Section 5: Conveyance System Analysis

5.1 Introduction

City wide storm water infrastructure has been investigated through a combination of existing document review, discussions with City staff, site visits, and limited system survey. Six collection and conveyance systems with discrete surface outfalls were modeled. Current City staff concerns, current areas experiencing flooding events, ongoing problematic maintenance needs, and areas of expected growth guided the modeling efforts. The model examines the current hydraulic capacity of each system and provides an understanding of future system hydraulics for systems that are anticipated to require additional capacity. The model was built and analyzed using Autodesk Storm and Sanitary Analysis 2012 (SSA) and the EPA Stormwater Management Model (SWMM) hydrology model. Development of the model and interpretation of the output is discussed in this section.

In general, the SSA model has two main functions: flow generation and flow conveyance. The model generates a runoff hydrograph based on a simulated rainfall event. The flow conveyance function consists of a junction and link representation of the pipes, manholes, and other structures typically found in storm water collection systems. Flows represented by the runoff hydrograph are injected over time into the appropriate junction, typically a manhole, and are dynamically routed through the collection system, allowing flow propagation to attenuate through the system, and to generate an overall hydrograph at a terminal outfall. Outfalls are usually real structures, such as river discharges.

The model was set up and adjusted to produce results that generally agreed with observations by City staff. However, much of the observed flooding is believed to be related to UICs and there was no infiltration included in the modeling. It is also important to note that flow monitoring during actual flood events was not included in this study. Therefore, the model has not been calibrated against actual events. The results from the model may vary from the actual conditions, and the City may wish to verify the model output by performing flow monitoring in the future.

5.2 Conveyance System Model

A model was developed for each of the six surface collection and conveyance systems with surface outfalls. The model was developed starting with the GIS data provided by the City for the existing storm water system, which included some elevation information for manhole and UIC rims, and general pipe locations. The GIS data was updated with available record and design drawings; however, the diameter, material type, inverts and slopes of less than two percent of the system is recorded. Some additional data was collected through survey of top of manhole rim elevations and measure-downs to pipe inverts.

Model pipes, basins, and junctions were defined to provide a realistic representation of the most likely existing system, but do not correspond exactly to the existing conditions or to the network represented in the GIS, in some cases. The model is understood to contain the ‘best available’ information. Assumptions were made as to pipe slope, invert elevations and diameter, when

necessary, to best represent the existing conditions reported by the City. Pipes, junctions and subbasins were defined in the GIS using the existing system information as a guide. Their geometric dimensions such as length, area and slope were calculated and the elements were imported into SSA.

5.2.1 Conveyance System Assumptions

All pipe was assumed to have a Manning's n of 0.015, which is an average value for concrete sewer pipe with inlets, manholes, etc. New pipe was also conservatively assigned a Manning's n of 0.015, although all new pipe is required by City design standards to be either polyvinyl chloride (PVC) or high-density polyethylene (HDPE), which may have a lower Manning's n and consequently a higher capacity. The minimum cover for all existing and new pipes was assumed to be 30 inches, which also aligns with City design standards for paved areas. The available grade constraint typically set the design pipe slope and pipes were designed to eliminate or minimize surcharge without specifically checking for the minimum required flow velocity of three feet per second. The minimum diameter used for new pipe was 12 inches.

When unknown, manholes were assumed to be seven feet deep to the lowest pipe invert with a 1.5 foot deep sump, which is the minimum sump depth required by City design standards. Where new manholes were modeled, they were generally spaced at the required maximum 500-foot spacing in the absence of other criteria. Each manhole was modeled to flood a circle of 50 foot diameter (an area of 123 square feet) which is the width of the right-of-way. The flooded depth is returned to the manhole when flows subside. UICs were in some cases used as junction locations and assumed to have a 27 foot sump depth with an initial water surface elevation 10 feet below the rim, but no infiltration was modeled at any point in the conveyance system.

5.2.2 Model Subbasins

Surface drainage subbasins were delineated starting from the basin delineations in the 1994 SDMP and are named to be consistent with that plan. The existing pipe network and catch basin locations in the GIS were used to further define the contributing area. U.S. Geological Survey (USGS)-Metro 10-foot interval contour data and manhole and UIC rim elevations were used to delineate basin boundaries. Flow directions, catch basin locations, and street level drainage breaks were in many cases identified using Google Maps satellite or street level imagery. The City of Canby requires that all stormwater from private lots, except the downtown core area including 1st, 2nd and 3rd streets, be managed on-site. The area contributing runoff to the City's systems is, therefore, estimated based on the area of the public right-of-way.

The entire area of the right-of-way is assumed to be impervious surface. An impervious surface is one in which water is unable to infiltrate. Impervious surfaces are typically man-made structures such as roadways, sidewalks and roofs. The assumption of 100% imperviousness in the right-of-way accounts for the area of paved roads and sidewalks, driveways and other impervious surfaces within the public right-of-way. The assumption also allows for some run-on to the right-of way from adjacent pervious surfaces, such as private driveways.

Basins were digitized in the GIS either using the tax lot boundaries to define the right-of-way limits, or crossing tax lot boundaries where other information was available. Basins were

developed for all right-of-way areas draining to surface systems. In the case of N Maple St, the adjacent agricultural area was also modeled, based on contour data and satellite imagery and routed to the conveyance system.

Based on the Comprehensive Plan Zones map, new residential development was assumed to take place on both sides of the north end of N Pine St between NE 15th Ave and NE Territorial Road, and on the east side of N Redwood Road from NE 12th Ave to NE 18th Pl. A basin representing a 500-foot or 750-foot long by 50-foot wide impervious right of way was assumed for each major tax lot and street intersection, with the length depending on the tax lot depth. Other future basins were existing basins that are anticipated to be connected to existing systems as CIPs.

5.2.3 Additional Basin Characteristics

Basin characteristics must be input into the model. These inputs are presented and discussed in the following list.

- Area – Subbasins were delineated and digitized in the GIS, which was also used to calculate their area in acres and square feet.

- Flow Path Length – The longest flow path length, or the maximum distance, in feet, a drop of rainfall must travel over land before it enters the conveyance system is used to calculate the time of concentration of the basin. The time of concentration is used to determine the speed of runoff response. Flow path length was either approximately digitized or estimated for all drainage basins, based on the longest distance from a basin boundary to an inlet.

- Equivalent Width – The equivalent width of each basin was approximated by dividing the area of each basin in square feet by the longest flow path length in the basin in feet. Equivalent width is used by the model in estimating time of concentration.

- Percent Slope – The average percent slope of the basin was estimated. Elevations along the flow path were used to define the estimated change in elevation where contour data was available. In many locations, the City is flat and level and a default basin slope of 0.01 percent was assumed.

- Percentage Depressions - Depth of depression storage within the basin. Storage depth is defined for both impervious, 0.08 inches, and pervious, 0.20 inches, portions of the basin.

- Curve Number – The SCS curve number represents the type of land cover and soil in the basin and is used in the EPA SWMM model only for infiltration, which occurs only in basins with less than 100% impervious cover.

5.2.4 Flow Generation Information

The design rainfall event is defined as precipitation intensity over a specified time period, which produces a total rainfall amount in inches. A SCS Type IA 24-hr storm distribution was used,

with precipitation amounts for Clackamas County. The 10-year and 25-year storms were modeled. The EPA SWMM model simulates rainfall on the defined basin surface, first filling the depression storage, then calculating the runoff rate to model junctions using Manning's equation and the time of concentration for the basin. A hydrograph is developed for each basin and routed through the pipe network junction by junction.

5.3 Hydraulic Criteria

5.3.1 Hydraulic Grade Line (HGL) Evaluation Technique

Modeled output is used to evaluate the hydraulic capacity of total pipe networks, subbasin networks, and individual pipe segments. In order to evaluate such a large quantity of output data, evaluation criteria must be developed. This criteria tests output data and determines the point at which the available capacity no longer allows for an acceptable factor of safety. These areas triggering a greater capacity are then translated into CIP projects.

The factor of safety associated with a piping configuration is typically related to the ratio between level of flow during a peak event divided by the total water level provided in the pipe, or more frequently called the d/D. For example, a ratio of 0.5 represents a half full pipe and a ratio of 1 represents a full pipe. As the ratio approaches 1, the factor of safety against an overflow event is decreased.

The modeling effort used two design storm events to develop an understanding of both the system as it functions now, and how it could react during a future storm event. Both current and future conditions were modeled using a 10-year, 24-hour 25-year, 24-hour design event. The basic criteria for evaluating whether a modeled system was adequate are that it does not surcharge during the 10-year event and that it does not flood during the 25-year event. Model outputs have been tabulated and are provided in Appendix A. Surcharged means that a pipe's maximum to total depth ratio is greater than 1.0, i.e. the water surface elevation exceeds the crown of the pipe. Flood means that the water surface elevation in a junction (manhole) exceeds the manhole rim elevation.

5.3.2 Sizing Evaluations

Flows developed in both the 10-year and 25-year modeling scenarios have shown surcharging events to be present in some systems. Most often, this surcharging is directly related to insufficient capacity in downstream piping. As downstream surcharging develops, an upstream backwater wave begins. As this backwater builds, upstream piping begins to surcharge as well. When a surcharge event is developed in the model, typically many pipe segments are affected. While numerous pipes may have contributed to the surcharging, it is important to note where the surcharging and backwater wave began, as it directly affects where the CIP project should focus. Additionally, since CIP projects are not always completed in one phase, it is important to begin improvements at the downstream end of the piping network. As downstream capacities are increased, surcharging related to backwater waves will be reduced.

5.4 Modeled Systems

Six surface outfall drainage systems were modeled in the City of Canby. Four additional systems were modeled for proposed CIP projects or anticipated futures conditions. Basic modeled-system characteristics are presented in Table 5.1 and model results are described briefly below. Much of the City is serviced by UICs connected to relatively short pipe segments or other small private systems, these areas are also presented in Table 5.1. The basins managed by ODOT along Hwy 99E drain to surface outfalls but were also not modeled. A complete basin specific map for each model and detailed model inputs and results are included in Appendix A.

Table 5.1: Modeled Basins

Basin Name	Description	Area (Acres)	Outfall
Canby Downtown	Existing system discharges to detention pond	48.9	Detention pond, swale to Molalla River
Canby Downtown CIP	Increase pipe sizes, add basin for decommissioned UICs	52.3	“
N Baker Dr	South end of N Baker Dr	2.1	Molalla River
Knights Bridge Rd	Existing system	3.5	Molalla River under bridge
Knights Bridge Rd plus N Baker St	Add pipe to collect N Baker St	4.3	“
Knights Bridge Rd and N Baker Rd plus NW 9th Ave	Add pipe to connect slow draining UICs on NW 9th and increase mainline pipe size	7.8	“
S Berg Parkway	Small basin in SW of City	3.2	Molalla River
North Maple St	Both agricultural and impervious area, includes area outside UGB	123 agricultural, 5.6 impervious 128.6 total	Willamette River
Redwood/Willow Creek	West of Willow Creek, including Territorial Rd	22.2	Willow Creek
10th Ave and Pine St to NE Territorial Rd plus Redwood/Willow Creek	Adds N 10th Ave and Pine St through to Territorial Rd	31.5	“
Redwood/Willow Creek plus N Pine St and N Redwood Rd New Development	Adds right-of-way for new residential development on available parcels	41.1	“
TOTAL MODELED	**EXISTING**	**209**	**Surface**
TOTAL MODELED	**FUTURE**	**239**	**Surface**
NOT MODELED	**UICs/Private/ODOT**	**3158**	**Infiltration-Surface**

5.4.1.1 Canby Downtown

This storm system drains the downtown commercial area and is the oldest part of the City of Canby's stormwater infrastructure. The discharge for this basin is to the Molalla River in the western part of the community. The area of the existing basin as modeled is 48.9 acres. There is an area within this basin in the vicinity of Wait Park that drains to drywells or infiltrates, and this area is not included as part of the basin. The outfall from the Canby downtown basin is a surface water discharge point via a detention pond, grassy swale, and natural drainage way adjacent to the new police station at the west end of NW 3rd Ave.

The model shows surcharging and flooding within the system due to shallow slopes in the existing pipe system and reverse slopes on the branch along N Ivy St. The mainline pipes along NW 3rd Ave were modeled as 30-inch diameter to reduce the flooding represented for existing conditions to a reasonable level, although other evidence suggests they might only be 20-inch diameter. There may be significant exfiltration through failing pipe systems that is not captured by the model.

5.4.1.2 Canby Downtown CIP

This model of a CIP adds an additional drainage basin on the south side of Wait Park to account for the decommissioning of drywells in the area. Pipe sizes are increased in several locations to eliminate flooding and minimize surcharging. The pipe segment on N Fir St between NW 3rd Ave and NW 2nd Ave is eliminated and a new pipe is added on NW 3rd Ave between N Fir St and N Cedar St.

5.4.1.3 N Baker Drive

The N Baker Dr basin drains to a pipe outfall that discharges runoff collected from the south end of N Baker Dr, NW 6th Pl, and NW 6th Ave. The area of the existing basin, as modeled, is 2.1 acres. The outfall is near the steep bank on the Molalla River, upstream of Knights Bridge Road. There is limited data for the conveyance system in this area and there may be additional outfalls that are not mapped. CIP #1 addresses issues in this basin. All pipes were assumed to have a 10-inch diameter and there were no issues identified in the modeling of this basin, except that one pipe has a reverse slope.

5.4.1.4 Knights Bridge Rd

This system drains approximately 1500 feet of Knights Bridge Road and immediate areas of intersecting streets. The area of the existing basin, as modeled, is 3.5 acres. The outfall of this storm system is a pipe located underneath the bridge on which Knights Bridge Rd crosses the Molalla River. The pipe is supported by the bridge structure underneath the roadway and has a free outfall approximately 30 feet above the Molalla River, above the approximate centerline of the river. The conveyance system includes a Contech Model CDS2015-4 hydrodynamic separator for particulate removal located at approximately N Ash St at the second-last manhole before the discharge point. The existing system model shows the system is adequate.

5.4.1.5 Knights Bridge Rd plus N Baker Rd

This model adds approximately 700 feet of N Baker Dr to the Knights Bridge Rd system. The new basin area is approximately 0.8 acres. The design standard 12-inch pipe was used and the model showed no other changes were needed.

5.4.1.6 Knights Bridge Rd and N Baker Rd including NW 9th Ave

This model adds another 3.5 acres to the system by connecting four slow-draining UICs on NW 9th Ave through a pipe along N Aspen St. The model shows that the Knights Bridge Rd mainline to the outfall needs to be increased from 12 to 18 inches to prevent flooding. However, no infiltration was assumed through the existing UICs and these would be expected to reduce peak flows somewhat. UIC flow modeling was beyond the scope of this plan; however, it should be completed to accurately determine the future required pipe size along Knights Bridge Rd.

5.4.1.7 S Berg Parkway

This conveyance system drains the 3.2 acres of SW Berg Parkway to an outfall near the Molalla River. Pipes were assumed to be 10-inch diameter and no issues were identified by modeling of this basin.

5.4.1.8 North Maple Street

This system collects stormwater in a series of daisy-chained UICs along Maple Street, which is located in the northern part of the City, which then combines with runoff from adjacent agricultural land and discharges directly to the Willamette River beyond the end of NE 34th Pl. The existing impervious area of this basin is 5.6 acres, and the contributing agricultural area is 123 acres. The system discharges through a deeply buried pipe at approximately the water surface elevation of the Willamette River.

The model of this system demonstrated slight surcharging and flooding of the manhole located adjacent to NE 31st Pl, with the remainder of the system functioning well. However, no infiltration was assumed through the existing UICs and which would be expected, and would reduce peak flows. In addition, the City has not observed any flooding problems in this basin, so it assumed the conveyance system is adequate. If more accurate flow rate estimates are required for future sizing of potential treatment devices in this basin, the reduction in flow rate due to the existing UICs should be considered in the model.

5.4.1.9 Redwood/Willow Creek

This system drains N Redwood St and adjacent side streets from NE Territorial Rd to Hwy 99E, a portion of the Molalla Forest Rd, some of the residential streets on the east side of N Pine St and NE Territorial Rd from N Maple St to Willow Creek. The modeled existing drainage basin is 22.2 acres. This system's outfall is to Willow Creek on the north side of NE Territorial Rd where it combines with flow from the main stem of Willow Creek and flows onto the Fish Eddy property. No issues were identified in the modeling of this system and it appears to be adequately sized.

5.4.1.10 N 10th Ave and N Pine St to NE Territorial Rd plus Redwood/Willow Creek

This model adds basins, manhole junctions and pipes to represent development of N 10th Ave and N Pine St and decommissioning of one UIC in the area. The additional basin area is 9.3 acres. Flows are piped along N Pine St to Territorial Rd. The model shows the existing system has adequate capacity for the new flows that are proposed to be added to the existing pipe along NE Territorial Rd.

5.4.1.11 Redwood/Willow Creek plus N Pine St and N Redwood Rd New Development

This model adds conceptual basins to account for runoff from potential future development. Pipes are added along N Pine St to connect to NE Territorial Rd and new connections are added along Redwood Rd from the basins to the east. The additional drainage area is 11.7 acres and the model shows the system has adequate capacity for the additional runoff.

5.1 Water Quality Facility Sizing Criteria

The design criteria for water quality facilities are specified in the City of Canby Public Works Design Standards for storm drainage design in Section 4.310 c. The standards require that water quality facilities be designed to the criteria in the Clean Water Services (CWS) Design Manual Chapter 4, paragraph 4.06. City of Canby design standards also required that overflow or bypass be provided for the storm events from the 25-year to 100-year 24-hour storm event such that the facility does not over-top or exceed the capacity of the overflow.

Chapter 4.06 of the CWS Design and Construction Standards specify that a wetland should have a permanent pool volume equal to 0.55 times the water quality volume (WQV), a detention volume equal to 1.0 times the water quality volume, and a drawdown time of 48 hours. The water quality volume is based on “a dry weather storm event totaling 0.36 inches of precipitation falling in 4 hours with an average storm return period of 96 hours.” The calculation for the required WQV used by CWS is 0.36 inches times the impervious area draining to the facility, with the amount of impervious area depending on the size and type of new development. The water quality flow rate (WQF) is calculated as the WQV divided by 14,400 seconds (equivalent to four hours). CWS also specifies other detailed design criteria that should be followed during wetland design.

Through discussion with CWS staff, it was determined that the above criteria and design guidelines are not applicable to a large regional treatment facility, and no alternative method has been developed by CWS for modeling the above specified design storm. Water quality facility evaluations were, therefore, completed using the SSA model and a 6-month 24-hour Type 1A SCS design storm, which was assumed to be 1.91 inches of rainfall or 72% of the 2-year 24-hour storm. This is a commonly used criteria for sizing water quality facilities and is expected to treat approximately 90% of annual stormwater runoff. The total volume flowing to the treatment location for the above storm is assumed to be the water quality volume.

5.2 Existing Detention Pond Capacity

The volumetric capacity of the existing detention pond below the west end of NW 3rd Ave near the police station was estimated based on field observations and satellite imagery. The

approximate depth of the pond was observed to be four feet, measured from the high water mark to the water surface. The water surface was observed on 14 February 2013 to cover approximately one quarter of the pond bottom only a few inches deep, in the lowest southeast corner of the pond, although the pond bottom appears to be relatively flat and level. The four-foot water depth in the pond would be controlled by the invert elevation of the swale leaving the east end of the pond, and ultimately by the three 36-inch culverts at the end of the swale. However, infiltration appears to be significant because the observed water level was significantly below this control point during the rainiest part of the year. The approximate surface area of the pond at the four-foot depth, digitized on satellite imagery, is 18,000 square feet. The approximate volume of the pond, as it is currently functioning, is therefore 72,000 cubic feet (1.7 acre-feet).

5.2.1 Detention Pond Capacity Analysis

The Canby Downtown stormwater models previously discussed in Sections 5.4.1.1 and 5.4.1.2 were used to analyze the detention pond capacity. Model assumptions for the pond and swale were based on visual observations and photographs obtained during one site visit and anecdotal reports of pond performance. Because of the many assumptions made to develop the model, results should be interpreted with caution; however, they generally indicate that the pond has sufficient capacity to convey the design storms for current and future conditions, and provides a level of water quality treatment.

A storage node representing the pond was added to the models. The pond was assumed to have an area of 18,000 square feet from an elevation of 110 feet to a maximum depth of 6 feet. The swale was assumed to leave the pond at elevation 113, have a bottom width of 10 feet, a depth of 3 feet with 4 to 1 side slopes, and a length of 625 feet. The swale was assumed to have a relatively flat slope so the inverts of the 3 culverts at the end of the swale were assumed to be at elevation 112.9. The road surface was assumed to be 3 feet above the top of the culvert. Refer to Appendix D for detailed model inputs and results.

Infiltration was not generally considered during modeling for this stormwater master plan, but because the modeled detention pond would contain a permanent pool, which has been observed not to occur, an exfiltration (equivalent to infiltration out of the pond) rate was assumed for the pond area. The assumed exfiltration rate was based on the design long term infiltration rate recommended by GeoPacific Engineers following infiltration testing in May 2003 at the Valentine meadows development located at NW Cedar and NW 3rd Ave. The exfiltration rate for the pond was modeled as 3.5 inches per hour, which is the recommended rate of 7 inches per hour divided by a safety factor of 2.

The previously discussed conveyance criteria of designing to provide capacity to pass a 25-year storm event with surcharge but without flooding was used as a capacity criteria for the pond. Specifically, the criteria was to keep the hydraulic grade line below the road surface at the three 36-inch culverts. For the detention pond, this criterion was also checked for the 100-year storm. The culverts do not flow full, and the freeboard is not less than 4.5 feet for any of the modeled scenarios, indicating that the pond has excess capacity. Refer to Appendix D for model results.

The previously discussed criterion for water quality volume is that detention facilities be designed to contain the volume of the 6-month 24-hour storm. For existing conditions at the

detention pond, this volume is 326,200 cubic feet, which is larger than the calculated pond volume. However, additional treatment volume and performance is provided by the swale through storage and vegetative filtration etc. Also, a significant portion of the runoff volume entering the pond is observed to infiltrate (the model indicates approximately 60% of the WQ storm exfiltrates for existing conditions), and, therefore, also receives treatment. More complex modeling would be required to accurately represent the true water quality benefit of the pond and swale, and this would require additional data collection beyond the scope of the current stormwater master plan.

5.3 Proposed Fish Eddy Wetland Sizing

A stormwater treatment wetland is proposed on the Fish Eddy property at approximately 1570 NE Territorial Rd and adjacent to the existing Willow Creek. This treatment wetland is part of a restoration of the entire Fish Eddy property to native seasonal wetland and wet prairie habitat as part of the Willamette Wayside Properties SWMP. The current plan estimates a total of 1,435,000 square feet (33 acres) is to be restored on the property, including the area for the treatment wetland.

5.3.1 Treatment Wetland Drainage Basins

The stormwater treatment wetland is sized to treat stormwater runoff from properties south of Territorial Rd, including Willow Creek. The wetland is sized to treat the existing conditions, plus any new impervious right-of-way, based on the assumption that new development will maintain discharges at or below existing levels. The watershed draining to Willow Creek at the proposed wetland location is approximately 393 acres and is estimated to be approximately 25% impervious when fully built out. The modeled subbasins used for sizing the wetland are shown on Figure 6 and described below.

The existing piped system discharging to Willow Creek on the north side of NE Territorial Rd drains areas including the existing piped systems along N Redwood Rd and NE Territorial Rd, the proposed piped system for NE 10th Ave and N Pine St, and potential new development at the north end of N Pine St and the west side of Redwood Rd. These subbasins total approximately 31 acres and are assumed to be 100% impervious. The existing Willow Creek subdivision, the undeveloped and rural area along Willow Creek between NE Territorial Rd and Hwy 99E, and the north side of Hwy 99E drain through Willow Creek to the Willow Creek Wetlands. These areas are modeled as a single basin of 120 acres that is 15% impervious. The Willow Creek wetlands discharge through a weir structure to two 36-inch culverts under NE Territorial Rd to join runoff from the piped systems on NE Territorial.

The north fork of Willow Creek also receives runoff from the basin, southeast of Hwy 99E via an existing culvert under the highway. This basin is approximately 142 acres and assumed to be 5% impervious. The south fork of Willow Creek receives runoff from two basins southwest of Hwy 99E, through an existing private detention pond located south of Hwy 99E. The first is an approximately 72 acre and 40% impervious piped system draining the privately owned commercial area adjacent to Hwy 99E. The second basin is the ODOT and County pipe and ditch system that drains S Ivy St and a portion of Hwy 99E. This basin is 18 acres and assumed to be 95% impervious.

5.3.2 Conceptual Treatment Wetland Size

The SSA model inputs and results are presented in Appendix E for the 6-month 24-hour water quality storm. The modeled flow at the Willow Creek outfall to the proposed Fish Eddy Treatment Wetland averages 6.7 cubic feet per second (cfs) and has a peak of 18.8 cfs. The modeled total runoff volume to this outfall, which is the water quality volume, is 814,000 cubic feet. Note that because of limited data, assumptions concerning channel dimensions, outlet structures, etc. were made based on satellite imagery and limited photographs or observations in the field while developing the model and basin characteristics. These estimates are conceptual only and should be refined prior to further design.

The depth of a stormwater treatment volume varies throughout the wetland, with deeper areas in the forebay and varied depths of the permanent pool to promote different zones of treatment and diverse vegetation; however, for conceptual sizing, the average depth can be assumed to be approximately three feet. Dividing the above water quality volume by three feet gives a required wetland surface area of approximately 6.2 acres. With the required side slopes, perimeter areas that are expected to flood during large storm events and additional volumes for sediment retention, the approximate disturbed area required for construction of the treatment wetland is 12 acres, or approximately two times the required surface area for treatment.

It is recommended that a flow monitoring station be established in Willow Creek North of NE Territorial Rd to characterize the rainfall runoff response for the existing watershed for one or two seasons prior to designing the treatment wetland. The collected data can be used to calibrate the hydrologic and hydraulic model so it can be used to size the wetland to be an efficient and cost-effective stormwater treatment facility.