Stormwater Master Plan Update

City of Manzanita

CITY OF MANZANITA

Submitted to:
City of Manzanita

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Prepared By:
Otak, Inc.
808 SW Third Ave, Suite 800
Portland, OR 97204

Project No. 19423

Otak

"I hereby certify that this Stormwater Master Plan Update for the City of Manzanita has been prepared by me or under my supervision and meets minimum standards of the City of Manzanita and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me."

Kyle Ayers, PE

Engineer, Transportation and Infrastructure

Email: Kyle.Ayers@otak.com

Phone: (503) 287-6825

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TABLE OF CONTENTS

Page

CHAPTER 1: EXECUTIVE SUMMARY 1
Introduction 1
Scope of Report 1
Water Quality Considerations 1
Wetland Considerations 1
Goals and Objectives 2

CHAPTER 2: WATERSHED CHARACTERISTICS 3
Watershed Characteristics 3
Study Area Delineation 3
Soils and Topography 3
Existing Drainage Features 3
Precipitation 4
Zoning and Development 4

CHAPTER 3: METHODOLOGY 9
Stormwater Model Parameters 9
Hydrologic Analysis 10
Hydraulic Analysis 11
Analysis Approach 11

CHAPTER 4: BASIN DELINEATION AND MODEL PARAMETERS 14
Basin Delineation and Model Parameters 14
Basin Delineation 14
Sub-Basin Parameters 15
Sub-basin Runoff Area 15
Soil Loss Parameter 15

CHAPTER 5: RAINFALL ANALYSIS 28
Rainfall Analysis 28
Design Storm Duration 28
Precipitation Depth-Frequency Relationship 28
Design Storm Hydrograph 31
Rainfall Intensity-Duration-Frequency Curves 31

CHAPTER 6: RUNOFF ANALYSIS 33
Runoff Analysis 33
Sub-Basin Flows 33
Peak Node Flows at Key Points 33

CHAPTER 7: IDENTIFICATION OF PROBLEM AREAS AND PROPOSED SOLUTIONS ... 36
Identification of Problem Areas and Proposed Solutions 36
General Discussion 36
Division Street Drainage – CIP Project #1 38
Problem Areas 38
Potential Solution 38
Sitka Street Drainage – CIP Project #2 40
Problem Areas 40

Potential Solution ... 40
Lakeview Drive Drainage – CIP Project #3 ... 43
Problem Areas ... 43
Potential Solution ... 43
Pine Street Drainage – CIP Project #4 ... 45
Problem Areas ... 45
Potential Solution ... 45
Hallie Lane Drainage – CIP Project #5 ... 47
Problem Areas ... 47
Potential Solution ... 47
Division Street (North) Drainage – CIP Project #6 ... 49
Problem Areas ... 49
Potential Solution ... 49
North Avenue & Epoh Avenue Drainage – CIP Project #7 ... 51
Problem Areas ... 51
Potential Solution ... 51
Greenridge Street Drainage – CIP Project #8 ... 53
Problem Areas ... 53
Potential Solution ... 53
Cherry Street Drainage – CIP Project #9 ... 56
Problem Areas ... 56
Potential Solution ... 56
Manzanita Avenue Drainage – CIP Project #10 ... 60
Problem Areas ... 60
Potential Solution ... 60

CHAPTER 8: CAPITAL IMPROVEMENTS – COST, PHASING AND IMPLEMENTATION ... 62
Capital Improvements – Cost, Phasing and Implementation ... 62
Design Assumptions and Unit Costs ... 62
Pipe Systems ... 62
Rock Filled Collection Trenches ... 62
Proposed Projects and Costs ... 63
Project 1:Dorcas Lane Drainage ... 66
Project 2: Sitka Lane Drainage ... 66
Project 3: Lakeview Drive Drainage ... 67
Project 4: Pine Ave and Cedar Street Drainage ... 67
Project 5: Hallie Lane Drainage ... 68
Project 6: Division Street (North) Drainage ... 68
Project 7: North Ave & Epoh Ave Drainage ... 69
Project 8: Greenridge Street Drainage ... 69
Project 9: Cherry Street Drainage ... 70
Project 10: Manzanita Ave Drainage ... 70
Project Summary and Phasing ... 71

CHAPTER 9: STORMWATER QUALITY, SEDIMENTATION AND EROSION ... 72
Runoff Control ... 72
Best Management Practices for Maintenance ... 72
Landscape Plantings ... 73
Roadside and Perimeter Vegetated Swales ... 73
Infiltration/Detention Basin ... 74
Catch Basin Cleaning ... 74
Vegetated Swale Maintenance ... 75

Erosion Control 75
Sediment Control 75
Principles of Erosion and Sediment Control 76
Vegetation as a Solution 77
Ordinance for Construction Erosion Control 78
Erosion and Sediment Control Plans 78
Guidelines for Erosion Control Plans 79
Plan Check 79

Appendices

Appendix A—Wetland Map

CHAPTER 1: EXECUTIVE SUMMARY

Introduction

The City of Manzanita last updated its Storm Drainage Master Plan in 2005. This report is an update of the 2005 report and has been renamed as the 2020 Stormwater Master Plan (SWMP) per industry standard for stormwater master plans.

Currently, the City of Manzanita has only a limited number of drainage facilities and minimal number of integrated storm drainage systems. In recent years, Manzanita has seen a great deal of development. Infill and greenfield development each have increased the volume of surface water runoff and channeled that runoff into previously developed areas where ponding and flooding cannot be tolerated. The resulting increase in drainage related problems, compounded by continued rapid development, warrants a comprehensive plan to integrate storm drainage improvements with recommendations for new storm drainage facilities required to serve future development.

In conclusion, this updated SWMP is meant to provide the City with a long-term planning tool that can be implemented with available funding, addresses community needs to solve existing drainage problems, and mitigates problems that continuing development might cause.

Scope of Report

This study addresses surface water quantity and storm drainage issues for a 20-year planning period, assumed to span the period from 2020 to 2040. Identifying specific water quality and wetland issues is beyond the scope of this study, but these issues may need to be considered in the final design and permitting stage. As discussed in Chapter 2, the Manzanita Urban Growth Boundary (UGB) was selected as the study area boundary. The study was intended to outline improvements required to solve problems due to existing and anticipated future runoff conditions. Proposed solutions were intended to describe the required design parameters: conveyance alignment, approximate length, pipe diameter or ditch dimensions and approximate slope required.

Water Quality Considerations

Currently, no stormwater quality regulations are in effect for Tillamook County or the City of Manzanita. Stormwater discharge permit requirements adopted by the Oregon Department of Environmental Quality currently do not apply to small municipalities such as the City of Manzanita. However, non-point source pollution has become of increasing concern to regulatory agencies. As such, changing regulations and community concern may affect drainage projects recommended for later phasing periods. An investigation of the impacts of development in the Manzanita area to surface water quality is beyond the scope of this study. However, Chapter 9 Stormwater Quality, Sedimentation and Erosion has been included and should be used as a guide for implementing the current water quality standards by the City of Manzanita.

Wetland Considerations

City and County zoning maps do not specifically identify any regulated wetland areas within the study area. However, the Federal Wetland Inventory identifies several low-lying areas within the Urban Growth Boundary (See Appendix A – Federal Wetland Inventory Map) which may fall under federal regulation. Approximate locations of these features have been shown on the included basin maps. Federal regulations currently prohibit fill or drainage of a regulated wetland under most conditions. Areas shown on the Federal Wetland Inventory Map are not necessarily regulated wetlands. However, a wetland delineation or field inspection by regulatory agencies is usually required to prove otherwise.

Determining which areas fall under federal regulation was beyond the scope of this study. Proposed storm drainage improvements assume full development as indicated by City and County zoning maps within the UGB. These capital improvements are designed to provide adequate surface water drainage in all areas zoned for development. Wetland issues, however, should not be ignored and will need to be resolved during the final design and permitting stage for each of the proposed storm drainage improvements.

Goals and Objectives

The City of Manzanita's SWMP Update was compiled to provide the City staff with a useful planning tool by outlining the improvements and associated costs expected to meet the City's storm drainage needs for the 20-year planning period. The specific goals and objectives of this report are as follows:

A. PROVIDE COST EFFECTIVE SOLUTIONS TO IMMEDIATE DRAINAGE PROBLEMS

1. By identifying existing drainage facilities which function poorly, cause frequent ponding and/or inhibit existing activities and future development.
2. By identifying existing areas where recent development has resulted in a need for new or additional storm drainage collection systems.
3. By providing a detailed description and cost estimate for proposed storm drainage improvements required to serve existing problem areas.

B. PROVIDE COST EFFECTIVE SUGGESTIONS FOR FACILITIES REQUIRED TO MEET FUTURE DRAINAGE REQUIREMENTS

1. By identifying future areas of development and using zoning and comprehensive plan assumptions to determine the impact of expected development on existing drainage facilities.
2. By evaluating zoning and land use considerations and determining where new storm drainage collection and conveyance facilities will be required.

C. IDENTIFY DRAINAGE IMPROVEMENTS THAT ARE FEASIBLE TO IMPLEMENT AT MINIMIZED COSTS

1. By analyzing possible alternative solutions to existing problems, other than simply replacing faulty pipes. Alternatives include diversion to systems with adequate capacity, detention, accepting a lower design storm frequency and incorporating existing improvements with improvements necessary for future development.
2. By prioritizing improvements, allowing those with the highest cost-benefit ratio to be implemented immediately, while other improvements can be postponed to a later phasing period.

CHAPTER 2: WATERSHED CHARACTERISTICS

Watershed Characteristics

This chapter describes the drainage characteristics of the City of Manzanita's watershed, including seasonal rainfall characteristics, topography, existing and future land use conditions and main drainage features. In later chapters, this information will be used to quantify the City's future and existing drainage needs. A vicinity map is provided in Figure 2.1.

Study Area Delineation

The City of Manzanita's Urban Growth Boundary (UGB) was selected as the study area boundary. The City limits and UGB are shown on the aerial map in Figure 2.2. With the exception of the creek flowing out of Neahkahnie Lake, drainage paths within the UGB are affected only by lands contained within this boundary.

The entire UGB was studied in order to provide flow information and mapping tools that can be applied to long term drainage planning. However, the main emphasis of the study was on lands expected to develop in the near future. These lands are contained primarily within the City limits, but also include the Necarney City subdivision area of South Manzanita.

Soils and Topography

The Manzanita watershed contains three regions with distinct topography and soils characteristics. Watershed topography is shown in Figure 2.3, Watershed Topography. Soils information is presented in Chapter 4, Figures 4.3A and 4.3B.

Lands in the eastern portion of the UGB are hilly and steep, with several ridges and deep swales. Overland slopes range from 3 to 30%. These lands drain in natural swales primarily toward Nehalem Bay. Most soils in this area are Netarts fine sandy loam. Southern sections also include Waldport fine sands, Yaquina loamy fine sands and dune lands. Soils in this area have widely varying infiltration characteristics. Yaquina soils have very low infiltration rates and are saturated frequently. Waldport sands and active dune areas have very high infiltration rates. Netarts fine sandy loam have high infiltration rates.

The northern portion of the City comprises the foothills of Neahkahnie Mountain. These lands slope to the southwest and have overland slopes ranging from 7 to 15%. Runoff from this area flows toward the flatter region near Manzanita Avenue and Laneda Avenue. Soils in this area are Netarts find sands, with moderately high infiltration rates.

The portion of the City south of Laneda Avenue and west of 3rd Street is relatively flat and slopes evenly towards the ocean. These lands have overland slopes ranging from 0.5 to 3%. Soils generally have moderately high to high infiltration rates and include Waldport fine sands and Heceta fine sands. Heceta fine sands are typified by minimal ground slopes, high water table conditions, and very poor drainage. Several patches are found between South 3rd Street and S. Carmel Ave and in Necarney City east of Necarney City Road.

Existing Drainage Features

Figures 4.2 A & 4.2 B show the existing drainage features in the Manzanita watershed. The primary natural drainages in the watershed are the creek flowing south and east out of Neahkahnie Lake in the northeastern portion of the UGB and the Golf Course Creek, flowing west to the Pacific Ocean within the City. Several deep natural swales drain the eastern portion of the UGB toward Nehalem Bay.

The City is served by six beach discharges located at Manzanita Avenue, Washington Avenue, Laneda Avenue, Treasure Cove, Pacific Avenue, at the north boundary of the City and at Sitka Lane in Necarney City. Runoff is conveyed to these discharges primarily via roadways and overland flow, with the exception of Laneda Avenue, see Figure 4.1.

Four collector systems are currently in service. A system running south along Division Street dead-ends at a private driveway near the golf course. Historically, this system drained to the Golf Course Creek. A system running south along South 3rd Street intercepts flow from Laneda Avenue and routes it to a 36-inch diameter trunk line along Pacific Avenue. This trunk line also routes flow from the Golf Course Creek to the Pacific Ocean. Carmel Avenue and Beach Street have been reconstructed since the previous report and now contain trunk lines. These pipes discharge to the Pacific Ocean at the Treasure Cove Lane system and the Pacific Lane system. The final collector system is the culverted ditch system in Necarney City that consists of ditches along Necarney Boulevard, Windward Lane and Sitka Lane. This system discharges into the Pacific Ocean at Sitka Lane through a 24-inch diameter storm drain culvert.

System capacity needs for existing and future conditions, as well as associated problem areas with each, are discussed in detail in Chapter 7 - Identification of Problem Areas and Proposed Solutions.

Precipitation

On the average, the City of Manzanita receives 90 inches of rainfall per year. Precipitation characteristics in the Manzanita area are nearly equivalent to conditions in Seaside and Tillamook. These coastal cities receive moderate precipitation in comparison to other areas in Western Oregon. For example, the average annual precipitation in the Willamette Valley is 40 inches. The annual precipitation on Neahkahnie Mountain and in the Coast Range averages 120 to 140 inches.

A majority of the drainage problems in the City may be attributed to intense rainfall events. For instance, available data indicates that in any given year there is a 50% chance that more than 3.5 inches of precipitation will occur in a single 24-hour period. In addition, Manzanita's moderately high annual rainfall often leads to wet antecedent conditions, which means that the ground is already saturated when a large storm occurs. A saturated soil will generate more runoff than a dry soil. Compounded with steep slopes, these conditions can result in large increases in runoff volume even in sandy soils with high infiltration rates.

Zoning and Development

The 2015 aerial photography shown in Figure 2.2 shows the current level of development and existing land uses. In Chapter 4, Figure 4.4, the existing zoning map is presented, indicating the character of expected future development.

The central portion of the City is fully developed. Commercial lands are located adjacent to Laneda Avenue and account for only a small portion of development. Single and multi-family residences occupy a majority of the developed area within the watershed. All platted streets have been constructed. A few buildable residential lots still remain, but a majority of the lots have been developed.

The northern portion of the City hosts significant residential development on steep slopes at the Classic Ridge Beach area. Developable land remains and is expected to be built out within the planning period.

Some commercial development has emerged and more is expected adjacent to existing commercial lands near Highway 101 in the northeast portion of the City. The commercial development on Manzanita Avenue from Division Street to Highway 101 is largely impervious and generates high runoff.

While a majority of the lands zoned for Special Residential Development have been developed as residential/golf course there is room for expansion in the eastern portion of the City. This plan assumes that the remainder of the lands zoned for Special Residential Development will develop with similar characteristics to the existing residential / golf course development.

The lands within the UGB east of the City limits are zoned for residential and mobile home development. Residential development has begun in the eastern portion of the UGB as well as within the Pine Ridge subdivision. These lands are developing very quickly and are expected to be at full build-out within the planning period. Further development is expected, but may, however, occur in the later portion of the planning period.

Lands in Necarney City are zoned for residential development. A majority of the development in Necarney City has occurred in beach front areas. Lands east of Necarney Boulevard are platted and are expected to develop as residential lots within the planning period. Necarney City is expected to be annexed within the planning period.

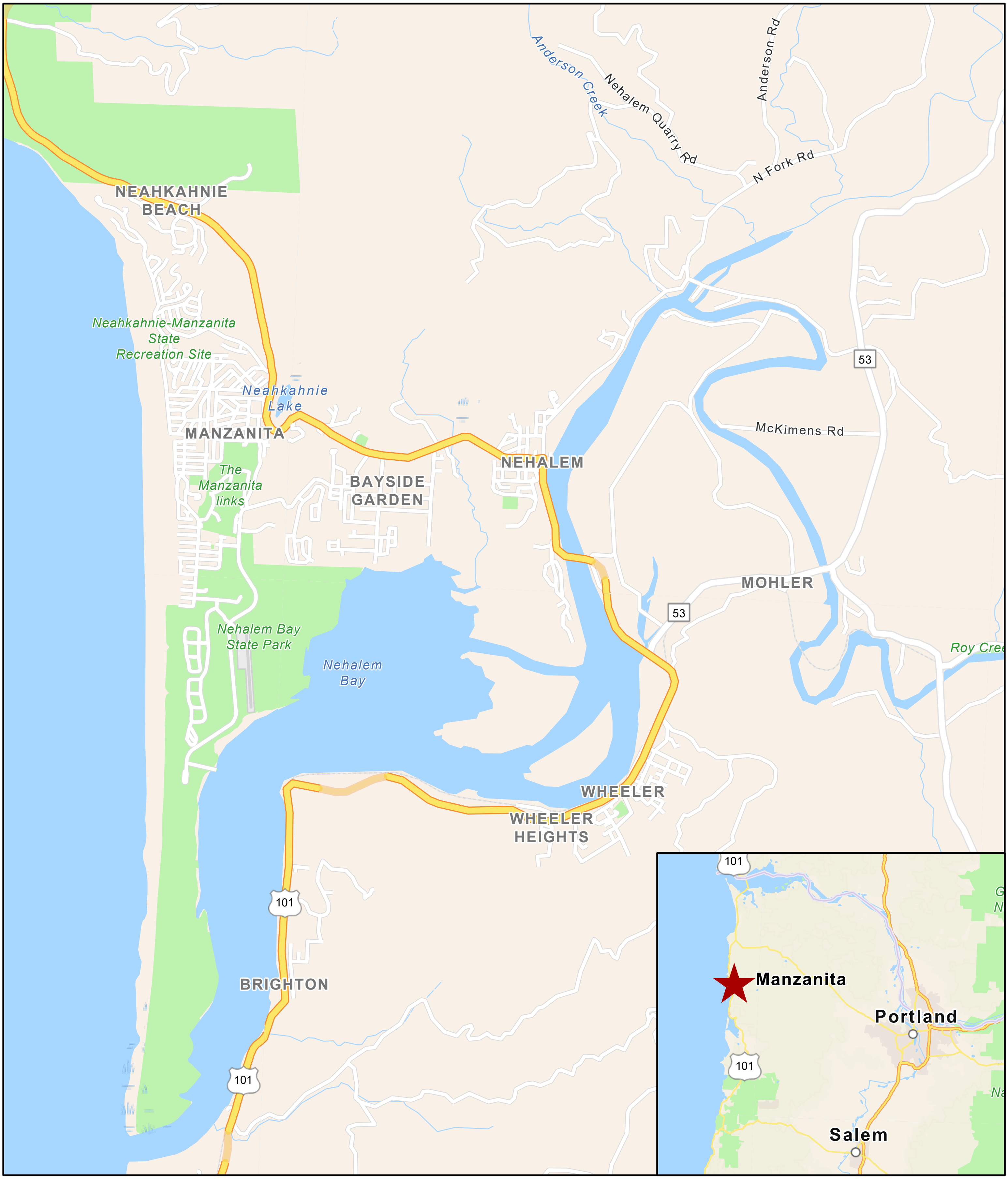


FIGURE 2.1

VICINITY MAP

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

MILES

0 0.5 1

Otak

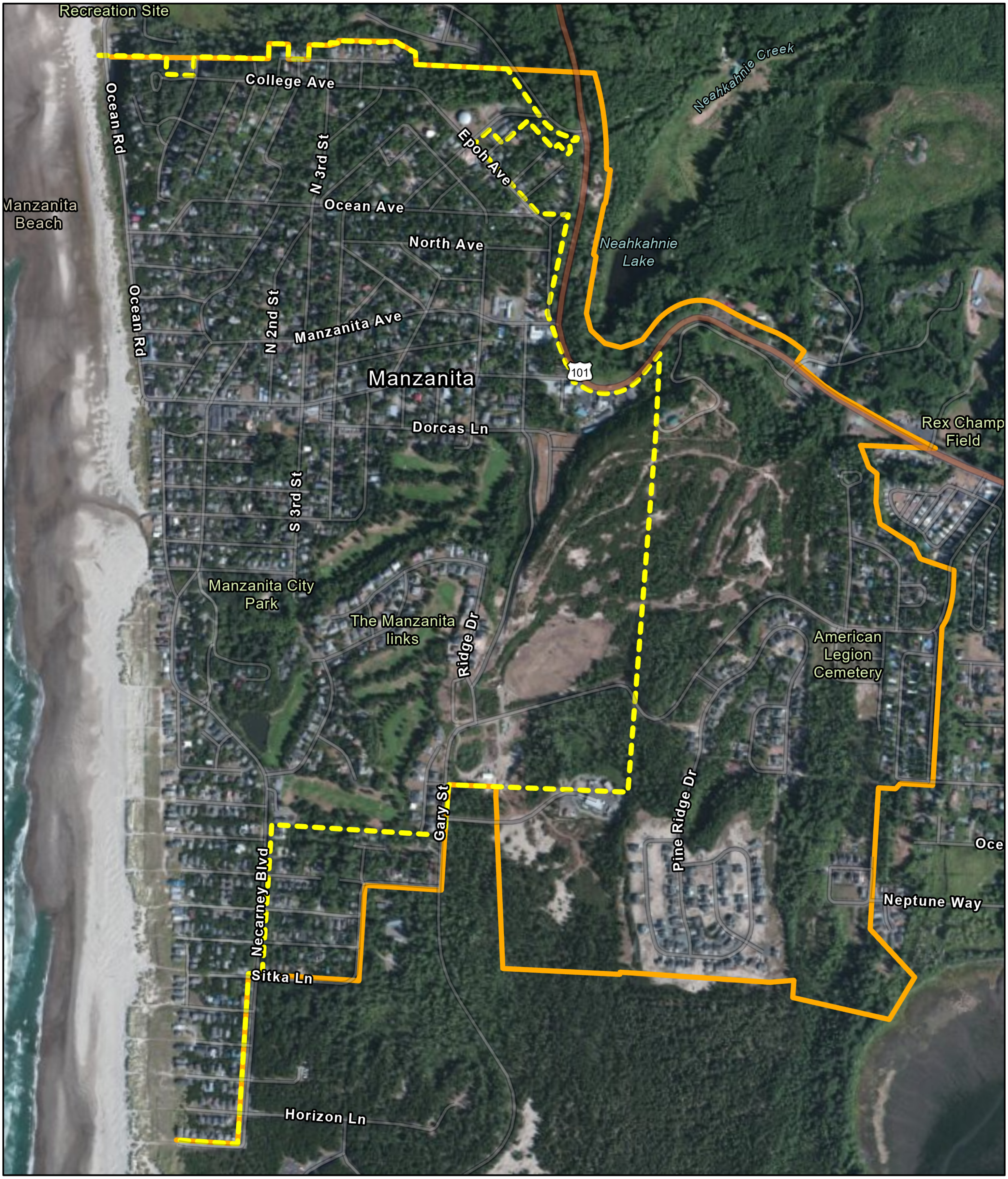


FIGURE 2.2

CITY LIMITS AND UGB

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

Legend

Manzanita City Limits

Manzanita UGB

FEET

0 500 1,000

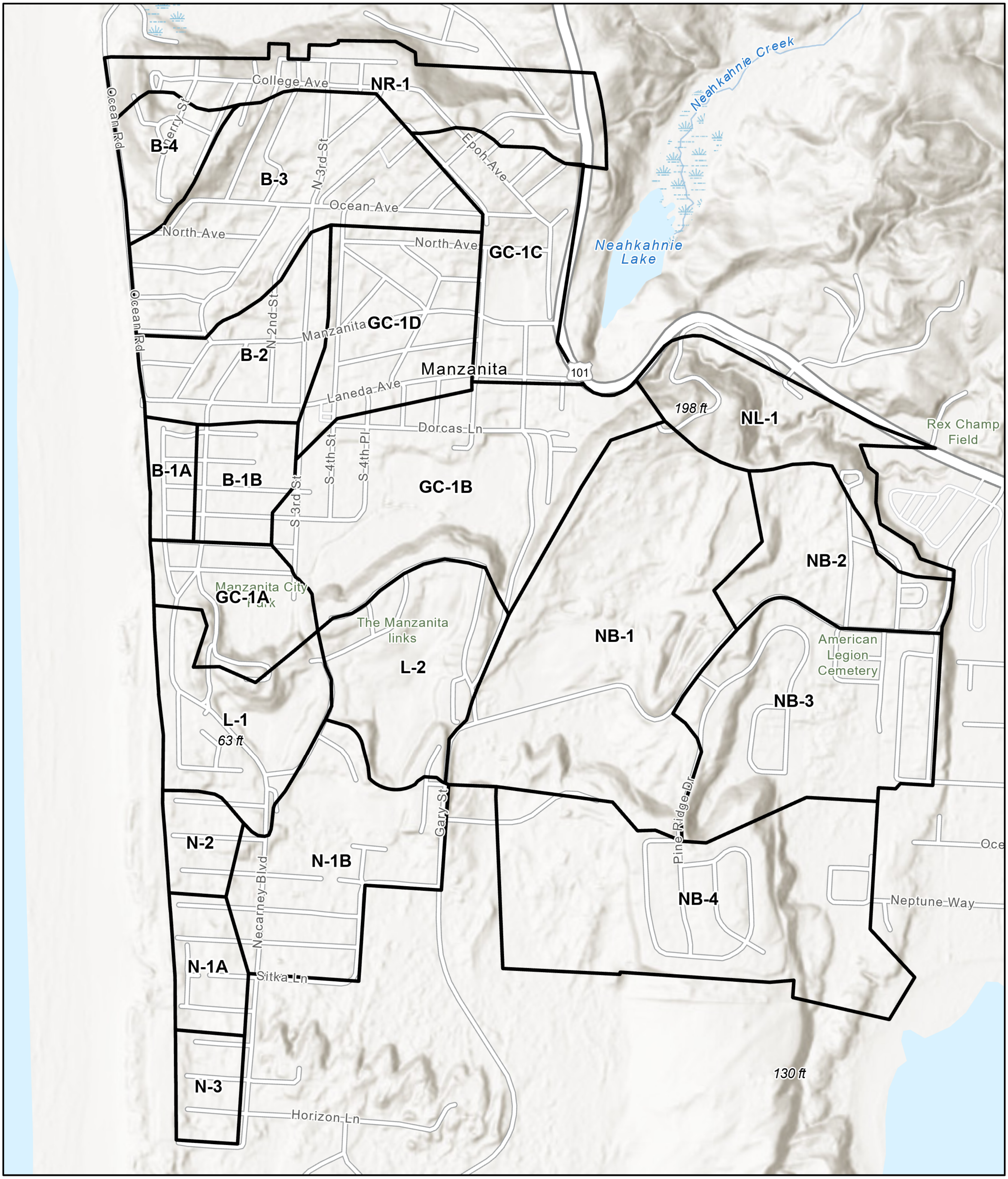


FIGURE 2.3

WATERSHED TOPOGRAPHY

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

CHAPTER 3: METHODOLOGY

This chapter describes the analytical techniques and outlines the methodology used in this study to determine the existing and future drainage system requirements. First, several basic assumptions are presented. Next, analytical techniques used for hydrologic and hydraulic analyses are described. Finally, this chapter briefly describes how drainage facilities will be evaluated.

Stormwater Model Parameters

The following basic parameters for the hydrologic and hydraulic analyses were made:

1. Future Mapped Impervious Areas (MIA) for commercial zones C1 and LC were assumed to be 90%. Future MIA for zones R2, R3, R4, RMD and SRR were assumed to be the maximum lot coverage allowed by the City's Zoning Ordinance, or 40%, 55%, 60%, 40% and 40%, respectively, see Figure 4.4.
2. Graveled parking areas in commercial zones were treated as 90% MIA for existing conditions and were assumed to be paved in the future.
3. Driveways and parking areas in residential zones were treated as gravel areas, in the existing and future conditions.
4. Pipe capacities were based on the design flows given and were adjusted for upstream ponding in problem areas where upstream storage was identifiable.
5. Development outside of the City limits, but inside the UGB, was assumed to meet City Zoning requirements.
6. Runoff rates for road rights-of-way (ROW), which have high MIA, were calculated separately from runoff rates for properties.
7. Future flows were calculated assuming full build-out within the UBG except in developments containing the Manzanita Golf Course, the Manzanita City Park, the Nehalem Bay State Park or Neahkahnie Lake as these areas are expected to maintain their open space.
8. Existing residential development constructed prior to 1990 was assumed to manage some of its runoff on-site using drywells, based on observations. Therefore, contributing runoff area was reduced by 30% within pre 1990 residential developments.
9. Existing commercial development constructed prior to 1990 was assumed to manage roof runoff on-site using drywells, based on observations. Therefore, contributing runoff area was reduced by 20% within pre 1990 commercial developments.
10. Existing residential development constructed since 1995 and future residential developments were assumed to manage runoff from 90% of the site area on-site using drywells, in accordance with the City’s on-site storm drainage policy. The associated driveways and parking areas adjacent to streets, or 10% of the area, were assumed to drain to the City’s storm drainage system. Therefore, contributing runoff area was reduced by 90% for these areas.
11. Existing commercial development constructed since 1990 and future commercial developments were assumed to manage runoff from 80% of the site area using drywells, in accordance with the City’s on-site storm drainage policy. All new commercial developments utilize on-site infiltration or detention systems to handle flows generated by a 25-year storm. These systems are often equipped with an overflow connected to the City’s storm drainage system in case of a failure in their system or for larger storm events. Therefore, contributing runoff area was reduced by 80% for these areas.

Hydrologic Analysis

In planning for required improvements to the storm drainage conveyance systems, the engineer or planner must know the frequency, or probability, of system failures. For this purpose, hydrologic events such as precipitation and peak flows are presented in terms of their frequency of recurrence. The recurrence interval of a given hydrologic event is equivalent to the probability of recurrence within a given year. For instance, the 2-year peak flow for a drainage basin is the flow rate expected to be equaled or exceeded once every 2 years and has a 50% chance of being equaled or exceeded in any given year. The 25-year flow is expected to be equaled or exceeded once every 25 years and has a 4% chance of occurring in any given year. For the purposes of this study, a hydrologic model was developed to determine peak rates and volumes of runoff at key design points for the 2, 5, 10, 25 and 100-year recurrence intervals.

Peak rates of runoff used for drainage design depend on the volume of runoff and the rate at which it occurs. The volume component of runoff depends upon the size of the drainage basin, the total volume of precipitation that occurs and the volume of runoff that is "lost" due to infiltration and evapotranspiration. The rate of runoff depends upon the hydraulic conditions of the basin (travel length, slope and roughness) and the varying intensity of rainfall throughout the design storm.

Several methods are available for calculating peak flows. The method most appropriate for a given study depends upon basin size, basin characteristics, available input data, ease of implementation and the ability to vary design parameters for changing conditions. A general discussion of three typical stormwater runoff analysis methods is as follows:

1. The **Rational Method** is a simple formula often used for design of local storm drain systems. This method is regarded by numerous regulatory agencies as unacceptable for larger basins because it only indirectly accounts for such conditions as varying rainfall intensity distributions during large storms, soil conditions, impervious area and the hydraulic routing of rainfall from one end of a basin to the basin outlet.

2. **Hydraulic methods** which compute flows based on theoretical hydraulic equations are very data intensive and often do not adequately account for imperfect drainage conditions found in basins with a high degree of natural and overland drainage.

3. **Hydrograph methods** use empirically determined "unit" hydrographs to approximate basin specific flow vs. time relationships based on direct input of anticipated rainfall intensity distributions, soil and ground cover parameters, hydraulic routing parameters and impervious area information.

The Rational Method was determined to be too simplistic for the basin sizes analyzed in this study. Hydraulic methods were found to be too complex and inflexible for efficient use in a general study of this type. Consequently, Hydrograph methods were selected to model the watershed hydrology.

A variety of unit hydrograph methods are available for calculating flows in ungauged watersheds. Many are useful only if detailed empirically based studies have been conducted on nearby gauged watersheds with similar characteristics to the ungauged basin of interest. Discussions with the Corps of Engineers and the Soil Conservation Service (SCS) revealed that no empirically based studies have been conducted on small watersheds in this area or in other areas demonstrating similar characteristics to the Manzanita area. Studies of large watersheds in the area used regression techniques which would encompass too large an area to be applicable to this study.

Since detailed studies of similar gauged watersheds are not available, the Santa Barbara Unit Hydrograph Method was chosen as the most applicable means of calculating flows in the Manzanita watershed. While other methods are very sensitive to local conditions and are often not applicable when limited data is available, the SCS method is well adapted to areas where limited hydrologic data is available since it uses a single parameter to represent the hydraulic conditions in the basin.

Applying unit hydrograph methods to a multi-basin study area can only be done efficiently with computer methods. A variety of computer programs are available which model complex multi-basin watersheds using the Santa Barbara Unit Hydrograph method. HydroCAD v10.0 was used to calculate representative existing and future flows for the City of Manzanita's watershed. The input parameters required by HydroCAD are described below:

Design Precipitation Hydrograph

The design hydrograph is the hypothetical distribution of precipitation vs. time that is assumed to produce the peak flow used for design purposes. Chapter 5 - Rainfall Analysis discusses the derivation of the design rainfall hydrographs.

Sub-Basin Area (acres)

A sub-basin is the surficial watershed within which runoff can be assumed to flow to a single discharge point. The area was adjusted to account for areas within the sub-basin draining to dry wells and not contributing to runoff.

Soil Infiltration and Evapotranspiration Loss Parameter

The SCS Curve Number method was chosen to estimate evapotranspiration and infiltration losses. This method uses a single "Curve Number" which has been empirically determined by the SCS for a variety of soil types and ground cover conditions to estimate soil infiltration and evapotranspiration losses.

Time of Concentration (min)

Time of concentration of a basin, or the time it takes water to travel from the most remote part of the basin to the basin outlet. The specific assumptions, values and means of computation for each of the above sub-basin parameters are discussed in Chapter 4, Basin Delineation and Model Parameters and Chapter 5, Rainfall Analysis, presents frequency information for expected rainfall and the development of the design rainfall hydrographs.

The result of the HydroCAD modeling process is the computation of sub-basin runoff hydrographs (runoff from each individual sub-basin vs. time). The peak flows for each subbasin for both existing and ultimate (full buildout) development conditions are presented in Chapter 6, Runoff Analysis.

Hydraulic Analysis

The hydraulic analysis involves calculating the hydraulic capacity and resulting water surface profiles under existing and ultimate land use conditions, based on the estimated peak flows generated from the hydrologic analysis. Several computer and analytical methods were utilized to model the different hydraulic elements.

The computer program HydroCAD, in-house spreadsheets which solve Manning's Equation (for gravity flow in conduits) and the Hazen-Williams Equation (for head loss per length of pipe under surcharged conditions) were used to model for piped drainage reaches.

Analysis Approach

The first step in analyzing the City of Manzanita's existing storm drainage facilities and determining future drainage needs was to sub-divide the City's watershed into more manageable drainage basins. The watershed was divided into "major drainage basins" based upon topographic drainage boundaries and the location of the discharge points. Then each major drainage basin was sub-divided into sub-basins for the purpose of determining flows using the HydroCAD model. Sub-basin outlets were chosen to correspond to key points where flow information was required and then delineated based on existing drainage facilities, zoning and topographic information. The specific drainage basin delineations arrived at are presented in Chapter 4, Basin Delineation and Model Parameters. Once delineated, sub-basins were used as the basis of the flow calculations. Rainfall data was evaluated and input into the model as described in Chapter 5, Rainfall Analysis. Hydrologic parameters were then determined (see Chapter 4, Basin Delineation and Model Parameters) for each sub-basin and flows for each design frequency calculated (see Chapter 6, Hydrology). The calculated flows apply to the point where the entire sub-basin has contributed flow. When flows were required to analyze a conveyance receiving flow from only a portion of the basin, a one-to-one relationship between contributing area and peak flow was assumed—i.e. the flow due to 10% of the basin is equivalent to 10% of the peak flow due to the whole basin. In hydrology, this assumption seldom holds true, however for the medium sized basins used in the calculations, the error is small and has a negligible effect on drainage planning.

Existing drainage facilities were analyzed with respect to their frequency of failure under existing conditions. Existing detention areas that are expected to remain were considered when determining the capacity of the downstream conveyance. Site visits, topographic mapping and discussions with City staff were used to estimate the impacts of flow exceeding the system's capacity. The impacts were then weighed with respect to the estimated frequency of failure to evaluate the need for improvements. Impacts considered were flooding of properties, frequent nuisance ponding in commercial areas, frequent ponding and poor drainage which could cause road surfaces to wear out prematurely, and flooding of main street intersections.

In addition to determining problems due to the capacity of existing drainage facilities, field surveys and topographic maps were used to determine areas where inadequate surface drainage facilities have been supplied. Erosion along roadways, poor drainage causing frequent ponding and low areas subject to flooding were considered problem areas requiring service.

After determining existing problem areas, future drainage needs were evaluated. Zoning information and 2020 aerial photography were compared and used to determine where future development was likely to occur. Based on previous flow calculations, drywell assumptions and judgments on how flow would channelize after development, future problem areas were isolated.

After the various locations of required improvements were identified, alternatives for alleviating each problem were developed. The following alternatives were considered in this approximate order:

1. Bypass — Re-route the flow around or away from the problem area in order to alleviate the problem and avoid replacement of system components.
2. Detention — Consider the utilization of existing detention areas or the construction of new detention facilities to store excess flow that might otherwise cause flooding.
3. Ditch System — Ditch systems were considered to collect flow in residential areas with free draining roadways (i.e. no curb and gutter) at slopes above 2%. Piped systems in steeper areas without a curb and gutter system are not able to adequately collect the runoff in catch basins. Ponding, poor roadway drainage and erosion of roadway shoulders can still occur when using piped systems.
4. Piped System — Replace undersized pipes or construct a piped system to serve unserved areas. Where piped systems would function well, they are preferable to ditches since maintenance is less costly. Piped systems are preferable in flat areas where standing water would remain in ditches, and in areas where conveyance of flow, and not collection, is the main purpose of the conveyance.

Solutions implementing the above alternatives were determined by:

a. Effectiveness in solving the problem;
b. Practicality of implementation based on cost (i.e. a $1,000,000 improvement would likely take several years to accumulate enough money for the project or would never be built);
c. Ability to subdivide the required system into reasonable projects which could be constructed during different phasing periods;
d. Long term maintenance requirements;
e. Practicality of implementation in conjunction with road improvements; and
f. The overall benefits of the improvement.

A map of all proposed solutions was developed, and then separate improvement projects were identified. An "Improvement" constitutes those lengths of pipe or ditch systems which should be constructed during a single project.

Once individual improvement projects were identified, each proposed project was prioritized relative to the others, based on known upcoming road projects, magnitude of the problem being solved, impact on phasing of upstream improvements, total cost and feasibility of immediate construction, and overall benefit of the improvement. Chapter 7, Identification of Problem Areas and Proposed Solutions discusses existing facilities, the problem areas identified and the rationale for each proposed solution.

Chapter 8, Capital Improvements – Cost, Phasing and Implementation presents the cost of each individual storm project and summarizes the recommended construction phasing.

This plan is intended for improvements phased over about a 20-year period. Naturally, uncertainties involving funding availability and changes in development plans will impact the solutions proposed in this plan. Given these considerations, the emphasis in this plan was to provide adequate information and documentation of results so that in spite of future changes, engineers and planners will have a tool to use as a basis for altering solutions to fit the specific instance. Flexibility of future storm drainage improvements with regards to phasing and cost was also a factor in determining the final solutions. In conclusion, this updated SWMP is meant to provide the City with a long-term planning tool that can be implemented with available funding, addresses community needs to solve existing drainage problems, and mitigates problems that continuing development might cause.

CHAPTER 4: BASIN DELINEATION AND MODEL PARAMETERS

Basin Delineation and Model Parameters

In order to effectively analyze existing facilities and plan future improvements, the Manzanita watershed must be subdivided into meaningful sub-areas. These sub-areas can then be used to calculate needed flow information as well as for drainage planning reference purposes. This chapter describes the delineation of the Manzanita watershed into smaller drainage sub-areas and quantifies the HydroCAD parameters used to calculate flow within each sub-area.

Basin Delineation

The Manzanita watershed was first divided into seven major basins based on discharge location, main drainageways, drainage boundaries, geographic and planning boundaries and similarity throughout the basin. Each major drainage basin was assigned a one or two letter mnemonic, which is used to reference sub-basins within the major drainage basin. They are identified on Figure 4.1, Sub-basin Drainage Map, showing the major basin and sub-basin delineation used in the analysis.

The following major drainage basins were defined:

1. Golf Course Creek Drainageway (GC) — This basin includes all lands historically draining to the Golf Course Creek.
2. Main Manzanita Beach Drainage (B) — This major basin was delineated based on all areas draining to the culverts under Ocean Road and Beach Street which outfall onto the main Manzanita Beach area near Laneda Avenue.
3. Lake Drainages (L) — This includes areas draining to the golf course lake or low areas along the golf course with no defined discharge to the Pacific Ocean.
4. Necarney City Basins (N) — Necarney City is within the Manzanita UGB. The N basins drain either to a culverted system along Sitka Lane or directly to the beach area between Chinook Lane and Glenesslin Lane.
5. Nehalem Bay Drainage (NB) — This area includes the eastern portion of the UGB and drains east to the Nehalem Bay.
6. North UGB (NR) — This area includes the northernmost portion of the UGB draining off the north side of the ridge into a creek system outside of the UGB.
7. Neahkahnie Lake Creek Drainage (NL) — The northeast portion of the UGB drains to the large creek which flows out of Neahkahnie Lake. Peak flows in the creek were beyond the scope of this project. However, sub-basin flows for existing and future conditions were determined for the purpose of planning local improvements.

Each major drainage area was then divided into sub-basins which were used to calculate flow quantities in the HydroCAD v10.0 model. Sub-basins were delineated by locating key "node" points where flow information was required and then based on zoning and topography, the area draining to each node was delineated.

Sub-basins were named based on the one or two letter designation assigned to each main drainage basin. Disconnected sub-basins within the major basin were then numbered consecutively, i.e. B-1, B-2, etc. Connected sub-basins (sub-areas of a single drainage reach) were designated by consecutive letter codes with A at the downstream end, i.e. B-1B is directly upstream of B-1A.

Figures 4.2A, Drainage North, and 4.2B, Drainage South, show the actual storm drain infrastructure.

Sub-Basin Parameters

Sub-basin Runoff Area

For existing conditions, it was assumed that the entire sub-basin contributes to runoff. However, as discussed in Chapter 3 Methodology, future residential developments are required to drain impervious surfaces to on-site dry wells. Areas of future development in sub-basins have been adjusted to account for this requirement. It was assumed that 90% of all future residential lands will be routed to on-site drywells. Since future impervious areas will not contribute to runoff, the future runoff area for sub-basins where dry wells can be implemented will be smaller than the existing runoff area. On-site dry wells are defined as those wells that are on private property and not in the road ROW.

Soil Loss Parameter

The effective impervious area method described above is used to determine the volume of runoff due to the impervious portions of the basin. For the pervious areas within the basin, infiltration and evapotranspiration significantly reduces runoff. The degree of these losses can be estimated using a soil loss parameter developed by the Soil Conservation Service (SCS). This parameter, called the Runoff Curve Number (CN), depends on the soil type, ground cover and antecedent moisture of the area.

In HydroCAD the sub-basin land cover is defined solely by the CN input for each watershed. CN values rely on landcover type determined by zoning and hydrologic soil group described below. Hydrologic soil group data for the City of Manzanita is from the Natural Resource Conservation Service. The CN values for various land uses and land cover characteristics for each soil classification are described below. Due to the complex nature of the basins and MIA methods, calculations for each basin CN was performed in excel, then input into HydroCAD.

The SCS rates soils as belonging to one of four hydrologic groups: A, B, C, D. Soils are defined as follows:

Group A: Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 0.30 in./hr.).

Group B: Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately course textures. These soils have a moderate rate of water transmission (0.15 to 0.30 in./hr.).

Group C: Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately coarse textures. These soils have a moderate rate of water transmission (0.05-0.15 in./hr.).

Group D: Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high-water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0.00 to 0.05 in./hr.).

Type A soils cover most of the City of Manzanita with some type D soils in the southeast quadrant and type B soils in the northeastern most corner. The soils type distribution in the area is shown in Figures 4.3A, Soils Legend, and 4.3B, Basin Delineation and Soils Map.

Based on the hydrologic classification of the soil, a numerical Curve Number can be determined from tables listing SCS Curve Numbers for various antecedent moisture and ground cover conditions. For the Manzanita area, it was assumed that antecedent moisture condition (AMC) III applies. The SCS defines AMC III as the condition applying to areas where the total precipitation 5 days previous to the design storm is in excess of 2.1 inches of rainfall. Data published in the Climatological Handbook for the Columbia Basin States, Precipitation Volume 2, lists the antecedent rainfall 4 days previous to the largest recorded event (approximately a 50-year event) as 4.1 inches of rain fall. While smaller events will have correspondingly smaller antecedent moisture levels, AMC III appears applicable to the wet coastal conditions.

To account for ground cover, assumptions were made for developed areas based on zoning type. Figure 4.4 shows City of Manzanita zoning within each delineated basin.

Curve numbers for single family residential areas are representative of fairly well developed, but not dense, lawns or sparse brush and trees. Curve numbers for multi-family areas represent a higher degree of runoff assuming some graveled areas, possibly landscaped areas with dirt or bark dust and sparser lawns. Curve numbers for commercial areas assume mostly gravel open areas and high runoff. Curve numbers for the golf course lands assume dense grass. Curve numbers for undeveloped areas assume dense brush or forest with dense underbrush.

Figures 4.5A and B show the open space assumptions used for the existing and future models. The open space was factored into the CN for each basin. Most of the open space in the existing model is undeveloped land. The open space in the future model consist of areas that will not be developed like the golf course and lakes. Open space was delineated based on a 2020 aerial from Google Maps.

Composite CN values were calculated for input into HydroCAD. Figure 4.5B shows the Composite CN values for each basin. Soils, land cover and CN for each land cover type used to calculate the composite CN are presented in Figures 4.6 and 4.7. The figures also include lag time and time of concentration.

Time of Concentration

The time of concentration (Tc) is the travel time from the most hydraulically remote point in the sub-basin to the sub-basin outlet. As described in SCS Technical Release #55, the total travel time can be computed by summing the time of travel required for each of the following components of runoff: overland flow, shallow concentrated flow (overland flow in shallow swales), gutter flow, channel flow, and pipe flow.

Sheet Flow — Sheet flow is water sheeting over a plane surface, such as an even amount of water flowing over a parking lot. It is the first component of Tc and starts at the hydraulically most distant watershed point. Sheet flow normally occurs at a depth of 0.1 feet or less, and the length of sheet flow rarely exceeds a few hundred feet. The maximum length of sheet flow in most cases is 300 feet. For commercial areas, overland flow was assumed to occur for 200 feet before being collected by a shallow swale, gutter or main channel.

- Calculation of sheet flow requires the following information: a two-year rainfall amount; length of flow; average slope along flow path; and ground roughness over which the water is sheeting (measured in Manning's n factor).

Overland slope — Overland slopes were calculated based on the topographic mapping.

Shallow Swale Flow — Shallow flow is water flowing in natural drainage depressions and swales, and usually begins after a maximum of 300 feet of sheet flow. The average velocity of shallow concentrated flow is determined by watershed slope and channel material (paved or unpaved). Typical areas where you have shallow flow are in swales between houses and the gutter section of a roadway.

- Calculation of shallow flow requires the following information: flow length; average slope; and a determination of whether the surface is paved or unpaved. The time of travel for shallow swale flow was calculated by using Manning's equation to calculate swale velocities for a typical swale as a function of slope and vegetation.

Channel Flow — Velocities in defined channels and pipes receiving flow from a majority of the basin were estimated using Manning's calculations based on the size, slope and standard n-value for each channel or pipe: n = 0.013 was used for concrete culverts, n = 0.024 for corrugated metal pipes (CMP), n = 0.0375 for average defined channels, and n = 0.08 for overgrown channels.

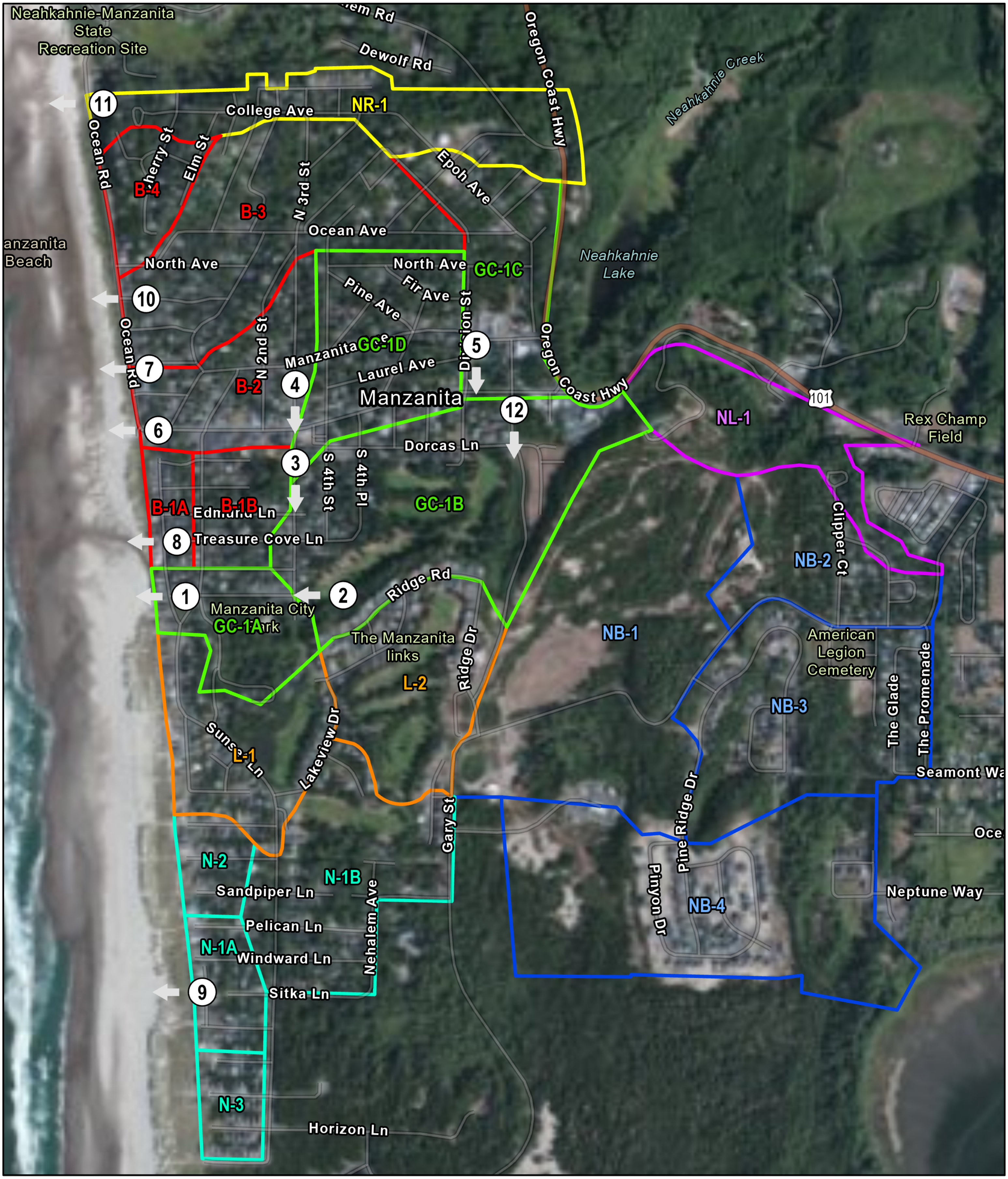


FIGURE 4.1
SUB-BASIN DRAINAGE
CITY OF MANZANITA
STORM WATER MASTER PLAN
MANZANITA | OREGON

Legend

B
GC
L
N
NB
NL
NR
Node ID
Flow Direction

FEET
0 500 1,000

Otak

City of Manzanita

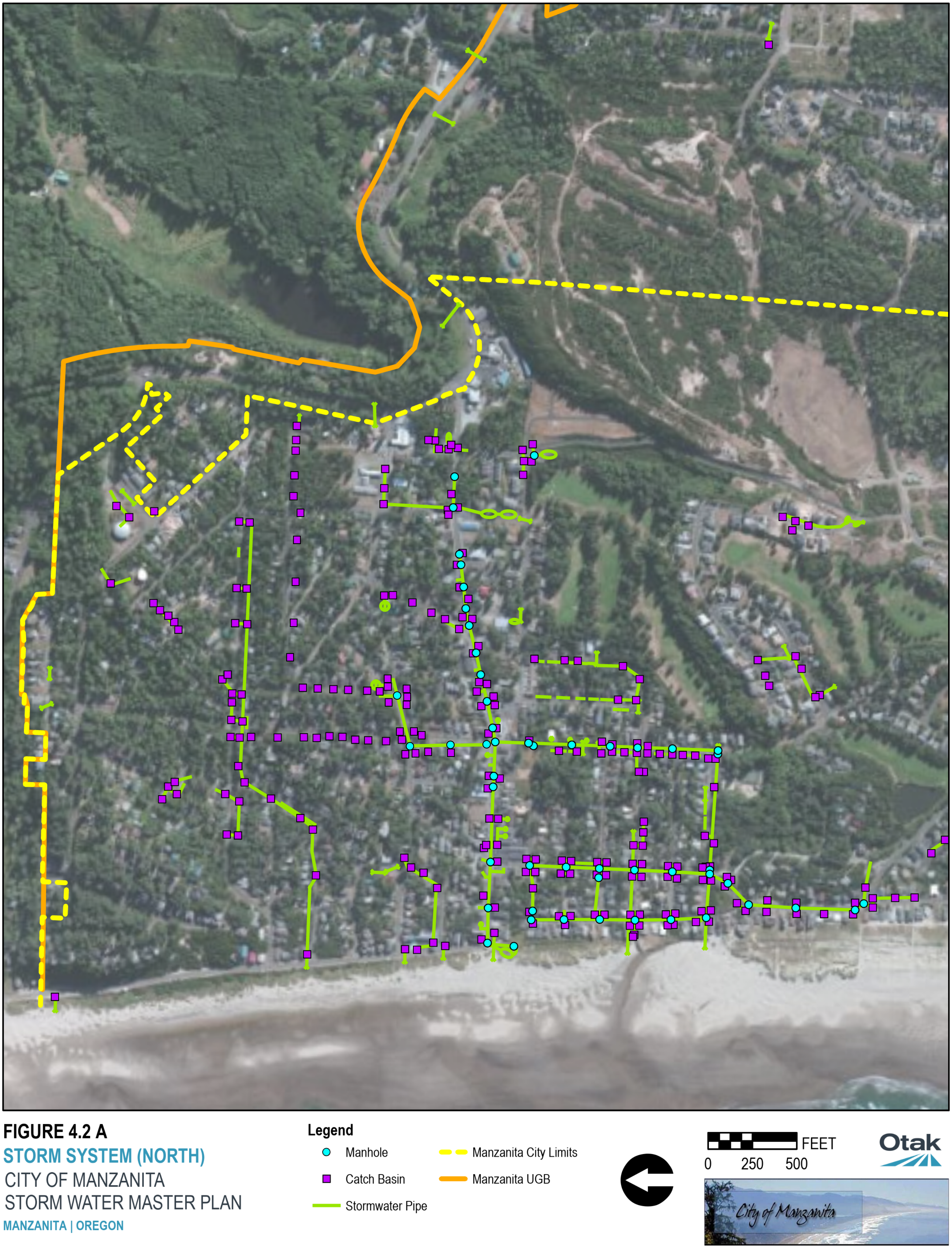


FIGURE 4.2 A

STORM SYSTEM (NORTH)

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

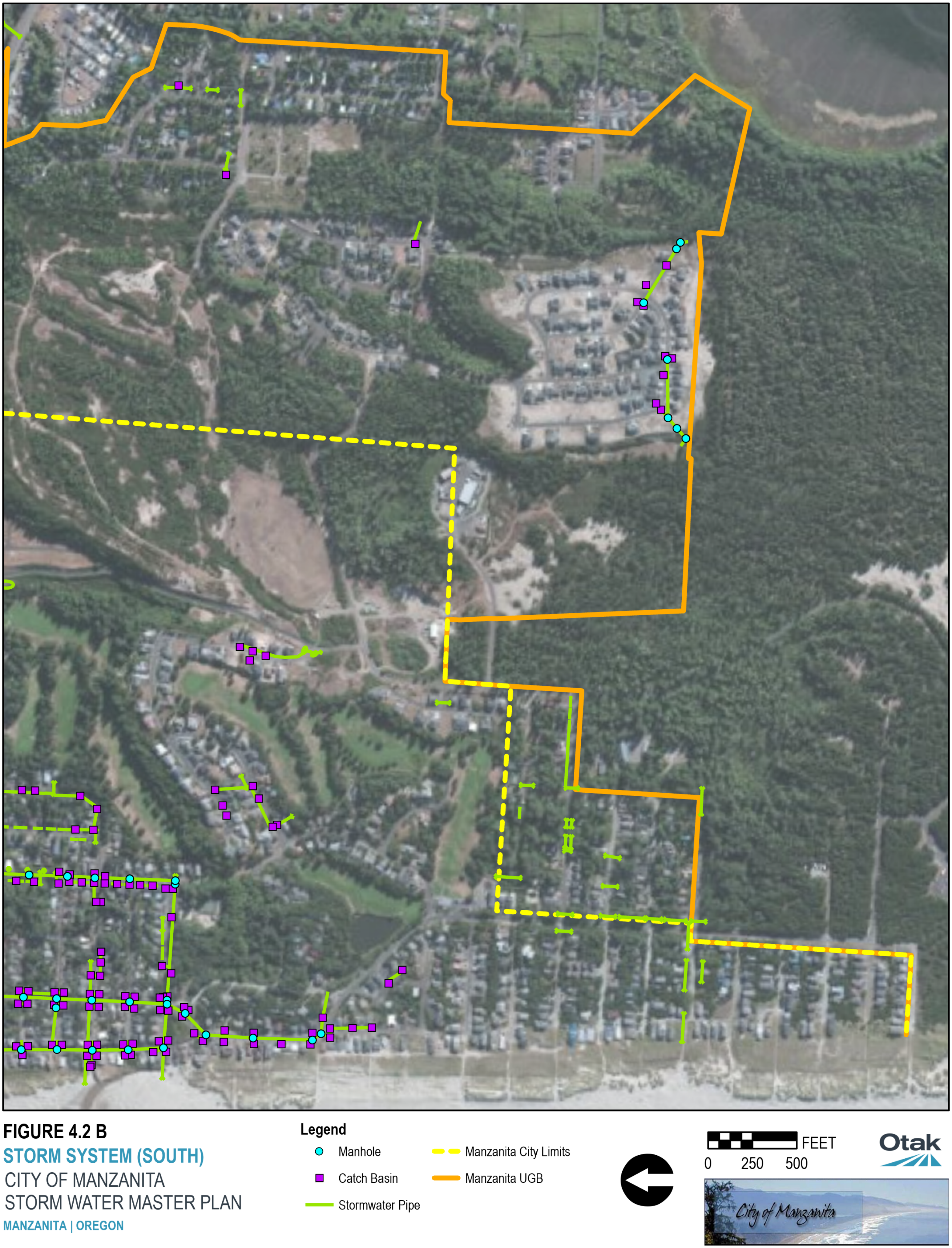


FIGURE 4.2 B

STORM SYSTEM (SOUTH)

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

Figure 4.3A – Soils Legend

HYDROLOGIC GROUP	SOIL NUMBER	SOIL TYPE
A	7	Dune Land
	10B	Waldport fine sand
	10C	
	10E	
	11B	Netarts fine sandy loam
	11D	
	13B	Waldport,thin surface-Heceta fine sands
	9C	Waldport fine sand
	9D	
	9E	
B	2A	Fluvaquents-Histosols complex
	30E	Templeton-Ecola medial silt loams
D	12B	Yaquina loamy fine sand
	14A	Heceta fine sand
	W	Water

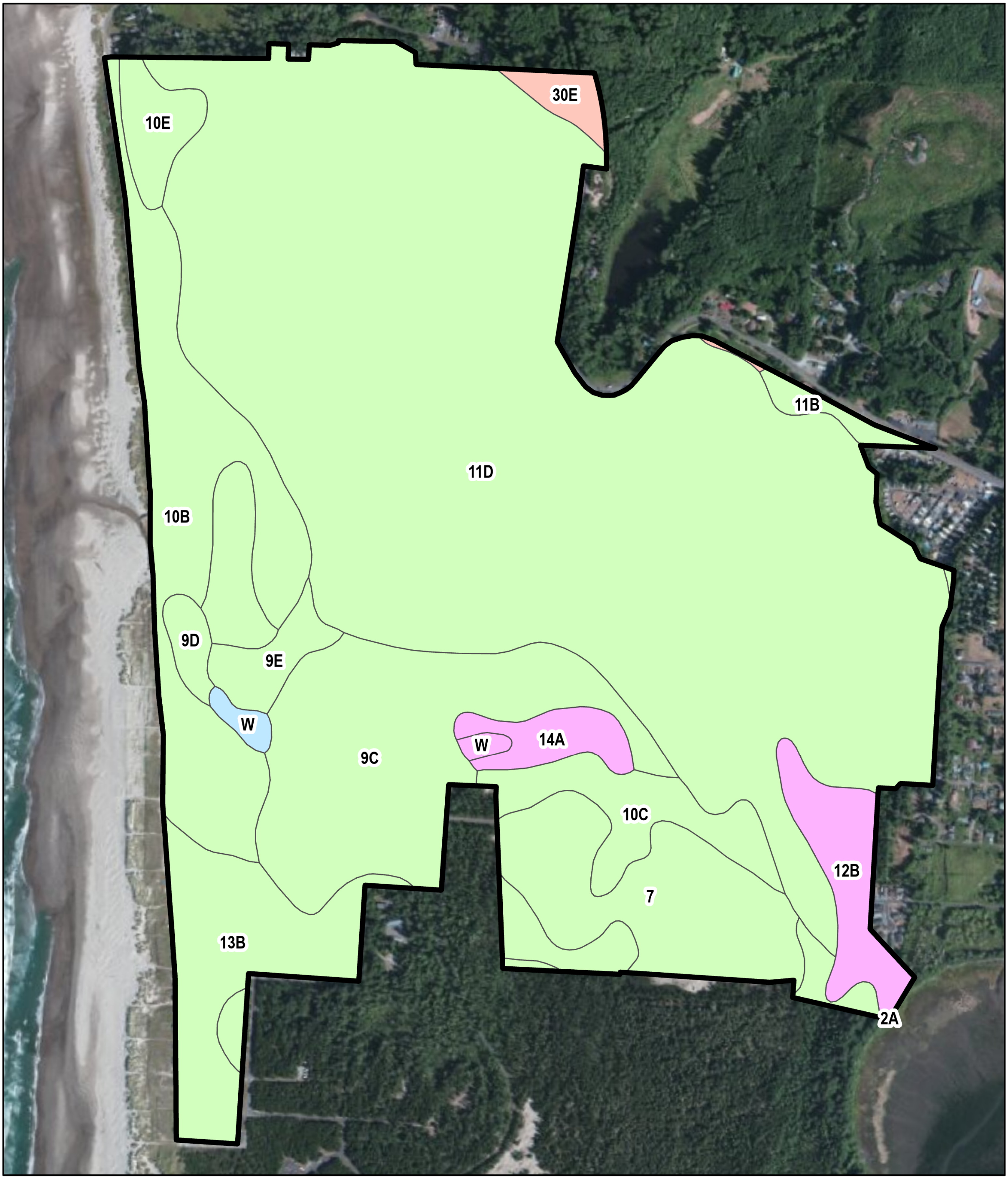


FIGURE 4.3 B

BASIN DELINEATION AND SOILS

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

Hydrologic Soil Group

A

B

A/D*

Lake

* D soils were used for calculations and modeling

FEET

0 500 1,000

Otak

City of Manzanita

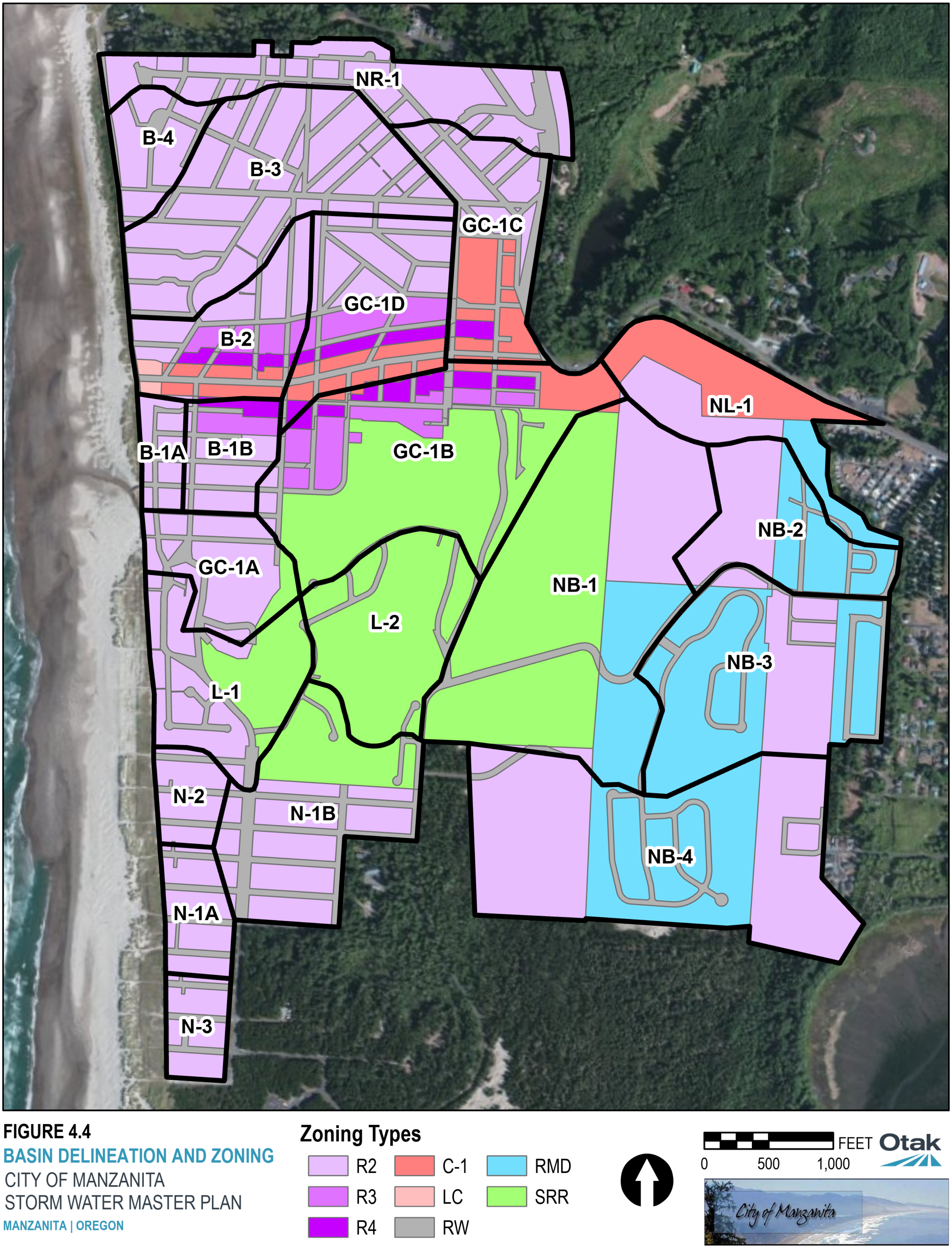


FIGURE 4.4

BASIN DELINEATION AND ZONING

CITY OF MANZANITA

STORM WATER MASTER PLAN

MANZANITA | OREGON

FIGURE 4.5 A

MAPPED OPEN SPACE

CITY OF MANZANITA
STORM WATER MASTER PLAN

MANZANITA | OREGON

Legend

- Full Build Out Open Space
- Current Open Space

FEET
0 500 1,000

Otak

City of Manzanita

Figure 4.5B – Runoff Curve Numbers (CN) for Urban Areas[1]

Cover Description		Curve Numbers for Hydraulic Soil Group			
Cover Type and Hydrologic Condition	Average % Impervious Area[2]	A	B	C	D
Fully Developed Urban Areas (Vegetation Established)					
Open Space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50 to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious Areas:					
Paved parking lots, roofs, driveways, etc. (excluding ROW)		98	98	98	98
***Streets and Roads*:**					
Paved, curbs and storm sewers (excluding ROW)		98	98	98	98
Paved, open ditches (including ROW)		83	89	92	93
Gravel (including ROW)		76	85	89	91
Dirt (including ROW)		72	82	87	89
Urban Districts:					
Commercial business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential Districts by Average Lot Size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	91	82

(Source: U.S. Soil Conservation Service 1986.)

1. Average runoff condition, and $I_a = 0.2S$.
2. The average% impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.
3. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open-space cover type.

Figure 4.6 - Existing Basin Parameters Calc Sheet

Assumes Existing degree of pavement, disconnected system

Unit Values per Zoning Type									
Land Use at Full Buildout	R/W	R2	RMD	R3	R4	LC	C1	SRR	Open
(MIA / CN)	50	40	40	55	60	90	90	40	0
CN - A	83	56	58	61	61	89	89	52	49
CN - B	89	75	75	75	75	92	92	79	69
CN - C	92	83	83	83	83	94	94	87	79
CN - D	93	87	87	87	87	95	95	93	84
Future Contributing Runoff %	100%	70%	70%	70%	70%	80%	80%	70%	100%

			Zoning									Hydrologic Class							
SUB-BASIN	Runoff Area (acres)	Adj. Runoff Area (acres)	R/W	R2	RMD	R3	R4	LC	C1	SRR	Open	A	B	C	D	Composite CN	Lag Time	Tc (Hr)	Tc (Min)
Manzanita Beach	**115.6**	**91.1**																	
B-1A	6.9	5.3	24%	76%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.39	0.65	39
B-1B	13.6	11.2	21%	45%	0%	10%	4%	0%	0%	0%	20%	100%	0%	0%	0%	61	0.66	1.10	66
B-2	26.1	20.4	22%	38%	0%	14%	8%	3%	15%	0%	0%	100%	0%	0%	0%	69	0.43	0.72	43
B-3	56.3	43.6	25%	75%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.47	0.78	47
B-4	12.8	10.6	25%	57%	0%	0%	0%	0%	0%	0%	18%	100%	0%	0%	0%	61	0.29	0.48	29
Golf Course	**152.8**	**129.9**																	
GC-1A	19.5	16.8	20%	39%	0%	0%	0%	0%	0%	7%	34%	100%	0%	0%	0%	59	0.52	0.87	52
GC-1B	68.6	60.3	13%	2%	0%	14%	7%	0%	7%	13%	44%	100%	0%	0%	0%	59	0.56	0.93	56
GC-1C	30.2	25.5	35%	34%	0%	0%	4%	0%	21%	0%	6%	100%	0%	0%	0%	72	0.33	0.55	33
GC-1D	34.4	27.3	25%	36%	0%	14%	9%	0%	16%	0%	0%	100%	0%	0%	0%	69	0.32	0.53	32
Golf Course Lake	**64.6**	**55.4**											0%	0%	0%				
L-1	29.9	24.7	20%	43%	0%	0%	0%	0%	0%	15%	22%	92%	0%	0%	0%	55	0.37	0.62	37
L-2	34.8	30.8	10%	0%	0%	0%	0%	0%	0%	38%	51%	99%	0%	0%	1%	54	0.27	0.45	27
Necarney City	**71.3**	**59.3**																	
N-1A	11.5	9.1	22%	70%	0%	0%	0%	0%	0%	0%	8%	100%	0%	0%	0%	61	0.35	0.58	35
N-1B	42.4	36.7	25%	34%	0%	0%	0%	0%	0%	11%	29%	100%	0%	0%	0%	60	0.95	1.58	95
N-2	8.6	6.6	20%	80%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	62	0.36	0.60	36
N-3	8.8	7.0	21%	67%	0%	0%	0%	0%	0%	0%	12%	100%	0%	0%	0%	61	0.36	0.60	36
Neah-Kah-Nie Lake	**29.2**	**26.1**																	
NL-1	29.2	26.1	1%	8%	18%	0%	0%	0%	14%	0%	60%	99%	1%	0%	0%	57	0.50	0.83	50
Nehalem Bay	**256.5**	**232.6**																	
NB-1	92.5	89.5	5%	1%	3%	0%	0%	0%	0%	7%	84%	89%	0%	0%	11%	55	0.50	0.83	50
NB-2	27.5	24.1	10%	0%	41%	0%	0%	0%	0%	0%	49%	100%	0%	0%	0%	56	0.43	0.72	43
NB-3	55.3	43.9	15%	12%	57%	0%	0%	0%	0%	0%	16%	95%	0%	0%	5%	62	0.64	1.07	64
NB-4	81.2	75.1	7%	9%	16%	0%	0%	0%	0%	0%	68%	82%	0%	0%	18%	59	0.37	0.62	37
North Ridge	**36.6**	**28.3**																	
NR-1	36.6	28.3	27%	72%	0%	0%	0%	0%	0%	0%	0%	86%	14%	0%	0%	65	0.35	0.58	35

Figure 4.7 - Future Basin Parameters Calc Sheet

Assumes full pavement, disconnected system

Unit Values per Zoning Type									
Land Use at Full Buildout	R/W	R2	RMD	R3	R4	LC	C1	SRR	Open
(MIA / CN)	50	40	40	55	60	90	90	40	0
CN - A	83	56	58	61	61	89	89	52	49
CN - B	89	75	75	75	75	92	92	79	69
CN - C	92	83	83	83	83	94	94	87	79
CN - D	93	87	87	87	87	95	95	93	84
Future Contributing Runoff %	100%	70%	70%	70%	70%	80%	80%	70%	100%

			Zoning									Hydrologic Class							
SUB-BASIN	Runoff Area (acres)	Adj. Runoff Area (acres)	R/W	R2	RMD	R3	R4	LC	C1	SRR	Open	A	B	C	D	Composite CN	Lag Time	Tc (Hrs)	Tc (Mins)
Manzanita Beach	**115.6**	**89.5**																	
B-1A	6.9	5.3	24%	76%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.34	0.57	34
B-1B	13.6	10.4	21%	52%	0%	16%	10%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.55	0.92	55
B-2	26.1	20.4	22%	38%	0%	14%	8%	3%	15%	0%	0%	100%	0%	0%	0%	69	0.40	0.67	40
B-3	56.3	43.5	25%	75%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.41	0.68	41
B-4	12.8	9.9	25%	75%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	63	0.21	0.35	21
Golf Course	**152.8**	**127.8**																	
GC-1A	19.5	16.7	20%	41%	0%	0%	0%	0%	0%	7%	32%	100%	0%	0%	0%	59	0.46	0.77	46
GC-1B	68.6	58.8	13%	2%	0%	14%	7%	0%	9%	20%	36%	100%	0%	0%	0%	60	0.45	0.75	45
GC-1C	30.2	25.2	35%	34%	0%	0%	4%	0%	27%	0%	0%	100%	0%	0%	0%	75	0.23	0.38	23
GC-1D	34.4	27.3	25%	36%	0%	14%	9%	0%	16%	0%	0%	100%	0%	0%	0%	69	0.24	0.40	24
Golf Course Lake	**64.6**	**55.4**											0%	0%	0%				
L-1	29.9	24.7	20%	43%	0%	0%	0%	0%	0%	15%	22%	92%	0%	0%	0%	55	0.37	0.62	37
L-2	34.8	30.8	10%	0%	0%	0%	0%	0%	0%	38%	51%	99%	0%	0%	1%	54	0.27	0.45	27
Necarney City	**71.3**	**57.2**																	
N-1A	11.5	8.8	22%	78%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	62	0.29	0.48	29
N-1B	42.4	35.2	25%	46%	0%	0%	0%	0%	0%	11%	18%	100%	0%	0%	0%	61	0.87	1.45	87
N-2	8.6	6.6	20%	80%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	62	0.34	0.57	34
N-3	8.8	6.7	21%	79%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	62	0.34	0.57	34
Neah-Kah-Nie Lake	**29.2**	**23.7**																	
NL-1	29.2	23.7	1%	29%	18%	0%	0%	0%	25%	0%	28%	99%	1%	0%	0%	63	0.43	0.72	43
Nehalem Bay	**256.5**	**188.9**																	
NB-1	92.5	66.1	5%	23%	17%	0%	0%	0%	0%	56%	0%	89%	0%	0%	11%	59	0.35	0.58	35
NB-2	27.5	20.1	10%	48%	41%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	60	0.28	0.47	28
NB-3	55.3	41.2	15%	28%	57%	0%	0%	0%	0%	0%	0%	95%	0%	0%	5%	63	0.40	0.67	40
NB-4	81.2	61.6	7%	53%	27%	0%	0%	0%	0%	0%	12%	82%	0%	0%	18%	63	0.24	0.40	24
North Ridge	**36.6**	**28.3**																	
NR-1	36.6	28.3	27%	72%	0%	0%	0%	0%	0%	0%	0%	86%	14%	0%	0%	65	0.27	0.45	27

CHAPTER 5: RAINFALL ANALYSIS

Rainfall Analysis

Once sub-basin parameters have been input into the model, the only remaining variable is rainfall. While rainfall is the driving force of the hydrologic model, it is also the most uncertain. However, statistical analysis of historical storms and use of regional data can allow the engineer or planner to estimate the approximate frequency with which a storm exceeding a given magnitude will occur. In addition, regional studies conducted by the SCS are useful in estimating the time distribution of rainfall during the peak period of a storm likely to cause flooding. Using these sources, a hypothetical "design storm" (or rainfall distribution) can be developed for use in the hydrologic models.

Design Storm Duration

The duration of the design storm depends upon the characteristics of the basins being studied. Typically, short duration storms should be used for design of steep, impervious basins where a high intensity of rainfall produces the greatest peak runoff. In larger, more pervious basins, a lesser intensity of rainfall preceded by a large volume of rain resulting in saturated conditions will produce the greatest peak flows. Because of the pervious nature of the basins being studied in Manzanita, a 24-hour storm duration was selected as the design storm. This value is typically used for basins of this size, and only under impervious conditions would it be likely that a 6-hour duration would be more applicable.

Precipitation Depth-Frequency Relationship

Values for the 24-hour precipitation for the 2, 5, 10, 25, 50, and 100-year design frequencies were determined from isopluvial maps published in the NOAA Precipitation-Frequency Atlas of the Western United States Volume X- Oregon 1973. These values were then compared to daily gauge data from the Tillamook rainfall gauge as described below.

Computer programs were used to extract the largest daily recorded precipitation for each year of record. These values were then sorted in ascending order. Theoretically, when the data is sorted in this order, the 2-year event should be the value that is halfway down the list; the five-year event should be exceeded by 1/5 of the total number of values, etc. However, when daily data and not hourly data is used, a clock adjustment must be made to account for the fact that the largest 24-hour precipitation value will not be in sequence with the 24-hour period in which the gauge is read. The NOAA Atlas recommends that daily data be increased by a clock adjustment factor of 1.13. For the purposes of comparison, the NOAA values in Figure 5.1 should be increased by this factor, should an additional engineering study be required.

The distribution of gauge data verifies the NOAA predictions for this area and justifies use of the regional values. When significant differences in distribution are observed between regional values and local gauge data, further study is often necessary. However, the reasonable correspondence of the NOAA values with the gauge data justifies use of the regional values for use in the final model.

Figure 5.1 – Largest Recorded Daily Rainfall per Annum (Gauge Data from NOAA - TILLAMOOK)

FIGURE 5.1	LARGEST RECORDED DAILY RAINFALL per ANNUM for TILLAMOOK, OR												
Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1948	M	M	M	M	M	M	0.38	0.43	1.04	1.48	2.45	2.19	**2.45**
1949	0.9	3.17	1.26	0.58	2.5	0.18	0.32	0.27	0.67	1.3	1.63	1.8	**3.17**
1950	4.42	1.5	2.01	1.43	0.83	0.65	0.85	0.56	0.97	1.61	3.02	1.73	**4.42**
1951	2.44	1.91	2.49	1.6	0.72	0.16	0.37	0.2	1.34	2.12	1.92	1.76	**2.49**
1952	1.92	2.52	1.72	0.67	0.6	0.5	0.05	0.41	0.48	0.42	0.81	1.5	**2.52**
1953	1.97	1.18	1.36	1.06	1.15	M	0.28	1.71	1.94	2	1.75	2.55	**2.55**
1954	3.53	3.06	1.79	1.17	0.76	0.61	0.56	1.36	0.77	1.42	1.95	2.7	**3.53**
1955	1.21	2	2.35	1.97	0.4	0.87	1.5	0.15	1.3	3.02	3.22	4.4	**4.4**
1956	2.6	1.5	1.89	0.62	0.72	0.72	0.23	1.24	1.16	1.42	0.93	1.61	**2.6**
1957	1.02	1.79	2.23	1.74	1.35	1.05	0.65	0.88	0.24	0.93	1.76	2.95	**2.95**
1958	1.91	1.76	0.86	1.93	0.47	1.14	0.02	0.32	0.79	1.21	3.17	1.86	**3.17**
1959	2.26	1.03	1.12	1.31	0.71	1.5	0.86	0.3	1.86	2.19	2.42	1.37	**2.42**
1960	1.84	2.43	2.72	1.1	1.26	0.62	T	1.05	0.57	1.77	4.02	2.72	**4.02**
1961	1.59	2.66	1.94	0.8	0.75	0.31	0.36	0.94	0.75	1.88	2.02	1.62	**2.66**
1962	1.67	1.74	1.78	2.92	0.58	0.78	0.19	1.14	2.29	1.26	3.04	1.52	**3.04**
1963	1.81	1.41	1.27	0.93	1.09	0.77	0.84	1.57	0.85	3.02	2.04	1.73	**3.02**
1964	3.82	0.85	1.27	0.9	0.56	1.07	0.82	0.58	0.67	0.53	2.45	2.65	**3.82**
1965	4.65	1.28	0.71	2.04	0.53	0.51	0.23	0.4	0.43	1.02	1.89	2.5	**4.65**
1966	1.22	1.05	2.2	0.69	0.55	0.83	0.24	0.4	0.52	1.72	1.86	2.55	**2.55**
1967	2.02	1.57	1.28	1.15	0.62	0.64	0	0.07	1.32	1.78	1.83	3	**3**
1968	2.85	2.94	2.05	1	0.88	2.5	0.32	1.53	1.15	1.75	3.1	1.62	**3.1**
1969	2.35	2	0.74	1.12	1.05	2	0.22	0.4	1.43	1	3	1.85	**3**
1970	2.7	1.8	1.13	1.48	1.55	0.48	0.43	0.13	1.2	1.73	1.37	3.25	**3.25**
1971	4.25	1.75	1.78	1.87	0.94	1.02	1.45	0.71	2.65	2.15	2.21	3.42	**4.25**
1972	4.2	1.4	2.17	1.55	0.55	0.63	0.41	0.26	1.72	1.12	1.97	3.3	**4.2**
1973	1.45	0.83	1.4	0.83	1.67	1.4	0.11	0.69	1.32	2.8	2.58	2.78	**2.8**
1974	2.77	1.57	3.14	2.12	0.92	1.58	2	0.37	0.64	0.98	2.83	3.52	**3.52**
1975	2.03	1.66	1.37	1.25	1.68	0.54	0.49	1.02	0.03	1.72	4.26	3.1	**4.26**
1976	2.57	2.32	1.85	0.82	0.72	0.23	1.06	0.69	1.42	1	0.61	1.49	**2.57**
1977	0.74	1.31	1.52	0.48	1.26	0.74	0.15	0.93	1.88	1.52	3.44	4.92	**4.92**
1978	1.27	1.5	0.85	1.2	1.45	0.63	0.5	0.88	1.01	0.78	1.94	0.95	**1.94**
1979	1.76	1.72	1.6	0.86	M	0.9	0.58	0.25	1.49	2.04	2.14	2.54	**2.54**
1980	1.8	1.76	1.34	1.3	0.89	0.91	0.39	0.34	0.9	0.89	1.9	2.92	**2.92**
1981	0.91	1.8	1.75	1.97	0.82	1.95	0.38	0.3	0.75	3.57	1.85	2.24	**3.57**
1982	5.22	2.04	1.22	2.13	0.32	0.56	1.1	0.31	1.44	1.16	1.68	3.35	**5.22**
1983	2.95	1.7	2.9	0.64	1.11	1.59	1.5	0.45	1.04	1.17	2.8	1.6	**2.95**
1984	2.43	2.17	0.57	1.12	0.96	1.65	1.07	0.18	0.9	2.18	2.29	1.29	**2.43**
1985	0.2	1.8	1.5	0.69	0.52	2.31	0.4	0.85	0.89	2.34	1.97	0.95	**2.34**
1986	1.68	2.35	1.64	0.86	0.98	0.35	1.08	0.22	1.25	2	1.94	1.54	**2.35**
1987	1.68	2.03	3.75	0.64	1.24	0.85	0.89	0.6	0.52	0.62	0.84	2.7	**3.75**
1988	2.2	1.91	1.73	1.05	1.2	0.94	0.47	0.78	1.45	0.71	1.62	1.73	**2.2**
1989	2.09	2.3	2.1	0.63	0.95	1.26	1.64	1.16	0.5	1.52	1.31	2.5	**2.5**
1990	4.14	2.05	1.4	1.51	0.87	0.78	0.26	0.29	0.2	1.31	2.34	1	**4.14**
1991	1.95	3.33	1.52	3.65	1.2	0.52	0.29	0.48	0.1	0.98	2.68	2.12	**3.65**
1992	2.25	2.05	0.38	1.99	0.36	0.46	0.1	0.5	1.15	2.06	2	1.37	**2.25**
1993	1.38	0.6	1.77	1.15	0.89	1.03	1.33	0.16	0.06	0.95	0.91	2.2	**2.2**
1994	2.03	1.89	1.4	2	0.86	1.05	0.1	0.41	0.73	4.67	3.01	4.02	**4.67**
1995	2.05	3.6	2	1.23	0.48	0.87	0.2	0.58	1.48	1.54	3.98	2.23	**3.98**
1996	2.19	4.84	0.87	4.35	1.28	0.75	0.67	0.68	1.77	1.97	3.42	2.2	**4.84**
1997	3.32	0.97	2.5	1.16	1.31	1.85	0.87	1.29	2.78	2	3.35	3.66	**3.66**
1998	2.29	1.4	1.48	0.6	0.89	0.86	0.18	0.01	0.45	3.11	3.85	2.48	**3.85**
1999	2.3	3.04	1.55	0.67	1.45	0.66	0.49	0.41	0.07	1	3.98	2.23	**3.98**
2000	1.35	1.55	0.88	0.82	1.4	1.53	0.3	0.39	1.1	1.25	0.68	2.61	**2.61**
2001	1.07	0.86	0.9	1.14	1.17	0.7	0.5	2.2	0.53	1.1	1.89	1.95	**2.2**
2002	2.95	1.02	2.75	1.24	0.68	0.97	0.27	0.18	0.51	0.91	1.19	2.03	**2.95**
2003	4.32	1.29	2.23	1	1.35	0.21	0.19	0.07	0.82	1.41	1.87	2.16	**4.32**
2004	4.13	0.94	0.91	1.43	0.89	1.94	0.11	1.4	2	1.31	1.1	2.81	**4.13**
2005	2.02	1.05	2.04	2	1.18	2.02	0.52	0.48	1.93	2.29	1.67	2.19	**2.29**
2006	2.49	1.46	1.16	0.93	1.15	0.75	0.61	0.08	0.81	1.7	3.49	2.58	**3.49**
2007	2.59	2.3	2.1	0.99	0.75	0.94	0.64	0.46	0.98	1.14	1.43	5.06	**5.06**
2008	1.29	2	1.39	1.6	0.52	1.32	0.15	1.1	0.36	1.42	2.56	2.12	**2.56**
2009	3.38	0.8	1.52	1.1	1.55	0.47	0.31	0.19	0.98	1.17	1.33	2.31	**3.38**
2010	2.19	1.13	1.53	2.15	1.32	1.1	0.64	0.43	0	0	M	M	**2.19**
2011	M	M	M	M	M	M	M	M	M	M	M	M	**M**
2012	M	M	M	0.48	0.9	0.62	0.27	0.02	0.02	2.2	3.3	4.65	**4.65**
2013	1.2	1.57	1.25	1.05	2.3	0.6	0.01	0.52	2.57	0.7	1.47	1.35	**2.57**
2014	1.33	1.75	2.59	1.87	1.4	1.24	1.03	0.3	2.17	1.55	1.77	2.12	**2.59**
2015	2.55	1.95	2.29	1	0.9	0.39	0.05	1.03	0.82	1.15	2.64	4.64	**4.64**
2016	1.8	1.97	1.6	0.63	0.8	0.63	0.58	0.25	1.1	4.02	3.3	2.3	**4.02**
2017	3.3	3.6	2.88	1.09	1.3	1.94	0.15	0.5	1.25	4.2	1.69	1.7	**4.2**
2018	2.04	1.06	2.99	1.93	0.47	1.11	0.09	0.29	1.75	0.68	2.37	1.85	**2.99**
2019	2.92	0.7	0.94	1.8	0.73	0.27	0.43	0.22	0.96	13.92	0.38	7.26	**13.92**
2020	10.32	8.52	M	M	1.29	M	M	M	M	M	M	M	**10.32**
Mean	2.43	1.92	1.7	1.33	0.99	0.95	0.52	0.59	1.06	1.81	2.22	2.48	**3.56**
Max	10.32	8.52	3.75	4.35	2.5	2.5	2	2.2	2.78	13.92	4.26	7.26	**13.92**
	2020	2020	1987	1996	1949	1968	1974	2001	1997	2019	1975	2019	**2019**
Min	0.2	0.6	0.38	0.48	0.32	0.16	0	0.01	0	0	0.38	0.95	**1.94**
	1985	1993	1992	2012	1982	1951	1967	1998	2010	2010	2019	1985	**1978**

Figure 5.2 represents these values after statistical distribution to highlight the precipitation vs the Design Storm Frequency.

Figure 5.2 – Design Precipitation Frequency – Design Values

Design Frequency	24 Hour Design Precipitation Depth (inches)
2	3.5
5	4.5
10	5.0
25	6.0
* 50	6.5
100	6.9

** 50-yr storm event shall be used for all future design and planning*

Figure 5.3 – City of Manzanita Monthly Rainfall Data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yearly Total
Avg. Record Data 1969 to 1996	12.22	8.56	7.82	6.11	4.89	2.93	0.50	2.69	4.40	8.56	13.44	15.16	87.28
1996	11.86	20.59	4.79	14.03	6.35	2.08	2.14	1.06	4.83	11.85	15.66	24.89	120.13
1997	15.79	6.26	17.43	8.29	4.80	6.09	1.77	2.23	8.20	13.15	8.56	12.42	104.99
1998	17.10	12.90	11.07	3.16	5.75	3.13	0.69	0.03	1.83	8.70	23.50	24.58	112.44
1999	14.26	19.42	10.26	4.18	7.08	4.54	0.69	1.74	0.24	4.67	21.75	17.84	106.67
2000	12.59	6.60	6.35	4.75	7.25	5.44	0.61	0.97	4.11	7.41	5.33	6.34	67.75
2001	6.98	4.47	7.65	7.35	3.56	4.36	1.14	4.97	1.79	5.48	18.13	15.25	81.13
2002	19.37	7.10	10.06	7.35	2.80	3.42	0.22	0.09	0.86	1.95	7.36	18.83	79.41
2003	16.96	5.63	13.67	8.45	3.83	1.27	0.21	0.13	2.96	9.03	12.11	14.94	89.19
2004	21.82	9.01	7.83	6.11	6.34	2.81	0.28	5.40	7.72	12.01	8.99	10.78	99.10
2005	9.35	3.79	10.95	8.82	9.06	5.76	3.11	1.65	2.70	13.31	13.91	15.89	98.30
2006	33.49	6.61	10.17	4.55	5.63	4.48	1.4	0.4	2.93	5.69	30.7	13.7	119.72
2007	11.34	16.95	12.65	6.68	2.56	4.04	4.17	1.33	2.97	6.95	8.65	16.4	94.72
2008	11.39	7.78	11.12	8.70	3.72	3.61	1.24	4.56	1.55	5.28	16.5	15.4	90.84
2009	12.74	4.10	13.07	7.94	6.89	1.87	0.76	2.11	3.3	10.8	22.1	10.9	96.58
2010	19.62	13.33	15.28	18.82	11	10.3	1.15	0.99	17.4	15.4	22.8	26.1	172.10
2011	21.20	7.10	9.60	7.20	4.31	1.55	1.74	0.2	2.75	3.72	10.2	2.9	72.49
2012	7.44	7.02	14.01	9.30	4.29	4.37	0.86	0.34	0.22	17.8	16	16	97.72
2013	12.76	8.10	4.66	6.23	7.69	3.05	0.02	2.39	13.5	2.46	8.55	5.14	74.51
2014	8.00	9.63	15.00	8.37	4.66	2.99	1.49	0.8	4.26	12.8	9.1	13.7	90.74
2015	10.58	9.00	7.89	4.96	2.42	0.68	0.46	1.74	2.31	9.6	16.6	27.3	93.52
2016	14.87	11.65	14.23	3.15	1.89	3.2	2.61	1.14	2.87	19.2	20.9	11.7	107.27
2017	8.56	15.21	18.36	9.89	6.51	5.09	0.1	0.48	3.29	12.2	19.3	11.1	110.02
2018	12.79	8.08	7.45	10.57	1.2	2.94	0.25	1.02	3.82	8.55	8.33	12.1	77.09
2019	6.51	11.05	3.66	6.85	2.91	1.16	2.03	0.41	6.84	7.47	3.55	9.24	61.68
2020	20.27	10.52	16.33	3.03	5.14	4.52	1.53	1.48	5.96				-

**The Engineer or Designer shall use the 50 year data, with the adjusted value of the NOAA rainfall data for future design and planning.*

Design Storm Hydrograph

The design storm hydrograph is the actual time distribution of rainfall that is input into the models. Once total 24-hour precipitation values were determined, design storm hydrographs were developed by applying these total values to the standard SCS Type 1A rainfall distribution. The SCS has published typical rainfall distributions for a variety of regions throughout the United States. Type 1A distributions were found to be representative of large storms along the Oregon and Northern California coast.

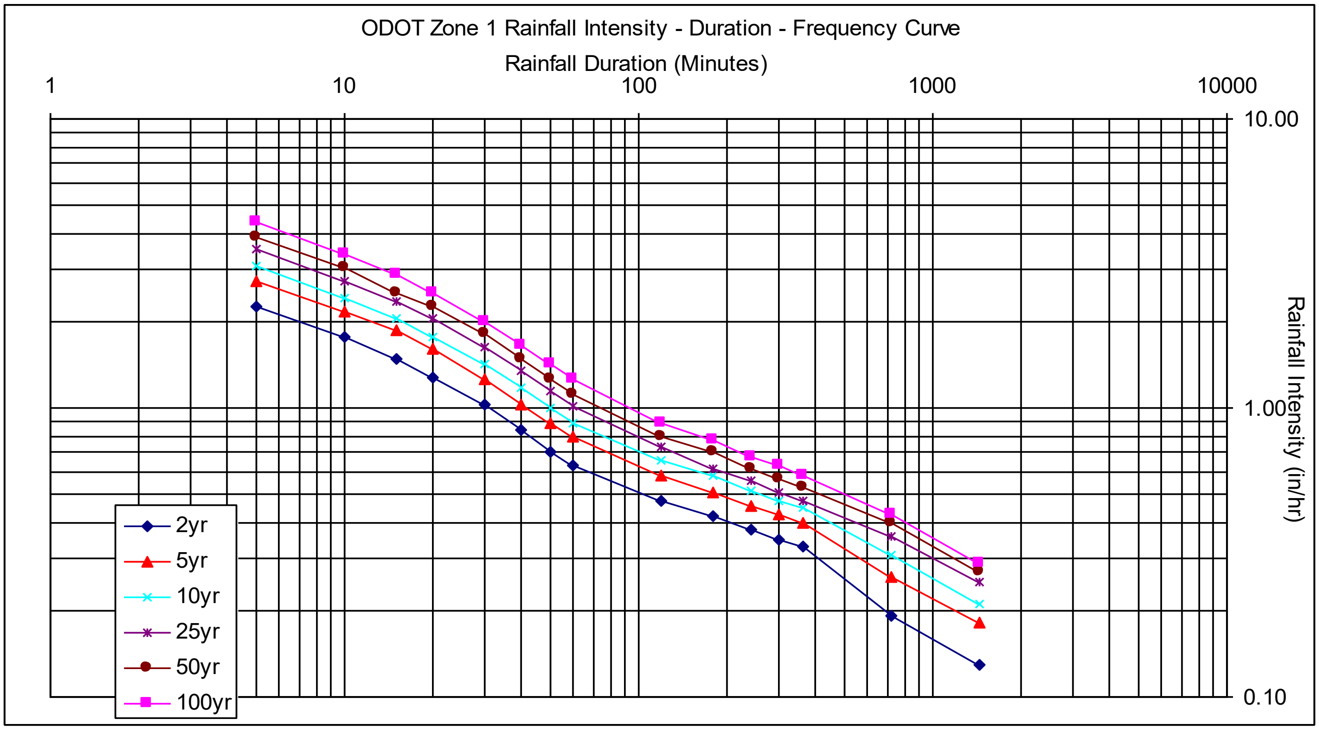
Rainfall Intensity-Duration-Frequency Curves

Rainfall intensity-duration-frequency (IDF) curves are used to determine the rainfall intensity in inches per hour associated with a given storm duration and design frequency. This information is required when computing flows using the Rational Method. While not used for calculations in this study, the Rational Method is a popular method of computing small basin flows and is often used by planners and engineers. Because of the Rational Method's popularity, IDF curves were included based upon Oregon Department of Transportation Hydraulics Manual. The 24-hour precipitation depths presented in Figure 5.2 were integrated with the ODOT IDF Curves. Manzanita falls at the edge of Zone 2 within the ODOT map, however, due to Manzanita's unique micro-climate caused by Neahkahnie Mountain, rainfall events tend to align with the Zone 1 IDF table. It is because of the collected data and observations throughout the past 20 years that we recommend the City of Manzanita be modeled using the Zone 1 IDF table. The resulting IDF tables are shown in Figure 5.4, and the corresponding curves are shown in Figure 5.5. These curves are intended to assist City staff and engineers with future design projects.

Figure 5.4 – Zone 1 IDF Tables

Zone 1 - Rainfall Intensity - Duration - Frequency Curves											
2 Year		5 Year		10 Year		25 Year		50 Year		100 Year	
Duration	Intensity	Duration	Intensity	Duration	Intensity	Duration	Intensity	Duration	Intensity	Duration	Intensity
5	2.24	5	2.75	5	3.09	5	3.53	5	3.92	5	4.40
10	1.75	10	2.15	10	2.40	10	2.75	10	3.07	10	3.40
15	1.47	15	1.85	15	2.05	15	2.35	15	2.50	15	2.88
20	1.28	20	1.60	20	1.75	20	2.05	20	2.24	20	2.50
30	1.03	30	1.25	30	1.42	30	1.63	30	1.80	30	2.00
40	0.84	40	1.03	40	1.18	40	1.34	40	1.48	40	1.65
50	0.70	50	0.89	50	1.00	50	1.15	50	1.26	50	1.42
60	0.63	60	0.80	60	0.89	60	1.02	60	1.12	60	1.25
120	0.48	120	0.58	120	0.66	120	0.73	120	0.80	120	0.88
180	0.42	180	0.51	180	0.58	180	0.62	180	0.70	180	0.77
240	0.38	240	0.46	240	0.52	240	0.56	240	0.62	240	0.68
300	0.35	300	0.43	300	0.48	300	0.51	300	0.57	300	0.63
360	0.33	360	0.40	360	0.45	360	0.48	360	0.53	360	0.58
720	0.19	720	0.26	720	0.31	720	0.36	720	0.40	720	0.43
1440	0.13	1440	0.18	1440	0.21	1440	0.25	1440	0.27	1440	0.29

Figure 5.5 – Zone 1 IDF Curves



CHAPTER 6: RUNOFF ANALYSIS

Runoff Analysis

Using the sub-basin parameters developed in Chapter 4 and the design storm developed in Chapter 5, design flows were determined for each sub-basin and at key node points.

The resulting flows are presented in this chapter as Figure 6.1 and Figure 6.2 and were used in Chapter 7 Identification of Problem Areas and Proposed Solutions to analyze the existing and future hydraulic requirements of the City's storm system.

Sub-Basin Flows

Individual sub-basin flows are presented in Figure 6.1, Peak Sub-basin Flows. These flows represent the runoff due to the sub-basin areas delineated in Figure 4.1 based on the sub-basin parameters given in Figure 4.7. Flows due to a portion of the sub-basin were estimated by assuming a one-to-one relationship between contributing area and peak flow. For example, for master planning purposes, it was assumed that a point in the sub-basin receiving runoff from 50% of the area would have a peak runoff rate of 50% of the rate calculated for the entire basin.

The sub-basin flows shown in Figure 6.1 show that in sub-basins where it was assumed that private residential dry wells will be used for all future development, future flows are equal to or slightly less than the calculated existing flows. In these areas future sub-basin area contribute less runoff than under existing conditions because new impervious areas will drain to drywells. In addition, existing flows were calculated assuming that after initial infiltration losses, the entire sub-basin area contributes to runoff at the sub-basin outlet. Local ponding in undeveloped low areas throughout the sub-basin was not accounted for in the calculations. Consequently, observed existing flows are expected to be significantly less than calculated existing flows. Since under future conditions, much less local ponding is expected, the estimated future flows should adequately approximate future flows. Existing flows, however, should be regarded as potential flows before full buildout, but after upstream restrictions have been removed or local ponding areas have been filled.

The correct flow to use in design of future improvements is the larger of the existing or future calculated flow. This represents the most conservative case, before drywells have been implemented throughout the entire basin, but after upstream improvements and some development have channeled a majority of the water to the basin outlet.

Peak Node Flows at Key Points

In Figure 4.1 several key points are indicated by a circled number and a large area. Flows due to all upstream sub-basins were computed at these points, unlike flows from a single sub-basin as presented in Figure 6.1. These node flows are the actual flow that should be used for design of a system when the combined flow from more than one sub-basin must be conveyed. Flows corresponding to these key node points are presented in Figure 6.2, Peak Node Flows at Key Points.

Figure 6.1 – Peak Sub-Basin Flows (CFS)

	Existing Flows						Future Flows					
SUB-BASIN	2yr	5yr	10yr	25yr	50yr	100yr	2yr	5yr	10yr	25yr	50yr	100yr
Ocean Rd.												
B-1A	1.2	1.9	2.3	3.2	3.6	3.9	1.3	2.1	2.5	3.4	3.8	4.2
B-1B	1.7	2.9	3.5	4.9	5.6	6.2	2.0	3.3	3.9	5.3	6.1	6.6
B-2	5.7	8.7	10.3	13.5	15.1	16.4	5.8	9.0	10.6	13.9	15.6	16.9
B-3	9.0	14.7	17.7	24.0	27.3	28.9	9.6	15.6	18.8	25.4	28.8	31.6
B-4	2.3	4.0	4.8	6.6	7.6	8.3	2.9	4.6	5.6	7.5	8.5	9.3
Golf Course												
GC-1A	2.7	4.6	5.7	7.9	9.1	9.9	2.8	4.8	5.9	8.3	9.5	10.4
GC-1B	9.2	16.0	19.5	27.3	31.3	34.5	10.8	18.2	22.2	30.6	34.9	38.4
GC-1C	8.9	13.3	15.5	20.0	22.3	24.1	11.2	16.2	18.7	23.7	26.3	28.3
GC-1D	8.6	13.2	15.5	20.4	22.8	24.8	9.6	14.7	17.3	22.6	25.3	27.4
Golf Course Lake												
L-1	3.6	6.6	8.3	11.8	13.7	15.2	3.6	6.6	8.3	11.8	13.7	15.2
L-2	4.6	8.8	11.1	16.1	18.6	20.7	4.6	8.8	11.1	16.1	18.6	20.7
Necarney City												
N-1A	1.9	3.1	3.8	5.3	6.0	6.6	2.1	3.5	4.2	5.7	6.5	7.1
N-1B	4.8	8.0	9.7	13.3	15.2	16.8	4.8	8.0	9.7	13.4	15.3	16.9
N-2	1.4	2.4	2.9	3.9	4.5	4.9	1.5	2.4	3.0	4.0	4.6	5.0
N-3	1.4	2.4	2.9	4.0	4.6	5.0	1.5	2.5	3.0	4.1	4.7	5.1
Neah-Kah-Nie Lake												
NL-1	3.6	6.5	8.1	11.4	13.2	14.6	5.1	8.3	10.0	13.6	15.4	16.9
Nehalem Bay												
NB-1	11.3	20.8	26.1	37.4	43.4	48.2	12.5	21.6	26.5	36.8	42.2	46.5
NB-2	3.6	6.4	8.0	11.3	13.0	14.4	4.5	7.6	9.3	12.8	14.5	16.0
NB-3	7.3	12.2	14.8	20.2	23.0	25.3	9.2	14.9	18.0	24.3	27.6	30.2
NB-4	13.9	23.9	29.4	40.8	46.7	51.5	16.9	27.5	33.0	44.5	50.4	55.2
North UGB												
NR-1	7.3	11.9	14.3	19.4	22.0	24.0	8.5	13.3	15.9	21.1	23.7	25.8

Figure 6.2 – Peak Flows at Key Node Points (CFS)

	Description of Drainage Area	Existing Upstream Runoff Area (ac)	Future Upstream Runoff Area (ac)	Existing Flows						Future Flows					
				2yr	5yr	10yr	25yr	50yr	100yr	2yr	5yr	10yr	25yr	50yr	100yr
1	36" Beach Outfall at Pacific Ave.	153	128	29	47	56	75	85	93	34	54	64	85	96	105
2	Golf Course Creek and 3rd St. systems at 3rd and Pacific Ave	133	111	27	42	51	68	76	83	32	49	58	77	86	94
3	3rd St. pipe south of Laneda Ave. connection	53	42	12	18	21	28	32	34	13	20	24	31	35	38
4	Inlet to 3rd St. system at Laneda Ave.	19	14	3	5	6	8	9	10	3	5	6	8	10	11
5	Division St. system	34	27	9	13	16	20	23	25	10	15	17	23	25	27
6	Flow at Laneda Ave. Beach Outfall	26	20	6	9	10	13	15	16	6	9	11	14	16	17
7	Flow at Manzanita Ave. Beach Outfall	21	16	4	6	7	9	11	11	4	6	7	10	11	12
8	Flow at Treasure Cove Beach Outfall	20	16	3	5	6	8	9	10	3	5	6	9	10	11
9	Flow at Sitka Lane Beach Outfall	54	44	7	11	14	19	21	23	7	11	14	19	22	24
10	Flow at Washington St. Beach Outfall	41	32	7	11	14	19	21	23	8	12	15	20	23	25
11	Flow at N. Basin Beach Outfall	9	7	2	3	4	5	5	6	2	3	4	5	6	6
12	Flow at Classic St. & Dorcas Lane Intersection	37	31	10	15	17	23	25	28	12	18	21	27	30	32

CHAPTER 7: IDENTIFICATION OF PROBLEM AREAS AND PROPOSED SOLUTIONS

Identification of Problem Areas and Proposed Solutions

Using the methodology discussed in Chapter 3, the planning information compiled in Chapter 4 and flows determined in Chapter 6, undersized facilities and areas needing more storm drainage facilities to serve future development were determined. This chapter discusses existing and future drainage needs for each main drainage system within the UGB. Separation of improvements into project groups, costs for each project and phasing are discussed in Chapter 8.

General Discussion

The problem areas have seen some change. While Division Street drainage to the Golf Course is still unresolved, it is listed as Project 1.

In October 2020, Otak met with Manzanita Public Works to review the priority areas within the Manzanita UGB that were presenting significant issues to the City of Manzanita. The main issues common to the problem areas are stormwater ponding and non-channelized flow. While there may be other issues with the storm drainage for the City, ten (10) projects and their priority are listed below.

In addition to the ten street drainages, the beach outfalls listed in the 2005 SWMP are still in use, however, maintenance could be done on all of them. State of Oregon State Parks would necessarily be involved in this maintenance. The Sitka Lane, Pacific Lane and Treasure Cove outfalls were discussed with Jay Sennewald of State Parks, May 10, 2020, during a walk-through tour.

Lastly, there are stormwater outfalls that occur naturally onto the beach. Five (5) natural outfalls on the north end of Manzanita were noted. These areas could benefit the City by “channelizing” the specific drainage, confining it to a known area where it could be managed. There may also be the possibility that two or more of these five areas could be combined. State Parks should be consulted if enhanced use or combining of these outfalls is desired by the City.

The ten areas that are problematic are discussed within the parameters of 1) immediate necessity to remove stormwater, 2) safety of residents, 3) street drainage components necessary to control flow and, 4) the priority set by the City.

The areas and their priority are:

1. Division Street Drainage
2. Sitka Street Drainage
3. Lakeview Drive Drainage
4. Pine Street Drainage
5. Hallie Lane Drainage
6. Division Street (North) Drainage
7. North Avenue Drainage
8. Greenridge Street Drainage
9. Cherry Street Drainage
10. Manzanita Avenue Drainage

These 10 CIP Priorities are presented on Figure 7.1. On the following pages, each problem and potential solution are discussed.

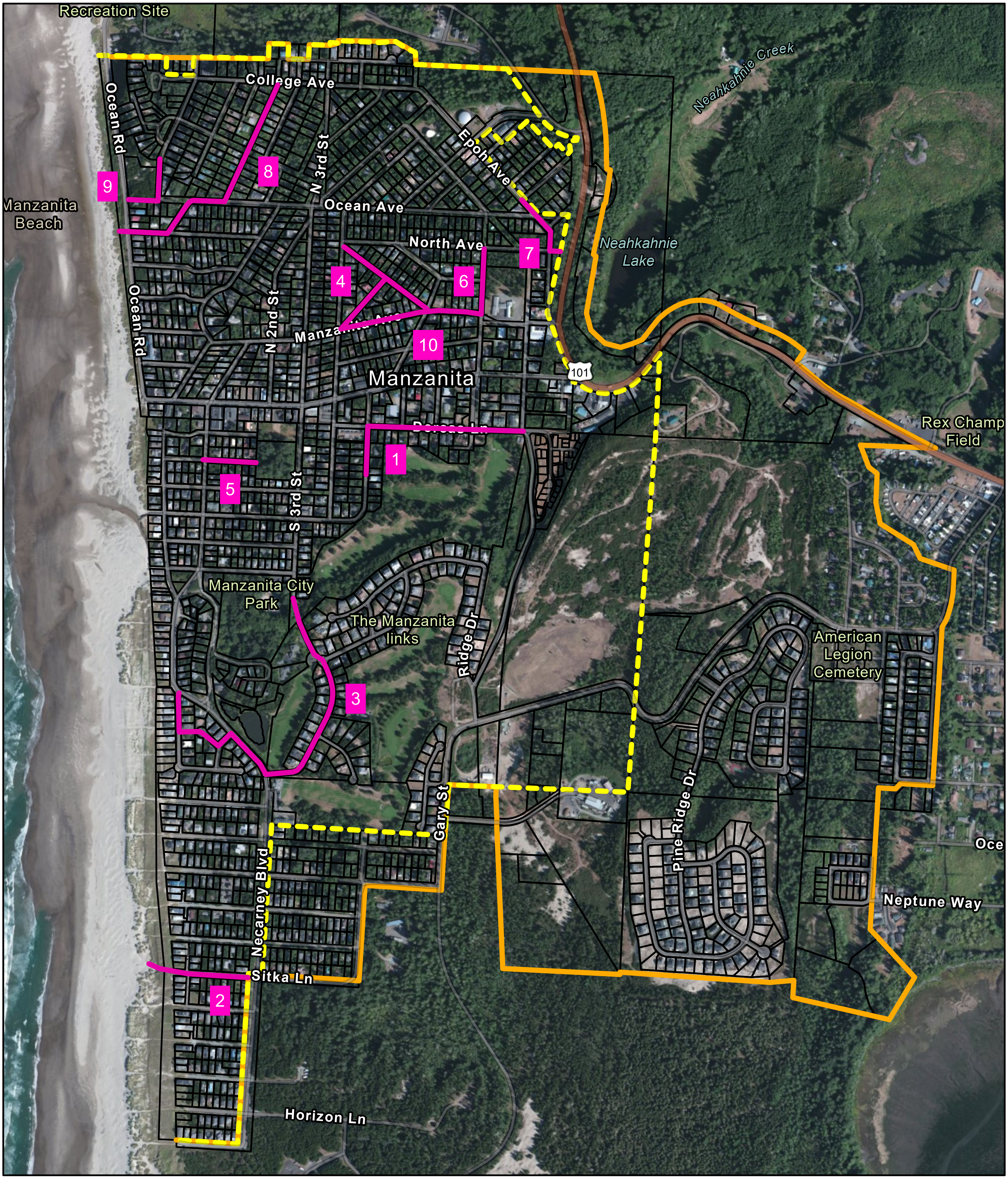


FIGURE 7.1

CIP PRIORITIES

CITY OF MANZANITA
STORM WATER MASTER PLAN

MANZANITA | OREGON

Legend

- Manzanita City Limits
- Manzanita UGB
- CIP Lines

FEET
0 500 1,000

Otak

City of Manzanita

Division Street Drainage – CIP Project #1

Problem Areas

The issues with this drainage were discussed in the 2005 SWMP update. They continue today. Water flows down the ROW on Division Street and onto the Golf Course. This water eventually makes it into the Golf Course Creek and then into the Pacific Lane ocean outfall.

The Golf Course and the City are now motivated to capture these flows and channelized them into the Manzanita stormwater system.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled sheet flows on Division are creating issues with the homeowners and the Golf Course. These water issues have not been resolved and appear to be worsening. Left unchecked or unchanneled by a municipal system, property damage is likely to occur. Discussions with the State have taken place to see what remedies may be available.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the strong velocity of uncontrolled runoff.

Parameter 3 – Components Necessary to Control Flows

A preliminary investigation of the area shows that the flows may be channeled into the City's storm drainage system via the addition of storm drain catch basins, manholes, pipe and a concrete valley gutter to channel overland sheet flow.

Parameter 4 – City Priority

The City has set this as the number one priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority. The west end of Dorcas Lane may be connected into the north-south drainage on South 3rd Street. The flows would then go to the Pacific Lane drainage and outfall.

Solution Perspective 1 – Estimated Components

To channelize this overland sheet flow during storm events, it is necessary to address the flows in two ways. The first way is to capture the area flows with catch basins and piping, then put the larger flows into the City stormwater system of storm pipes and manholes. The second way is to reshape the road base and add concrete curb and valley gutter to control individual site runoff such that it flows into the storm drain system.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP. A more in-depth estimate was developed for the City as the Dorcas & 4th Storm Drainage Project that was presented to City Council in October 2020.

Solution Perspective 3 – City Priority

Division Street Drainage is a large area of settling ponds and drainage ditches. To keep stormwater from causing private property damage and street flooding Public Works must clean approximately 2500' feet of storm ditches per year along with three settling ponds located in the 5th Street area. The area of greatest concern is west of S. 5th Pl. The past several years' storm flows have overwhelmed ditches and ponds in the area causing flooding of ditches on S. 4th Pl. and debris washout. Homeowners in the area have expressed frustration and concern over the standing water crossing onto their property in the area of S. 5th Street. Most stormwater in this area is generated from the ROW and City properties.

During recent surveying it also was discovered that City infrastructure was constructed on two different private properties.

City installed a storm outfall onto the Classic Street and Dorcus intersection. This outfall is located on City property. The stormwater runs off city property on to private land.

Sitka Street Drainage – CIP Project #2

Problem Areas

This is an extremely low area, with an Ocean Outfall cut into the western edge of the platted ROW. The outfall drainage piping stops just prior to crossing the N-S vegetation line. Should the City desire to extend the outfall piping west of the N-S vegetation line, it will require input from Oregon State Parks and possibly permits. The ditching silts up and needs cleaning, and the road elevation should be brought up.

From the 2005 Storm Drainage Master Plan Update:

"The 24-inch [high density polyethylene] HDPE outfall to the beach at the west end of Sitka Lane occasionally becomes blocked due to sand movement and large debris from the ocean as shown below. The combination of the low elevation of the outfall pipe, the lack of outfall protection and high tides cause it to be submerged throughout most of the year. The blockage of this outfall cripples the entire Necarney Boulevard collection system and causes flooding throughout Necarney Boulevard" p. 54.

Parameter 1 – Necessity to Remove Stormwater

Severe ponding from heavy storm events and King tides make travel hazardous, if not impossible, for short periods of time.

Parameter 2 – Safety of Residents

While ponding may not affect large, high emergency vehicles, ponding may inhibit property owners and City maintenance vehicles from passage.

Parameter 3 – Components Necessary to Control Flows

From the 2005 Storm Drainage Master Plan Update:

"Topography in this area reduces available options for a storm drainage collection and conveyance system. The low areas run parallel with the beach/dunes and hinder the use of a collection system along the roadways. Collection systems along the roadways would require installing beach discharge pipes at each dead-end road. Pipes that discharge onto the beach, such as the Sitka Lane discharge pipe, are problematic with high maintenance and should be avoided if possible. Therefore, the City must look to utilize the existing discharge pipe already established. The Nehalem Bay Wastewater Agency (NBWA) utilizes a utility easement running from Spyglass Lane to as far north as Chinook Lane. In order to construct a collection system that uses the Sitka Lane discharge, the City would need to utilize this utility easement as well. Installing a collector pipe that parallels the existing sanitary sewer main will help alleviate the existing ponding and help with future drainage issues once these roads become paved" p. 55.

Parameter 4 – City Priority

The City has set this as the second highest priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated. Otak has met with Jay Sennewald, Oregon State Parks Permit Specialist to look at the issues with this drainage. During this visit, it was obvious that the drainage was partially blocked by silts and ocean debris.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

From the 2005 Storm Drainage Master Plan Update:

Problems with the Sitka Lane system are most likely due to siltation of the culverts, rather than poor design. Maintenance of existing culvert entrances, use of rip-rap at the inlet of the 24-inch HDPE appears to be in suitable condition and therefore, replacement was not included in the proposed capital improvements. Rip-rap at the outfall and a trash rack should, however, be installed at the west end of the Sitka Lane discharge pipe to prevent it from becoming buried and submerged. Regular maintenance of the trash rack will be required after high tides and excessive rainfall" p. 55.

As highlighted in the 2005 SWMP and discussions with Manzanita Public Works Department, there is a recognized need for a larger pipe and manholes, possibly as large as 36-inch HDPE. This pipe would run in the NBWA utility easement, from Spyglass in the south to as far north as Chinook Lane. Further, a trash rack should be installed to protect the outfall from ocean debris. A storm drain manhole should also be installed at Necarney Boulevard. Further engineering analysis is required here.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

The Sitka outfall pipe in its current state must be monitored multiple times a day and night during rain events. Currently there is a small surge ditch/pond on the inlet of the beach outfall. This 24-inch pipe can have 8 to 10 inches covering the pipe while on the beach side less than half of the pipe is flowing water. This is believed to be caused by a high point in the pipe. Area must be cleared of brush and cleaned out every 5 years. Failure of this outfall would cause flooding to neighborhoods.


24" OUTFALL PIPE SILTS - OCEAN DEBRIS
24" HDPE PIPE
SITKA STREET DRAINAGE
VIEW: LOOKING WEST AT SITKA BEACH OUTFALL
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA STORM WATER MASTER PLAN - PROJECT 2
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

Lakeview Drive Drainage – CIP Project #3

Problem Areas

Lakeview Drive is located adjacent to the Golf Course and consists of residential development with large impervious areas and steep sloping roadways and driveways. This subdivision around the golf course was constructed with little-to-no storm drainage catchment or conveyance systems, which produces large amounts of sheet flow, developing into channelization along the edges of pavement. Erosion and large ruts have plagued this area and have increased in recent years due to development reaching maximum buildout.

The two legs or reaches of this drainage system are from Lakeview Drive east to 3rd Avenue, and the second leg is from the intersection shown, proceeding northwest to the newly constructed Carmel Avenue system. It may be possible with a favorable topographic analysis to drain north by gravity.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled flows are creating issues with the homeowners and the Golf Course. These issues have not been resolved and appear to be getting worse. Left unchecked or unchanneled by a municipal system, property damage is likely to occur.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the strong velocity of uncontrolled water runoff.

Parameter 3 – Components Necessary to Control Flows

Preliminary examination of the elevations indicates that approximately five manholes and 2,500 feet of 36-inch HDPE pipe will be required. A new outfall and outfall protection by rip-rap will be required. This outfall may not be extended west of the vegetation line determined by Oregon State Parks.

Parameter 4 – City Priority

The City has set this as the number three priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

This would be a new storm drain alignment and outfall for the City. In addition to the 2,500 feet of pipe and manholes, trenching along Lakeview, Necarney and Chinook may require trench stabilization and a significant amount of rock.

Due to the size of outfall pipe required here, the outfall onto the beach should be covered by a trash rack to prevent storm debris from blocking the outfall. Further, this would enhance the safety of families using the beach.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Lakeview drive drainage area has few ditches and drain structures. These structures must be inspected often during the rainy season. Large rain events cause wash-out of road edges and plugging of storm drains in the area. Flooding occurs on Beach Pine Dr. This was a street project that collected stormwater with no outfall to move water out of the area. Waters then collect and flood over the properties along the south side of the street.

Pine Street Drainage – CIP Project #4

Problem Areas

As is evidenced by the project photo, heavy flows along Pine Avenue create ponding. This ponding can damage not only the road surface, but also the subgrade, making future repairs costlier, as the design engineer adds aggregate to stiffen the subgrade. On some streets such as this, there may also be an opportunity to grind the existing asphalt material and re-lay and re-compact it.

The alignment for this drainage proceeds south along N Cedar Street. The unchannelized flows here have the potential for nuisance property damage.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled flows along Pine Street are creating difficulties for the homeowners and hazardous travel through the ponding in heavy rain events.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the ponding and water runoff.

Parameter 3 – Components Necessary to Control Flows

Preliminary investigation with the Public Works Department indicates that catch basin inlets and one manhole would be needed to collect the ponding into the new storm system at Manzanita Avenue.

In addition to catch basins, manholes, pipe and a concrete valley gutter would be added to channel overland sheet flow.

The addition of these components may also require re-paving.

Parameter 4 – City Priority

The City has set this as the number four priority to be resolved.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

A portion of the new storm drainage that was added along Manzanita Avenue and extended north along Cedar could be continued here. The continuation would be north along Cedar, and then east and west along Pine. The storm drain would alleviate the ponding along Pine Avenue and allow flows to be channelized into the Manzanita Avenue storm drain system.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Pine Street and Cedar Street have large standing water puddles that form and cause gravel to wash out. City staff fill in potholes several times a year.


FLOW
(DOWN N CEDAR ST)
PONDING/SHEET FLOW ISSUES
PINE STREET DRAINAGE
VIEW: LOOKING NORTHWEST ON PINE AVENUE
DATE: NOVEMBEER 17, 2020
NUMBER: MANZANITA STORM WATER MASTER PLAN - PROJECT 4
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

Hallie Lane Drainage – CIP Project #5

Problem Areas

The localized ponding is creating water hazards for the property owners and visitors along Hallie Lane. To alleviate this issue, the area should have catch basins and new pipe installed. The storm drain system along Carmel Ave has recently been improved, with a stub-out for any future Hallie Lane improvements.

The pieces of existing asphalt pavement are in a very deteriorated condition. This deteriorated condition may allow water to pool underground and may create hidden infrastructure damage to the subgrade underneath Hallie Lane.

Future heavy equipment that may be used to develop the lots on the east end of Hallie Lane may create further damage to the road surface. As an alternative to routing heavy equipment on N 3rd Street, sub-grade testing could be done by a Geotechnical firm and the proper subgrade specifications developed if the Hallie Drainage Project precedes the development of the empty lots. The heavy equipment should approach the undeveloped area via 3rd Street, if the repairs to Hallie Lane are not done prior to the development.

Parameter 1 – Necessity to Remove Stormwater

Drain standing water from Hallie Lane.

Parameter 2 – Safety of Residents

Removes standing water, via catch basins connected to existing storm drain manholes.

Parameter 3 – Components Necessary to Control Flows

This improvement will require double catch basins at the east end, connected to the recent Carmel Ave storm drain improvements manhole stub-out. The pipe should be approximately 375 feet in length, and 8-inch to 12-inch in diameter. Overland flows should be controlled by a valley gutter on both sides.

Due to the condition of the asphalt and the installation of the new piping system, Hallie Lane should be re-paved.

Parameter 4 – City Priority

The City has set this as the number five priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

This improvement will require double catch basins at the east end, connected to the recent Carmel Ave storm drain improvements manhole stub-out. The pipe should be 8-inch to 12-inch. Overland flows should be controlled by a valley gutter on both sides.

Note that this storm drainage should have a stub-out on the catch basins to the east, should the property to the east be developed.

Due to the condition of the asphalt and the installation of the new piping system, Hallie Lane should be re-paved.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Hallie Lane Drainage has two parts. Hallie west of S. Carmel has large standing water puddles that form and cause gravel to wash out. City staff fill in potholes several times a year.

Hallie Ln. East of S. Carmel has a failing drainage pipe. The pipe inlet at the east end of Hallie is monitored multiple times a day and night during large rain events. Staff has had to run pumps in the area to avoid damage to homes.

Division Street (North) Drainage – CIP Project #6

Problem Areas

As evidenced by the attached project photo, due to the height of the road and the lower elevation to the east, water sheet flows onto the properties to the east. No visible water is standing, even in stormy conditions.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled sheet flows on Division, left unchecked may cause property damage and a nuisance to property owners on the east side of Division.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the volume of uncontrolled water runoff.

Parameter 3 – Components Necessary to Control Flows

Valley curb and gutter should be installed on the east side of Division to control the sheet flows. A catch basin should be installed on the north end, near the intersection of Division and North Avenue, on the east side of Division.

Approximately 550 feet of 12-inch HDPE piping would connect the catch basin to the existing system in Manzanita Avenue. This would be covered by a 4-foot trench patch of asphalt.

Parameter 4 – City Priority

The City has set this as the sixth priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

Valley curb and gutter should be installed on the east side of Division to control the sheet flows. A catch basin should be installed on the north end, near the intersection of Division and North Avenue, on the east side of Division.

Approximately 550 feet of 12-inch HDPE piping would connect the catch basin to the existing system in Manzanita Avenue. This would be covered by a 4-foot trench patch of asphalt.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Division Street drainage has no current ditches or infrastructure. Roadsides are gravel that require constant maintenance and erosion control. Roadsides wash out and must be rebuilt several times a year.


SHEET FLOW ISSUES
DIV ISION STREET (NORTH) DRAINAGE
VIEW: LOOKING SOUTH DOWN DIVISITON ST
DATE: NOVEMBER 17, 2020
NUMBER MANZANITA STORM WATER MASTER PLAN - PROJECT 6
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

North Avenue & Epoh Avenue Drainage – CIP Project #7

Problem Areas

This is a very short drainage from 19th Street, down to North Avenue, and then east into the woods to the east ROW of Epoh Avenue. The catch basin lid at the southwest corner of the intersection of Epoh Avenue and North Avenue gets covered by organic debris and then the water flows onto the property just south of Epoh. Epoh is very steep (+/- 15% slope) with sheet flow, channelized flow and erosion along the south edge of pavement. This uncontrolled flow is evident in the project photo, attached.

Parameter 1 – Necessity to Remove Stormwater

While lives are not in danger, properties are, from the volume of uncontrolled water runoff.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the strong velocity of uncontrolled water runoff.

Parameter 3 – Components Necessary to Control Flows

Water should be directed into an underground piping system. Further, a curb and valley gutter should be placed on the south edge of Epoh Avenue pavement. A larger manhole or catch basin with a larger suburban frame and cover should replace the existing catch basin at the intersection. The length of this project is approximately 550 feet of pipe, manholes and catch basins, prior to the pipe crossing Epoh Avenue, and directing the water into the east ROW.

The asphalt trench patch crossing Epoh Avenue should also have a small asphalt water bar around the south edge of the manhole rim. This would direct water into the manhole and away from the property just south.

Parameter 4 – City priority

The City has set this as the seventh priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

A larger manhole or catch basin with a larger suburban frame and cover should replace the existing catch basin. Approximately 100 feet of 12-inch HDPE should replace the existing pipe crossing Epoh Avenue.

The asphalt trench patch crossing Epoh Avenue should also have a small asphalt water bar around the south edge of the manhole rim. This would direct water into the manhole and away from the property just south.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

North Avenue and Epoh Ave Drainage area has very steep road grades that causes road edge washouts. This requires constant monitoring of a single drain located at the base of the road. This drain is overwhelmed quickly and causes flooding onto private property. Road edges must be cleaned and maintained often, and erosion control bags cleaned and replaced.


DRAINAGE ISSUES - EROSION
NORTH AVE & EPOH DRAINAGE
VIEW: LOOKING EAST ON EPOH AVE
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA STORM WATER MASTER PLAN - PROJECT 7
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

Greenridge Street Drainage – CIP Project #8

Problem Areas

The steep grade south on Greenridge accelerates the water as it flows toward Ocean Avenue. The flow is uncontrolled, either by catch basins and piping or curbing and valley gutters. This flow causes erosion south of Ocean Avenue and should be controlled. The photograph shows an approximate 8% grade toward Ocean Avenue.

Several of the local residents have used sandbags in an attempt to control flows down their driveways.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled sheet flows on Greenridge Street are creating issues with the homeowners and the topography south of Ocean. The water, when the catch basin is overwhelmed, flows between house number 136 and 128, as shown in photos. These water issues have not been resolved and appear to be getting worse. Left unchecked or unchanneled by a municipal system, private property and City property damage is likely to occur.

Parameter 2 – Safety of Residents

While lives are not in danger, properties are, from the strong velocity of uncontrolled water runoff. In fact, even some of the downstream structures may be affected by uncontrolled water flows.

Parameter 3 – Components Necessary to Control Flows

An underground, piped system, to the west, along Ocean Avenue, southwest along Surf Lane, then out to the County Road, combined with a curb and valley gutter should provide the necessary runoff protection in heavy storm events. A trash rack should be included due to pipe size outfall protection.

In addition to the underground and surface system along Greenridge, consideration should be given to placing water bars along the east side driveways in addition to the curb and gutter.

These flows could then be channeled south, then southwest along Surf Lane and finally west along North Avenue to a new outfall, which would drain into the large swale just west of Ocean Road. A careful study of the area must be made during engineering to evaluate the location of the N-S vegetation line, so that the drainage occurs on City property.

This area drainage improvement would also involve catch basins, manholes and an asphalt trench patch.

Parameter 4 – City Priority

The City has set this as the eighth priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

To channelize this overland sheet flow during storm events, it is necessary to address the flows in two ways. The first way is to capture the area flows with catch basins and piping, then channelize the larger flows toward an outfall near Ocean Road, via a system of storm pipes and manholes. The second way is to add curbs and valley gutters to control initial sheet flow. Greenridge Street should be trench patched after the installation of the piping system and the curb and valley gutter. Water bars should be considered in steep areas.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Greenridge Street is a narrow paved road with little to no road edges. Water flows down the steep grade causing stormwater runoff damage to road edges and to private property. Staff has to repair road edges and place erosion control bags in the area on heavy rain events.


136 OCEAN AVE
128 OCEAN AVE
SHEET FLOW PATH WHEN
CATCH BASIN OVERFLOWS
GREENRIDGE ST DRAINAGE PHOTO B
(OF A B)
VIEW: LOOKING SOUTH FROM OCEAN AVE/GREENRIDGE ST INTERSECTION
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA STORM WATER MASTER PLAN - PROJECT 8
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

Cherry Street Drainage – CIP Project #9

Problem Areas

The heavy flows are creating ponding issues and heavy slope erosion immediately west of Cherry Street. These flows appear to have destabilized the slope, with heavy erosion evident, as shown in photos.

Parameter 1 – Necessity to Remove Stormwater

The uncontrolled flows to the south on Cherry Street are creating issues with the homeowners. There is heavy ponding near the address of 730, due to a dip in the roadway that collects water. These water issues have not been resolved and appear to be getting worse. Left unchecked or unchanneled by a municipal system, property damage due to destabilized dunes is likely to occur.

Parameter 2 – Safety of Residents

While lives are not in danger, downstream properties are, from the strong velocity of uncontrolled water runoff. People may not be able to access their homes for short periods of time due to the depth of the ponding, particularly at 730 Cherry Street.

Parameter 3 – Components Necessary to Control Flows

From interviews with the Public Works Department, the road immediately adjacent to 730 Cherry Street should be raised approximately 2.0 feet. A piping system with catch basins should be dug in to move the water in this area, in a controlled way to an Ocean Avenue.

Further, the area along Cherry Street should be protected with a curb and valley gutter on the west side. A manhole would be placed at the south end of Cherry, in the half-street.

Then, as the piping system actually goes down the steep grade, with dune stabilization underway, the piping system could be installed by boring, and go under the County Road (Ocean Road). A crossing at Ocean Road would need to be constructed, and the outfall protected by rip-rap and a trash rack.

After the piping system and the curb and valley gutter are placed, Cherry Street should be re-paved, in part where the roadway was elevated, and a trench patch placed where the piping and curb and valley gutter was placed.

Parameter 4 – City Priority

The City has set this as the ninth priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

From interviews with the Public Works Department, the road immediately adjacent to 730 Cherry Street should be raised approximately 2.0 feet. A piping system with catch basins should be dug in to move the water in this area in a controlled way to an Ocean Avenue.

Further, the area along Cherry Street should be protected with a curb and valley gutter on the west side.

As the piping system actually goes down the steep grade, the piping system should be anchored to the slope.

After the piping system and the curb and valley gutter are placed, Cherry Street should be re-paved, in part where the roadway was elevated, and a trench patch placed where the piping and curb and valley gutter was placed.

A crossing at Ocean Road would need to be constructed, and the outfall protected.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

The Cherry Street drainage area has standing water and water runoff to the west onto private property and a narrow ROW. Runoff has moved multiple yards of sand onto Ocean Ave. Staff is having to monitor the area closely as property is developed.


SIGNIFICANT EROSION ISSUES OFF OF CHERRY ST
CHERRY STREET DRAINAGE PHOTO B
(OF A B C)
VIEW: LOOKING WEST FROM OVER THE BANK ON CHERRY STREET
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA STORM DRAIN WATER MASTER PLAN - PROJECT 9
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020


SIGNIFICANT EROSION CHANNEL TO WEST
CHERRY STREET DRAINAGE PHOTO C
(OF A B C)
VIEW: LOOKING WEST DOWN AN EROSION CHANNEL OFF CHERRY ST
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA STORM WATER MASTER PLAN - PROJECT 9
REPORT: MANZANITA STORM WATER MASTER PLAN UPDATE - 11/2020

Manzanita Avenue Drainage – CIP Project #10

Problem Areas

The heavy flows are creating flooding and ponding issues east of 5th Street. As evidenced by the attached photograph, unchannelized water is flowing down the south side of Manzanita Avenue. As the water flows west sheet flow affects properties to the south. As the water flows in this manner further west, toward N 4th Street, the unchannelized flow ponds approximately 300 feet east of N 4th Street, and then into private driveways to the south of Manzanita Avenue.

Further, if the ponding becomes significant, emergency and police vehicles may lose critical time in their operational response, in having to approach Manzanita Avenue from the opposite side.

Parameter 1 – Necessity to Remove Stormwater

To properly control the flows from the east, a storm drainage system should be put in on the south side of Manzanita Ave. This would remove water by an underground piping system below the road surface and a concrete valley curb at the road surface, on the south side of Manzanita Ave.

Parameter 2 – Safety of Residents

While lives are not in immediate danger, downstream properties along with potential emergency vehicle response times are.

Parameter 3 – Components Necessary to Control Flows

This project will require storm piping underground, manholes, catch basins, and the addition of curb and gutter along the south side of Manzanita Avenue. In addition, it may require basic landscape restoration and driveway apron repair or replacement.

Parameter 4 – City Priority

The City has set this as the tenth priority to be resolved. While there has been broad discussion of methods, the actual project with engineering and construction has not been initiated.

Potential Solution

Potential solutions are discussed at a high-level overview from the perspectives of estimated components, budget, and City priority.

Solution Perspective 1 – Estimated Components

This project will require storm piping underground, manholes, catch basins, and the addition of curb and gutter along the south side of Manzanita Avenue. In addition, it may require basic landscape restoration and driveway apron repair or replacement.

Solution Perspective 2 – Estimated Budget

A simple budget for the standard components and contractor expenses has been prepared. It is part of the SWMP.

Solution Perspective 3 – City Priority

Manzanita Avenue Drainage collects water runoff from many streets to the north of Manzanita Avenue and Manzanita Avenue itself. Water runs down both sides of Manzanita Avenue causing moderate edge erosion damage to gravel areas. Water collects at Manzanita and 4th Street where it enters into an improved storm system.


DOWNHILL EROSION & PONDING
MANZANITA AVENUE DRAINAGE
VIEW: LOOKING WEST ON MANZANITA AVENUE
DATE: NOVEMBER 17, 2020
NUMBER: MANZANITA AVENUE DRAINAGE - PROJECT 10
REPORT: MANZANITA STORMWATER MASTER PLAN UPDATE - 11/2020

CHAPTER 8: CAPITAL IMPROVEMENTS – COST, PHASING AND IMPLEMENTATION

Capital Improvements – Cost, Phasing and Implementation

This chapter discusses the actual implementation of the solutions presented in Chapter 7. First, a brief discussion of drainage design assumptions regarding pipe and collector trench construction is presented. Unit costs for pipe and collector trench systems are described. General solutions are separated into logical project groups and detailed cost estimates for each proposed project presented. Priorities are assigned to each project, based on cost, magnitude of problem, and expected impact and phasing of future development. A summary table of cost and proposed phasing is then presented in Figure 8.1.

Design Assumptions and Unit Costs

Proposed drainage improvements implement either pipe systems or rock filled collection trenches. The design assumptions and unit costs for each are discussed below.

Pipe Systems

In calculating the final capacities and determining unit costs, it was assumed that corrugated exterior, smooth interior, HDPE pipe (ADS SaniTite) shall be used, per the City's standards. HDPE is a high density, polyethylene pipe that has gained popularity for drainage improvements in recent years. It has a much longer life span and less friction than metal pipe. In comparison to concrete pipe, HDPE pipe is easier to install, more durable, has less friction and is typically less expensive. All HDPE pipes, fittings and connections specified in the capital improvements must be water-tight, with rubber gaskets, and no exceptions to this standard are allowed.

Ground slopes throughout most of the City are adequate for sufficient drainage and it was assumed that pipes were constructed at the average ground slope, maintaining minimum cover of two feet.

In place unit costs for piped systems are given in Figure 8.2. These costs include material, labor and mobilization costs. They are based on available previous bids and estimates of backfill, repaving, excavation and pipe material costs. Engineering and contingency costs are not included in the unit costs presented in Figure 8.2. Engineering and contingency costs are accounted for in the individual cost estimates presented in Section 8.2. At present, 21-inch HDPE pipe is not available. Because 21 inches is a standard diameter for most other kinds of pipe and may be available for HDPE pipe in the future, segments where a 21-inch pipe is sufficient are noted in the improvements. However, to reflect current availability, cost estimates assumed 24-inch pipe will be required instead of 21-inch pipe.

Rock Filled Collection Trenches

In steep, residential areas, a rock filled infiltration trench may be preferable to pipes or ditches. These trenches should be 2 feet deep, have a 2.5-foot bottom width and be filled with very coarse, very clean 3-inch to 4-inch railroad ballast. Permeable geotextile fabric will be required to prevent infiltrating of the drain rock with sand. About 6 inches of 3/4-inch gravel should be used on top of the fabric to form the roadway shoulder. The roadway shoulder will need to be graded at about 2 to 3% to channel water into the collection trench.

The advantage to such a trench is that it is very robust and maintenance free. The 3-inch to 4-inch rock is assumed to have a permeability of about 280,000 ft/day or 32 ft/sec and at reasonably steep slopes should be able to convey 0.5 to 1.5 cfs. For local drainage this will sufficiently convey the majority of the storms, though it is not adequate for the 2-year or larger flow. Flows exceeding the trench's capacity will be primarily conveyed in the roadway shoulder and are not expected to cause significant flooding.

In addition to limited capacity, such a trench does not collect water as well as a ditch system. The permeable liner, necessary to prevent infiltration of sand, has a permeability of 1200 ft/day or 0.013 ft/sec. This is two orders of magnitude greater than the existing soils, but one order of magnitude less than the overlying gravels. However, calculations show that this should be sufficient for a majority of the annual storms.

Proposed Projects and Costs

The drainage solutions shown in Figure 4.6 were separated into isolated project groups for planning purposes. For each project, cost estimates were developed. A plan view and cost sheet for each project is included in this section. Cost estimates include the approximate number of catch basins and manholes that will be needed in the final design. Exact locations of catch basins and manholes are beyond the scope of this study and should be determined during the final design of the system.

Figure 8.1 Proposed Capital Improvement Projects

Project Ranking	Project Name	Est. Project Length	Sub-Basin	Issue to Be Addressed	Storm System Tie-In	Description of Improvements	Estimated Cost
1	Division St south to Dorcas Ln Drainage	2,800 feet	GC-1B	Sheet flows, off of ROW flowing into the Golf Course	Tie-in at intersection of S 4th PL, to the existing collector.	Tight-line with drainage components, under road piping to remove overland flow onto Golf Course	$ 809,000
2	Sitka St Drainage	550 feet	N-1A	Serious localized ponding	Ocean outfall at Sitka	Maintenance pipe, outfall cleaning, add trash rack	$ 144,000
3	Lakeview Dr Drainage	2,500 feet	L1	Localized ponding, pulling storm flows off private property. The exact route not set - requires more study	Unknown at this time - perhaps a new ocean outfall on west end of Chinook Lane	Tight-line with drainage components, under road piping, includes new trash rack	$ 730,000
4	Pine St Drainage	1,130 feet	GC-1D	Heavy flows to intersection of Pine and N Cedar, localized ponding	Tie into the existing storm system on Manzanita Ave	Tight-line drainage components, under road piping, new curbing as valley gutter, repave asphalt	$ 389,000
5	Hallie Ln Drainage	375 feet	B-1B to B-1A	Localized ponding, additional storm drainage for east	Tie into recent Carmel Avenue improvements	Tight-line drainage components, new piping and catch basins, under road piping	$ 155,000

Project Ranking	Project Name	Est. Project Length	Sub-Basin	Issue to Be Addressed	Storm System Tie-In	Description of Improvements	Estimated Cost
6	Division St (North) Drainage	650 feet	GC-1C	Heavy erosion at edges of asphalt	Tie in on Manzanita Ave - existing system	Add drainage components, piping, within east ROW	$ 217,000
7	North Ave and Epoh Ave	100 feet	GC-1C	Localized flooding, ponding, catch basin lid plugs easily	Epoh Ave east ROW	Add drainage components, piping, trench patch repave	$ 150,000
8	Greenridge St Drainage	1,775 feet	B3 to B4	Heavy erosion on east side of Greenridge Road and properties south of Ocean Avenue	Possible construction of new outfall into swale just west of Ocean Road.	Add drainage components, piping, pipe anchors, trash rack, trench patch repave	$ 536,000
9	Cherry St Drainage	500 feet	B4	Heavy erosion on west side of Cherry Street	Possible construction of new outfall into swale just west of Ocean Road - HDD boring steep slope.	Add road leveling, drainage components, under road piping, trash rack, repave with concrete valley curb	$ 271,000
10	Manzanita Ave Drainage	2000 feet	GC-1D	Localized flooding, ponding	Tie in on Manzanita Ave - existing system	Tight-line drainage components, new piping and catch basins, repave with concrete valley curb	$ 347,000

Figure 8.2, Projects No. 1-10 Costs (figure spans multiple pages)

Project 1:Dorcas Lane Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 17,500	$ 17,500
2	EROSION CONTROL, TEMP STORM AND DEWATERING	LS	1	$ 10,400	$ 10,400
3	CONNECTION #1 - 18" CONNECTION (AT LANEDA)	LS	1	$ 3,000	$ 3,000
4	CONNECTION #2 - 18" CONNECTION (AT 4th PL)	LS	1	$ 3,000	$ 3,000
5	CONNECTION #3 - 18" CONNECTION (AT N 4th ST)	LS	1	$ 3,000	$ 3,000
6	F&I 18" STORM PIPE, TESTING, COMPLETE	LF	2800	$ 90	$ 252,000
7	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	10	$ 8,000	$ 80,000
8	F&I CATCH BASINS - NYLOPLAST	EA	19	$ 2,200	$ 41,800
9	3/4" SHOULDER ROCK, COMPACTED IN-PLACE	CY	100	$ 50	$ 5,000
10	CURB & VALLEY GUTTER, IN-PLACE, COMPLETE	LF	1000	$ 35	$ 35,000
11	AC, SAWCUT, DEMO, WASTEHAUL, PAVING	TONS	500	$ 200	$ 100,000
12	PIPE OUTFALL PROTECTION	EA	0	$ 2,500	$ -
13	F&I LANDSCAPE RESTORATION	LS	1	$ 6,250	$ 6,250
				Rounded Subtotal =>	$ 557,000
	Project Contingency @	20%			$ 112,000
	Survey, Engineering, Permitting & Administration @	25%			$ 140,000
	Project Estimate with Contingency				$ 809,000

Project 2: Sitka Lane Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 5,000	$ 5,000
2	EROSION CONTROL, MONITORING, COMPLETE	LS	1	$ 6,900	$ 6,900
3	CONNECTION #1 - NEW STORM PIPE	LS	1	$ 3,000	$ 3,000
4	CONNECTION #1 - NEW STORM PIPE TO EXISTING OUTFALL	LS	1	$ 3,000	$ 3,000
5	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	550	$ 100	$ 55,000
6	F&I 48" STORMWATER MANHOLE, TESTING, COMPLETE	EA	2	$ 3,800	$ 7,600
7	F&I CATCH BASINS - NYLOPLAST	EA	0	$ 2,200	$ -
8	3/4" SHOULDER ROCK	CY	50	$ 50	$ 2,500
9	CURB & VALLEY GUTTER, COMPLETE	LF	0	$ 35	$ -
10	AC, SAWCUT, DEMO, WASTEHAUL, PAVING	TONS	0	$ 200	$ -
11	F&I OUTFALL PROTECTION, CL-50 RIP/RAP	LS	1	$ 4,000	$ 4,000
12	F&I LANDSCAPE RESTORATION & TRASH RACK	LS	1	$ 11,250	$ 11,250
				Rounded Subtotal =>	$ 99,000
	Project Contingency @	20%			$ 20,000
	Survey, Engineering, Permitting & Administration @	25%			$ 25,000
	Project Estimate with Contingency				$ 144,000

Project 3: Lakeview Drive Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 20,000	$ 20,000
2	EROSION CONTROL, MONITORING, COMPLETE	LS	1	$ 5,200	$ 5,200
3	CONNECTION #1 - 18" CONNECTION (AT SOUTH THIRD)	LS	1	$ 3,000	$ 3,000
4	F&I 18" STORM PIPE, TESTING, COMPLETE	LF	2500	$ 90	$ 225,000
5	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	9	$ 8,000	$ 72,000
6	F&I CATCH BASINS - NYLOPLAST	EA	25	$ 2,200	$ 55,000
7	3/4" SHOULDER ROCK	CY	200	$ 50	$ 10,000
8	CURB & VALLEY GUTTER	LF	2000	$ 35	$ 70,000
9	AC, SAWCUT, DEMO, WASTEHAUL, PAVING	TONS	100	$ 200	$ 20,000
10	F&I OUTFALL PROTECTION, CL-50 RIP/RAP	LS	1	$ 2,500	$ 2,500
11	F&I LANDSCAPE RESTORATION	LS	1	$ 20,000	$ 20,000
		Rounded Subtotal =>			$ 503,000
	Project Contingency @	20%			$ 101,000
	Survey, Engineering , Permitting & Administration @	25%			$ 126,000
	Project Estimate with Contingency				$ 730,000

Project 4: Pine Ave and Cedar Street Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 11,250	$ 11,250
2	EROSION CONTROL, TEMP STORM AND DEWATERING	LS	1	$ 6,900	$ 6,900
3	CONNECTION #1 - MANZ AVE	LS	1	$ 2,000	$ 2,000
4	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	1130	$ 80	$ 90,400
5	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	4	$ 8,000	$ 32,000
6	F&I CATCH BASINS - NYLOPLAST	EA	12	$ 2,200	$ 26,400
7	3/4" SHOULDER ROCK	CY	200	$ 50	$ 10,000
8	CONCRETE CURB - VALLEY GUTTER (PINE AND N CEDAR)	LF	1130	$ 35	$ 39,550
9	AC REPAVE	TONS	235	$ 200	$ 47,000
10	F&I OUTFALL PROTECTION, CL-50 RIP/RAP	LS	0	$ 2,000	$ -
11	F&I LANDSCAPE RESTORATION	LS	1	$ 2,500	$ 2,500
		Rounded Subtotal =>			$ 268,000
	Project Contingency @	20%			$ 54,000
	Survey, Engineering, Permitting & Administration @	25%			$ 67,000
	Project Estimate with Contingency				$ 389,000

Project 5: Hallie Lane Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 5,000	$ 5,000
2	EROSION CONTROL, TEMP STORM AND DEWATERING	LS	1	$ 5,200	$ 5,200
3	CONNECTION #1 - NEW STORM PIPE NEW MANHOLE	LS	0	$ 2,000	$ -
4	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	375	$ 70	$ 26,250
5	F&I 48" STORMWATER MANHOLE, TESTING, COMPLETE	EA	2	$ 8,000	$ 16,000
6	F&I CATCH BASINS - NYLOPLAST	EA	4	$ 2,200	$ 8,800
7	3/4" SHOULDER ROCK	CY	100	$ 50	$ 5,000
8	CONCRETE CURBING - VALLEY GUTTER	EA	375	$ 35	$ 13,125
9	AC REPAVE	TONS	117	$ 200	$ 23,400
10	F&I OUTFALL PROTECTION, CL-50 RIP/RAP	LS	0	$ 2,000	$ -
11	F&I LANDSCAPE RESTORATION	LS	1	$ 2,500	$ 2,500
		Rounded Subtotal =>			$ 106,000
	Project Contingency @	20%			$ 22,000
	Survey, Engineering, Permitting & Administration @	25%			$ 27,000
	Project Estimate with Contingency				$ 155,000

Project 6: Division Street (North) Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL#	LS	1	$ 7,500	$ 7,500
2	EROSION CONTROL, MONITORING, COMPLETE	LS	1	$ 5,200	$ 5,200
3	CONNECTION #1 - (AT MANZANITA AVE)	LS	1	$ 3,000	$ 3,000
4	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	670	$ 80	$ 53,600
5	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	3	$ 8,000	$ 24,000
6	F&I CATCH BASINS - NYLOPLAST	EA	7	$ 2,200	$ 15,400
7	3/4" SHOULDER ROCK	CY	50	$ 50	$ 2,500
8	CONCRETE CURBING - VALLEY GUTTER	LF	650	$ 35	$ 22,750
9	AC TRENCH PATCH	TONS	53	$ 200	$ 10,600
10	F&I OUTFALL PROTECTION, RIP/RAP	LS	0	$ 2,000	$ -
11	F&I LANDSCAPE RESTORATION	LS	1	$ 3,750	$ 3,750
		Rounded Subtotal =>			$ 149,000
	Project Contingency @	20%			$ 30,000
	Survey, Engineering, Permitting & Administration @	25%			$ 38,000
	Project Estimate with Contingency				$ 217,000

Project 7: North Ave & Epoh Ave Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 5,000	$ 5,000
2	EROSION CONTROL, MONITORING, COMPLETE	LS	1	$ 5,200	$ 5,200
3	CONNECTION #1 -(EPOH AVE TO CB AT]NORTH.)	LS	1	$ 2,000	$ 2,000
4	CONNECTION #2 -(NORTH AVE/EPOH AVE.)	LS	1	$ 2,000	$ 2,000
5	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	500	$ 80	$ 40,000
6	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	2	$ 8,000	$ 16,000
7	F&I CATCH BASINS - NYLOPLAST	EA	3	$ 2,200	$ 6,600
8	3/4" SHOULDER ROCK	CY	24	$ 50	$ 1,200
9	CONCRETE CURBING - VALLEY GUTTER	LF	550	$ 35	$ 19,250
10	AC - TRENCH PATCH	TONS	4	$ 200	$ 800
11	OUTFALL PROTECTION, RIP/RAP	EA	1	$ 2,000	$ 2,000
12	F&I LANDSCAPE RESTORATION	LS	1	$ 2,500	$ 2,500
				Rounded Subtotal =>	$ 103,000
	Project Contingency @	20%			$ 21,000
	Survey, Engineering, Permitting & Administration @	25%			$ 26,000
	Project Estimate with Contingency				$ 150,000

Project 8: Greenridge Street Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 15,000	$ 15,000
2	EROSION CONTROL, TEMP STORM AND DEWATERING	LS	1	$ 17,300	$ 17,300
3	F&I 18" STORM PIPE, TESTING, COMPLETE	LF	1775	$ 90	$ 159,750
4	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	6	$ 8,000	$ 48,000
5	F&I CATCH BASINS - NYLOPLAST	EA	18	$ 2,200	$ 39,600
6	3/4" SHOULDER ROCK	CY	65	$ 50	$ 3,250
7	CONCRETE CURBING - VALLEY GUTTER	EA	922	$ 35	$ 32,270
8	AC TRENCH PATCH	TONS	188	$ 200	$ 37,600
9	F&I OUTFALL PROTECTION, RIP/RAP, ANCHORS	LS	1	$ 4,000	$ 4,000
10	F&I LANDSCAPE RESTORATION	LS	1	$ 11,250	$ 11,250
				Rounded Subtotal =>	$ 369,000
	Project Contingency @	20%			$ 74,000
	Survey, Engineering, Permitting & Administration @	25%			$ 93,000
	Project Estimate with Contingency				$ 536,000

Project 9: Cherry Street Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 7,500	$ 7,500
2	EROSION CONTROL, MONITORING, COMPLETE	LS	1	$ 17,300	$ 17,300
3	HDD BORING ON SIDE SLOPE - DUNE STABILIZATION	LF	320	$ 200	$ 64,000
4	F&I 12" STORM PIPE, TESTING, COMPLETE	LF	230	$ 90	$ 20,700
5	F&I CATCH BASINS - NYLOPLAST	EA	3	$ 2,200	$ 6,600
6	3/4" SHOULDER ROCK, COMPACTED IN-PLACE	CY	115	$ 50	$ 5,750
7	CONCRETE CURB - VALLEY GUTTER	LF	350	$ 35	$ 12,250
8	AC, SAWCUT, DEMO, WASTEHAUL, PAVING	TONS	182	$ 200	$ 36,400
9	F&I OUTFALL, RIP/RAP	LS	1	$ 4,000	$ 4,000
10	F&I LANDSCAPE RESTORATION	LS	1	$ 11,250	$ 11,250
				Rounded Subtotal =>	$ 186,000
	Project Contingency @	20%			$ 38,000
	Survey, Engineering, Permitting & Administration @	25%			$ 47,000
	Project Estimate with Contingency				$ 271,000

Project 10: Manzanita Ave Drainage

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Total
1	MOBILIZATION, BOND, TRAFFIC CONTROL	LS	1	$ 15,000	$ 15,000
2	EROSION CONTROL, TEMP STORM AND DEWATERING	LS	1	$ 17,300	$ 17,300
3	F&I 18" STORM PIPE, TESTING, COMPLETE	LF	1000	$ 90	$ 90,000
4	F&I 48" STORMWATER MANHOLES, TESTING, COMPLETE	EA	3	$ 8,000	$ 24,000
5	F&I CATCH BASINS - NYLOPLAST	EA	10	$ 2,200	$ 22,000
6	3/4" SHOULDER ROCK	CY	65	$ 50	$ 3,250
7	CONCRETE CURBING - VALLEY GUTTER	EA	1000	$ 35	$ 35,000
8	AC TRENCH PATCH (4' x 1000') @ 4"	TONS	103	$ 200	$ 20,600
9	F&I LANDSCAPE RESTORATION	LS	1	$ 11,250	$ 11,250
				Rounded Subtotal =>	$ 239,000
	Project Contingency @	20%			$ 48,000
	Survey, Engineering, Permitting & Administration @	25%			$ 60,000
	Project Estimate with Contingency				$ 347,000

Project Summary and Phasing

The projects were prioritized based on the magnitude of the problem and benefits of the improvements. Based on cost and priority, the projects could then separate into four 5-year phasing periods. Improvements in Phase 1 are assumed to be implemented within the next five years, or 2020 to 2025. Phase 2 spans the period between 2025 and 2030, Phase 3 occurs between 2030 and 2035, and Phase 4 approximately between 2035 and 2040.

There are several methodologies that the City can use to project these costs into the future.

One method is a straight-line method of a rate of increase. For example, the City could multiply the cost given in the Cost Summary by 2 to 3%. This method is effective and could be used for the first phasing period as it is straight-forward in calculations.

A second method that may be more accurate as the time extends to Phase 2 through 4 may be use of the ENR Construction Cost Index, which would calculate the inflation rate based on the current date, and a later period of time.

Anticipated construction costs also could be pro-rated accordingly if they are completed in a different phasing period than proposed, or different construction conditions and methods exist in the construction industry.

The estimated costs, for each project, as of November, 2020 are summarized in Figure 8.3, below.

Figure 8.3 CIP Cost Summary*

COST SUMMARY		
PROJECT PRIORITY	PROJECT NAME	COST
1	DIVISION STREET DRAINAGE	$ 809,000
2	SITKA STREET DRAINAGE	$ 144,000
3	LAKEVIEW DRIVE DRAINAGE	$ 730,000
4	PINE STREET DRAINAGE	$ 389,000
5	HALLIE LANE DRAINAGE	$ 155,000
6	DIVISION STREET (NORTH) DRAINAGE	$ 217,000
7	NORTH AVENUE DRAINAGE	$ 150,000
8	GREENRIDGE STREET DRAINAGE	$ 536,000
9	CHERRY STREET DRAINAGE	$ 271,000
10	MANZANITA AVENUE DRAINAGE	$ 347,000
TOTAL		$ 3,748,000

**Cost and project information are also located in Figure 8.1 Proposed Capital Improvement Projects.*

Drainage issues at the locations listed above are creating issues for the homeowners and visitors to the City of Manzanita. Further, they may also hamper development. This listing is not all-inclusive, and as development proceeds forward, other areas may present developmental issues and must be dealt with. No large contingency for materials, construction or time of year has been added.

Because of the complexities of these individual CIP projects, we would recommend further study, drainage analysis and citizen input before actual engineering and preparation of an actual construction cost estimate is done.

CHAPTER 9: STORMWATER QUALITY, SEDIMENTATION AND EROSION

Runoff Control

Construction activities usually result in the removal of vegetative cover and increases in impermeable surfaces, both of which increase the volume and velocity of runoff. Increases of stormwater volume and velocity lead to increased erosion (gulling) in the sediment transport and off-site delivery (sedimentation). These increases must be addressed when implementing erosion and sediment control.

Runoff control measures are those practices which mitigate for the erosive and sediment transport forces of stormwater during and after construction activities. Some examples of runoff control might include, outlet protection (energy dissipaters), diversion dikes and swales, temporary slope drains, rock lined channels, grass-lined channels, and temporary stream crossings.

Runoff control involves the use of structures to reduce velocities and/or safely carry stormwater in a manner which reduces erosion and sediment transport.

The energy equation, E = mv2, where E = erosive Energy, m = unit density of water, and v = the runoff velocity, demonstrates that if the velocity of running water is reduced by half, then the erosive energy will be reduced by four times. To reduce runoff energy, it is recommended that the City and the citizens of Manzanita implement practices which use the 4 D's of runoff control.

- Decrease - decrease the amount of runoff
- Detain - decrease the velocity
- Divert - divert runoff to less erodible areas
- Dissipate - spread the runoff out

Rill and gully erosion is caused by concentrated runoff. Methods which reduce runoff velocity, such as check dams, vegetated channels or riprap, will also reduce the potential for sediment transport. An alternative to reducing the runoff energy of stormwater is to convey that runoff through or along non-erodible surfaces, i.e., culverts and slope drains. Conversely, conveying runoff through culverts generally results in an increase in downstream velocities.

Temporary check dams, especially straw bale dams, are not recommended for any flowing water conditions. Straw bales and silt fences are sediment control Best Management Practices, not runoff control Best Management Practices and should not be placed in channel flow (runoff) areas.

Best Management Practices for Maintenance

For the purposes of this report, pollution prevention is defined as the use of materials, processes or practices that reduce or eliminate the creation of stormwater runoff pollutants or wastes at the source. Best Management Practices (BMPs) are structural and managerial techniques that are recognized to be the current, most effective means to prevent and/or reduce pollution from stormwater runoff. There are generally two reasons to implement BMPs from a surface and ground water quality standpoint: to protect the existing level of water quality from future degradation; and, to correct existing water quality problems.

This report discusses both structural and managerial practices to be implemented as a part of the proposed pollution prevention strategy for the City of Manzanita. These practices are intended to minimize pollutants and/or improve stormwater runoff quality. The emerging philosophy of stormwater management emphasizes controlling stormwater where it falls and incorporates both structural and vegetative measures to detain and “treat” the water.

Specific needs regarding quantity and quality of stormwater runoff addressed for this project include:

1. Minimizing any increase in the current volume of runoff.
2. Minimizing any increase in the velocity of runoff.
3. Provisions for improving water quality to the highest degree possible with currently available, passive technology.

Landscape Plantings

Landscaping for stormwater management concerns is a science that is still in its infancy. It is known that proper landscaping techniques can create both a beautiful physical setting and one that benefits the environment and saves water. In fact, attractive, water-efficient, low-maintenance landscapes can increase home values between 7% and 14% (USEPA, 1993). In addition, using trees and shrubs to provide shade in the summer and sunlight in the winter can cut in half home cooling and heating costs. Other environmental benefits include:

1. Reduced irrigation water use
2. Reduced runoff of stormwater and irrigation water that carries soils, fertilizers and pesticides
3. Fewer yard wastes that need to be disposed
4. More habitat for plants and wildlife

Roadside and Perimeter Vegetated Swales

Vegetated swales are most applicable in residential areas of moderate density such as found in Necarney City. This practice requires that site planning and design respect the natural drainage patterns in lieu of elaborate underground, piped, storm drain systems. Vegetated swales are considered as a method of biofiltration using terrestrial grasses and other fine herbaceous plants for stormwater treatment.

Roadside swales should be used in lieu of a conventional curb and gutter drainage systems. A perimeter swale may be planned either on private property (as a part of the property development) or within the ROW of public streets. Both swale systems can also discharge into a vegetated infiltration/detention basin. The perimeter swale can also be provided with overflow weirs and/or level spreaders to dissipate concentrated flows and provide additional sediment settling capabilities. During larger, less frequent runoff events, the weirs can overtop and function as a level spreader dispersing flow over a larger cross-sectional area.

Swale design incorporates the use of Manning's equation of open channel flow to obtain a width for a given flow and slope and selected water depth. The velocity resulting in a sized channel is then compared to a criterion, and the length is calculated using a hydraulic residence time criterion. A recent key study showed that a residence time of nine minutes is needed to achieve the highest and most reliable performance. Deterioration of performance was noticeable when the residence time fell below five minutes, which is recommended as the absolute minimum. Other construction features that help to maximize the success in establishing swales as biofilters are:

1. Sited or located away from building and tree shadows to avoid poor plant growth from lack of sunlight.
2. If the longitudinal slope is less than 2% or the water table can reach the root zone of vegetation, water tolerant species should be planted.
3. As much as practicable, the lateral slope is entirely uniform to avoid any tendency for the flow to channelize.
4. Entrance flow is dissipated quickly, and flow distributed uniformly to avoid erosion.

Establishing an effective and productive grass cover in the swales will begin with an evaluation of site-specific conditions and proposed topsoil cover. Based upon ongoing localized studies being conducted to evaluate biofiltration capabilities of varying cover crops, the following grass mixture applied at a rate of 130 to 175 lbs/acre, dependent on detained runoff volumes, is recommended:

Species/Variety	***lbs./1000 sq. ft.***
Perennial Rye	
Edge	0.45 to 0.60
Kentucky bluegrass	
Ram	0.45 to 0.60
Meri	0.45 to 0.60
Chewings fescue	
SR510	0.90 to 1.20
Hard fescue	
SR3100	0.75 to 1.00

One example of an effective vegetated perimeter swale in Manzanita can be found at the city park. A close examination of the perimeter swale indicates that sediments and oils that run off of the parking lot are being effectively trapped in the perimeter swale that is a mowed grassy surface.

Infiltration/Detention Basin

Infiltration/detention basins are probably the most common management practice for the control of stormwater runoff. If properly designed, constructed and maintained, infiltration/detention basins can be effective in controlling peak runoff flow rates.

Design and construction of the basin where the swales will discharge to, incorporates measures which will account for enhanced pollutant attenuation in the soil column. A combination of first flush detention (i.e., plunge pool at inlet) followed by infiltration will protect water quality while filtering out a portion of those substances which contribute to basin failure (i.e., sediment).

Generally, basins should be located in a naturally low area of the landscape. Optimally, the floor should be 18 to 24 inches above the seasonal high-water table to avoid direct contact with ground water and allow treatment by infiltration through the soil.

Pollutant removal is governed by basin size. Removal rates of residual pollutants can be enhanced by increasing the surface area of the basin reserved for exfiltration. Exfiltration refers to the amount of runoff that is effectively infiltrated through the soil profile. For example, an infiltration basin sized to store and exfiltrate half an inch (first flush) of runoff volume will only be effective for the removal of pollutants from runoff volumes equal to or less than half an inch. Conversely, a basin designed to capture a 2-year storm will treat all the intermediate sized storm events. Recommended vegetative seeding mixtures for the basin and rates are the same as those for grass swales.

Catch Basin Cleaning

It is recommended that catch basin sumps be cleaned out at least once a year, and optimally twice, to accomplish effective pollutant removal and to protect downstream waters. Catch basins are usually cleaned with a vacuum truck, however, if sediment volume is minimal, manual cleaning can be equally effective. The resultant slurry of water, sediment and other debris can be transported to an approved treatment plant or landfill for disposal. Proper cleaning helps to reduce the re-suspension of sediments during runoff.

It is important to keep maintenance records and clean-out schedules as part of the catch basin maintenance process. A periodic review of the records will then highlight those areas needing more maintenance and those needing less, streamlining the cleaning process.

Vegetated Swale Maintenance

Homeowners and City Public Works staff should be encouraged to use mulching mowers for lawn and grassed swale mowing. By mowing the grass on a regular basis, the vegetative state of the grasses will be enhanced while increasing the pollutant removal potential of the swale. Grasses should generally be mowed to a height of approximately 2 to 4 inches.

Grassed swales should be inspected monthly and after large storm events to determine whether there are erosion problems that need to be controlled, to remove accumulated debris and to inspect the vegetation. Care should be taken in the removal of sediments so that underlying vegetation is not destroyed, especially at the culvert ends and outlets, if any. Handheld tools should be used and leaves raked out as necessary.

Erosion Control

Erosion control is any practice that protects the soil surfaces and prevents the soil particles from being detached by rainfall or wind. Erosion control, therefore, is a source control that treats the soil as a resource that has value and should be kept in place.

The most efficient and economical method of controlling sheet, rill and raindrop impact erosion is to establish vegetative cover from seed. Vegetation can reduce erosion by more than 90% by protecting the soil from raindrop impact and sheet erosion.

When erosion control BMPs are implemented and maintained, the amount of sediment associated with runoff waters can be dramatically reduced. Whenever possible are provide erosion control first and sediment control second. Some important points include:

- Vegetative cover is the primary erosion control practice.
- Retain existing vegetation by minimizing disturbance and scheduling large land disturbances during periods of expected dry weather in the late summer months.
- Establishing cover immediately after disturbance (staging) is important.
- Temporary erosion control is usually achieved by seeding with fast growing annual grasses and/or protecting the soil with mulch.
- Permanent erosion control usually involves planting perennial grasses, shrubs, and trees.

The selection of the right plant material for the site, choosing the correct mulching technique and proper seedbed preparation are critical for effective erosion control. Surface roughening, contour furrows, and stepped slopes are essential to establish vegetation on sloping surfaces.

Non-structural erosion control practices are generally more cost-effective than sediment control. For example, the cost of temporary seeding one acre would be comparable to the cost of installing 200 linear feet of silt fence (for one acre drainage) or equivalent to the cost of constructing a temporary sediment trap designed for a one acre drainage. However, the practice of temporary seeding would probably be more effective while the silt fence and sediment trap will require regular and costly maintenance.

Erosion control is, generally, more cost-effective than sediment control and requires less maintenance and repair.

Sediment Control

Sedimentation is the deposition of soil particles that have been transported by water or wind. The amount of sediment produced during construction is directly proportional to the degree and effectiveness of erosion control practices implemented. The quantity and size of the particles transported increases with the velocity of the runoff.

Sediment control is used to keep sediment, the product of erosion, on site. Sediment control involves the construction of structures that allow sediment to settle out of suspension. Sediment control structures, therefore, require frequent inspection and maintenance.

Generally, sediment is retained on site by two methods: a) slowing runoff velocities, as they flow through an area, sufficiently so that sediment cannot be transported, and b) impounding sediment-laden runoff for a period of time so that the soil particles settle out.

Sediment controls are not filtering. Practices referred to as "sediment filtering" actually work by slowing velocities and allowing sediment impoundment to de-water in a very slow and controlled manner. For effective sediment control planning and design, materials such as geotextiles, silt fences, and straw bales should be considered for their ability to impound water and slow runoff velocities, not for their ability to "filter" sediment.

Effective sediment control involves ponding sediment-laden runoff long enough for the soil particles to settle out of suspension. Reducing runoff velocities will also reduce sediment transport and thereby help retain sediment on-site.

Structural sediment control can be divided into three general types; 1) sediment basins, 2) sediment traps, and 3) sediment barriers.

Temporary Sediment Basins are recommended for the outlet of disturbed drainage areas ranging from 5 acres to 100 acres. Sediment Basins should be designed by a qualified professional.

Temporary Sediment Traps are recommended for disturbed drainage areas less than 5 acres. A typical Sediment Trap designed to handle 0.5 inches of runoff over a 24 hour period would require a settling zone capacity of 67 cubic yards/acre of contributing drainage area and a sediment storage capacity of 33 cubic yards/acre of drainage area.

Excavated Sediment Traps require less rigorous design work, are smaller in size and they are easier to construct, therefore, a preferable alternative is to sub-divide large projects into smaller subareas (less than 5 acres) and utilize numerous sediment traps. Multiple traps and/or additional volume may be required to accommodate site specific rainfall and soil conditions. This approach may facilitate phased construction along relatively narrow highway ROW.

Excavated Storm Drain Inlets are small excavated sediment traps located at storm drain inlets are effective as part of phased construction. The design capacity of excavated inlet sediment traps shall be 67 cubic yards/acre of contributing drainage area. These excavations are temporary, and they are not very effective for trapping small particles (silt and clay) and they should not be used where runoff velocities are high.

Sediment Barriers are BMPs that are intended to separate sediment from sheet flow runoff. They function by reducing runoff velocity and ponding small quantities of stormwater. Sediment barriers are only intended for areas experiencing sheet flow and they must be installed in areas that can pond water and accumulate sediment and, most importantly, the must be accessible for cleanout. Sediment barriers are the most common type of practices used on construction sites.

Examples of sediment barriers include:

- Silt Fence
- Straw Bale Dike
- Continuous Berms
- Storm Drain Inlet Barriers

Principles of Erosion and Sediment Control

Severe erosion is caused by the action of wind, rainfall, and runoff on bare soil. Clearing, grading, and other construction activities remove the vegetation and compact the soil, increasing both runoff and erosion. Excessive runoff then causes gully erosion, increased streambank erosion, and results in increased off-site erosion, sedimentation and flooding problems. Effective erosion and sediment control can be achieved by careful attention to the following principles:

- Protect the land surface from erosion.
- Manage runoff and keep velocities low.

- Capture sediment on site.
- Integrate erosion and sediment control with the construction schedule.
- Inspect and maintain the erosion and sediment control practices.

The following are principles for controlling erosion and off-site sedimentation from construction sites:

- Fit the development to the existing topography, soils, and vegetation as much as is possible.
- Schedule construction operations in order to minimize soil exposure during the rainy season.
- Minimize disturbance and soil exposure by retaining natural vegetation, adopting phased construction techniques, and using temporary cover.
- Vegetate and mulch all denuded areas to protect the soil from winter rains. The primary effort for controlling sediment pollution from construction sites should be to minimize raindrop impact on bare soil.
- Utilize proper grading, barriers, or ditches to minimize concentrated flows and divert runoff away from denuded slopes or other critical areas.
- Minimize the steepness of slopes and control the length of slopes by utilizing benches, terraces, contour furrows, or diversion ditches.
- Utilize riprap, channel linings, or temporary structures in the channel to slow runoff velocities and allow the drainage ways to handle the increased runoff from disturbed and developed areas.
- Keep the sediment on-site by utilizing sediment basins, traps, or sediment barriers.
- Monitor and inspect sites frequently to assure the measures are functioning properly and correct problems promptly.

Vegetation as a Solution

Dense, healthy vegetation and the associated leaf litter protects the soil from raindrop impact. Raindrop impact is a major force in dislodging soil particles which then allows them to move downslope or form a crust on the soil surface. When a crust forms on the soil surface the rainfall infiltration rate decreases and runoff increases.

Vegetation also protects the soil from sheet and rill erosion. It shields the soil surface from the transport of soil particles and scour from overland flow (sheet flow) and it decreases the erosive energy of the flowing water by reducing velocity.

The shielding effect of the plant canopy and leaves is augmented by roots and rhizomes that hold the soil in place, improve the soil's physical condition, and increase the rate of infiltration, further decreasing runoff. Plants also remove water from the soil through transpiration, thus increasing its capacity to absorb water.

Suitable vegetative cover provides excellent erosion protection, and reduces the need for high cost, low efficiency, high maintenance sediment control measures. Vegetative cover is relatively inexpensive to achieve and tends to be self-healing; it is often the only practical, long-term solution of stabilization and erosion control on most disturbed sites.

Initial investigation of site characteristics and planning for vegetation stabilization reduces its cost, minimizes maintenance and repair, and makes other erosion and sediment control measures more effective and less costly to maintain. Permanent erosion control (post-construction landscaping) is also less costly where soils have not been eroded.

Exposed subsoils are generally difficult to amend, are infertile, and require more irrigation. Natural, undisturbed areas can provide low-maintenance landscaping, shade, and privacy. Large trees increase property values when they are properly protected during construction.

Besides preventing erosion, healthy vegetative cover provides a stable land surface, reduces heat reflectance and dust, restricts weed growth, and complements architecture. The result is a pleasant environment for employees, tenants and customers, and an attractive site for homes.

Property values can be increased dramatically by small investments in erosion control. The final landscaping represents a small fraction of total construction costs, but it can contribute greatly to an increased market value of the development. Healthy vegetation and planned development will reduce concentrated flows and peak discharge, thus reducing channel erosion and flooding. Good, healthy vegetative cover greatly reduces the environmental impacts that poor water quality and habitat reduction is having on rivers and streams.

Ordinance for Construction Erosion Control

We first recommend that the city staff become completely knowledgeable about the areas within the City where an erosion control ordinance could apply. The City staff should prepare an overlay map of the assessor's tax lot maps to mark those lots that either will or may be subject to the requirements of this ordinance. All drainage basins can readily be identified from the maps within this SWMP. Therefore, all lots that are within 100 feet from the Golf Course Creek and other watercourses can readily be identified. Similarly, all lots with slopes that exceed 12% can generally be identified, at least on a global basis. There may always be individual areas on a particular lot that are, however, less than 12% in slope.

We recommend that the City staff should be conservative in its approach to implementing this section of the ordinance. It is better to request that a property owner comply with the ordinance and then allow the property owner to provide documentation that the subject property in question is not subject to the requirements of this section of the ordinance. Given the many steep areas within the City of Manzanita, there are numerous lots within the city that are located on slopes greater than 12%.

The City should develop and disseminate additional information related to erosion control during construction. As a basis for developing such information, we have included at the end of this section a sample of the ordinance and erosion control guidance document used by the City of Cannon Beach. The erosion control guidance document is relatively simple and straightforward for easy understanding and implementation by property owners and contractors without the need for professional engineering assistance. In addition to the grading and erosion control ordinance provided from the City of Cannon Beach, we have also included a copy of the City of Lincoln City's more extensive ordinance related to erosion control and earthwork. Those two ordinances are provided as reference material only for the future use of the City of Manzanita as it addresses the appropriate revisions needed to the Zoning Ordinance.

Ultimately, it is the property owners and the excavating contractors who live and/or work in the City of Manzanita who will need to understand and implement correct grading and erosion control methods in order to reduce sedimentation in the watercourses within the City that lead to the ocean and Nehalem Bay.

Erosion and Sediment Control Plans

The erosion control plan must be prepared before construction begins, ideally during the project planning and design phases. The erosion control plan shall be submitted with the grading plan as required by local ordinances or be prepared as part of the Stormwater Pollution Prevention Plan (SWPPP).

If the grading permit allows work to be done during the wet weather season, the permit may require a wet weather operating and erosion control plan. This plan must be approved prior to the commencement of any work and include all necessary temporary and permanent erosion control measures, including those to be followed should the work stop at any time during the wet weather season.

If the site or portion of the site is planned to be idle for more than 45 days, then vegetative stabilization must be accomplished within seven days. The wet weather plan should include a plan for the immediate (within 24 hours of the first forecast of a stormfront) installation of emergency erosion control measures.

Guidelines for Erosion Control Plans

The plan should consist of three parts:

1. A narrative, containing:

- A brief description of the proposed land-disturbing activities, existing site conditions, and adjacent areas (such as creeks and buildings) that might be affected by the proposed clearing and grading;
- A description of critical areas on the site - areas that have a potential for serious erosion problems, including the name, location and aerial extent of moderate and highly erodible soils and slopes on the project site;
- The date grading will begin and the expected date of stabilization;
- A brief description of the measures that will be used to control erosion and sedimentation on the site;
- When these measures will be implemented; and
- A description of an inspection and maintenance program, with provisions for frequency of inspection, reseeding, repair and reconstruction of damaged structures, cleanout and disposal of trapped sediment, duration of maintenance program, and final disposition of the measures when site work is complete.

2. A map showing:

- Existing site contours at an interval and scale sufficient for distinguishing runoff patterns before and after disturbance;
- Final contours;
- A legend, if necessary;
- Limits of clearing and grading;
- Existing vegetation, such as grassy areas or vegetative buffers, that may reduce erosion or off-site sedimentation;
- Critical areas within or near the project site, such as streams, lakes, wetlands, or the aerial extent of erodible soils; and
- The location and types of erosion and sediment control measures, including the aerial extent of vegetative treatments.

3. Plan details, including:

- Detailed drawings of erosion and sediment control structures and measures, showing dimensions, materials, and other important details;
- Design criteria and calculations such as design particle size for sediment basins and peak discharge for channel design and outlets;
- Seeding or vegetative specifications; and
- Inspection and maintenance notes.

The narrative and details should be placed on the Erosion Control Plan Map.

Plan Check

The following items provide a general approach and guidelines that the City of Manzanita, as the review agency or plan checker, might find useful:

- <u>Responsibility</u>: It is not the responsibility of the plan reviewer to ensure that the plan is appropriate for the level of work suggested by the proposed project. The reviewer can only ensure that the plan meets the minimum standards set by the City and its authorizing ordinance.
- <u>Communications</u>: Encourage informal communications between the plan reviewer and the plan preparer. This will enable the reviewer to make informal suggestions that may save the developer

money and the preparer time, and it may result in a better, more effective plan. It will also enable the preparer to explain and justify the plan.

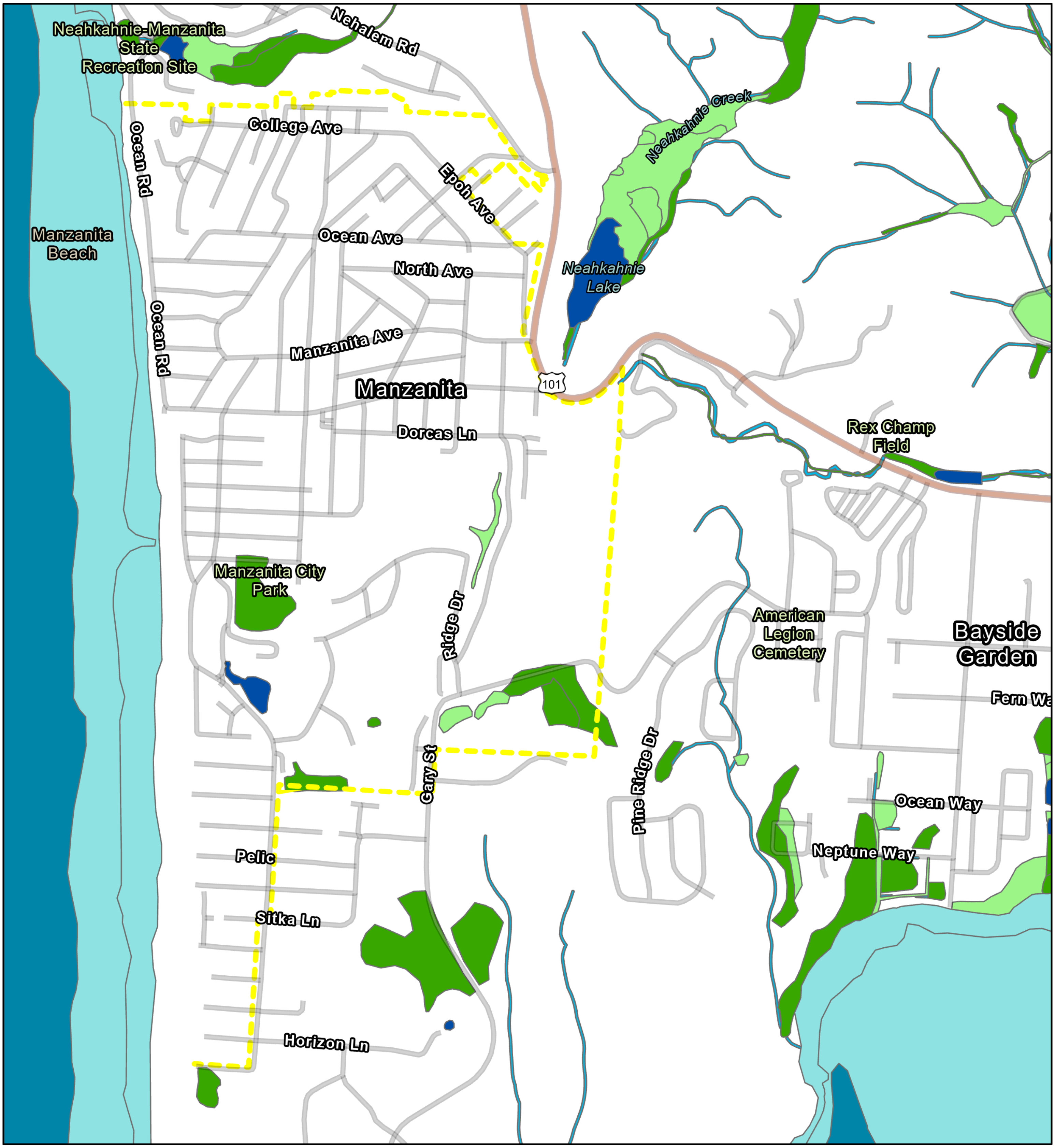
- Incomplete Plans: Do not review seriously incomplete plans. Send them back with a request for the missing information.
- Required Information: Make sure all the required information has been submitted. A checklist can be used by both plan reviewers and plan preparers; however, checklists can encourage laziness. Having everything checked off does not necessarily mean that everything is in order.
- Plan Concept: The concept should be examined first, starting with the general and moving to the specific. Does the plan make sense?
- Schedule: Examine the construction schedule. Will grading be completed before the wet weather season or before the summer thunderstorm months? When will storm drainage facilities, paving, and utilities be installed in reference to the wet weather season? If grading will take place during months when there is a high probability of heavy rains, what extra precautions will be taken to protect against erosion, sedimentation, and changing drainage patterns (Is a wet weather plan necessary)?
- Minimize Disturbance: Does the plan show areas that are not to be disturbed? If possible, native vegetation should be retained and stream buffer areas should be designated on the plan and flagged in the field. A well-conceived erosion control plan will minimize erosion by attempting to minimize disturbance and retain natural vegetation. A phased approach to development can assure that the extent and timing of grading does not exceed the contractor's ability to perform erosion and sediment control.
- Site Drainage: Make sure you understand where all drainage comes from on and above the site, where it goes, and how it traverses the site. For large sites, require or prepare a drainage area map. If drainage patterns are unclear, ask for clarification.
- Sediment Basins and Traps: Locate all sediment basins and traps and define their tributary areas.
- Erosion Control: Check the method used to prevent erosion. Hydraulic seeding and mulching may adequately stabilize some areas, but other areas, because of their proximity to sensitive features such as watercourses, or their steepness and erosive soil, may need far more intensive revegetation efforts. On steep and critical slopes, a reliable backup system for hydraulic planting, such as punched straw, bonded fiber matrix, or erosion control blankets is strongly recommended.
- Channels and Outlets: Examine all drainageways where concentrated flows will occur. Be sure adequate erosion protection is provided both along channels and at channel and pipe outlets. Check the sources of runoff to be sure that all the runoff comes from undisturbed or stabilized areas or has been desilted by sediment basins or other sediment retention devices.
- Miscellaneous: Look for haul roads, stockpile areas, and borrow areas. They are often overlooked and can have a substantial effect on drainage patterns. Have construction or access roads been surfaced with rock, as a minimum treatment, before the rainy season? Look at all points of vehicle access to the site and be sure mud and soil will not be tracked onto paved streets and that sediment-laden runoff will not escape from the site at these points. Pay particular attention to watercourses and their protection.
- Plan Details: Once the plan concept has been shown to be adequate, check the details to be sure the concept is adequately described in the plans.
- Structural Details: Be sure that sufficiently detailed drawings of each structure (sediment basin, dike, ditch, silt fence, etc.) are included so there is no doubt about location, dimensions, or method of construction.
- Calculations: Determine if calculations have been submitted to support the capacity and structural integrity of all structures. Were the calculations made correctly? Non-engineered structures, such as straw bale barriers, do not generally need hydrologic calculations, however, supporting information such as drainage area and peak flow should be available if requested.
- Vegetation: Review seed, fertilizer, and mulch specifications. Check quantities and methods of application to be sure they are appropriate and consistent with local guidelines. Are there stipulations so that ineffective revegetation and/or damage can be remedied immediately?

- Maintenance: Be sure that general maintenance requirements and, where necessary, specific maintenance criteria, such as the frequency of sediment basin cleaning, are included. Are there stockpiles of spare materials (filter fabric, straw bales, stakes, gravel, etc.) to repair damaged control measures? Routine maintenance inspections should be part of the plans.
- Contingencies: The plan must provide for unforeseen field conditions, scheduling delays, and other situations that may affect the assumed conditions. For example, straw mulch may need to be installed as an emergency measure during severe summer thunderstorms, or sediment basins may need to be cleaned more frequently.
- Technical Review: Where applicable, the erosion and sediment control plan should be reviewed by the soils, certified professional in erosion and sediment control (CPESC), or geotechnical consultant for the project.
- Signature: Where applicable, the erosion and sediment control plan should be signed by the preparer who shall be a qualified professional for large development projects.

The City may wish to prepare examples of Erosion Control Plans, which would be required for all single-family site development (individual lots) have been included for the City of Manzanita. Locally, the City of Lincoln City and the City of Cannon Beach have adopted ordinances to require the preparation and implementation of an approved erosion control plan for all site development work within those cities.

Appendix A

Wetland Map



Legend

Estuarine and Marine Deepwater	Freshwater Emergent Wetland
Estuarine and Marine Wetland	Freshwater Forested/Shrub Wetland
Freshwater Pond	Riverine

APPENDIX A

WETLAND MAP

CITY OF MANZANITA
STORM WATER MASTER PLAN

MANZANITA | OREGON

FEET
0 500 1,000

City of Manzanita