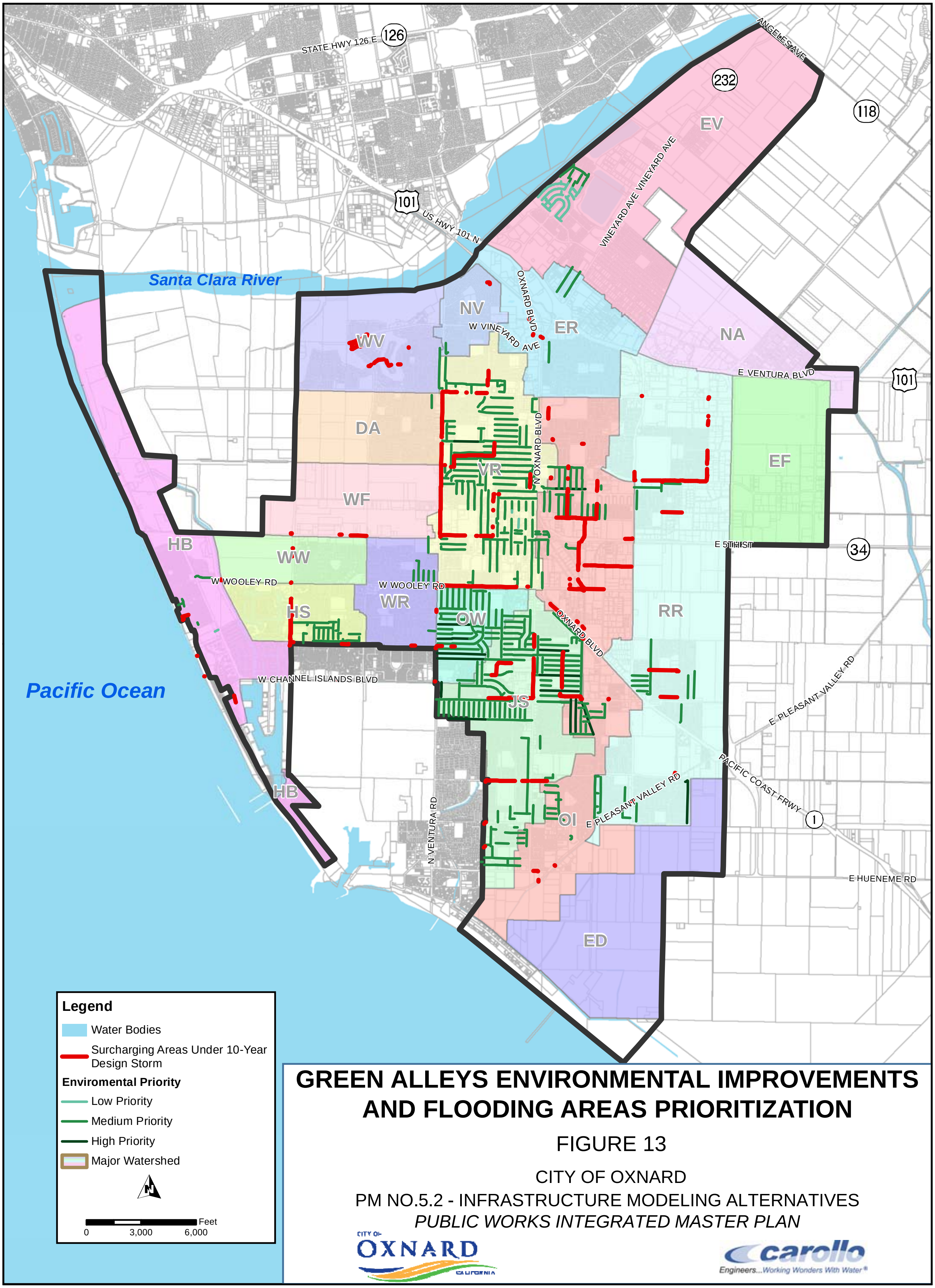
These basic techniques may be implemented singly within a basin or in combination to manage present and expected future stormwater problems.

6.1.1 No Action

The no action approach implies that no improvements whatsoever will be made to the existing drain system. It will be included in the analyses for all ten basins for comparison purposes. It is always possible to not improve the system, at the cost of continued damages and inconveniences where drainage facilities are inadequate or nonexistent. To ensure that system improvements are justified, it is necessary to compare the costs and advantages of those improvements to the no action alternative.

6.1.2 Upgrading the Existing System

This approach would involve constructing replacement or parallel pipes and upgrading or piping existing ditches to provide adequate capacity for the design flow. Upgrading of existing ditches may consist of vegetation and debris removal, regrading, shaping, and channel enlargement. This is often the most straightforward alternative since it involves the existing drainage system and easements.



GREEN ALLEYS ENVIRONMENTAL IMPROVEMENTS AND FLOODING AREAS PRIORITIZATION

FIGURE 13

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6.1.3 Storage/Detention

The concept of runoff detention is simple: hold back the excess upstream runoff that would cause flooding problems downstream. This excess water is released later at a rate the downstream drainage structures are capable of conveying without flooding. The rate of release from the detention site may be based on the capacity of existing downstream drainage structures. Alternatively, the rate of flow release may be a reduction to a lesser design storm flow (e.g., the system design storm may have a 10-year recurrence interval, and the detention facility outlet may be sized to release only 5-year storm flows). Runoff detention facilities can be on-site or regional.

On-site detention may be accomplished using small detention ponds, underground pipe storage, or rooftop and parking lot detention. Regional detention basins are defined as basins, which receive runoff from a large drainage area, usually tens to hundreds of acres, and are large enough to attenuate the peak in that runoff. A policy of requiring on-site detention in residential areas results in several small detention facilities throughout the community. On-site detention in commercial and industrial areas generally consists of parking lot and rooftop detention. This can be a feasible option where large parking lots or structures are available and will be well maintained. These alternatives will be evaluated at a later date and will allow to refine the proposed improvements presented in Section 6.3.

6.1.4 Green Stormwater Infrastructure (GSI)

GSI is based on the concept of keeping or returning a watershed to more natural conditions in which rainwater can effectively infiltrate into the soil and thus not cause extensive peak flows downstream. A concept similar to detention ponds is infiltration ponds. Flow is routed into a pond, as with detention, but the runoff is not released. Rather, the stormwater infiltrates into the soil. This option would only be feasible in a location having soils with high infiltration rates (hydrologic soil types A and B), and an overflow route should be included in design of infiltration facilities. Such a facility would dispose of stormwater without taxing existing storm drains with runoff from existing or new development areas. Swales, permeable pavement, and other GSI techniques are also available. GSI, beyond the brief discussion above the Green Alley Program will be limited in this PM. These techniques can be further explored in Phase 2 during the optimization approach.

6.2 Approach

For conveyance improvements that address flooding problems, a new parallel pipe to the existing pipe was generally proposed. For the purposes of this PWIMP, it is assumed that the new parallel pipe will be installed at the same slope and alignment as the existing pipeline and will have the same size as the existing pipe. However, the decision to replace the existing pipe with a larger diameter pipe or parallel the existing pipeline should be made during the preliminary design phase for each specific project. The flow capacity criteria used to determine whether an existing pipeline should be replaced is based on the existing pipeline's hydraulic capacity. Replacement is "triggered" when a pipeline in the model has a peak HGL at a greater slope than the slope of the pipes and cause upstream flooding. Additional modeling will be conducted to refine the preliminary improvements recommended in this PM during Phase 2 of the PWIMP.

6.3 Recommended Piping Improvements

The basis of the CIP projects are the results of the hydraulic modeling assessment, and application of the replacement criteria presented above. Table 6 below summarizes all proposed improvement projects. A total of 13 storm drainage improvement projects with a total of 18,998 feet of drains, approximately 3.6 miles have been identified to address the deficiencies predicted by the system analysis.

The columns used in Table 6 refer to the following:

- ID: Each pipe segment is assigned an ID. This is an alphanumeric number that starts with the CIP project number, which is one letter indicating the type of project (P = pipe) and is followed by a number. The projects are grouped by geographical location. Projects located in the same area but with different existing size pipe have an additional letter following the number (for instance, P-4A).
- US Manhole: Model Node ID of the upstream (US) manhole of a proposed pipeline project.
- DS Manhole: Model Node ID of the downstream (DS) manhole of a proposed pipeline project.
- Drainage Basin: This is the drainage basin the project is located in.
- Location: Street in which the improvement is proposed.
- Ex. Size.: Diameter of the existing pipeline (in inches).
- Length: Estimated length of the proposed pipeline (in feet).
- Phase Ranking: This is the proposed project ranking as described below.

6.4 Project Prioritization

To assist in future planning, all proposed improvements presented above are grouped in accordance with four priorities that were defined upon urgency of needs. The four phase rankings are defined as follow:

1. Phase 1: Proposed facilities located in an area that has a significant flooding problem under existing modeling conditions for the 10-year design storm. These facilities are primarily located in the most developed part of the City.
2. Phase 2: Proposed facilities located in an area that has a significant flooding problem under planning year 2020 modeling conditions for the 10-year design storm.

Table 6 **Recommended Improvement Projects and Prioritization**
Public Works Integrated Master Plan
City of Oxnard

Project ID	Upstream Manhole	Downstream Manhole	Basin	Location	Ex. Size	Length (ft)	Phase Ranking
P-1	CB-J15-308	CB-I15-306	WV	Along Indian Wells Ct between N Patterson Rd and Spyglass Trl W	24-inch	444	2
P-2	MH-J15-111	JS-J16-503	WV	Along Bermuda Dunes Dr between N Patterson Rd and Bermuda Dunes Pl	36-inch	748	4
P-3	JSM-O17-505	MH-O17-106	OI	Along Entrada Dr north of Martin Luther King Jr Dr	24-inch	607	2
P-4A	MH-S20-100	TS-R20-810	RR	Along Camino Del Sol between N Graves Ave and Gibraltar Street	4.5' x 6.58'	901	3
P-4B	TS-R20-810	TS-R20-807	RR		4.5' x 7.5'	500	3
P-4C	TS-R20-807	IN-R20-1006	RR		4.5' x 7.75'	218	3
P-4D	MH-S20-103	MH-S20-100	RR		4.5' x 6.25'	817	3
P-5A	MH-P23-105	MH-P23-107	OI	Along Pacific Ave between Mountain View Ave and E Wooley Rd and along E Wooley Rd between Pacific Ave and Richmond Ave	30-inch	741	4
P-5B	MH-P23-107	JS-P24-504	OI		42-inch	1,647	4
P-6A	TS-L18-810	TS-L19-807	VR	Along S Ventura Rd between W Fifth St and Devonshire Dr., Wagon Wheel, and Channel Islands Blvd	66-inch	1,169	1
P-6B	TS-L21-800	JS-L22-504	VR		3' x 8'	1,402	1
P-6C	JS-L20-507	TS-L21-800	VR		4.25' x 8'	2,133	1
P-6D	TS-L19-807	JS-L20-507	VR		5' x 7.25'	1,168	1
P-7A	MH-O27-107	MH-P28-109	JS	Along E Channel Islands Blvd between Cloyne St and Alley St	42-inch	1,369	1
P-7B	MH-P28-110	JS-P28-504	JS		36-inch	52	1
P-8	JS-O27-508	TS-O28-800	JS	Along Saviers Rd between Coach C Ln and Redwood St	3' x 5'	1,292	2
P-9	MH-N28-103	JS-N28-504	JS	Along Redwood St between Saviers Rd and S "A" St	3' x 6'	426	2
P-10	JS-N28-503	MH-N28-101	JS	Along Redwood St between S "A" St and S "C" St	3' x 7'	457	2
P-11	JS-N28-500	JS-N28-511	JS	Along Redwood St between S "C" St and Oleander Dr	3' x 8'	655	2
P-12	JS-N28-509	JS-M28-503	JS	Along Redwood St between Oleander Dr and S "J" St	3' x 8'	701	2
P-13	CB-H25-304	TS-H26-803	HS	Along S Victoria Ave between Leeward Way and W Hemlock St	24-inch	1,552	2
Total (ft.)						**18,998**	
Total (mile)						**3.6**	

3. Phase 3: Proposed facilities located in an area that has a significant flooding problem under planning year 2030 modeling conditions for the 10-year design storm.

4. Phase 4: Proposed facilities located in an area that has a significant flooding problem under planning year 2040 modeling conditions for the 10-year design storm.

Figure 14 illustrates the locations of the identified improvements and their phase rankings. Hydraulic profiles of each proposed project can be found in Appendix A. These profiles compare the HGL during existing and year 2040 conditions and show the impact the proposed improvements has on the HGL at these locations.

6.5 Condition Assessment

A total of 304 pipes, catch basins, manholes, and channels were assessed in this condition assessment report. Limited areas that have flooded in the past were also investigated. This represents approximately 2 percent of the entire stormwater collection system. Assets for inspection were chosen based on age, slope, and proximity to areas prone to flooding. Groupings of old assets with small slopes near areas prone to flooding were top priority. Overall, 29 sites were assessed. Additional information and details on site specific findings are provided in PM 5.3, Stormwater System - Condition Assessment.

This condition assessment found that approximately 12 percent of the assets assessed need immediate attention or need attention within the next five years. The location of these assets is illustrated in Figure 15 where phase 4 assets in orange are in poor conditions and phase 5 assets in red require immediate attention.

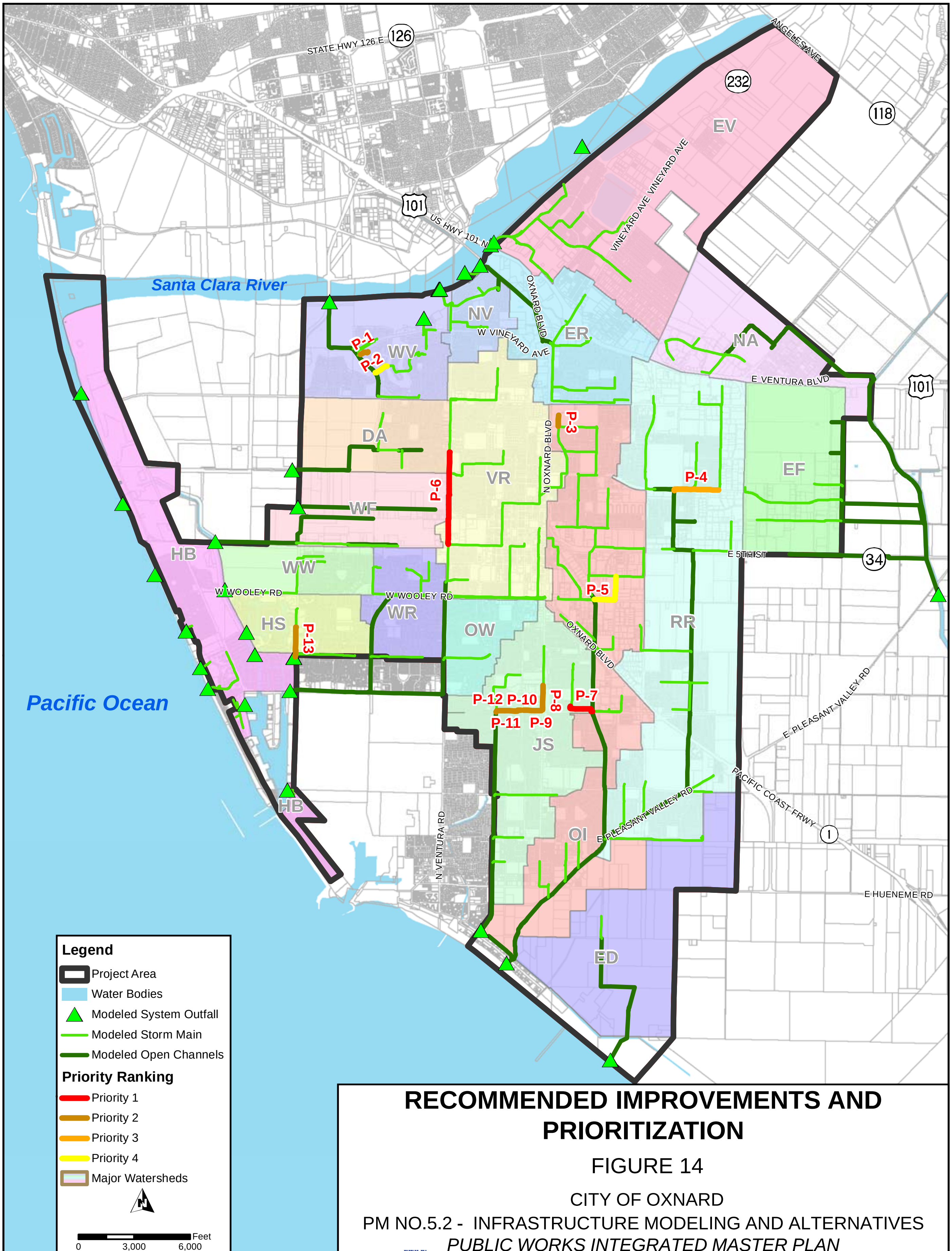
The location of these high priority condition assets was overlay with improvements recommended for capacity purposes. One condition area overlaps with the proposed improvements presented in the section above. The City should consider combining improvements to this area located at the intersection of E Wooley Rd and Pacific Ave.

6.6 Discussion and Additional Recommendations

6.6.1 Additional Studies Recommended

Additional analysis should be performed on all recommended flooding projects prior to construction. In some cases, additional solutions or alternatives could be considered. Additional information should also be collected to evaluate potential construction issues and to provide more detailed cost estimates. A detailed field survey should be performed to determine accurate flood damage elevations on potentially affected private property.

Additional studies are recommended for potential mitigation projects, ravine-top discharges, detention pond retrofits, and ditch retrofits.



RECOMMENDED IMPROVEMENTS AND PRIORITIZATION

FIGURE 14

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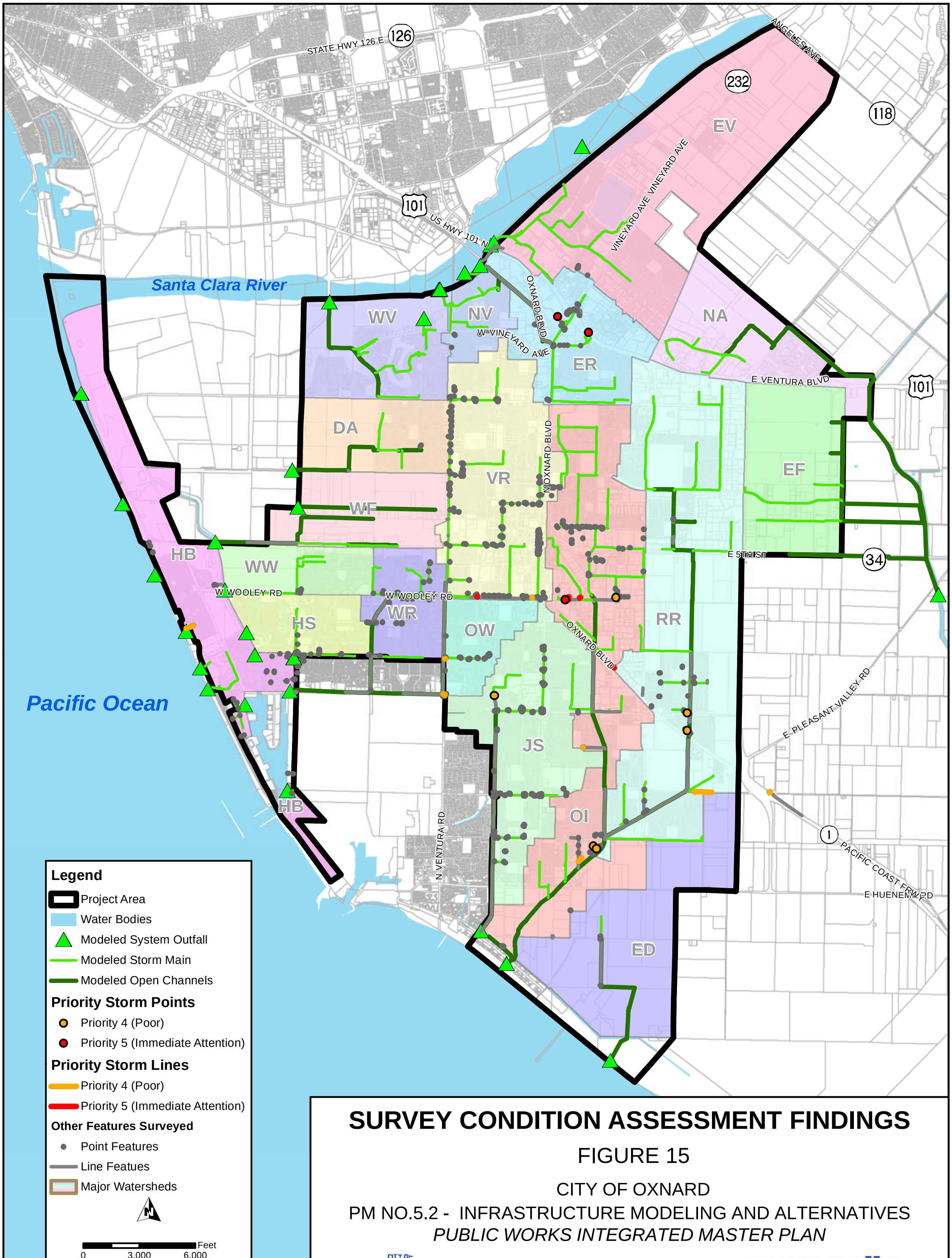

SURVEY CONDITION ASSESSMENT FINDINGS

FIGURE 15

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6.6.2 Potential Mitigation Projects

Some other alternatives to proposed projects could still be considered as potential mitigation projects. In particular, the small detention ponds in the watershed could potentially be considered as mitigation projects to help offset the increases in flow related to future development. The City may wish to further analyze the potential for these or other projects to help reduce future peak runoff.

It is generally recommended that for situations when the City plans to use heavy equipment near streams for earthwork on projects (such as replacing culverts), the City might also consider installing large woody debris (typically downstream of the project) and plant conifer trees as appropriate for site conditions. These activities are intended to incrementally improve habitat conditions at a marginal additional cost to projects.

6.6.3 Additional Modeling Evaluation

The improvements presented in this PM consist of preliminary recommendations and further modeling should be performed to refine these projects and include other types of mitigation projects, such as LID or retention during Phase 2 of the PWIMP.

6.6.4 Further Development Connection Assumptions

Part of the City's study area is currently undeveloped and consists of mainly open space and agriculture type usage with no drain storms. As the City develops these areas, new storm drains should be constructed to collect runoff from these future areas. The expansion of the City's current drainage system into these undeveloped areas will be evaluated during Phase 2 of the PWIMP.

7.0 RECOMMENDED PROJECT – COSTS AND PRIORITIZATION

Cost estimates, implementation phase and schedule were also developed for the recommended projects for the water supply and treatment projects, as summarized in the previous section. This information will be included in the overall Capital Improvement Program (CIP) and used as the basis for the financial analysis portion of the PWIMP to determine financial impact of the project to the City and its rate payers. The costs and timing presented in this PM represent Carollo's best professional judgment of the capital expenditure needs of the City and of the timing needed to maintain a reliable and compliant system that can meet current and future stormwater needs. Timing was set to align with the seven master plan drivers, namely: R&R, regulatory requirements, economic benefit, performance benefit, growth, resource sustainability, and policy decisions. Timing is also based on input from City staff and the condition assessments performed.

While the costs developed in this PM match the costs analyzed as part of the Cost of Service Study, the timing presented may differ. The Cost of Service Study will balance not only the CIP projects identified but also the rates and rate payer affordability based on a yearly balance and also the integrated costs for the different City funds and enterprises.

7.1 Cost Summary

The cost estimates presented in this section have been prepared for general master planning purposes and for guidance in project evaluation and implementation. Final costs of a project will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors such as preliminary alignment generation, investigation of alternative routings, and detailed utility and topography surveys. A more detailed discussion of the basis of costs is included in PM 1.4, *Overall - Basis of Cost*.

7.1.1 Construction Unit Costs

Base construction cost estimates used in developing the rehabilitation/replacement CIP were based upon the unit costs presented in Table 7. The unit costs are for “typical” open-trench field conditions utilizing trench boxes in stable soil. The unit costs include:

- Pipe, lower lateral, and manhole materials and appurtenances.
- Conventional open-trench w/ trench box type shoring, excavation, and backfill.
- Pavement removal and replacement.

The unit cost estimates presented in Table 7 were derived assuming conventional open trench construction methods. The specific method of construction for each specific project should be determined on a project-by-project basis during preliminary and final design efforts.

**Table 7 Pipeline Unit Cost
Public Works Integrated Master Plan
City of Oxnard**

Pipe Size (inch. / ft.)	Unit Cost ($/lf)(1)(2)(3)
24-inch	252
30-inch	316
36-inch	379
42-inch	442
66-inch	694
96-inch	1,010
3 by 8 feet and less	442

Table 7 Pipeline Unit Cost Public Works Integrated Master Plan City of Oxnard	
Pipe Size (inch. / ft.)	**Unit Cost ($/lf)(1)(2)(3)**
4.75 by 11 feet and less	694
8 by 6.75 feet and above	1,010

Notes:
(1) Unit project replacement costs include pipe and pipe installation, manhole and appurtenances, allowance for lower lateral replacement, open-trench excavation w/ trench box, backfill, pavement removal and replacement, and contractor overhead and profit.
(2) $/lf = unit project cost per lineal foot.
(3) Unit costs do not include replacement of upper laterals.

Using the cost assumptions presented in the above sections, project cost estimates were developed and are summarized in Table 8. The total improvement cost is estimated at $14.7 M, where $6.7 M (or 46 percent) are phase 1 (high priority) projects.

7.1.2 Rehabilitation Projects

In addition to the projects developed in the sections above, costs were also allocated in the CIP for rehabilitation of structures and pipes identified as deficient in the condition assessment done in *PM 5.3 - Stormwater - Condition Assessment.* Rehabilitation costs for pipes and catchment basins identified with a Level 4 or 5 rating (Poor or Immediate Attention, respectively) were included in the CIP and shown as the last project in Table 9. There were a total of 21 assets with a Level 4 rating and one asset with a Level 5 rating.

One location that the City would like specifically addressed in the near term is flooding that occurs in the Perkin Road area near the OWTP. This area was part of the condition assessment (see PM 5.3). This area represents Site 5 in the condition assessment report (page 63) and was inspected. One of the catch basins in the area was flooded during dry weather so there may be excessive debris and/or sediment that is trapped in the stormwater outfalls that drain this area. Further inspection of this area is needed by the City to ascertain if this is a maintenance issue or if additional capital improvements may be needed for rehabilitation purposes. Since this area is tidally influenced, there may be limited options.

7.2 Prioritization

As noted in Section 6.4 and 6.5, the recommended projects were ranked according to their need for storm capacity as well as condition of the existing system infrastructure. The phase ranking of each project is included in Table 8 as well as the recommended projects are summarized by phase in Table 9.

Table 8	Recommended Projects, Cost Estimates, and Phasing for Stormwater System Drainage Basins[(1)(2)(3)] Public Works Integrated Master Plan City of Oxnard			
Project ID[(1)]	**Drainage Basin**	**Length (ft)**	**Recommended Project Cost ($)**	**Phase Ranking**
P-1	WV	444	$173,403	2
P-2	WV	748	$439,233	4
P-3	OI	607	$237,094	2
P-4	RR	2,436	$2,620,545	3
P-5	OI	2,388	$1,491,298	4
P-6	VR	5,872	$5,768,336	1
P-7	JS	1,421	$968,359	1
P-8	JS	1,292	$885,191	2
P-9	JS	426	$292,038	2
P-10	JS	457	$312,883	2
P-11	JS	655	$448,554	2
P-12	JS	701	$480,307	2
P-13	HS	1,552	$606,278	2
Total		**18,998**	**$14,723,519**	

Notes:

(1) Projects from Table 6 are combined by location. For instance, projects P-4A and P-4B are combined under P-4 in this table.

(2) 20-City Average Index ENR CCI of 9,962 was used for February 2015. A R.S. Means Location Factor of 106.6 for Oxnard was used.

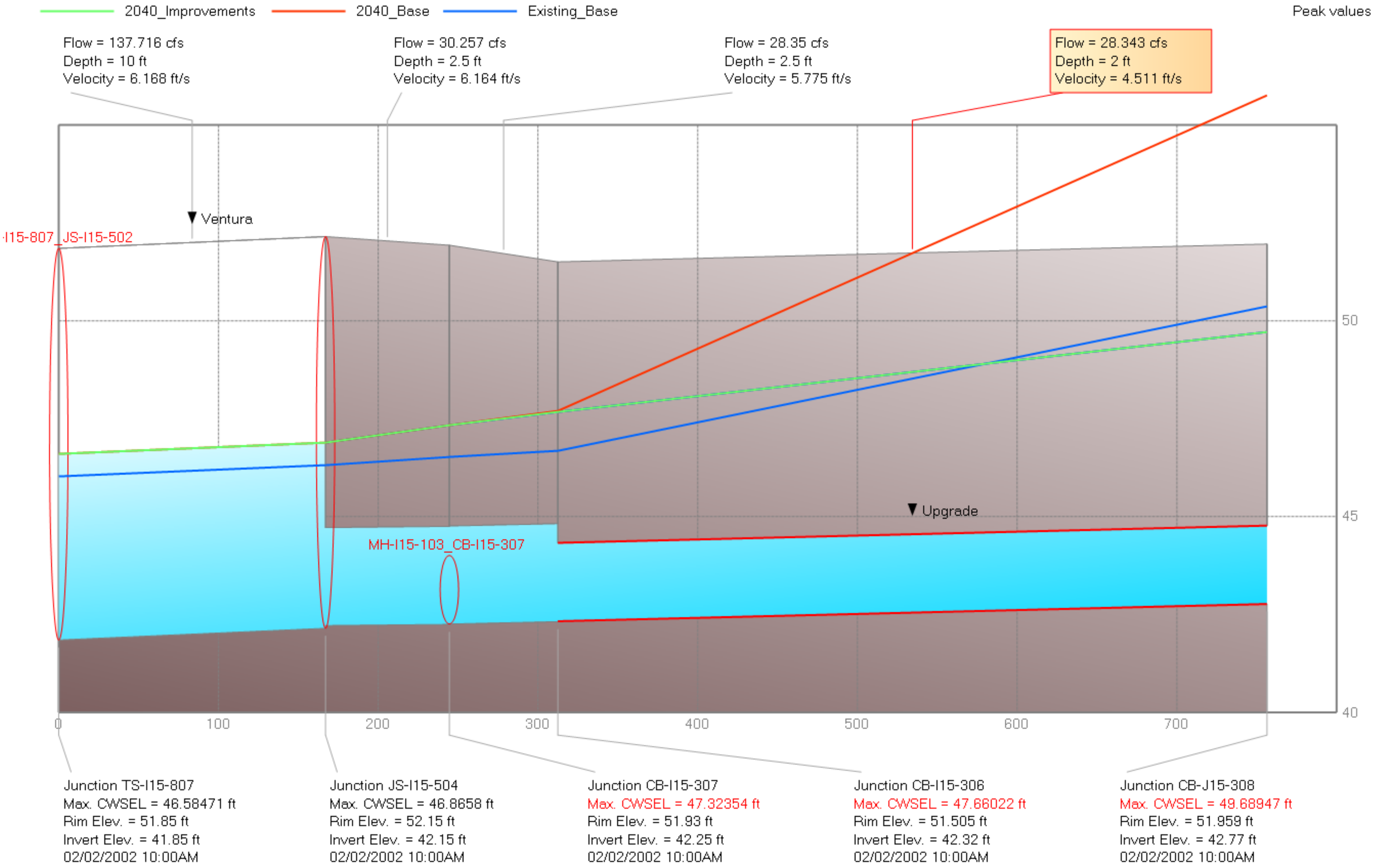
(3) Project costs, schedules, and phasing are based on data and information available at the time of the original date of preparation – December 2015. The updated CIP is contained in the Brief History section of the PMs, the Summary Report, and the Executive Summary.

Table 9	Overall Projects Costs by Phase for Stormwater System Drainage Basins[(1)] Public Works Integrated Master Plan City of Oxnard		
Phase Ranking	**Total Length (ft) or # Assets**	**Capital Improvement Cost ($)**	**% of Total Capital Improvement Cost**
1	7,292 ft	$6,736,694	37.3%
2	6,134 ft	$3,435,748	19.0%
3	2,436 ft	$2,620,545	14.5%
4	3,136 ft	$1,930,531	10.7%
Varies	22 assets	$3,324,000	18.4%
Total	**18,998**	**$18,047,518**	**100.0%**

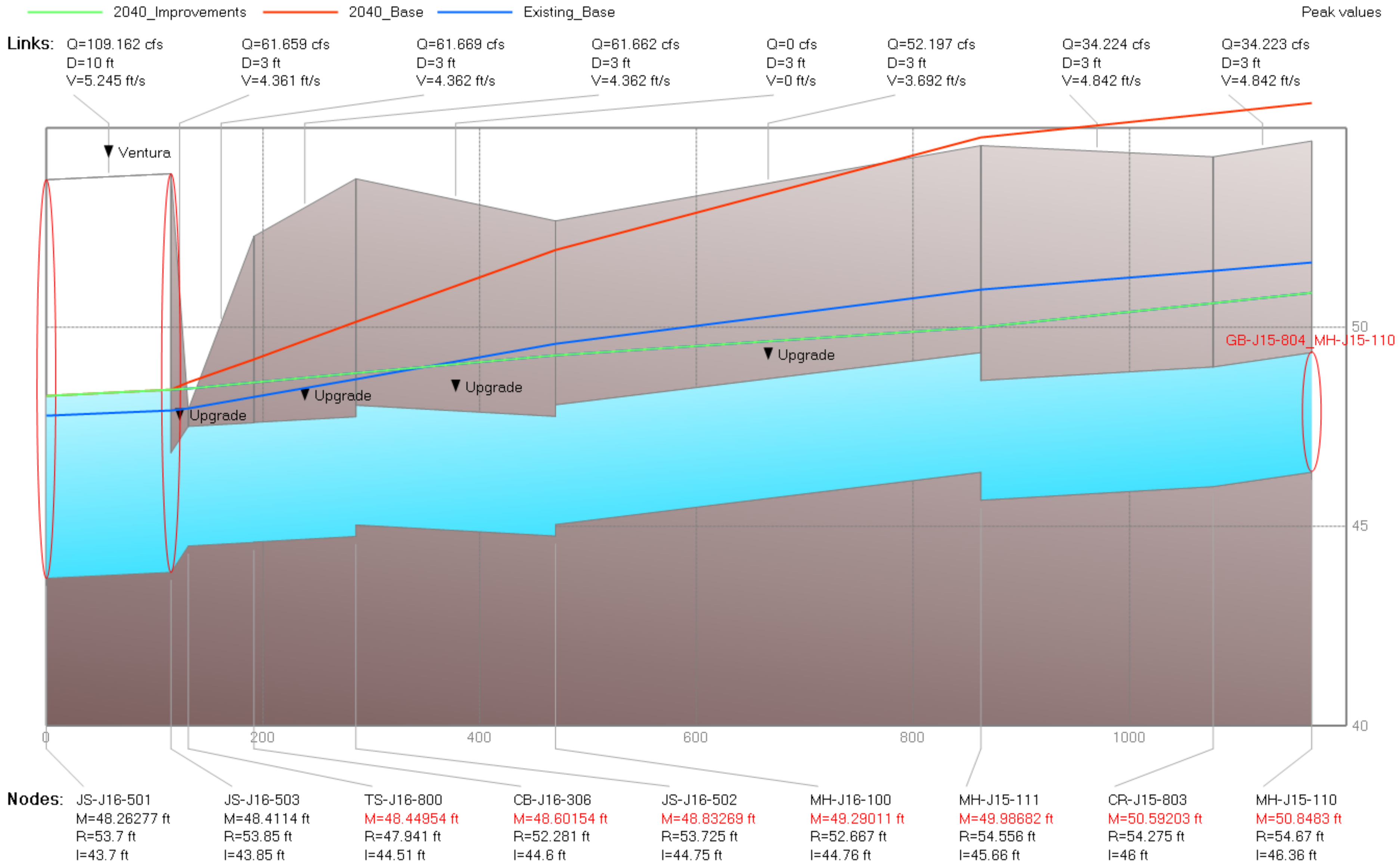
Notes:
(1) Project costs, schedules, and phasing are based on data and information available at the time of the original date of preparation – December 2015. The updated CIP is contained in the Brief History section of the PMs, the Summary Report, and the Executive Summary.

APPENDIX A – RECOMMENDED IMPROVEMENT PROJECT HYDRAULIC PROFILES

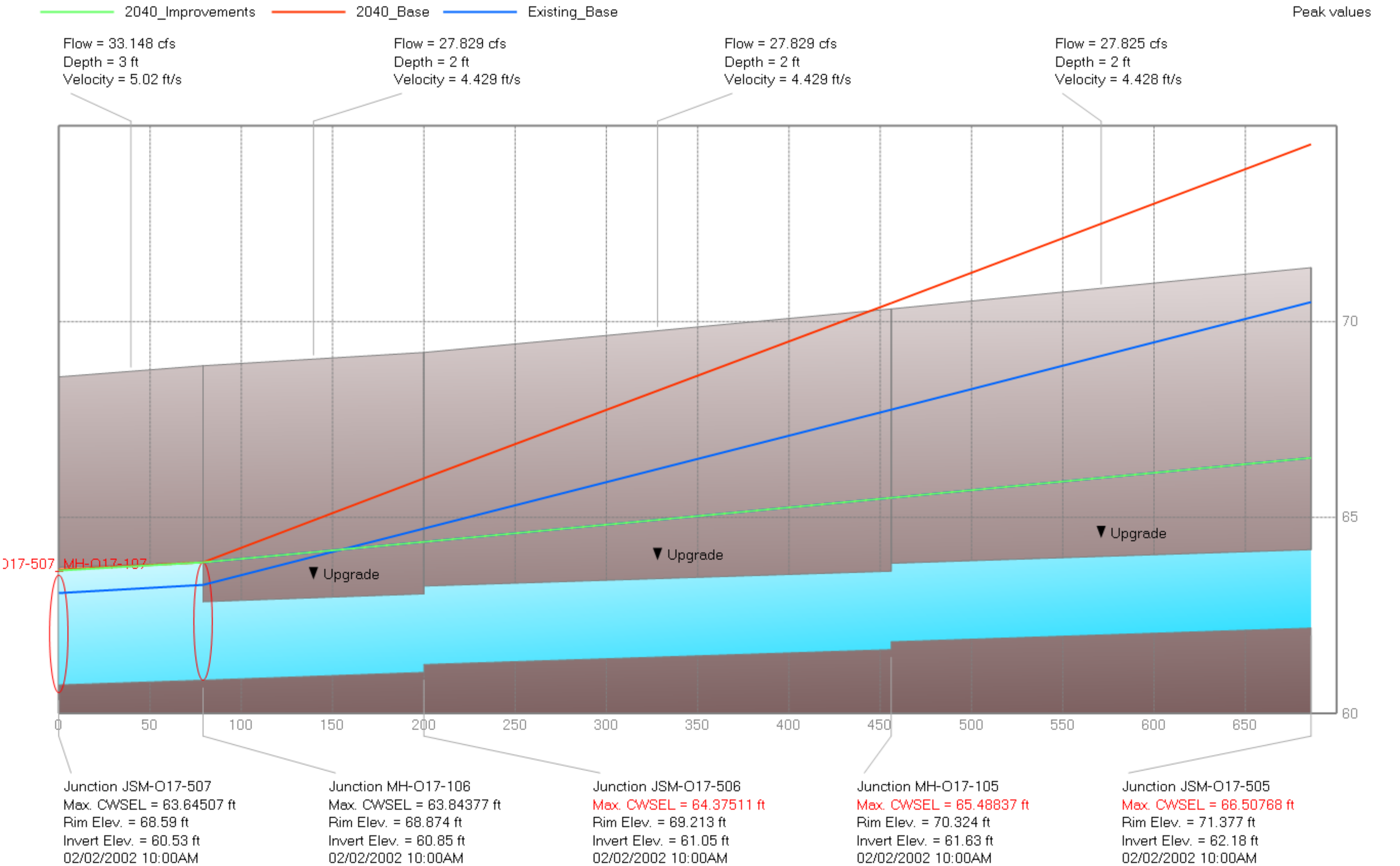
Project P-1 – Priority 2 (Year 2020)



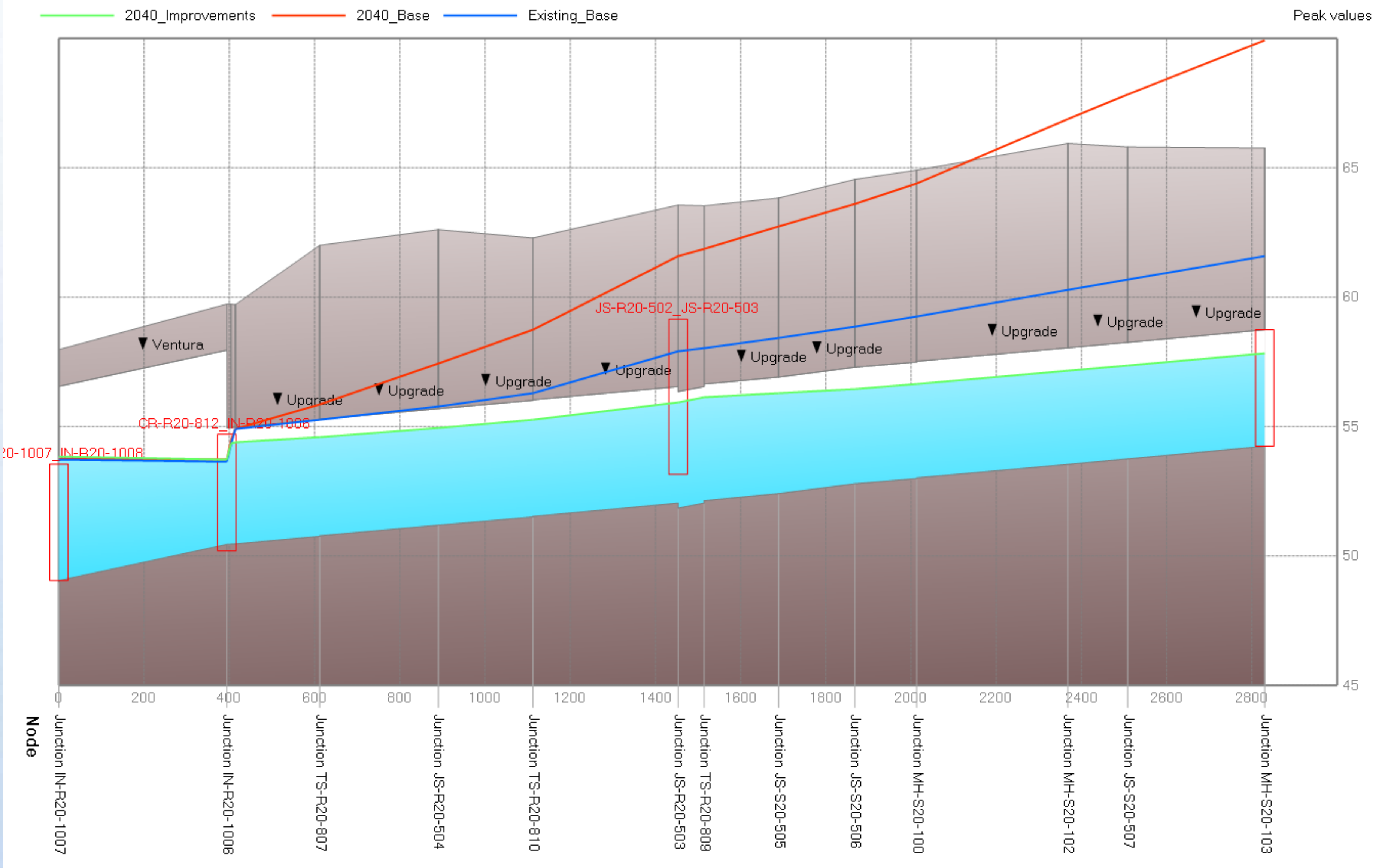
Project P-2 – Priority 4 (Year 2040)



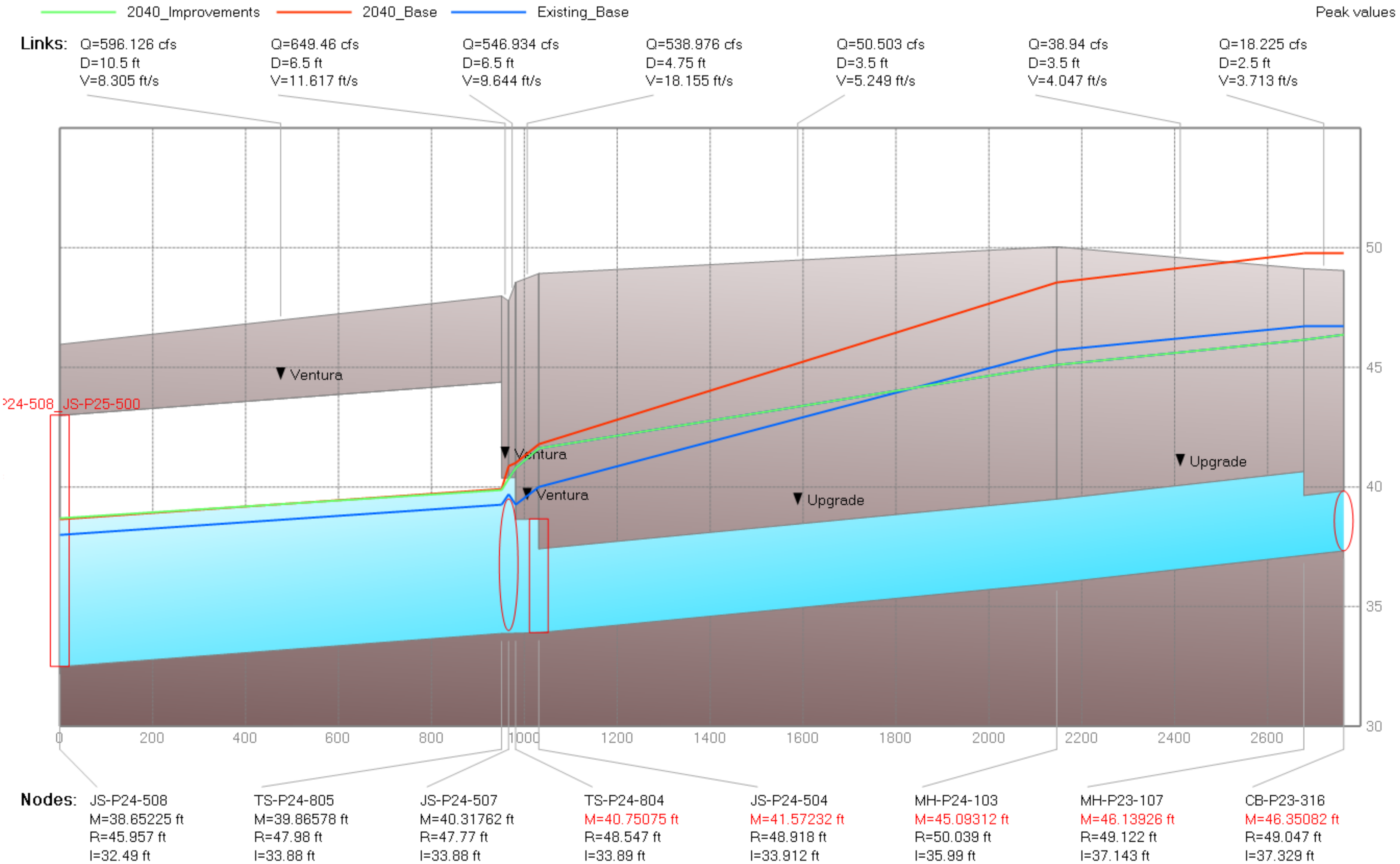
Project P-3 – Priority 2 (Year 2020)



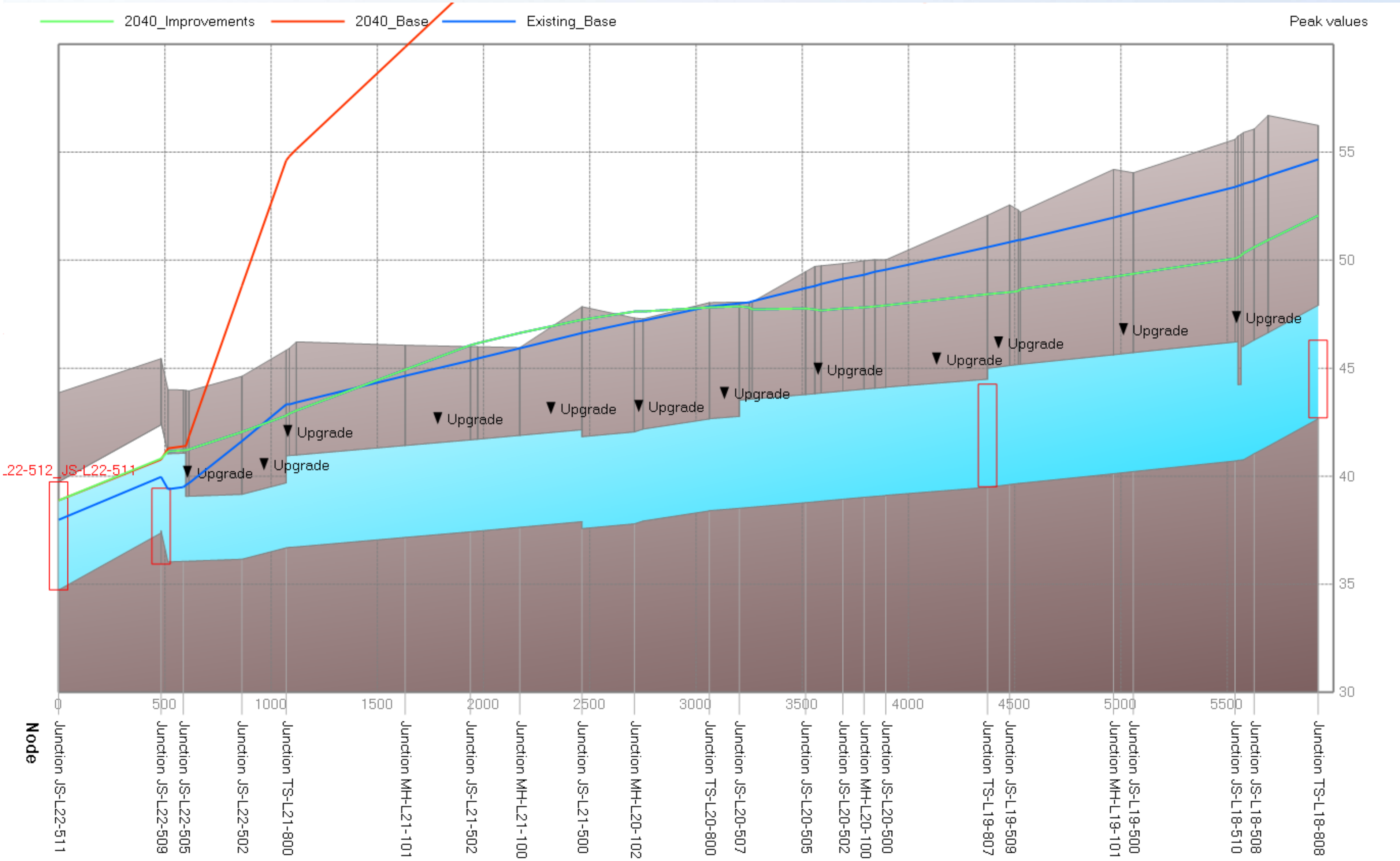
Project P-4 – Priority 3 (Year 2030)



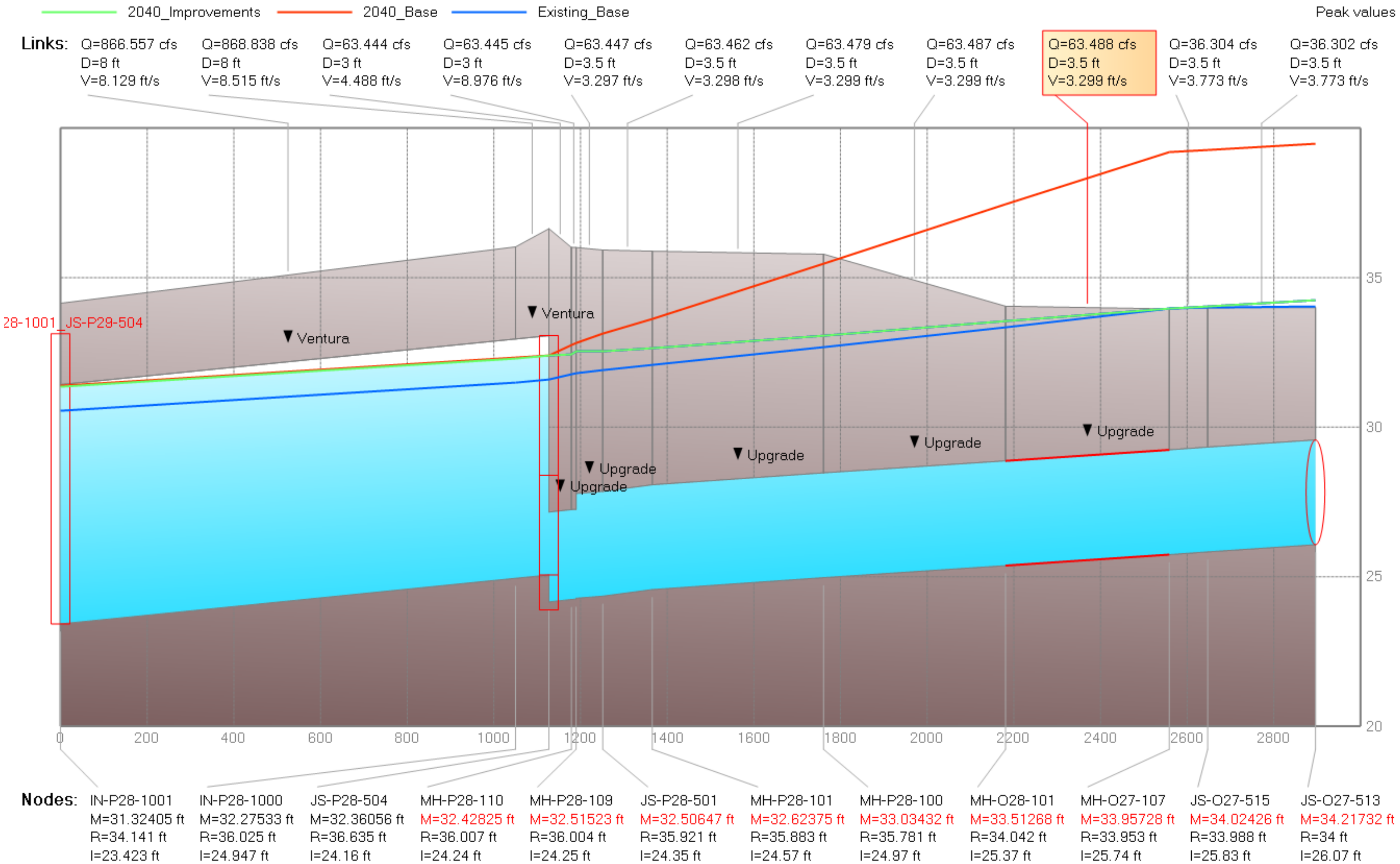
Project P-5 – Priority 4 (Year 2040)



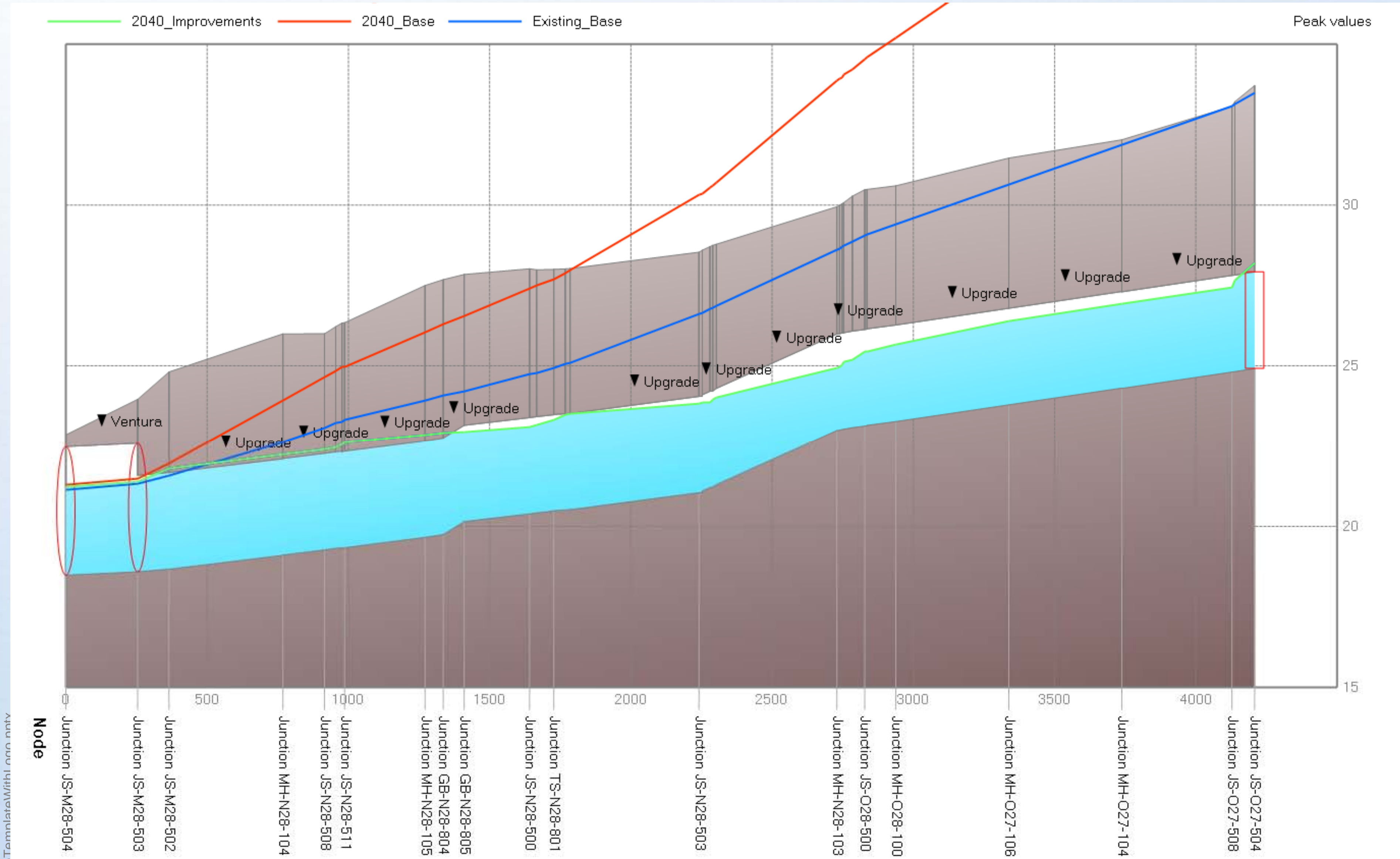
Project P-6 – Priority 1 (Current Conditions)



Project P-7 – Priority 1 (Current Conditions)



Projects P-8, 9, 10, 11, 12 – Priority 2 (Year 2020)



Project P-13 – Priority 2 (Year 2020)