# Lummi Nation

# CHIEF MARTIN RD
# UTILITIES MASTER PLAN

**July, 2024**

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# ACKNOWLEDGMENTS

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# EXECUTIVE SUMMARY

## 1. Introduction

The purpose of this plan is to provide a comprehensive evaluation of the water, wastewater, stormwater, transportation, and power supply relevant to development of the Chief Martin Rd corridor. This plan evaluates the capacity of utilities relative to the total planned buildout growth for moderate-density and high-density scenarios. Water and wastewater capacities were evaluated in the greatest detail, which included computer modeling of the water distribution network and the sewer collection network. This study identifies the recommended system improvements that are needed to meet the existing and future demands. Reasonable order of magnitude (ROM) cost estimates are provided, where appropriate, for recommended improvements. All ROM costs are total project costs, which include construction costs, design and permitting services costs, and construction administration costs.

The planned buildable areas maps (Figure 1-1 and 1-2) provide low-density and high-density scenarios for build-out growth. Figure 1-2 includes land currently zoned Forest. in both an extended scenario where the parcels to the East of Chief Martin Rd have been extended and a non-extended scenario. The range of additional dwelling units is 777 to 3,717. The actual buildout density is likely to fall somewhere in the middle of these estimate. The utilities evaluations assumed the highest density scenario, so that it would be certain that utilities would not be undersized. Following is a summary of the discussions and recommendations for each major utility system studied.

## 2. Water Distribution System

The tribal water system has undergone improvements since the last Water system Plan was completed in 2007. However, there are many improvements in the system that need to be made to accommodate future growth. The full extent of those improvements will be presented tin the next Water System Plan. This plan presents needed improvements directly related to the Chief Martin Rd development. Of most significance is that, based on the 2015 capacity report, the peninsula water system lacks the water supply needed to support full buildout of the corridor. Additional groundwater or surface water source (e.g., Nooksack River) will be needed.

In this evaluation water distribution system was analyzed using computerized hydraulic modeling for existing and future buildout conditions, including “peak hour demand” and “fire flow capacity.”

Chief Martin Rd water system infrastructure should consist of the following:

- New 12-inch Water Main running the entire length of Chief Martin Rd from Kwina Rd to Cagey Rd.
- New Hydrants every 500 feet along Chief Martin Rd.
- A new 250,000-gallon storage reservoir and booster station on Chief Martin Rd. The booster station would supply a 300-ft pressure zone for parcels above 160-170 ft in elevation in the vicinity of the new reservoir and booster station. An 8-inch water main for the 300-ft pressure zone would parallel the 12-inch, 230/240-ft pressure zone water main.
- Booster station upgrades at the Mackenzie and Kinley Storage Reservoirs.

The capacity of the existing peninsula water distribution system:

- System is adequate to provide at least 30 PSI at most locations on the peninsula.
- For fire flow, the system is inadequate to satisfy all commercial and residential fire flow requirements. Fire flow capacity is quite variable across the peninsula.
- At the high population growth level scenario, the fire flow capacity is reduced systemwide. Planning for upgrades to system bottlenecks is needed.

The capacity of the existing water distribution system to supply water to Chief Martin Rd:

- Existing system operating pressure is not high enough to provide the minimum 30 PSI at higher elevations of Chief Martin Rd. A Booster Station is needed at the high elevation location.
- The Chief Martin Rd water main will greatly improve the fire flow for the northern half of the peninsula. For example, Kwina Rd fire flows of about 600 GPM increase to about 1800 GPM.

Total estimated cost for buildout water system improvements:

$6,450,000 to $7,740,000

# 3. Sanitary Sewer

The Lummi Peninsula is split into multiple sewer basins, but for the purpose of this report only the Kwina Point Basin and Gooseberry Point Basin were analyzed. The Kwina Point WWTP is near capacity with a total operation capacity of 100,000 gallons/day (GPD), but it is only expected to need to handle the additional flows from the Turkey Shoot II development and proposed Lummi Utility Building. The Gooseberry Point WWTP has recently been expanded and upgraded to 0.375 MGD average annual flow.

Chief Martin Rd sewer system infrastructure should consist of the following:

- About 2,500 LF of gravity sewer flowing south to Kwina Rd on Chief Martin Rd.
- A new pump station south and west of Scott Rd with about 10,000 LF of 8-inch to 21-inch sewer mains in Chief Martin Rd..
- 7,620 LF of 15-inch force main flowing from the pump station south along Chief Martin Rd.
- 6,050 LF of 21-inch gravity main to connect to the Lummi Shore Eastern Interceptor. The last 4,200 LF of this connection would require a new sewage corridor easement acquisition.
- Other sewer mains to serve developments adjacent to Chief Martin Rd.

The sanitary sewer collection systems for the Gooseberry Point Basin and Kwina Basin both have capacity constraints that need to be addressed as the population grows.

- The eastern interceptor along Lummi Shore Dr and pump stations located south of Smokehouse Rd will need be upgraded to accommodate all the increased flows coming from Chief Martin Rd.

- All downstream pump stations and piping on Lummi Shore Drive will need to be upgraded to handle buildout flow from the Chief Martin corridor. The extent and timing of theses upgrades will require further evaluation. Design and construction of these upgrades would occur in phases as growth proceeds.

- Gooseberry Point WWTP will need a capacity upgrade by the time 565 ERUs are added.

Total estimated cost for buildout sewer system improvements:

$22,600,000 to $29,800,000

# 4. Stormwater

Building out Chief Martin Road will result in a minimum of 600,000 square feet (13.5 acres) of impervious area within the 60-ft ROW. Runoff from this area will be collected via a catch basin and storm pipe system located at the curb and gutter of Chief Martin Rd. Collected runoff will flow to the four different low points. At each of these low points, stormwater treatment and detention are needed. The most practical and economical method to achieve the two goals is to install combined detention ponds. A combined detention pond includes a permanent pool of water and a “live” storage volume above the permanent pool, and typically one-foot of freeboard. The design permanent pool water treatment volume is equal to the six-month storm volume. The detention volume is equal to approximately 8” of rain spread out over the impervious area plus the runoff from the pervious area.

Each of four drainage areas (labeled herein as south, south middle, north middle and north) would each have a combined detention pond. The location and approximate size of each of the four combined ponds are shown in the conceptual  plans in the appendices. The South and North ponds would discharge to the existing ditch and storm piping on Cagey Rd and Kwina Rd, respectively. The other two ponds would discharge to the forest via level spreaders to adequately disperse the flow.

Land acquisition or easements will be required for the ponds. Alternatively, stormwater flow control within the road right of way would be very challenging, and more costly, to implement,. However, it would be feasible.

Other stormwater management approaches should be further investigated after more detailed investigations of soils and hydrologic conditions. Flow control methods could include roadside bioretention, infiltration basins, infiltration trenches, or roadside direct dispersion into dedicated forested land. Treatment methods could include bioretention, vegetated filter strips, proprietary filters vaults, or proprietary modular (vaulted) wetlands.

The stormwater piping and catch basins are sized based on the total peak flow from the roadway and the forested areas. The primary storm drain piping would range in size from 12 inches to 24 inches in diameter. There would be no ditches or culverts.

The estimated cost for stormwater for the entire Chief Martin corridor (excluding roads and developments branching off from Chief Martin Rd) is

$5,500,000.

# 5. Electrical Power

The future buildout peak load is based on what would be expected for 3,000 homes. The peak demand from each home is conservatively 10 KW. Total peak demand would be 30,000 KW. To provide this service would require two circuits of 12,470 volt, 3-phase, 600 amp. Two parallel stepped down circuits of 12,470 volt, 3-phase, 200 amp would be needed. Puget Sound Energy would provide these distribution lines in buried duct banks.

Additional infrastructure would include:

- 600A, 12,470V, 3 phase, Padmount Switches
- 600A primary vaults (approx. every 800')
- 200A primary vaults (approx. every 400')

Service into each neighborhood would be 200A.

The estimated cost for power, on Chief Martin Rd only, for 3,000 new residences is:

$32,000,000.

The estimated cost for power, on Chief Martin Rd only, for 1,500 new residences is:

$17,000,000.

Costs for service into neighborhoods is not included in the cost estimate.

Costs do not include cost that PSE may impose for increasing power supply to the vicinity (this cost is unknown).

# 6. Transportation

Transpo Group prepared a detailed transportation evaluation for the Chief Martin Rd development and adjacent connecting roads. A detailed memorandum is included in the appendices. The analysis of existing and future traffic volumes was used in determining road classification and roadway cross sections for Chief Martin Road for master planning purposes. Transpo considered existing and future traffic volumes, connections with external roadways, and daily and peak hour traffic loads on Chief Martin Road..

1. Widening Kwina Road to four or even five lanes between Haxton Way and Lummi Shore Road.
2. Improving the intersection of Kwina Road, Lummi Shore Road, and Marine Drive.
3. Improving Scott Road to allow an alternative route out of the development area, which may in turn require improvements to Lummi Shore Road north of Scott Road.
4. Constructing new east-west roadways from Chief Martin Road to Lummi Shore Road or Haxton Way.
5. Improvements to the roundabout at Haxton Way/Kwina Road.
6. Internal intersections may require widening or other improvements, such as signalization, to facilitate smooth traffic flow (see below).

For the lower density development scenario (up to 1,200 units added on Chief Martin Rd), a two-lane road with left turn lanes at key points will probably be sufficient. A left turn lane should be installed at the north end of Chief Martin R (at Kwina Rd) when the road is first improved. At later dates, left turn lanes may be needed at Scott Rd and Cagey Rd as well. Chief Martin Road would carry higher volumes than Kwina Road or other area roadways do today. Kwina Road in its existing condition should be able to accommodate the amount of traffic projected in the low-density scenario without additional impacts.

The initial development of Chief Martin Rd would include two travel lanes, bike lanes, sidewalks, utilities, and lighting. The ROW would be widened to accommodate a left turn lane at Kwina Rd.

For planning purposes, once the development exceeds approximately 1,200 units, additional mitigation measures and/or road widening will be needed to accommodate high traffic volumes. For example, Chief Martin Rd could need to be expanded to four lanes in the northern segments to meet traffic demand.

Additional work will be needed to plan for high-density growth. Planning for right of way acquisition should begin early on. Future evaluations of final buildout density should be undertaken as development of the Chief Martin Rd corridor proceeds.

Transit planning should include land acquisition for pullouts or design of methods to safely allow buses to pull over and reenter traffic with obstructing traffic and bike lanes.

When developing plans for individual developments, consideration should be given to sufficient intersection designs. Spacing and number of intersections will be key considerations when plots are created. For instance, in the high-density scenario, a single access intersection for a development of 1,000 homes would not function at acceptable levels with side street stop sign controls. Additional access points might need to be created.

The estimated cost (see appendices for detail) for the Chief Martin Rd construction (not including utilities) is:

$32,000,000

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## 7. Total Buildout Costs

The “planning level” estimated cost (in 2024 dollars) for the full buildout of Chief Martin Rd, assuming only a two-lane roadway but with utilities sufficient for high density growth:

$133,000,000

# 1. INTRODUCTION

## 1.1. *Purpose*

The purpose of this plan is to provide a comprehensive evaluation of the water, wastewater, stormwater, transportation, and power supply relevant to development of the Chief Martin Rd corridor. This plan evaluates the capacity of utilities relative to the total planned buildout growth for moderate-density and high-density scenarios. This study identifies the recommended system improvements that are needed to meet the existing and future demands. Reasonable order of magnitude (ROM) cost estimates are provided, where appropriate, for recommended improvements. All ROM costs are total project costs, which include construction costs, design and permitting services costs, and construction administration costs.

The systems evaluated in this study include: Water distribution (domestic), sanitary sewer, transportation, stormwater, and power. Information on other utilities is not provided as part of this report. Water and wastewater capacities were evaluated in the greatest detail, which included computer modeling of the water distribution network and the sewer collection network.

## 1.2. *Background*

**Previous Studies – Water and Sewer.** The Lummi Water and Sewer District's peninsula's water and sewer systems have each been evaluated two times over the last two decades. The water system evaluations were: (1) Water Facilities Plan (BHC Consultants, 2007) and (2) Source Water Physical Capacity Analysis (Indian Health Service, 2015). The sanitary sewer system evaluations were: (1) Sewer Facilities Plan (BHC Consultants, 2007) and (2) Gooseberry Point Wastewater Treatment Plant Facility Plan (Gray and Osborn Inc., 2017). Little information is available for the stormwater or power utility.

**Residential Development Capacity.** The buildable areas map (Figure 1-1) and the extended buildable areas (extends into the area zoned Forested) map (Figure 1-2) provide a framework and total number of lots that may be developed within the wetland constraints according to planning conversations between Wilson Engineering and the LIBC. All of the projected growth will be based on this mapping. Growth can be divided into multiple build-out phases as needed. These buildable areas maps show the potential number of housing units in each subarea for medium density (3 units per acre) and high density (9 units per acre).

## 1.3. *Master Plan Area*

The plan area is limited to the Chief Martin Rd corridor from Kwina Rd to the South where it meets Cagey Rd. The buildable areas map in Figure 1-1 shows the total area that may be developed, excluding wetlands, according to planning conversations between Wilson Engineering and the LIBC. The extended buildable areas map in Figure 1-2 includes additional area extending into the area zoned Forest. Figure 1-3 shows the zoning, land ownership type and wetlands all on one map in order to show the overlap between these land classifications. A detailed 39 sheet conceptual plan for the full construction of a Chief Martin Road corridor is included in the Appendices. This plan shows the full-width road improvements and all of the utilities for a two-lane road corridor. The plan set used Lidar-derived topographic mapping to design the road elevations and longitudinal slopes of the road and utilities.

## 1.4. *Master Plan Area Zoning and Land Ownership*

**Zoning.** The majority of the area is zoned residential. Mixed use land is located at the north end of the corridor and is already developed. Land zoned as Forest is included in the "extended" Master Plan area shown in Figure 1-2, under the assumption that the zoning could be changed in the future.

**Land Ownership.** There are four types of land ownership in the Master Plan Area

1. Individual Native Trust
2. Native Trust
3. Tribal Fee
4. Fee

The largest type (by area) is Individual Native Trust, which is also the most difficult to develop due to the diluted ownership. The other three ownership types are the most likely to be developed in the near future. For example, there are four large fee parcels in the middle of the Master Plan Area – these could be developed by the owners or purchased by LIBC to be developed.

## 1.5. *Recent Growth and Planned Upcoming Growth*

Planned buildings and substantial recent growth include the following:

- Turkey Shoot Phase II – 14 Lots (mixed use zoning, Tribal Trust
- Lummi Utilities Building (Tribal Trust)

## 1.6. *Future Growth*

The planned buildout maps present medium-density growth and high-density growth scenarios for future buildout along Chief Martin Rd. Figures 1-1 and 1-2 show the growth in residential units per acre of new development (medium density and high density) that can possibly occur over the next 30 years.

**Residential Development Capacity.** The buildable areas map (Figure 1-1) and the extended buildable areas (extends into the area zoned Forested) map (Figure 1-2) provide a framework and total number of lots that may be developed within the wetland constraints according to planning conversations between Wilson Engineering and the LIBC. All of the projected growth used in the utility capacity and transportation analyses will be based on this mapping. Growth can be divided into multiple build-out phases as needed. These buildable areas maps show the potential number of housing units in each subarea for medium density (3 units per acre) and high density (9 units per acre). The high-density scenario is unlikely based on past building history; however, utility capital improvement planning should consider the high growth scenario so that utilities infrastructure is sized appropriately (so that future upgrade replacement would not be needed).

The total anticipated buildout increase is 777 at 3 units/acre and 2,330 at 9 units/acre (Figure 1-1). In Figure 1-2, Forest zoned land is included and buildout increase is 1239 at 3 units/acre and 3,717 at 9 units/acre.

In this study, the utilities are sized for the high-density buildout (9-lot/acre density).

The high-density scenario is unlikely to happen do to (1) the difficulty in dividing up Individual Native Trust lands, (2) high density development is not popular, and (3) providing roads with effective traffic conveyance for high density will be difficult. Also, it might take 50 years to achieve those high number of additional dwelling units, even if it could be done. However, the high-density scenario provides an upper limit baseline with which to plan and design utility improvements. For example, sewer and water pipes have a long life span and thus should be sized to accommodate growth over 50 years.

## Table 1-1. Buildout Growth by Subarea and Housing Density

| | Buildable Area | | | | Low Density | | | | | | High Density |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Area | ac | units/ac | 1 /ac | 2 /ac | 3 /ac | 4 /ac | 5 /ac | 6 /ac | 7 /ac | 8 /ac | 9 /ac |
| 1 | 111.5 | Units | 112 | 223 | 335 | 446 | 558 | 669 | 781 | 892 | 1004 |
| 2 | 3.3 | Units | 3 | 7 | 10 | 13 | 16 | 20 | 23 | 26 | 29 |
| 3 | 5.2 | Units | 5 | 10 | 16 | 21 | 26 | 31 | 36 | 41 | 47 |
| 4 | 20.4 | Units | 20 | 41 | 61 | 81 | 102 | 122 | 142 | 163 | 183 |
| 5 | 2.3 | Units | 2 | 5 | 7 | 9 | 12 | 14 | 16 | 18 | 21 |
| 6 | 44.5 | Units | 45 | 89 | 134 | 178 | 223 | 267 | 312 | 356 | 401 |
| 7 | 80.0 | Units | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 8 | 32.0 | Units | 32 | 64 | 96 | 128 | 160 | 192 | 224 | 256 | 288 |
| 9 | 39.8 | Units | 40 | 80 | 119 | 159 | 199 | 239 | 278 | 318 | 358 |
| | | | | | | | | | | | |
| **Total** | **338.9** | **Units** | **259** | **518** | **777** | **1035** | **1294** | **1553** | **1,812** | **2,071** | **2,330** |
| Including Area Extending into Forest Zone | | | | | | | | | | | |
| Area | ac | units/ac | 1 /ac | 2 /ac | 3 /ac | 4 /ac | 5 /ac | 6 /ac | 7 /ac | 8 /ac | 9 /ac |
| 1 | 151.2 | Units | 151 | 302 | 454 | 605 | 756 | 907 | 1058 | 1210 | 1361 |
| 2 | 3.3 | Units | 3 | 7 | 10 | 13 | 16 | 20 | 23 | 26 | 29 |
| 3 | 5.2 | Units | 5 | 10 | 16 | 21 | 26 | 31 | 36 | 41 | 47 |
| 4 | 63.4 | Units | 63 | 127 | 190 | 254 | 317 | 380 | 444 | 507 | 571 |
| 5 | 2.3 | Units | 2 | 5 | 7 | 9 | 12 | 14 | 16 | 18 | 21 |
| 6 | 115.9 | Units | 116 | 232 | 348 | 464 | 580 | 696 | 812 | 928 | 1043 |
| 7 | 80.0 | Units | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 8 | 32.0 | Units | 32 | 64 | 96 | 128 | 160 | 192 | 224 | 256 | 288 |
| 9 | 39.8 | Units | 40 | 80 | 119 | 159 | 199 | 239 | 278 | 318 | 358 |
| | | | | | | | | | | | |
| **Total** | **493.0** | **Units** | **413** | **826** | **1239** | **1652** | **2,065** | **2,478** | **2,891** | **3,304** | **3,717** |

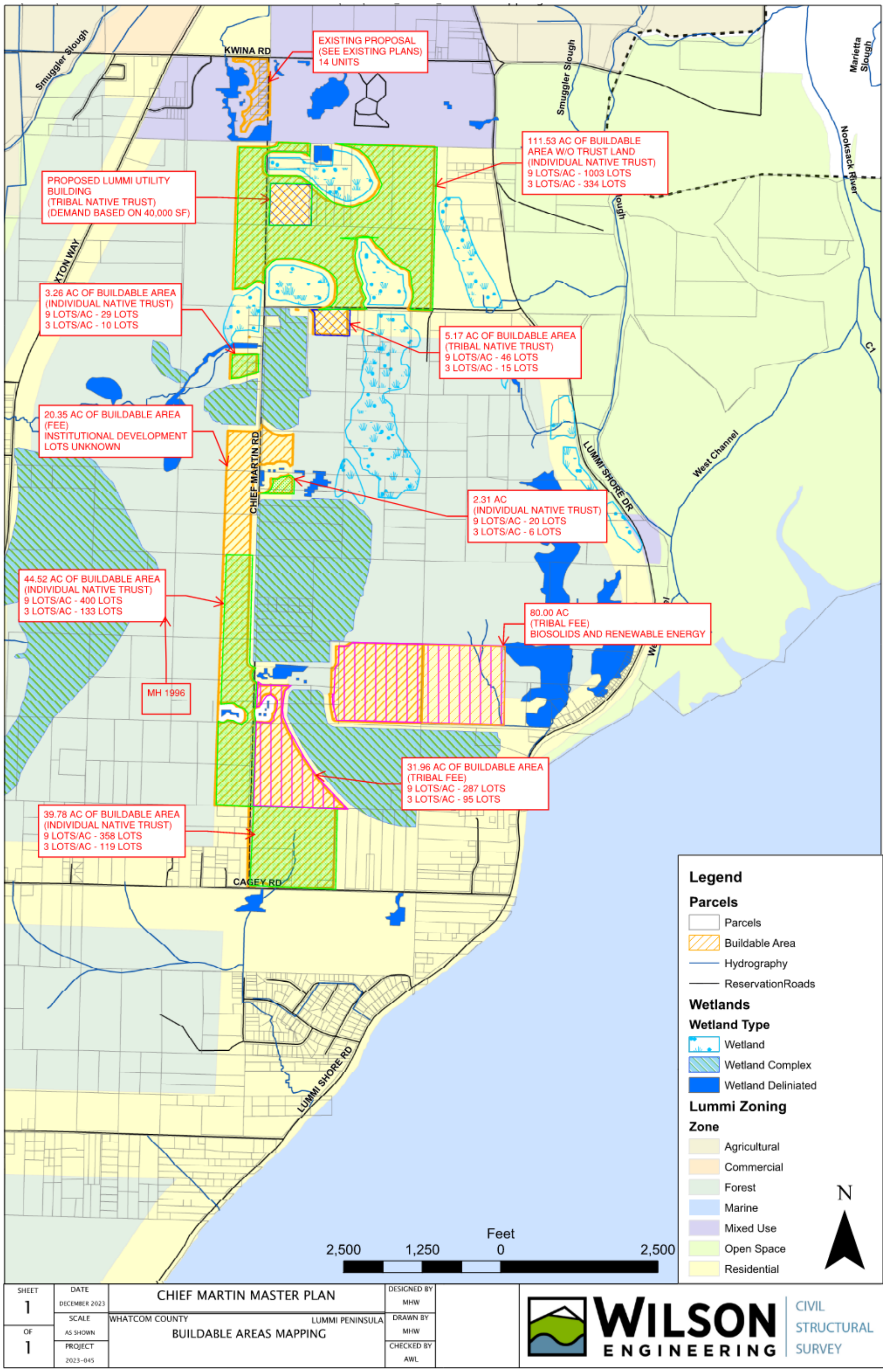



**Figure 1-1. Land Use Map with Projected Growth (dwelling units/acre).**

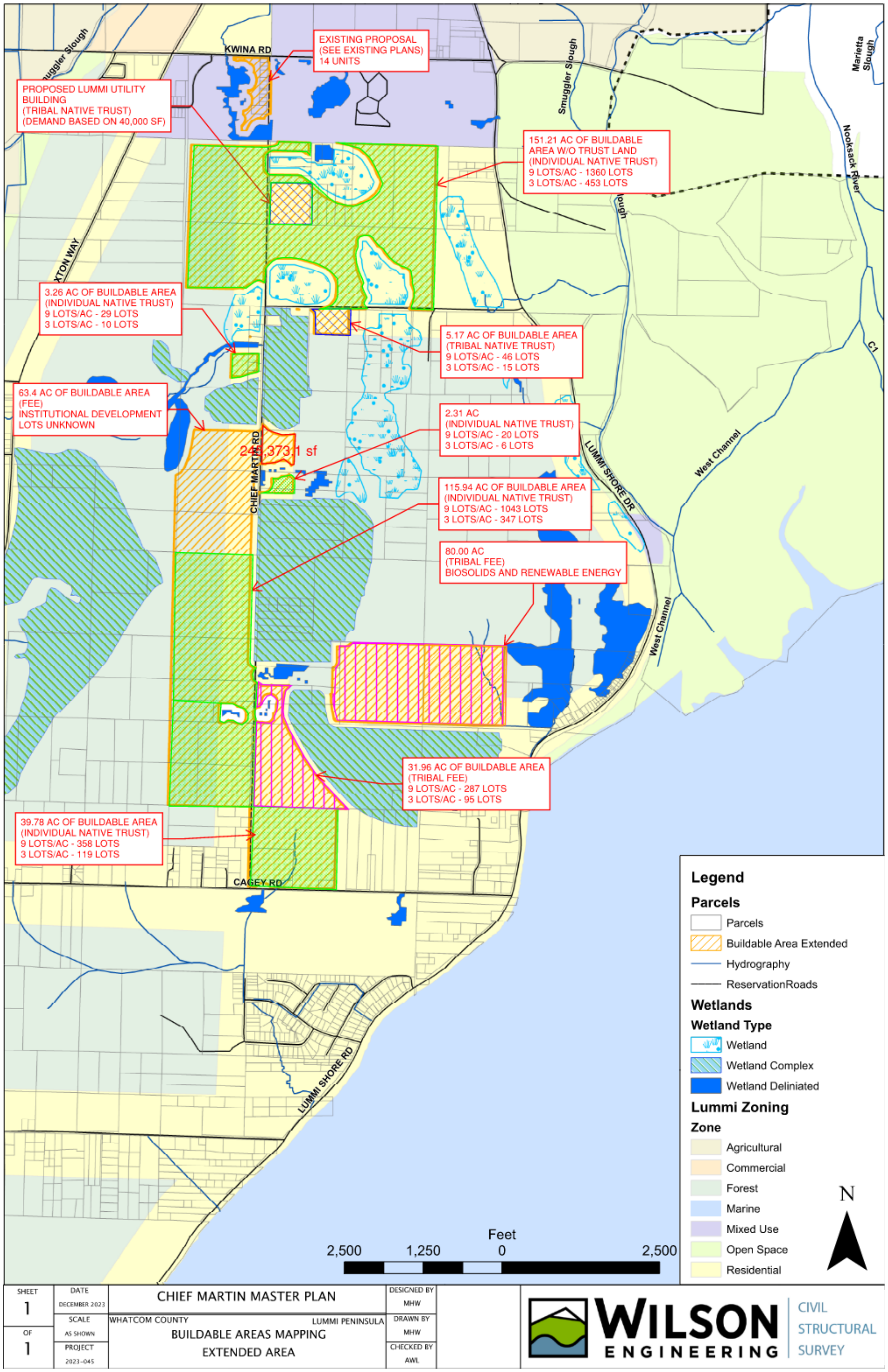



**Figure 1-2. Land Use Map with Extended Projected Growth (dwelling units/acre).**

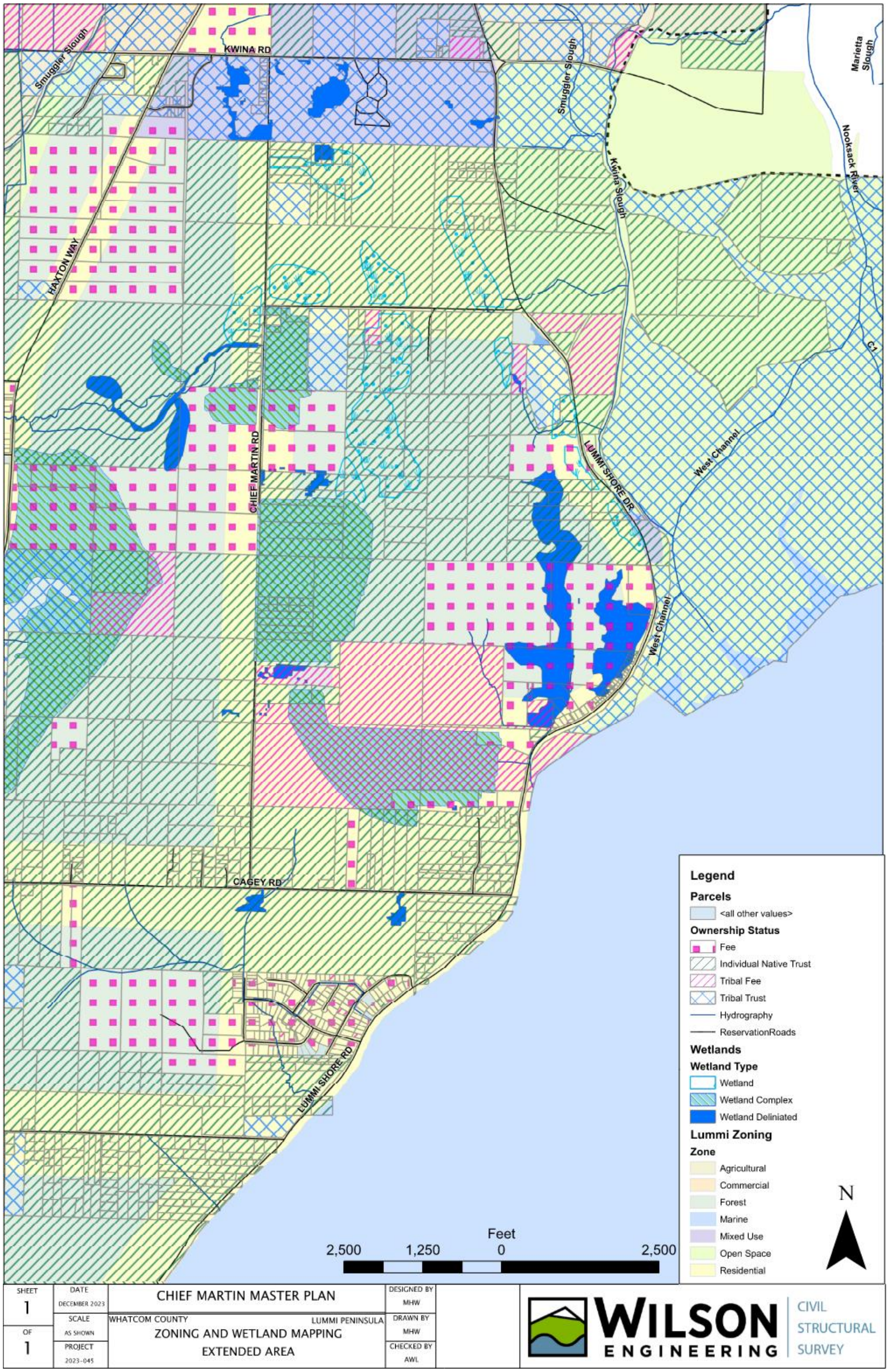



**Figure 1-3. Combined Zoning, Land Ownership Types, and Wetland Locations Map**

# 2. WATER DISTRIBUTION SYSTEM

## 2.1. *Existing System*

### 2.1.1. Description

The water distribution system has had several improvements made since the original study in 2007. Since 2007, the system has been extended and improved with an additional expansion to serve the Silver Reef Casino.

The Lummi Tribal Water System has only one pressure zone (static elevation of 230 ft asl). It feeds the entirety of peninsula with the assistance of an intertie from the City of Bellingham.

The majority of the water distribution system water mains are located on the peninsula south of Slater Rd.

Distribution system losses (DSLs) have been about 15% (of produced water) for the peninsula water system (BHC Consultants, 2007 and Indian Health Service, 2015). In comparison, typical public water distribution system losses average 16% (USEPA, 2013, Water Audits And Water Loss Control For Public Water Systems). For the analysis of the water system capacity, a DSL of 15% (which is consistent with typical water systems) is assumed in the analysis.

### 2.1.2. Lummi Tribal Water Supply

The tribal water system receives its water supply from multiple wells located throughout the peninsula as shown in Figure 2-2 below.

The **total production capacity** of the active peninsula wells is **347 GPM**.

Also provided below in Table 2-1 is pertinent information for each well.

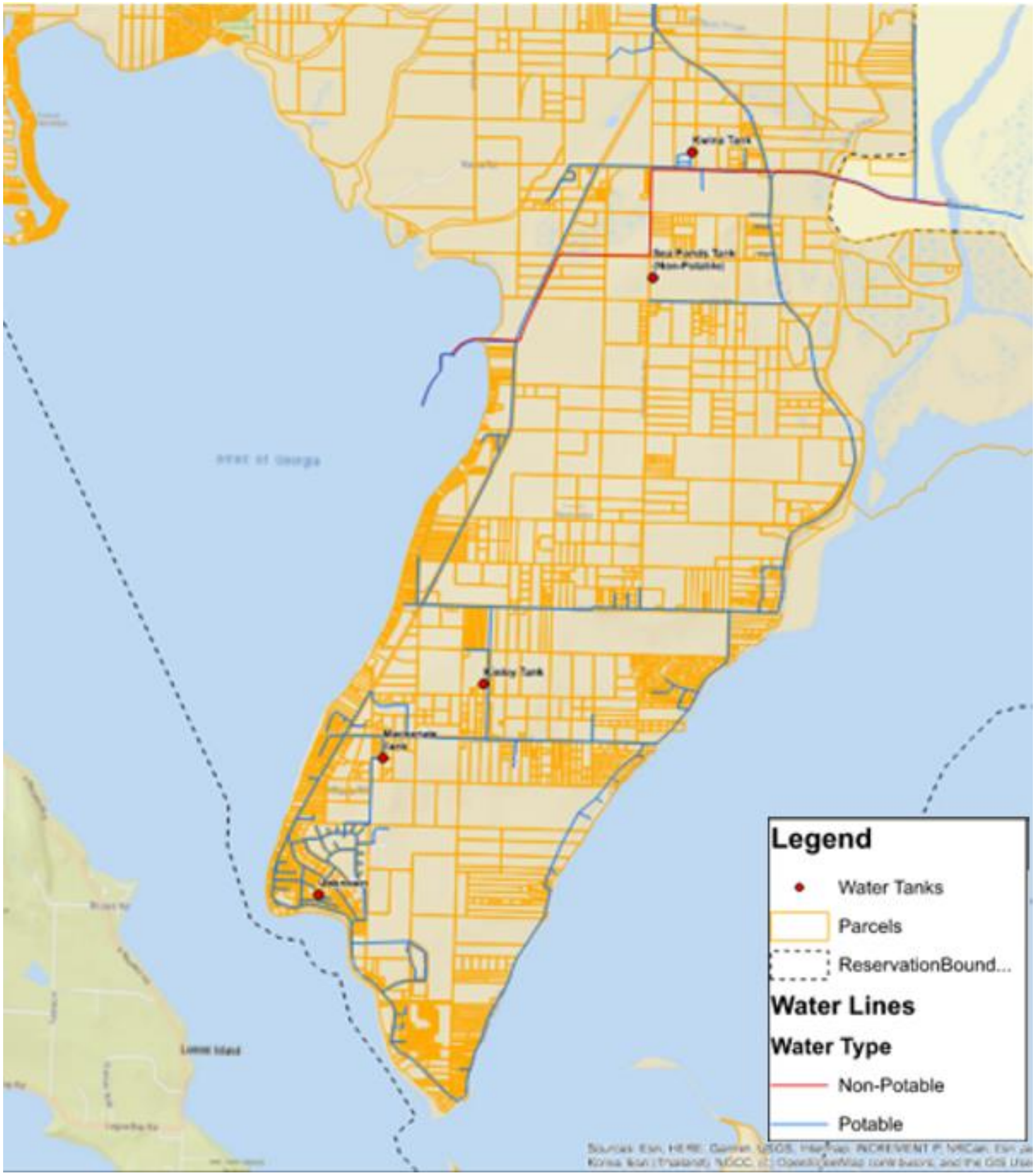



**Figure 2-1. Overall Water System Map**

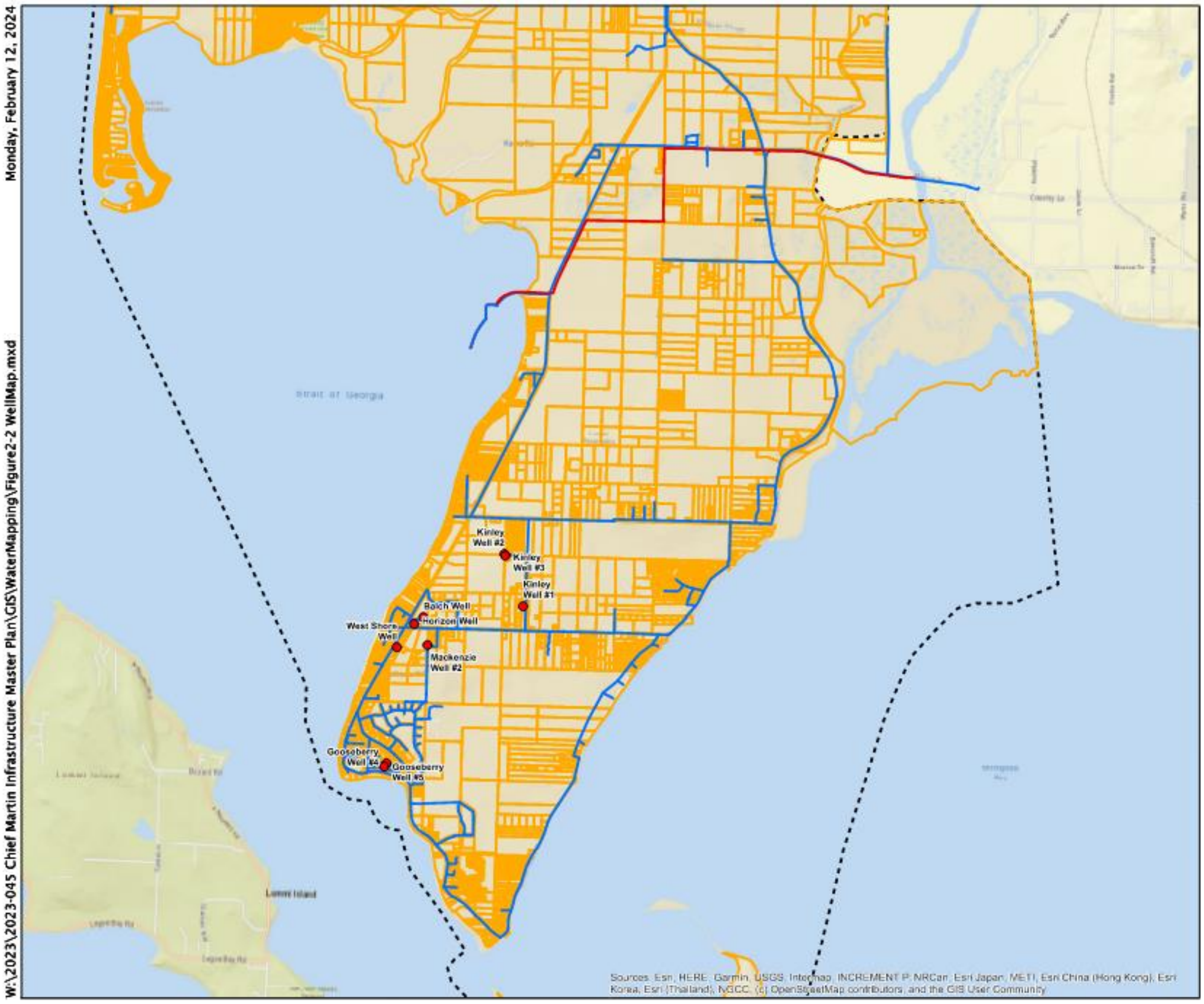



**Figure 2-2. Well Locations.**

**Table 2-1. Well Information Summary (Source: IHS, 20150)**

| Summary of Sources | | | | |
|---|---|---|---|---|
| **Name/Location** | **Date Drilled** | **Pumps from:** | **Pumps to:** | **Pump Capacity** |
| | | | | |
| Balch Well | 1987 | 200' bgs Peninsula Aquifer | Kinley Way Tank | 14 gpm |
| Horizon Well | 1968 | 165' bgs Peninsula Aquifer | Kinley Way Tank | 51 gpm |
| Kinley Well #1 | 1971 | 185' bgs Peninsula Aquifer | Kinley Way Tank | 50 gpm |
| Kinley Well #2 | 1999 | 200' bgs Peninsula Aquifer | Kinley Way Tank | 0 gpm |
| Kinley Well #3 | 2001 | 196' bgs Peninsula Aquifer | Kinley Way Tank | 80 gpm |
| Mackenzie Well #2 | 1987 | 200' bgs Peninsula Aquifer | Mackenzie Tank | 14 gpm |
| West Shore Well #1 | 1994 | 175' bgs Peninsula Aquifer | Northwest Tank | 18 gpm |
| Northwest Well #1 | 2001 | 155' bgs NW Lummi Aquifer | Arsenic Plant | 50 gpm |
| Northwest Well #2 | 2001 | 185' bgs NW Lummi Aquifer | Arsenic Plant | 70 gpm |
| | | | | |
| Vern Johnson Well (LDNR - Fish Hatchery) | 1994 | 185' bgs NW Lummi Aquifer | Arsenic Plant | 55 gpm |
| | | | | |
| Gooseberry Well #4 | 2001 | 125' bgs Peninsula Aquifer | Offline Radium 228 MCL | 0 gpm *(25 gpm)* |
| Gooseberry Well #5 | 2001 | 122' bgs Peninsula Aquifer | Offline Radium 228 MCL | 0 gpm *(25 gpm)* |
| | | | | |
| Bellingham* | | Lake Whatcom | Kwina Tank | 404 gpm |
| | | | | |
| *(bgs) – below ground surface* | | | ** denotes emergency source* | |

Due to low yield, Kinley Well #2 was shut down permanently. Gooseberry Wells #4 and #5 water quality testing found Radium 228 present above the minimum contaminant level (MCL). Due to this, these two wells have also been shut down permanently. The Vern Johnson Well is also used mainly for supplying water to the fish hatchery and is not used as a potable water source. An additional source of water is the intertie with the City of Bellingham, which provides up to 404 GPM of water. This intertie is used intermittently as a water supply source to supplement well water. Due to some pipe and tank (Bellingham) sizing limits, the intertie cannot supply the contracted 1,000 GPM. This has not been an issue, but may need to be resolved in the future.

### 2.1.3. Water Storage

The water system has four tanks, each with a variable effective volume dependent upon on the levels in the other tanks. Currently, the entire system operates around the level of the Mackenzie Water Tank. The two tanks at lower elevations, Kinley and Kwina Tanks, are operated manually, thus there are no pump setpoint on and pump off elevations are noted. In the future, additional booster pumps can be installed at the lower elevation tanks to provide additional storage to supplement water from the higher elevation tanks. Within Table 4 is a summary of the existing tanks and the storage they provide to the system.

**Table 2-2.Tank Information Summary (BHC, 2007)**

| Parameters | Mackenzie | Kinley Way | Kwina Tower | Northwest |
|---|---|---|---|---|
| Ground Elevation | 171.2 | 140.0 | 45 | 210 |
| Elevation at nearest existing service | 150 | 140 | 47 | 170 |
| Tank diameter in feet | 26 | 16 | 30 | 30 |
| Tank height in feet | 60 | 60 | 126 (2) | 60 |
| Maximum Elevation | 231 | 200 | 171 | 270 |
| Elevation to provide 30 PSI | 219 | 209 (3) | 115 | 279 |
| Elevation to provide 20 PSI | 196 | 186 | 92 | 256 |
| Total Volume (gallons) | 226,000 | 90,600 | 125,000 | 317,000 |
| All Pumps Off – elevation | 230.0 | --- | --- | 269 |
| Last Pump On – elevation | 228.5 | --- | --- | 267 |
| Effective Volume (gallons) | 139,000 | 85,000 (1) | 125,000 | 280,000 |

Notes:
1 – Fire pump at tank site allows nearly the entire volume to be used and a jockey pump circulates the volume back to the system for water quality concerns
2 – Bottom of elevated tank water volume is about elevation 151
3 – Maximum water surface will not provide 30 PSI to adjacent residences

The existing water distribution system, including approximate locations of reservoirs, pipes, and wells is shown in Appendices Figure 2-1-3 and in detail in Figures 2-1-4 – Figure 2-1-9. Also, in Figure 2-1-10 are Lummi Nation community buildings, streets, and topography. Attachment 1 shows additional identification and location information for the nodes (junctions) used in the system hydraulic model analysis. Model information including junction pressure zone, demand type, and elevation and pipe diameter, length, installation year, owner, and start.

## 2.2. *Existing Conditions Demand Evaluation*

### 2.2.1. Basis of Analysis

The peninsula water system supplies flows for domestic and non-domestic users belonging to the tribe and for non-tribal users. Metering data exists for each connection type. For all domestic demands it was assumed that peak consumptions for both tribal and non-tribal users occur at the same time. All the demands will be considered together during modeling.

The three measures of domestic water demand (consumption) used herein are Average Day Demand (ADD), Maximum Day Demand (MDD) and Peak Hour Demand (PHD). The ADD was taken from water meter records in 2014 and calculated in the physical capacity report. As part of our updated model, we assumed a growth of approximately 20 ERUs (Equivalent Residential Units) per year to simulate growth throughout the peninsula. The MDD is equivalent to the highest expected 24-hour demand, expressed in gallons per day (GPD). The PHD is equal to the highest expected 1-hour demand, expressed in gallons per minute (GPM).

In determining the distribution system capacity to meet domestic water demand, the system must be able to supply the PHD and maintain a 30 PSI minimum pressure at all service connections in the distribution system. Service connections where minimum water pressure is below 30 PSI, indicate that improvements to the system are needed.

In determining the distribution system capacity to meet fire flow demand, the system must be able to provide both fire flow and MDD simultaneously, while maintaining a minimum pressure of 20 PSI throughout the system. The system must have the capacity to meet the 20 PSI minimum for the duration of a fire flow event (until the full depletion of the fire suppression storage volume). It is assumed that all new developments will need to meet or exceed these requirements and were modeled as such.

### 2.2.2. Existing Domestic Flow Demand

Water demands for the Tribal Water system were evaluated using water meter data for from 2014. It was previously analyzed and separated into separate usage categories within the Source Water Physical Capacity Analysis from 2015.

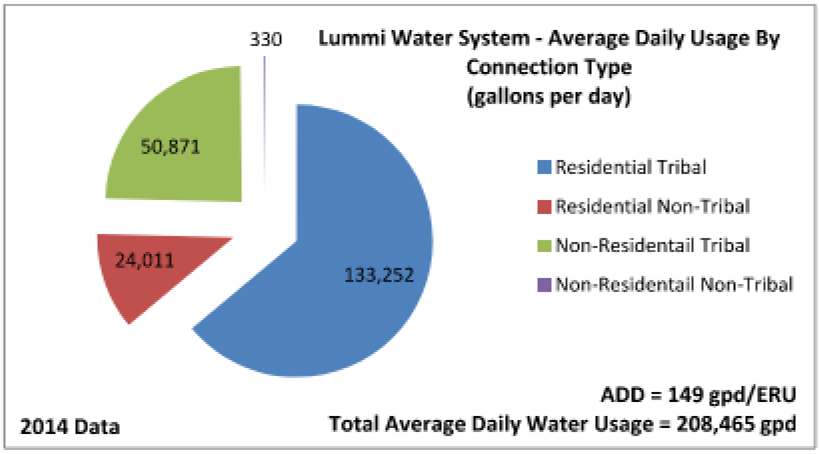



**Figure 2-3. Average Usage by Connection Type**

For Residential Tribal, the ADD is 149 GPD (this is 1 ERU). For Residential Non-Tribal, the ADD is 125 GPD. For non-residential connections, the meter data was used to determine the ADD for each connection. For each connection, the number of ERUs was determined by dividing by the Residential ADD. Table 5 below provides the ADD of the water system based on all of the service connections and the recorded water usage.

**Table 2-3.Summary of Service Connections**

| Summary of Service Connections | | | |
|---|---|---|---|
| Connection Type | Number | Average Usage Per Day | Average Daily Usage per Connection |
| | | | |
| Residential Tribal | 893 | 133,252 gallons | 149 gpd |
| Residential Non-Tribal | 192 | 24,011 gallons | 125 gpd |
| Non-Residential Tribal | 76 | 50,871 gallons | 673 gpd |
| Non-Residential Non-Tribal | 7 | 330 gallons | 45 gpd |
| | | | |
| **Total** | **1,168** | **208,465 gallons** | |

Existing Maximum Day Demand

The Maximum Daily Demand (MDD) is calculated by multiplying the ADD by a peaking factor. The 2023 MDD peaking factor is assumed to be similar to the 2015 peaking factor. The peaking factor was previously calculated as 2.0 times the ADD. Th peaking factor was herein recalculated as the ratio of the maximum month's daily demand (MMAD) to the ADD multiplied by a standard MDD:MMAD peaking factor of 1.7. The MDD is calculated as 440,148 GPD/1397 ERU or 315 gpd/ERU (see below table). The ratio of MDD to ADD (315/149) is 2.11 (the peaking factor).

**Table 2-4.Summary of Existing Service Connections**

| Maximum Daily Demand (MDD) | |
|---|---|
| | |
| Average Daily Demand (ADD) | 208,465 gpd |
| | |
| Maximum Month Annual Demand (MMAD) (July) | 258,911 gpd |
| | |
| MADD Peaking Factor = 1.7 | |
| | |
| Maximum Monthly Demand (MDD) (MDD = MADD x Peaking Factor) | 440,148 gpd |
| | 315 gpd/ERU |
| | |

Existing Peak Hour Demand

PHD was calculated using the following criteria from Section 3.4.2 of the 2020 design manual.

**Equation 3-1: Determine PHD**

$$PHD = (ERU_{MDD} / 1440)\ [(C)(N) + F] + 18$$

Where
- **PHD** = Peak Hourly Demand, total system (gallons per minute)
- **C** = Coefficient Associated with Ranges of ERUs
- **N** = Number of ERUs based on MDD
- **F** = Factor Associated with Ranges of ERUs
- **$ERU_{MDD}$** = Maximum Day Demand per ERU (gallons per day)

| Number of ERUs (N) | C | F |
|---|---|---|
| 15 – 50 | 3.0 | 0 |
| 51 – 100 | 2.5 | 25 |
| 101 – 250 | 2.0 | 75 |
| 251 – 500 | 1.8 | 125 |
| > 500 | 1.6 | 225 |

**Figure 2-4. Equation 3-1 PHD Calculation**

$$PHD = \frac{315\ GPD}{1{,}440} \times ((1558 * 1.6) + 225) + 18 = 612.4\ GPM$$

Based on the calculation the PHD for 2023 was estimated to be approximately 612 GPM or 4.11 times the ADD.

### 2.2.3. Existing Irrigation Flow Demand

Assumed to be zero or included in domestic flow demand.

### 2.2.4. Existing Fire Flow Demand

Fire flow demands for the system were taken from the Washington Survey and Fire Rating Bureau by BHC Consultants in the 2005 report. It is likely that these standards have been updated to be similar to International Fire Code Standards (IFC) which have been adopted by many local municipalities including Whatcom County. The fire protection standards require a minimum residual pressure of at least 20 PSI in the mains for fire flow, whether it is temporary or permanent, or commercial or residential. The fire flow requirement for each building was calculated by identifying the flow requirement based on square feet of fire area and building construction type and then making modifications depending on sprinkler system installations, hazard classification, proximity to other buildings, and fire/smoke detection system installations.

Base Fire Flow

The “base” fire flow requirement (i.e., not including flow reductions for sprinklers, etc.) for each residential building was assumed to be under 3,600 sq ft with no sprinkler system to simplify assumptions.

**Table 2-5.Residential Fire Flow Summary**

| FIRE-FLOW CALCULATION AREA (square feet) | AUTOMATIC SPRINKLER SYSTEM (Design Standard) | MINIMUM FIRE-FLOW (gallons per minute) | FLOW DURATION (hours) |
|---|---|---|---|
| 0-3,600 | No automatic sprinkler system | 1,000 | 1 |
| 3,601 and greater | No automatic sprinkler system | Value in Table B105.1(2) | Duration in Table B105.1(2) at the required fire-flow rate |

Table 2-6 below provides the Fire Flow and Duration for a range of commercial buildings.

**Table 2-6. Commercial Fire Flow Summary**

| FIRE-FLOW CALCULATION AREA (square feet) | | | | | FIRE-FLOW (gallons per minute)[b] | FLOW DURATION (hours) |
|---|---|---|---|---|---|---|
| Type IA and IB[a] | Type IIA and IIIA[a] | Type IV and V-A[a] | Type IIB and IIIB[a] | Type V-B[a] | | |
| 0-22,700 | 0-12,700 | 0-8,200 | 0-5,900 | 0-3,600 | 1,500 | 2 |
| 22,701-30,200 | 12,701-17,000 | 8,201-10,900 | 5,901-7,900 | 3,601-4,800 | 1,750 | |
| 30,201-38,700 | 17,001-21,800 | 10,901-12,900 | 7,901-9,800 | 4,801-6,200 | 2,000 | |
| 38,701-48,300 | 21,801-24,200 | 12,901-17,400 | 9,801-12,600 | 6,201-7,700 | 2,250 | |
| 48,301-59,000 | 24,201-33,200 | 17,401-21,300 | 12,601-15,400 | 7,701-9,400 | 2,500 | |
| 59,001-70,900 | 33,201-39,700 | 21,301-25,500 | 15,401-18,400 | 9,401-11,300 | 2,750 | |
| 70,901-83,700 | 39,701-47,100 | 25,501-30,100 | 18,401-21,800 | 11,301-13,400 | 3,000 | 3 |
| 83,701-97,700 | 47,101-54,900 | 30,101-35,200 | 21,801-25,900 | 13,401-15,600 | 3,250 | |
| 97,701-112,700 | 54,901-63,400 | 35,201-40,600 | 25,901-29,300 | 15,601-18,000 | 3,500 | |
| 112,701-128,700 | 63,401-72,400 | 40,601-46,400 | 29,301-33,500 | 18,001-20,600 | 3,750 | |
| 128,701-145,900 | 72,401-82,100 | 46,401-52,500 | 33,501-37,900 | 20,601-23,300 | 4,000 | 4 |
| 145,901-164,200 | 82,101-92,400 | 52,501-59,100 | 37,901-42,700 | 23,301-26,300 | 4,250 | |
| 164,201-183,400 | 92,401-103,100 | 59,101-66,000 | 42,701-47,700 | 26,301-29,300 | 4,500 | |
| 183,401-203,700 | 103,101-114,600 | 66,001-73,300 | 47,701-53,000 | 29,301-32,600 | 4,750 | |
| 203,701-225,200 | 114,601-126,700 | 73,301-81,100 | 53,001-58,600 | 32,601-36,000 | 5,000 | |
| 225,201-247,700 | 126,701-139,400 | 81,101-89,200 | 58,601-65,400 | 36,001-39,600 | 5,250 | |
| 247,701-271,200 | 139,401-152,600 | 89,201-97,700 | 65,401-70,600 | 39,601-43,400 | 5,500 | |
| 271,201-295,900 | 152,601-166,500 | 97,701-106,500 | 70,601-77,000 | 43,401-47,400 | 5,750 | |
| 295,901-Greater | 166,501-Greater | 106,501-115,800 | 77,001-83,700 | 47,401-51,500 | 6,000 | |

## 2.2.5. Existing System Capacity

### 2.2.5.1 Hydraulic Model

The existing peninsula water distribution system was modeled using Innovyze InfoWater for ArcGIS computer program.

<u>Data Sources</u>

Data for constructing the model was obtained from a number of sources. The LIBC was able to provide the GIS linework along with a hard copy of model results produced by BHC in 2007. The GIS database was missing information on the pipe age, material, and diameter, but as much information as possible was incorporated from the 2007 model. The pipe sizes assumed for the computer model, and presented in this report, may not be perfectly correct, but are deemed sufficient for the planning level calculation herein, but should not be used for any other purpose.

<u>Pipe Characteristics</u>

The roughness of a pipe’s interior affects the flow capacity of the pipe. Flow capacity is proportional to the roughness factor (i.e. a pipe with a Hazen-Williams C value [roughness factor] of 130 has twice the flow capacity of a pipe with a C value of 65). Pipe roughness values were assumed to be approximately 110, which allowed for our model to provide similar results to the calibration study performed by BHC in 2007. For all new pipes the C value was assumed to be 130. Modeling information and nodes and pipes are shown in Attachment 1, additional information is available on the compact disc or USB Drive.

Model Boundary Conditions

The tribal water distribution system is contained on the peninsula and is supplied by groundwater wells (an intertie with Bellingham is also available).. The following assumptions were made to simplify the model because our area of concern was mainly around Chief Martin Rd.

- The tribal water system runs on a 230 Pressure Zone (approximately) based on the Mackenzie tank operating levels. The storage tanks were assumed to be full for the model.

Model Calibration Data

Available system and pressure tests and hydrants flow tests were used to calibrate the model to ensure that results were accurate. The latest flow tests were from 2007 from the original model calibration. Residual pressure data at non-flowing hydrants was used to calibrate or verify the computer model. Residual pressure data from flowing hydrants was not used because the pressure drop in a flowing hydrant can be substantially higher than pressure drop in the water main. The computer model evaluates the residual pressure in the water main not in hydrant service lines. Calculated C values based on the flow testing results were incorporated into the updated model and the model provides results within 5-10 PSI of expected results.

#### 2.2.5.2 Existing Maximum Day Demand Capacity

It is important to note that currently the existing system of wells and storage tanks does not have the capacity to support greater than 18 hours of MDD demand without an additional source or pumping (2017 Physical Capacity Report). Ideally the capacity should be at least 24 hours.

#### 2.2.5.3 Existing Peak Hour Demand Capacity

The existing tribal water system was analyzed at each junction to determine to determine minimum water pressure (as compared to the required minimum of 30 PSI). The model showed that the existing water distribution system is capable of providing at least 30 PSI to almost all water service connections at ground level during estimated peak hour demands (there are about 10 out of 185 nodes in the model at which pressure is less than 30 PSI).

#### 2.2.5.4 Existing Fire Flow Capacity

The existing system was analyzed to determine the fire flow capacity. Available fire flows were determined for each fire hydrant/junction in the system at the design conditions of: MDD and a minimum 20 PSI residual pressure in the water main throughout the system (per WAC 246-290-230[6]). The available flows were determined assuming, conservatively, that the hydrant of interest is the only hydrant flowing.

Available Fire Flows

Figures 2-2-2 to 2-2-7 shows each fire hydrant and node in the existing system and its maximum flow available while maintaining a 20 PSI minimum residual pressure throughout the entire system. This analysis shows fire flow capacity at each node regardless of whether a hydrant is actually installed. The large majority areas have at least 1000 GPM. Many areas, including Kwina Rd have over 2,000 GPM. There are scattered areas where fire flow is lower than 1000 GPM or even lower than 500 GPM, due to higher elevations and distribution system bottlenecks.

## 2.3. *Future Conditions Demand Evaluation*

### 2.3.1. Description

The Master Plan Area full buildout of 9 units/acre was assumed for calculating water demand, for the computer model evaluation, and for sizing the needed water distribution infrastructure for Chief Martin Rd.

The Chief Martin Rd proposed water mains include 2.5 miles of 12-inch pipe and about 2,000 feet of 8-inch pipe served by a booster station. The capacity of 12-inch proposed water main was evaluated using the computer model. Adding the proposed booster station to the model was not needed to complete the analysis. The existing nearby distribution system water mains were also included in the computer model in order to identify any capacity issues caused by the increased demands from Chief Martin Rd. It is assumed in model that no infrastructure improvements have occurred since the GIS file was provided to Wilson Engineering.

The sizing of the Chief Martin Rd water mains was determined based on the model and the amount of anticipated peak flow through the piping. The pipe diameter selected was the minimum diameter needed to limit fire flow velocity to 8 feet per second (fps) or less. Although this system is not required to adhere to this velocity limit, it is good practice because excessive velocities may reduce pipe service life, cause excessive energy consumption, and increase the risk of pipe damaging hydraulic transients. In addition, unplanned high velocity flow may scour interior pipe surfaces and cause water quality problems for consumers.

Phasing was determined geographically assuming that that the district would construct from the Northern end of the Chief Martin Road to the southern intersection (the actual development will likely be different it should be noted). The number of lots developed per phase was determined based on geographical/topographical constraints and the need for upgrades throughout the system.

### 2.3.2. Future Domestic Flow Demand

The LWSD water meter data records for the existing system analysis were used to estimate the water demands for the Chief Martin Expansion. Based on the previous calculation in section 2.2.2, the existing MDD is approximately 315 GPD/ERU. With the full buildout of Chief Martin Rd, the domestic water demand will increase as follows:

- MDD: 1,297 GPM (currently 453 GPM) (844 GPM or 186% increase)
- PHD: 1,782 GPM (currently 612 GPM) (1170 GPM or 191% increase)

The estimated future demands were applied to the model for the Chief Martin Rd buildout. Total growth for the area was assumed to be concentrated within the Chief Martin Rd Development.

### 2.3.3. Future Irrigation Flow Demand

Irrigation flows are assumed to be already included in each connection’s demand.

### 2.3.4. Future Fire Flow Demand

Fire flow demands throughout for the Chief Martin Expansion will be the same as presented in the existing conditions fire flow demands Table 2-2. It should be noted that for simplicity no fire flow demand reductions due to sprinkler systems are included.

### 2.3.5. Future System Capacity

#### 2.3.5.1 Future Maximum Day Demand Capacity

It is important to note that currently the existing system of wells and storage tanks does not have the capacity to support greater than 18 hours of MDD demand without an additional source or pumping (2017 Physical Capacity Report). Ideally the capacity should be at least 24 hours. As growth proceeds and MDD increases, this duration will only get shorter without improvements to source of supply and storage.

#### 2.3.5.2 Future Peak Hour Demand

The model includes all of the proposed Chief Martin Rd infrastructure (12-inch water main, and the storage tank with booster station and 8-inch water main). In order to account for pipe aging prior to full buildout, the estimated pipe friction C-values for existing water mains were reduced by 5.

Figures 2-5-1 through 2-5-6 show the model results for peak hour demand at high density buildout. The Chief Martin Rd Development will have a small effect on the pressure at each exiting demand node. The majority of nodes will still have the minimum required 30 PSI at the water service connection for PHD. At the existing nodes that were already less than 30 PSI, the pressure is reduced more. Chief Martin Rd services will all be over 30 PSI. It is important to note that even though minimum pressures can be met for the Chief Martin Rd, the peninsula water system currently lacks source of supply and storage capacity to support the full buildout development - additional water source will be needed.

#### 2.3.5.3 Future Available Fire Flows

Figures 2-6-1 through 2-6-6 show the available fire flows for the system after all upgrades and improvements are made to the system. The maps show, for each fire hydrant and node in the system, its maximum flow available while maintaining a 20 PSI minimum residual pressure throughout the entire system. This analysis shows fire flow capacity at each node regardless of whether a hydrant is actually installed. Although the installation of the Chief Martin Rd water main improves hydraulic capacity of flow, the higher domestic flows throughout the system result in lower fire flow capacity throughout the distribution system. Fire flows are 10%-30% less. Upgrading water main sizes at bottlenecks should be included as part of the overall water system planning. The Chief Martin Rd fire flows range from 760 GPM to 1770 GPM, except on the hill at the south end. At the south end fire flow could be provided by the new storage tank and/or by upsizing the 6-inch water main on Cagey Rd.

It should be noted that upgrades to the source of supply to the water storage tanks will be needed well before full buildout can be completed. Also, additional improvements (see Section 2.4) will be needed to provide the full recommended fire flow.

## 2.4. *Recommended Improvements*

The following recommended improvements are planning level improvements. Therefore, the sizing and extent of the improvements must be further refined if selected for implementation. Table 2-6 provides a summary of each node and its fire flow status. Figure 2-7 shows the location of the following minimum recommended improvements to provide fire flows and domestic demands. As additional lots are constructed for the Chief Martin Project, capacity studies will need to be performed in order to ensure that the current wells are adequate for future construction.

1. Install 12-inch water main full length of Chief Martin Road. Connect to water mains in Kwina Rd, Scott Rd and Cagey Rd. Install 8-inch crossings at each new intersection. Install service lines at each developed or developing parcel fronting Chief Martin Road. Install 12-inch gate valves every 500 ft to 1000 ft. Install Fire hydrants every 500 ft.
2. Install one 250,000-gallon Water Storage Reservoir at Chief Martin Road north of Cagey Rd (at about 160-ft ground elevation).
3. Install a Booster Station at Water Storage Reservoir at Chief Martin Road north of Cagey Rd to create 300-ft Pressure Zone from Chief Martin Rd Station 12+00 to Station 32+00 (this is the highest segment of the road). Install 8-inch water main (supplied by this booster station) to distribute water to parcels higher than 160 ft in elevation.
4. Install Booster Station at Kinley Reservoir (240 ft TDH)
5. Increase Mackenzie Booster Station pressure capacity to 240 ft TDH
6. Miscellaneous water system improvements (e.g., upsize 6-inch Cagey Rd main).

Final Note: The new storage tank location on Chief Martin Rd might be a good site to drill a new supply well.

## 2.5. *Cost Estimates*

Reasonable order of magnitude cost estimates (2024 dollars) for water system upgrades including design and installation (actual costs could be more or less depending on the simplicity of design and installation):

| IMPROVEMENT | ESTIMATED COST | | |
|---|---|---|---|
| Water Mains for Chief Martin Rd | $2,549,000 | to | $3,058,800 |
| Chief Martin Rd Booster Station and Storage Reservoir | $2,100,000 | to | $2,520,000 |
| Kinley Booster Station Upgrades | $1,200,000 | to | $1,440,000 |
| Mackenzie Booster Station | $600,000 | to | $720,000 |

## 2.6. *Conclusions*

Chief Martin Rd water system infrastructure should consist of the following:

- New 12-inch Water Main running the entire length of Chief Martin Rd from Kwina Rd to Cagey Rd.
- New Hydrants every 500 feet along Chief Martin Rd.
- Updates to the existing booster stations at the Mackenzie and Kinley Tanks
- A new 250,000-gallon reservoir and booster station on Chief Martin Rd. The booster station would supply a 300-ft pressure zone for parcels above 160-170 ft in elevation in the vicinity of the new reservoir and booster station. An 8-inch water main for the 300-ft pressure zone would parallel the 12-inch, 230/240-ft pressure zone water main.

The capacity of the existing water distribution system:

- System is adequate to provide at least 30 PSI at most locations on the peninsula.
- For fire flow, the system is adequeqate fire flow to most commercial and residential locations. However, fire flow capacity is quite variable across the peninsula.

The capacity of the existing water distribution system to supply water to Chief Martin Rd:

- Existing system operating pressure is not high enough to provide the minimum 30 PSI at higher elevations of Chief Martin Rd. A Booster Station is needed.

Impact to the existing water distribution system from Chief Martin Rd:

- At the high population growth level scenario, the fire flow capacity is reduced systemwide. Planning for upgrades to system bottlenecks is needed.

## 2.7. *Appendices (Figures and Tables)*

- Figure 2-1 Existing Water Service Area and Tanks
- Figure 2-1-2 Existing Wells
- Figure 2-1-3 Well, Tanks, and Piping
- Figure 2-1-4 – Figure 2-1-9 – Pipe Mapping
- Figure 2-1-10 Community, Streets, Topography
- Figure 2-3-1 – 2-3-6 – Existing PHD Pressure Mapping
- Figure 2-4-1 – 2-4-6 – Available Fire Flows Mapping
- Figure 2-5-1 – 2-5-6 – Future Buildout PHD Pressure Mapping
- Figure 2-6-1 – 2-6-6 – Future Buildout Available Fire Flows Mapping
- Attachment 1 Pipe Table

# 3. SANITARY SEWER

## 3.1. *Existing System*

### 3.1.1. Description

The existing sanitary sewer system is shown in Figure 3-1 and the proposed Chief Martin Rd sewer is shown in Schematic Figure 3-2. Existing systems are shown in more detail in Appendices Figure 3-2 for the Kwina Point WWTP Basin and Figure 3-3 for the Gooseberry Point Basin. The Kwina and Gooseberry Point sanitary sewer systems consist of a series of sewer mains, manholes, lift stations and force mains of various sizes, slopes, and depths.

<u>Kwina Basin</u>

The Kwina Basin consists of the areas North of Kwina Road to Slater Road including the Silver Reef Casino. Because only a small portion of the Chief Martin Expansion will direct flows to this basin, this system is not evaluated in detail. The most upstream point of the basin is at the Silver Reef Casino, from where sewage is pumped via force main south along Haxton Way and Lummi Shore Dr. Additional pump stations and gravity sewer mains, most notably the Kwina Rd sewer mains, contribute to the system until the sewage reaches the Kwina WWTP. The sewer system map is provided in Figure 3-2 in the appendices.

<u>Gooseberry Point Basin</u>

The northern boundary for the Gooseberry Point basin is approximately 800 LF north of the intersection of Scott Rd and Lummi Shore Dr on the eastern portion of the peninsula. Two interceptors on the eastern and western side of the peninsula convey flow to the southern portion of the peninsula and the WWTP at Gooseberry Point. These sewers consist of a combination of pump/force mains and gravity interceptors. At Gooseberry Point, PS1, PS1A, PS10, or PS2 pump sewage into the headworks of the WWTP. Sewage is then treated and discharged to the Salish Sea via Hale’s Passage. It is assumed that the WWTP will need to be upgraded along with the outfall in the future; however, upgrades are not detailed in this report. The sewer system map is provided in Figure 3-3 in the appendices.

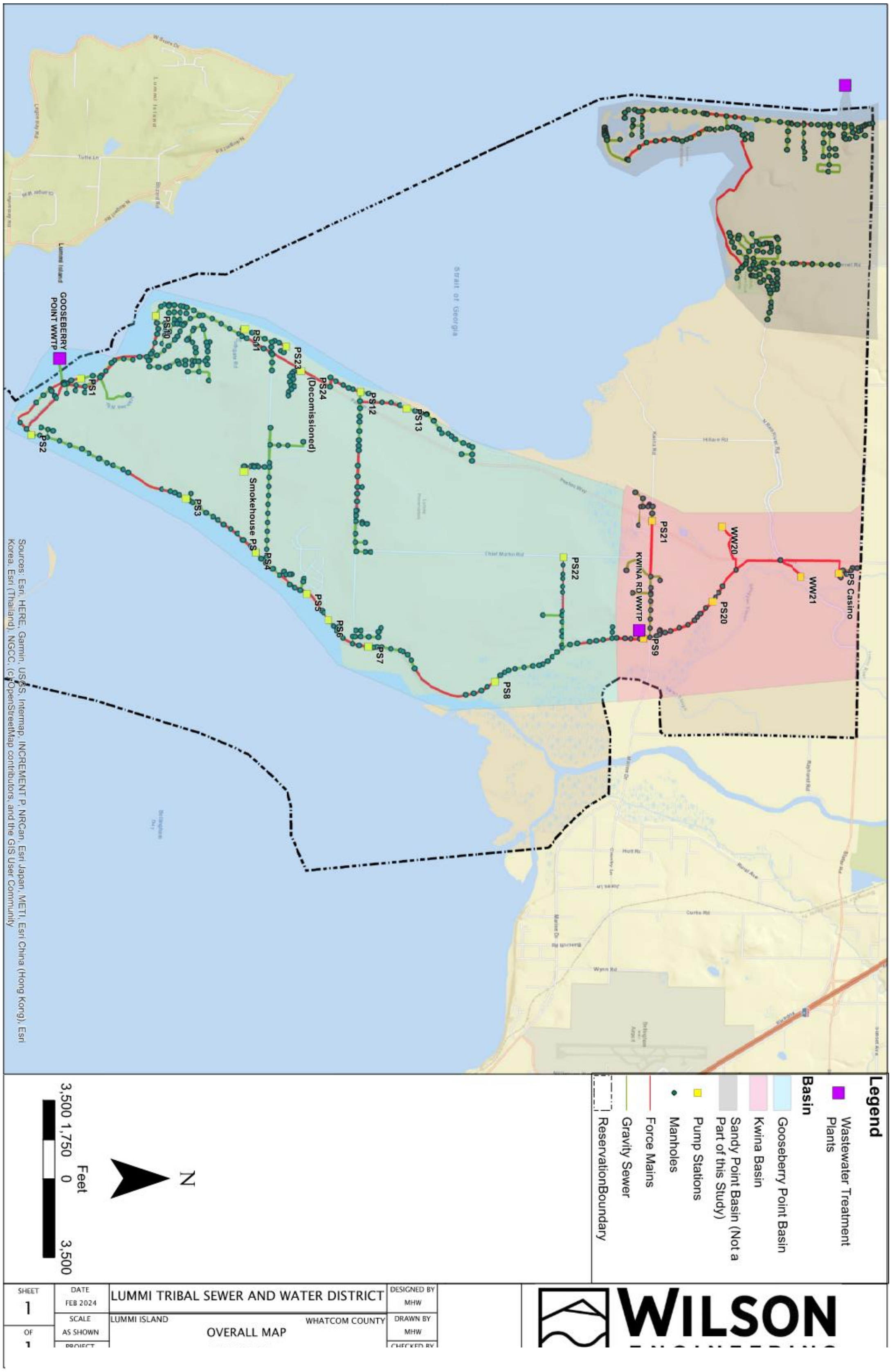
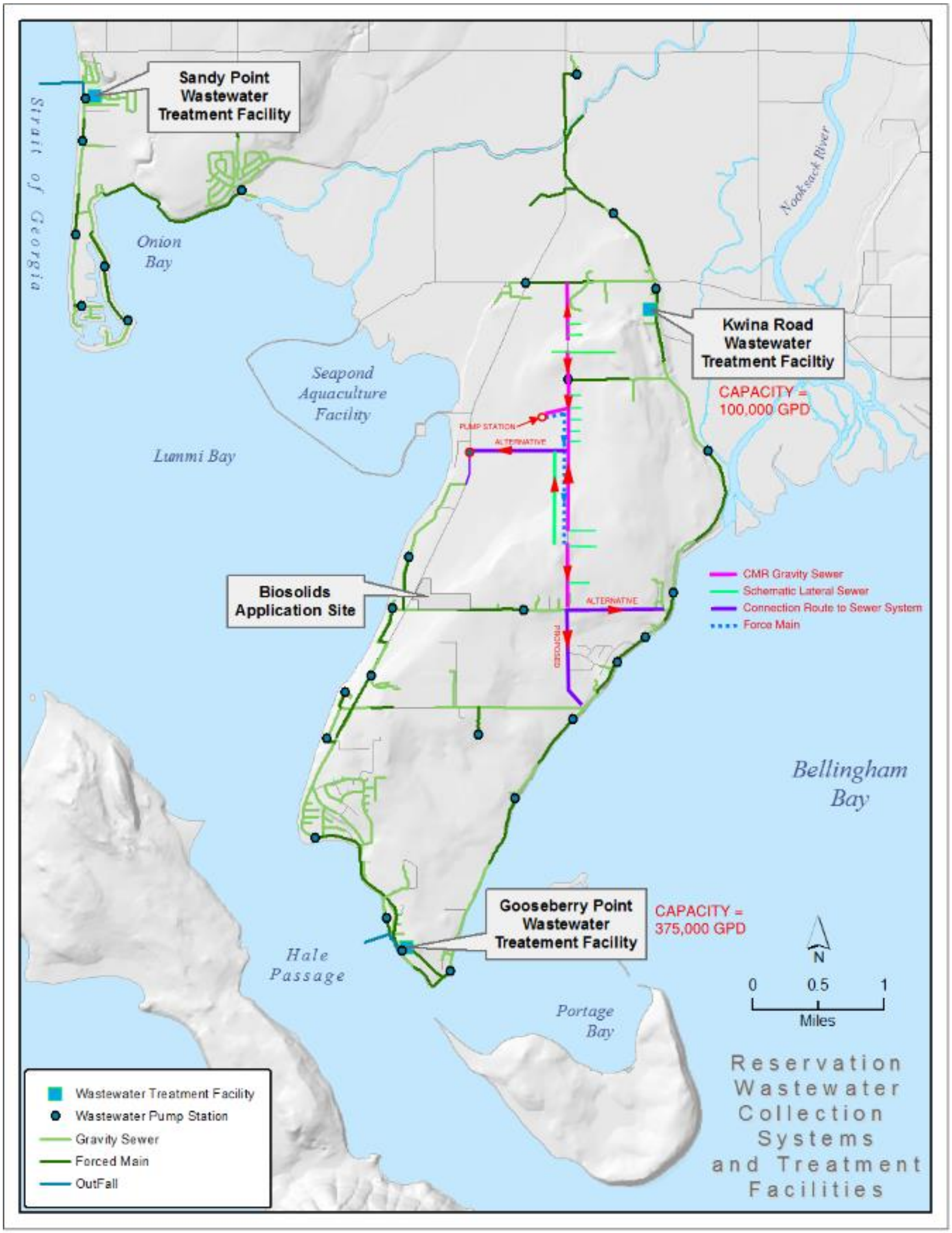



**Figure 3-1. Overall Sewer System Map**




Map 26. Wastewater Collection and Treatment Facilities on the Lummi Indian Reservation

**Figure 3-2. Schematic Layout of Proposed New Sewer System**

## 3.2. *Existing Conditions Evaluation*

### 3.2.1. Flow Evaluation

Existing sanitary flow rates were estimated based on the following factors, total number of single-family residences (SFRs) located with Lummi Tribal Sewer and Water District (LTSWD) and SFRs owned privately along with nonresidential water/sewer users. The 2017 analysis performed by Gray and Osborne (wastewater report, 2017) showed that each SFR residence will generate approximately **154 GPD.** In addition, there are **43 nonresidential ERUs** (estimated based on water usage). In 2017, a total number of **913 ERUs** was estimated for the Gooseberry Point Basin. Assuming a constant growth rate of 20 ERUs/Year, a total of 1,033 ERUs is assumed to be contributing to the Gooseberry WWTP for the existing system (2023).

<u>Mean Daily Flows and Inflow/Infiltration</u>

The mean daily flow for each ERU was also checked against the average dry and wet weather flows. Table 3-1 shows the existing dry weather flow and average annual flow for the WWTP and from these values the GPD/ERU was confirmed along with the total inflow and infiltration (I/I) entering the system through existing manholes and pipes.

<u>Peak Flows</u>

The Peak Hour Flow (PHF) of 950 GPM was estimated by summing the pump flows from PS 10 and PS 2 (single pump flow). A peaking factor was calculated as PHF divided by the Annual Average Flow (AAF). A summary of these values is included below in Table 3-2.

**Table 3-1.Sewer Flow Summary**

| YEAR | ERUs | WW Base Flow<br>MGD | Flow per ERU<br>GPD/ERU | Influent at WWTP (AAF)<br>MGD | I/I<br>MGD | I/I per MH*<br>GPD |
|---|---|---|---|---|---|---|
| 2017 | 913 | 0.141 | 154.44 | 0.255 | 0.114 | 0.261 |
| 2023 | 1033 | 0.159 | 153.92 | 0.288 | 0.129 | 0.307 |

*Assumed approximate 303 Manholes in system

### 3.2.2. Sanitary Sewer Capacity

The flows and flow capacities of the main sewer system pipes are shown in Attachment 2 along with all the basic pipe data: diameter, length, age, invert elevations, and slope (Please note that not all pipes had provided data and minimum slope was assumed for sections without adequate data). A color-coded map of q/Q (ratio of actual flow over pipe flow capacity) is provided in Figure 3-4 and 3-4-1 for existing flows. In the computer model, the estimated flows are input at each manhole based on the density of lots around the manhole. No backflow analyses were performed at each manhole due to incomplete data.

### 3.2.3. Gooseberry Point WWTP

Gooseberry Point serve the entire Lummi Peninsula area of the Reservation, except for the Silver Reef Casino to Kwina Rd area to the north. The National Pollutant Discharge Elimination System (NPDES) permit issued by the Environmental Protection Agency (EPA) for this facility authorizes an annual average day flow of 0.375 MGD. The wastewater collection system for the Gooseberry Point wastewater treatment plant includes 15 pump stations. Flow to Gooseberry Point WWTP is about 0.288 MGD (annual average 2023) compared to the permitted capacity of 0.375 MGD. Therefore, the WWTP has capacity for an additional 565 ERU.

### 3.2.4. Kwina WWTP

The Kwina Road wastewater treatment plant is a membrane bioreactor (MBR) that became operational in June 2006. This facility now serves the northeast portions of the Reservation including the Silver Reef Hotel, Casino & Spa. It produces “Class A” reclaimed water that is currently discharged into the ground through a series of underground injection wells. The ground water in the vicinity of this facility is too salty for potable uses. The MBR facility currently has the capacity to treat 0.10 million gallons per day. The wastewater collection system for this facility extends south from the casino complex and includes 4 pump stations.

## 3.3. *Future Conditions Evaluation*

The development of the Chief Martin corridor will require sewer infrastructure within the corridor and will also impact all downstream sewer system facilities, many of which lack the capacity for substantial increases in sewer flows.

Most of the Chief Martin Rd sewer flows ae anticipated to flow to the Lummi Shore Dr sewer and then to Gooseberry Point WWTP. As flows are added into the Lummi Shore Dr sewer system downstream of the Chief Martin Rd developments, downstream pump stations and force mains and gravity mains on Lummi Shore Dr will need to be upgraded. It is calculated that only about 80 lots can be added before major downstream upgrades will begin to be needed.

These upgrades will be phased over time as development of Chief Martin Rd and population growth will likely occur in many phases occurring over many years.

The Gooseberry Point WWTP capacity will need to be upgraded by the time 565 ERUs are added to the system. Evaluation of the WWTP upgrades is beyond the scope of this report, but does need to be addressed before this level of growth occurs. Cost estimates for the WWTP upgrade are not included in this plan.

The Kwina Point WWTP is close to capacity. New flows from the additional planned lots from Turkey Shoot and the planned utility building will likely put it at capacity.

Future flows and future improvements were modeled using InfoSewer. Results of the model are included in the appendices.

The future Chief Martin Rd gravity mains were sized with two considerations in mind. The 1st being total flow capacity ratio q/Q as previously discussed in section 3.2.2 and the minimum scouring velocity as referenced by the Department of Ecology Orange Book Section C1-4.4. All sewers must be designed to give a mean velocity of no less than 2.0 fps flowing full. With these two criteria in mind future sewers were sized.

## 3.4. *Recommended Improvements*

### 3.4.1. Recommended Improvements and Operations

Recommendations are primarily for upgrading the system to handle additional flows. LTWSD should plan to continue to monitor and replace aging pipes.

**Monitoring and Maintenance of the Existing System**

1. **Monitor the sanitary sewer system for infiltration and inflow**
   Monitor the sanitary sewer system for infiltration and inflow (I&I, aka leaks) problems during wet weather (e.g., during January). This could be as simple as placing peak level recording devices in manholes (such as a vertical plastic pipe with cork dust) to record manhole surcharging during wet weather.

2. **Monitoring Sewer Condition**
   Monitor the conditions of the sewer pipe and manholes to identify potential failures, blockages or leak problems. LTWSD should plan to contract or perform sewer video inspections on problematic areas. These inspections should continue to be performed every regularly depending on the age and condition of piping.

3. **Flush the sewer systems on an as-needed basis**
   Flush or jet clean the sewer systems on an as-needed basis. The necessity and frequency should be as determined by operations staff observations. If standing water is observed in manhole outlet pipes, then flushing should be performed.

**Improvements and Phasing for Chief Martin Rd**

1. Phase 1 – Minor upgrades will be needed for Phase 1 (Figure 3-4-1). The sewer improvement will consist of 2,100-2,500 LF of 8-inch sewer main flow north down Chief Martin Rd to Kwina Rd). For the adjacent Turkey Shoot Development, a parallel 8-inch sewer main will extend 760 LF north to Kwina Rd. If connecting to this sewer is allowed then the total length of the sewer main could be reduced. A minimum of 1,400 LF 8-inch sewer extension will be needed to provide sewer service to the proposed new Public Works Utilities Building, if connecting to the Turkey Shoot sewer (or 2,100 LF if extending all the way to Kwina Rd).

2. Phase 2 – Phase 2 will begin constructing lots within the highlighted areas in Figure 3-4-2. Installation of 300 LF of 8-inch sewer, 890 LF of 12-inch sewer, and 730 LF of 18-inch sewer will be needed to serve Phase 2. This area is currently served by the existing pump station at the intersection of Chief Martin Rd and Scott Rd (PS 22). **Approximately 80 lots can be constructed before PS 22** will be at capacity. The existing downstream pump stations will receive more flow as shown

in Table 3-2 below. The capacity of these pump stations should be checked using the drawdown method or other means.

**Table 3-2. Phase 2 Pump Station Summary**

| PS Name | Existing Flow Rate (GPM) | Proposed Phase Flow Rate (GPM) | Buildout Flow Rate (GPM) |
|---|---|---|---|
| PS8 | 100 | 150 | 150 |
| PS7 | 100 | 180 | 200 |
| PS6 | 210 | 210 | 210 |
| PS5 | 240 | 240 | 240 |

**3.** Phase 3A – Phase 3 will construct the additional lots in the highlighted area as shown in Figure 3-4-3. Installation of 60 LF of 16-inch sewer and 2,750 LF of 18-inch sewer (connecting to the pump station) will be needed to serve Phase 3A. A force main will also be constructed as described below.

Because PS 22 will be at capacity, construction of a new pump station and force main will be needed as shown. Because the new pump station will receive higher and higher flows in the future, it will be important that it be designed so that it may be upgraded easily in the future. After construction of the new pump station, PS 22 can be decommissioned and flows rerouted to the new pump station via gravity main. The new pump station will need a combination new force main and newer gravity sewer to convey sewage 7,620 LF of 15-inch force main and 6,050 LF of 21-inch gravity main to connect to the Lummi Shore Eastern Interceptor. The last 4,200 LF of this connection would be through a new sewer corridor, which would require easements. The alternative to this 4,200 LF corridor would be to use Cagey Rd (this would require more upgrades to pump stations and sewer pipe on the Lummi Shore Interceptor. On an interim basis, the Scott Rd force main could be used for until that system is built (this would limit building capacity in the interim). This would require constructing a temporary force main to connect to the Scott Rd force main. This would also help the forcemain to be put online when it has enough connections to make the large size work (i.e., sewage in a long, large forcemain will stagnate in the pipe for days if the inflow is small).

**Pump Station Upgrades.** Based on their existing capacity, the following existing downstream pump stations would need to be upgraded to accommodate the added flows due to growth.

**Table 3-3. Phase 3 Pump Station Summary**

| PS Name | Existing Flow Rate (GPM) | Proposed Phase Flow Rate (GPM) | Buildout Flow Rate (GPM) |
|---|---|---|---|
| PS4 | 100 | 1,000 | 2,350 |
| PS3 | 310 | 1,100 | 2,500 |
| PS2 | 500 | 1,150 | 2,410 |
| New PS (near Scott Rd) | N/A | 840 | 1,800 |

For both Phases 3A and B, existing force mains and gravity sewers will need to be upgraded, as shown in maps 3-4-4 and 3-4-5, respectively.

Phase 3B – Phase 3 will construct lots in the highlighted area as shown in Figure 3-4-4. Based on existing capacity values, the same existing downstream pump stations would need to be upgraded to accommodate the added flows.

**Table 3-4. Phase 3B Pump Station Summary**

| PS Name | Existing Flow Rate (GPM) | Proposed Phase Flow Rate (GPM) | Final Flow Rate (GPM) |
|---|---|---|---|
| PS4 | 100 | 1,400 | 2,350 |
| PS3 | 310 | 1,500 | 2,500 |
| PS2 | 500 | 1,500 | 2,410 |
| New PS | N/A | 1,250 | 1,800 |

Because these same pump stations are being used to convey flow to the Gooseberry Point WWTP, upgrading the pump stations in advance for Phases 3A and 3B is recommended.

Approximately 2,040 LF of 8-inch sewer and 2,070 LF of 12-inch sewer are needed to serve the new Phase 3B areas.

**Existing Pipe Upgrades.** For both Phases 3A and B, existing force mains and gravity sewers will need to be upgraded, as shown in maps 3-4-4 and 3-4-5, respectively.

4. Phase 4 – Phase 4 continues to construct lots further south along Chief Martin Rd as shown in Figure 3-4-5. New infrastructure includes 2,035 LF of 8-inch sewer and 4,000 LF of 12-inch sewer to serve these new lots. These Phase 4 additions are not included in the overall cost estimate in Section 8, because they are not crucial to development of the corridor.

   **Existing System Upgrades.** The continued construction of these lots will impact the same downstream infrastructure as the previous phases. LTWSD can reassess piping and pump stations and upgrade piping and pump stations to accommodate full buildout or other design capacity as needed.

5. Phase 5-6 – Phases 5 and 6 continue to construct lots south of Phase 4 as shown in Figures 3-4-7 and 3-4-8. It is recommended that all pump station upgrades be done in Phase 4. Thus, Phases 5 and 6 would include the addition of gravity sewer mains. Phase 5 would include 2,050 LF of 8" sewer 1700 LF of 12" sewer parallel to and west of Chief Martin Rd. These Phase 5 and 6 additions are not included in the overall cost estimate in Section 8, because they are not crucial to development of the corridor.

### 3.4.2. Cost Estimates

Reasonable order of magnitude cost estimates (2024 dollars) for Chief Martin Road Sewer and downstream system upgrades, including design and installation (actual costs will be more or less depending on the simplicity of design and installation):

| IMPROVEMENT | ESTIMATED COST |
|---|---|
| Phase 1 | $300,000 to $400,000 |
| Phase 2 | $350,000 to $450,000 |
| Phase 3A * | $15,000,000 to $20,000,000 |
| Phase 3B | $1,000,000 to $1,200,000 |
| Phase 4 * | $5,000,000 to $6,500,000 |
| Phase 5 | $700,000 to $900,000 |
| Phase 6 | $250,000 to $350,000 |
| Total | $22,600,000 to $29,800,000 |

* Costs includes substantial existing system upgrades.

## 3.5. *Conclusions*

- The sanitary sewer collection systems for the Gooseberry Point Basin and Kwina Basin both have capacity constraints that need to be addressed as the population grows. The peninsula sewer collection system was not designed and built to accommodate substantial additional population growth. The lack of capacity stems from each individual pump station or sections of gravity mains that will quickly reach capacity after a certain number of service connections are added upstream. And, as the upstream sewer and pump stations become overloaded so too will the larger downstream pump stations and eventually the wastewater treatment plants.

- Therefore, each phase of growth on Chief Martin Rd will require upgrades to various specific downstream sewer infrastructure components.

- An important aspect of maintaining capacity of existing sewer infrastructure throughout both basins is keeping sewer pipes and manholes in good condition, replacing aging pipe before it deteriorates, and preventing/correcting increases in infiltration and inflow (I&I).

- The eastern interceptor along Lummi Shore Dr and pump stations located south of Smokehouse Rd will need be upgraded to accommodate all the increased flows coming from Chief Martin Rd Phases 3-6.

- Phase 1 will convey flows towards the Kwina WWTP which is understood to be at near capacity. A new 8-inch gravity sewer would be constructed on Chief Martin Rd. No upgrades to the existing system are needed, as long as the WWTP can accept the added flows.

- Phase 2 could add 80 new lots, which would all be located in the Gooseberry Point WWTP basin. A new 8-inch gravity sewer main would be constructed on Chief Martin Rd. No additional upgrades are needed. The Scott Rd pump station and force main would be used to convey flow.

- Phases 3 – 6 could add a large number of new lots, which would all be located in the Gooseberry Point WWTP basin. A new pump station and force main and new gravity sewer mains of varying sizes would be constructed on Chief Martin Rd. The new pump station would pump sewer flows south via a new force main to the high point on Chief Martin Rd. At that point, a new gravity main would convey flow to Cagey Rd. From there, a new gravity main could convey flow south to Lummi Shore Rd at Smokehouse Rd or east or west on Cagey Rd. The new sewer main to Smokehouse Rd option would bypass several existing pump stations, which would allow for avoiding upgrades to those pump stations and force mains. These phases will require the largest capital investment in onsite and downstream upgrades.

- All downstream pump stations and piping on Lummi Shore Drive will need to be upgraded to handle buildout flow from the Chief Martin corridor. The extent and timing of theses upgrades will require further evaluation. Design and construction of these upgrades would occur in phases as growth proceeds.

- Gooseberry Point WWTP will need a capacity upgrade by the time 565 ERUs are added.

## 3.6. *Appendices (Figures and Tables)*

# 4. STORMWATER

## 4.1. *Regulatory Requirements*

The Lummi Nation Water Resources Protection Code titled:

> *TITLE 17 LUMMI NATION CODE OF LAWS, WATER RESOURCES PROTECTION CODE*

contains the Large Projects Stormwater Management Requirements in Section 17.05 as follows:

*Chapter 17.05 Storm Water Management*

*Subsection 17.05.060 Requirements for Large Projects:*

*All large projects must prepare and submit a large project plan and a permanent Storm Water Pollution Prevention Plan prepared by a licensed professional engineer. In addition, the following requirements must be met for all large projects and must be addressed in conditions of a Lummi land use permit issued under Title 15 of the Lummi Code of Laws:*

- *(a) Erosion and sediment control. All new development and redevelopment shall comply with erosion and sediment control requirements through the implementation of an approved Large Project Erosion and Sediment Control Plan.*
- *(b) Natural drainage patterns will be maintained and discharges from the site will occur at the natural location to the maximum extent practicable.*
- *(c) Source control BMPs shall be applied to all projects to the maximum extent practicable and will be selected, designed, and maintained according to methods approved pursuant to 17.05.*
- *(d) Runoff treatment BMPs will be provided for the treatment of all storm water. Direct discharge of untreated storm water to surface water is prohibited. All treatment BMPs will be selected, designed, and maintained according to methods approved pursuant to 17.05.*
- *(e) Streambank erosion control is required where storm water runoff is discharged directly or indirectly to a stream. As the first priority, streambank erosion control BMPs shall utilize infiltration only when, to the fullest extent practicable, the site conditions are appropriate and ground water quality is protected. Streambank protection will be selected, designed, and maintained according to methods approved pursuant to 17.05.*

*(f) When storm water discharges directly or indirectly through a conveyance system into a wetland, the following additional requirements must be met:*

*(1) Storm water discharges to wetlands must be controlled and treated to the extent necessary to meet appropriate water quality standards.*

*(2) Discharges to wetlands shall maintain the hydroperiod and flows of existing site conditions to the extent necessary to protect the characteristic functions of the wetland.*

*(3) Created wetlands that are intended to mitigate for loss of wetland acreage, function, and value shall not be designed to also treat storm water.*

*(g) All large development projects will conduct an analysis of off-site water quality impacts resulting from the project and shall mitigate these impacts. The analysis will extend a minimum of one-fourth mile downstream from the project. The existing or potential impacts to be evaluated and mitigated include, but are not limited to, excessive sedimentation, streambank erosion, discharges to ground water or recharge areas, violations of water quality standards, and spills and discharges of priority pollutants identified under Section 307(a) of the Federal Clean Water Act.*

*(h) An operation and maintenance schedule shall be provided for all proposed storm water facilities and BMPs; the party responsible for maintenance and operation shall be identified.*

*(i) If it is determined by the Water Resources Manager that the minimum requirements of this Code do not provide adequate protection of water quality or of sensitive areas (e.g., high value wetlands, aquifer recharge areas, tidelands, and estuaries) either on-site or within a designated area or basin, more stringent controls shall be required to protect water quality or the sensitive area.*

## 4.2. *Hydrologic Modeling*

Hydrologic modeling is used to determine stormwater runoff quantities and peak flow rates. Stormwater treatment and flow control calculations were performed using the 2019 Western Washington Hydrology Model (WWHM). Pipe sizing calculations were performed using 24-hour single-event models or the Rational Method as appropriate.

## 4.3. *Stormwater Flow Control and Treatment*

The existing Chief Martin Road stormwater system consist of ditches. These ditch flow alongside Chief Martin Road and discharge at four different locations: (1) Cagey Rd ditches, (2) unnamed stream 1 flowing west, (3) unnamed stream 2 flowing west, and (4) Kwina Rd storm system. There are no detention or treatment systems. The existing road is narrow with no sidewalks and no curb and gutter. The soils are mostly hydrologic group C soils, which have high runoff and usually a shallow groundwater table. The next most common group is hydrologic group D soils, which have very high runoff. There is a small area of Group B soils (which have good infiltration capacity and lower runoff rates) at the highest elevations.

Building out Chief Martin Road will result in a minimum of 600,000 square feet (13.5 acres) of impervious area within the 60-ft ROW. Runoff from this area will be collected via a catch basin and storm pipe system located at the curb and gutter of Chief Martin Rd. Collected runoff will flow to the four different low points. At each of these low points, stormwater treatment and detention are needed. The most practical and economical method to achieve the two goals is to install combined detention ponds. A combined detention pond includes a permanent pool of water and a “live” storage volume above the permanent pool, and typically one-foot of freeboard. These volumes are calculated using the WWHM. The design permanent pool water treatment volume is equal to the six-month storm volume. The detention volume is equal to approximately 8” of rain spread out over the impervious area plus the runoff from the pervious area.

These four drainage areas are labeled herein as south, south middle, north middle and north. The areas of impervious surfaces within the 60-ft ROW and the pervious surfaces, which are primarily forested land, are shown in Table 4-1. The peak flow shown in Table 4-1 is calculated using the SBUH method (based on a 24-hour, 3.7-inch storm event). For the pervious areas, a curve number of 70 was used based on forest in good condition in Type C soils (note that the runoff from these pervious areas will be much higher after they are developed; and, stormwater controls will be needed for any development).

Water quality treatment and detention are sized to provide the control needed for the road runoff only. The added flows from pervious areas will add to flow through the ponds. Flow control structures will have to be designed in consideration of this. It may be desirable or required that the permanent volume is increased to account for the added flows from these areas. It may also be prudent to increase the net detention volume for these ponds as a more conservative approach to managing current and future flows (as the data for the model is imperfect). The location and approximate size of each of the four combined ponds are shown in the plans. The South and North ponds would discharge to the existing ditch and storm piping on Cagey Rd and Kwina Rd, respectively. The other ponds would discharge to the forest via level spreaders to adequately disperse the flow.

**Table 4-1. Peak Flows for the Roadway and Adjacent Forested Land**

| | Road Length Stationing | Impervious Area | Pervious Area | Impervious Area Peak Flow | Total Area Peak Flow |
|---|---|---|---|---|---|
| | 100ft + ft | acres | acres | cfs | cfs |
| South End | 0+80 - 27+30 | 2.7 | 20 | 2.3 | 3.2 |
| South Middle | 27+30 - 80+00 | 5.4 | 241 | 4.7 | 17.4 |
| North Middle | 80+00 - 104+00 | 2.5 | 112 | 2.2 | 8.1 |
| North End | 104+20 - 132+00 | 2.9 | 17 | 1.0 | 2.1 |

Other stormwater management approaches should be further investigated after more detailed investigations of soils and hydrologic conditions. Flow control methods could include roadside bioretention, infiltration basins, infiltration trenches, or roadside direct dispersion into dedicated forested land. Treatment methods could include bioretention, vegetated filter strips, proprietary filters vaults, or proprietary modular (vaulted) wetlands.

The stormwater piping and catch basins are sized based on the total peak flow from the roadway and the forested areas and the road slope using the Manning's equation (Table 4-2). It should be noted that in the future, these pipes may not have adequate capacity if future developments do not provide proper flow control. The primary pipe conveyance would be located on the west side of the road at the gutter. Catch basins on the east side of the road would be connected to the primary piping via 42-ft long, 8-inch pipes. Catch basins are Type 1 for pipe 12 inches and smaller, Type 1L for 15-18 inches, and Type 2 for 24 inches.

**Table 4-2. Pipe Segments and Hydraulic Flow Capacity**

| Pipe Segment Location | | | Pipe Diameter inches | Pipe capacity cfs | Pipe Length LF | Peak Flow Flow cfs | Extra Capacity cfs |
|---|---|---|---|---|---|---|---|
| South | 0+55 | 13+30 | 12 | 5.9 | 1,275 | 3.2 | 2.7 |
| | 13+30 | 23+70 | 12 | 2.7 | 1,040 | 1.6 | 1.1 |
| South | 30+10 | 34+30 | 12 | 7.0 | 420 | 2.4 | 4.6 |
| Mid | 34+30 | 36+60 | 15 | 6.2 | 230 | 3.1 | 3.1 |
| | 36+60 | 50+20 | 15 | 12.6 | 1,360 | 7.6 | 5.0 |
| | 50+20 | 61+00 | 18 | 11.3 | 1,080 | 11.1 | 0.1 |
| | 61+00 | 75+00 | 24 | 17.2 | 1,400 | 15.7 | 1.4 |
| | 75+00 | 77+00 | 12 | 2.7 | 200 | 1.6 | 1.1 |
| North | 82+40 | 86+40 | 12 | 2.7 | 400 | 2.1 | 0.6 |
| Mid | 86+40 | 96+40 | 18 | 8.0 | 1,000 | 6.0 | 2.0 |
| | 96+40 | 102+60 | 12 | 8.8 | 620 | 2.7 | 6.1 |
| North | 107+30 | 115+30 | 12 | 11.2 | 800 | 0.8 | 10.3 |
| | 115+30 | 132+00 | 15 | 4.9 | 1,670 | 2.1 | 2.8 |
| Total | | | | | 11,495 | | |

## 4.4. *Costs*

The more detailed cost estimate is included in the Appendices. The estimated cost for stormwater for the entire Chief Martin corridor (excluding roads and development branching off from Chief Martin Rd) is $5,500,000.

# 5. ELECTRICAL POWER AND TELECOMM

## 5.1. *The Existing System*

Kwina Road, Lummi Shore Drive, and Haxton Way have 3-phase, 3-wire circuit distribution systems (overhead). Cagey Road is served by single-phase overhead power. Scott Rd also has single-phase overhead power. The south end of Chief Martin Road has power service extended from Cagey Rd to several roadside parcels. The north end has a short run of buried power lines extending south from Kwina Rd along the west side of Chief Martin Rd.

## 5.2. *Future Loads*

The likely maximum future buildout peak load is based on what would be expected for 3,000 homes. The needed infrastructure is basin on this assumption. If, for example, only half the demand is realized, then the needed infrastructure could be halved. The peak demand from each home is conservatively 10 KW. Total peak demand would be 30,000 KW. To provide this service would require two circuits of 12,470 volt, 3-phase, 600 amp. Two parallel stepped down circuits of 12,470 volt, 3-phase, 200 amp would be needed. Puget Sound Energy would provide these distribution lines in buried duct banks. Additional infrastructure would include:

- 600A, 12,470V, 3 phase, Padmount Switches
- 600A primary vaults (approx. every 800')
- 200A primary vaults (approx. every 400')

Service into each neighborhood would be 200A.

Life expectancy of electrical cables and equipment is expected to be 20-30 years or more.

## 5.3. *Costs*

The estimated cost for power, on Chief Martin Rd only, for 3,000 new residences is $32,000,000.

The estimated cost for power, on Chief Martin Rd only, for 1,500 new residences is $17,000,000.

Costs do not include cost that PSE may impose for increasing power supply to the vicinity (this cost is unknown).

# 6. TRANSPORTATION

Transpo Group prepared a detailed transportation evaluation for the Chief Martin Rd development and adjacent connecting roads. A detailed memorandum is included in the appendices. The analysis of existing and future traffic volumes was used in determining road classification and roadway cross sections for Chief Martin Road for master planning purposes. Transpo considered existing and future traffic volumes, connections with external roadways, and daily and peak hour traffic loads on Chief Martin Road. The results include various cross sections for the low-density and high-density development scenarios, as well as consideration of internal intersections along the road. This section presents the most relevant results of the evaluation in brief.

Project Description

The proposed development includes development of seven areas of land that lie along Chief Martin Road between Kwina Road and Cagey Road, within Lummi Nation lands in Whatcom County, Washington. The individual areas range from 3 to 72 acres of buildable area, with the total buildable area of about 324 acres. Low- and high-density scenarios were studied. The low-density scenario equates to a density of 3 single-family units per acre and the high-density scenario equates to 9 units per acre. For each scenario, trip generation per plot of land was forecast and distributed to the roadway system. This information was used to determine the appropriate road classification and cross-sectional elements for Chief Martin Road. Operational analyses were also considered for the two terminal intersections along Chief Martin Road, at Kwina Road on the north and Cagey Road on the south.

## 6.1. *Existing Conditions*

Roadways

Figure 6-1 shows the general study area and the existing road network serving the developable sites along Chief Martin Road.

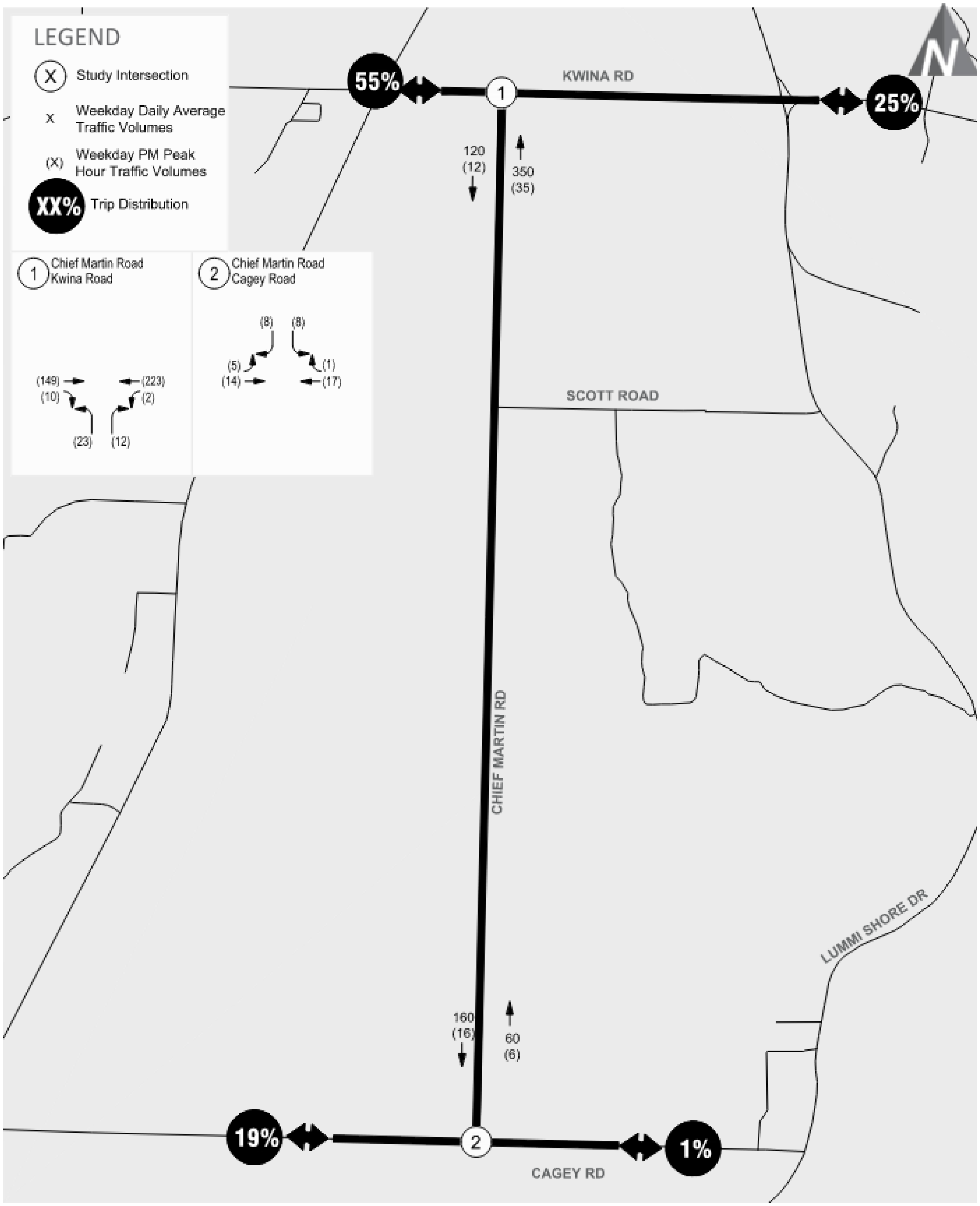



**Figure 6-1. Area Roadways and Existing PM Peak Hour Volumes**

**Chief Martin Road**. This is a 2-lane, north-south, rural local road that travels between Kwina Road and Cagey Road. The roadway is relatively narrow with limited shoulders. The road ends in two "T" intersections, at Kwina Road and Cagey Road. There are a handful of driveways to single- family homes along this corridor. No transit lines operate along this road.

**Kwina Road.** This is a 2-lane, east-west, rural major collector road located at the north end of Chief Martin Road. The road has curbs, gutters, and sidewalks along much of its length. Kwina Road is lined with commercial, institutional, and residential parcels and is the most urban-feeling road in the area. Several Lummi Transit shelters and stops lie along Kwina Road. Whatcom Transit Authority (WTA) Route 50 also stops on this road.

**Cagey Road**. This is a 2-lane, east-west, rural local road located at the south end of the study area. This road is narrow with limited shoulders. Some low-density housing lies along this road. There are several Lummi Transit stops on this road.

**Scott Road.** This roadway provides a "T" intersection with Chief Martin Road approximately ¾ of a mile south of Kwina Road. This 2-lane rural local road travels east from Chief Martin Road to Lummi Shore Road.

**Transit**

The surrounding area is served by both Lummi Transit and WTA buses/vans. No bus service is currently provided along Chief Martin Road.

**Traffic Volumes**

Weekday PM peak hour traffic counts were collected in November 2023 at two intersections:

- Chief Martin Road/Kwina Road
- Chief Martin Road/Cagey Road

These volumes form the baseline of existing traffic in the area and inform future roadway design. These volumes are shown in Figure 6-1. Roadway volumes at these intersections are relatively low, with about 420 pm peak hour trips at the Kwina Road intersection and about 50 pm peak hour trips at the Cagey Road intersection.

## 6.2. *Future With Project Conditions and Recommendations*

For this master planning process, the amount of future traffic is critical for roadway cross-section development and for testing operations at nearby key intersections. For this study, we have considered two scenarios – a low-density scenario with just under 1,000 single-family dwelling units, and a higher density scenario with almost 3,000 single-family dwelling units. The two situations would create the need for different roadway cross sections for Chief Martin Road.

For this master planning process, the amount of vehicular traffic generated by the development along Chief Martin Road was estimated. Traffic was assigned to the roadway network based on anticipated future trip distribution patterns. Potential cross sections for each of the two scenarios were then developed to accommodate the forecasted traffic volumes and desired aesthetic. Chief Martin Road will carry higher volumes in the future than other area roadways. The future housing density and higher traffic volumes fit a more urban aesthetic than the current form of the road. In keeping with the more urban land uses along Kwina Road and its more urban cross section, the proposed Chief Martin Road cross sections tend to the more urban form.

**Trip Generation**

As described above, two scenarios were analyzed: a low-density scenario with 3 dwelling units per acre; and a high-density scenario with 9 dwelling units per acre. With a total of 324 building acres, the two scenarios are anticipated to include 972 and 2,916 single-family dwelling units, respectively. It should be noted that the total development, at even the low density, may take many years or even decades to come to fruition. The high-density buildout scenario is unlikely to occur over a twenty-year planning period and perhaps not at all

At the low end of the range of buildout development (3 units/buildable acre), 843 PM peak hour trips (both directions) would be generated (see Table 6-1). The daily trips along Chief Martin Road, by road segment, are shown Figure 6-2. Figure 6-2 also shows the 2043 with projected weekday PM peak hour traffic volumes at each development area and at the two study intersections.

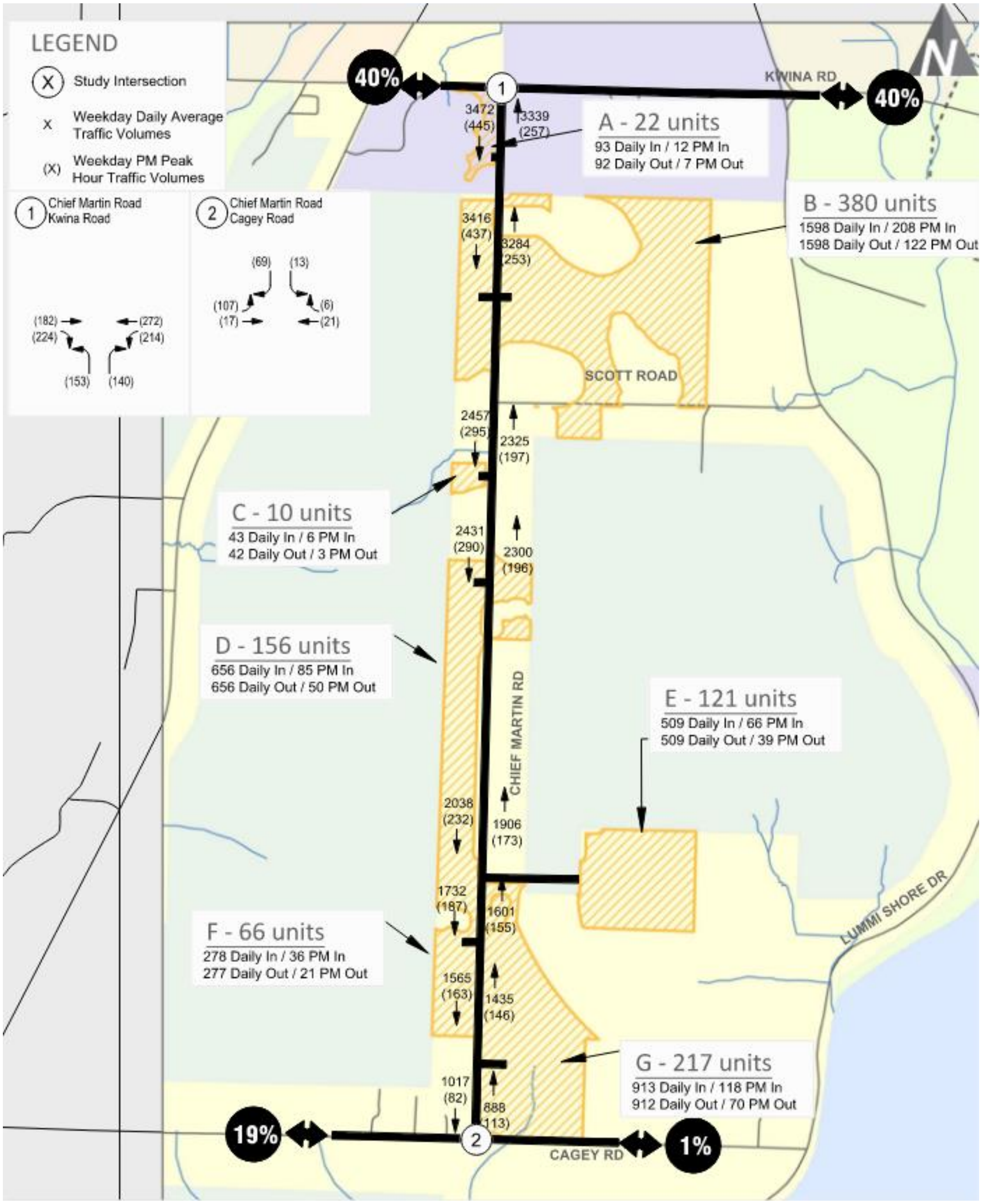



**Figure 6-2. Low Density Scenario Daily Volumes and Future PM Peak Hour Intersection Volumes (972 dwelling units).**

**Table 6-1. Estimated Vehicle Trip Generation, Low-Density Scenario**

| Land Use[1] | Size | Daily Trips | Daily Trips In | Daily Trips Out | Daily Trips Total | PM Peak Hour Trips In | PM Peak Hour Trips Out | PM Peak Hour Trips Total |
|---|---|---|---|---|---|---|---|---|
| Single Family Home (LU 210) LOW Density | 972 du | 8,176 | 4,088 | 4,088 | 8,176 | 531 | 312 | 843 |

Note: du = dwelling unit.

1. Trips rates from ITE Trip Generation Manual, 11th Edition.

At the high end of the range of development (9 units/buildable acre), 2,367 PM peak hour trips would be generated. The daily trips along Chief Martin Road, by road segment, are shown Figure 6-3. Figure 6-3 also shows the PM peak hour turning movement volumes at the two terminal intersections. Note that the volumes increase and decrease at intervening intersections along the length of Chief Martin Road. This represents drivers turning into and out of the individual buildable areas.

**Table 6-2. Estimated Vehicle Trip Generation,** High**-Density Scenario**

| Land Use[1] | Size | Daily Trips | Daily Trips In | Daily Trips Out | Daily Trips Total | PM Peak Hour Trips In | PM Peak Hour Trips Out | PM Peak Hour Trips Total |
|---|---|---|---|---|---|---|---|---|
| Single Family Home (LU 210) HIGH Density | 2,916 du | 22,466 | 11,233 | 11,233 | 22,466 | 1,491 | 876 | 2,367 |

Note: du = dwelling unit.

1. Trips rates from ITE Trip Generation Manual, 11th Edition.

### Trip Distribution

Existing trip distribution patterns to/from the project site the patterns shown by the collected turning movement counts at the study intersections. The existing distribution of trips in the area are approximately 55% west and 25% east of the Chief Martin Road/Kwina Road intersection. About 19% of study area trips travel west and 1% east at Chief Martin Road/Cagey Road. We would anticipate that future buildout distribution of trips would change to approximately 40% west and 40% east of the Chief Martin Road/Kwina Road intersection.

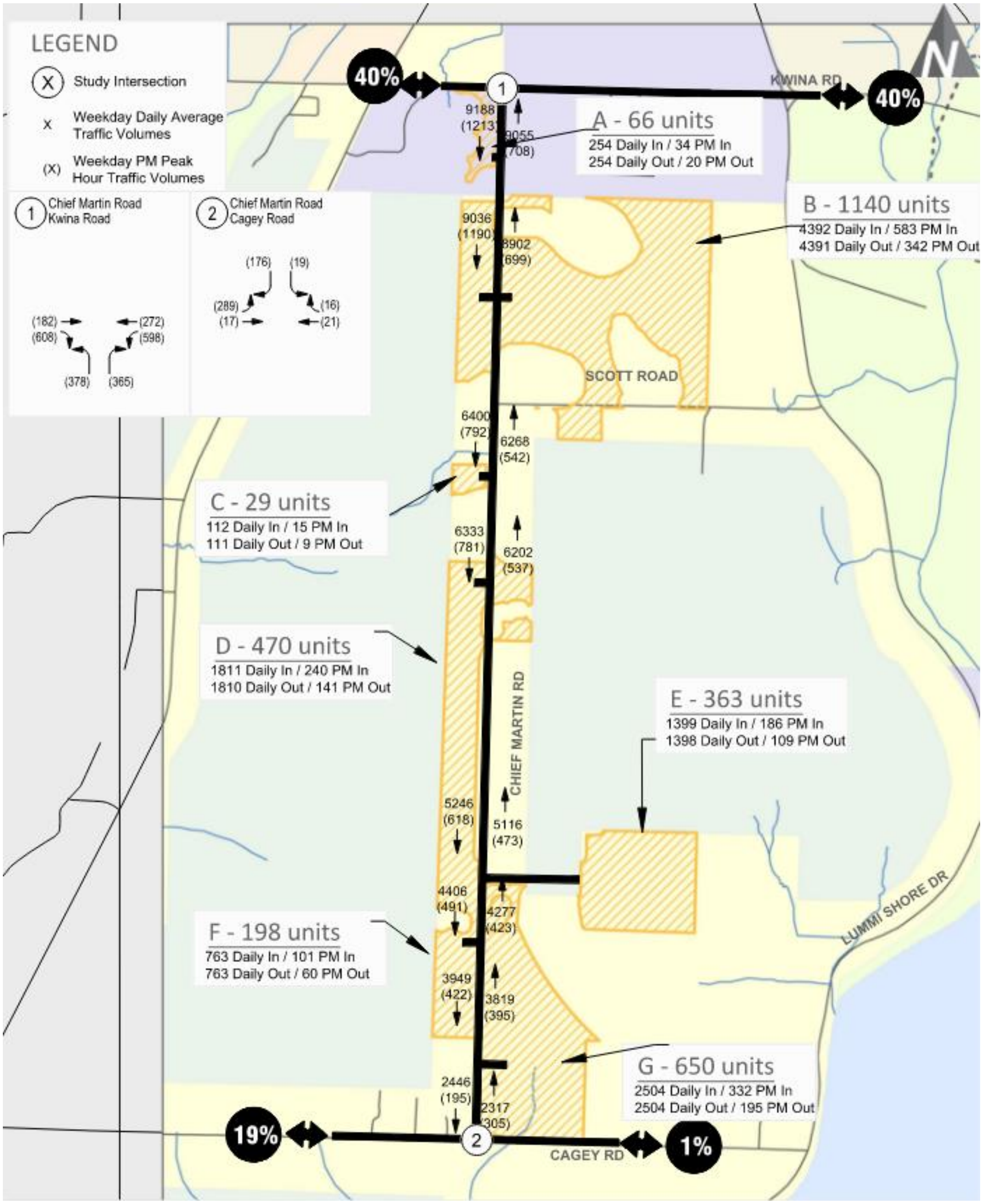



**Figure 6-3. High Density Scenario Daily Volumes and Future PM Peak Hour Intersection Volumes (2,916 dwelling units).**

## 6.3. *Roadway Cross Sections – Chief Martin Road*

**Low-Density Development Scenario**

The surrounding roadways include a variety of segment cross sections. Some have just narrow travel lanes and swales, others have segments with curb/gutter/sidewalk, transit stops and shelters. Some area roadways have wider travel lanes with wider shoulders than others. As discussed under Existing Conditions, the existing area roadways are a mix of rural major collectors, rural minor collectors, and rural local roads. **The proposed cross section of Chief Martin Road in the low-density scenario would most closely resemble that of Kwina Road.** Kwina Road has areas of bike lanes/shoulders wide enough for bikes, and curbs, gutters, and sidewalks.

Figure 6-4 shows a typical Whatcom County Minor Urban Arterial cross section. This would include 12’ travel lanes, 5-foot sidewalks and bike lanes, curb and gutter, and lighting/utilities (not shown here) in the drainage slope outside the sidewalk. The total roadway cross section is 46 feet (without the lighting) in a 60-foot right of way, which is the right of way width for Chief Martin Rd.


**Figure 6-4. Whatcom County Typical Cross Section, Minor Urban Arterial**

The daily trip volumes associated with the low-density development level indicate a two-travel lane roadway would accommodate demand. Improvements are recommended to the existing Chief Martin Road cross section, for functional and aesthetic reasons. We propose a Minor Urban Arterial classification. The road would be constructed within a, ideally, 70-ft right-of-way. The total roadway width would be 50 feet, with 11-foot travel lanes, 5-foot bike lanes, curb and gutter, 5-foot sidewalks, and an additional 3 feet on either side for street trees and lighting. Consideration should be given to providing transit stops or shelters, as Lummi Transit already operates on the surrounding roadways. The posted speed limit should be 30 to 35 mph. Transit shelters could be accommodated in the sidewalk area, or in small concrete bump outs created for just the shelter.

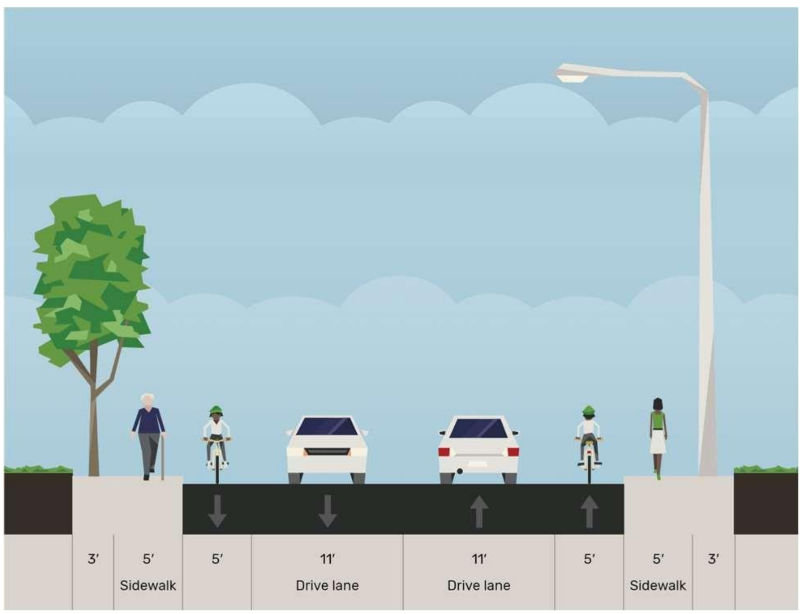



**Figure 6-5. Low Density, Minor Urban Arterial, 2-lane Scenario**

The sidewalks or mixed-use trail sections could be connected into the new neighborhoods to facilitate use of transit. Sidewalk/mixed use trails would also facilitate walking trips for either recreation or to reach destinations. If a mixed-use trail is desired, it might be similar to the Red River Trail that parallels Haxton Way north of Kwina Road. With an 8–9-foot mixed-use trail on each side, the total cross-section width could be reduced by 2-4 feet.

A number of neighborhood road access points will occur along Chief Martin Road. This will likely require roadway widening at some intersections, to facilitate turning traffic. A two-way-left-turn lane (TWLTL) could also be used to facilitate turns. Widening at intersections approaches is recommended over the TWLTL. Continuous TWLTLs widen the visual roadway, which in turn encourages driver speeding. The potential cross-section at intersections (or with a TWLTL) is shown in Figure 6-7. This cross section would include 11-foot travel lanes, a 10-foot turn lane, curb and gutter, 5-foot bike lanes, 5-foot sidewalks, and street trees and lighting, for a total roadway width of 60 feet and a right-of-way of a minimum of 70 feet (this would require acquisition of 10 ft of additional right-of-way.

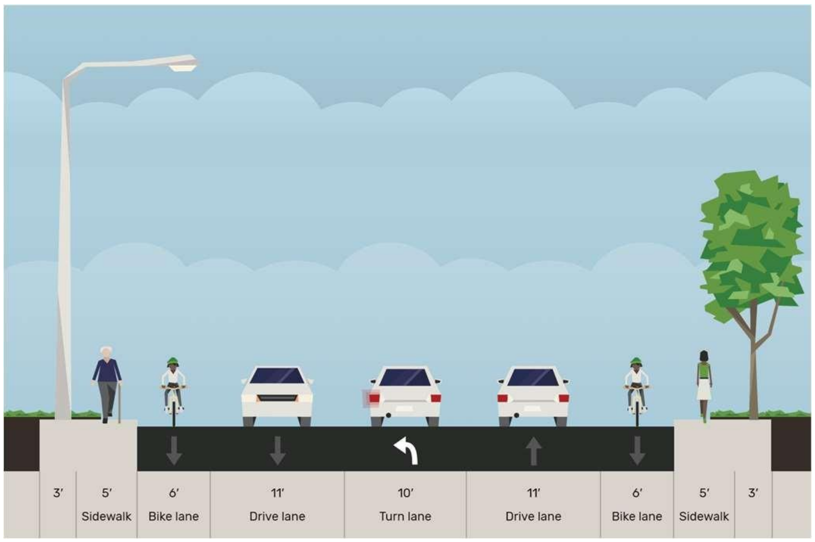



**Figure 6-6. Low Density, Minor Urban Arterial, 3-lane Scenario**

**High-Density Development Scenario**

As shown in Table 2, above, the PM peak hour and daily volumes related to the high-density scenario would generate much greater traffic volumes than the low-density scenario. The volumes would far exceed that currently carried by any of the surrounding roadways. At these high daily and peak hour volumes, a 4-lane cross section is advised. This would be the largest roadway in the general area. Center turning lanes should also be added. This would effectively make it a five-lane cross-section. The total right of way width would be 90 feet, requiring acquisition of 30 feet of additional right of way. The means top address these high traffic volumes is described in more detail in the appendices

## 6.4. *Intersection Operations*

A second type of operational analysis was used to develop the Chief Martin Road cross sections: intersection operational levels of service. These analyses result in a level of service that ranges from LOS A (good) to LOS F (poorest). If a roadway or intersection will operate at a lower level of service (LOS E or F), then the configuration should be modified to allow for a smoother driver experience.

For the intersections of Chief Martin Road/Kwina Road and Chief Martin Road/Cagey Road, analyses are based on the Highway Capacity Manual (HCM) 7th Edition, Transportation Research Board methodology using Synchro software version 12.8 Table 6-3 summarizes the existing, future (2043) without-project and future (2043) with-project operational conditions. These analyses assume the current intersection configurations: one travel lane on each approach and stop sign controls on the Chief Martin Road leg of the intersections.

**Table 6-3. Weekday PM Peak Operation Summary – Current Intersection Configurations.**

| | 2023 Existing | | | 2043 With-Project (Low) | | | 2043 With-Project (High) | | |
|---|---|---|---|---|---|---|---|---|---|
| Intersection | LOS[1] | Delay[2] | WM[3] | LOS[1] | Delay[2] | WM[3] | LOS[1] | Delay[2] | WM[3] |
| ***Weekday PM Peak Hour*** | | | | | | | | | |
| Chief Martin Road/Kwina Road | B | 10 | NB | E | 42 | NB | F | 300> | - |
| Chief Martin Road/Cagey Road | A | 9 | SB | A | 9 | SB | B | 11 | SB |

1. Level of service, based on 2016 Highway Capacity Manual methodology.
2. Average delay in seconds per vehicle.
3. Worst movement reported for unsignalized intersections.

**Chief Martin Road/Kwina Road Intersection**

In the low-density scenario with the same intersection configuration as today, the intersection of Chief Martin Road/Kwina Road would operate at LOS E. The longest delays would be experienced by those drivers on Chief Martin Road waiting to turn left or right onto Kwina Road. Consideration should be given to additional turn lanes and perhaps higher traffic controls (such as a roundabout, all-way stop, or possibly a signal) for this intersection. Drivers may get frustrated with the relatively long waits at this intersection.

For the high-density scenario, average delays for drivers become excessive, at over 300 seconds average delay. The high volumes at this intersection would create the need for higher level traffic controls such as a traffic signal or roundabout at this intersection. Additional turn lanes would also be needed to accommodate the high volumes. The high volumes would also create the need for a wider cross-sectional width for Chief Martin Road.

**Chief Martin Road/Cagey Road Intersection**

The intersection of Chief Martin Road/Cagey Road would continue to operate well in both scenarios. No changes are recommended at this point in the master planning process.

## 6.5. *Potential Improvements to alleviate traffic volumes:*

1. Widening Kwina Road to four or even five lanes between Haxton Way and Lummi Shore Road.
2. Improving the intersection of Kwina Road, Lummi Shore Road, and Marine Drive.
3. Improving Scott Road to allow an alternative route out of the development area, which may in turn require improvements to Lummi Shore Road north of Scott Road.
4. Constructing new east-west roadways from Chief Martin Road to Lummi Shore Road or Haxton Way.
5. Improvements to the roundabout at Haxton Way/Kwina Road.
6. Internal intersections may require widening or other improvements, such as signalization, to facilitate smooth traffic flow (see below).

## 6.6. *Cost Estimates*

The estimated cost (see appendices for detail) for the Chief Martin Rd construction (not including utilities is $32,000,000.

## 6.7. *Conclusions*

The initial development of Chief Martin Rd would include two travel lanes, bike lanes, sidewalks, utilities, and lighting.

For the lower density development scenario, Chief Martin Road would carry higher volumes than Kwina Road or other area roadways do today. Kwina Road in its existing condition should be able to accommodate the amount of traffic projected in the low-density scenario without additional impacts.

For up to 1,200 units added on Chief Martin Rd, a two-lane road with left turn lanes at key points will probably be sufficient. A left turn lane should be installed at the north end of Chief Martin R (at Kwina Rd) when the road is first improved. At later dates, left turn lanes may be needed at Scott Rd and Cagey Rd as well.

For planning purposes, once the development exceeds approximately 1,200 units, road upgrades will be needed to accommodate high traffic volumes. For example, Chief Martin Rd would need to be expanded to four lanes in the northern segments to meet traffic demand.

Additional work will be needed to plan for high-density growth. Planning for right of way acquisition should begin early on. Future evaluations of final buildout density should be undertaken as development of the Chief Martin Rd corridor proceeds.

Transit planning should include land acquisition for pullouts or design of methods to safely allow buses to pull over and reenter traffic with obstructing traffic and bike lanes.

Internal Intersections

Some of the buildable areas would have a high number of units, which translates to higher volumes turning in and out from Chief Martin Road. When developing plans for individual buildable areas, consideration should be given to sufficient intersection designs. Spacing and number of intersections will be key considerations when plots are created. For instance, in the high-density scenario, a single access intersection for Lot B would not function at acceptable levels with side street stop sign controls. Additional access points might need to be created along the face of this buildable area, or access provided along Scott Road, to relieve the burden at a single intersection.

# 7. GLOSSARY

| | |
|---|---|
| A | Amps |
| ac-ft | Acre-feet. A volume equal to 1 foot of water over 1 acre (~325,000 gallons) |
| ADD | Average Daily Demand for water system |
| bgs | below ground surface |
| CFS | Cubic feet per second (usually stormwater flow rate) |
| DOH | Washington State Department of Health (Division of Drinking Water) |
| EPA | Environmental Protection Agency |
| ERU | equivalent residential units |
| GPD | gallons per day |
| GPM | Gallons per minute |
| I&I | Infiltration and Inflow (in sewers) |
| KW | Kilowatt (measure of electricity flow; equals 1000 Watts) |
| KWh | Kilowatt-hours (standard measure of electricity consumption) |
| LF | linear feet |
| LOS | Level of Service |
| MDD | Maximum Day Demand for water system |
| MGD | million gallons per day |
| NEC | National Electrical Code |
| NFPA | National Fire Protection Association |
| PHD | Peak Hour Demand for water system |
| PS | pump station |
| PSE | Puget Sound Energy (electric utility) |
| PSI | pounds per square inch, pressure |
| ROW | Right of Way |
| SFR | single-family residential units |
| TDH | Total dynamic head (pressure – static head plus other head) |
| TWLTL | two-way-left-turn lane |
| V | volts |
| WWHM | Western Washington Hydrology Model |
| WWTP | Wastewater Treatment Plant |

# 8. APPENDICES

**Appendix A – Water System Appendices**

**Appendix B – Sewer System Appendices**

**Appendix C – Transpo Group Technical Memorandum**

**Appendix D – Overall Cost Estimate for Buildout**

**Appendix E – Chief Martin Road Conceptual Plan Set**