3. POTABLE WATER SYSTEM

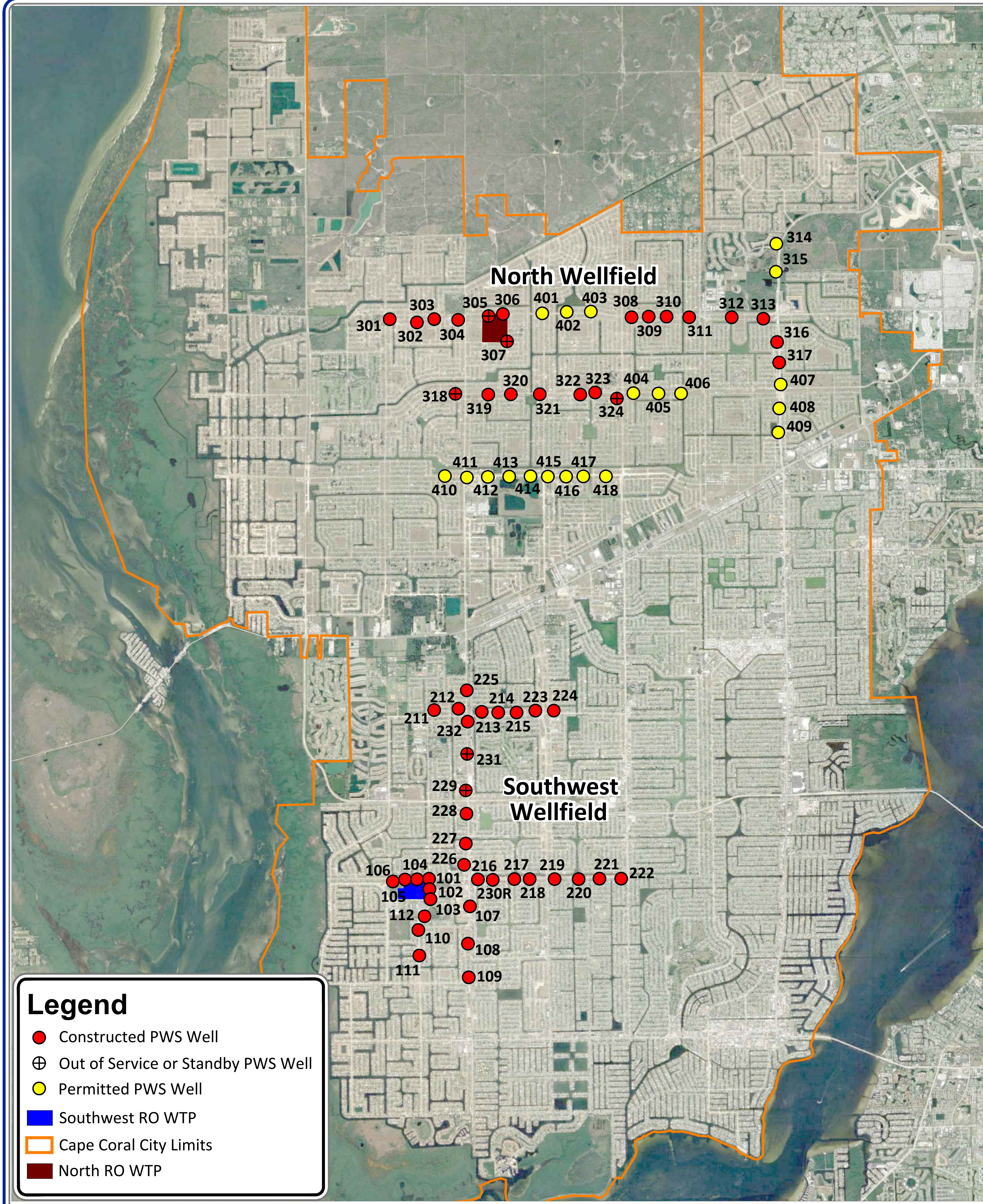
The system includes raw water supply wellfields, water treatment plants and a water transmission and distribution system consisting of pumping facilities, storage tanks, and transmission and distributing network.

The City's existing potable water system is comprised of a raw water supply and transmission system, two potable water treatment plants, a transmission and distribution system consisting of pumping facilities and piping network and a concentrate disposal system at each treatment facility.

3.1 Raw Water

The City maintains approximately 25 miles of raw water transmission mains as of the July 2022 inventory, which transport raw water from two brackish water wellfields to the 18 MGD Southwest RO WTP and the 12 MGD North RO WTP as shown in **Figure 3-1**. The wellfields consist of 56 wells that withdraw brackish groundwater from the Lower Hawthorn aquifer of the Upper Floridan aquifer system. There are currently 50 operational wells. The remaining 6 wells are either used only if needed, or equipment has been removed and the well is no longer operational. The City's raw water wells are generally open to between 450 and 850 feet below land surface (bls).

The wellfields currently have the capacity to provide approximately 31 MGD as indicated in **Table 3-1 and 3-3**. Total annual combined raw water supply from both the wellfields during the past 5 years has ranged between 300 and 575 MGM (10 and 20 MGD) with an average of 400 MGM (12 MGD) as shown in **Figure 3-2**.



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	N 0 4000 8000 APPROXIMATE SCALE IN FEET
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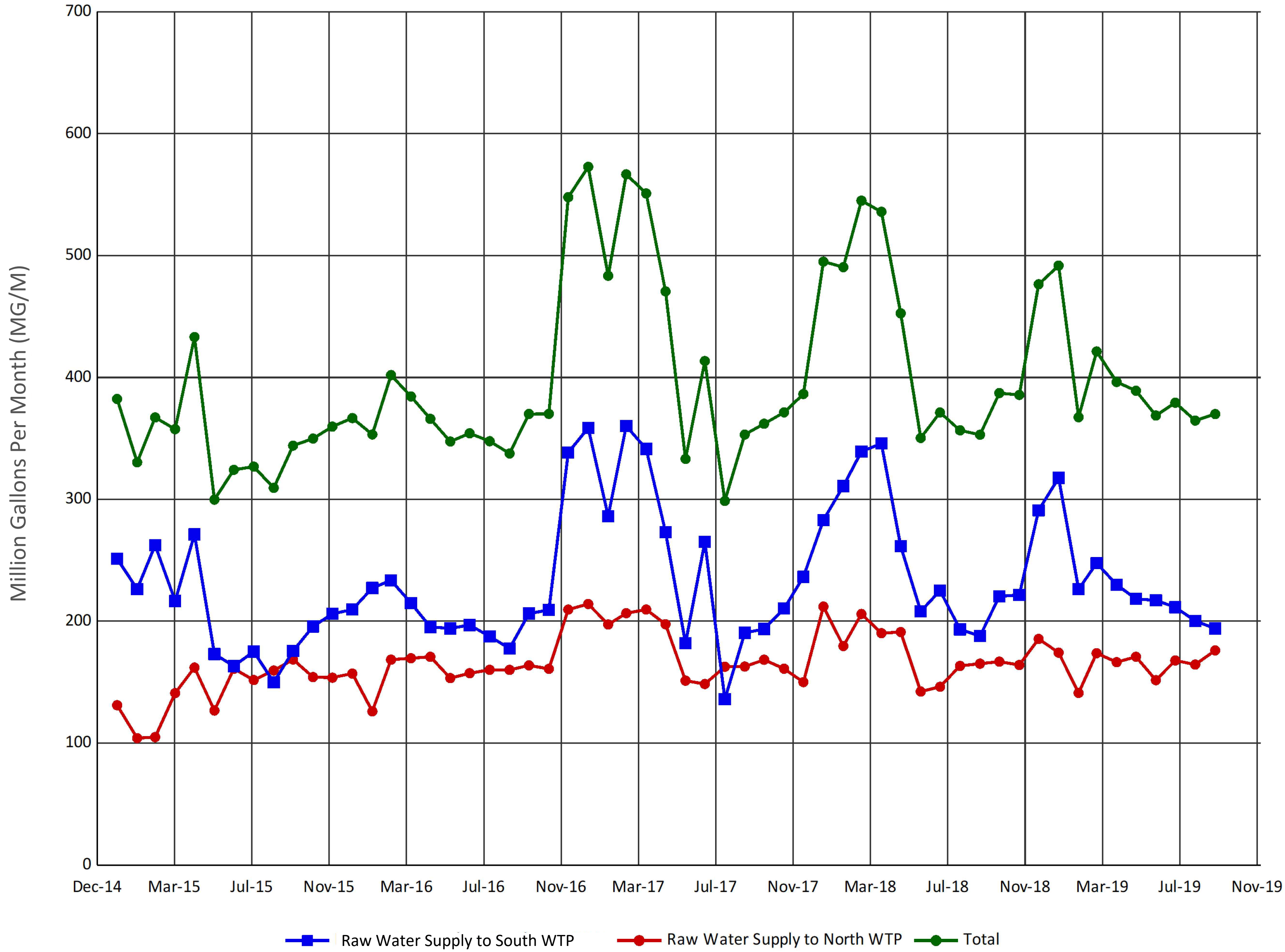
Figure 3-1: Brackish Water Wellfield Locations.



PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437
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Figure 3-2: 5-Year Raw Water Supply.

The water treatment plants have a combined treatment capacity of 30 MGD (18 MGD for the Southwest WTP and 12 MGD for the North WTP). To generate 30 MGD of finished water requires 37.5 MGD raw water supply (assuming 80% recovery efficiency). Considering the current operational maximum capacity of the wellfields is roughly 31 MGD, the deficit in raw water capacity to run the WTPs at full capacity is approximately 6.5 MGD.

There are 18 Water Use Permitted proposed wells for the North wellfield that have not yet been installed. However, some of the permitted well locations may no longer be desirable and will be evaluated based on the Comprehensive Water Supply evaluation currently under way. The proposed wells when installed are expected to increase the raw water capacity by between 10 to 12 MGD.

The permitted maximum monthly and average annual allocations from the raw water wells are 1,312 MG and 14,326 MG respectively which is equivalent to 43.7 and 39.2 MGD. The current wellfields can produce approximately 70% of the maximum monthly permitted quantity. Detailed discussion of the individual WTPs and the associated wellfields are provided below.

3.1.1 Southwest Wellfield

The Southwest wellfield supplies raw water to the Southwest RO WTP which began operation in 1977 and is located on SW 32nd Street, west of Agualinda Boulevard. This wellfield consists of 32 operational production wells and is aligned in a north-south direction along the Chiquita Boulevard and in the east-west direction along Gleason and Trafalgar Parkways (**Figure 3-1**). The monthly raw water pumped from the Southwest wellfield generally ranges between approximately 140 and 360 MGM, (i.e., between 5 and 12 MGD), with an average of 250 MGM or 8 MGD. Approximately 80% of the feedwater at the Southwest RO WTP is recovered as usable finished water for potable system distribution with the remaining 20% RO concentrate disposed via two on-site Class I injection wells located at the SW RO WTP/ SW WRF. **Table 3-1** shows the individual well total and cased depths and flow.

Table 3-1: Well Depths & Flow Rates of Raw Water Supply Wells at the South Wellfield

Well ID	Cased Depth (ft)	Total Depth (ft)	Flow Rate (gpm)	
101	362	745	450	
102	347	685	500	
103	345	705	450	
104	350	700	325	
105	345	765	400	
106	564	800	300	
107	357	752	525	
108	345	752	450	
109	350	748	300	
110	350	748	500	
111	500	750	400	
112	455	721	400	
211	599	762	400	
212	599	742	450	
213	590	765	350	
214	529	702	300	
215	558	782	350	
216	456	707	525	
217	440	700	550	
218	495	722	425	
219	490	710	525	
220	508	720	450	
221	510	720	500	
222	515	642	500	
223	420	652	450	
224	389	709	400	
225	440	715	475	
226	460	715	450	
227	429	715	350	
228	460	714	400	
229	460	712	Out of Service	
230R	450	700	450	
231	435	703	Out of Service	
232	470	634	500	
			Total	13,800 GPM
			Total	20 MGD

3.1.1.1 Southwest Wellfield Water Quality Considerations

A review of average composite wellfield chloride concentrations, weighted by individual recommended well flow rates, indicates that the composite salinity of the wellfield has steadily increased during the last 10 years from approximately 825 milligrams per liter (mg/L) to 975 mg/L or about 20%. Average total pumpage from all the wells in the wellfield during this time increased from approximately 5 to 8 MGD. Refer to **Figure 3-3** for composite chloride concentrations and total pumpage from all the wells from the Southwest wellfield.

Approximately 50% of the wells comprising the wellfield have chloride concentrations less than 1,000 mg/L, approximately 38% have chloride concentrations in the range of 1,000 and 2,000 mg/L, and only four wells (RO-104, RO-214, RO-228, and RO-229) exhibit chloride concentrations above 2,000 mg/L. Data indicates that the wellfield is generally pumped at the recommended flow rate. Refer to **Figure 3-4** and **Figure 3-5** for maps showing chloride concentration ranges and increases in chloride concentration per 100 million gallons (MG) of water withdrawn for wells comprising the wellfield. Some stabilization of chloride concentrations indicated over the last two years may be attributed to implementation of a wellfield management plan which establishes a recommended maximum pumping rate for each well.

3.1.1.2 Southwest Wellfield Production Capacity Considerations

Specific capacity data based on pumping tests conducted on each well in 2011 and 2019/2020 are tabulated and compared in **Table 3-2**. The period of 2011 to 2018 corresponds to a time when efforts to restore productive capacity, by well rehabilitation methods, declined. Since early 2019 the City's well rehabilitation efforts have increased. Decline in specific capacity (defined as pumping rate divided by pumping water level decline from static) is a natural occurrence which happens due to precipitation of calcium carbonate in the near borehole area, thus occluding porosity and reducing well yield. Data suggests that about 22% of the wells in the wellfield have lost more than 40% of their capacity and about 28% of the wells have lost between 20% and 40% of their capacity. The remaining 50% of the wells show less than 20% reduction in capacity. As the wellfield ages, it is normal to see a reduction in capacity typically caused by the precipitation of calcium carbonate along the borehole walls. Wells showing significant reduction in specific capacity (>40%) are candidates for rehabilitation treatments such as rapid rate high dosage hydrochloric acid treatment. The reduction in capacities is currently being managed through regular and periodic treatments with acid. Refer to **Figure 3-6** for a map showing wells with high, medium, and low decline in specific capacities over time.

Table 3-2: Summary of Specific Capacity Change in the Southwest Wellfield

Well ID	2011	Current (See Note)	% Decline or Increase
101	26.2	16	-39%
102	16	14.4	-10%
103	26.3	14.3	-46%
104	22.3	5.3	-76%
105	16.6	9.5	-43%
106	3.1	5.5	77%
107	9	7.4	-18%
108	8.3	12.2	47%
109	2.5	25.7	928%
110	15.6	8.1	-48%
111	9.2	8.5	-8%
112	13.2	8.2	-38%
211	7.3	15.3	110%
212	5.7	9.7	70%
213	45.3	17.9	-60%
214	14.3	7.9	-45%
215	10.6	8.2	-23%
216	11.9	9.5	-20%
217	13.5	11.8	-13%
218	9.6	9.8	2%
219	9	7.6	-16%
220	7.9	8	1%
221	6.9	16.4	138%
222	8.3	12.7	53%
223	27.1	23.8	-12%
224	12.5	9.6	-23%
225	8.8	7	-20%
226	10.9	8.5	-22%
227	11.6	6.3	-46%
228	11.3	7.9	-30%
229	OS	OS	NA
230R	4.9	15.6	218%
231	OS	OS	NA
232	10.3	7.5	-27%

Note: Data for Well IDs shown in black are obtained from the Quarterly Report of Monthly Wellfield Data Review – Third Quarter, 2019, Table 9 (RMA, 2019). Data for Well IDs shown in red were obtained from the City staff and are based on 2020 acidification results. OS denotes "Out of Service".

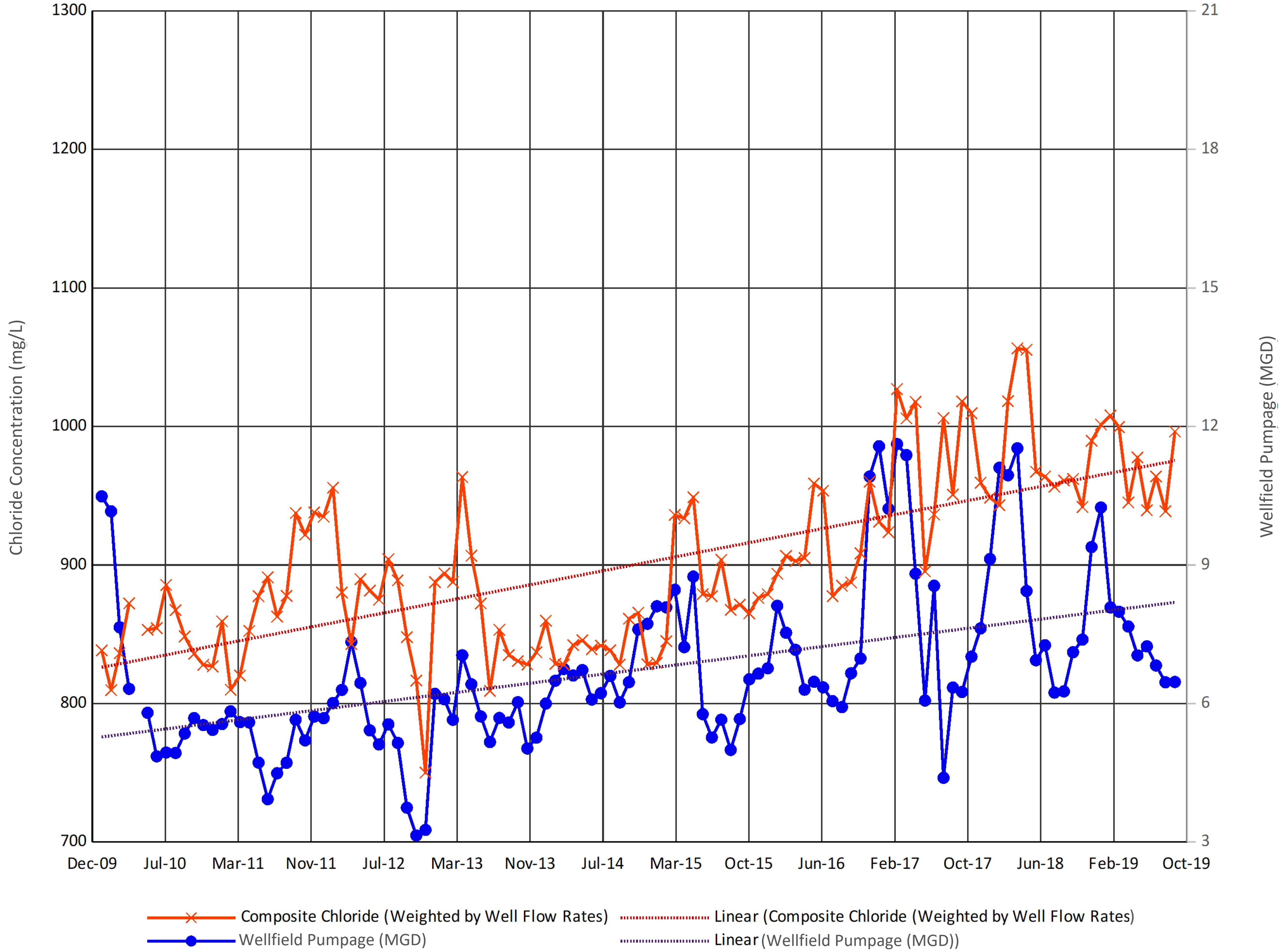

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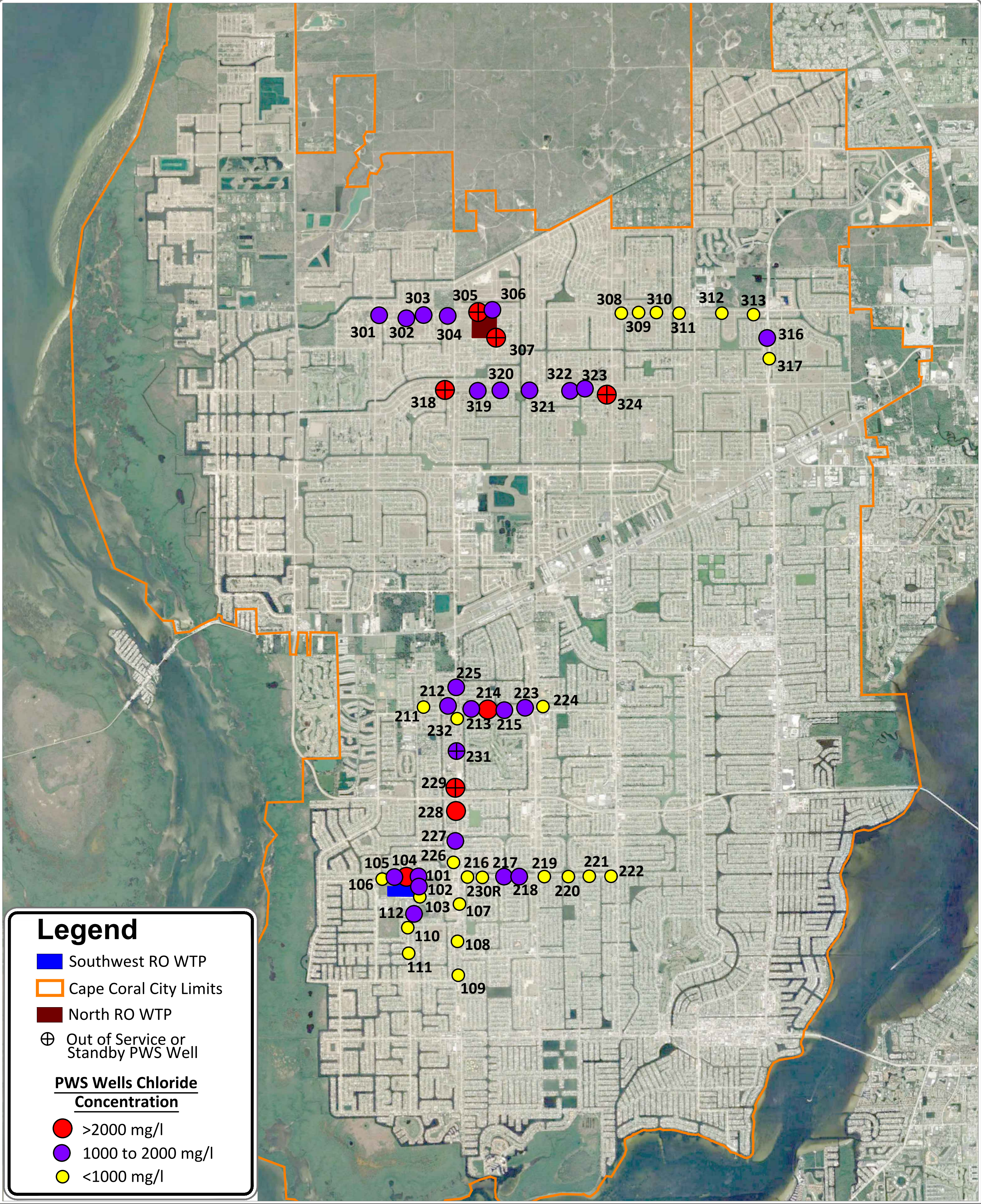
Figure 3-3: Plot Showing Composite Chloride Concentrations and Total Daily Pumpage in the Southwest Wellfield.



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	N 0 4000 8000 APPROXIMATE SCALE IN FEET
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-4: Map Showing Chloride Concentrations in Individual Supply Wells as of 2019.

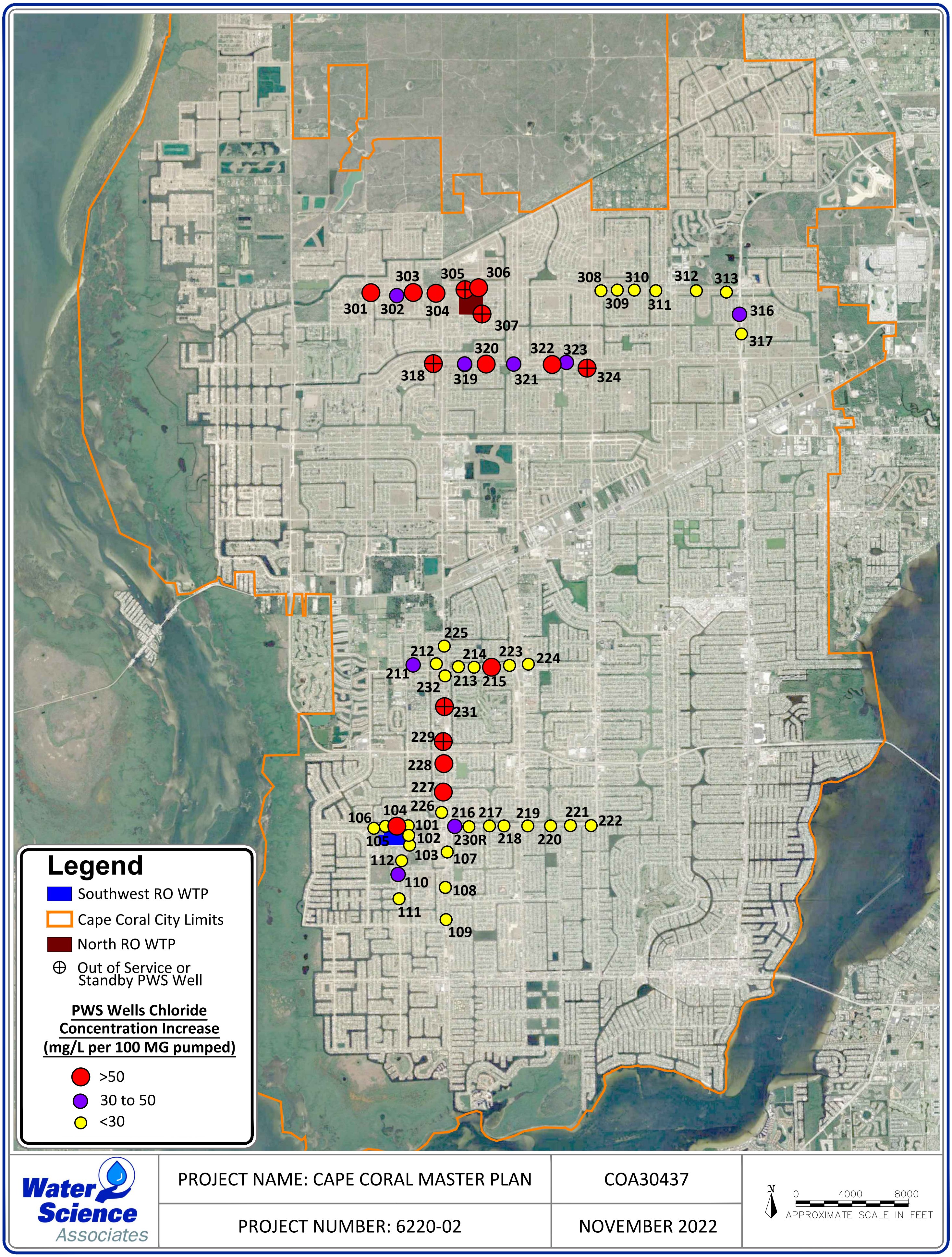
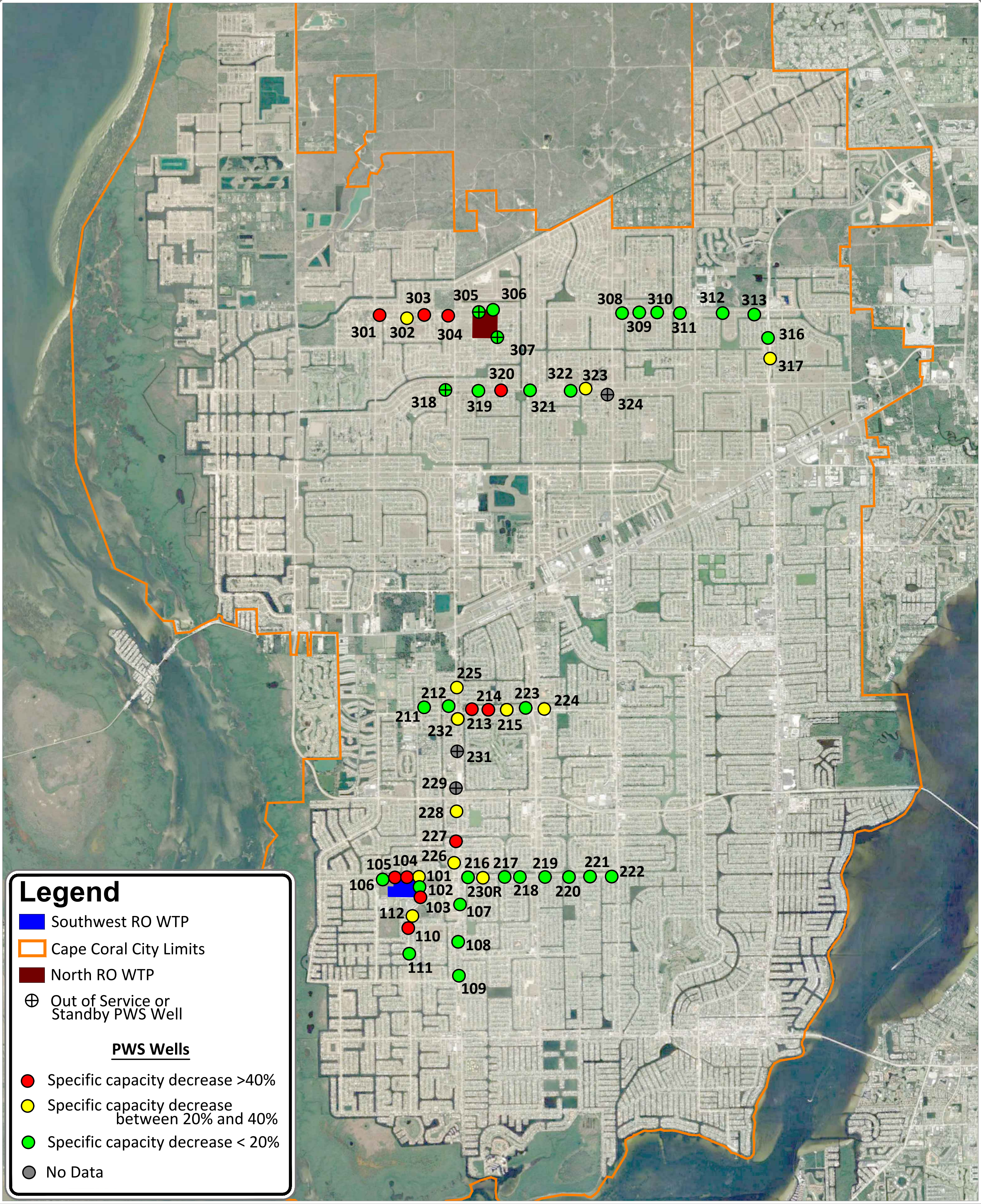


Figure 3-5: Map Showing the Chloride Concentration Increase Per 100 MG Withdrawal in Individual Wells (2011-2020).



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	N 0 4000 8000 APPROXIMATE SCALE IN FEET
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-6: Map Showing Decline in Well Specific Capacities (2011 to 2020).

3.1.2 North Wellfield

The North wellfield supplies raw water to the North RO WTP, which is located north of Pine Island Road (SR-78) and began operation in 2010. The North wellfield consists of 18 operational production wells and is aligned in an east-west direction along Kismet Parkway and Diplomat Parkway **(Figure 3-1).** The monthly raw water pumped from the North wellfield generally ranges between approximately 100 and 200 MGM, (i.e., between 3 and 7 MGD), with an average of 150 MGM or 5 MGD. Approximately 80% of the feedwater at the North RO WTP is recovered as usable finished water for potable system distribution with the remaining 20% concentrate disposed via a single Class I injection well located at the North RO WTP. Refer to **Table 3-3** for individual well total and cased depths and flow. Refer to **Figure 3-7** for monthly pumpage from the North wellfield.

Table 3-3: Well Depths & Flow Rates of Raw Water Supply Wells at the North Wellfield

Well ID	*Cased Depth (ft)*	*Total Depth (ft)*	*Flow Rate (gpm)*
301	500	762	225
302	453	714	300
303	453	672	300
304	476	712	300
305	440	636	0
306	445	645	350
307	460	614	0
308	505	702	700
309	503	702	650
310	520	722	575
311	575	802	450
312	428	520	650
313	542	805	450
316	590	883	450
317	745	1100	550
318	655	864	0
319	468	536	250
320	592	840	250
321	565	800	325
322	630	840	250
323	630	832	400
324	643	866	0
		Total	7475 GPM
		Total	11 MGD

3.1.2.1 North Wellfield Water Quality Considerations

A review of average composite wellfield chloride concentrations, weighted by individual recommended well flow rates, indicates that the salinity in the wellfield has steadily increased during the last 10 years from approximately 800 mg/L to 1,100 mg/L or about 35%. Average total pumpage from all the wells in the wellfield during this time remained stable at approximately 5 MGD. Refer to **Figure 3-7** for composite chloride concentrations and total pumpage from all the wells from the North wellfield. Based on 2019 data, four wells (RO-324, RO-318, RO-305 and RO-307) exhibit chloride concentrations above 2,000 mg/L. These wells are currently out of service or in a standby status to be used only when needed. Approximately 50% of the wells comprising the wellfield have chloride concentrations in the range of 1,000 and 2,000 mg/L and approximately 32% of the wells exhibit chloride concentrations less than 1,000 mg/L.

Data indicates that the wellfield is generally pumped at the recommended flow rate and noticeable decline in water quality (chloride concentration above 2,000 mg/L) has occurred in only a limited number of wells. Some stabilization of chloride concentrations indicated over the last two years may be attributed to the implementation of a wellfield management plan which establishes a recommended maximum pumping rate for each well. Refer to **Figure 3-4** and **Figure 3-5** for maps showing chloride concentration ranges and increase in chloride concentration per 100 million gallons (MG) of water withdrawn for wells comprising the wellfield. In general, the wells in the western portion of the wellfield show higher salinity compared to the eastern portion of the wellfield. Degradation of the water in these wells may be occurring due to vertical upconing or preferential flow of higher salinity water present in deeper aquifer(s). Individual well sites or areas of the wellfield showing accelerated increases in salinity are indicative of enhanced vertical flow beneath the primary production zone.

3.1.2.2 North Wellfield Production Capacity Considerations

Specific capacity data based on tests conducted on each well in 2011 and 2019/2020 are tabulated and compared in **Table 3-4**. Data suggests that approximately 19% of the wells in the wellfield have lost more than 40% of their capacity and about 10% of the wells have lost between 20% and 40% of their capacity. The remaining 71% of the wells show less than 20% reduction in capacity. Most of the wells are maintaining their initial capacities, which reflects the relatively young age of the wellfield. The reduction in capacities is typically caused by precipitation of calcium carbonate along the borehole walls. The reduction in capacities is currently being managed through regular and periodic treatments with acid. Refer to **Figure 3-6** for a map showing wells with high, medium, and low decline in specific capacities over time.

Table 3-4 Summary of Specific Capacity Change in the North Wellfield

Well ID	2011	Current (See Note)	% Decline or Increase
301	11.4	5.2	-54%
302	13.8	11.2	-19%
303	13.9	8.4	-40%
304	15.8	8	-49%
305	18.4	43.1	134%
306	18.4	23.5	28%
307	13.2	23.2	76%
308	9.2	13.2	43%
309	15.8	16	1%
310	13	11.2	-14%
311	5.1	7.7	51%
312	18.4	19.3	5%
313	9.5	10.6	12%
316	15.7	13.7	-13%
317	16.1	10	-38%
318	8.2	8	-2%
319	15.8	12.9	-18%
320	9.3	4.7	-49%
321	8.5	7.8	-8%
322	7.1	8.5	20%
323	15.4	10.6	-31%
324	7.6	OS	OS

Note: Data for Well IDs shown in black are obtained from the Quarterly Report of Monthly Wellfield Data Review – Third Quarter, 2019, Table 9 (RMA, 2019). Data for Well IDs shown in red were obtained from the City staff and are based on 2020 acidification results. OS denotes "Out of Service"

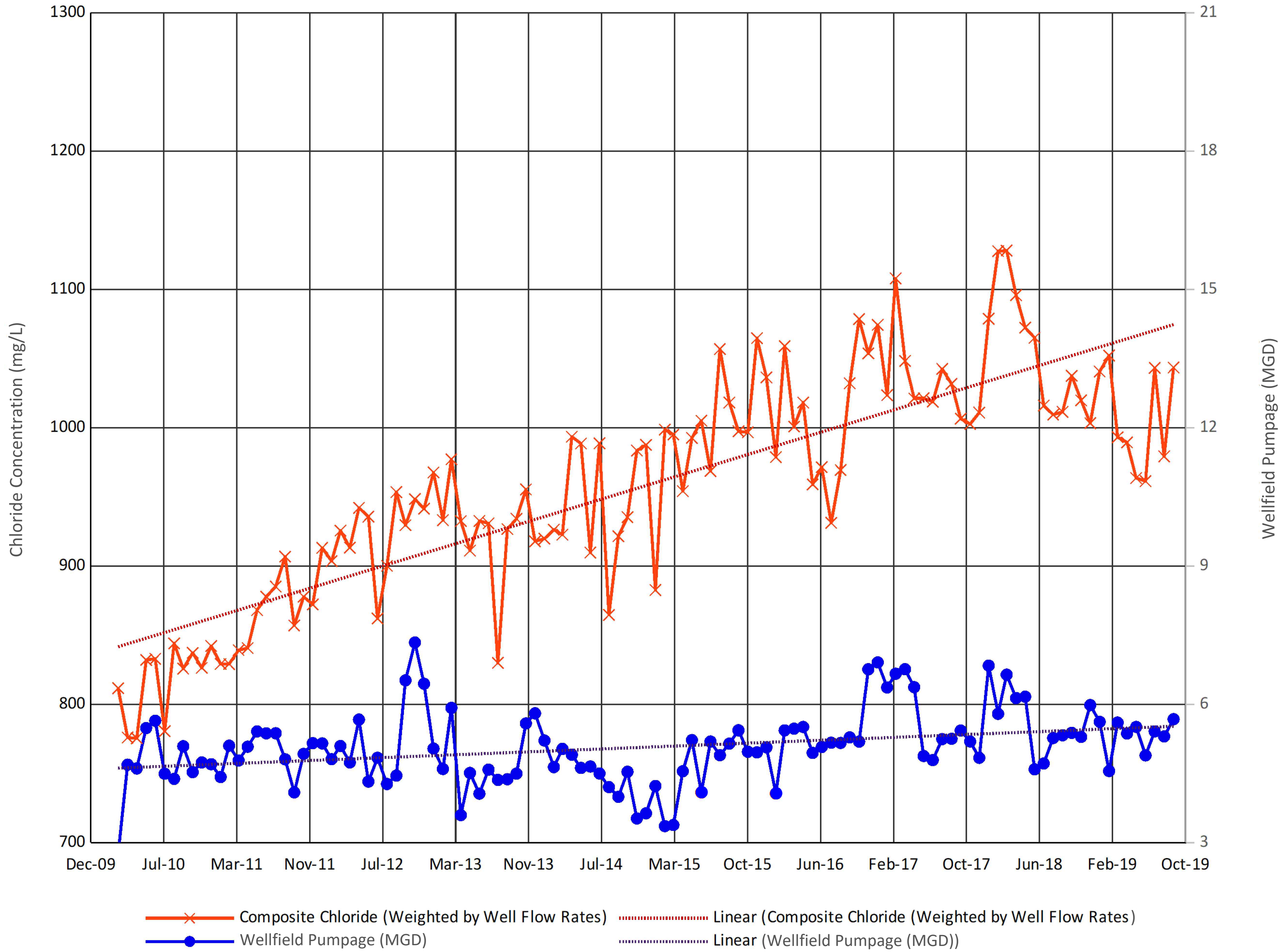


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Figure 3-7: Plot Showing Composite Chloride Concentrations and Total Daily Pumpage from the North Wellfield.

3.1.3 General Summary of Wellfield Conditions

The previous sections provided details regarding the conditions of both the North and South wellfield. A summary of the wellfield conditions is provided below:

- The current operational maximum capacity of both the north and southwest wellfields is roughly 31 MGD. There is currently a deficit of 6.5 MGD in raw water capacity to run the WTPs at full capacity. Future treatment plant expansions will require additional well construction.
- There are an additional 18 Water Use Permitted proposed wells for the North wellfield that have not yet been installed. The permitted well locations should be evaluated. The proposed wells when installed are expected to increase the raw water capacity by between 10 to 12 MGD.
- The permitted maximum monthly and average annual allocations from the raw water wells are 1,312 MG and 14,326 MG respectively which is equivalent to 43.7 and 39.2 MGD. The current wellfields can produce approximately 70% of the maximum monthly permitted quantity.
- The salinity in the Southwest wellfield has steadily increased during the last 10 years from approximately 825 milligrams per liter (mg/L) to 975 mg/L or about 20%. The chloride concentrations have been relatively stable. Stabilization of chloride concentrations during the last two years is the result of implementation of a wellfield management plan which establishes a recommended maximum pumping rate for each well. Data indicates that about 22% of the wells in the wellfield have lost more than 40% of their capacity and about 28% of the wells have lost between 20% and 40% of their capacity. Wells showing significant reduction in specific capacity (>40%) are candidates for rehabilitation treatments such as rapid rate high dosage hydrochloric acid treatment. The reduction in capacities is currently being managed through regular and periodic treatments with acid.
- The salinity in the North wellfield has steadily increased during the last 10 years from approximately 800 mg/L to 1,100 mg/L or about 35%. Noticeable decline in water quality (chloride concentration above 2,000 mg/L) has occurred in only a limited number of wells. Stabilization of chloride concentrations during the last two years is the result of implementation of a wellfield management plan which establishes a recommended maximum pumping rate for each well. Data indicates that approximately 19% of the wells in the wellfield have lost more than 40% of their capacity and about 10% of the wells have lost between 20% and 40% of their capacity. Most of the wells are maintaining their initial capacities, which reflects the relatively young age of the wellfield. The reduction in capacities is currently being managed through regular and periodic treatments with acid.

3.2 Treatment Facilities

The City currently owns and operates two water treatment plants that can process brackish groundwater resources the North RO WTP and the Southwest RO WTP.

3.2.1 North RO WTP

This facility is a two stage RO process and has a current permitted capacity of 12.0 MGD. The facility has a firm capacity of 9.0 MGD with one RO skid unit out of service. Construction of the North RO WTP facility began in 2006 and was completed in 2010. A treatment process block diagram for the North RO WTP is presented in **Figure 3-8** and the facility site plan is shown in **Figure 3-9**.

Raw water to this facility undergoes pretreatment using sulfuric acid, polyacrylic acid, and filtration. The sulfuric acid reduces the water's pH, while the polyacrylic acid is added as a scale inhibitor. Flow through a 5-micron cartridge filter allows for removal of any materials that can otherwise damage the membranes. Following pretreatment, pressurized feed water is conveyed to each of four parallel treatment trains at the facility. Each train has a dedicated 400 horsepower (HP) feed pump and a production capacity of 2.5 MGD. Each membrane treatment train is a two-stage system with a first stage of 48 pressure vessels, which can be expanded to 54 pressure vessels; and a second stage of 24 pressure vessels, which can be expanded to 27 pressure vessels. With the feed water under pressure, pre-treated water can pass through the membranes, and the concentrate from the first stage becomes feed for the second stage. The product water from both stages is blended with raw water to add back minerals to meet target water quality parameters, improve the taste of the drinking water and increase overall recovery efficiency.

Figure 3-8: North RO WTP Process Flow Diagram

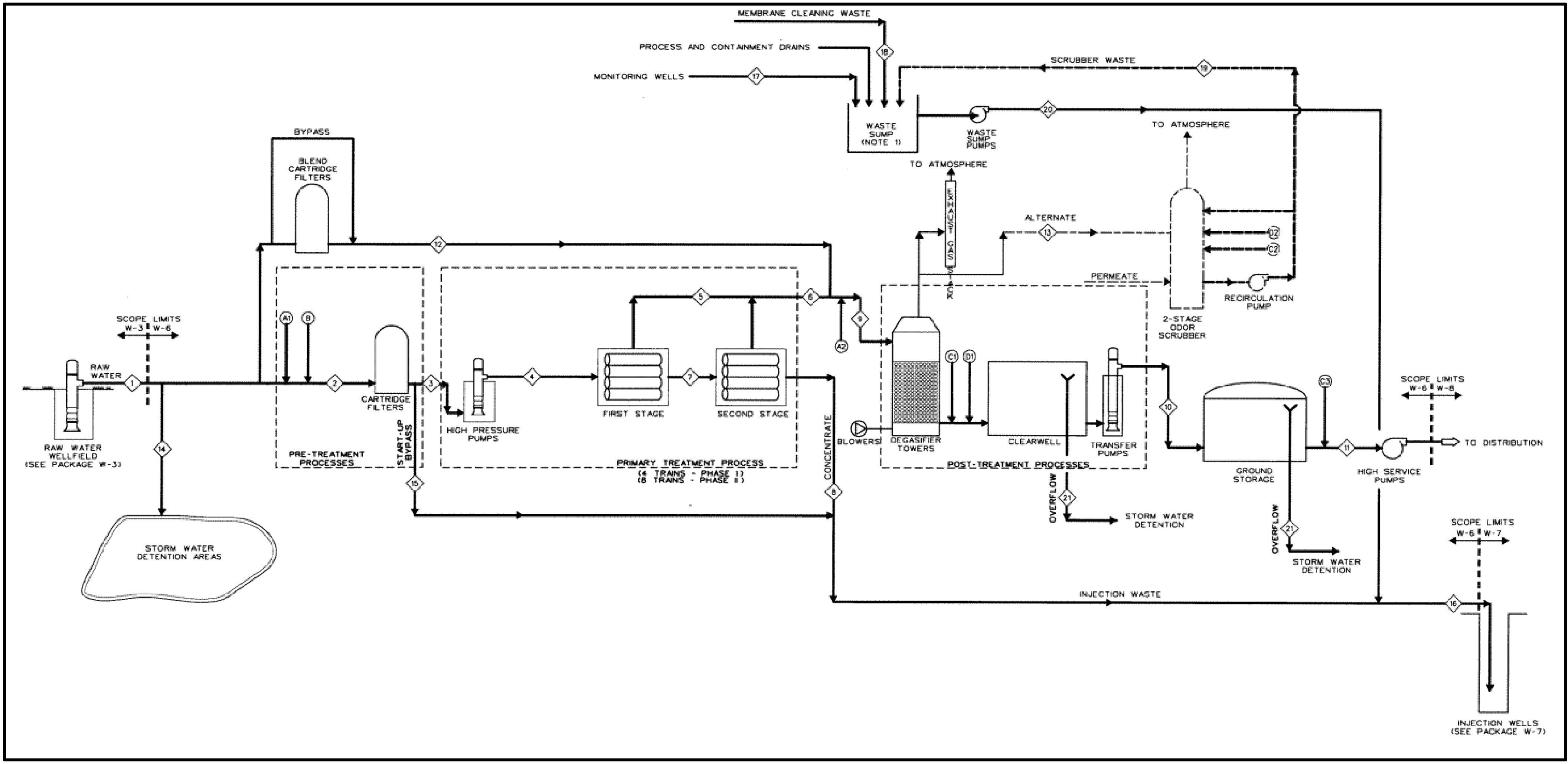
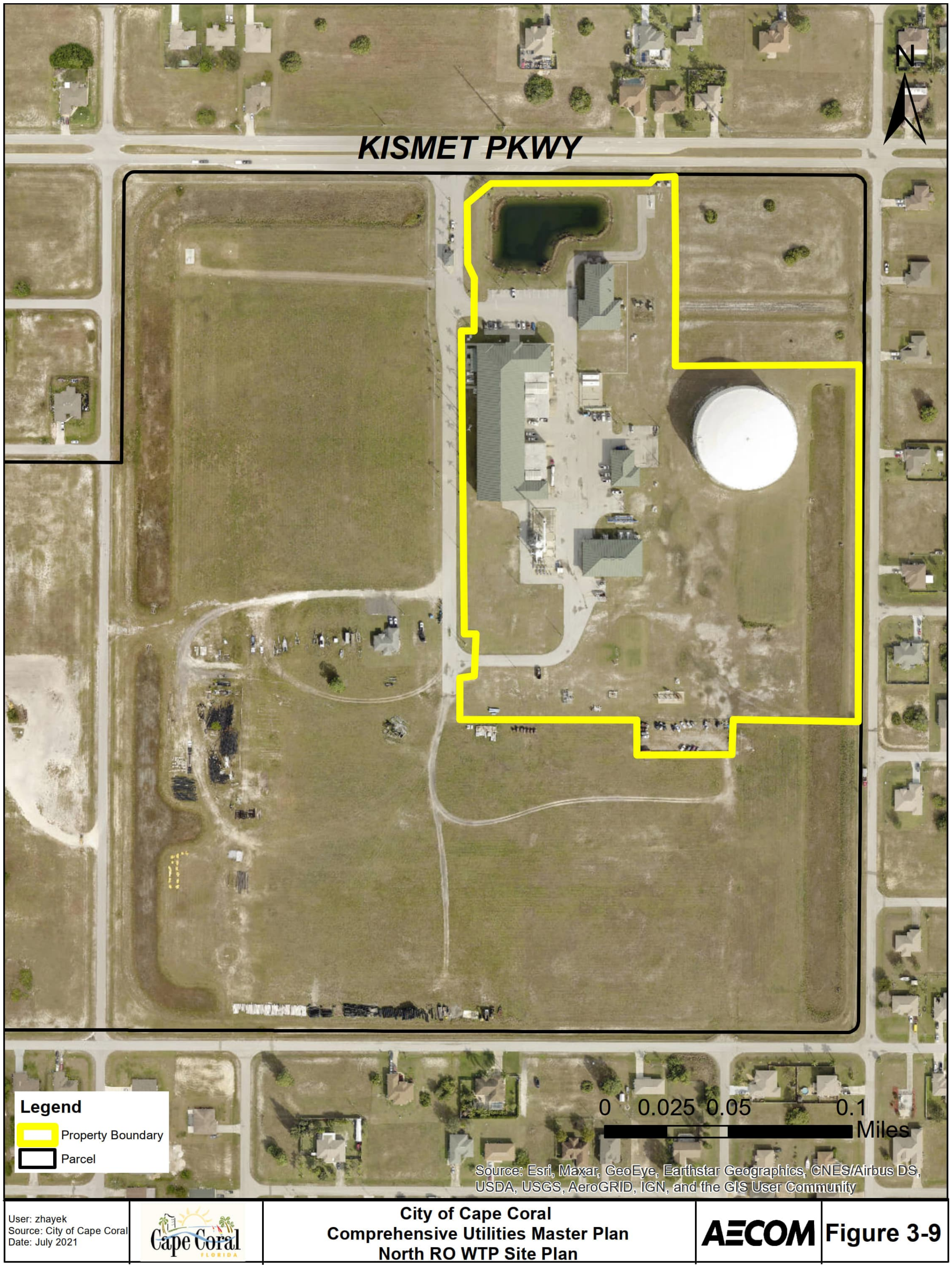


Figure 3-9: North RO WTP Site Plan



Post treatment follows the primary RO treatment. Blended product water is post treated for the removal of gases such as hydrogen sulfide and carbon dioxide using stripping towers. After the degasification system, the blended product water enters a 170,000-gallon clearwell where sodium hydroxide is added to attain the desired pH. The product water is then disinfected using sodium hypochlorite and pumped to the 12.0 MG prestressed concrete ground storage tank by means of three (two duty, one standby) 100 HP transfer pumps. The chlorine levels are continuously recorded and monitored by remote wet panel sensors. Grab samples are collected by operators daily to ensure the accuracy of the chlorine sensors. Water from the ground storage tank on-site is pumped into the City's potable water distribution system using four pumps located at the High Service Pump Station. This pump station has a total capacity of 22,500 GPM (32.4 MGD) and a firm capacity of 15,000 (21.6 MGD).

Power to the North RO WTP is supplied by Lee County Electric Cooperative (LCEC). In addition, there are two 2,250 KW diesel generator units located at the facility, each having sufficient capacity to power 100 percent of the process equipment at its current constructed capacity. The North RO WTP participates in a load shed program with LCEC during peak power demand times on their system, which results in recurring cost savings for the facility.

There are also portable diesel generators that provide standby power at the production well sites. Therefore, this facility meets FAC requirements for auxiliary power noted in Chapter 62-555.

The injection well system at the North RO WTP facility is used for disposal of non-hazardous RO concentrate. The injection well system consists of two Class 1 injection wells (a proposed well IW-1 and an existing well IW-2) and a dual-zone monitor well (DZMW-1) which monitors for upward migration from the injection zone. IW-2 was constructed in 2007 to FDEP's Class I injection well standards. The maximum permitted daily disposal volume for the North RO facility is 7.4 MGD. The North RO WTP has a current capacity of 12 MGD and a maximum anticipated production rate of RO concentrate of 3.0 MGD at an 80% recovery efficiency. Therefore, the current capacity of the injection well is considered to be adequate to handle anticipated disposal needs. Also, the Southwest RO WTP is able to supply the City with water during periods of regulatory agency mandated testing or other maintenance activity which would take the North RO WTP offline. The City has plans to construct a second injection well with the construction of the proposed North WRF. However, while the construction of IW-2 has good disposal capacity, the amount of overlying confinement was notably less than typically seen in other injection wells in the area. A new well for backup disposal of wastewater effluent will likely be built off of the current RO WTP in an area with better confinement potential.

The North RO WTP received an Outstanding Membrane Plant Award at the Southeast Desalting Association (SEDA) in 2016 in recognition of the outstanding plant operations, maintenance, and exemplary membrane treatment plant performance.

The majority of the North RO WTP infrastructure is constructed of reinforced concrete to protect against storms and to meet current building codes. The WTP's Operations/Control Center, High Service Pump Building, Maintenance Building, Generator and Electrical Buildings have sufficient area for future equipment expansion needed to meet buildout conditions. With a total floor area of 24,000 square feet, the existing Process Building is capable of meeting the facility's needs through expansion to between 24.0 and 30.0 MGD, and, if needed, vacant area on site has been reserved for future expansion of the building to meet the 36.0 MGD build-out capacity. Therefore, the system treatment process will be able to stay on-line with minimal disruption during future upgrades.

3.2.2 Southwest RO WTP

This facility is an 18.1 MGD facility, which consists of two plants. The facility has a firm capacity of 16.0 MGD with one RO skid unit out of service at each plant. The facility was constructed in 1977 as the first of its kind in the United States and was originally a 3.0 MGD facility. In 1980 and 1985, its treatment capacity was expanded from 3.0 to 6.0 MGD and then to 15.0 MGD with the addition of Plant 2. The facility was then modified in 2007 to expand treatment capacity from 15.0 MGD to 18.1 MGD. The City has no current plans for capacity expansion of this facility beyond 18.1 MGD. A treatment process block diagram for the Southwest RO WTP is presented in **Figure 3-10** and a site plan for the facility is presented in **Figure 3-11**.

The pre-treatment process for both plants at this facility consists of filtration and chemical adjustment. Sulfuric acid is added to reduce the pH of the water and polyacrylic acid is added as a scale inhibitor. Particulates in the raw water supply are physically removed through 5-micron cartridge filters.

Figure 3-10: Southwest RO Plant Process Flow Diagram

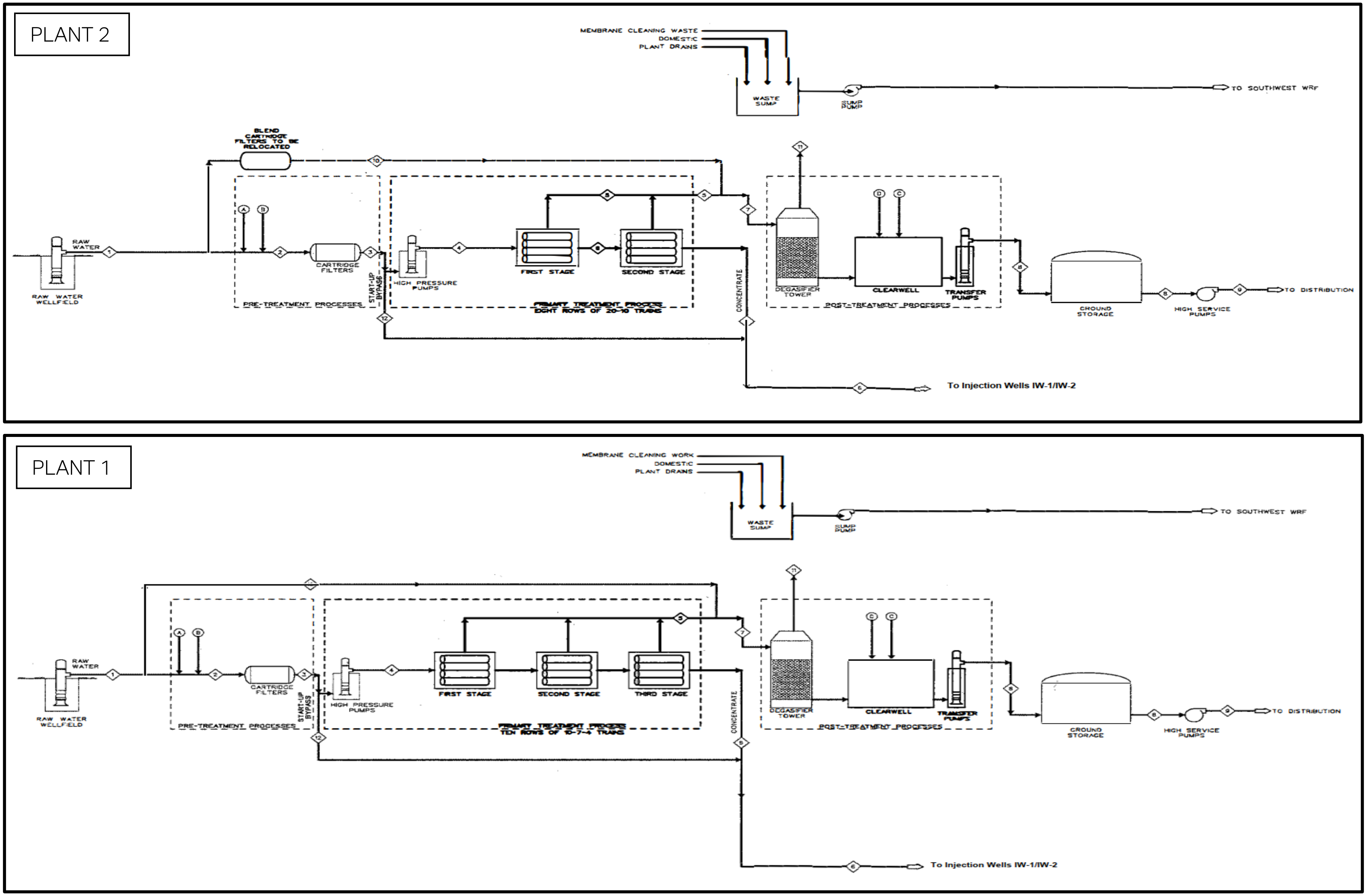
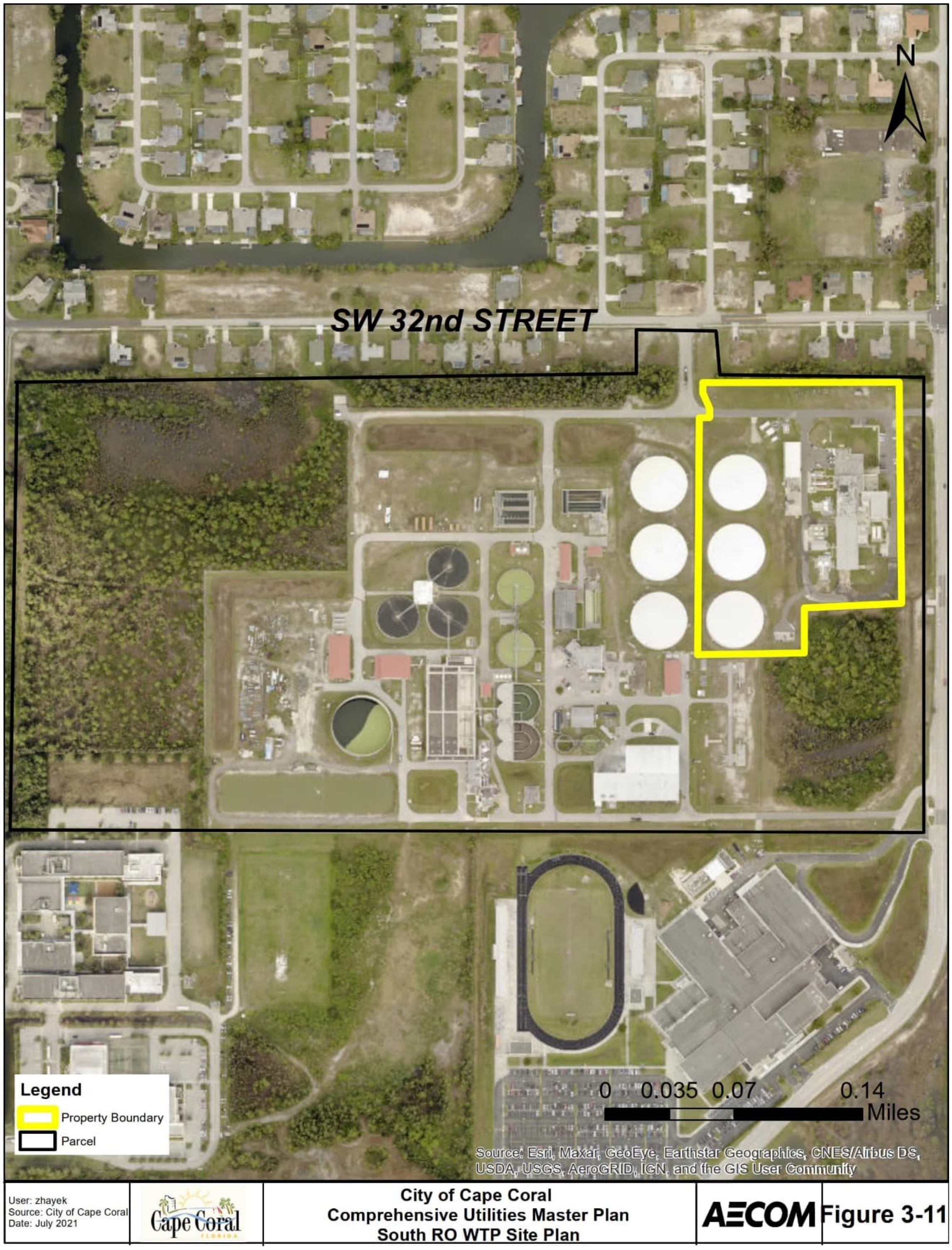


Figure 3-11: Southwest RO WTP Site Plan



Pre-treated raw water is conveyed to the RO process where finished water is produced. Plant No. 1 is a three-stage RO process with 10 trains and Plant No. 2 is a two stage RO process with 8 trains. The rejected concentrate is re-processed to increase the yield of finished water and reduce the final quantity of concentrate. Each treatment train at Plant No. 1 consists of 10 first stage pressure vessels, 7 second stage pressure vessels, and 4 third stage pressure vessels. Plant No.

2 consists of 20 first stage pressure vessels and 10 second stage pressure vessels. The product water from both plants is blended with raw water to add back minerals to meet target water quality parameters, improve the taste of the drinking water, and increase overall recovery efficiency. Concentrated water is disposed of in accordance with FDEP regulations and the blended water is conveyed to the next step of treatment.

Blended product water is post-treated for the removal of gases such as hydrogen sulfide and carbon dioxide using stripping towers. The blended water then enters a clearwell (41,000 gallons at Plant No. 1 and 117,500 gallons at Plant No. 2) where sodium hydroxide is then added to attain desired pH. The product is then disinfected using sodium hypochlorite and pumped via three transfer pumps at each plant to three 5-MG ground storage tanks. The chlorine levels are continuously recorded and monitored by remote wet panel sensors. Grab samples are collected by operators daily to ensure the accuracy of the chlorine sensors. Water is pumped from the ground storage tanks into the City's potable water distribution system using the High Service Pump Station. Pressure and flow are monitored and maintained by on-site equipment and with operator input.

The injection well system for the Southwest RO WTP is permitted as dual-purpose to serve not only the Southwest RO WTP but also the adjacent Southwest WRF for disposal of non-hazardous RO concentrate and treated municipal effluent. The injection well system consists of two Class 1 injection wells (IW-1 and IW-2) and a dual-zone monitor well (DZMW-1). IW-1 construction was completed in 2008 and IW-2 construction was completed in 2018. The maximum permitted daily disposal volume for IW-1 is 9.7 MGD and IW-2 is 10.2 MGD. The Southwest RO WTP has a current rated capacity of 18 MGD and a maximum anticipated disposal need for RO concentrate of 4.5 MGD assuming 80% treatment efficiency. The Southwest WRF currently has a capacity of 15 MGD. Except during extreme wet periods, all of the reclaimed water is used for irrigation supply. Therefore, since use of the injection well system for backup wastewater effluent disposal is extremely rare, the current capacity of the injection well system (about 20 MGD) is considered to be adequate to handle anticipated disposal needs of the Southwest RO WTP and Southwest WRF.

Power to the Southwest RO WTP is provided by LCEC. Also, two 2,250 KW diesel generators located at the facility, provide standby backup power, each having sufficient capacity to power 100 percent of the process equipment at its current constructed capacity. Also, portable diesel generators provide standby power at the production well sites. This facility therefore meets FDEP and FAC requirements for auxiliary power per Chapter 62-555.

3.3 Water Storage and Pumping Facilities

The City owns and operates two water storage and pumping facilities to provide sufficient fire flow capacity and distribution system pressures:

- Van Loon Storage and Booster Pump Station
- Palm Tree Storage and Booster Pump Station

Table 3-5 presents details of the City's storage and booster pump stations.

Table 3-5: Storage and Pumping Facilities

Storage & Pumping Station	Palm Tree	Van Loon
Ground Storage Tanks		
Number of Tanks	1	1
Capacity of Tanks	2.0 MG	1.0 MG
High Service Pumps		
Number of Pumps	4	2
Capacity of Pumps	1,500 GPM	1,000 GPM
Total Capacity	6,000 GPM	2,000 GPM

The storage and pumping facilities currently provide sufficient distribution storage required per the FDEP regulations. The Palm Tree Storage and Booster Pump Station, which originally functioned as a lime softening plant in 2000, currently utilizes a single line for both suction and discharge for the on-site storage tank. The City has plans to construct a new 16-in. suction main to the Palm Tree Storage Tank which will improve fill times and water age in the distribution system. The site of the Palm Tree facilities also has adequate space for an additional 2 MGD storage tank that is planned for the future.

3.4 Water Transmission and Distribution Network

The City's water transmission and distribution system network distributes finished water from the water treatment plants to the customers of the City. The network consists of the two storage and booster stations, 944 miles of water transmission and distribution mains as of the July 2022 inventory, 4, 288 fire hydrants, and 68,575 water meters.

3.5 Potable Water Regulatory Compliance

Water treatment is regulated federally by the United States Environmental Protection Agency (USEPA). All drinking water regulations fall under the Safe Drinking Water Act (SDWA). The Florida Department of Environmental Protection (FDEP) is responsible for enforcing the requirements of the SDWA at the state level.

3.5.1 Current Regulations

Drinking water standards are divided into two main groups: primary and secondary drinking water standards. Primary standards are health-based standards and include the four following categories: microbiological, disinfectants and disinfection byproducts (DBPs), inorganic compounds, and organic compounds. Secondary standards are primarily aesthetic in nature (color, taste, odor) and are not health-based standards. All drinking water regulations fall within the Safe Drinking Water Act (SDWA). The SDWA was passed by Congress in 1974 and applies to all public water systems. FDEP has primacy (i.e., primary responsibility to enforce) for the SDWA and drinking water regulations in the State of Florida. Florida drinking water regulations are contained in Chapters 62-550 F.A.C., 62-555 F.A.C. and 62-560 F.A.C. A summary of current drinking water regulations applicable to the City is provided in **Appendix A**.

3.5.2 New and Proposed Regulations

There are several recent drinking water regulatory developments at the federal level, most notably revisions to the Lead and Copper Rule and forthcoming regulations for poly-and perfluoroalkyl substances (PFAS). In addition, several additional updates to the Florida Administrative Code (F.A.C.) have been proposed regarding potable water and its treatment and distribution. Much of the proposed changes to the F.A.C. involve increased monitoring for contaminants. Proposed amendments also include efforts to prevent cross contamination of public water systems. Proximity of water mains to sanitary and stormwater sewer systems are specified to be of a certain distance of separation for any new or updated water mains. More details regarding the changes in federal regulations and the F.A.C. are provided in **Appendix A**.

3.5.3 Lead and Copper Rule Revision

The Lead and Copper Rule (LCR) was designed to protect the public health by minimizing lead (Pb) and copper (Cu) levels in drinking water. Lead and copper are seldom present in appreciable concentrations in source water. Rather, the LCR addresses lead and copper by reducing corrosivity of the finished water. Unstable water can be corrosive to iron, lead, copper, and alloys that contain lead and copper as are often found in household plumbing or service lines. Lead service lines were an obvious source of lead corrosion that the LCR sought to address. When the LCR was first established in 1991, all community water systems were required to comply.

In January of 2021, the United States Environmental Protection Agency (USEPA) published the Lead and Copper Rule Revisions (LCRR) in the Federal Register. In June 2021, USEPA affirmed the rule requirements and revised the effective date of the rule to December 16, 2021, and the compliance date to October 16, 2024.

While the copper requirements under the rule remain essentially unchanged, this is not the case for lead. Though USEPA opted not to lower the lead action level (AL) from its current value of 15 ug/L, the revisions establish a new lead trigger level (TL) of 10 ug/L. Compliance and associated actions by a water system are based on the 90th percentile of lead monitoring results in comparison to the AL and TL.

The City's Annual Consumer Reports on the Quality of Tap Water from 2017 to 2020 show both lead and copper are well in compliance. Ninetieth percentiles for lead range from 0.0011 to 2.5 ug/L and for copper range from 0.025 to 0.047 ppm. To ensure that future compliance is maintained, it is recommended to not make appreciable or sudden changes to finished water chemistry, particularly with respect to pH and alkalinity. Because of the lower alkalinity content in the finished water, pH adjustment plays an important role in maintaining the City's ability to provide corrosion control.

In addition to the TL and AL changes, the LCRR includes substantive changes to streamline the rule requirements and improve public health protection while ensuring effective implementation, including development of service line inventories and lead service line (LSL) replacement plans, promoting corrosion control optimization, strengthening of tap sampling requirements, increasing transparency and sampling of schools and childcare facilities, and public education requirements. A summary of the LCRR is provided in **Appendix A.**

In anticipation of the LCRR compliance date of October 16, 2024, it is recommended the City proceed with implementing the new LCRR requirements by:

- Begin preparing the service line inventory to document materials of construction on the public and private side of the meter and document presence/absence of LSLs.
- Review compliance sampling pool and identify new monitoring locations to focus on areas with the high lead risk and vulnerable members of the community.
- Evaluate historical compliance data including mapping to identify worst-case conditions within the City's service areas and assess risk of exceeding the new TL of 10 ug/L.
- Prepare for monitoring changes with LCRR standard sampling occurring twice per year starting on January 1, 2025.
- Prepare for "Find and Fix" requirements for customer notification, follow-up sampling and corrective measures for individual sample locations exceeding the 15 ug/L lead Action Level.
- Conduct a desktop evaluation to identify gaps and conduct planning for LCRR implementation.

- Initiate planning for outreach and sampling with the local school district(s) and childcare facilities and determine past monitoring activity and results, begin to coordinate a strategy as to how to collaborate with customers and community stakeholders.

3.5.4 Poly-and Perfluoroalkyl Substances

Poly-and Perfluoroalkyl Substances (PFAS) are a group of synthetic chemicals that have been in use since the 1940s. PFAS are used in a variety of products and processes, including aqueous firefighting foams (AFFF), electroplating mist suppressants, semiconductor manufacturing, aerospace and electronics applications, materials commonly used in laboratories and environmental sampling, ScotchGard™, and Teflon™. Because of their widespread use, they are frequently detected in blood samples from the general public and in the environment.

In October 2021, USEPA issued its PFAS Strategic Roadmap: EPA's Commitments to Action 2021-2024, which includes the following actions which could impact the City:

1. Undertake nationwide monitoring for PFAS in drinking water. The Unregulated Contaminant Monitoring Rule 5 (UCMR5) will provide occurrence data for 29 PFAS in the nation's drinking water systems including the four compounds listed in **Table 3-6**. Going forward, USEPA has indicated it will continue to prioritize additional PFAS for inclusion in the Unregulated Contaminant Monitoring Rule 6 (UCMR6) and beyond, as techniques to measure these additional substances in drinking water are developed and validated.
2. Establish a national primary drinking water regulation (NPDWR) for perfluorooctanoic acid (PFOA) and perfluorooctane sulfonate (PFOS). A proposed rule is expected in the Fall of 2022 and a final rule by the end of 2023. In March 2021, USEPA published the Fourth Regulatory Determinations, including a final determination to regulate PFOA and PFOS in drinking water. The agency is now developing a proposed NPDWR for these chemicals.
3. Publish the final toxicity assessments for hexafluoropropylene oxide dimer acid and its ammonium salt, known as GenX chemicals, and five additional PFAS by the end of 2021. USEPA can use this information to develop health advisories levels for these PFAS compounds. In addition, USEPA is currently developing toxicity assessments for five other PFAS: perfluorobutanoic acid (PFBA), perfluorohexanoic acid (PFHxA), perfluorohexane sulfonate (PFHxS), perfluorononanoic acid (PFNA), and perfluorodecanoic acid (PFDA). (Toxicity assessments for GenX and perfluorobutane sulfonic acid (PFBS) were issued in 2021.)
4. Publish health advisories for GenX and PFBS by the Spring of 2022. Similar to what happened with PFOA and PFOS, this will certainly start to elicit concern in communities where GenX and PFBS is detected in their water supply. USEPA has indicated they will continue to develop health advisories as the agency completes toxicity assessments for additional PFAS.

Although no regulatory standards or maximum contaminant levels (MCLs) for PFAS currently exist, the USEPA had initially established a lifetime Health Advisory Level (HAL) for drinking water of 70 parts per trillion (ppt or ng/L) for the combined concentration of PFOS and PFOA. On June 15, 2022, the USEPA re-issued interim updated lifetime drinking water health advisories for PFOS and PFOA and also established final lifetime drinking water health advisories for GenX and PFBS. The revised health advisories, summarized in **Table 3-6**, are much lower than those previously communicated by USEPA. The USEPA is now recommending further treatment if the chemicals listed in **Table 3-6** below are detected with the new minimum reporting levels.

Table 3-6: Summary of USEPA Health Advisories as Revised in June 2022

Chemical	Lifetime Health Advisory Level/Value (ppt)	Minimum Reporting Level (ppt)
PFOA	0.004 (interim)	4
PFOS	0.02 (interim)	4
GenX Chemicals	10 (final)	5
PFBS	2,000 (final)	3

Note that the health advisory levels for PFOA and PFOS are up to 1000x lower than the minimum reporting level (MRL). This essentially means that any detection of PFOA or PFOS could require treatment.

The City is required to conduct its UCMR5 monitoring between 2023 and 2025. Two rounds of sampling must be done during a consecutive 12-month period and the sampling must be between 5 and 7 months apart. Sampling is required at each of the City's entry points to the distribution system (EPTDS).

It is worth noting that the City conducted monitoring for six PFAS compounds under the Third Unregulated Contaminant Monitoring Rule (UCMR3) in 2014. None were present above the detection limit at that time. However, the City should begin developing a response strategy in the event any of the four PFAS identified above are detected above the new health advisory levels.

3.5.5 Senate Bill 64

Senate Bill 64 (SB 64) was approved by the Governor of Florida in June of 2021. The bill focuses on eliminating effluent, reclaimed water, and reuse water discharges to surface waters by domestic wastewater utilities and regulates authorized discharges that are beneficially used. Beneficial use or regulated authorized discharges include discharges associated with indirect potable reuse projects. With regards to potable water, the bill not only incentivizes the potable reuse projects, but also specifies that potable reuse is an alternative water supply, making these projects eligible for alternative water supply funding.

3.5.6 Potable Reuse

Indirect potable reuse (IPR) is the planned discharge of reclaimed water or advanced treated water to a ground or surface water to supplement potable water supply. Direct potable reuse (DPR) is the introduction of advanced treated water directly into a drinking water system (either through an existing water treatment plant or directly to the water distribution system). Advanced treated water is reclaimed water that has been treated to a higher level of treatment than is typically required for ground or surface waters due to the increased pathogenic risk and presence of constituents of emerging concern (CECs) in reclaimed water.

FDEP is in the process of revising and developing new regulations for potable reuse which will impact future treatment requirements for both IPR and DPR. Revisions to 62-550 F.A.C. (Drinking Water Standards, Monitoring and Reporting) and 62-555 F.A.C. (Permitting, Construction, Operation, and Maintenance of Public Water Systems) may not be finalized for several more years. The resulting uncertainty in potable reuse treatment requirements poses a significant challenge to water utilities considering potable reuse.

Though final revisions to 62-550 F.A.C. may not happen for several years, FDEP has made some indication regarding future "advanced treated water" requirements. In May 2021, FDEP issued a notice of proposed revisions to 62-550 F.A.C. which included a preliminary definition of advanced treatment and established a benchmark for "advanced treated water." In the draft rule, advanced treatment includes membrane filtration, reverse osmosis, and advanced oxidation – an "RO-based" treatment process for the purposes of the discussion in this chapter. The draft rule language allowed for testing of alternate treatment processes provided it could be demonstrated that the alternate process provided equivalent public health protection to the RO-based process.

There are two main treatment concerns with potable reuse of reclaimed water: increased pathogenic risk and CECs. The May 2021 draft of 62-550 F.A.C. indicated preliminary treatment technique requirements for potable reuse. A summary of preliminary disinfection requirements is provided in **Table 3-7**. Note there are two levels of disinfection indicated. "From headworks" indicates the total required disinfection of the combined wastewater, advanced treatment and drinking water treatment facilities. "From reclaimed water" indicates the required disinfection of the combined advanced treatment and drinking water treatment processes. Preliminary rule language indicates water must meet reclaimed water standards to be used for potable reuse, thus, unless a wastewater treatment facility includes a process that provides a significant additional disinfection barrier, e.g., membrane bioreactors, most utilities will likely provide additional disinfection "from reclaimed water." If a utility desires, site specific disinfection criteria can also be established by conducted microbiological monitoring of one's reclaimed water supply.

Table 3-7: Preliminary FDEP Potable Reuse Disinfection Requirements

Parameter	Required Log Removal/Inactivation	
	From Headworks	From Reclaimed Water
Giardia	10	6
Cryptosporidium	10	5.5
Virus	12	8

In addition to enhanced disinfection requirements, the May 2022 draft of the proposed revisions to 62-610 F.A.C. includes the following:

- Reuse projects shall be designed and operated to meet the primary and secondary drinking water standards…prior to reclaimed water or advanced treated water discharging to ground or surface waters or distribution system. (Draft language does seem to indicate that compliance with secondary parameters is not required for injection into an aquifer greater than 3,000 TDS.)
- Potable reuse projects regulated by Part V of this chapter shall be designed and operated to meet the pathogen reduction requirements established in Rule 62-550 (**Table 3-7**). A separate treatment process may be credited with no more than 6-log reduction, with at least two processes each being credited with no less than 1.0-log reduction. A single treatment process may receive log reduction credits for one or more pathogens.
- Potable reuse projects shall include a multi-barrier framework composed of source control and appropriate treatment technology that incorporates resiliency (i.e., ability to adjust to upsets), redundancy, and robustness (i.e., features that simultaneously address multiple constituents) for control of pollutants, which includes emerging constituents and pathogens.

The key takeaway from this language for DPR is the level of required treatment. Though the disinfection requirements may change, much higher levels of disinfection will be required. It is also known that a multi-barrier process will be required, and that process must address CECs, as well as pathogens.

For IPR, the future regulatory requirements are a bit more unclear. Based on the draft language, FDEP could require advanced treatment for disinfection and CECs prior to discharge to the aquifer, in addition to compliance with primary standards as is currently required. That appears to be the direction the rules are headed. This needs to be considered in the planning and conceptual design of any future IPR project.

3.5.7 Microbial Protection and Disinfection Byproducts

USEPA conducts a review of every Primary Drinking Water Regulations (PDWR) at least once every six years. The most recent review was conducted in 2016. Though several potential issues were raised (volatile organic compounds, fluoride, acrylamide and epichlorohydrin), the primary areas of focus (at least until the next review, which could be in 2022) are microbial protection and disinfection byproducts.

With regard to microbial protection, there are two areas of interest to the City: 1) requirements to maintain a minimum disinfectant residual in the distribution system, and 2) CT criteria for virus disinfection.

Maintaining a disinfectant residual above a set numerical value in the distribution system may improve public health protection from a variety of pathogens. Such a change could have benefits for controlling occurrence of all types of pathogens in distribution systems, except for those most resistant to disinfection, such as Cryptosporidium. While no specific residual level has been proposed, the impact of different minimum distribution system residuals on disinfection byproducts (DBP) levels and total organic carbon (TOC) limits in the finished water will be evaluated during bench testing.

USEPA is also considering whether the current CT (disinfectant residual x contact time) criteria based on hepatitis A virus (HAV) are sufficiently protective against other types of viruses. This could impact the City's future disinfection requirements (i.e., require a higher chlorine dose or longer contact time) and ultimately DBP levels in the City's water distribution system.

With regard to DBPs, there is discussion as to whether the existing disinfection byproduct MCLs are sufficient to protect health. Specific DBPs noted by USEPA for potential revision include total trihalomethanes (TTHM), haloacetic acids (HAA5), brominated DBP species, chlorate, chlorite, and nitrosamines (specifically N-nitrosodimethylamine [NDMA]). NDMA is not a concern for the City because they do not use chloramination for secondary disinfection.

3.6 Potable Water Historical and Projected Demands

3.6.1 Historical Potable Water Production

Potable water supplied by the City is produced at the North RO WTP and the SW RO WTP. Treated water is transferred to ground storage tanks located at each WTP and at two different remote locations in the City and then pumped into the distribution system. Potable water production data for the years from 2010 to 2019 was provided by the City.

Table 3-8 shows the combined system production from both WTPs. The production data is summarized by Annual Average Daily Demand (AADD), Maximum Month Daily Demand (MMDD), Maximum 3-Day Demand (M3DD), and Maximum Daily Demand (MDD); all expressed in million gallons per day (MGD), where:

- AADD is the average quantity of water supplied over a one-year period
- MMDD is the highest of monthly averages over a one-year period
- M3DD is the maximum quantity of water supplied over three consecutive days over a one-year period
- MDD is the maximum quantity of water supplied in a single day over a one-year period

Table 3-8: 10-Year Historical Potable Water Production

Fiscal Year	Annual Average Daily Demand (AADD) MGD	Maximum Month Daily Demand (MMDD) MGD	Maximum 3-Day Demand (M3DD) MGD	Maximum Daily Demand (MDD) MGD
2010	9.02	10.07	10.87	12.22
2011	9.64	10.13	10.69	11.89
2012	9.49	10.33	11.03	11.94
2013	9.38	10.37	11.31	11.85
2014	9.63	10.15	10.64	11.07
2015	9.81	10.61	11.93	12.20
2016	10.05	10.92	11.38	11.58
2017	10.88	12.09	13.12	13.31
2018	11.10	12.68	13.24	13.43
2019	11.22	11.98	12.58	12.86
10-Year Average	10.02	10.93	11.68	13.43

Over the past 10 years the AADD has generally increased as the served population grew with an increase in demand of 24.4%. The MDD produced over the same period has been generally trending upward. It is expected that the AADD will continue increasing as new service connections are added throughout the existing and future service areas.

3.6.2 Nonrevenue Water

The difference between water produced at the plant and water billed is called nonrevenue water, which can be attributed to pipe losses, water usage for pipe and hydrant flushing, and other types of unbilled water use. The amount of nonrevenue water in a system is a good indicator of the overall system condition; that is, the lower the amount of nonrevenue water, the "better" the system. The Florida Administrative Code [25-30.4325 (1)(e)] defines excess to be above 10% of the total water produced. Records of daily plant production as well as water billed data for the last seven years were provided by the City.

Table 3-9 shows the annual total number of gallons produced and billed per year, and the annual nonrevenue producing water expressed both in million gallons per day and as a percentage of the water produced.

Table 3-9: Nonrevenue Potable Water Summary

Fiscal Year	Average Daily Production (MGD)	Average Daily Billed (MGD)	Average Daily Accounted for Water for Flushing/Leaking (MGD)	Average Nonrevenue (%)
2013	9.38	7.99	0.48	10.00%
2014	9.63	8.14	0.51	10.20%
2015	9.81	8.31	0.67	10.10%
2016	10.05	8.80	0.63	6.40%
2017	10.88	9.54	2.68	5.00%
2018	11.10	9.63	1.98	5.90%
2019	11.22	9.63	1.09	7.70%
Average	10.30	8.86	1.15	7.90%

The average nonrevenue water for the system was 7.9% for the time period of 2013-2019, with three years having nonrevenue water below 6.4%. This analysis reflects good condition of the system and proactive management of leaks and other line losses.

3.6.3 Per Capita Historical Water Demand

The historical per capita water demand is used to project the system-wide water demand based on the population in the service area. The annual average daily demand (AADD) for each year over the 10-year period was divided by the served population to determine an average per capita demand, as shown in **Table 3-10**.

Table 3-10: Historical Per Capita Water Usage

Fiscal Year	Served Population	Annual Average Daily Demand (AADD)	
		Potable Water Demand (MGD)	Per Capita Demand (gpcd)
2010	119,349	9.02	76
2011	121,047	9.64	80
2012	122,212	9.49	78
2013	123,747	9.38	76
2014	125,005	9.63	77
2015	127,111	9.81	77
2016	133,627	10.05	75
2017	137,255	10.88	79
2018	140,857	11.10	79
2019	143,605	11.22	78
10-Year Average	129,381	10.02	77

The per capita demand for potable water has remained under 80 gallons per capita per day over the 10-year period of 2010-2019. This relatively low demand is attributed to a combination of the City's water conservation policies, an ongoing program for replacing aging pipes which reduces losses due to leaks, and most significantly the expansion of the City's IQ water distribution network as a separate water supply system. It is anticipated this low per capita demand will continue, however, 80 gpcd is recommended to be used to provide conservative future potable water demand projections. It is recommended that the City continues implementing water conservation measures to maintain demand reduction, including measures such as water conservation control device requirements for new construction, water conservation-based rate structure, and meter replacement program as per the City's 2017 Water Supply Facilities Work Plan.

3.6.4 Supplemental Irrigation Water Demand

During Fiscal Years 2012, 2013, 2015, and 2017-2019, the water produced at the water treatment plants was also used to supplement the IQ water system. The amount of water produced by the plants for supplemental irrigation is presented in **Table 3-11**. These values are not included in **Table 3-10** to calculate per capita demand since the City has several projects underway that will expand IQ water supply and potable water used for irrigation will no longer be needed. Therefore, this master plan only considers the potable water demands for consumptive usage.

Table 3-11: Historical Supplemental Irrigation Water Demand

Fiscal Year	Annual Average Daily Demand (AADD)	Maximum Daily Demand (MDD)
2010		
2011		
2012	1.17	3.49
2013	2.54	3.27
2014		
2015	3.16	5.19
2016		
2017	3.97	4.93
2018	3.01	4.35
2019	2.30	4.38
10-Year Average	2.69	4.27

Over the past ten years supplemental irrigation water was provided by the WTPs for six total years including the past three consecutive years. The annual average demand supplied is 2.7 MGD with the maximum daily flow supplied being 5.19 MGD in FY 2015.

3.6.5 Peaking Factors

Potable water demands tend to vary based on seasonal population and other factors and therefore master plans utilize peak demands for projections to ensure the potable water service will be reliable at all times. The City follows the requirements stipulated in FDEP permits and also guidelines provided by the Ten States Standards and the U.S. Environmental Protection Agency. The City's water treatment facilities permitted capacity is based upon the MDD and therefore the reliable treatment capacity must be able to meet both the AADD and the MDDs. The AADD and the MDD for the years 2010 through 2019 are shown in **Table 3-12**. The MDD peaking factor (PF) for each of the years listed was calculated by dividing the MDD by the AADD for the respective year.

Table 3-12: Maximum Daily Demand Peaking Factors

Fiscal Year	AADD (MGD)	MDD (MGD)	MDD Peaking Factor
2010	9.02	12.22	1.35
2011	9.64	11.89	1.23
2012	9.49	11.94	1.26
2013	9.38	11.85	1.26
2014	9.63	11.07	1.15
2015	9.81	12.20	1.24
2016	10.05	11.58	1.15
2017	10.88	13.31	1.22
2018	11.10	13.43	1.21
2019	11.22	12.86	1.15
10-Year Average	10.02	12.24	1.22
Highest PF			1.35

From the tabulated values, the MDD PF has ranged from 1.15 to 1.35 during the period of 2010-2019, with an average factor of 1.22. For determining the potable water demand projections, an MDD PF of **1.35** is utilized, which is highest PF over the last ten years. This provides a conservative estimate of MDDs to provide added reliability when evaluating potable water treatment capacity.

Table 3-13 presents the M3DD PF. As indicated from the tabulated values, the M3DD PF has ranged from 1.10 to 1.22 during the period 2010 – 2019, with an average peaking factor of 1.17. A M3DD PF of 1.20 is recommended to be used for M3DD projections. **Table 3-14** summarizes the MMDD PF. The PFs for M3DD and MMDD are calculated by dividing the M3DD and the MMDD by the AADD for the respective year.

Table 3-13: Maximum 3-Day Daily Demand Peaking Factors

Fiscal Year	AADD (MGD)	M3DD (MGD)	M3DD Peaking Factor
2010	9.02	10.87	1.21
2011	9.64	10.69	1.11
2012	9.49	11.03	1.16
2013	9.38	11.31	1.21
2014	9.63	10.64	1.10
2015	9.81	11.93	1.22
2016	10.05	11.38	1.13
2017	10.88	13.12	1.21
2018	11.10	13.24	1.19
2019	11.22	12.58	1.12
10-Year Average	10.02	11.68	1.17

Table 3-14: Maximum Month Daily Demand Peaking Factors

Fiscal Year	AADD (MGD)	MMDD (MGD)	MMDD Peaking Factor
2010	9.02	10.07	1.12
2011	9.64	10.13	1.05
2012	9.49	10.33	1.09
2013	9.38	10.37	1.11
2014	9.63	10.15	1.05
2015	9.81	10.61	1.08
2016	10.05	10.92	1.09
2017	10.88	12.09	1.11
2018	11.10	12.68	1.14
2019	11.22	11.98	1.07
10-Year Average	10.02	10.93	1.09

From the tabulated values, the MMDD PF has ranged from 1.05 to 1.14 during the period 2010 – 2019, with an average peaking factor of 1.09. The highest MMDD PF of 1.14 is used for MMDD projections.

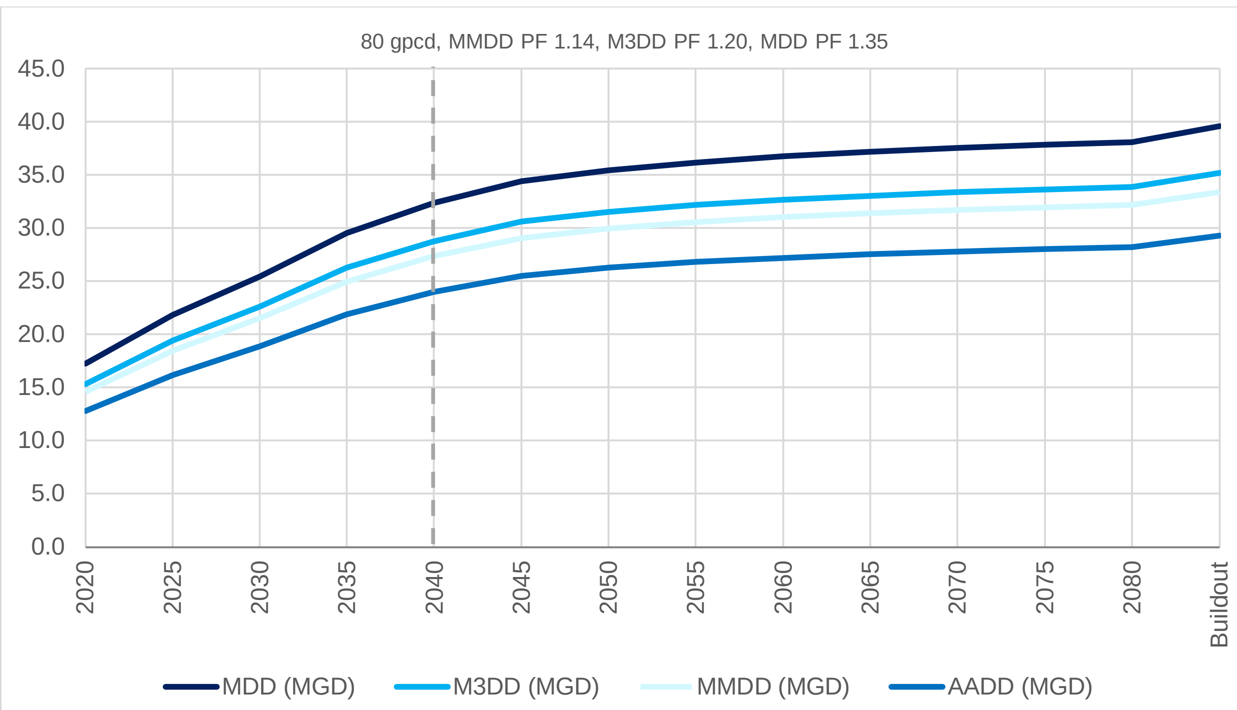
3.6.6 Potable Water Demand Projections

Potable water demand projections were prepared using the projected populations developed as part of this master plan and the per capita water demand of 80 gpcd calculated based upon the historical analysis completed. The projected MDD was estimated using the AADD projections and the system wide MDD PF of 1.35. Projections for MMDD and M3DD were estimated with PFs of 1.14 and 1.20 respectively to provide an overall understanding of higher demand periods although they are not the basis for determining reliable potable water treatment capacity or potable water storage requirements. The potable demand projections are presented in **Table 3-15** and **Figure 3-12**.

Table 3-15: Potable Water Demand Projections

Fiscal Year	Total Served Population	AADD (MGD)	MMDD (MGD)	M3DD (MGD)	MDD (MGD)
2020	159,922	12.79	14.58	15.35	17.3
2025	202,004	16.16	18.42	19.39	21.8
2030	235,530	18.84	21.48	22.61	25.4
2035	273,407	21.87	24.93	26.25	29.5
2040	299,534	23.96	27.32	28.76	32.3
2045	318,407	25.47	29.04	30.57	34.4
2050	328,065	26.25	29.92	31.49	35.4
2055	334,816	26.79	30.54	32.14	36.2
2060	339,887	27.19	31.00	32.63	36.7
2065	343,897	27.51	31.36	33.01	37.1
2070	347,276	27.78	31.67	33.34	37.5
2075	350,116	28.01	31.93	33.61	37.8
2080	352,520	28.20	32.15	33.84	38.1
Buildout	366,393	29.31	33.42	35.17	39.57

Figure 3-12: Potable Water Demand Projections



The dashed line indicates the start of forecasts provided exceeding the 20-year planning horizon (FY 2020 through FY 2040). This is meant to indicate that projections after this point are more uncertain and should be revisited for future planning. As shown by **Table 3-15** and **Figure 3-12**, the MDD is expected to increase from 17.3 MGD in FY 2020 to 32.3 MGD in FY 2040 or an increase of 86.7%. This demand is projected to increase to 39.57 MGD at buildout or an increase of about 129% from FY 2020 which should be viewed with caution due to uncertainty after the 20-year planning period.

3.7 Level of Service Standards/Performance Criteria

Level of Service Standards and Performance Criteria are established to provide a basis for determining demand generated by a planned development, the availability of facility capacity, and a basis to measure the overall performance of the utility service provided. A baseline assessment of the City's LOS Standards/Performance Criteria to be utilized when evaluating the potable water system's infrastructure needs was completed and the results are summarized below.

The City has adopted potable water service LOS Standards for per capita demand and design criteria for peaking factors. This information is documented in City of Cape Coral Design Procedure Manual and the infrastructure element of the Cape Coral Comprehensive Plan. As shown in **Table 3-16**, the potable water demand recommended by the City Design Procedures Manual and the City's Comprehensive Plan is 200 gpd per ERU. This equates to 91.2 gpd per capita based on current City demographics provided by Metro Forecasting Models LLC (2.55 persons per residence and 86% occupancy rate).

As noted in the Section 3.6.3 previously, it is recommended that 80 gpcd be utilized for determining treatment capacity. However, it was agreed that a more conservative value of 91.2 gpcd (per City design manual) should be used for North 1 when completing the hydraulic modeling analyses to evaluate needed infrastructure improvements to the potable water transmission and distribution system.

After completing a review of various reference documents/engineering reports and completing a historical analysis of demands and peaking factors, a list of LOS Standards/ Performance & Design Criteria was recommended for potable water service as summarized in **Table 3-16**. The most significant additions/changes to the performance criteria include:

- Using 80 gpcd for per capita water usage
- Determining the needed wells for potable water supply based upon the design capacity of the water treatment plants and the amount of raw water required to produce finished water **where 1 in every 4 wells is redundant.**
- Evaluating deep injection well capacity based upon maximum anticipated RO concentrate and annual average daily flows for wastewater.

Table 3-16: Potable Water Level of Service/Design Criteria for Cape Coral

Service Value	Design Criteria Statement	2020 Design Criteria/LOS	Unit of Measure	Driver	Reference Material
Capacity & Access	Annual average day per capita demand	80	gpcd	Master Planning Demand Projections – Hydraulic Modeling & Gap Analyses	Historical Data Analysis
Capacity & Access		91.2	gpcd	Master Plan Hydraulic Modeling for North 1 & Design	Cape Coral Design Procedures Manual
Capacity & Access	Treatment plant process design basis	Max Day	MGD	Design	FAC 62-555.320 (6)
Capacity & Access	Residential fire flow demand	500	gpm	Design	Cape Coral Design Procedures Manual
Capacity & Access	Commercial fire flow demand	1,000	gpm	Design	Cape Coral Design Procedures Manual
Capacity & Access	Minimum annual average, max day and peak hour demand operating pressure	35	psi	Design	Cape Coral Design Procedures Manual
Capacity & Access		90	psi	Design	Cape Coral Design Procedures Manual
Health & Safety	Water quality complies with Federal and State of Florida Safe Drinking Water Act standards	Treatment plants and distribution system meet permit requirements for water quality		Regulatory	FDEP Rule Chapter 62-555 Drinking Water Standards, Monitoring, and Reporting.
Quality & Reliability	Meets treatment demands with one of the largest units out of service	Capacity with one of the largest units out of service	-	Reliability	2012 Recommended Standards for Water Works Sec 4.0 by The Great Lakes - Upper Mississippi River Board of State and Provincial Health and Environmental Managers
Quality & Reliability	% of additional supply requirement as reliable capacity from brackish groundwater source	25	% of supply requirement	Reliability	The supply requirement is based upon the design capacity of the water treatment plants and the amount of raw water required to produce finished water where 1 in every 4 wells is redundant.
Quality & Reliability	Maximum allowable unaccounted-for water loss percentage	10	%	Reliability, Environment	Basis of Review for Water Use Permit Applications Within the SFWMD Sec 2.6.1.C Water Conservation Plans July 21st, 2013
Quality & Reliability	Reliable capacity for deep injection well	Average Annual Daily Flow	MGD	Reliability	Recommended criterion when DIW is secondary disposal method

The recommended criteria outlined above was used as a basis to evaluate the potable water system. System deficiencies over the planning horizon are identified and a prioritized list of these recommended improvements in conjunction with water quality goals, treatment requirements and available water resources form the basis for a summary of recommended UEP and utility improvements.

3.8 Potable Water System Future Needs

3.8.1 Water Treatment Plant Capacity Gap Analysis

The total capacity of the City's water treatment plants must be able to meet the projected MDD. A potable water treatment plant gap analysis compares the projected MDD to the firm treatment capacity to determine "if" and "when" additional capacity is needed. The total firm treatment capacity of the City's WTPs is 25.1 MGD considering the largest treatment process unit is out of service at each WTP (one RO skid offline at the North RO WTP and one RO skid offline at Plant 1 and Plant 2 of SW RO WTP).

A potable water treatment capacity gap analysis was completed and is shown in **Figure 3-13**. The gap analysis was based upon a per capita demand rate of 80 gpcd and a 1.35 MDD peaking factor. As shown, treatment plant capacity expansions are needed to keep up with the projected potable water MDDs. The timeline for expansions identified in the gap analysis was developed based upon using 3 MGD RO skids for future expansions. In addition, the timeline included replacement of the SW RO WTP based on the age, condition, and concern that the existing RO membranes are no longer being manufactured. **Table 3-17** summarizes the projected timeline for capacity expansion of each WTP through buildout conditions determined by the gap analysis.

Figure 3-13: Potable Water System Treatment Gap Analysis

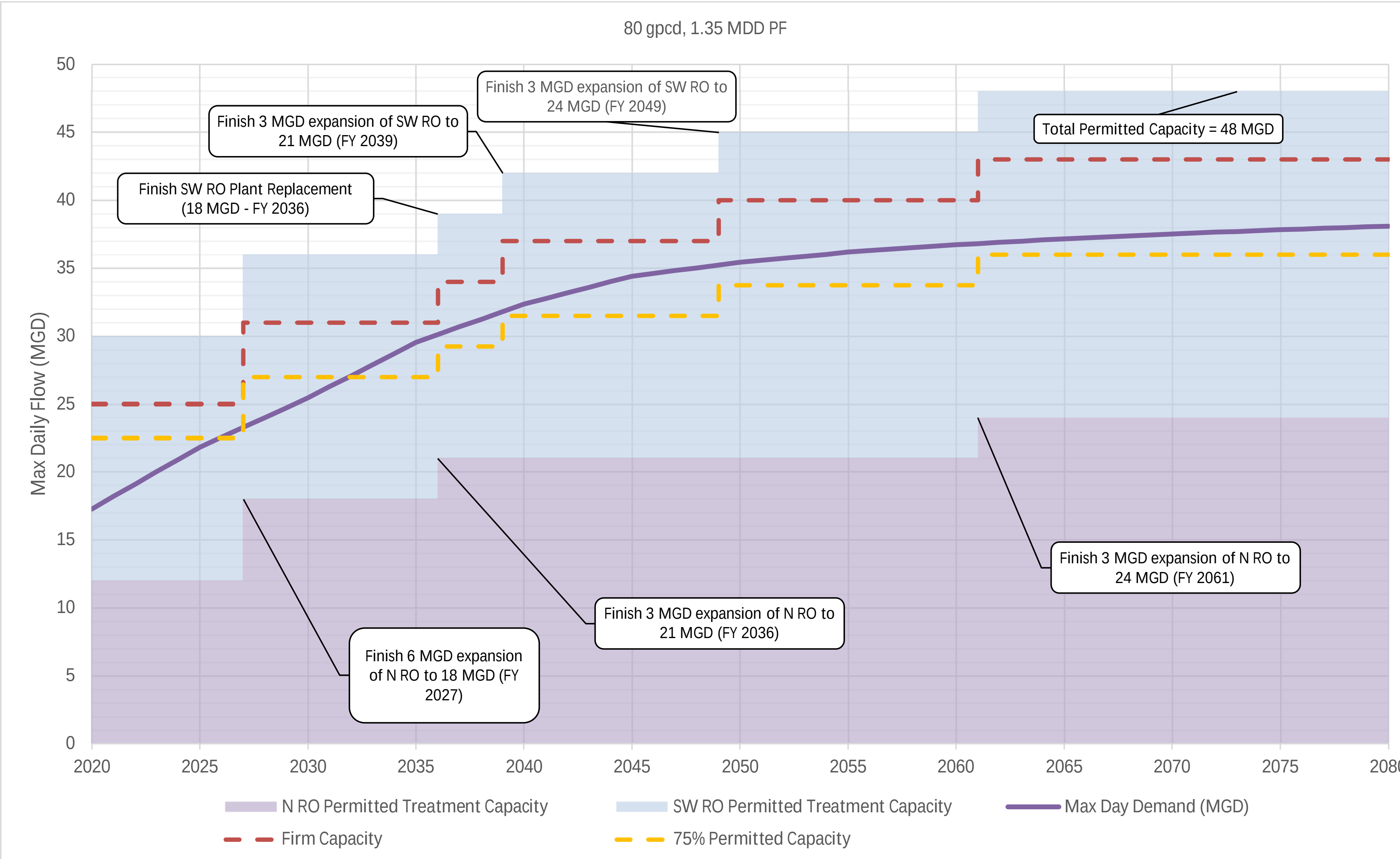


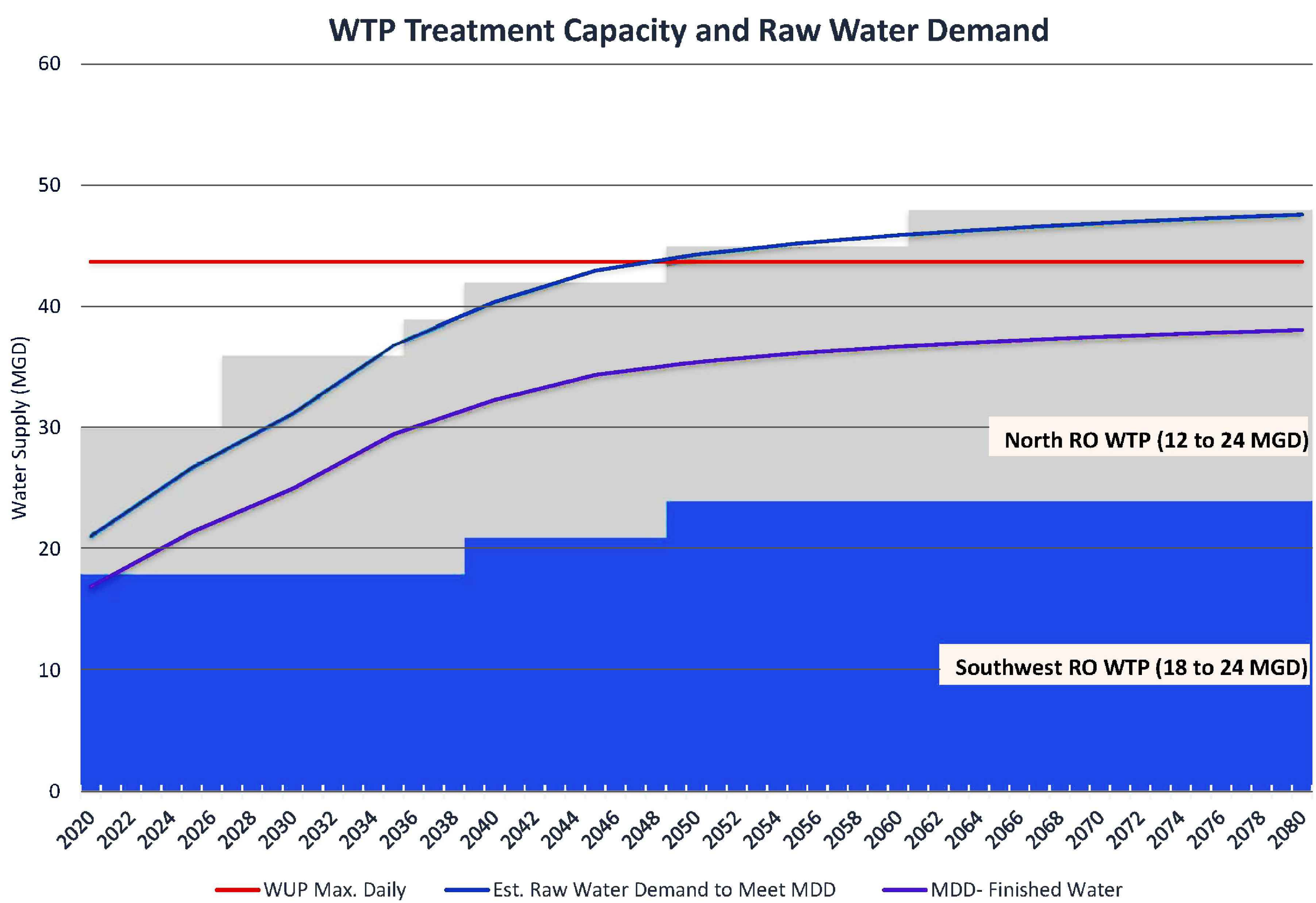
Table 3-17: WTP Treatment Capacity Timeline

RO Facility Expansion Timeline Summary	
2025	Start SW RO Plant Replacement Design
2027	Finish 6 MGD expansion of N RO to 18 MGD
2036	Finish SW RO Plant Replacement (18 MGD)
	Finish 3 MGD expansion of N RO to 21 MGD
2039	Finish 3 MGD expansion of SW RO to 21 MGD
2049	Finish 3 MGD expansion of SW RO to 24 MGD
2061	Finish 3 MGD expansion of N RO to 24 MGD

3.8.2 Brackish Supply and Infrastructure Needs

The potable water demand projections generated for the years 2021 through 2080 for the maximum daily demand (MDD) are used to estimate projected raw water production required from the wellfields for the same period with an assumed RO WTP treatment efficiency of 80%. A plot showing the current and proposed treatment capacities of the City's WTPs, projected MDD of finished water, and projected raw water demands are shown in **Figure 3-14**. The finished water MDD is projected to increase from 17.3 MGD in 2021 to 38.1 MGD in 2080. Proportionally, the total raw water demand is anticipated to increase from 21.6 MGD in 2021 to 47.6 MGD in 2080. The City's active Public Water Supply permit (No. 36-00046-W) allows the City to withdraw up to 1,312 million gallons on a maximum monthly basis which translates to roughly 43.7 MGD. The permitted annual average withdrawal is 14.3 billion gallons (39.2 MGD).

The current combined brackish treatment capacity for the City's WTPs is 30 MGD, which includes 18 MGD at the Southwest RO WTP and 12 MGD at the North RO WTP. Based upon the Potable Water Treatment Plant expansion plan shown in **Table 3-17**, the Southwest RO WTP will expand to 21 MGD capacity in 2039 and 24 MGD capacity in 2049. The North RO WTP will expand to 18 MGD capacity in 2027, 21 MGD capacity in 2036, and 24 MGD capacity in 2061. Refer to **Figures 3-15** and **3-16** for plots showing treatment capacities for each phase of planned WTP expansion for both the WTPs and brackish water needed to supply the WTPs at full capacities.



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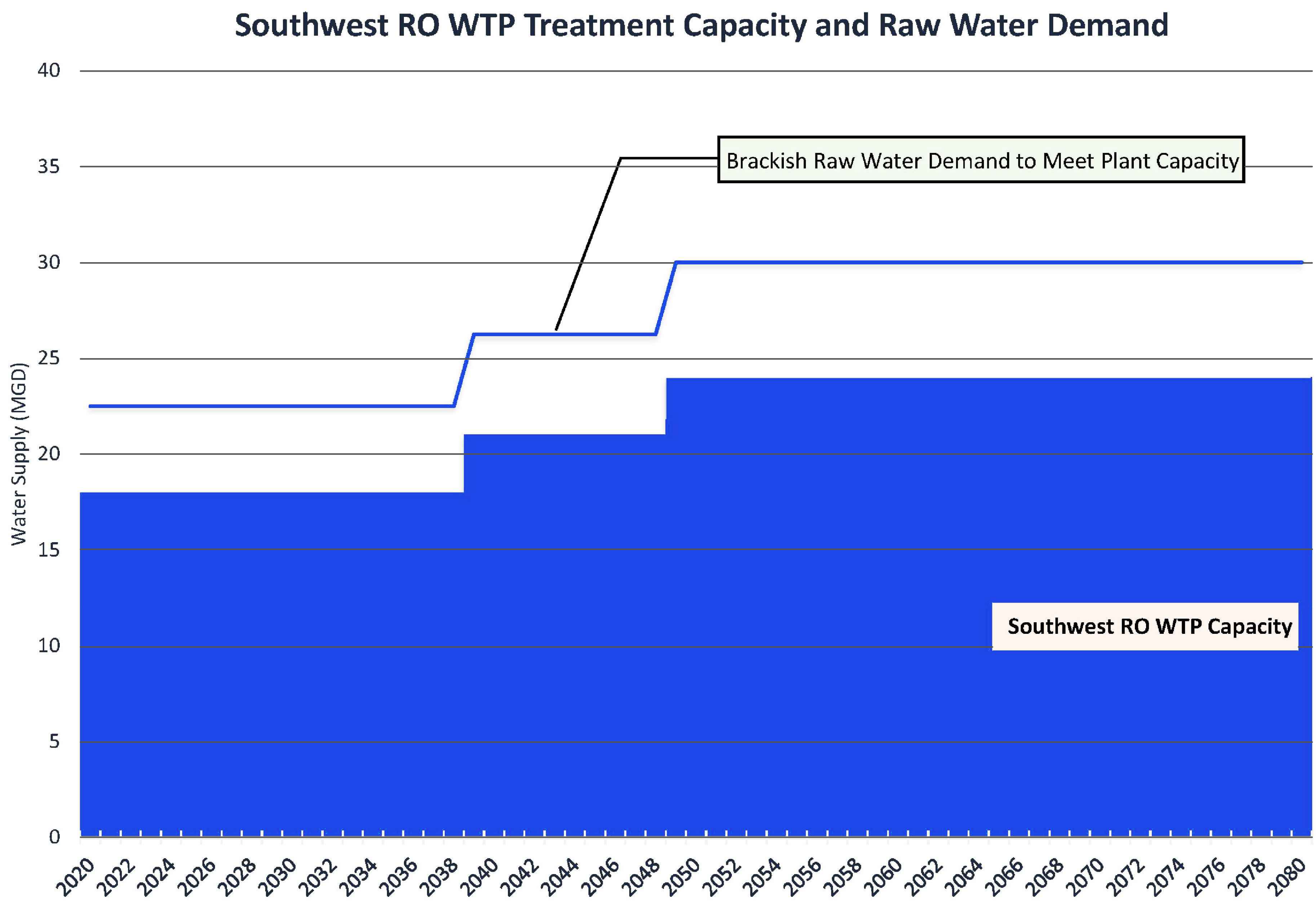
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-14: North and Southwest WTP Treatment Capacities and Raw Water Demands.



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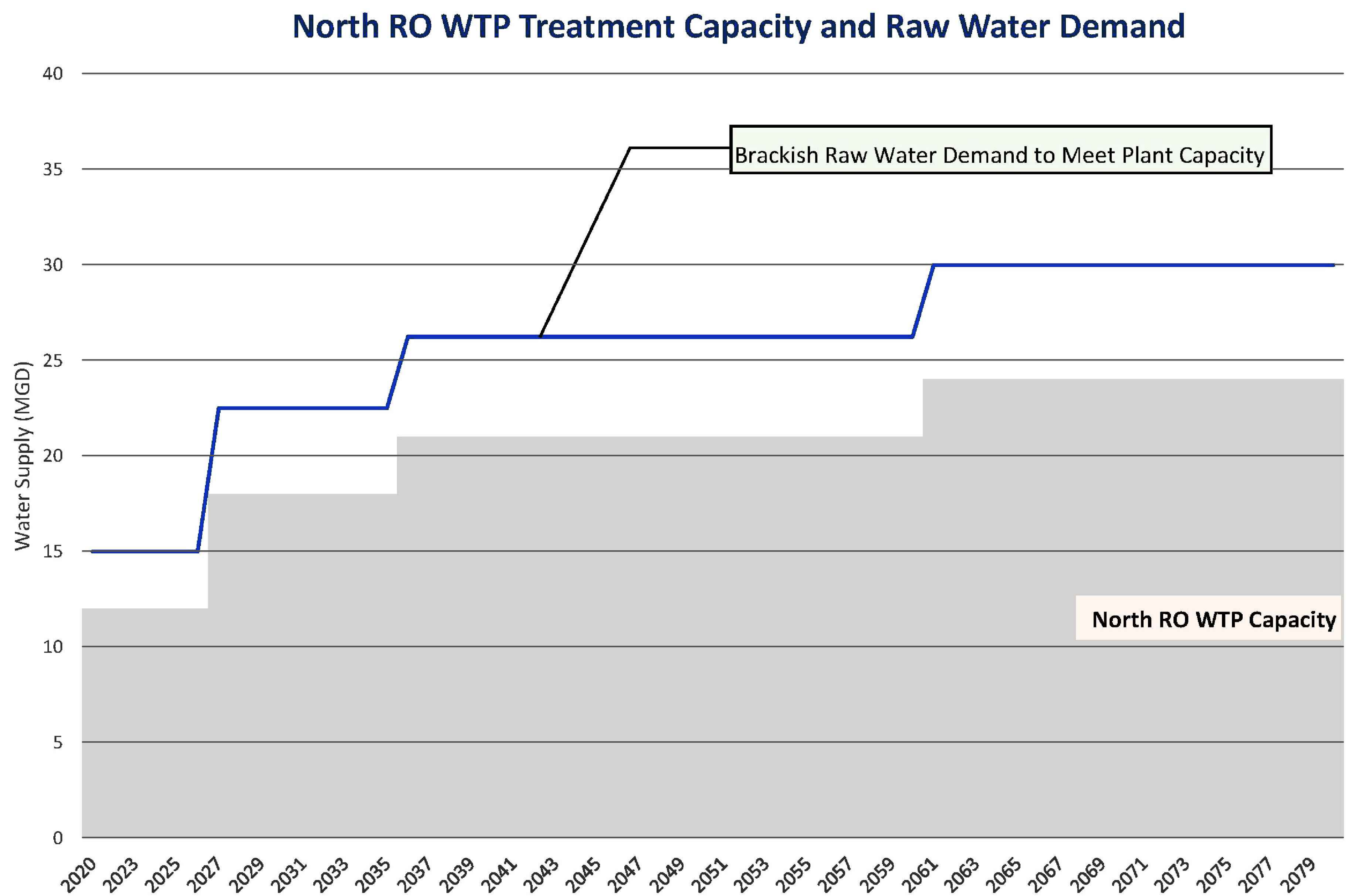
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-15: Southwest RO WTP Treatment Capacity and Raw Water Demand.



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	NOT TO SCALE
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-16: North RO WTP Treatment Capacity and Raw Water Demand.

The total raw water demand and number of wells needed to meet that demand were estimated based on the following assumptions:

- The WTP treatment efficiency for brackish raw water is 80%.
- The average operational flow rate of each brackish wells is 400 gpm.
- The brackish water wellfields will have a reliability or redundancy factor of 25%, meaning every fifth well is a redundant well.

On average, the Southwest wellfield has a flow rate of 432 gpm per well and North wellfield has an average flow rate of 347 gpm. A flow rate of 400 gpm was selected in coordination with the City for planning purposes as a representative flow rate for a brackish well.

The number of wells needed to meet the projected potable demands is presented on **Figure 3-17**. Currently there are 50 operating wells feeding a total WTP capacity of 30 MGD. In the year 2040, the City will need a total of 88 wells to meet a total raw water demand of 40.4 MGD. In the year 2080, the City will need 103 wells to feed a total raw water demand of 47.6 MGD. This indicates that the City will need an additional 53 operational wells to meet the projected MDD through 2080. The recommended timeline for construction of production wells is presented in **Table 3-18**.

Table 3-18 Recommended Timeline for the Construction of Wells*

Time Period	No. of Production Wells Needed
2021 to 2025	8
2026 to 2030	10
2031 to 2035	12
2036 to 2040	8
2041 to 2045	6
2046 to 2050	3
2051 to 2055	2
2056 to 2060	1
2061 to 2065	1
2066 to 2070	1
2071 to 2075	1
2076 to 2080	1
Total	53

*For cost estimating purposes, wells were bundled up and associated with plant expansions.

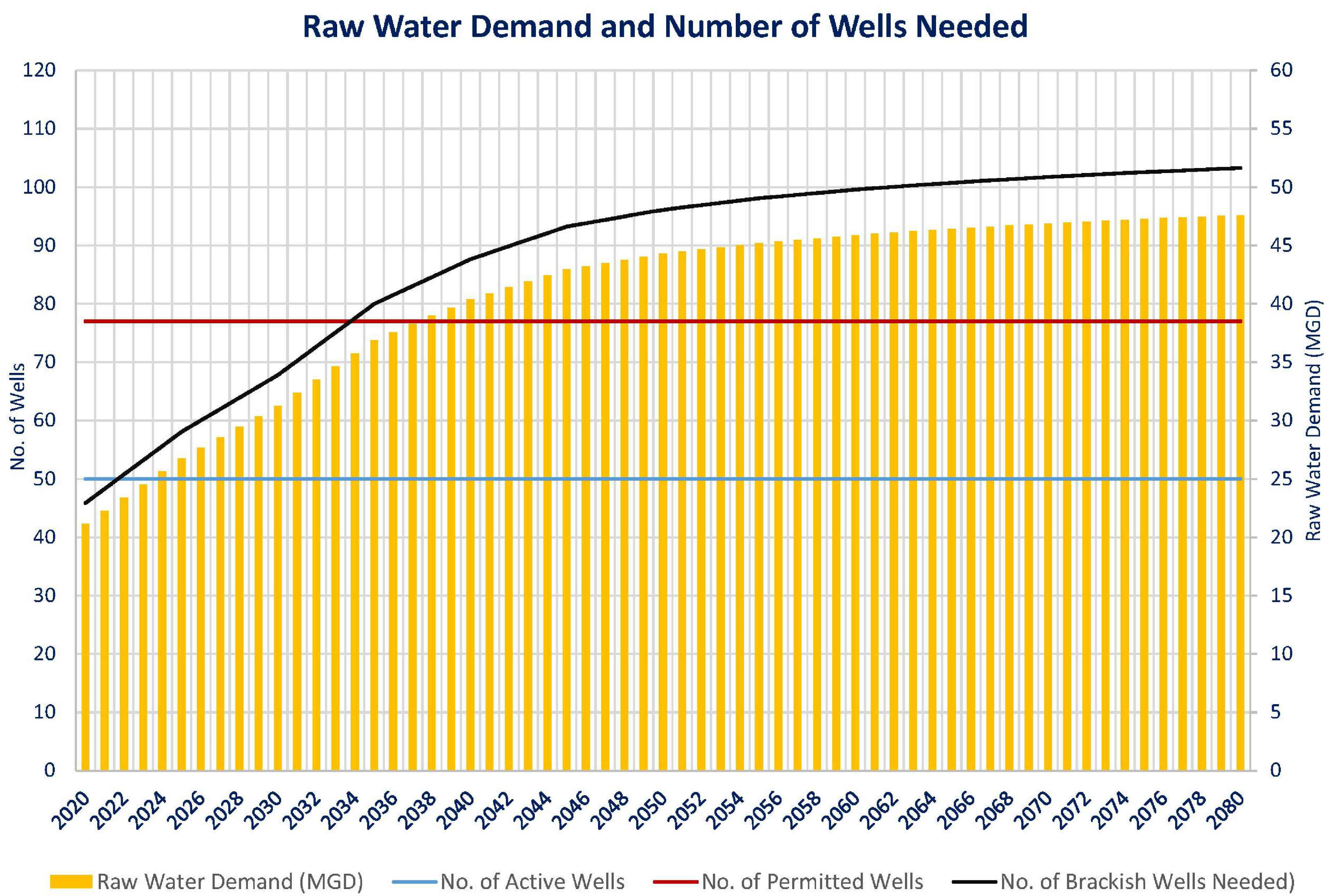


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Figure 3-17: Raw Water Demand and Number of Wells Needed.

As noted on **Figure 3-17**, the current number of wells is sufficient to meet the current demands. However, the City is projected to need an average of 2 new production wells every year until 2040, an average of 1 new well per year from 2040 to 2050, and an average of 1 new well every 4 years from 2050 to 2080.

Prospective locations for additional brackish wells are shown on **Figure 3-18**. In general, the ideal locations of new wells are along, or close to, existing transmission pipelines, at least 1,500 feet away from existing active wells, and on City owned parcels. For future wellfield expansions, which involves construction of up to 53 new wells, the City will need to plan for approximately 15 miles of linear wellfield alignments as shown on **Figure 3-18**.

Raw Water By-Pass Blend

The Mid-Hawthorn aquifer is a suitable freshwater source for by-pass blending of the RO permeate produced at City's WTPs. Typically, raw water by-pass blends with RO permeate are used to add needed mineral content and chemical stability to the finished drinking water product. The potential for raw water by-pass blending with brackish water is typically limited because of the high dissolved solids content of the source aquifer. The Mid-Hawthorn aquifer has been essentially off limits to any new use of significance for the past two decades. However, with the recovery of water levels south of Pine Island Road, the aquifer may have renewed viability for targeted use as a blending source. The Mid-Hawthorn aquifer wells typically yield between 100 and 200 gallons per minute within the City limits. If the City determines that there is a need for additional fresh water raw blending, we recommend installing one Mid-Hawthorn aquifer well each at the WTP facilities initially and expanding the number of wells based on evaluation of need and finished water quality. Prospective locations for additional Mid-Hawthorn aquifer blend wells are shown on **Figure 3-19**.

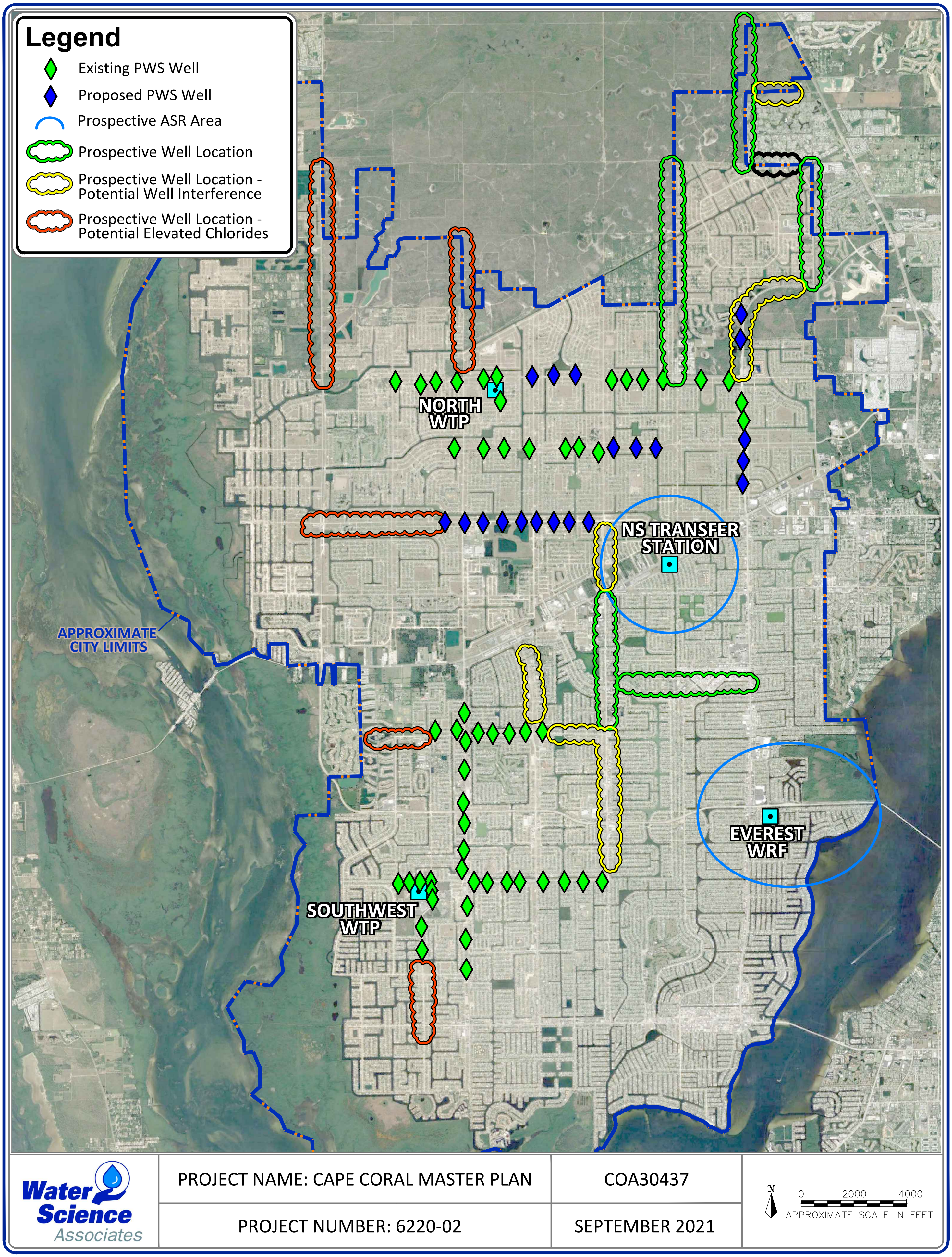
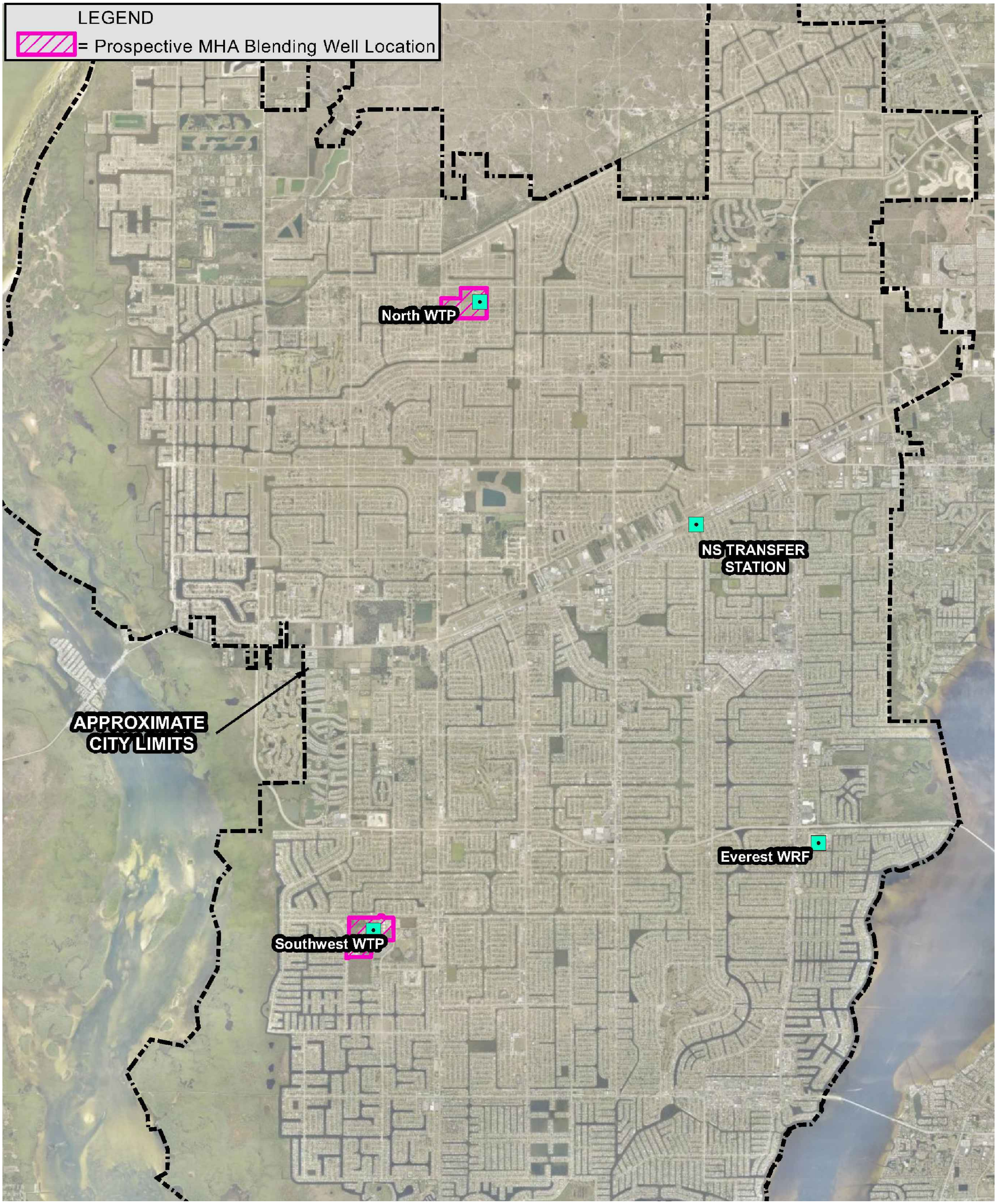


Figure 3-18: Prospective Locations for Additional Brackish Wells.



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	N 0 3500 7000 APPROXIMATE SCALE IN FEET
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-19: Prospective Locations for Mid-Hawthorn Aquifer Blend Wells.

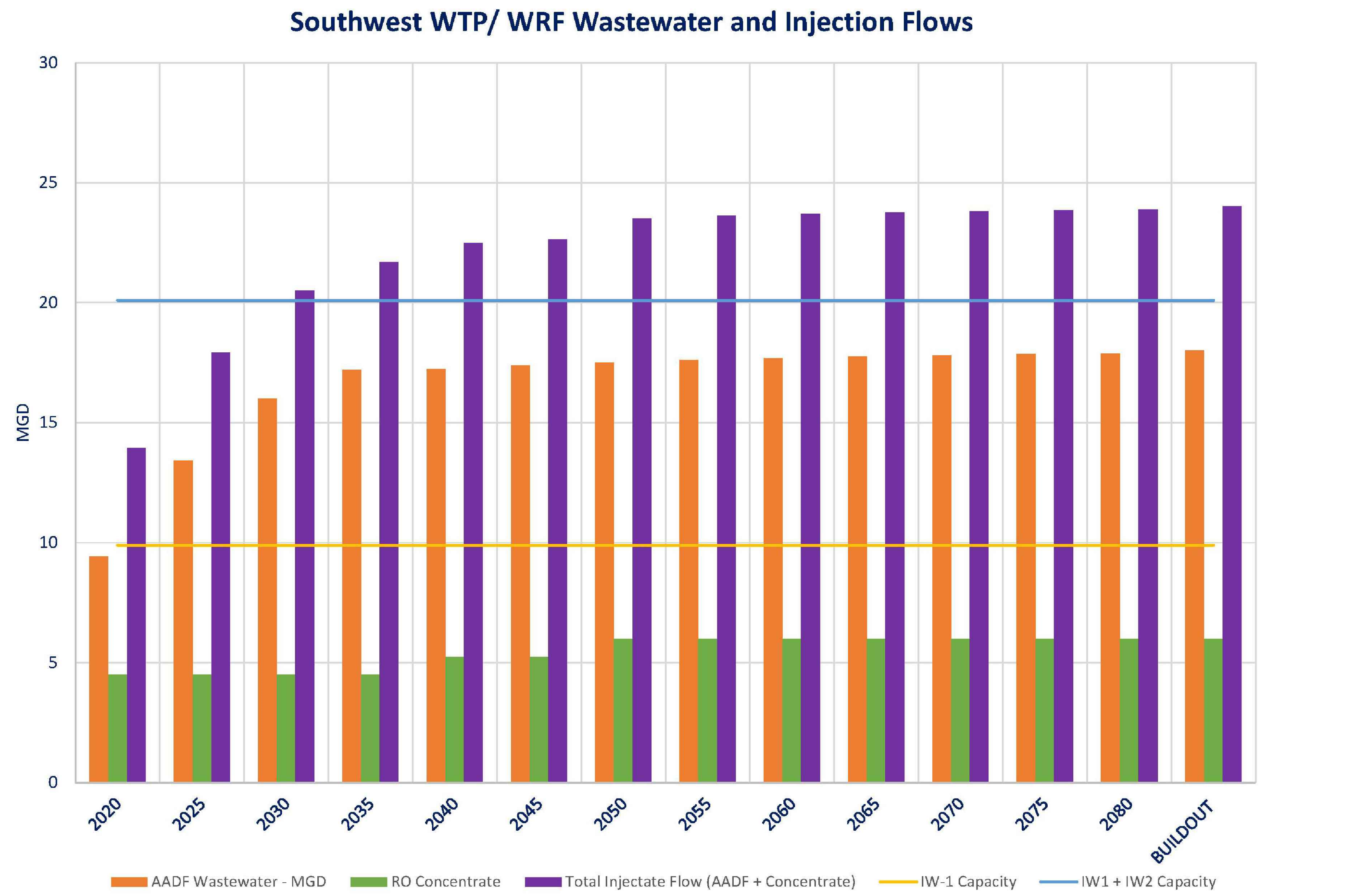
3.8.3 Concentrate Effluent Disposal Gap Analysis

The City's two RO WTPs require deep injection well systems for disposal of RO concentrate. The RO treatment systems at the Southwest and North water treatment facilities are projected to have an ultimate combined treatment capacity of 48 MGD (24 MGD each) and a treatment efficiency of 80%. These facilities will generate RO concentrate at a rate of 6 MGD each when operating at full capacities.

Southwest RO WTP/WRF

The injection well system at the Southwest RO WTP facility is used for disposal of both RO concentrate and treated municipal effluent from the SW WRF. The maximum projected RO concentrate production for the Southwest RO WTP ranges between 4.5 MGD (2020) to 6.0 MGD when the plant capacity is expanded to 24 MGD. The Southwest WRF has a rated treatment capacity of 15 MGD. The City has plans to expand the treatment capacity to 20 MGD by 2025. The projected Annual Average Daily Flow (AADF) to the Southwest WRF ranges between 9.4 MGD (2020) and 18.0 MGD (Buildout). However, the primary disposal for the WRF is provided through irrigation reuse with the backup disposal provided by the injection well system.

The injection well system at this facility consists of 2 deep injection wells (IW-1 and IW-2) and a dual-zone monitor well (DZMW-1). The total injection capacity of the injection well system at this facility is approximately 20 MGD which includes a permitted capacity of 9.7 MGD for IW-1 and 10.2 MGD for IW-2. Projected wastewater flows for the expanded WRF, RO concentrate flows, and existing IW capacity are shown on **Figure 3-20**. The projected total injectate flow for the Southwest RO WTP/WRF at buildout is approximately 24 MGD, which is slightly higher than the current injection capacity of 20 MGD at the facility. However, because the City is able to use all of its wastewater effluent daily for irrigation, use of the injection well system for backup wastewater effluent disposal is extremely rare. Construction of an additional injection well to cover this small projected deficit that would essentially never be utilized is not recommended as there are other means to manage the available effluent. Injectate flow exceeding the injection capacity available at the Southwest WTP/WRF can be stored in the reject tanks.



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	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-20: Wastewater and RO Concentrate Flows and Existing Injection Well Capacity at the Southwest RO WTP/WRF.

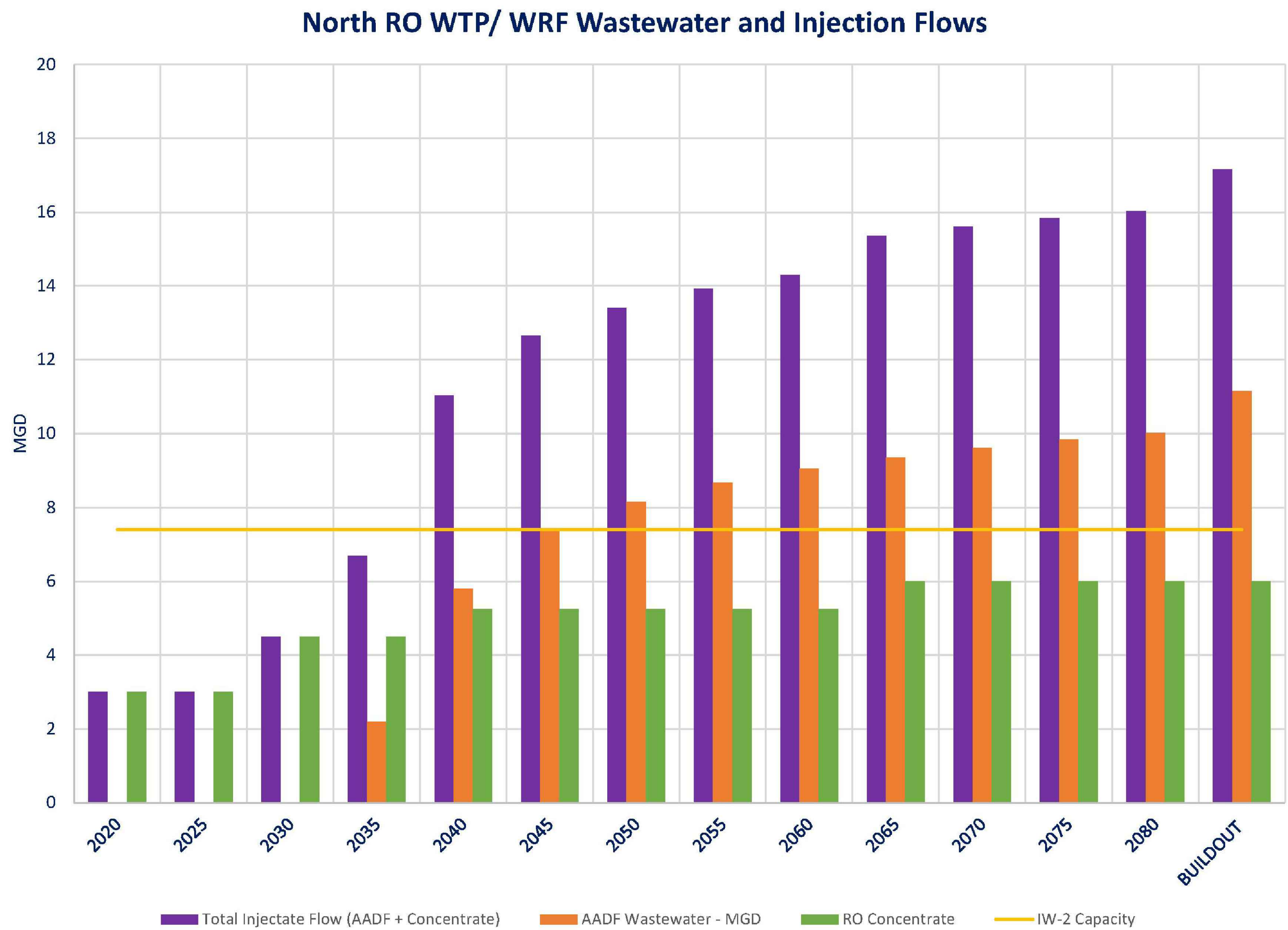
North RO WTP/WRF

The North RO WTP has one existing injection well (IW-2) with a permitted capacity of 7.4 MGD and an associated Dual Zone Monitoring Well (DZMW) which monitors for upward migration from the injection zone. The facility will need a second injection well (IW-1) and associated DZMW. A second injection well is anticipated to be operational by 2027 and will also serve for the excess treated effluent from the proposed North WRF, which is expected to be operational in 2035. The primary disposal mechanism for the North WRF will be provided through irrigation reuse however, backup disposal for the WRF is provided by the injection well system. The second DZMW will be required since the second injection well is proposed to be constructed off the site of the North RO WTP/WRF. A separate injection well site is proposed due to the less than typical amount of confinement strata encountered during construction of IW-2 at the North RO WTP.

The maximum injection well disposal need at the North RO WTP ranges between 3.0 MGD (2020) to 6.0 MGD when the plant capacity is expanded to 24 MGD. The projected AADF wastewater flow for the North WRF ranges between 2.2 MGD (2035) and 11.1 MGD (Buildout). The anticipated total injectate flow (wastewater AADF flow + concentrate) at buildout is 17.1 MGD, which indicates that the facility will require additional injection capacity of approximately 10 MGD. A single 10-MGD wastewater tubing and packer injection well will serve as a back-up well for RO concentrate disposal as well as for the planned expansion of wastewater disposal requirements. The well locations should be selected based on the results of the seismic survey and modeling analysis that are currently underway. Refer to **Figure 3-21** for a summary of wastewater flow components for the North RO WTP and the proposed North WRF facilities.

3.8.4 Analysis of Transmission and Distribution Systems

Hydraulic modeling analyses of the potable water transmission and distributions system was completed using the City's hydraulic model in InfoWater which was updated and calibrated as a part of this master planning effort. The hydraulic model was used to evaluate the existing and future transmission and distribution system conditions and to identify system deficiencies and improvements needed. **Figure 3-22** provides a map of the existing water transmission and distribution system.



Water Science Associates	PROJECT NAME: CAPE CORAL MASTER PLAN	COA30437	NOT TO SCALE
	PROJECT NUMBER: 6220-02	NOVEMBER 2022	

Figure 3-21: Wastewater and RO Concentrate Flows and Existing Injection Well Capacity at the North RO WTP/WRF.

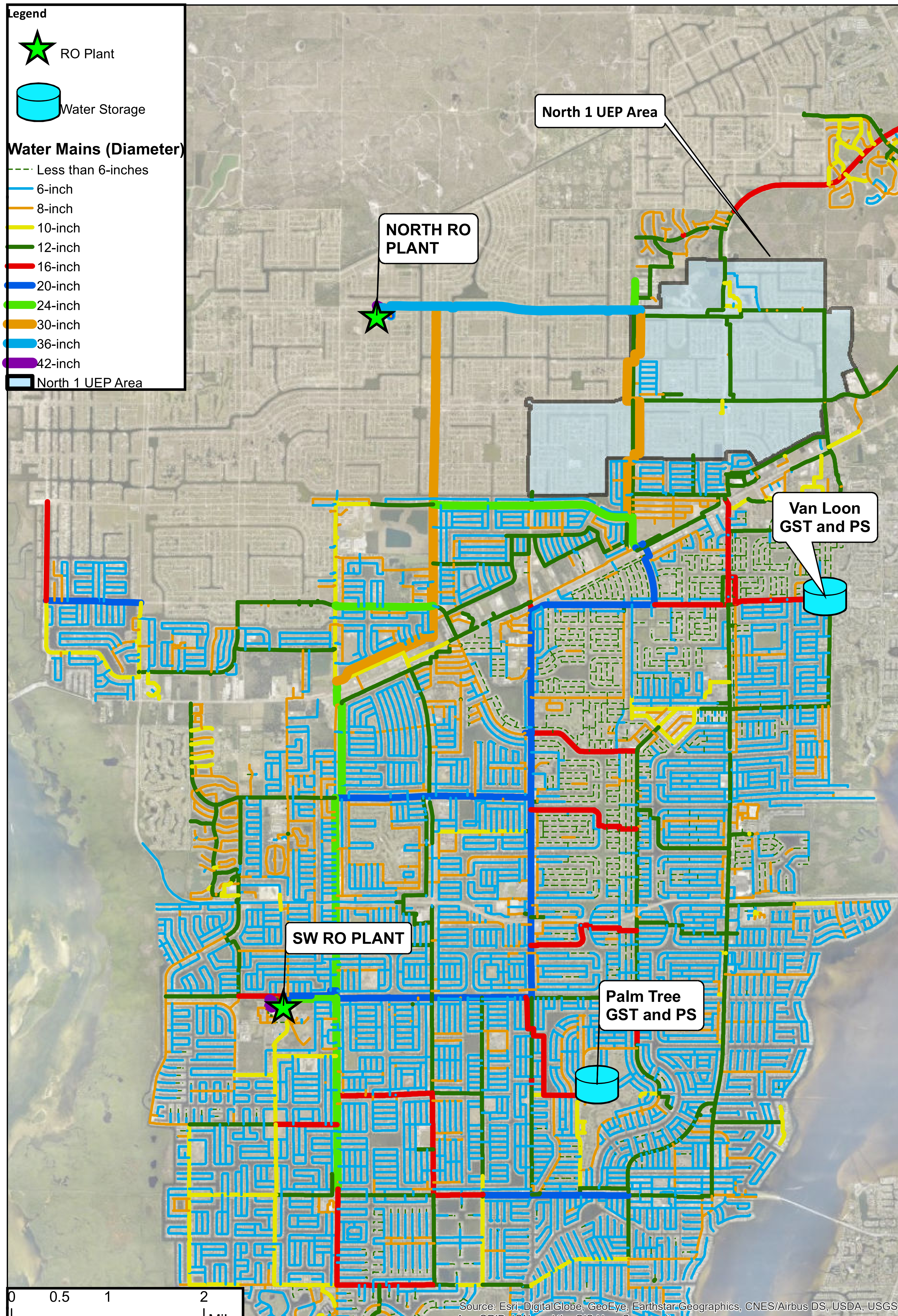


Figure 3-22
Existing Water Distribution System

City of Cape Coral
Comprehensive Utilities Master Plan Update

CDM Smith

HOLOWELLJD E:\4.0 Current Project Files\Cape Coral\Potable Water Master Plan (AECOM)\GIS Figures\Maps\Task 3 - North 1 UEP Memo\Figure 1-1 - Existing System.mxd 2/12/2021

3.8.4.1 North 1 Analysis

The City is moving forward with extending potable water service (along with irrigation and wastewater services) to the North 1 UEP by FY 2024. Therefore, a hydraulic modeling analysis was completed to simulate various potable water system conditions to determine water main sizes to meet the current and future needs of the North 1 project area. This analysis was completed earlier in the master planning process and updates to service areas, population projections, and demands due to planned development projects occurring later in the master planning process were updated prior to final submission in March 2021. The final hydraulic modeling analysis was completed in October 21 and this analysis captured further modifications which impacted potable water demand projections and needed potable water system infrastructure improvements. A summary of the results and recommendations of these analyses are provided below. Additional details of the hydraulic modeling evaluation are included in the Technical Memorandum “Preliminary Potable Water Distribution System Model Buildout Analysis for the North 1 Utilities Extension Project Area”.

There were eight scenarios simulated for this evaluation. The scenarios included the following:

- Scenario 1 (Buildout Conditions): This scenario evaluated the pressures and water age in the North 1 UEP area during a 336-hour extended period simulation (EPS) of the City’s water distribution system at buildout conditions. The system demands are based on the projected buildout AADD of 34.5 MGD. These system demands were initially based upon a service buildout population of 359,793, per capita demands of 91.2 gpcd, and an additional 1.00 MGD to account for development projects referred to as Pine Island East PUDs and PF for MDD of 1.35.
- Scenario 2 (Buildout Conditions): This scenario evaluated the pressures in the North 1 UEP area during a 48-hour extended period simulation (EPS) of the City’s water distribution system at buildout conditions. The system demands are based on the project buildout MDD of 46.6 MGD. This scenario included the peak hour demand conditions of 74.9 MGD (51,994 gpm) projected at buildout.
- Scenario 3 (Buildout Conditions): This scenario evaluated the available fire flow in the North 1 UEP area during a steady-state fire flow simulation of the North 1 UEP area at buildout conditions. The system demands are based on the projected buildout MDD of 46.6 MGD.
- Scenario 4 (Buildout Conditions): This scenario evaluated the pressures and water age in the North 1 UEP area during a 336-hour extended period simulation (EPS) of the City’s water distribution system at buildout conditions. The system demands are based on the projected buildout minimum day demand (MINDD) of 20.7 MGD.
- Scenario 5 (North 1 Constructed Conditions): This scenario evaluated the pressures and water age in the North 1 UEP area during a 336-hour extended period simulation (EPS) of the City’s water distribution system following construction of the North 1 UEP area. This

scenario includes the existing system, the North 2 UEP area, and the North 1 UEP area. The system demands are estimated in this scenario to be representative of the demand when the North 1 UEP area is in service. The estimated AADD used in this scenario is 14.7 MGD. The purpose of modeling the estimated demands around the time that North 1 UEP area comes into service is to evaluate any potential water age issues due to the lower starting system demand.

- Scenario 6 (North 1 Constructed Conditions): This scenario evaluated the pressures in the North 1 UEP area during a 48-hour extended period simulation (EPS) of the City's water distribution system following construction of the North 1 UEP area. The system demands are based on the project buildout MDD for the existing, North 1, and North 2 service areas of 31.9 MGD. This scenario included the peak hour demand conditions of 51.4 MGD (34,403 gpm) projected at buildout. The purpose of modeling the buildout demands for the existing, North 1, and North 2 service areas is to evaluate the water distribution conveyance system's ability to maintain pressures in the event that the remaining UEP areas are not constructed.
- Scenario 7 (North 1 Constructed Conditions): This scenario evaluated the available fire flow in the North 1 UEP area during a steady-state fire flow simulation of the North 1 UEP area following construction of the North 1 UEP area. The system demands are based on the projected buildout MDD of 31.9 MGD.
- Scenario 8 (North 1 Constructed Conditions): This scenario evaluated the pressures and water age in the North 1 UEP area during a 336-hour extended period simulation (EPS) of the City's water distribution system following construction of the North 1 UEP area. Similar to the AADD scenario (Scenario 5), the system demands are estimated in this scenario to be representative of the demand when the North 1 UEP area is in service. The estimated MINDD used in this scenario is 8.8 MGD.

Buildout Conditions

The MDD and MDD plus fire flow buildout condition scenarios were used to ultimately size the water mains in the North 1 UEP area. These scenarios also evaluated the existing transmission system and capacity deficiencies that could impact the conveyance to the North 1 UEP area. The results of the hydraulic model indicated that there are no capacity deficiencies with the existing transmission system. For the maximum day analysis, all portions of the North 1 UEP system are projected to exhibit minimum pressures above 50 psi. This is a result of an adequately sized transmission system supplying the project area at buildout. Based on the results of the fire flow simulation under MDD conditions, the majority of the North 1 UEP area is predicted to be able to produce greater than 3,500 gpm for fire flow needs. The northeast corner of the North 1 UEP area is predicted to have the lowest available fire flow. The predicted fire flow in this area is greater than 1,500 gpm with most junctions predicted to have greater than 2,000 gpm available fire flow as shown in **Figure 3-23.**

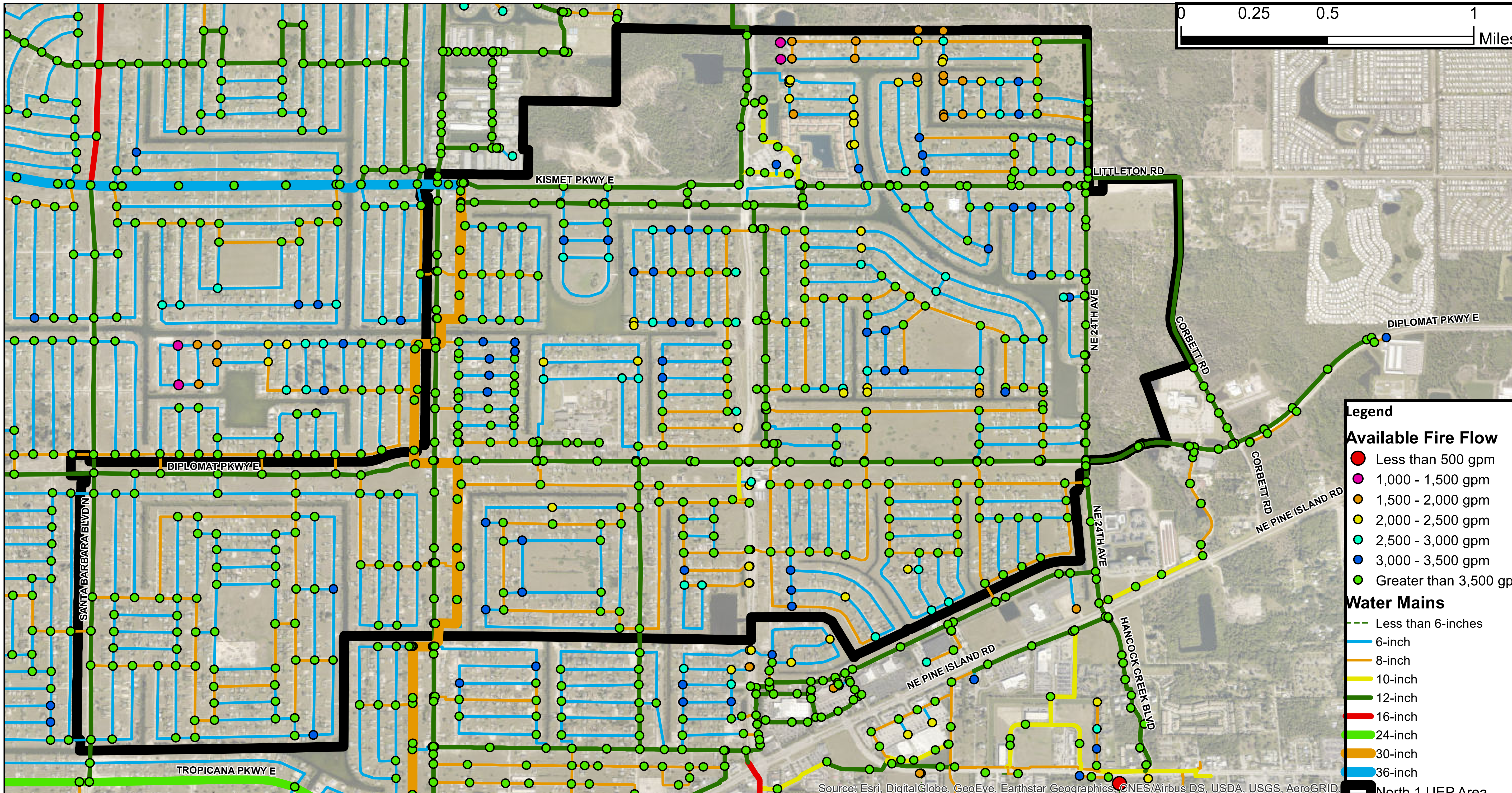


Figure 3-23
Buildout MDD Available Fire Flow

City of Cape Coral
Comprehensive Utilities Master Plan Update

CDM Smith

HOLOWELLJD E:\4.0 Current Project Files\Cape Coral\Potable Water Master Plan (AECOM)\GIS Figures\Maps\Task 3 - North 1 UEP Memo\Figure 3-2 - Buildout MDD Available Fire Flow.mxd 2/12/2021

The MINDD and AADD scenario were ultimately used to evaluate potential concerns associated with the selected water main sizes as it relates to water age. For the annual average demand analysis, the model predicts that the maximum water age in the North 1 UEP area is less than 7-days at buildout conditions. The minimum day analysis showed similar a pattern of water age in the North 1 UEP Area with the maximum water age in the northeast area between 7-10 days. This is not a big concern as this was a conservative extended period simulation with a minimum day occurring for 14 consecutive days.

North 1 Constructed Conditions

The MDD and MDD plus fire flow buildout condition scenarios were used to confirm that the sizes selected for the water mains would be adequate to meet the service area needs at buildout without the addition of the future UEP areas. These scenarios also evaluated the existing transmission system and capacity deficiencies that could impact the conveyance to the North 1 UEP area. The results of the hydraulic model indicate that there are no capacity deficiencies with the existing transmission system supplying the North 1 UEP Area.

For the maximum day analysis, all portions of the North 1 UEP system are projected to exhibit minimum pressures above 60 psi. The model did not predict any deficiencies in the existing transmission system with adding the North 1 UEP service area demands. The results of the fire simulation under MDD conditions did not change significantly from the buildout scenario with the majority of the North 1 UEP predicted to be able to produce greater than 3,500 gpm for fire flow needs. The model predicts that the lowest available fire flow in the northeast area is still greater than 1,500 gpm in this fire flow simulation. One observation noted was that the fire flows improved at buildout conditions due to the additional looping in the system with future expansions.

Similar to the buildout conditions, the MINDD and AADD scenario were ultimately used to evaluate potential concerns associated with the selected water main sizes as it relates to water age. For the annual average demand analysis, the model predicts that the maximum water age in the majority of the North 1 UEP area is less than 4-days. There were a few nodes that the model predicts the maximum water age to be greater than 10-days. This was generally the same for the minimum day analysis with the maximum water age being 4-days or less with a few nodes displaying the maximum water age of greater than 10-days. The results of both scenarios indicate that routine flushing should be expected in the near term as the majority of junctions displaying higher water age were at dead end runs.

Summary and Recommendations

The hydraulic model was used to evaluate and identify the sizes for the proposed potable water distribution and transmission mains in the North 1 UEP area. The recommended piping networks consist of primarily a 12-inch transmission backbone, an 8-inch feeder system within neighborhoods, and 6-inch piping for most streets. This combination provides an economic solution while balancing pressure, fire flow availability and water age within City goals.

The North 1 UEP area was broken up into four areas to provide higher resolution figures showing the proposed water main sizes. **Figure 3-24** provides a key map for the four areas delineated within the North 1 UEP area. **Figures 3-25 through 3-28** provides the recommended water main sizes for the four areas. Any changes to the piping sizes during the engineering design of the North 1 UEP area should be verified in the hydraulic model.

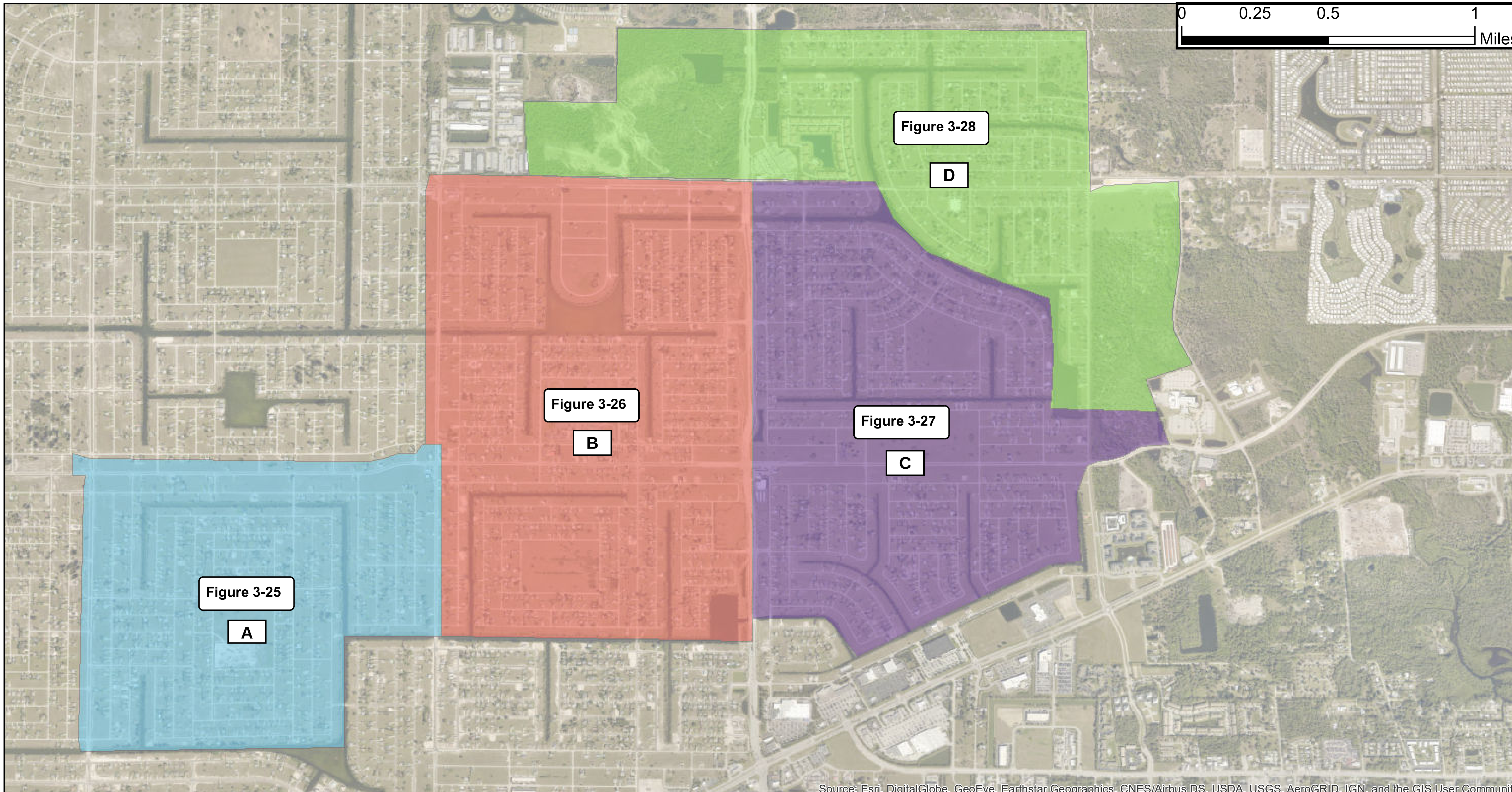


Figure 3-24
Proposed North 1 Water Main Improvements Key Map

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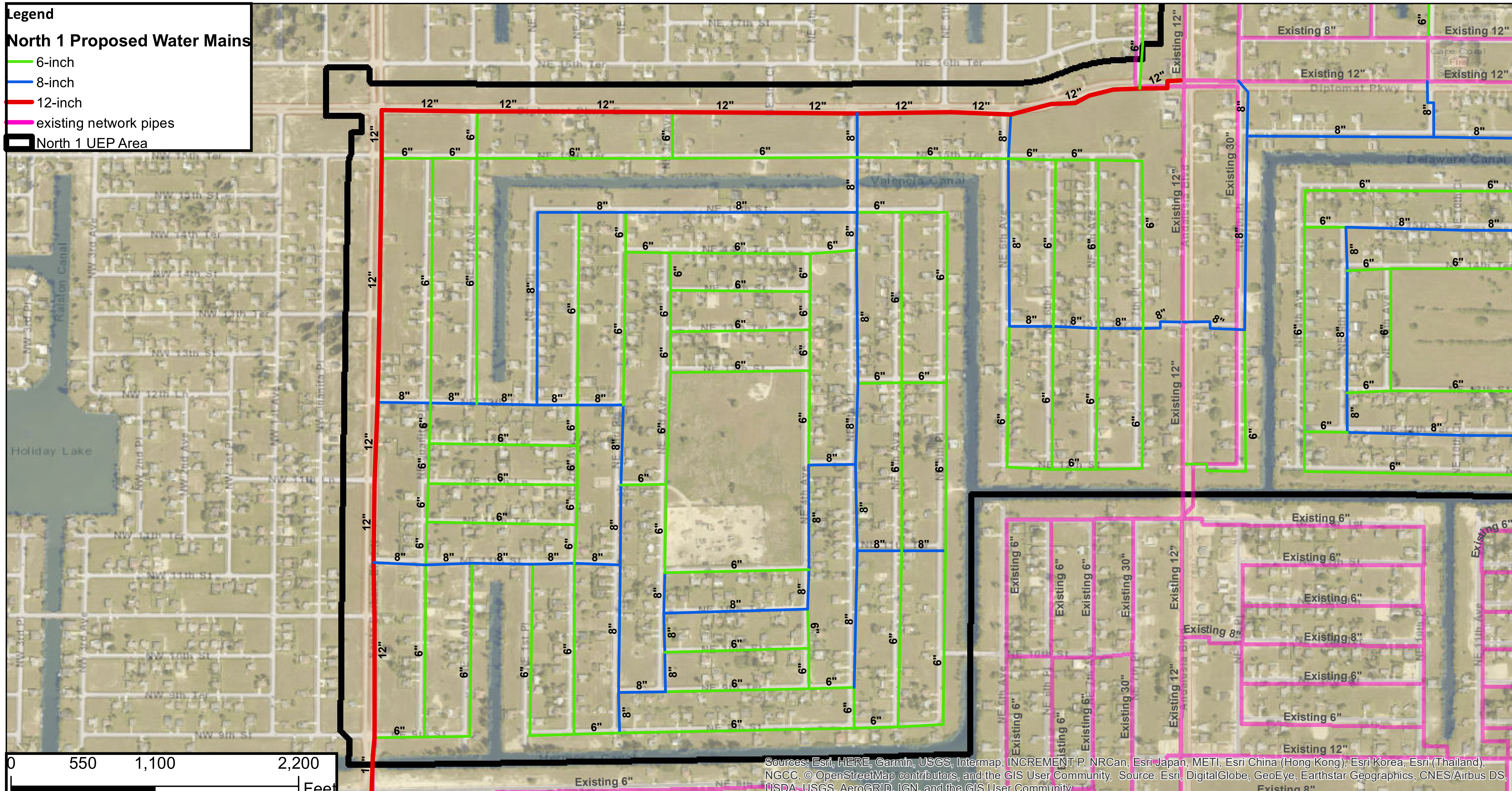
HOLOWELLJD E:\4.0 Current Project Files\Cape Coral\Potable Water Master Plan (AECOM)\GIS Figures\Maps\Task 3 - North 1 UEP Memo\Figure 4-1 - North 1 Key Map.mxd 2/12/2021



N

Figure 3-25

Proposed North 1 Water Main Improvements (A)

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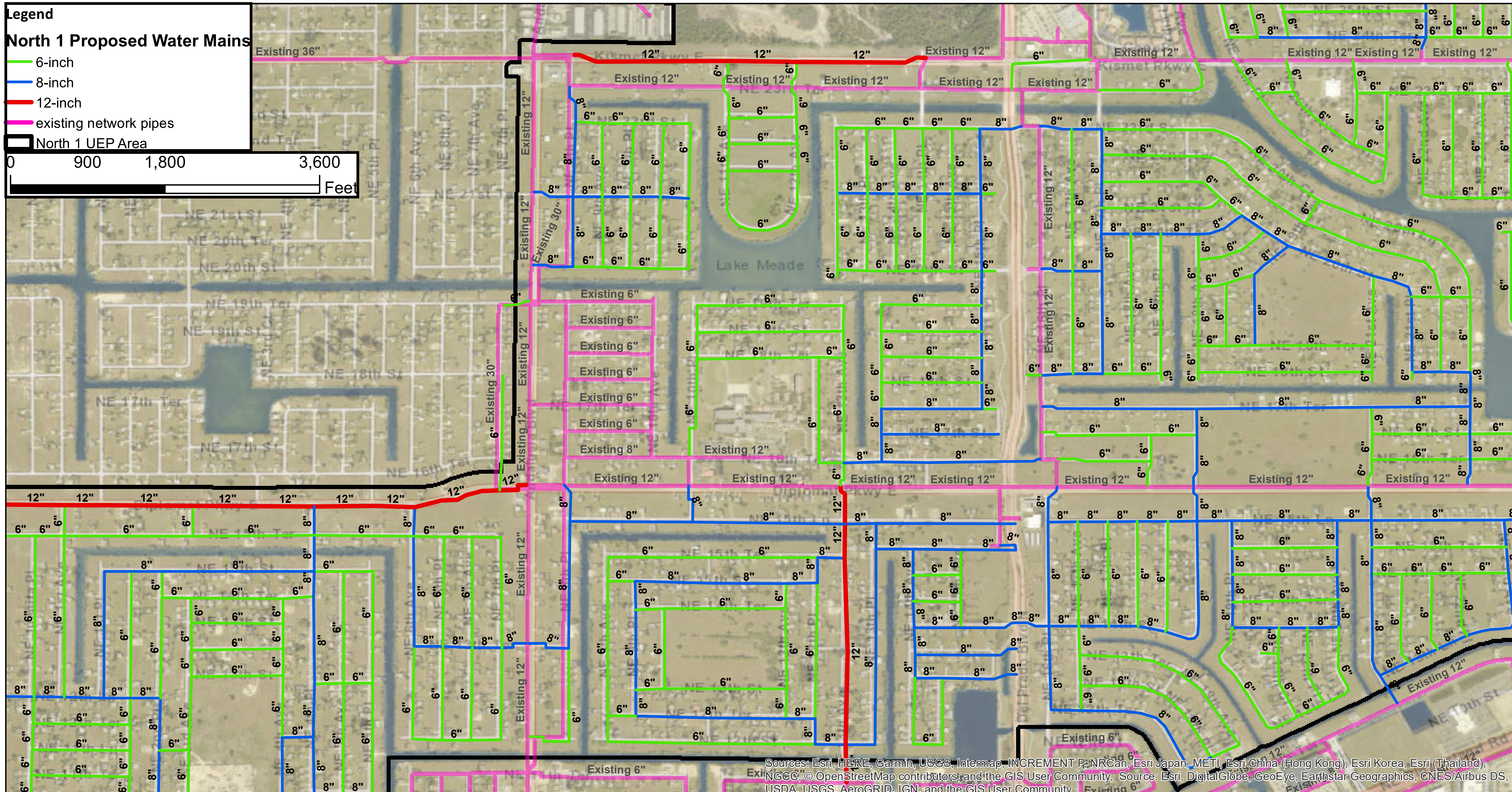

Figure 3-26

Proposed North 1 Water Main Improvements (B)

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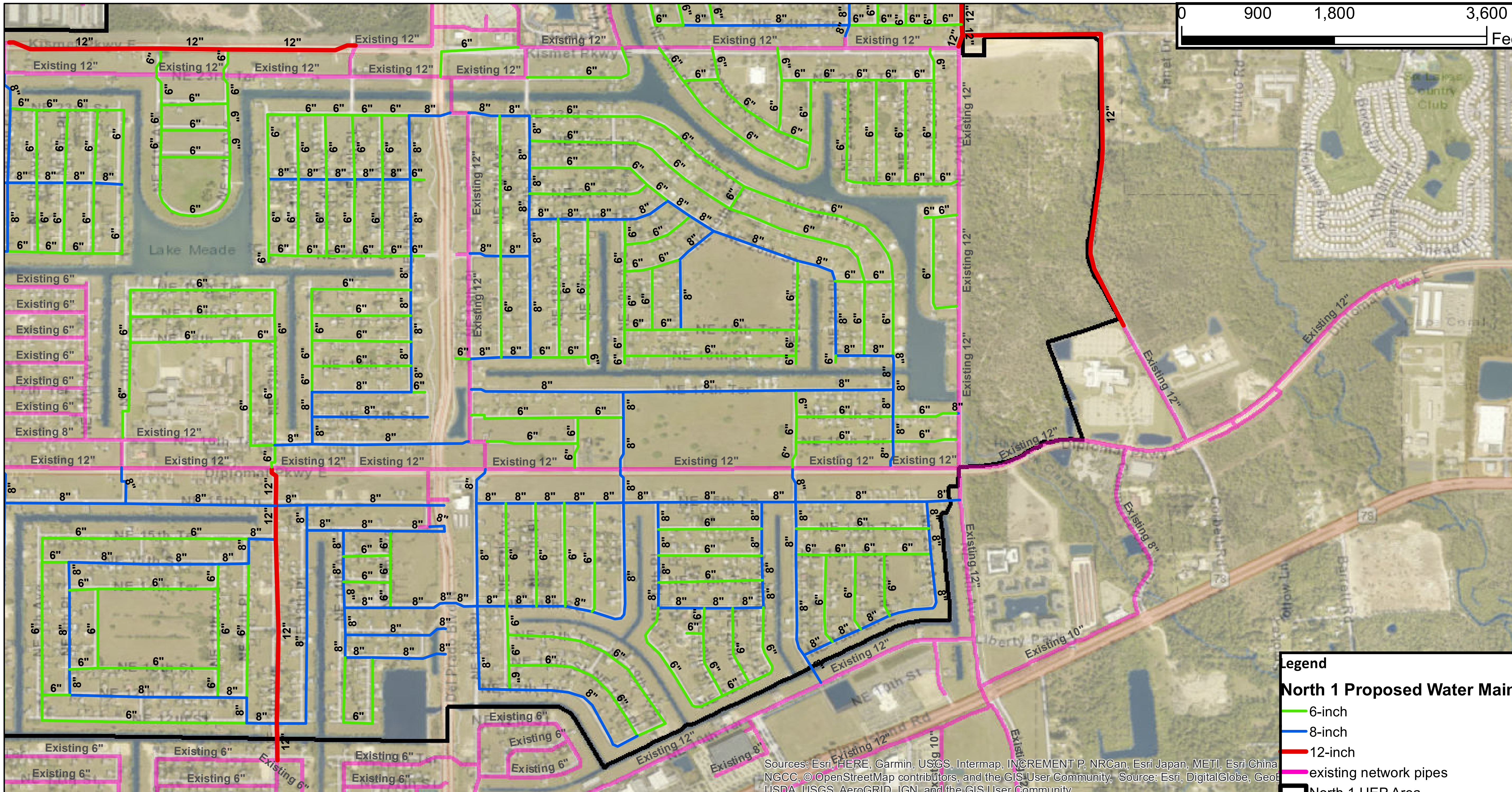

Figure 3-27

Proposed North 1 Water Main Improvements (C)

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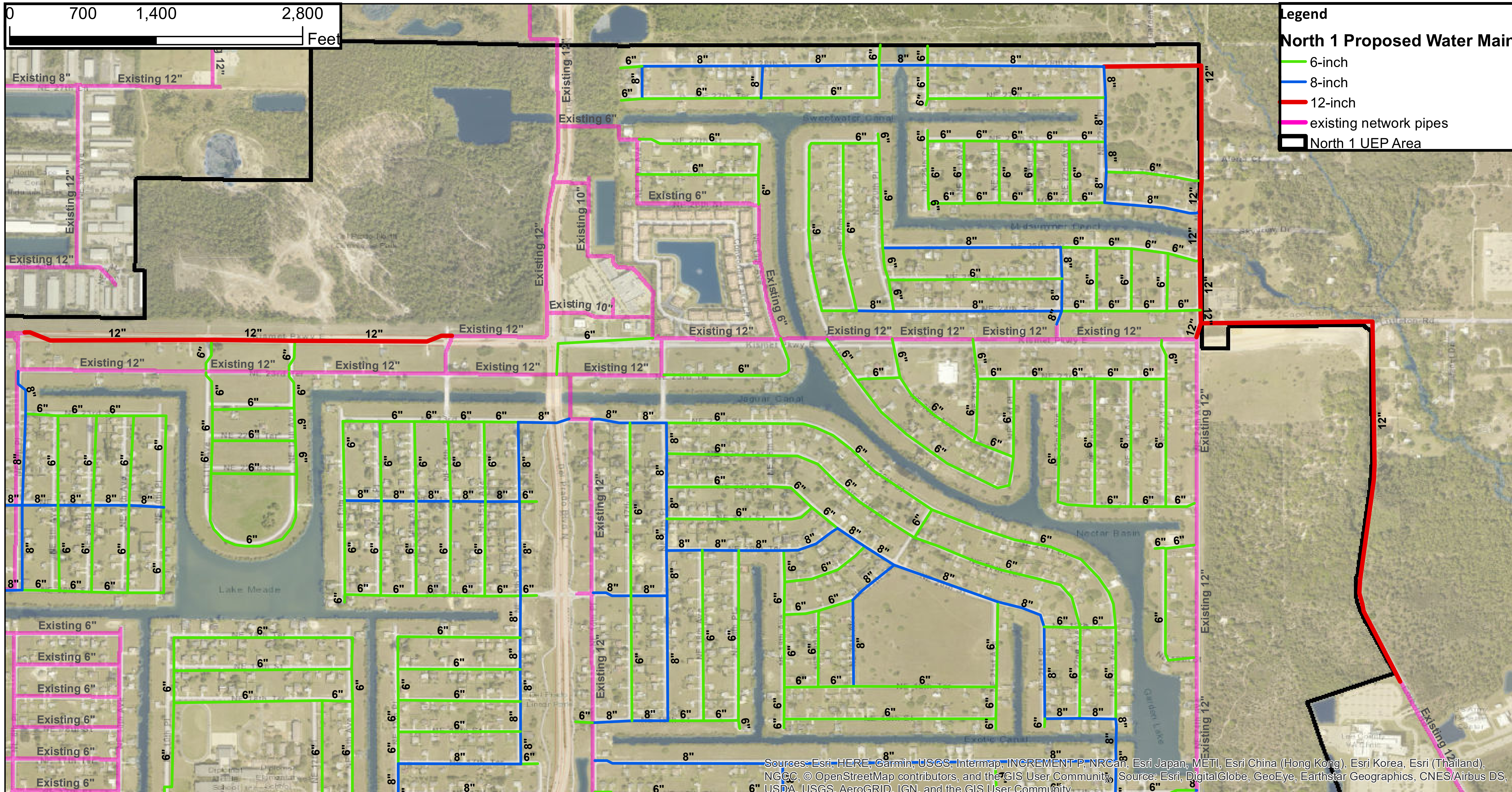

Figure 3-28

Proposed North 1 Water Main Improvements (D)

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HOLOWELLJD E:\4.0 Current Project Files\Cape Coral\Potable Water Master Plan (AECOM)\GIS Figures\Maps\Task 3 - North 1 UEP Memo\Figure 4-5 - North 1 WM Improvements.mxd 2/16/2021

3.8.4.2 Hydraulic Modeling Analysis

Near-term and long-term hydraulic modeling analyses were performed to determine the adequacy of the water distribution system's ability to meet level of service standards.

Five scenarios were evaluated using the hydraulic model:

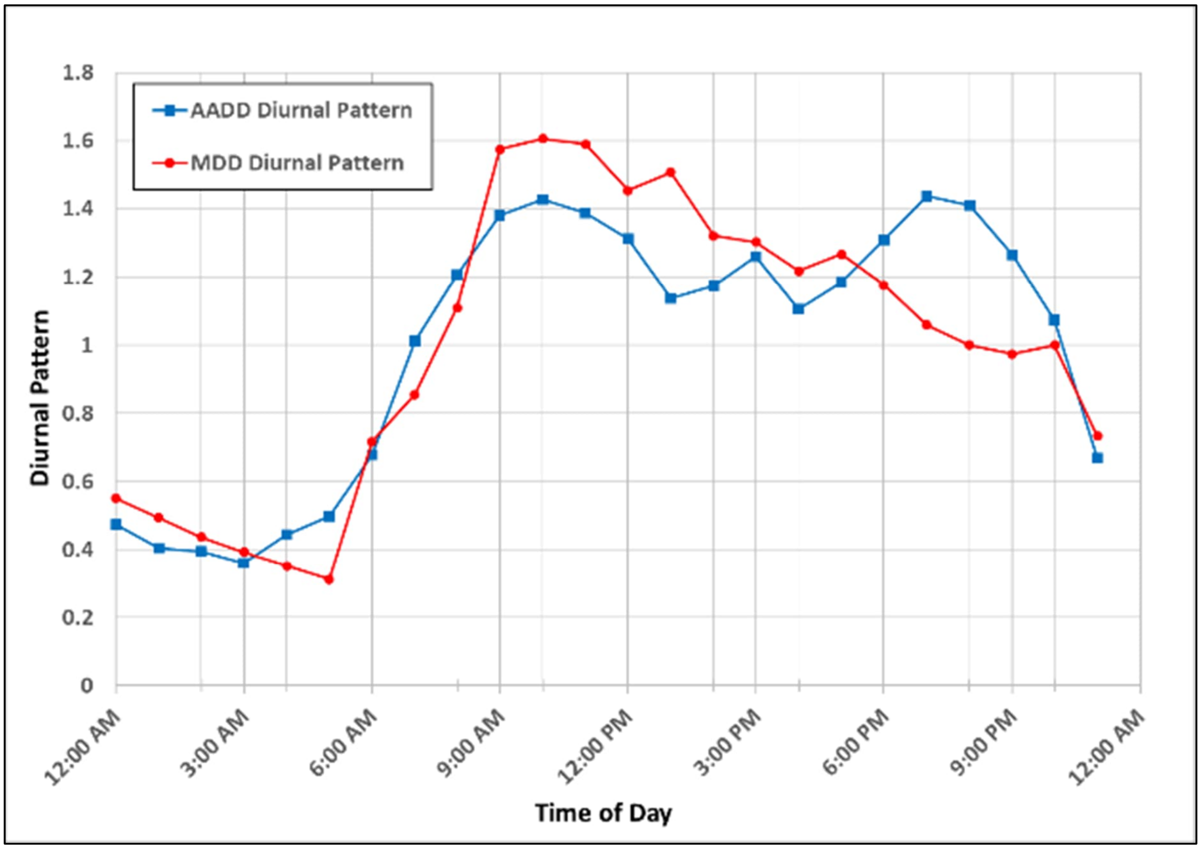
- 2020 Existing Conditions Scenario. Under this scenario the existing water distribution system was evaluated using an annual average daily demand (AADD) calculated based on billing records for the existing customers and an estimated AADD for the North 2 UEP area.
- 2025 (Short Term Scenario)
- 2030 (Mid-Term Scenario)
- 2040 (Long Term Scenario
- Buildout conditions

The hydraulic modeling analysis was performed for the following conditions:

- AADD – 14 day (336 hour) extended period simulation (EPS) to evaluate system pressures and water age
- Maximum Day – 2 day (48 hour) extended period simulation to evaluate system pressures, pumping capacities, and storage volumes to determine the system's response to consecutive high demand days. This simulation includes the peak hour demand condition
- Minimum Day – 14-day (336-hour) EPS to evaluate water age
- Maximum Day Fire Flow – to perform individual steady state simulations on each junction assigned with a fire flow goal. The fire flow simulations include maximum day demands.

The diurnal patterns used for the AADD and MDD model simulations are shown in **Figure 3-29**. As shown in **Figure 3-29**, the peak hour demand is set to occur at 10AM during a MDD condition with a multiplication factor of 1.61 (PHD = 1.61 x MDD).

Figure 3-29: Updated Annual Average Daily Demand and Maximum Day Demand Diurnal Patterns



A summary of the results of the hydraulic modeling is provided herein. More detailed results are provided in the Technical Memorandum “Potable Water Distribution System Model Update”.

2020 Existing Conditions Modeling Results

A 2020 existing conditions analysis was performed to evaluate the existing water distribution system, including the recently constructed North 2 UEP area, to identify system deficiencies where level of service standards are not being met. This analysis also identifies recommended improvements to address any noted system deficiencies and the timing of the implementation needed.

The most significant issue noted as a result of the 2020 Existing Conditions Modeling is that there are some areas where a fire flow of 1000 gpm could not be provided under a MDD condition. As noted in **Figure 3-30**, most of these areas with fire flow deficiencies are in older parts of the water distribution system with water main diameters less than 6-inches.

The City has implemented a small diameter pipe replacement program. This program already has plans to replace existing small diameter water mains within four areas between Santa Barbara Boulevard and County Club Boulevard as part of a phased program with completion expected before the 2030 planning year. Based on feedback from the City, replacement of the water mains in two of the areas are intended to be completed by 2025 and the other two by 2030. An additional four areas with large quantities of existing water mains less than 6-inches are anticipated to be completed by 2035 and 2040. **Figure 3-31** provides a map that displays the delineated areas with existing water mains less than 6-inches in diameter and the assumed implementation timing of the small diameter pipe replacement program. The remaining portions of the system with water mains less than 6-inches are anticipated to be replaced by buildout.

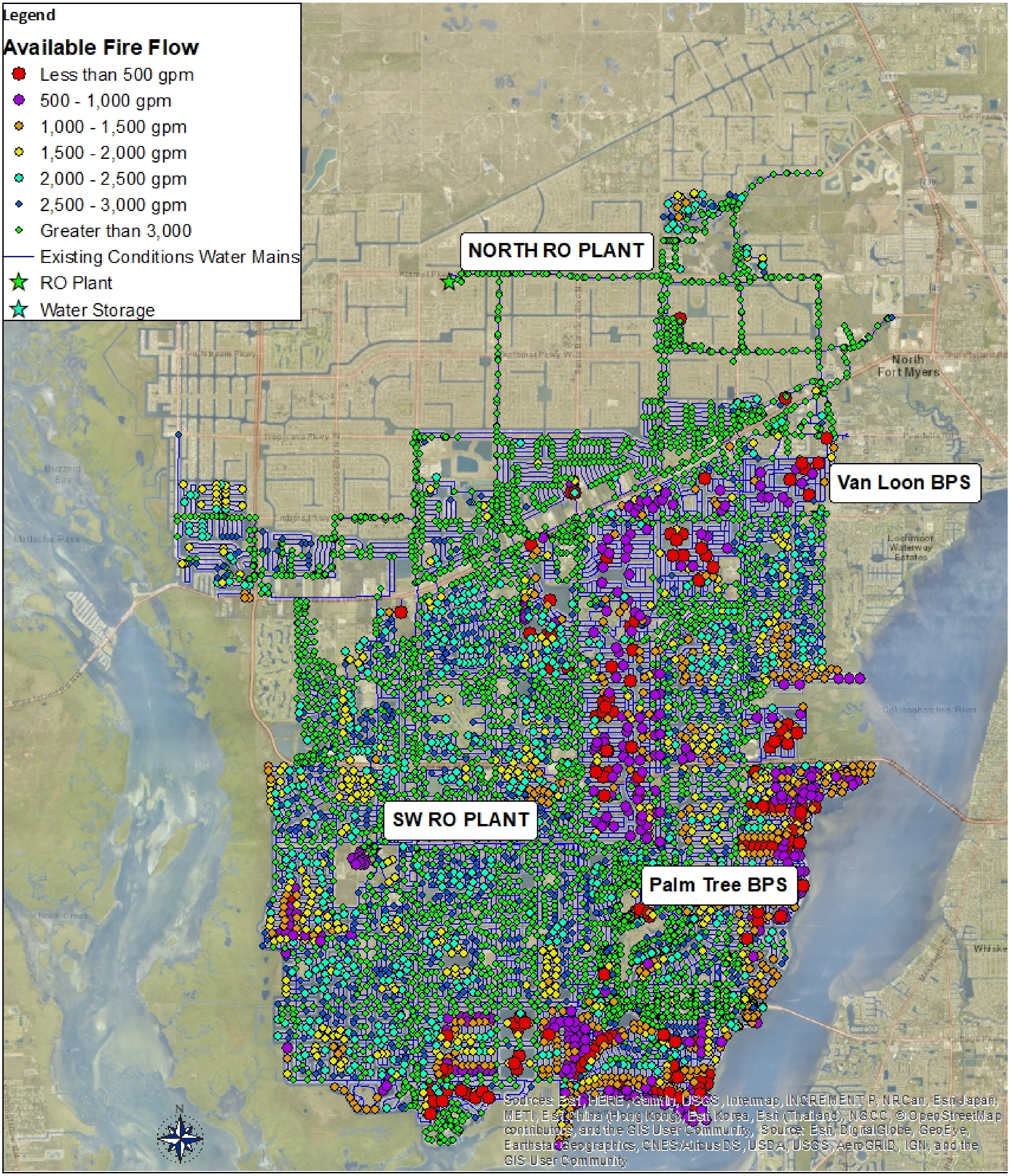


Figure 3-30
Existing Conditions MDD
Available Fire Flow

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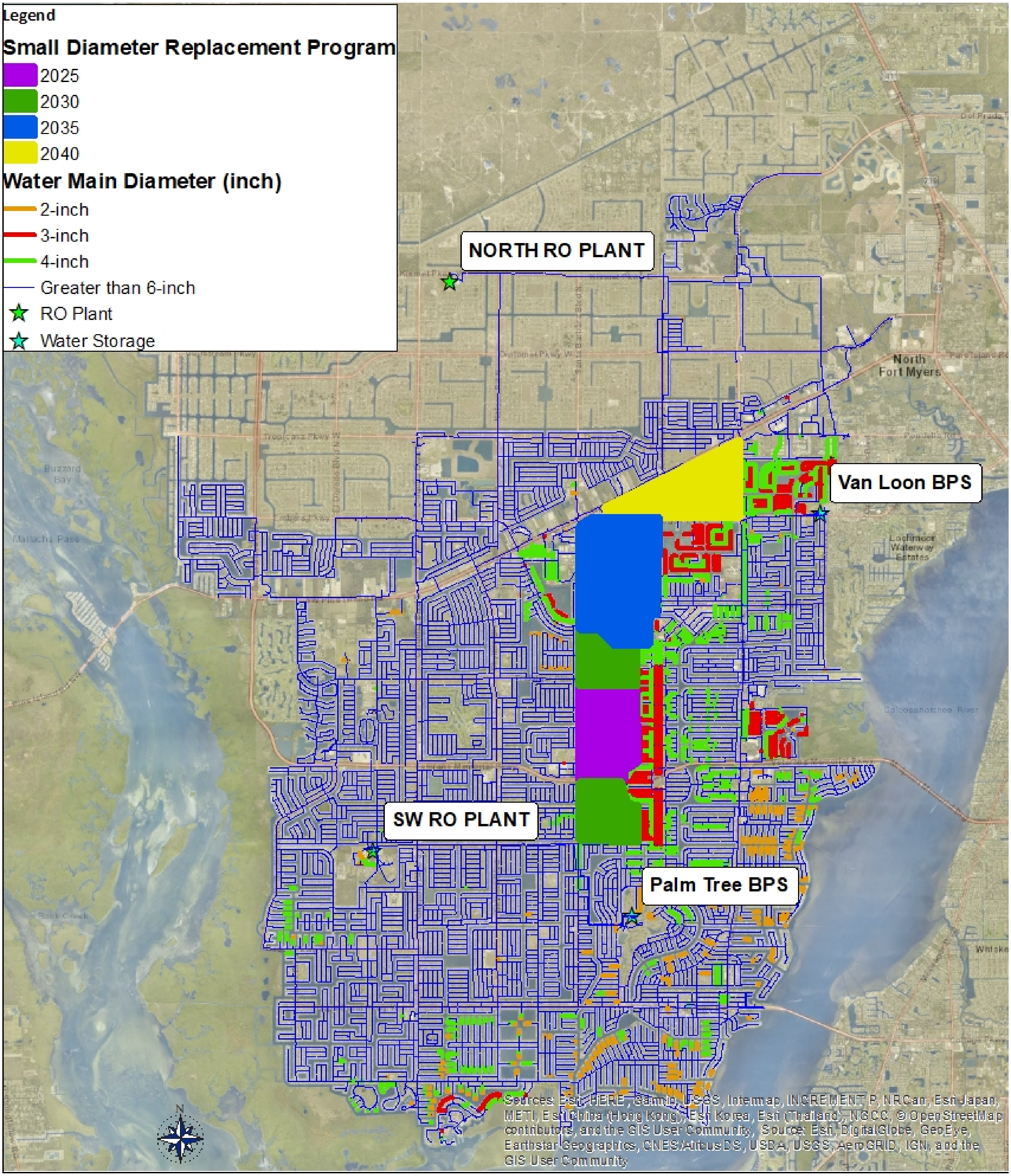

Figure 3-31
Delineated Areas with Water Mains Less than 6-Inches

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Planning Year 2025 Evaluation

Analyses for the 2025 planning year were performed to evaluate the water distribution system with the addition of new potable water service connections in the North 1 UEP and North 3 UEP service areas and multiple planned unit developments. **Figure 3-32** provides a map displaying the existing water distribution system, the North 1 UEP service area, North 3 UEP service area, and the planned unit developments (PUD) assumed to be connected to the water distribution system by 2025. The 2025 planning year also includes the following additional water main improvements:

- Water mains to serve the North 1 UEP service area as shown in **Figure 3-33.**
- Water mains to serve the North 3 UEP service area as shown in **Figure 3-34.**
- Water mains to serve new developments in the Pine Island Corridor between Chiquita and Burnt Store Road as shown in **Figure 3-35.**
- All the water mains shown in **Figure 3-36** with diameters less than 6 inches are assumed to be upsized to a 6-inch water main by 2025 as a part of the Small Diameter Replacement Program.
- New transmission mains required to serve the Burnt Store Road corridor to Hudson Creek PUD and to City limits as shown in **Figure 3-37.**

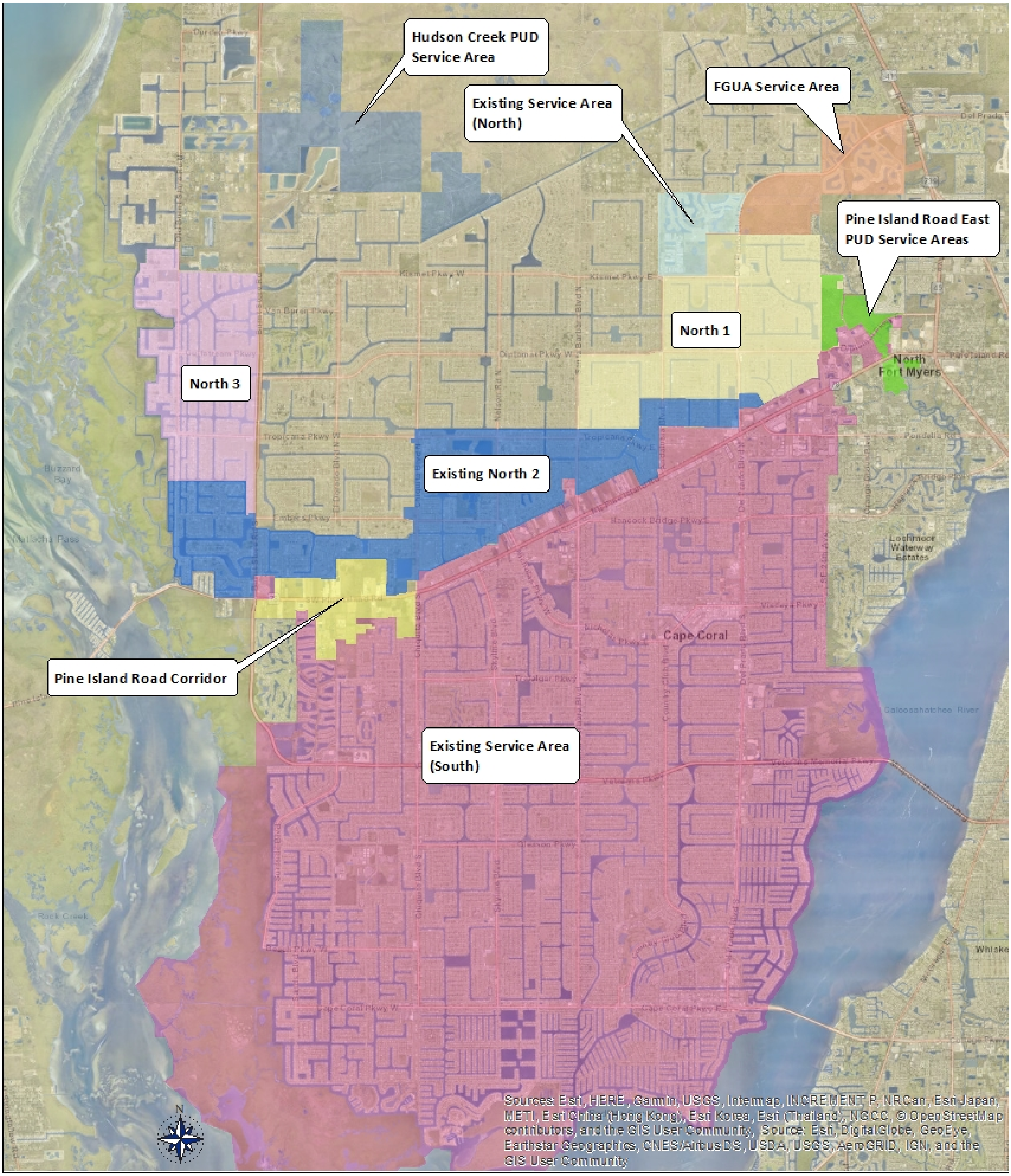

Figure 3-32
Planning Year 2025
Existing and Proposed Service Areas

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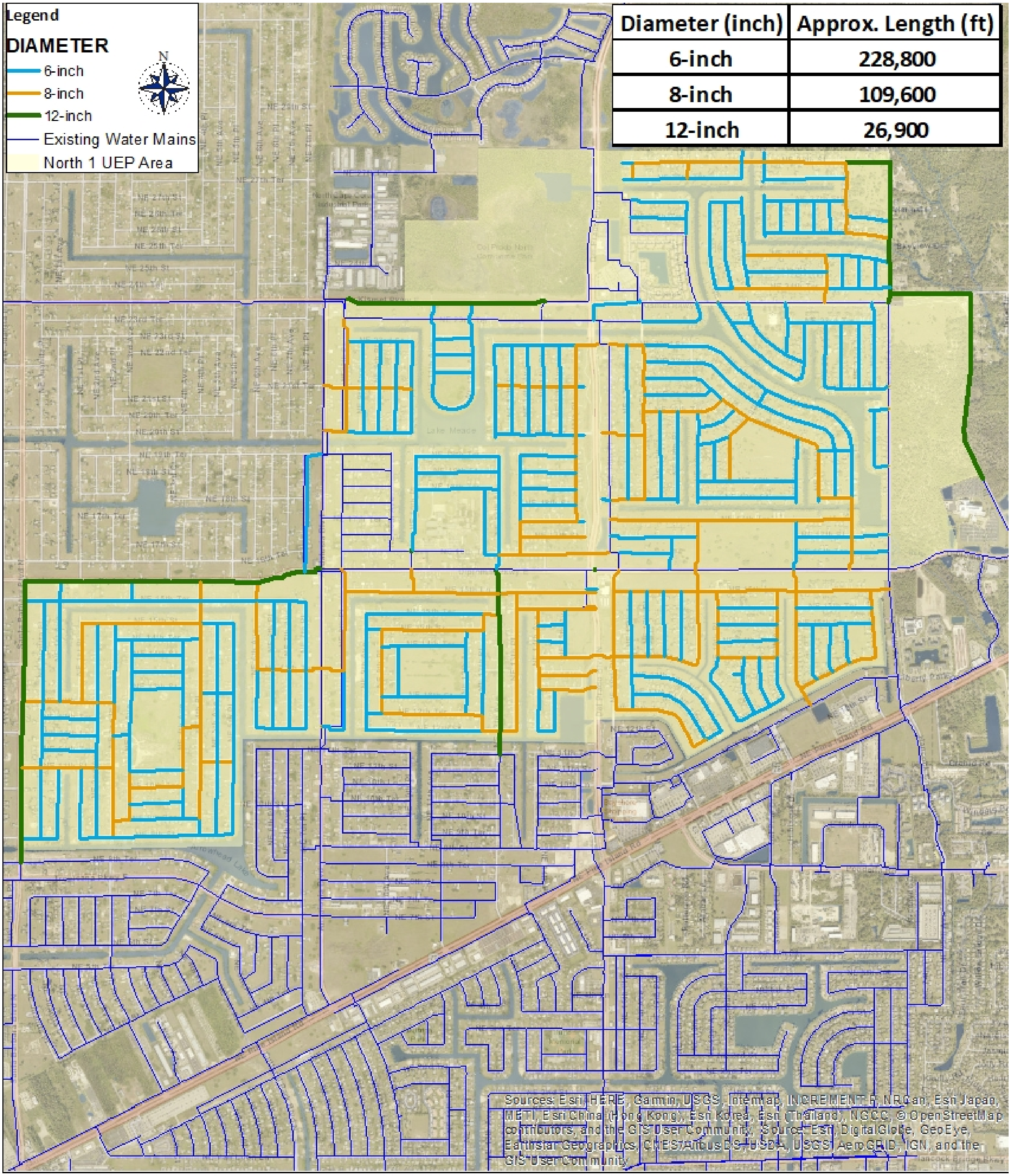

Diameter (inch)	Approx. Length (ft)
6-inch	228,800
8-inch	109,600
12-inch	26,900

1 in = 2,193 feet

Figure 3-33
Planning Year 2025
North 1 UEP Improvements

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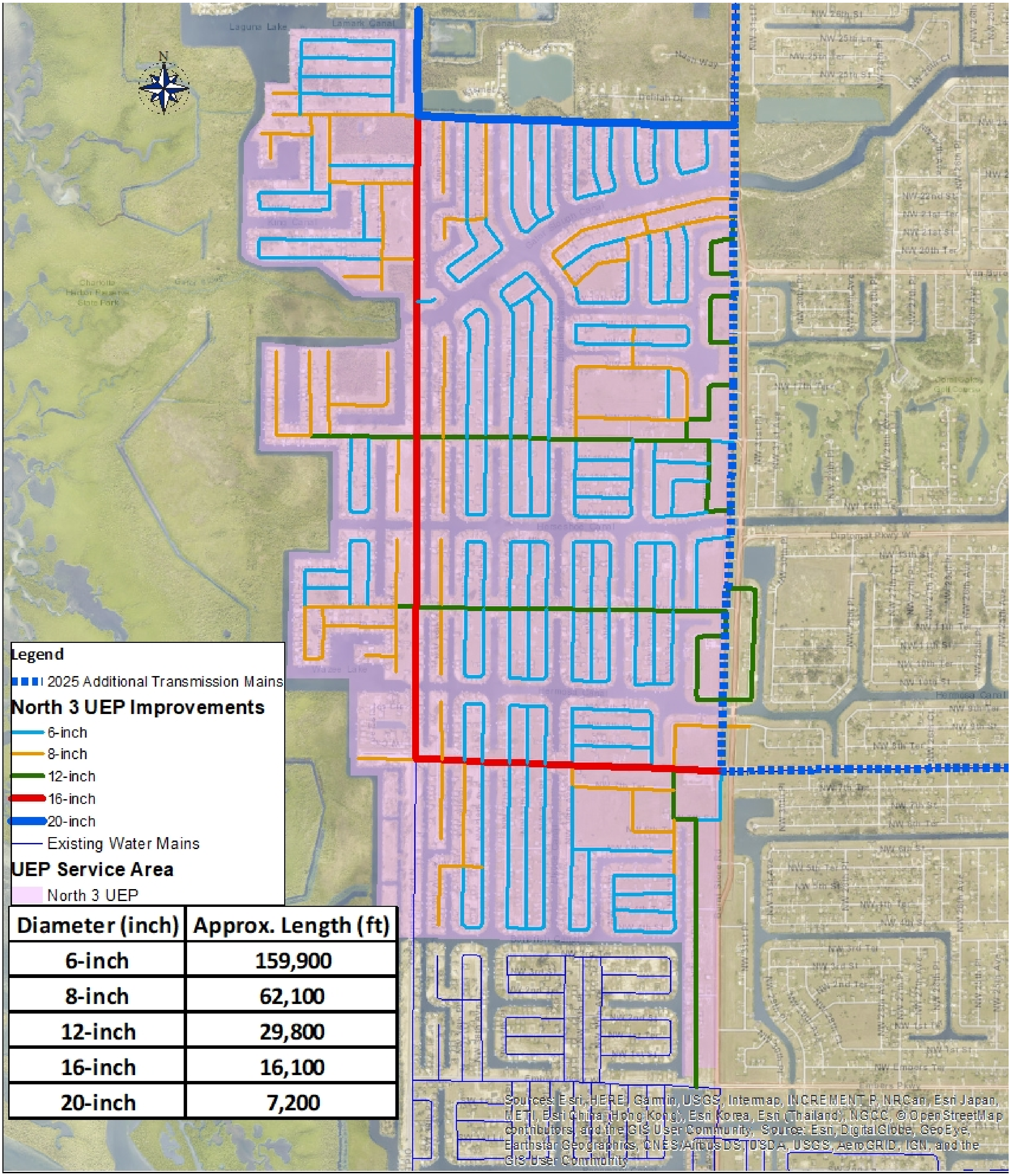

Diameter (inch)	Approx. Length (ft)
6-inch	159,900
8-inch	62,100
12-inch	29,800
16-inch	16,100
20-inch	7,200

1 in = 2,000 feet

Figure 3-34
Planning Year 2025
North 3 UEP Improvements

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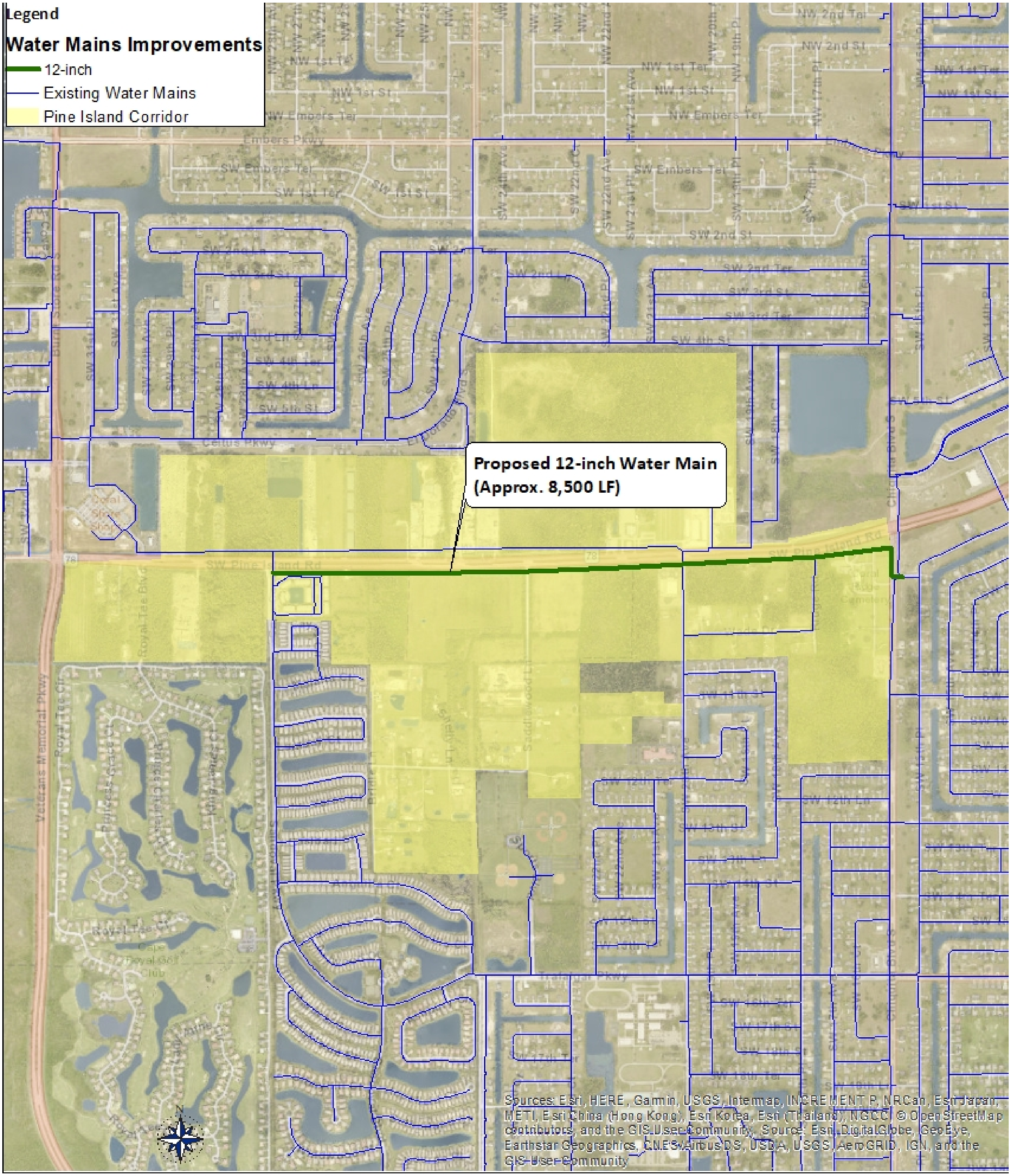

Figure 3-35
Pine Island Corridor
Water Main Improvements

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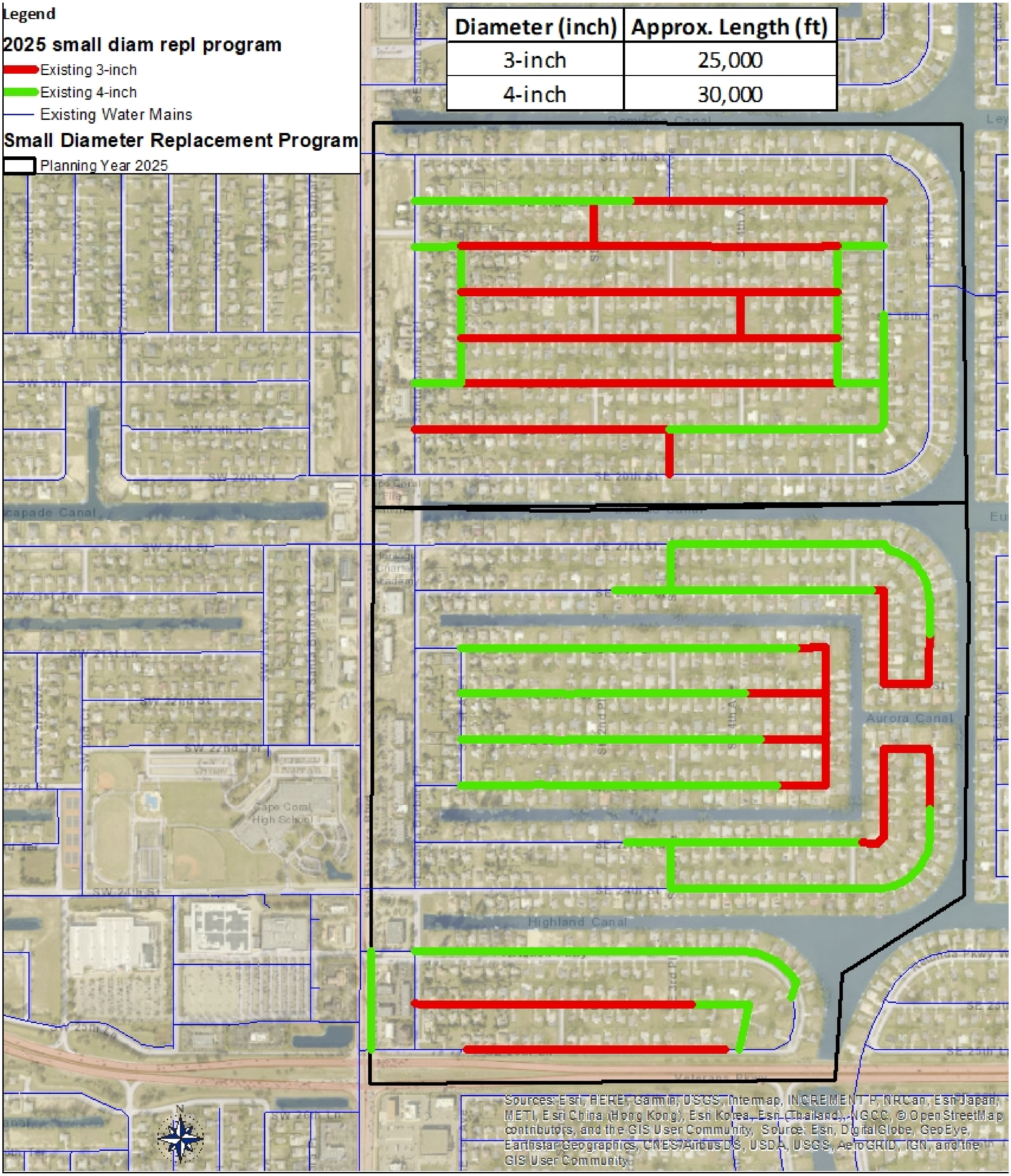

Diameter (inch)	Approx. Length (ft)
3-inch	25,000
4-inch	30,000

Figure 3-36
Planning Year 2025
Small Diameter Replacement Program

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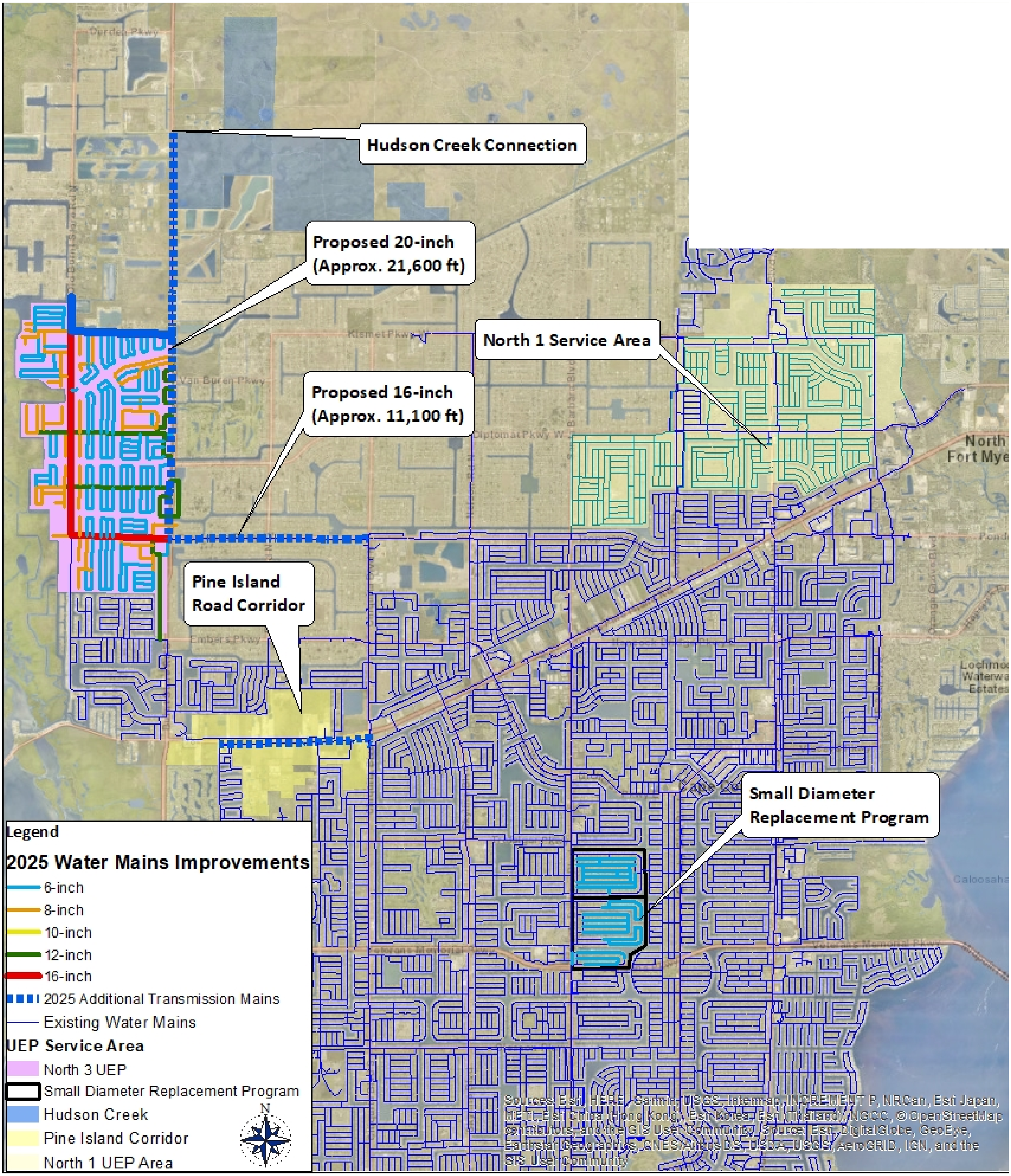

Figure 3-37

Planning Year 2025

Burnt Store Road Corridor
Hudson Creek PUD
Water Main Improvements

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There are three deficiencies identified as a result of the 2025 modeling that require upgrades. There are some areas where a fire flow of 1000 gpm could not be provided under a MDD condition. As noted in **Figure 3-38**, most of these areas with fire flow deficiencies are in older parts of the water distribution system with water main diameters less than 6-inches and the City has plans to upgrade these mains with the Small Diameter Pipe Replacement Program. **Figure 3-39** shows the model predictions for available fire flow in the area after replacement of water mains less than 6-inches as a part of the 2025 Small Diameter Replacement Program Area.

Also, upgrades to the high service pumps at both the North RO WTP and the SW RO WTP require upgrades (additional 7500 gpm and 2500 gpm high service pumps respectively).

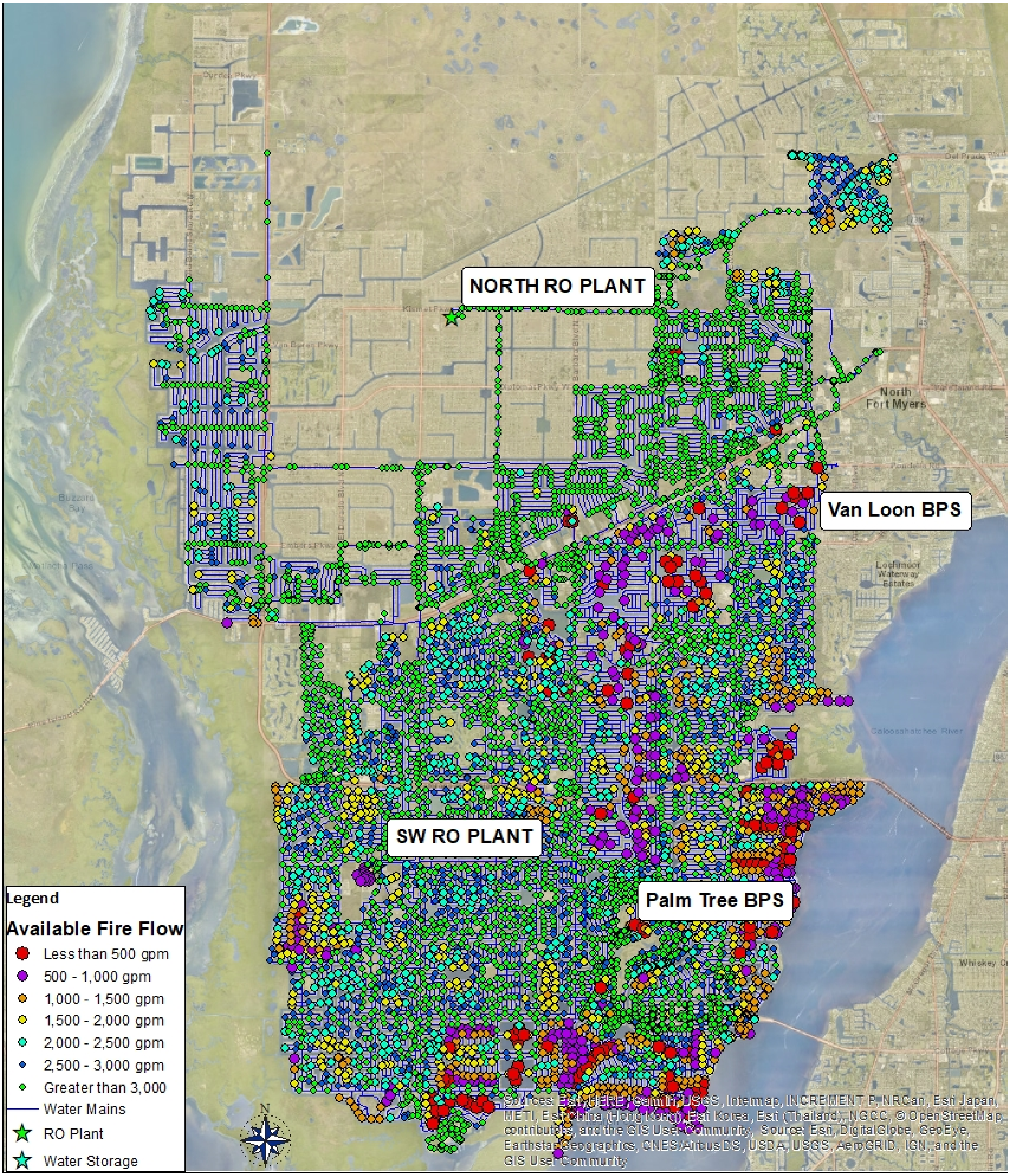

Figure 3-38
Planning Year 2025 MDD
Available Fire Flow

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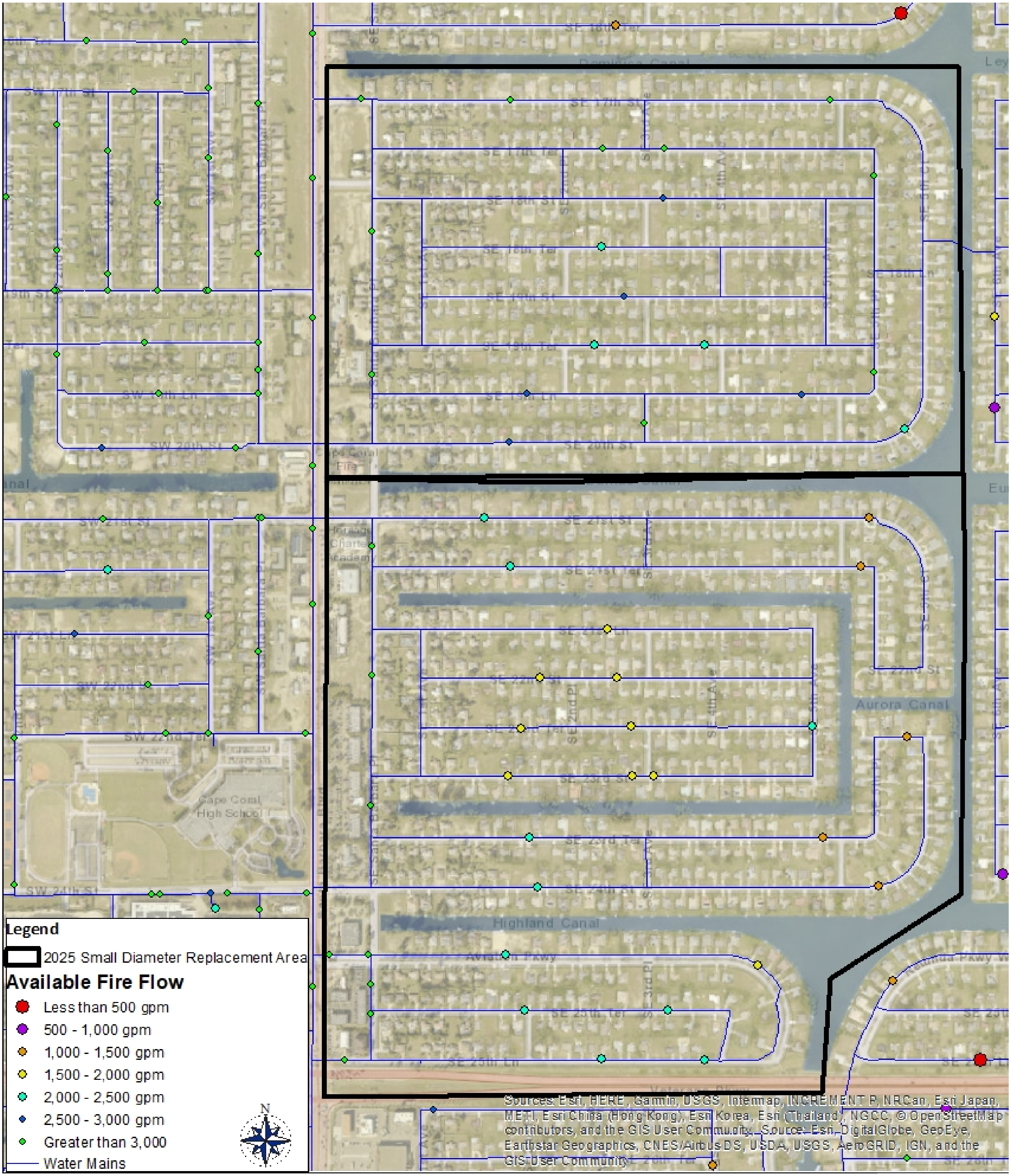

Figure 3-39
2025 Small Diameter Replacement Area
Available Fire Flow

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Table 3-19 summarizes the recommended water main improvements for the 2025 planning year. This includes the recommended water main sizes and the total length for each size.

Table 3-19: Summary of the Recommended UEP Improvement Projects for Planning Year 2025

Service Area	Diameter (inch)	Length (ft)
North 1 UEP Area	6	228,800
	8	109,600
	12	26,900
North 3 UEP Area	6	159,900
	8	62,100
	12	29,800
	16	16,100
	20	7,200
Additional Transmission Main (Burnt Store Road)	20	21,600
Additional Transmission Main (Tropicana Parkway)	16	11,100
Additional Transmission Main (Pine Island Corridor)	12	8,500
2025 Small Diameter Replacement Program	6	53,800
High Service 7,500 gpm Pump at the North RO WTP	-	-
High Service 2,500 gpm Pump at the Southwest RO WTP	-	-

Planning Year 2030 Evaluation

The 2030 analyses incorporated the addition of new potable water service connections in the North 4 UEP and North 5 UEP service areas, as well as growth in the existing and 2025 areas. **Figure 3-40** provides a map displaying the 2025 planned service area along with the North 4 and North 5 UEP areas anticipated for 2030. The 2030 planning year also includes the following additional water transmission main improvements:

- Water mains to serve the North 4 UEP service area as shown in **Figure 3-41**.
- Water mains to serve the North 5 UEP service area as shown in **Figure 3-42**.
- Water transmission mains to supply the Hudson Creek PUD from the east while maintaining the connection on the west side of the PUD as shown in **Figure 3-42**.
- All the water mains shown in **Figure 3-43** with diameters less than 6-inch are assumed to be upsized to a 6-inch water main by 2030 with the Small Pipe Diameter Replacement Program.
- Construction of the east storage tank and booster pump station for reliability by 2030.

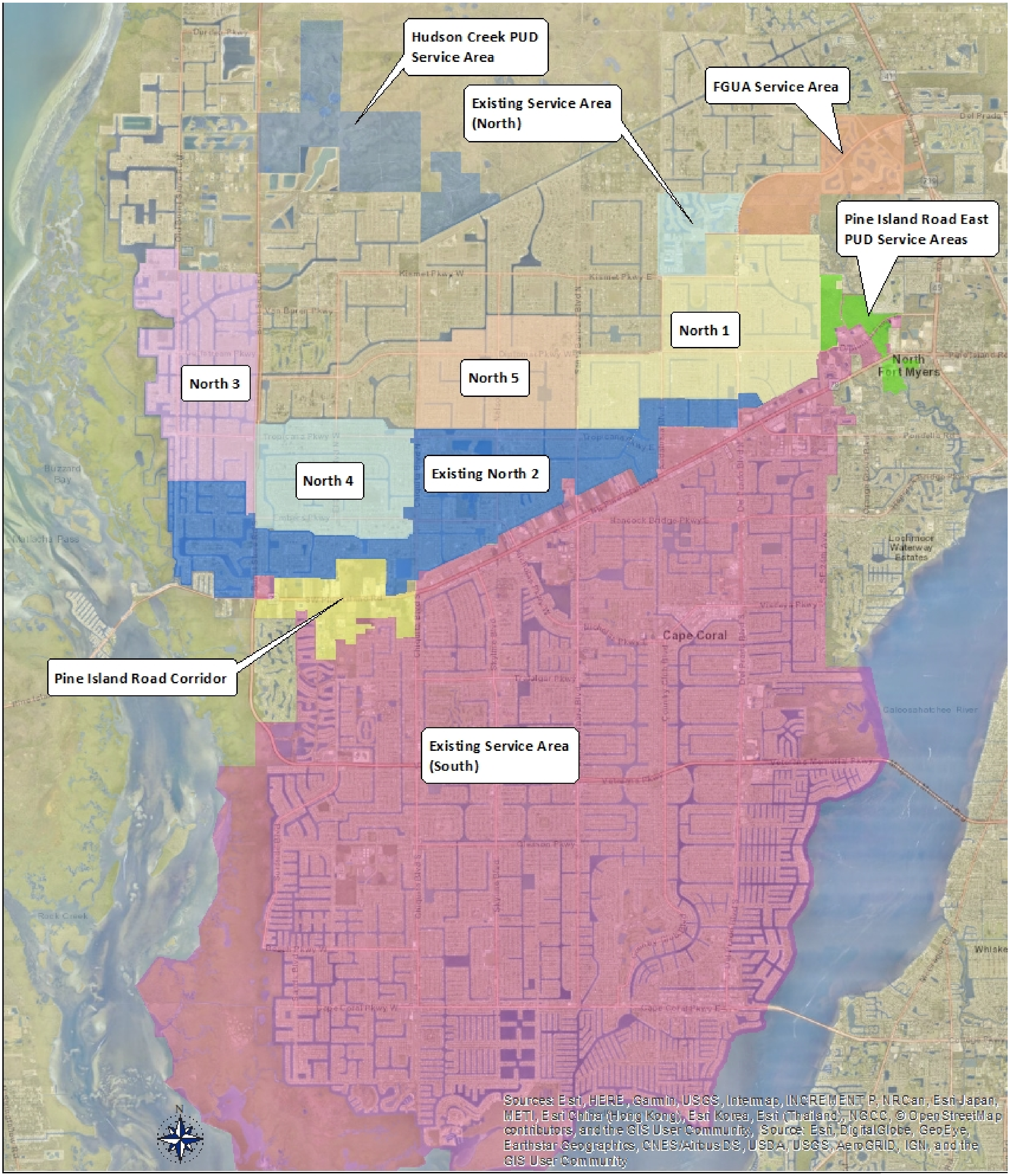

Figure 3-40
Planning Year 2030
Existing and Proposed Service Areas

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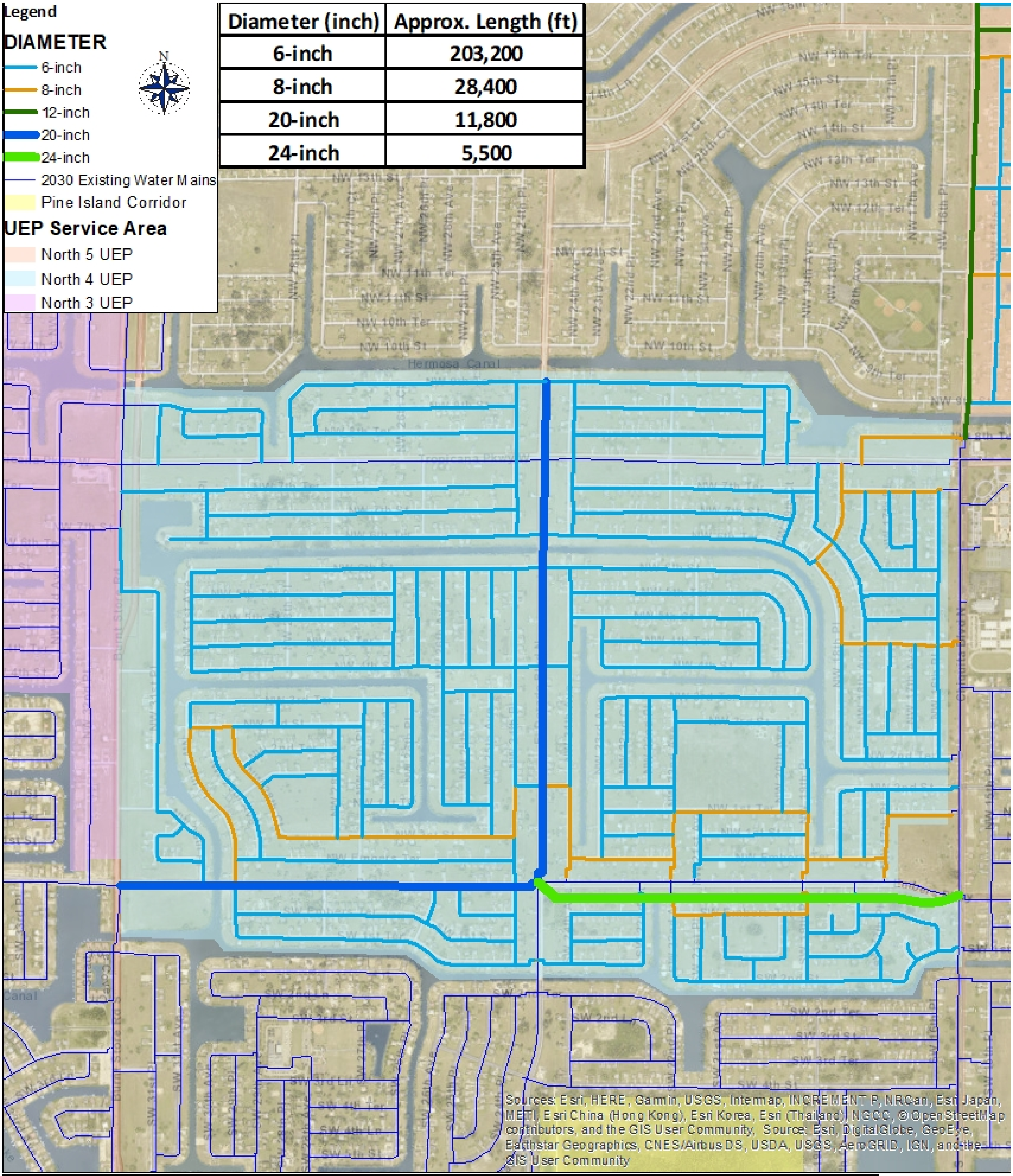

Diameter (inch)	Approx. Length (ft)
6-inch	203,200
8-inch	28,400
20-inch	11,800
24-inch	5,500

1 in = 1,509 feet

Figure 3-41
Planning Year 2030
North 4 UEP Improvements

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Diameter (inch)	Approx. Length (ft)
6-inch	197,000
8-inch	34,500
12-inch	19,100

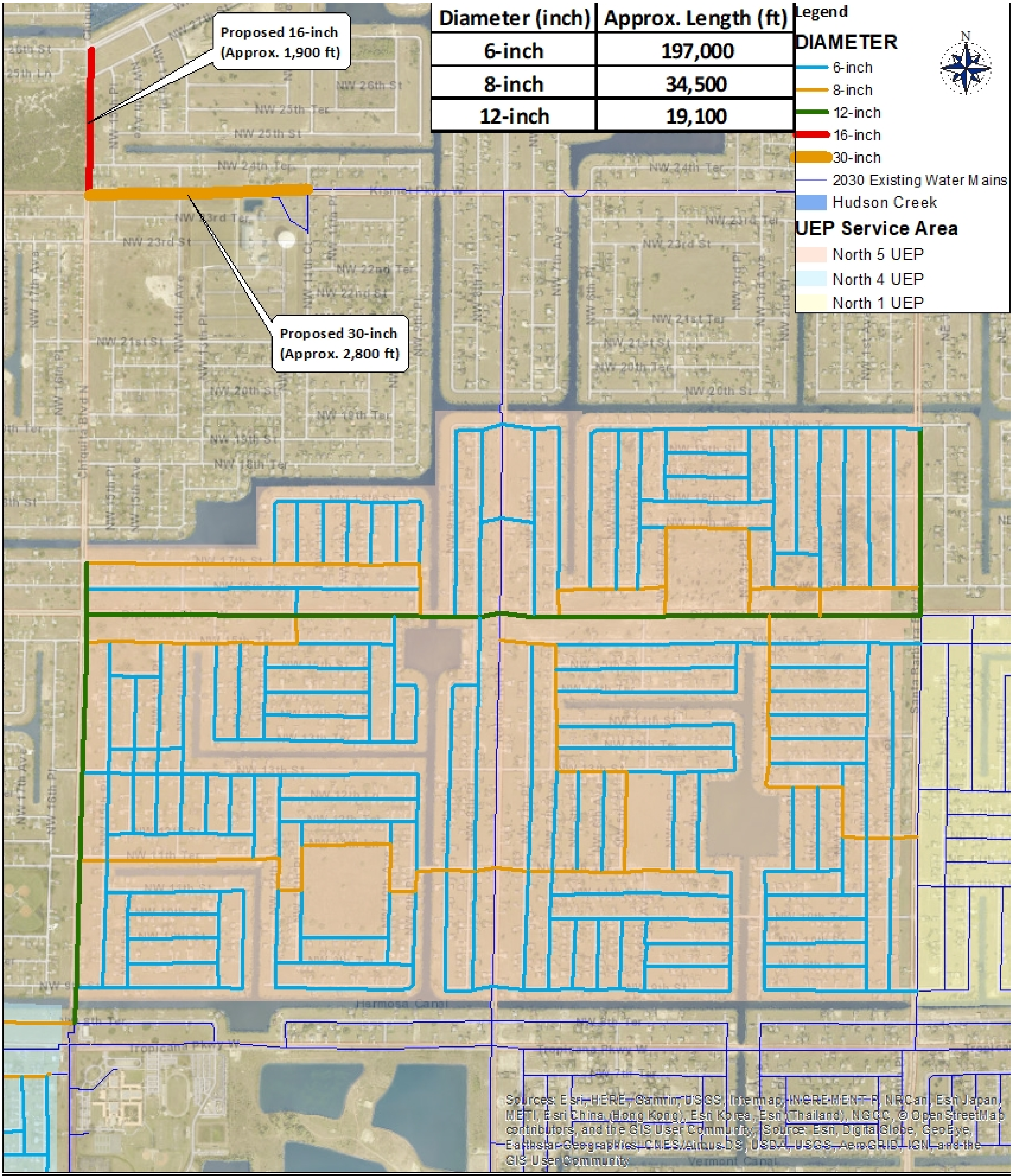

Figure 3-42
Planning Year 2030
North 5 UEP Improvements

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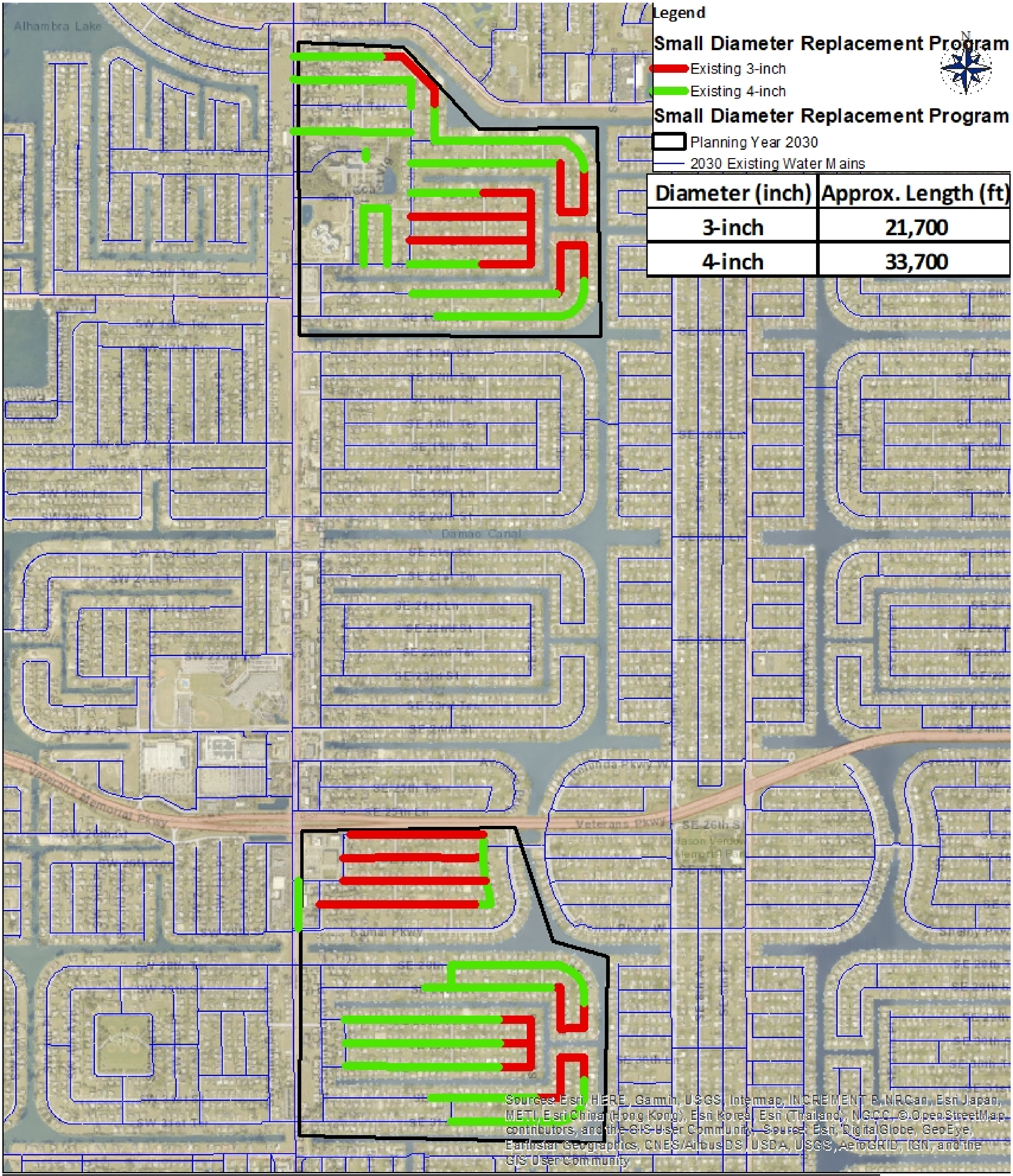

Diameter (inch)	Approx. Length (ft)
3-inch	21,700
4-inch	33,700

Figure 3-43

Planning Year 2030

Small Diameter Replacement Program

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Available fire flow was the most significant deficiency identified as a result of the 2030 Modeling. There are some areas where a fire flow of 1000 gpm could not be provided under a MDD condition. As noted in **Figure 3-44**, most of these areas with fire flow deficiencies are in older parts of the water distribution system with water main diameters less than 6-inches and the City has plans to upgrade these mains with the Small Diameter Pipe Replacement Program. **Figure 3-45** shows the model predictions for available fire flow in the area after replacement of water mains less than 6-inches as a part of the 2030 Small Diameter Replacement Program Area.

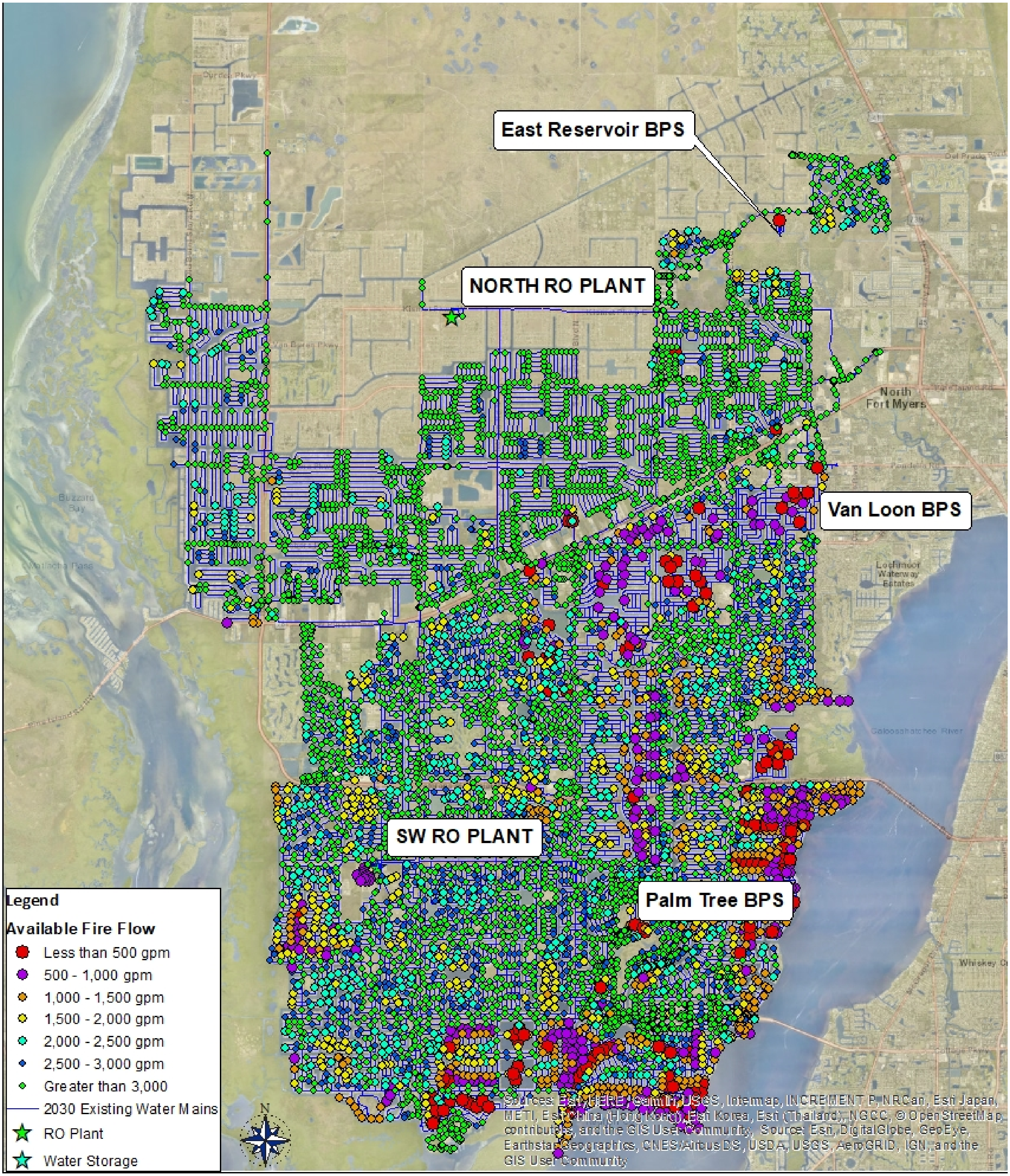

Figure 3-44
Planning Year 2030 MDD
Available Fire Flow

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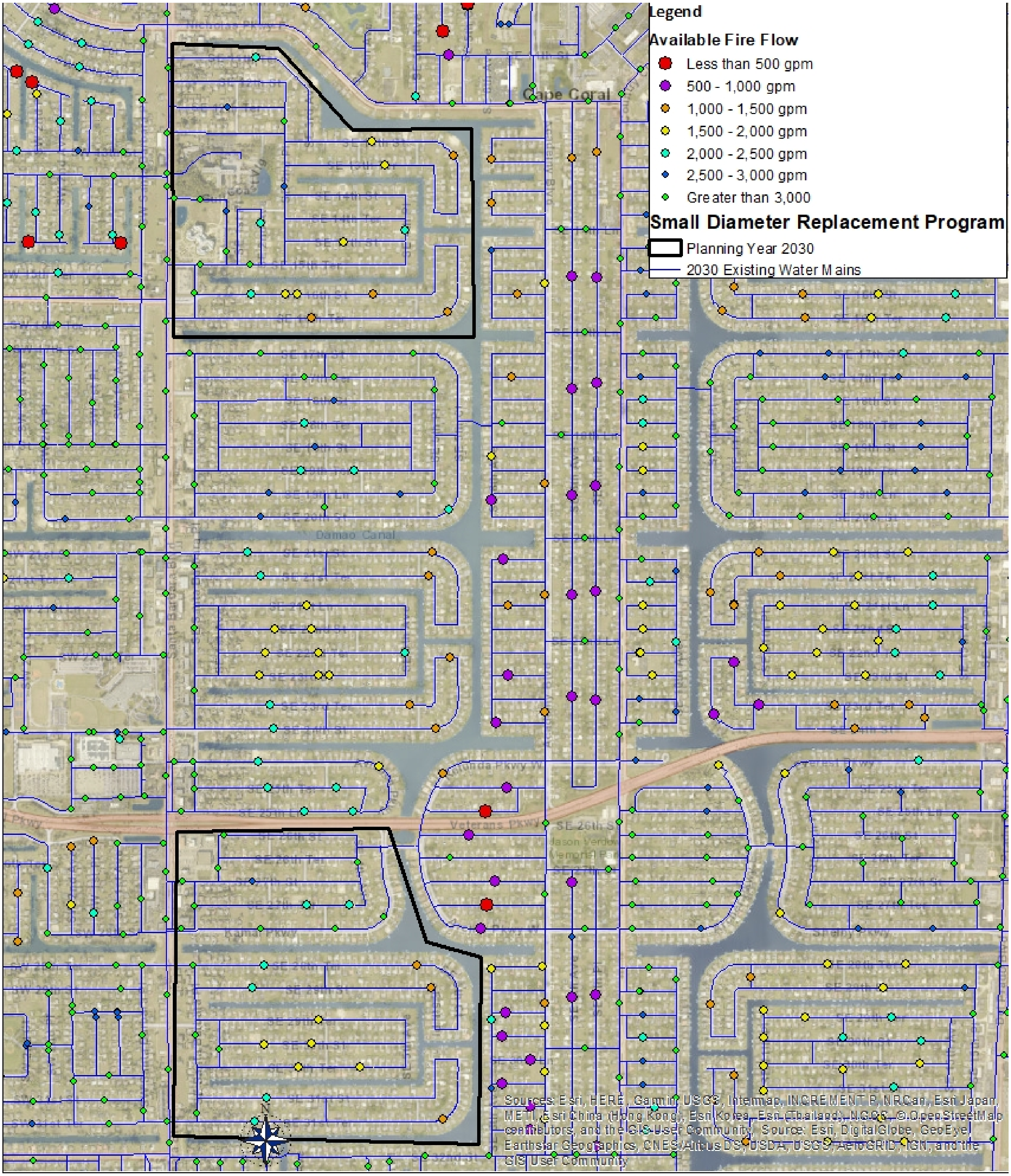

Figure 3-45
2030 Small Diameter Replacement Area
Available Fire Flow

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Table 3-20 summarizes the recommended water main improvements for the 2030 planning year. This includes the recommended water main sizes and the total length for each size.

Table 3-20: Summary of the UEP Improvement Projects for Planning Year 2030

Service Area	Diameter (inch)	Length (ft)
North 4 UEP Area	6	203,200
	8	28,400
	20	11,800
	24	5,500
North 5 UEP Area	6	197,000
	8	34,500
	12	19,100
East Reservoir Ground Storage Tank and Pump Station	-	-
East Reservoir Transmission Main	20	3,000
Additional Transmission Main (Chiquita Boulevard)*	16	1,900
Additional Transmission Main (Kismet Parkway)*	30	2,800
2030 Small Diameter Replacement Program	6	55,400

*Time frame for improvements were advanced due to developers' aggressive implementation schedule. These improvements are now planned for Year 2027.

Planning Year 2035 Evaluation

The year 2035 analyses build on the systemwide 2030 analyses with the addition of new potable water service connections in the North 6 UEP and North 7 UEP service areas. **Figure 3-46** provides a map displaying the 2030 planned service area along with the North 6 and North 7 UEP areas anticipated for 2035. The 2035 planning year improvements include:

- Water mains to serve the North 6 UEP service area as shown in **Figure 3-47**.
- Water mains to serve the North 7 UEP service area as shown in **Figure 3-48**.
- All the water mains shown in **Figure 3-49** with diameters less than 6-inch are assumed to be upsized to a 6-inch water main by 2035.

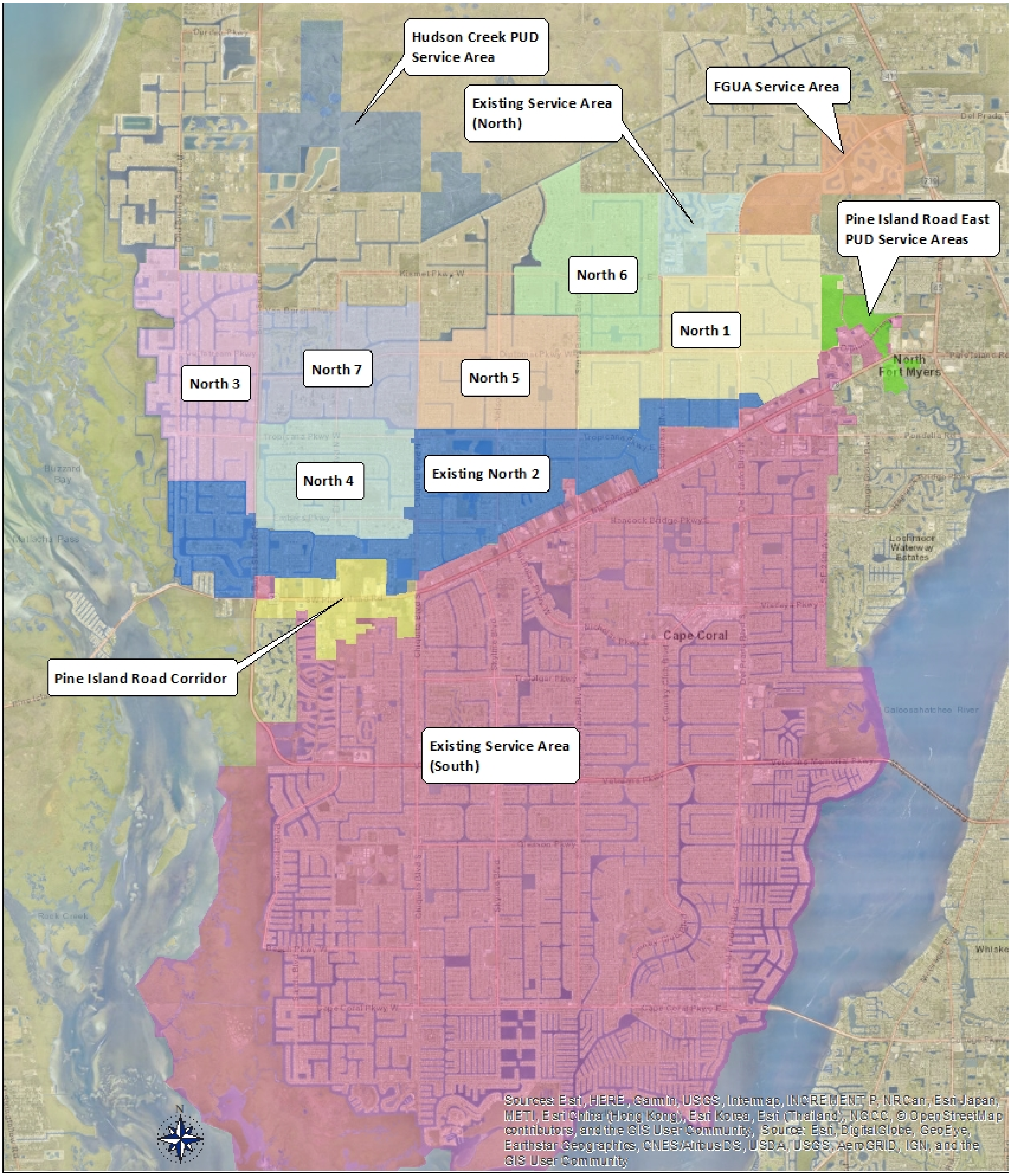

Figure 3-46
Planning Year 2035
Existing and Proposed Service Areas

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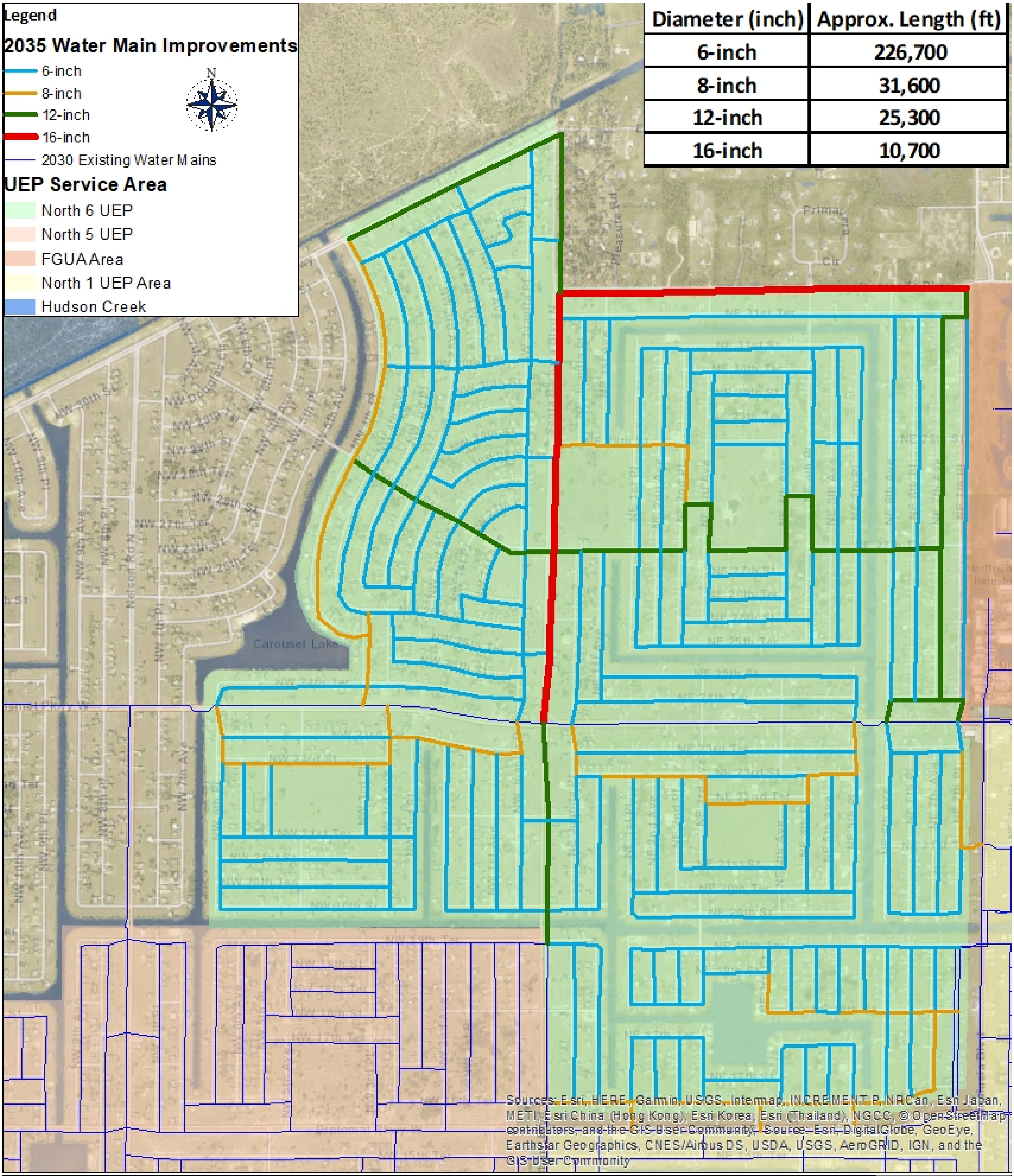

Diameter (inch)	Approx. Length (ft)
6-inch	226,700
8-inch	31,600
12-inch	25,300
16-inch	10,700

Figure 3-47
Planning Year 2035
North 6 UEP Improvements

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Diameter (inch)	Approx. Length (ft)
6-inch	171,900
8-inch	31,900
12-inch	19,400
20-inch	8,000

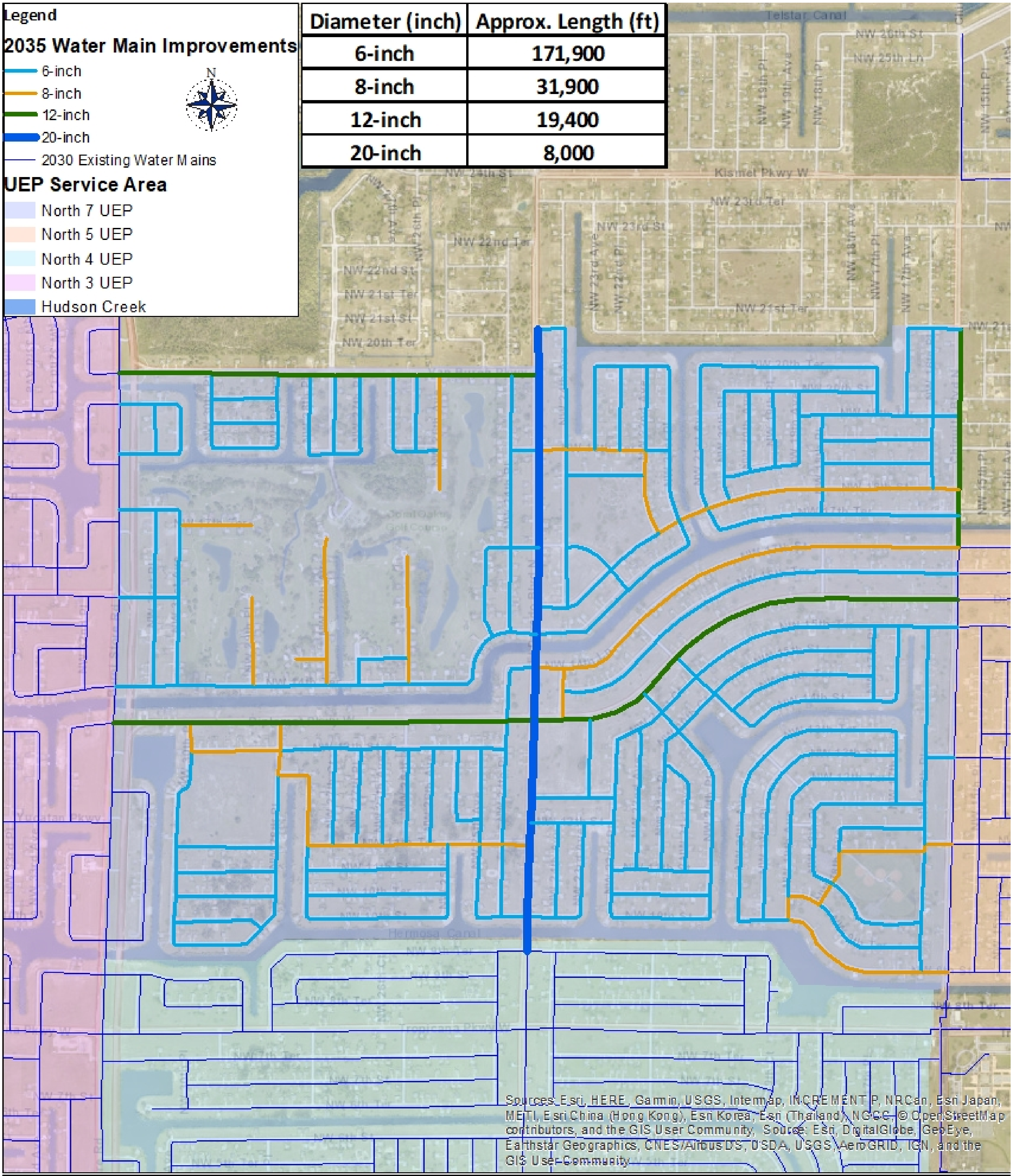

Figure 3-48
Planning Year 2035
North 7 UEP Improvements

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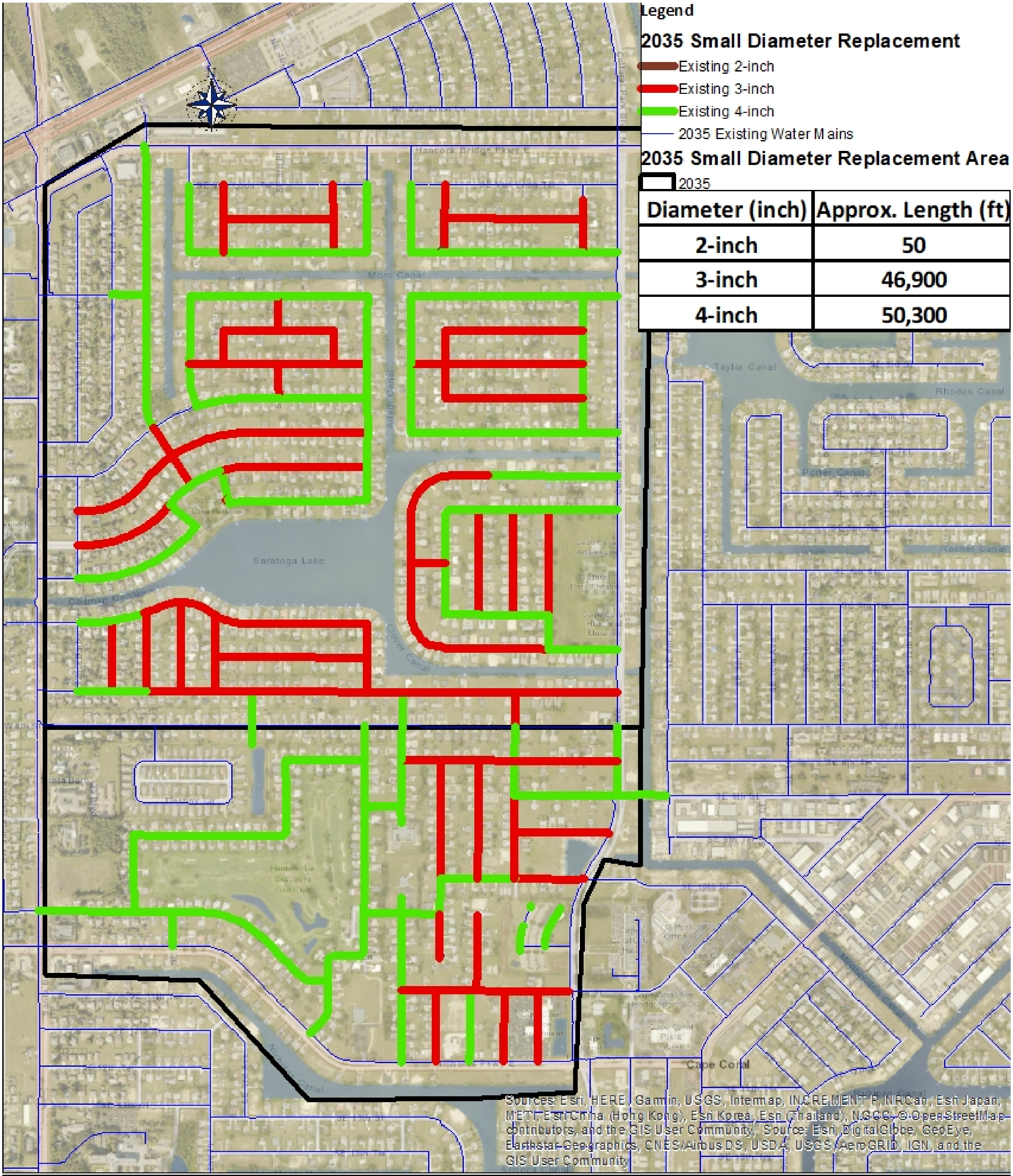

Diameter (inch)	Approx. Length (ft)
2-inch	50
3-inch	46,900
4-inch	50,300

Figure 3-49
Planning Year 2035
Small Diameter Replacement Program

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Two deficiencies were identified as a result of the 2035 Modeling. There are some areas where a fire flow of 1000 gpm could not be provided under a MDD condition. As noted in **Figure 3-50**, most of these areas with fire flow deficiencies are in older parts of the water distribution system with water main diameters less than 6-inches and the City has plans to upgrade these mains with the 2035 Small Diameter Pipe Replacement Program. **Figure 3-51** shows the model predictions for available fire flow in the area after replacement of water mains less than 6-inches as a part of the 2035 Small Diameter Replacement Program Area. In addition, a third 7,500 gpm pump is recommended to be installed at the SW RO WTP to increase the firm capacity to 32.4 MGD.

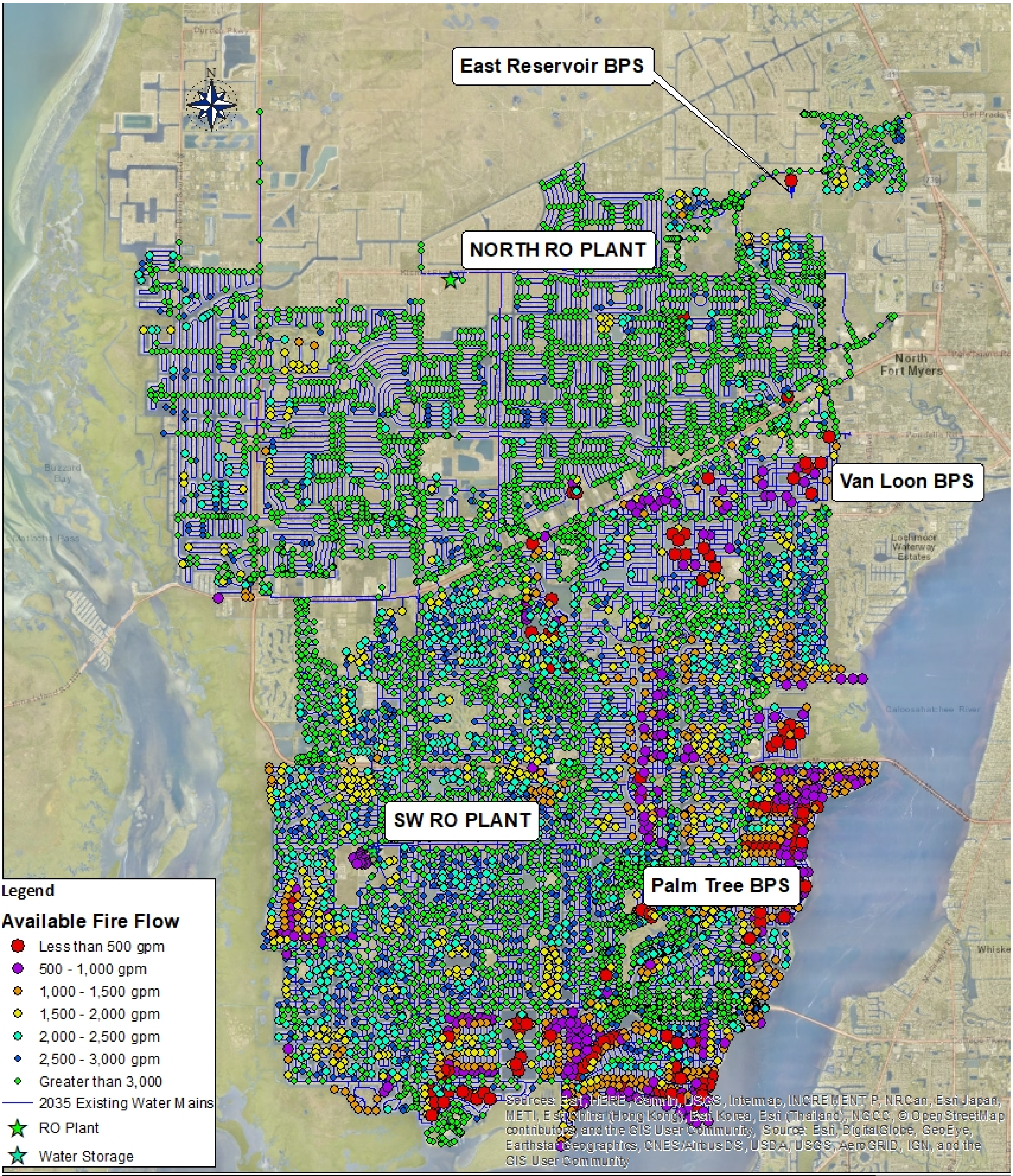

Figure 3-50
Planning Year 2035 MDD
Available Fire Flow

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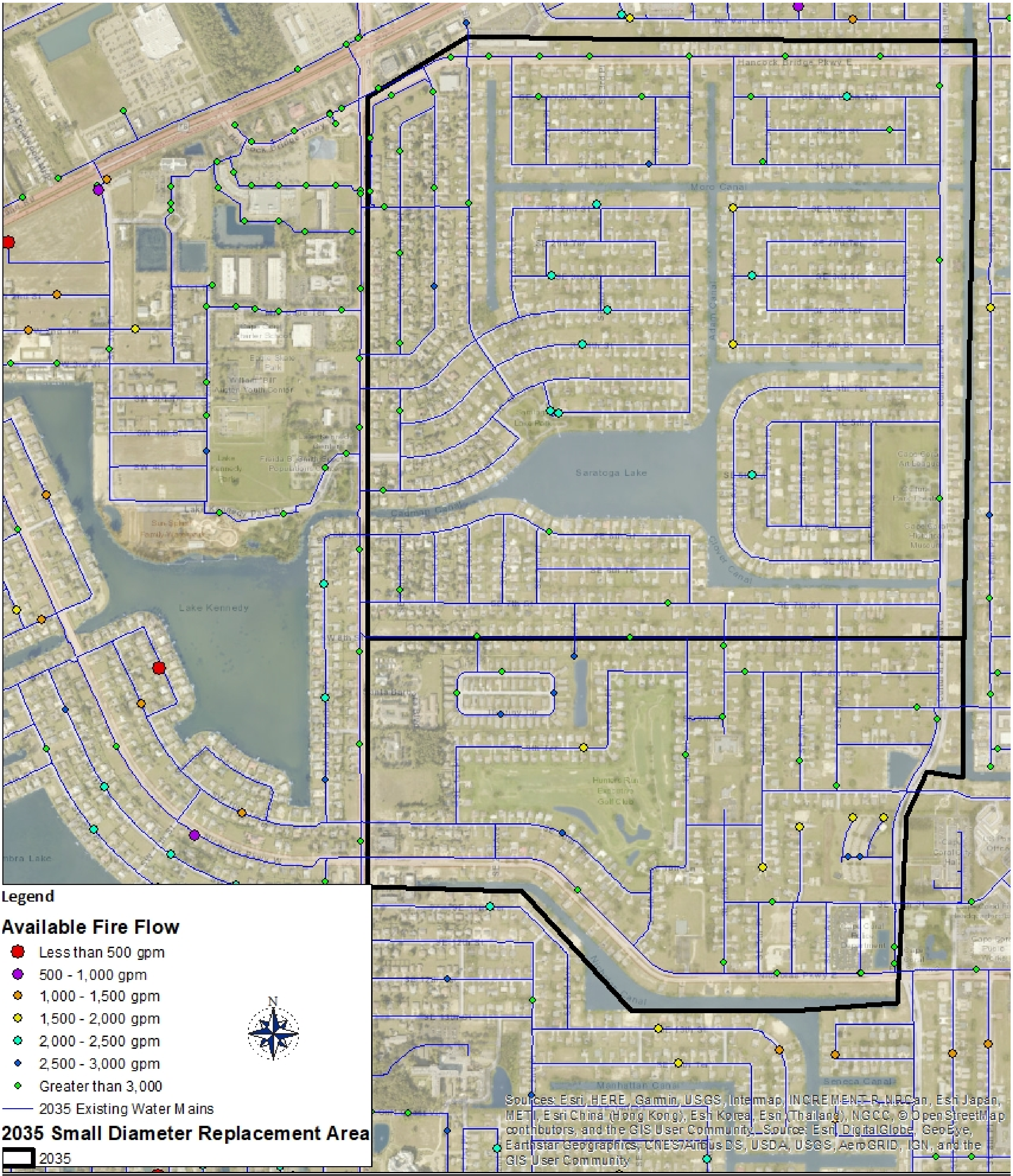

Figure 3-51
2035 Small Diameter Replacement Area
Available Fire Flow

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Table 3-21 summarizes the recommended water main improvements for the 2035 planning year. This includes the recommended water main sizes and the total length for each size.

Table 3-21: Summary of the Recommended UEP Improvement Projects for Planning Year 2035

Service Area	Diameter (inch)	Length (ft)
North 6 UEP Area	6	226,700
	8	31,600
	12	25,300
	16	10,700
North 7 UEP Area	6	171,900
	8	31,900
	12	19,400
	20	8,000
Entrada (Upgrades Existing Water Mains)	6 - 16	36,800
2035 Small Diameter Replacement Program	6	97,200
High Service 7,500 gpm Pump at Southwest RO WTP	-	-

Planning Year 2040 Evaluation

The 2040 planning year analyses build on prior years with the addition of new potable water service connections in the North 8 UEP and North 9 UEP service areas. **Figure 3-52** provides a map displaying the 2035 planned service area along with the North 8 and North 9 UEP areas anticipated for 2040. The 2040 planning year also includes the following additional water transmission main improvements:

- Water mains to serve the North 8 UEP service area as shown in **Figure 3-53.**
- Water mains to serve the North 9 UEP service area as shown in **Figure 3-54**.
- All the water mains shown in **Figure 3-55** with diameters less than 6-inch are assumed to be upsized to a 6-inch water main by 2040.

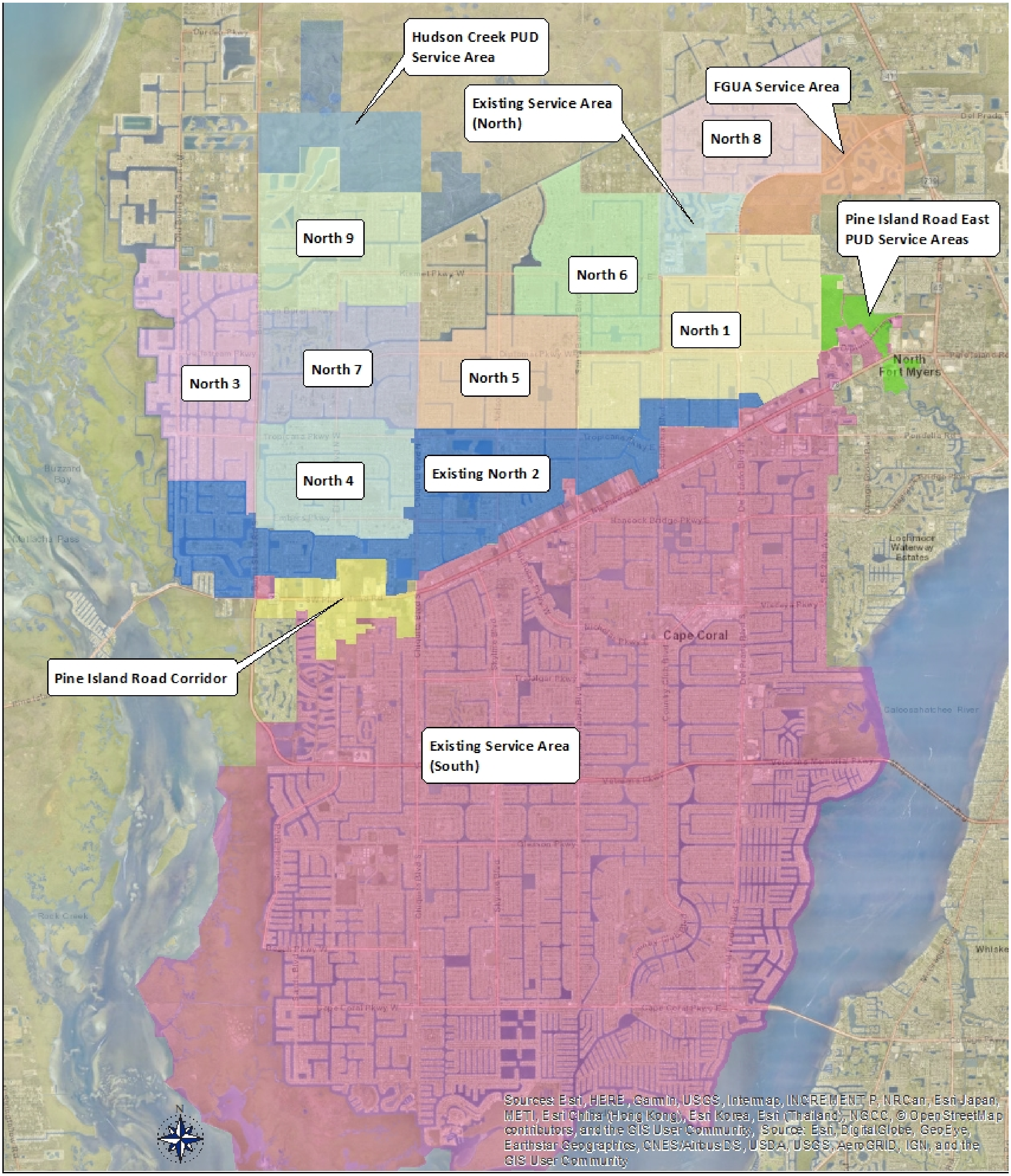

Figure 3-52
Planning Year 2040
Existing and Proposed Service Areas

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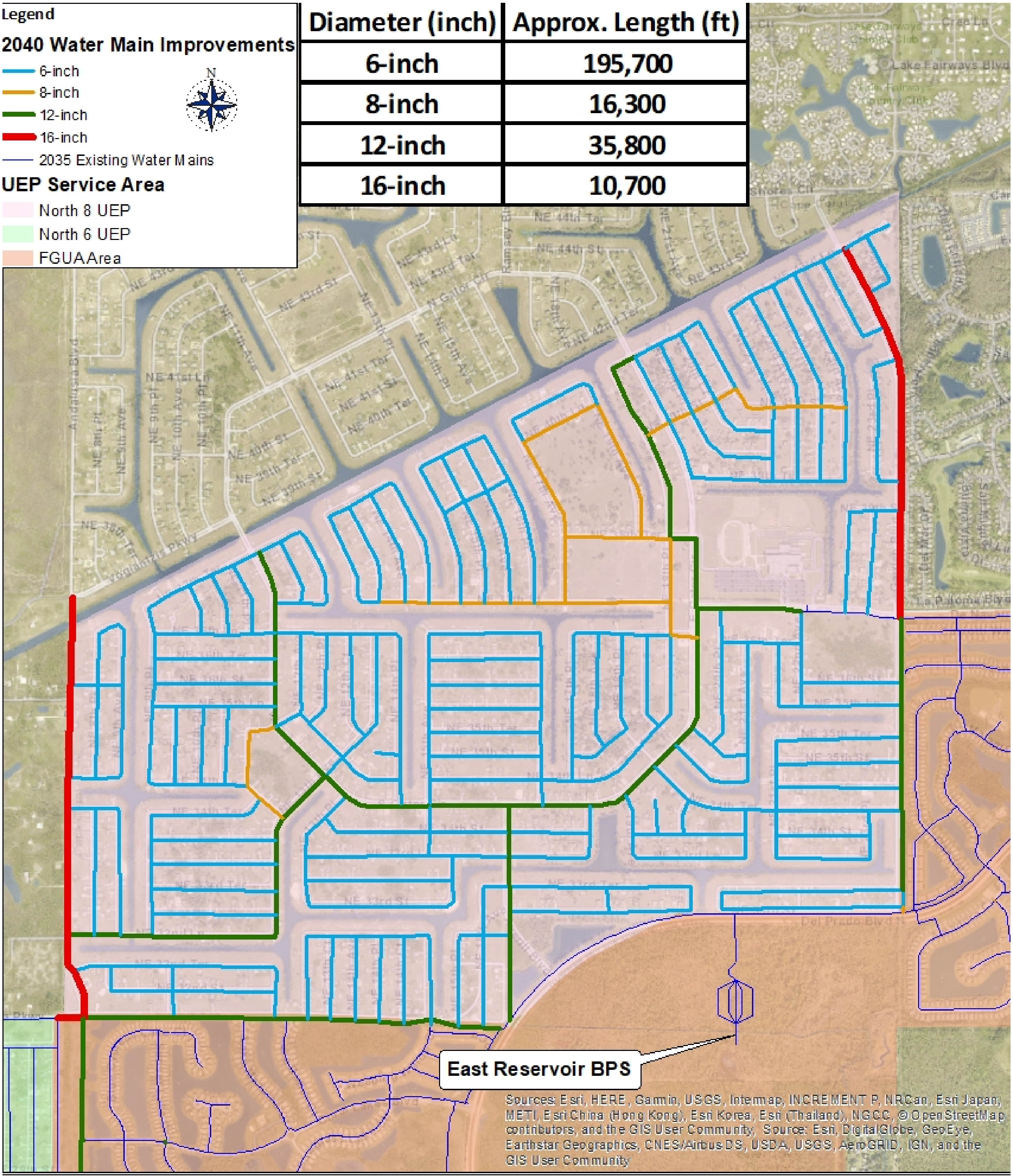

Diameter (inch)	Approx. Length (ft)
6-inch	195,700
8-inch	16,300
12-inch	35,800
16-inch	10,700

Figure 3-53
Planning Year 2040
North 8 UEP Improvements

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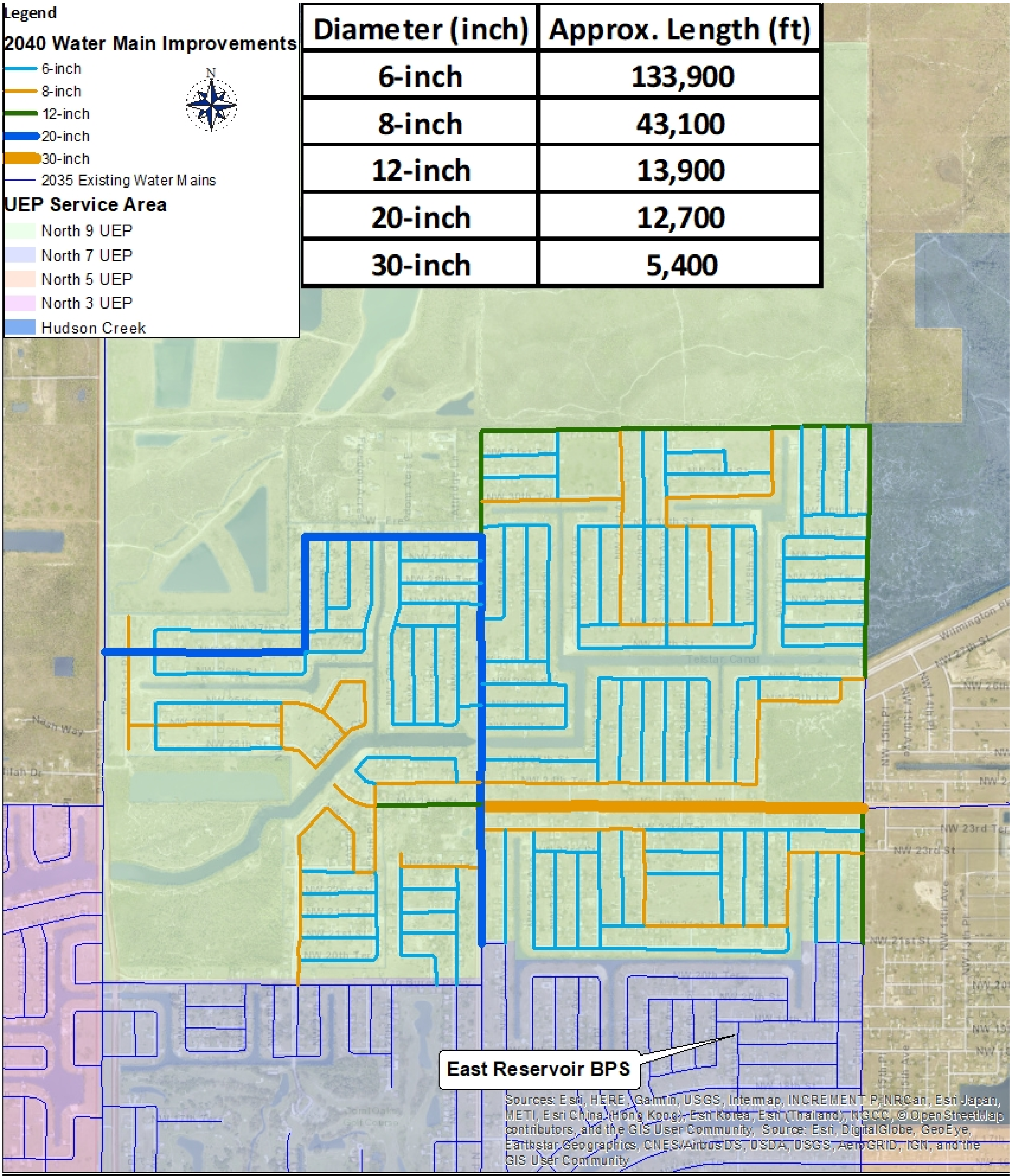

Diameter (inch)	Approx. Length (ft)
6-inch	133,900
8-inch	43,100
12-inch	13,900
20-inch	12,700
30-inch	5,400

Figure 3-54
Planning Year 2040
North 9 UEP Improvements

City of Cape Coral
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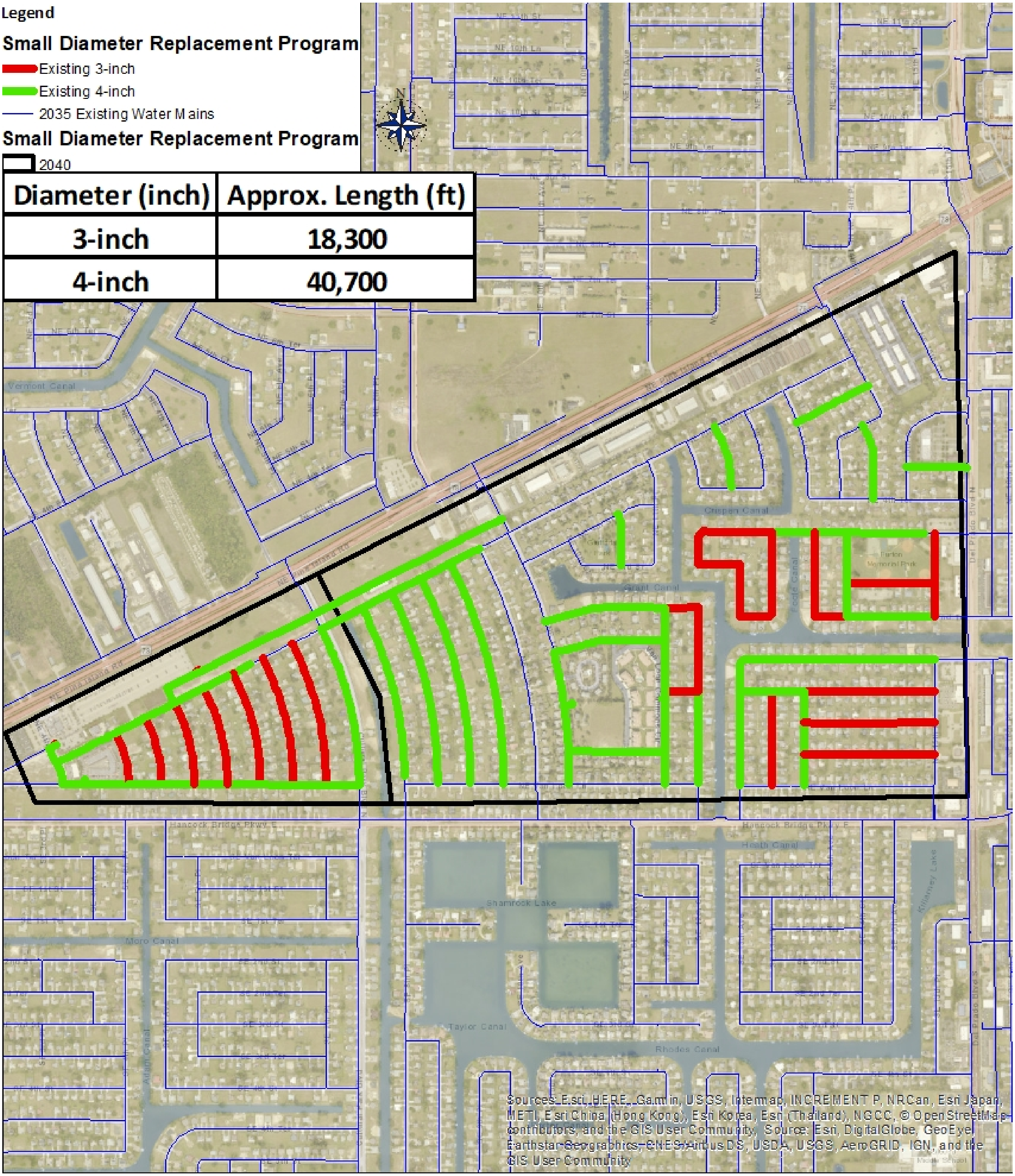

Diameter (inch)	Approx. Length (ft)
3-inch	18,300
4-inch	40,700

Figure 3-55
Planning Year 2040
Small Diameter Replacement Program

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Available fire flow was identified as a deficiency as result of the 2040 Modeling. There are some areas where a fire flow of 1000 gpm could not be provided under a MDD condition. As noted in **Figure 3-56**, most of these areas with fire flow deficiencies are in older parts of the water distribution system with water main diameters less than 6-inches and the City has plans to upgrade these mains with the Small Diameter Pipe Replacement Program. **Figure 3-57** shows the model predictions for available fire flow in the area after replacement of water mains less than 6-inches as a part of the 2040 Small Diameter Replacement Program Area.

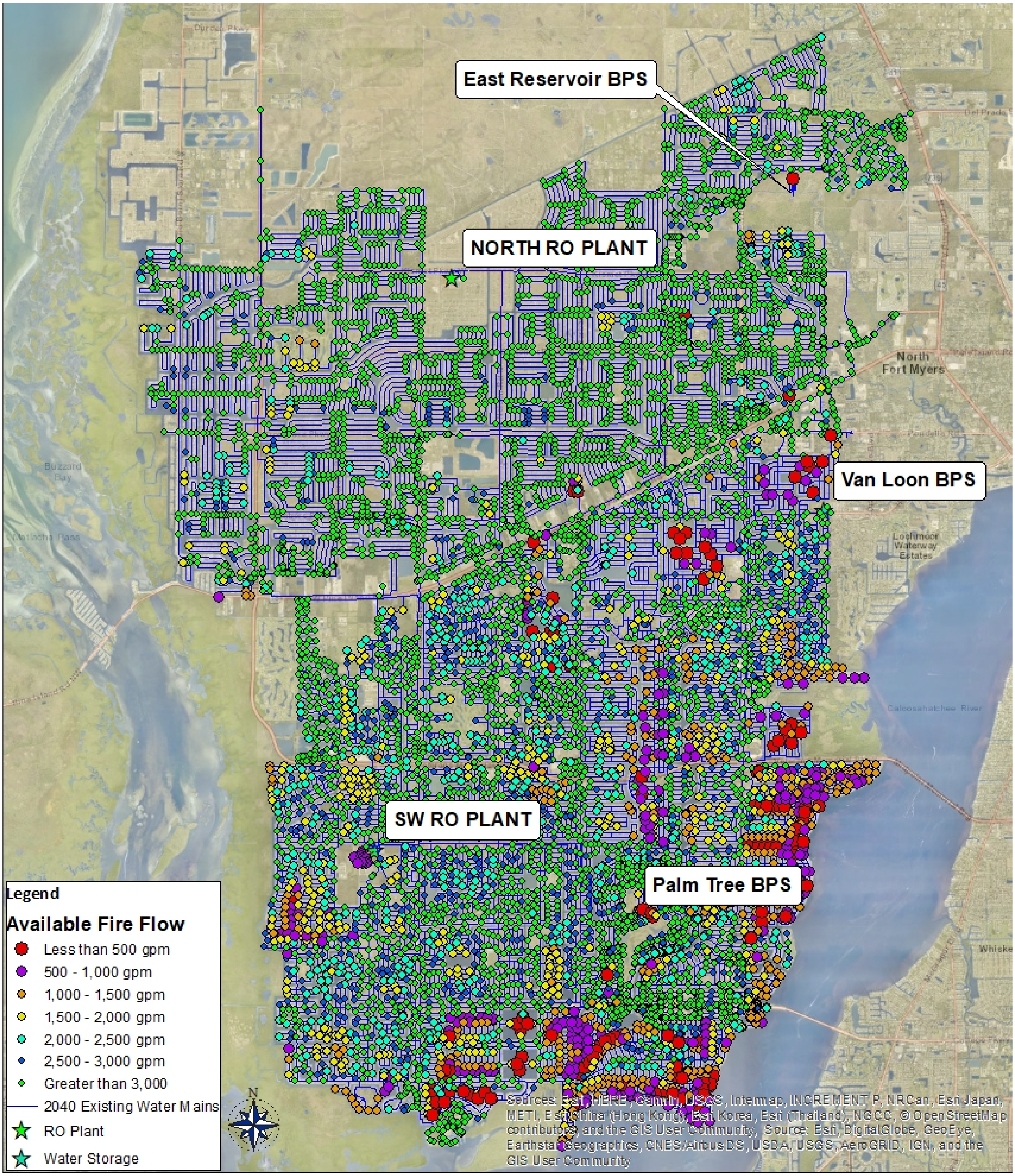

Figure 3-56
Planning Year 2040 MDD
Available Fire Flow

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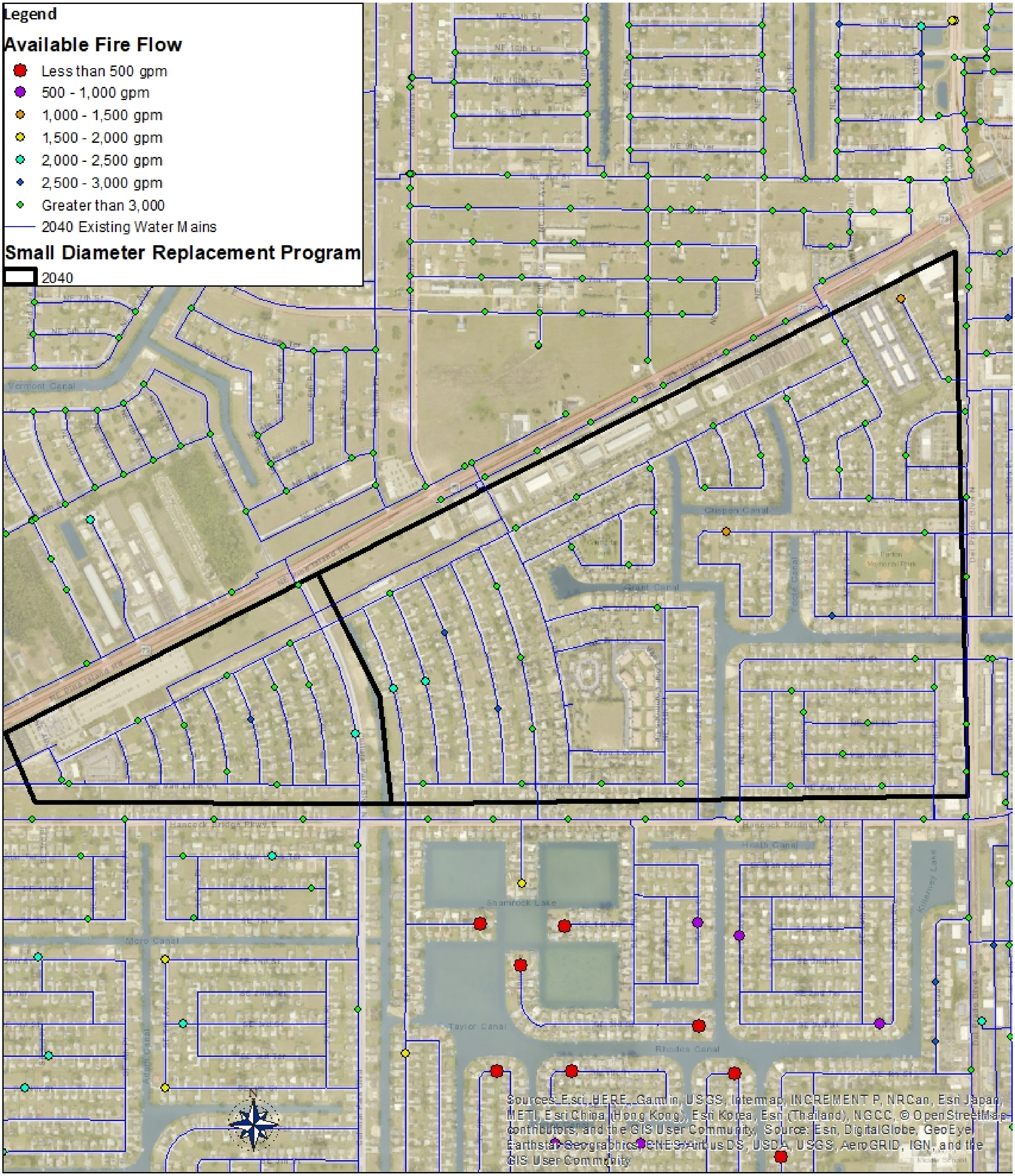

Figure 3-57
2040 Small Diameter Replacement Area
Available Fire Flow

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Table 3-22 summarizes the recommended water main improvements for the 2040 planning year. This includes the recommended water main sizes and the total length for each size.

Table 3-22: Summary of the Recommended UEP Improvement Projects for Planning Year 2040

Service Area	Diameter (inch)	Length (ft)
North 8 UEP Area	6	195,700
	8	16,300
	12	35,800
	16	10,700
North 9 UEP Area	6	133,900
	8	43,100
	12	13,900
	20	12,700
	30	5,400
2040 Small Diameter Replacement Program	6	58,900

Buildout Planning Year Evaluation

A final analysis was performed to evaluate the water distribution system with the addition of new potable water service connections in the North 10 UEP, North 11 UEP, and North 12 UEP service areas at buildout population projections. **Figure 3-58** provides a map displaying the 2040 planned service area along with the North 10, North 11, and North 12 UEP areas anticipated for buildout. The water main improvements assumed for buildout conditions are:

- Water mains to serve the North 10 UEP service area as shown in **Figure 3-59.**
- Water mains to serve the North 11 UEP service area as shown in **Figure 3-60**.
- Water mains to serve the North 12 UEP service area as shown in **Figure 3-61**.
- All the water mains shown in **3-62** with diameters less than 6-inch are assumed to be upsized to a 6-inch water main by buildout conditions.

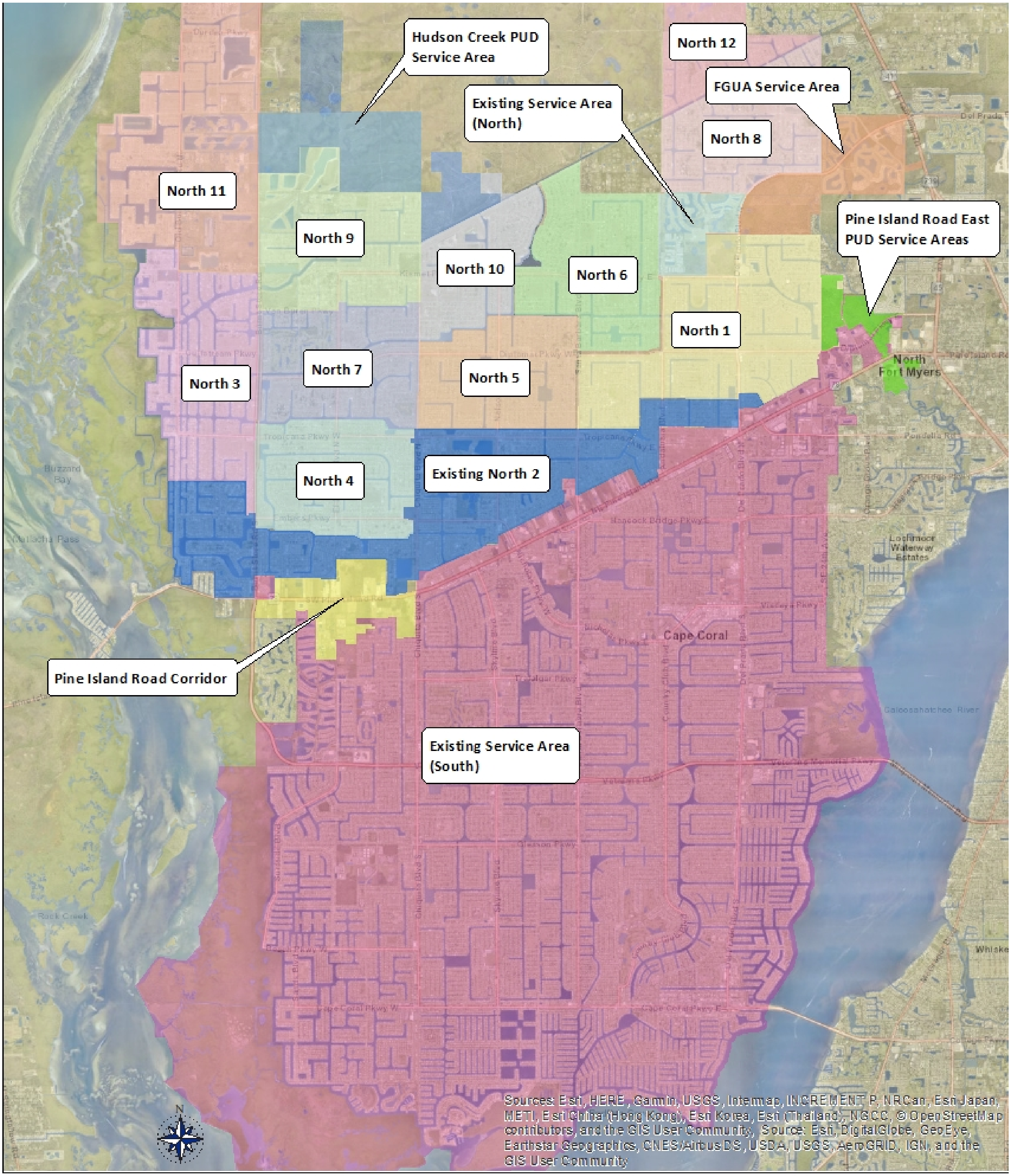

Figure 3-58
Buildout Planning Year
Existing and Proposed Service Areas

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Diameter (inch)	Approx. Length (ft)
6-inch	120,300
8-inch	32,200
12-inch	12,000

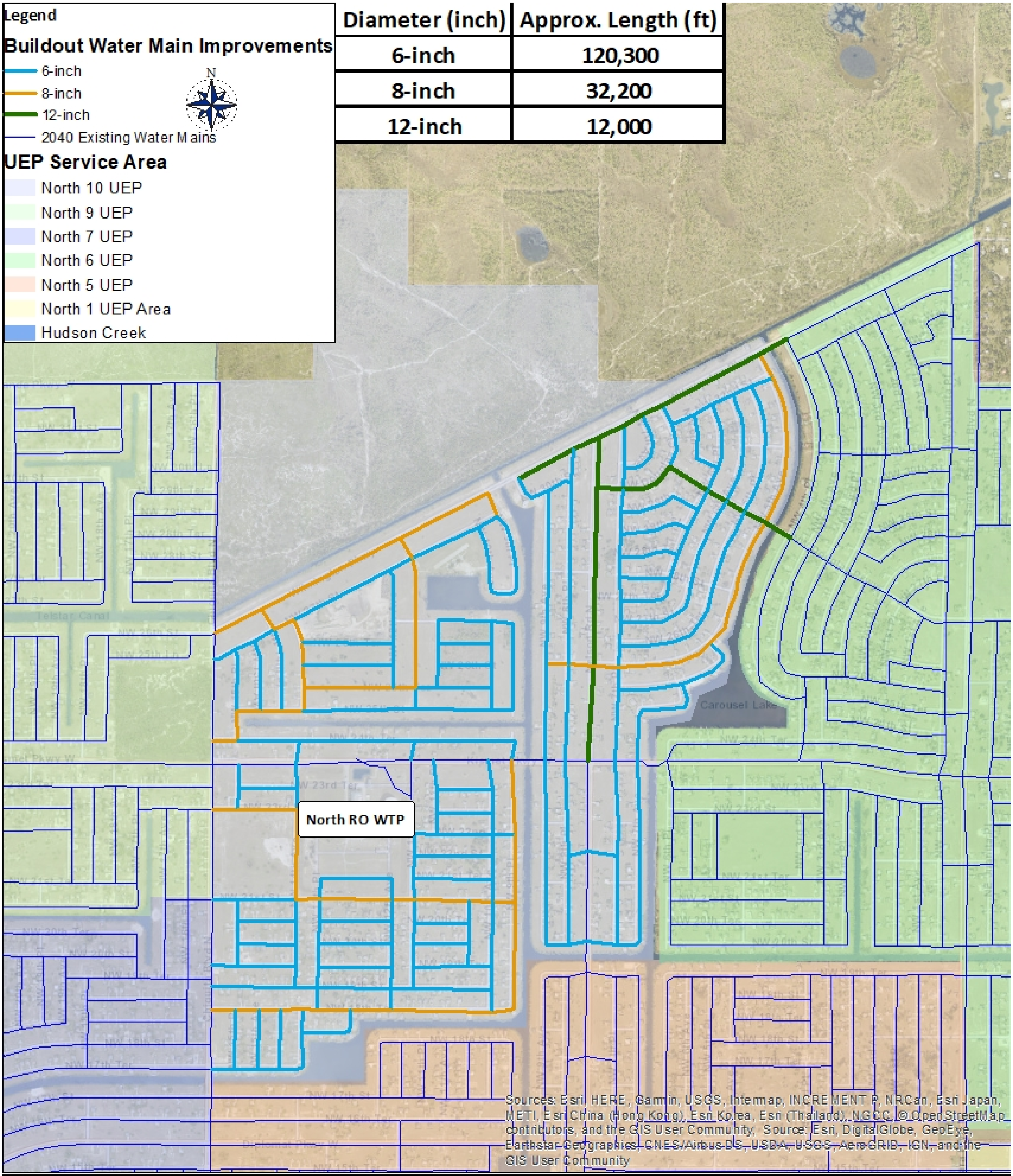

Figure 3-59
Buildout Planning Year
North 10 UEP Improvements

City of Cape Coral
Comprehensive Utilities Master Plan Update

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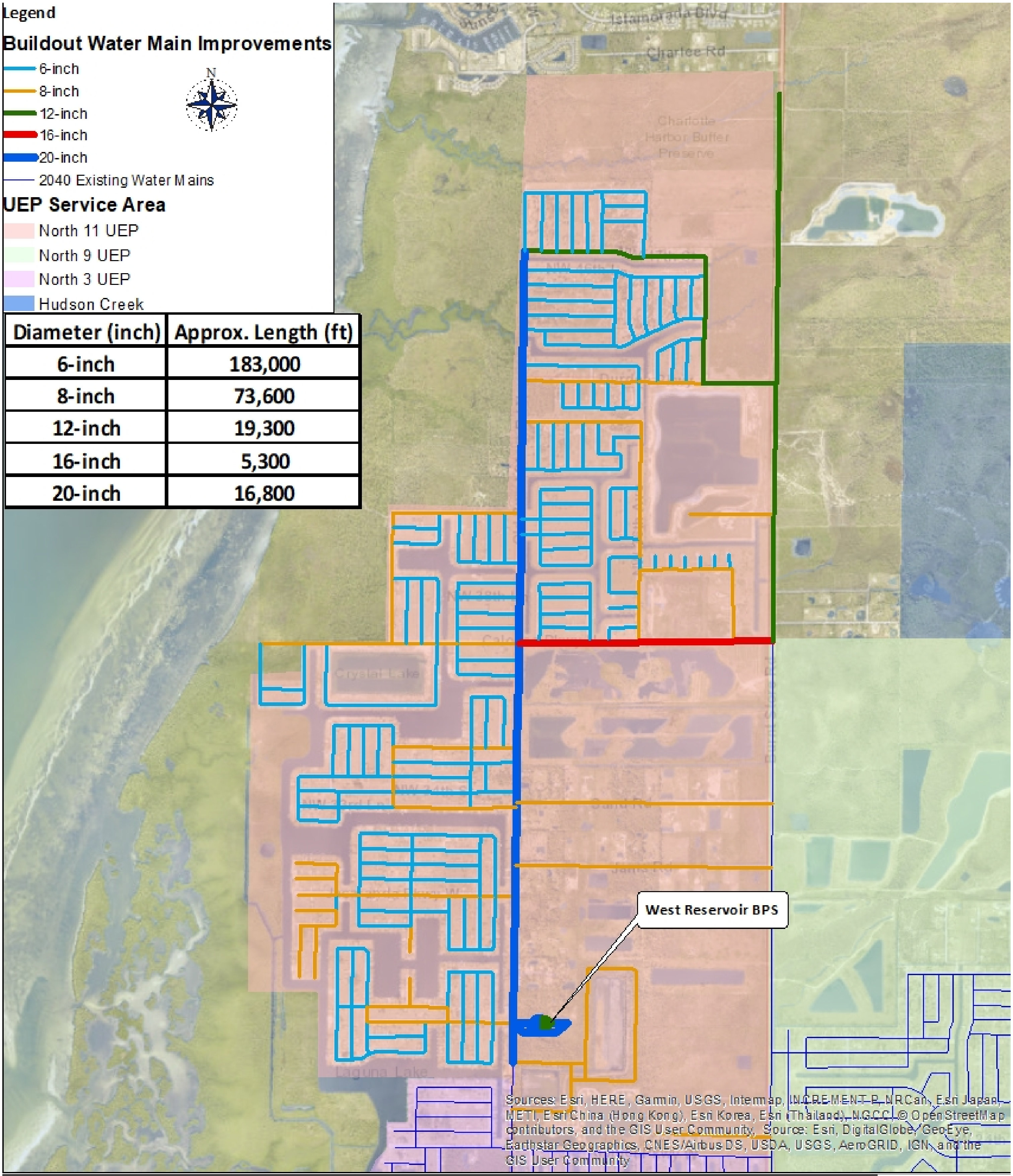

Diameter (inch)	Approx. Length (ft)
6-inch	183,000
8-inch	73,600
12-inch	19,300
16-inch	5,300
20-inch	16,800

Figure 3-60
Buildout Planning Year
North 11 UEP Improvements

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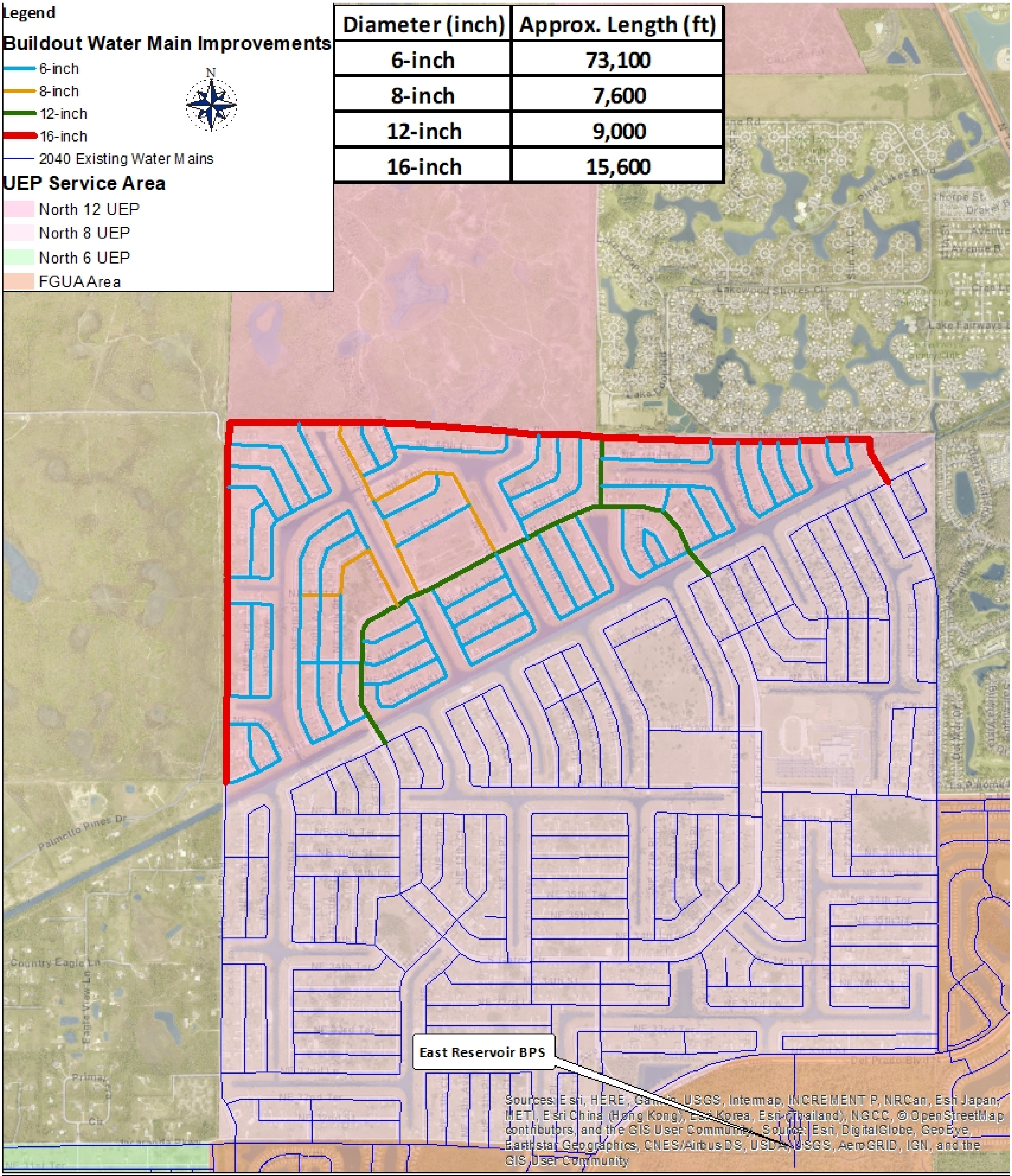

Diameter (inch)	Approx. Length (ft)
6-inch	73,100
8-inch	7,600
12-inch	9,000
16-inch	15,600

Figure 3-61
Buildout Planning Year
North 12 UEP Improvements

City of Cape Coral
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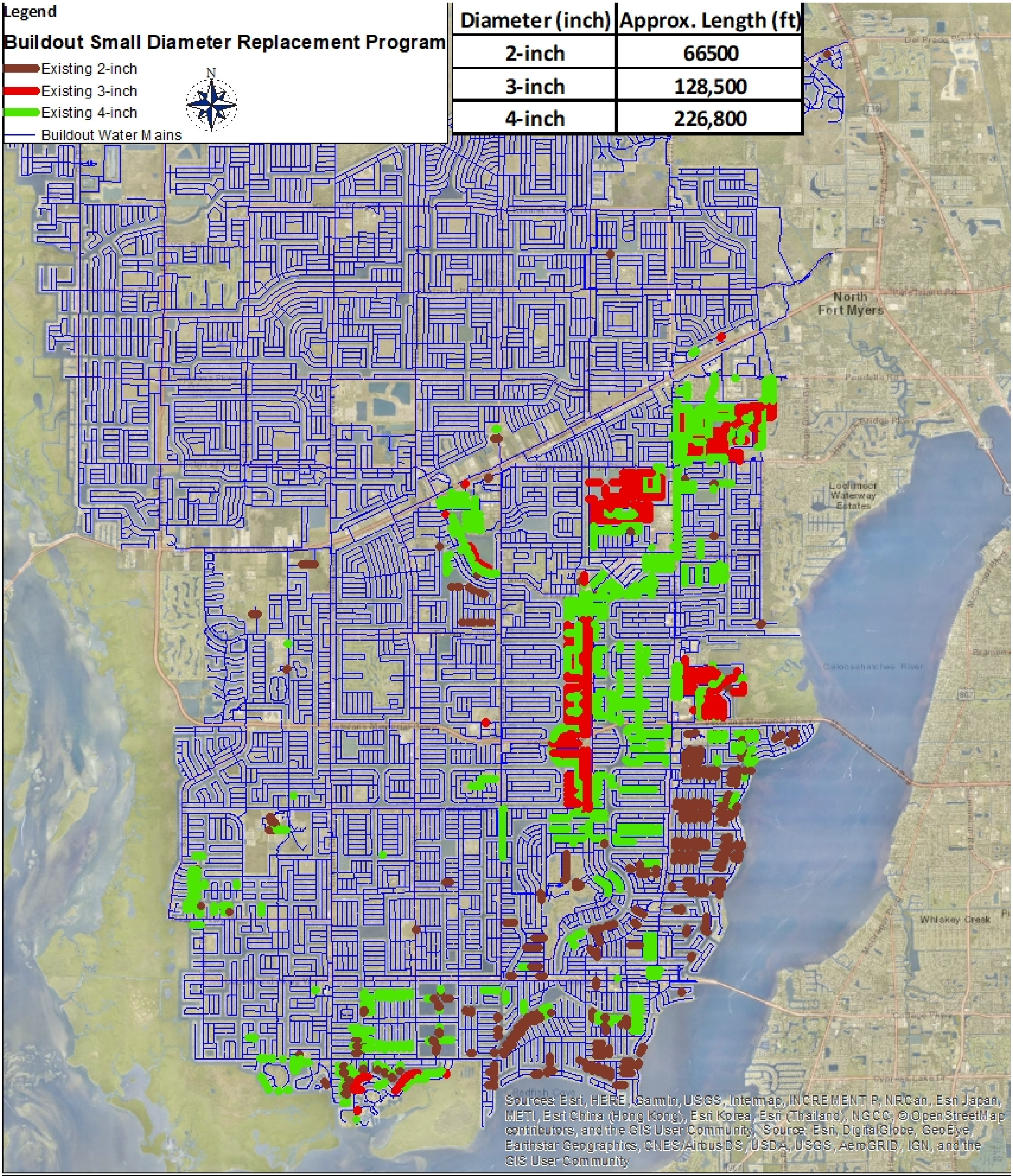

Diameter (inch)	Approx. Length (ft)
2-inch	66500
3-inch	128,500
4-inch	226,800

Figure 3-62
Buildout Planning Year
Small Diameter Replacement Program

City of Cape Coral
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High service pump upgrades at both the Southwest and North RO WTPs were identified as a deficiency as result of the Buildout Modeling. One additional 7,500 gpm high service pump is required at each plant. There were no fire flow deficiencies identified which indicates the Small Pipe Diameter Replacement Program is anticipated to be successful.

Table 3-23 summarizes the recommended water main improvements for the buildout planning year. This includes the recommended water main sizes and the total length for each size.

Table 3-23: Summary of the Recommended UEP Improvement Projects for Buildout Planning Year

Service Area	Diameter (inch)	Length (ft)
North 10 UEP Area*	6	120,300
	8	32,200
	12	12,000
North 11 UEP Area*	6	183,000
	8	73,600
	12	19,300
	16	5,300
	20	16,800
North 12 UEP Area*	6	73,100
	8	7,600
	12	9,000
	16	15,600
Buildout Small Diameter Replacement Program	6	421,800
West Reservoir Transmission Main	20	3,000
West Reservoir Ground Storage Tank and Pump Station	-	-
2 x 7,500 gpm High Service Pump (one at each plant)	-	-

*City plans to complete UEP areas 10, 11, and 12 by 2045.

Conclusions and Recommendations

This analysis identified adequate transmission and distribution improvements needed for expansion of utilities to North 1 through North 12 and new development projects including Hudson Creek and Pine Island Corridor in order to meet the performance criteria established in Section 3.7. The results of this analysis show that the Small Diameter Replacement Program is needed to improve fire flow conditions within the existing system south of Pine Island where deficiencies were identified. Furthermore, storage tank and pump station improvements as well as high service pump improvements are recommended in conjunction with the linear improvements.