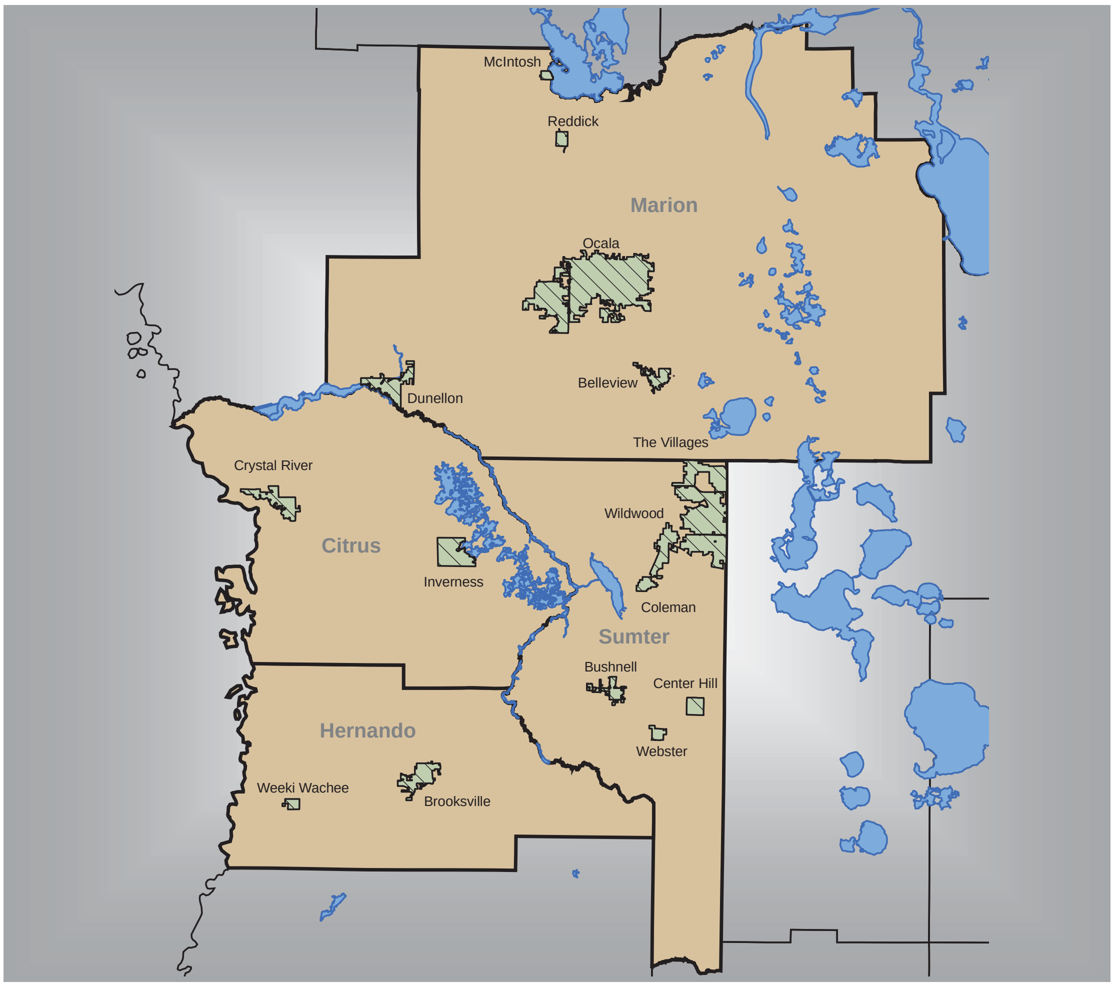
FINAL

Withlacoochee Regional Water Supply Authority

Phase II - Detailed Water Supply Feasibility Analyses



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Chapter 8 – Surfacewater Project Options

8.0 Key Points

Key Points

- Surfacewater is an alternative water supply source that will be available to utilities in the region after fresh groundwater, increasing water conservation, and additional beneficial reuse supplies are tapped.
- Potable surfacewater supply options in the WRWSA include the Withlacoochee River and the Lower Ocklawaha River. Long transmission distances exist between most of these locations and the projected demand areas.
- Individual surfacewater project options along the Withlacoochee River include a conjunctive use in North Sumter County, a reservoir system near Holder, and a supply from Lake Rousseau.
- The Withlacoochee River surfacewater project capacities range from 10 mgd to 25 mgd.
- Water supply yield from the Withlacoochee River is determined using the WRWSA proxy MFLs. Actual MFL adoption by the SWFWMD in 2011 will determine water availability from the river.
- The conceptual water production costs for the Withlacoochee River project options range from $2.38 to $3.15 per thousand gallons.
- Transmission costs range from about 25% to 50% of the water production costs for the Withlacoochee River options.
- The SJRWMD has initiated planning and facilitation efforts to develop the Lower Ocklawaha River. The river could provide cost-effective potable service to WRWSA members in Marion County.
- Long-range planning for surfacewater development should consider dispersed groundwater development in the vicinity of the river systems. Dispersed groundwater projects could transmit future river supplies through their transmission systems.

8.1 The Role of Potable Alternative Water Supply in the WRWSA

Chapters 1, 3, 4, and 5 demonstrated that existing permitted allocations, available local groundwater resources, conservation and reclaimed water will be sufficient to serve the projected 2030 groundwater demand in the WRWSA. Significant adjustments to these projected demands are also anticipated in the region, due to regulatory and incentive measures which have been proactively implemented by the SWFWMD and the SJRWMD in order to extend the lifetime of fresh groundwater. These measures are detailed in Chapter 4 for water conservation and Chapter 5 for beneficial reuse in the WRWSA.

Dispersed fresh groundwater project options were presented in Chapter 6 as opportunities for utilities facing local groundwater resource limitations to continue to rely on groundwater for potable supply. A number of the wellfield options have capacities that exceed identified demands so it is unlikely that all of those projects will be implemented within the 2030 planning horizon.

Water conservation, beneficial reuse, and dispersed groundwater all provide more cost-effective approaches to water supply in the WRWSA region than potable alternative water supplies. There are significant cost and implementation challenges associated with these strategies, but those hurdles pale in comparison to the costs and challenges of developing potable alternative water supplies. The rural character of the region and relative abundance of water resources suggests that smaller communities in the region will likely be able to rely on conservation, beneficial reuse, and planned groundwater for the long haul. The individual strategies will depend on the resources available to each specific utility and the actual rate of population growth.

Growth rates can change quickly and dramatically in rural areas such as the WRWSA region. Flexible strategies are needed within the 20-year planning horizon and beyond, because potable alternative water supplies can take an extremely long time (10-12 years) and are very costly to implement. For the purposes of this plan, potable alternative water supply strategies target larger population centers in the WRWSA where conservation, beneficial reuse, and dispersed groundwater may not meet water needs for the long haul. This strategy can be adjusted over time as growth occurs and additional data is gathered.

Two large river systems in the WRWSA have been identified as potential potable alternative water supply sources: the Withlacoochee River and the Lower Ocklawaha River. The water supply development potential of these systems has been discussed by the WRWSA in WRA (2007) and WRA (2009). As discussed above, neither source is anticipated to be developed for WRWSA members within the 20-year planning horizon. However, the lengthy and costly implementation process for these sources requires a flexible strategy. For this reason, both the Withlacoochee River and the Lower Ocklawaha River[1] are included in the potable alternative water supply strategies for the region.

There are three service areas in the WRWSA with permitted water allocations exceeding 15 mgd:

- The Villages
- Hernando County (Western Service Area)
- City of Ocala

Of these, The Villages is projected to build out prior to 2030. The City of Ocala's long range water demand will depend on the rate of infill and commercial development and whether the utility service area expands. The capacity of the dispersed groundwater projects generally exceeds the projected water demands of these two utilities in 2030, but both of these communities are located closer to the Lower Ocklawaha River and Withlacoochee River system than they are to the Gulf of Mexico. There is also available groundwater in Hernando County, which is located a similar distance from both Lake Rousseau and the Crystal River Power Plant. However, each of these communities is included in the alternative water supply strategy for surfacewater projects as they are the larger public suppliers in the region.

[1] The Lower Ocklawaha River is not a WRWSA project. The SJRWMD has initiated planning and facilitation efforts to develop this source.

When a potable alternative water supply is developed, smaller communities in close proximity to the source may elect to be served. For this reason, Citrus County, Marion County, and Wildwood are included in the alternative water supply strategy for surfacewater projects.

8.2 Water Supply Yield – Withlacoochee River

The Withlacoochee River travels north from its headwaters in the Green Swamp through the the four counties of the WRWSA, emptying into the Gulf of Mexico at Lake Rousseau near Yankeetown. As the river travels downstream, significant inflows occur at the Outlet River from Lake Panasoffkee, at the confluence with Rainbow River, and occasionally at the Tsala Apopka outfall canal (C-331). The Wysong-Coogler Water Conservation Structure (WCS) and the Inglis Dam are significant hydraulic features in the river system.

USGS gages record the river flows. Long-term gages, where flow records reach 60 years in duration, are the general locations where the available flow record is the best and where MFLs will be set. There are three long term gages from south to north along the river system: Trilby, Croom, and Holder. As discussed in Chapter 2, the flow records from these gauges are used to develop proxy MFLs which constrain the potential river withdrawals. Shorter term gages are located near Rital, Nobleton, Floral City, Inverness, the Wysong-Coogler WCS and the Inglis Dam.

This section presents the yield evaluation for the Withlacoochee River system. The evaluation is based on the proxy MFLs from Chapter 2 at Croom and Holder.[2] The yield at the Wysong-Coogler WCS and Lake Rousseau is also discussed. The yield evaluation is subject to actual MFL adoption for the Withlacoochee River in 2011.

Anthropogenic flow declines (due to changes in land use, groundwater withdrawals, etc), the Atlantic Multidecadal Oscillation (see Kelly, 2004), and climate change are not considered in this evaluation. These factors will be considered during the design of any river withdrawal.

8.2.1 Croom Gage

The Croom gage has a flow record to 1939 located about 18.6 miles upstream of the Outlet River from Lake Panasoffkee. The Withlacoochee River at Croom drains 810 square miles. The flows over the period of record for the gage can be used to estimate a median quantity for withdrawal at Croom.

Yield Evaluation

The Proxy MFLs seasonal blocks and estimated withdrawal quantities for Croom are shown on Table 8-1. The Withlacoochee River at this location has a heavily skewed flow distribution and narrow channel which will be sensitive to withdrawals. For this analysis, it is assumed that no withdrawal at Croom would occur within each block when flows are lower than the median. This assumption means that the withdrawal at this location is best suited for conjunctive use or aquifer recharge where periodic supply interruptions are acceptable, subject to actual MFL adoption. As shown, the withdrawals are based on the median daily annual flow (p50) over the period of record for each seasonal block, and the withdrawals vary seasonally. A percentage of

[2] The yield evaluation for the Trilby gage was presented in Chapter 7.

the median daily annual flow can be withdrawn without exceeding the proxy MFL constraint. The proxy MFL at Croom indicates that an estimated withdrawal of 21.95 mgd is available at Croom on a median annual basis.

When flow is above the low-flow threshold, but is not high enough to accommodate a withdrawal sufficient to meet the treatment plant design capacity, the supply would be in a deficit period. Based on the median percent flow reduction allowed at Croom, the deficit flow for a 15 mgd median annual withdrawal is 178 cfs. Additional yield may also be available at lower flows, however, the withdrawal may be less than the design withdrawal. To identify the river yield at lower flows, a percent flow reduction strategy would need to be identified using adopted MFLs. It would require consideration of the downstream MFLs and the development of a zero withdrawal threshold. During low flow periods in the river, the withdrawal would be less than the full quantity to protect the river's ecology.

Other Environmental Considerations

The Withlacoochee River supplies the Tsala Apopka Chain of Lakes through the Leslie Hefner and Orange State canals, in the vicinity of Floral City, roughly 8 miles upstream of the Outlet River from Lake Panasoffkee. With the large land area of Tsala Apopka, there may also be a meaningful subsurface relationship between the river and the Tsala Apopka Chain in this area. It is an area where the river receives groundwater seepage. The watershed features in the downstream reach make it difficult to extrapolate increasing yield from the Croom gage to nearby downstream areas as the river progresses. However, due to the greater than 10 mile length of the reach between Croom and the lake system, an acceptable withdrawal at Croom is unlikely to have indirect hydraulic effects on Lake Panasoffkee or Tsala Apopka. Hydraulic effects in the river channel would require further consideration during the design and permitting of the project. They are anticipated to be acceptable under current permitting criteria and can be optimized with multiple intake locations to minimize the hydraulic effects of the withdrawal.

Since any withdrawal at Croom would reduce downstream flows, the withdrawal must allow the downstream MFLs at Holder and the Tsala Apopka Chain to be met. Any withdrawal at Croom would be minimal on a percentage basis at high flows at Holder, so the primary downstream concern would be the low-flow MFL.

Table 8-1. Design Withdrawal from the Withlacoochee River at Croom.

Design Withdrawal[(1), (2)]			
Seasonal Block	Block I May 10 - July 26	Block II November 3 - May 9	Block III July 27 - November 2
Number of Days	78	189	98
Long-Term Daily Median Flow (mgd)[(3)]	71.7	120	295
Proxy Percent Withdrawal: Low-Flow MFL < Q < High-Flow MFL	13%	13%	15%
Daily Median Withdrawal (mgd)	9.32	15.60	44.25
Potential Annual Median Withdrawal (mgd)	21.95		

(1) Periods with withdrawals lower than the annual and block averages are anticipated. See Chapter 2 for a discussion of low-flow MFLs.
(2) Withdrawals assume that existing legal uses at other locations on the river do not affect available yield.
(3) Based on the 1939 – 2007 period of record for the Croom gage.

8.2.2 Wysong-Coogler Gage

The Wysong-Coogler Water Conservation Structure has a long history. The structure is intended to maintain levels in the Tsala Apopka Chain of Lakes and Lake Panasoffkee, and recharge the groundwater system in coastal Citrus County. The original inflatable fabridam was installed in 1964 and removed in the late 1980's after studies indicated it had little effect on water levels. After concerted citizens' lobbying efforts, the structure was rebuilt in 2002 as an operable, inflatable rubber dam. The regulation schedule for the dam calls for it to be lowered when the flow across it drops below a certain level.

The Wysong structure is typically submerged, making hydraulic analysis difficult, and the structure's historic effect on river hydrology is unclear. The short operational period for the new dam limits any assessment of its effects on river hydrology. In the absence of data on the structure's effect, the flow data for the period of record at the Wysong gage (without consideration of changes to the structure) is the best available predictor of future flows. The Withlacoochee River at Wysong drains approximately 1520 square miles.

Yield Evaluation

As discussed above, the proxy MFL upstream at Croom indicates that an estimated withdrawal of 21.95 mgd is available at Croom on a median annual basis. To protect low flows, the approach assumes no withdrawal would occur when flow is lower than the median at Croom. Based on the median percent flow reduction allowed at Croom, the deficit flow for a 15 mgd median annual withdrawal is 178 cfs. Historic river flows at Wysong exceed the Croom deficit line for the majority of the period of record (reflecting the increase in drainage area from Croom to Wysong). However, the watershed features in this reach, including the Tsala Apopka Chain and Lake Panasoffkee, make it difficult to extrapolate increasing yield from the Croom gage to Wysong as the river progresses downstream. Since the period of record is limited at Wysong, the water supply yield evaluation at Croom is applied at Wysong without adjusting for the

increased flow. This assumption means that the withdrawal at this location may be best suited for conjunctive use or aquifer recharge where periodic supply interruptions are acceptable, subject to actual MFL adoption.

Additional yield may be available at lower flows, however, the withdrawal may be less than the design withdrawal. To identify the river yield at lower flows, a percent flow reduction strategy would need to be identified using adopted MFLs. It would require consideration of the downstream MFLs and the development of a zero withdrawal threshold. During low flow periods in the river, the withdrawal would be less than the full quantity to protect the river's ecology.

Other Environmental Considerations

Lake Panasoffkee and the Tsala Apopka Chain both have adopted MFLs. In contrast to the proxy MFLs for the Withlacoochee River system, which are based on flow criteria, the adopted MFLs for the lake systems are based on stage criteria. There are hydraulic relationships between the river system, lake inflows and outflows, and lake stages that will require consideration in the permitting of the withdrawal. The Outlet River from Lake Panasoffkee has been structurally altered and has a complex hydraulic relationship with the river in the area of the confluence. Hydraulic effects in the river channel would require further consideration during the design and permitting of the project. They are anticipated to be acceptable under current permitting criteria and can be optimized with multiple intake locations to minimize the hydraulic effects of the withdrawal.

For the purposes of MFL development, water levels in Lake Panasoffkee and Tsala Apopka are classified as historic, meaning that there are no measurable impacts to due to withdrawals and structural alterations are similar to current conditions. Both of these systems will allow some general water supply development in their vicinity, as their long-term p50's are greater than the adopted MLL.[3] In addition, the District removed all lakes with adopted MFLs in the WRWSA from its Stressed Lakes List, [4] which eliminated a previous regulatory consideration to both of the lake systems.

Since any withdrawal at Wysong would reduce downstream flows, the withdrawal must allow the downstream MFL at Holder to be met. The yield at Wysong is based on the Croom gage and would be minimal on a percentage basis at high flows at Holder, so the primary downstream concern would be the low-flow MFL.

The proxy low-flow MFL for Holder is 90 mgd or 139 cfs. In comparison, the deficit line for the allowable flow reduction at Wysong is 178 cfs. Since the deficit line is higher than the proxy MFL and no water would be withdrawn when flows are below the deficit line, the 15 mgd withdrawal would not affect the low flow MFL at Holder. For this analysis, it is assumed that the additional contributing area and/or springs between Wysong and Holder do not contribute water at low flows.

[3] Withlacoochee Regional Water Supply Authority (2007). Review of Minimum Flows and Levels – 2006.
[4] Ibid

8.2.3 Holder Gage

The Holder gage has a flow record to 1928 located about 20 miles downstream of the Outlet River from Lake Panasoffkee. The Withlacoochee River at Holder drains about 1820 square miles, and includes the discharge from Tsala Apopka at outfall canal C-331. The flows over the period of record for the gage can be used to estimate a median quantity for withdrawal at Holder. Historic river flows are used to estimate potential future withdrawals.

Yield Evaluation

The Proxy MFLs seasonal blocks and estimated withdrawal quantities for Holder are shown on Table 8-2. The Withlacoochee River at this location has a moderately skewed flow distribution and an incised channel which will be sensitive to withdrawals at low flows. For this analysis, it is assumed when flow in the river is below the MFL low-flow threshold, that no withdrawals would be allowed. This assumption means that a withdrawal at this location may be best suited for a conjunctive use where periodic supply interruptions are acceptable or that reservoir storage may be needed to avoid supply interruptions (subject to actual MFL adoption). During Block 1, on a long-term median basis, flow is below the low-flow threshold 7.5% of the time. Assuming river low flows correlate within the block (serial correlation), a low-flow period would extend for the entire block.

Reservoir Storage Design

Since the proxy MFLs at the Croom gage were used to develop a conjunctive yield, the proxies at the downstream Holder gage are used to develop the storage duration for a reservoir. Future river flows are variable and not known with certainty, so the design of a storage facility is a conceptual optimization process that considers historic flows, the available area of the site, and the level of reliability of the supply storage during its design lifetime. There are two parameter types that are used to characterize system reliability (Nagy et al, 2002):

- Temporal reliability
- Supply reliability

Temporal reliability is the expected percent of time that the reservoir is able to meet demand - the full design capacity of the system. In contrast, supply reliability is the expected proportion of time that the reservoir can provide any water, not just its full design capacity. Ultimately, both of these reliability parameters are interrelated and would be optimized in design (e.g., intermittent water production can improve supply reliability while decreasing temporal reliability). For the purposes of this report, temporal reliability is used to develop a conceptual estimate of the storage duration for a year-round supply at the identified site.

For a year-round type of supply, a reservoir is assumed to be capable of serving its design capacity throughout a no inflow period historically occurring 7.5% of the time; therefore it must include storage for the 78-day duration of Block I. During Blocks 2 and 3, historic flow is below the low-flow threshold 2.5% of the time, on a long-term median basis. Assuming serial correlation within each block, a low-flow period would extend for the entire block. A reservoir design could be capable of serving its design capacity throughout a no inflow period occurring 2.5% of the time; this would include storage for the maximum 189 day duration of Block 2. Cost limitations would likely preclude 189 days of storage, so consideration of the Block I low-flow

regime would lead to a minimum storage requirement of 78 days with an estimated temporal reliability of 2.5%. This approach assumes a full reservoir at the beginning of the Block, and thus does not assume serial correlation between Blocks. However, drought conditions can span multiple years in Florida meaning that serial correlation between Blocks is likely.

Table 8-2. Proxy MFLs Flow Regimes at the Withlacoochee River near Holder Gage.

Block 1	**May 10 - July 26 = 78 day**				
	Flow Regime and Percent Flow Reduction	Block Annual P-Value Flow		Bounds of Flow Regime (mgd)	
			(mgd)	from	To
	High flow (12%)	p90	821	821	
	Middle flow (13%)	p50	332	332	821
	Low flow (0%)	p7.5	96.9	90	332
Block 2	**November 3 - May 9 = 189 days**				
	Flow Regime	Block Annual P-Value Flow		Bounds of Flow Regime (mgd)	
			(mgd)	from	to
	High flow (12%)	p90	1,105	1,105	
	Middle flow (13%)	p50	438	438	1,105
	Low flow (0%)	p2.5	91.5	90	438
Block 3	**July 27 - November 2 = 98 days**				
	Flow Regime	Block Annual Average P-Value Flow		Bounds of Flow Regime (mgd)	
			(mgd)	from	to
	High flow (8%)	p90	2,139	2,139	
	Middle flow (15%)	p50	711	711	2,139
	Low flow (0%)	p2.5	105	90	711

The minimum storage considered under the low-flow regime and the temporal reliability concept does not address other deficit periods that will occur during reservoir operations. When flow is above the low-flow threshold, but is not high enough to accommodate a withdrawal sufficient to meet the treatment plant design capacity, the reservoir would be in a deficit period. Based on an estimated withdrawal of 13% during the middle flow period, the deficit threshold or line would be 308 mgd or 477 cfs. Additional yield may be available at lower flows; however, the withdrawal may be less than the design withdrawal. To identify the river yield at lower flows, a percent flow reduction strategy would need to be identified using adopted MFLs. It would require consideration of the downstream MFLs and the development of a zero withdrawal threshold. During low flow periods in the river, the withdrawal would be less than the full quantity to protect the river's ecology.

More sophisticated analysis is beyond the needs of this report and the assumptions herein will also be affected by actual MFLs adoption. For conceptual design purposes, the reservoir will be sized for a 120 day storage period which is 50% greater than the minimum no-flow requirement. This assumption is likely to generate a cost estimate which is comparable to similar facilities in

west-central Florida. This assumption would be reviewed and adjusted as appropriate during design and permitting.

<u>Other Environmental Considerations</u>

Hydraulic effects in the river channel would require further consideration during the design and permitting of the project. They are anticipated to be acceptable under current permitting criteria and can be optimized with multiple intake locations to minimize the hydraulic effects of the withdrawal.

8.2.4 Lake Rousseau

Chapter 2 noted that a proxy MFL for the Lower Withlacoochee River (based on discharge from Lake Rousseau) can not be estimated at this time. It is anticipated that no or minimal raw water storage will be required for a withdrawal this location due to sufficient flows to the lower river during the Block 1 and Block 2 dry seasons. These flows will occur due to the contributing flows from the Rainbow River just upstream of Lake Rousseau.

Rainbow River has a relatively even flow distribution due to its spring source; and the historic Rainbow River p50 is 681 cfs. A 13% flow reduction from the Rainbow River p50, based on the middle flow reduction in the proxy MFLs, is 89 cfs or 57 mgd. This value does not consider incoming flows from the Withlacoochee River upstream of the confluence with Rainbow River.

It should be noted that estuarine conditions in the Lower Withlacoochee River downstream of Lake Rousseau reflect a different type of constraint than that considered in the proxy MFLs. Actual MFL adoption for the Lower Withlacoochee River will determine the yield and whether raw water storage is required at Lake Rousseau. It might also affect possible withdrawals upstream near Holder. In addition, the USACOE regulation schedule at the Inglis Dam will need to be considered during the design and permitting of a facility at either site.

8.3 Water Supply Yield – Lower Ocklawaha River

The Ocklawaha River travels north from its headwaters in Lake County through the eastern half of Marion County. As the river travels downstream, significant inflows occur at the confluence with Silver River, at Orange Creek. The Moss Bluff Dam and Rodman Dam are significant hydraulic features for the river system as it traverses Marion County.

Long-term USGS gages record the river flows. There are three long term gages from south to north along the river system: Moss Bluff, Conner, and Eureka. Though there are gaps in these data sets, the flow records from these gauges will be used to develop MFLs which constrain the potential river withdrawals. A shorter term gage is located at the Rodman Dam.

As discussed in WRA (2009), several estimates have been made of yield from the Ocklawaha River system. These estimates tend to focus on areas downstream of the confluence with the Silver River which is known as the Lower Ocklawaha River.

Just downstream of the Silver River confluence at the Conner gage, the p50 is 585.8 mgd and the river has a relatively even flow distribution due to the spring source of the Silver River. It is anticipated at this point, that no or minimal raw water storage will be required for this location

due to the contributing flows from the Silver River. The current yield estimated by the SJRWMD is 83.85 mgd at this location. The yield estimate is subject to actual MFL adoption for the Lower Ocklawaha River in 2011.

8.4 Service Area Demands

Potable surfacewater may serve communities located in Citrus, Hernando, Sumter and Marion Counties. However, more cost-effective water supply strategies than potable surfacewater, including conservation, fresh groundwater and reclaimed water, are likely to be sufficient to meet water supply needs in the WRWSA region to 2030.

Water demands have not been projected for this region on a utility-by-utility basis beyond 2030,[5] so general long-range planning values are used to determine a possible design capacity for potable surfacewater projects. These long-range values are roughly proportional to the permitted allocation in each service area. Table 8-3 below provides a summary of these potential consumers and the long-range planning demands.

Table 8-3. Potential Users for Surfacewater Supply.

#	Permitted Service Area	ADF
		mgd
1	Citrus County – Citrus County / WRWSA	2.5
2	Hernando County Utilities – West Hernando	10.0
3	City of Ocala	7.5
4	Marion County Utilities	2.5
5	City of Wildwood	2.5
6	The Villages	5.0
	Total:	**30.0**

8.5 Surfacewater Project Options in the WRWSA

The yield analyses utilizing the proxy MFLs suggest that certain types of surfacewater development may be best suited for different reaches of the Withlacoochee River, subject to actual MFL adoption. The reach from Croom to the vicinity of the Wysong-Coogler WCS may be best suited for a conjunctive use where periodic supply interruptions are acceptable. The reach in the area of the Holder gage may be an appropriate setting for a system that includes reservoir storage. Finally, Lake Rousseau may provide a steady supply without the need for supplemental storage.

Long transmission distances exist between most of these locations and the projected demand areas. The length of transmission in some cases is such that economies of scale associated with service to multiple users will be diminished by the need for transmission. For example, a small or conjunctive withdrawal from the Withlacoochee River reach upstream of Holder is likely to prove more cost-effective for northeastern Sumter County utilities than a similar withdrawal from Lake Rousseau, which would require about 15 miles of additional transmission which would require about 15 miles of additional transmission and regional-scale participation.

[5] Reference water demand projections to 2055 were included in Phase I, but they were developed on a county-by-county basis.

Similarly, for communities in Marion County, a withdrawal from the Lower Ocklawaha River may prove more cost-effective than a similar withdrawal from the Withlacoochee River system.

A menu of surfacewater options is identified for the WRWSA region for comparative purposes. Not all projects are likely to be implemented or serve all of the long range demands identified in Table 8-3, though some economies of scale are likely. Transmission distance, economies of scale with multiple users, and yield will inform the project selection for member communities.

Surfacewater project options to provide potable water year-round are identified for both Holder and Lake Rousseau based on the yield analyses. Transmission lengths are generally less for a Holder location than at Lake Rousseau, but a reservoir would be needed at Holder. These two options provide a comparison between two different potential locations on the Withlacoochee River which have different hydrologic constraints.

Surfacewater project options can also involve conjunctive use, meaning they would rely on surfacewater and groundwater in combination. A conjunctive project is identified in North Sumter County that provides a comparison with longer transmission distances from Lake Rousseau or Holder. This project is based on surfacewater use when available from the river, and groundwater use during low flows when surfacewater is not available. By utilizing groundwater during periods of low flow, the project would not require a costly reservoir that also loses water to evaporation. This type of project can extend groundwater availability by reducing the frequency and duration of groundwater withdrawals.

The project location, supply description and design capacity for the WRWSA surfacewater projects is listed in Table 8-4. The capacities of each project are loosely based on collective long-range planning demands beyond 2030. The intent of these projects is to provide a reasonable approximation of a project that could be needed over a 50-year long range outlook. Figure 8-1 shows the general location of the potable surfacewater project options available to WRWSA members.

Table 8-4. WRWSA Potable Surfacewater Projects.

Source	Location	Supply Description	ADF
			mgd
Withlacoochee River	North Sumter County	Conjunctive Use – No Reservoir	10
Withlacoochee River	Near Holder	Year-Round Supply – Reservoir	25
Lake Rousseau	Lake Rousseau	Year-Round Supply – No Reservoir	25

Notes:
1) *Listed projects and associated yield evaluations are for individual consideration. They are not evaluated on a cumulative basis.*

As previously mentioned, the Lower Ocklawaha River is not included in the table because it is not a WRWSA project. For comparative purposes, if the Lower Ocklawaha River project was conceived in a similar fashion for members in Marion County, it would be a year-round potable supply (no reservoir) with a design capacity of 15 mgd.

The SJRWMD has included in their water supply plan two concepts for potable service from the Lower Ocklawaha River. One concept is a very large system (83.85 mgd) near Conner. This concept was initially developed by the SJRWMD with thoughts of serving large demands in Orange County; its service was subsequently revised to consider Lake, Putnam, and Marion Counties. Another concept is a moderately sized system (20 mgd) near the Rodman Reservoir with supply to utilities located in Putnam County. With respect to WRWSA members, the latter concept now appears applicable near the Conner location. Actual water demands in the identified service area are unlikely to merit further consideration of the former concept in the foreseeable future.

8.6 Withlacoochee River Facilities

For conceptual design purposes, certain criteria were utilized when evaluating potential sites for the location of water supply options along the Withlacoochee River. These include:

- The property must be publicly owned by the SWFWMD, the County, the State, or any other government agency which should result in limited land acquisition costs;
- The parcel must be large enough to accommodate the facilities necessary for supply from that reach of the river (treatment plant, reservoir, etc); and,
- The site must be as close to the raw water intake as possible and have road access.

Based on these requirements, potential sites for the project options were identified. This section presents the conceptual project locations and supply facility layouts at each site. The river intake and raw water pumping facilities are also discussed in this section.

8.6.1 North Sumter

The site in North Sumter is a property consisting of multiple parcels owned by the SWFWMD. The parcel is adjacent to the Withlacoochee River and has access to SR 315A. The property is approximately 750 acres in size and is sufficient to accommodate the water supply facilities for the 10 mgd conjunctive use project. The Wysong-Coogler Water Conservation structure is about 1.8 miles downstream of the intake. Figure 8-2 depicts the location of the proposed site and water supply facilities.

8.6.2 Near Holder

The site near Holder is a property owned by the SWFWMD. It is located in Marion County, northeast of the town of Holder. The parcel is adjacent to the Withlacoochee River and has access to SR 200. The property is approximately 8,250 acres in size and is sufficient to accommodate the 25 mgd water supply facilities including a raw water storage reservoir. Figure 8-3 depicts the location of the proposed site and water supply facilities.

8.6.3 Lake Rousseau

The site near Lake Rousseau is located in Levy County. Lake Rousseau is approximately 3 miles to the south of the proposed location. The site consists of more than 10 parcels owned by the Florida Department of Agriculture and Consumer Services (FDACS) with a total area of approximately 7,200 acres. The site has access to SR 336 and is sufficient to accommodate

the 25 mgd water supply facilities. Figure 8-4 shows the location of the proposed site and water supply facilities.

Few publicly owned properties meeting the selection criteria were identified in the vicinity of Lake Rousseau. The identified site would require approximately 4 miles of raw water transmission north from the lake and a comparable length of finished water transmission back south towards the pipeline corridors. A better suited location south or east of the lake should be able reduce overall transmission lengths by 5 to 10 miles.

8.6.4 River Intake

A detailed study of the effect of the river intake on the natural environment in the area will need and on the river flow regime will need to be performed during design and permitting in order to determine the location and design of the intake structure. For the purposes of this section, a concrete intake structure is proposed on the bank of the river at a location reasonably proximate to the potential site.

The intake will consist of a submerged reinforced concrete weir structure. The weir would be set at an elevation equal to the water elevation, below which no withdrawals can occur. A floating barrier and screens will be installed to prevent entry into the structure. The design of the structure will address FDEP criteria for impingement and entrainment of aquatic organisms. Generally, an intake velocity of less than 2.0 feet per second will be developed and the screen design will prevent access by listed species.

8.6.5 Raw Water Pump Station

The raw water pump station will be constructed next to the intake structure. Water would flow from the intake structure through a culvert or large diameter pipe to the wet well of the raw water pump station. A small building housing the MCC and an emergency generator will be constructed. The pump station would include two or more vertical turbine pumps to pump raw water from the wet well to the head of the WTP. For the North Sumter and Lake Rousseau locations, the capacity of the pump station would be the same as the design capacity of the project. For the Holder location, the capacity of the pump station would be twice the capacity of the project in order to fill the reservoir during high flow periods. Standby pump capacity would be provided in accordance with the Ten State Standards and Chapter 62-550, F.A.C. The wet well would meet the hydraulic needs of the pumps but would not provide storage. The raw water pump station would pump the raw water to the treatment plant or reservoir through a large diameter concrete pipe.

8.7 Conceptual Design of Raw Water Storage Reservoir

The reach in the area of the Holder gage may be an appropriate setting for a system that includes reservoir storage, based on possible limitations to low-flow withdrawals from the Withlacoochee River. Recent experiences in the Tampa Bay region have pointed out the importance of design and construction for reservoirs in west-central Florida, particularly in the areas of seepage control and structural geology. Extensive site specific testing, evaluation and design will be needed in subsequent investigations for the reservoir. For the purposes of this report, this section describes the conceptual design for a raw water storage reservoir to support a 25 mgd year-round supply in the Holder area.

8.7.1 Reservoir Size

The function of the reservoir is to store raw water during the wet months for treatment and supply during the dry season when withdrawals are reduced in the river. In order to properly size the reservoir, a thorough water balance must be prepared in the consequent project phases; including river withdrawals based on adopted MFLs, rainfall, seepage losses, and evaporation rates for the proposed location of the reservoir. Further evaluation of the statistical frequency and duration of deficit periods, and of their relationship with the low-flow regime, would be required to optimize the size the reservoir and refine the estimate of reliability. As indicated earlier, the reservoir for this conceptual phase of the project will be sized for a 120 day storage period. This storage period for the project near Holder correlates to the storage volume below:

- 120 days storage * 25 mgd = 3.0 billion gallons

A storage depth of 20 feet is assumed. The area of the reservoir with this storage depth would be approximately 20,065,000 sq. ft. or 461 acres. Five feet of free board would be provided in accordance with 62-572, F.A.C. regulations. This would bring the total height of the reservoir berm to 28 feet with the accommodation of direct rainfall from large storm events. The reservoir would also meet requirement of the USACOE engineering manual, Chapter 15 (USACOE, 1997). Supplemental sources, either at the utilities or at the reservoir, may also be able to assist with optimization of the reservoir design.

8.7.2 Structural Geology Evaluation

Further evaluation will be needed to prove up the site specific geology and to document that there are no sinkholes in the proposed reservoir area and that the area is not susceptible to sinkhole formation. Current methodologies will be used to assess the potential for sinkhole development, including:

- Review of ancient and modern sinkhole distribution;
- Site specific assessment of surficial soil and bedrock geology;
- Site specific assessment of hydrogeologic information;
- Site specific geotechnical investigation including ground penetrating radar; and,
- Local experience.

If the potential for sinkhole development is identified, alternative site locations or specific construction contingency plans may be needed.

8.7.3 Hydrogeologic Evaluation

In conjunction with the water balance used to size the reservoir, site specific soil tests would have to be performed to determine soil percolation rates and potential seepage losses. Figure 8-5 shows the geology of Marion and Citrus Counties adapted from the Geologic Map of the State of Florida (Scott, et. al. 2001). Figure 8-6 shows the map legend. In the vicinity of the potential reservoir, the surface geology is Eocene Ocala Limestone. The Ocala Limestone consists of nearly pure limestones and occasional dolostones, composed of a white to cream-colored, fine to medium grained, poorly to moderately indurated, very fossiliferous limestone. The permeable, highly transmissive carbonates of the Ocala Limestone form an important part

of the FAS. It is one of the most permeable rock units in the FAS. The presence of this highly permeable and essentially unconfined surface formation in the vicinity of the proposed reservoir suggests that seepage losses will be extremely significant. For conceptual design purposes, it is assumed that a reservoir liner will be needed to prevent excessive water loss.

It is noted that similar surface geology exists along the river from Lake Panasoffkee north nearly to Lake Rousseau. Any year-round supply alternative along this reach (except for Lake Rousseau) will likely require a lined reservoir for storage, assuming actual MFLs effectively limit seasonal withdrawals. Alternatively, ASR wells could be considered, but the known geology in this region is not considered suitable for ASR due to the lack of consistent confinement.

8.7.4 Reservoir Construction

Reservoir construction will ensure dam stability and functionality for water storage. Specific issues that will be addressed include inside slope protection to protect against erosion from wave runup; seepage control on the outside slope; a spillway for emergency overflows; and shaping and compaction of the reservoir foundation and embankment.

Inside slopes will be protected from erosion by optimization of design alternatives such as soil-cement planting; stair step protection systems; vegetated berms; and optimization of interior slopes. Slopes may vary from 2:1 to 2.5:1. In general, flatter slopes are more desirable for maintenance and stability purposes.

Seepage control on the outside slope will consider the permeability of the embankment soils and the placement of those soils. A blanket system and perimeter toe-drain will collect seepage and return it to the reservoir through a HDPE collector and sump pump system. The outside slope would be 2:1 with a 20-feet maintenance access atop the berm.

The bottom of the proposed reservoir will be lined with an HDPE liner system to minimize water loss in the reservoir. The liner thickness will be established during the design phase based on geotechnical studies of the existing soils. The membrane thickness will likely be 30-45 mils.

The soil foundation and embankment areas will need to be prepared by removal of all stumps, roots and rocks. Next it will be shaped and compacted. Once this has been completed, liner sections will be installed and fusion welded. Final testing will include seam shear and peel testing to ensure an acceptable seal between the liner sections.

8.7.5 Transfer Pump Station

To convey raw water from the reservoir to the water treatment plant, a transfer pump station will be required. The station would have would utilize three or more horizontal split-case centrifugal pumps.

8.8 Conceptual Water Treatment Facility Design

This section presents the conceptual design for the surfacewater treatment facilities. Each facility will include treatment operations and processes to efficiently and cost effectively convert raw surfacewater into potable (finished) water with quality meeting all requisite local, state, and federal regulations. The design and permitting for each facility will identify and evaluate

potential project specific issues, including the siting and quantity of river withdrawals. Site specific considerations related to land acquisition, requisite permitting issues of the F.A.C., the SWFWMD, and local ordinances and regulations are not addressed herein.

For conceptual design purposes, the process selection at each facility is a common treatment train for a fresh surfacewater supply.[6] An enhanced conventional treatment process is selected consisting of powdered activated carbon, coagulation, ballasted flocculation, sedimentation, filtration, disinfection, finished water storage and pumping.[7] This process selection is generally based on the treatment trains at comparable facilities in west-central Florida. The intent to generate cost estimates comparable to operating surfacewater treatment plants. Each facility is assumed to be identical from a process perspective. Therefore, the conceptual design and process components are identical for each facility. They are provided for illustrative purposes to show the design elements of each facility.

Transmission routing and project costs are not included in this section because they will vary depending on the configuration of each individual project. Transmission routing and project costs for each individual project are provided in subsequent sections.

8.8.1 Basis of Design

In Florida, FDEP has jurisdiction over the drinking water standards described in Chapter 62-520 and 62-550, F.A.C. The primary drinking water standards, which are health-based and include the control of pathogens, are described in Rule 62-550.310, F.A.C., while the Secondary Drinking Water Standards are contained in Rule 62-550.320. Secondary standards generally apply to the aesthetic qualities of water (appearance, taste, and odor) that are typically desired for public acceptance and use. No known health effects are currently associated with the secondary standards. All primary and secondary standards are enforced for potable water supplies and, as such, compliance with all standards will occur when planning for and designing the new water supply facility.

Minimum capacity criteria for water supply facilities are described in Chapter 62-550, F.A.C. FDEP has jurisdiction over these criteria, which include design requirements for supply capacity, high service pumping capacity, stand-by power, and storage. The new water supply facility will meet all capacity criteria as well as the Ten State Standards. Key criteria are discussed in the applicable sections below.

8.8.2 Water Treatment Plant

The surfacewater treatment plant and appurtenant facilities would require a range of 10-20 acres depending on the project size. The process selection is an enhanced conventional treatment process consisting of powdered activated carbon, coagulation, ballasted flocculation,

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[6] This assumes that dissolved salts are not present in the water at sufficient concentrations (> 250 mg/L) to require membrane treatment

[7] Membrane processes are becoming increasingly common in the treatment of surfacewaters and offer considerable advantages to conventional processes in the areas of taste and odor control and disinfection byproduct formation. This process will likely require conventional pre-treatment and filtration to protect the membranes. This type of system may be considered during design when a project location is confirmed and water quality data has been gathered.

sedimentation, filtration, disinfection, finished water storage and pumping, as shown in the process flow diagram on Figure 8-7.[8]

The actual treatment process will be dependent on the water quality present at the specific site. The Withlacoochee River system is not currently used for potable supply, so further pilot study or jar testing will evaluate the full range of raw water quality that may be experienced. Water quality data should be gathered reflecting high and low flow conditions in the river. Surfacewater treatment processes are reasonably well understood in Florida waters; records from operational facilities should be reviewed during design. The major elements of the surfacewater treatment plant are discussed below.

8.8.2.1 Powdered Activated Carbon System

A powder activated carbon (PAC) system for taste and odor control will be used for the surfacewater treatment plant. When PAC is introduced into water, it adsorbs the taste and odor causing compounds and low concentrations of pesticides and some organic pollutants. The system will consist of concrete contact basins providing a minimum of 15 minutes of contact time during peak flows, PAC clarifiers, PAC storage silo, and PAC injector. PAC will be injected at the beginning of the contact chamber and will be removed from the water by sedimentation in the PAC clarifiers.

8.8.2.2 Coagulation / Ballasted Flocculation / Sedimentation System

A coagulation / ballasted flocculation / sedimentation system of the ACTIFLO type is assumed for the project evaluation. This will generate a comparable cost estimate to other West Central Florida SWTPs without requiring a detailed water quality review. If this project is selected for further design consideration, the proprietary ACTIFLO system will be compared with other conventional treatment systems, as appropriate, and water quality data requirements identified. The ACTIFLO system is used for the removal of organic and inorganic particulate constituents and portion of the dissolved organic matter from surfacewaters. This is achieved by conditioning the water by coagulation and sedimentation followed by sedimentation and filtration. Typical coagulants used are alum or ferric chloride, ferric sulfate, natural or synthetic polyelectrolytes. Detailed analysis of the water quality parameters will be required in the following phases of the project to determine the exact type of coagulant.

The proposed ACTIFLO system consists of two or more trains, each having a treatment capacity equal to a proportion of the design capacity. Each train consists of four tanks – coagulation tank, injection tank, maturation tank, and settling tank. A static mixer will be installed on the influent pipe of the ACTIFLO system where coagulant will be injected and will be mixed with the raw water. Raw water enters the coagulation tank where mixing is introduced by a static mixer for better reaction with the coagulant. From the coagulation tank water is routed to the injection tank where sand and polymer are added by hydrocyclones. The purpose of the sand is to serve as a media around which the floc will form with the help of the polymer. The maturation tank is where the actual flocculation occurs. Separation of the floc from the water occurs in the sedimentation tank. The tank is equipped with lamella tubes for reducing the settling time and thus reducing the size of the settler. A scraper at the bottom of the settling tank collects the solids which are pumped by recycle pumps to the hydrocyclones. There sand

[8] Ibid.

is separated from the floc and reused in the process. Sludge that remains is collected in a wet well and pumped to the sludge processing system.

8.8.2.3 Filtration System

A rapid gravity flow dual media bed filtration system following the ACTIFLO system is proposed for the project. It removes finer particles that were not removed by the plate settler of the ACTIFLO system. A schematic of the proposed filter system is included in Appendix A. The system consists of multiple cells each having a filtration area of 880 ft^2. The total filtration area depends on the capacity of the project; the filtration rate is 4 gpm/ft^2. Dual media consisting of 12" sand and 18" anthracite is currently proposed. A polymer can be fed to the influent filter pipe to aid the filtration process. Filters will be cleaned via backwashing and air scour. Backwashing will be provided by backwash pumps pumping water from the finished water storage tanks at a rate of 20 gpm/ft^2. Spent backwash water flows by gravity to a pump station and is pumped to the sludge processing system.

8.8.2.4 Disinfection

The proposed disinfection system consists of mixed oxidant generation system and concrete contact chambers. Onsite generation was selected based on previous studies conducted by URS when evaluating onsite generation versus bulk storage. The generation system uses an electrolysis process to convert saltwater brine to a mixed oxidant which contains hypochlorous acid and chlor-oxygen species. Disinfectant will be added before the filters for preventing microbial growth on the filter media and after the filters at the beginning of the contact chambers for disinfection. Concrete contact chambers will be constructed providing twenty minutes of contact time. Disinfected water will be pumped from the contact chambers to the finished water storage tanks.

The product water will require addition of chemicals for pH stability, corrosion inhibition, and scale control in the transmission system. The final configuration of post treatment chemical addition will be affected by the selection of disinfectant method, pilot testing, the transmission line material and feasible blending considerations identified in design. However, the utilities would be responsible for blending the finished water with the water in their distribution system(s).

8.8.2.5 Finished Water Storage

The water supply facility will typically be a new supply for member utilities. Storage for the product water would be provided in case of transmission interruption or other conflicts with the delivery and use system. Two or more storage tanks would be provided on site for plant downtime and transmission system interruptions. FDEP requirements for minimum storage stipulate that the total storage capacity of the facility meet at least 25% of the maximum daily demand of the system. For conceptual design, it is assumed that 50% of the projected average daily demand is sufficient storage to meet the storage requirements. The maximum daily demand and storage requirements will be determined during design and permitting through coordination with utility end users.

Storage will be provided by circular prestressed concrete storage tanks, constructed in accordance with AWWA D-110 (e.g., a composite similar to a CROM tank). The site will be

developed with enough area to install a future storage tank to meet expansion needs beyond the horizon of this study.

8.8.2.6 Finished Water Pump Station

In order to transfer water from the treatment facility to the communities served, a dedicated finished water pumping system would be installed. This system would consist of three or more horizontal split-case pumping units (possibly with variable speed drives) and would be controlled using pressure levels in the downstream transmission/distribution system, water levels in downstream storage tanks, or both. Results from the hydraulic modeling of the finished water transmission system should be used to establish sizing and selection requirements for the finished water pumping system.

8.8.2.7 Residuals Management

The sludge processing system consists of an equalization tank (EQ tank), gravity thickener, and sludge dewatering system. Residuals from the different treatment processes are routed to the EQ tank. The tank will be a pre-stressed concrete tank with a volume of 700,000 gallons. From the EQ tank, residuals are metered to the gravity thickener where they settle to the tank bottom. Supernatant is decanted and recycled back to the head of the plant. Thickened sludge is collected from the bottom of the thickener by a scraper and is pumped to the belt filter presses for dewatering. All dewatering equipment is housed in a sludge dewatering building. Six 2-meter belt filter presses are proposed for the project. Each press is fed by a single belt press feed pump of the progressive cavity type. Dewatered sludge from each belt filter press is discharged into a cake pump and routed to a trucking dock to be hauled offsite. A dedicated polymer system will be provided for each belt filter press which will enhance the dewatering performance of the presses. A schematic of the configuration of the proposed dewatering system is included in Appendix A.

The disposal method for dewatered sludge will be evaluated in preliminary design, and may include land application or landfilling. Dewatered sludge will not be disposed of to surfacewater bodies. Depending on the environmental requirements of the disposal method, its selection will affect the final design of the sludge processing system and the sludge disposal costs. Preliminary design will include identification of the preferred method and costs associated with sludge disposal.

A dedicated chemical building will be built on the site. The building will house all polymer and coagulant metering systems and storage containers. A separate room will be provided for the mixed oxidant generation system.

8.8.3 Conceptual Site Layout

Figure 8-8 is a conceptual site layout of the surfacewater treatment facility**.** It shows the major components of the site. Additional facilities required for the surfacewater treatment operations will include the following:

- Chemical building and storage tank facilities;
- Parking and access;
- Electrical feed and distribution system;

- Sanitary sewer service;
- Communication links (telephone, cable, telemetry);
- Stormwater management system;
- Landscaping and buffer zones; and
- Lighting.

8.8.4 Support Facilities

Operations, maintenance, and administration facilities will be constructed to support the overall operations of the water treatment plant and the staff who will work there. Two buildings are anticipated for this purpose. The design of the support facilities will be closely coordinated with the needs of the participating utilities.

Operations / Administration / Laboratory

A facility will be constructed to support the overall operations of the water treatment plant and the staff that will work there. The facility should have adequate office space for staff, a room from which the various plant components can be monitored and possibly controlled, a file storage and reference area, a room that could be used for meetings or breaks, and bathrooms. In addition, a room that could be used and equipped to serve as an on-site laboratory will be included.

Maintenance

A dedicated facility will be constructed to house various tools and equipment that would be needed to support the operation and maintenance of the treatment plant. This facility would include an adequate work space with benches, storage cabinets, common and specialty tools, spare equipment components and parts, and other materials that may be needed from time to time.

8.9 Transmission Systems

In order to deliver finished water produced by the new water supply facility to the users, a finished water transmission system will need to be evaluated, designed, and constructed. A conceptual transmission system for each wellfield was prepared for this element of the project. The transmission route typically assumes that water will be provided water to utilities at an approximate location within the respective service area, via easements acquired along public rights-of-way. Proposed pipe routes run along county or state roads for the purposes of this section.

Since a proposed facility would be a major water supply facility for the area, careful planning and consideration should be given to the location where the finished water supply should be routed and connected into the existing water distribution systems that are currently present in the local area. Actual pipeline routes and points of connection will be identified during design and permitting through coordination with the participating utility.

8.9.1 Conceptual Transmission Design

The conceptual design of the transmission piping is approximately based on the planning demands presented above and the overall capacity of the project. Hydraulic modeling and coordination with participating utilities will be performed during design and permitting to determine the actual transmission requirements. Actual transmission sizes will be based on maximum daily flows determined by participating utilities.

Typical flow velocities for average daily flows for large transmission systems are in the range of 5-5.5 feet per second. Maximum daily flows may increase the flow velocities to the range of 6-8 feet per second assuming a typical peaking factor of 1.5. The transmission design assumes that the existing local supply facilities will support peak needs for participating utilities, with limited support for peak flows provided by the new facility.

Normal pipeline life expectancy of 40 years exceeds the demands projected for this study. As previously mentioned, these water supply projects may provide water supplies for demands occurring after 2030. DIP is assumed as the pipeline material for the purposes of this report; other pipeline materials including cement-lined prestressed concrete and PVC may be evaluated during preliminary design. The pipe routes and sizes for the conceptual transmission systems are presented in the following sections.

Since the proposed pipe routes run along county or state roads, consideration should be given to potential road upgrades in the future. In order to avoid future pipe relocation, easement along the pipeline corridors should be acquired. Easement width will be 30 feet for pipes 16 inch or larger and 20 feet for smaller pipes.

8.9.2 North Sumter

Figure 8-9 shows the conceptual transmission route for the North Sumter surfacewater project. The locations of the connection points to the distribution systems of the different municipalities are approximate. The actual alignment will be determined during design and permitting. Finalizing the locations of the points of connection in later phases of the project would result in different pipe lengths and would also impact the conceptual cost estimate described in the following section. End users would be responsible for interconnection and distribution of combined water to their respective users. Table 8-5 summarizes the conceptual transmission system for the North Sumter project.

Table 8-5. Conceptual North Sumter Finished Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	feet	miles	acres
36	68,145	12.9	46.9
20	46,245	8.8	31.8
Total:	**114,390**	**21.7**	**78.7**

8.9.3 Holder

Figure 8-10 shows the conceptual transmission route for the Holder surfacewater project. The locations of the connection points to the distribution systems of the different municipalities are approximate. The actual alignment will be determined during design and permitting. Finalizing the locations of the points of connection in later phases of the project would result in different pipe lengths and would also impact the conceptual cost estimate described in the following section. End users would be responsible for interconnection and distribution of combined water to their respective users. Table 8-6 summarizes the conceptual transmission system for the Holder project.

Table 8-6. Conceptual Holder Finished Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	feet	miles	acres
48	8,440	1.6	5.8
42	69,460	13.2	47.8
36	109,230	20.7	75.2
24	69,660	13.2	48.0
12	13,090	2.5	6.0
Total:	**269,880**	**51.2**	**182.8**

8.9.4 Lake Rousseau Surfacewater

Figure 8-11 shows the conceptual transmission route for the Lake Rousseau surfacewater project. The locations of the connection points to the distribution systems of the different municipalities are approximate. The actual alignment will be determined during design and permitting. Finalizing the locations of the points of connection in later phases of the project would result in different pipe lengths and would also impact the conceptual cost estimate described in the following section. End users would be responsible for interconnection and distribution of combined water to their respective users. For this project, a raw water transmission system would also be required to deliver raw water from the intake location to the treatment plant. Tables 8-7 and 8-8 summarize the conceptual transmission systems for the Lake Rousseau project.

Table 8-7. Conceptual Lake Rousseau Raw Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	feet	miles	acres
48	22,704	4.3	13.6
Total:	**22,704**	**4.3**	**13.6**

Table 8-8. Conceptual Lake Rousseau Finished Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	feet	miles	acres
48	36,615	6.9	25.2
42	69,990	13.3	48.2
36	109,230	20.7	75.2
24	104,415	19.8	71.9
12	13,090	2.5	6.0
Total:	**333,340**	**63.2**	**226.5**

8.9.5 Blending

If finished water will not provide dedicated service, differences in the water chemistry between treated groundwater and treated surfacewater present potential issues that must be considered by the utility users in the planning process. This will require review of the treated surfacewater supply characteristics, existing groundwater supply of the utilities, the construction materials of the utilities' distribution systems, and the disinfection and corrosion issues associated with blending potable water from different sources.

The primary issues with blending are water quality as it relates to the disinfectant residual, DBP formation, and pipeline corrosion. Surfacewater contains higher levels of total organic compounds (TOC) and pathogens such as *Giardia*, and requires a different level of disinfection than groundwater. The TOC in surfacewater lends to increased levels of DBPs in comparison to groundwater. Potable water standards must be met in the transmission system in accordance with Rule 62-550.310, F.A.C., and meeting the disinfection and corrosion control needs in the Plant's transmission system will affect the design of the utility's blending facility.

After treated water from one source mixes with that from another source, changes in distribution system water chemistry can affect the stability of pipe coatings and disrupt the biofilms that protect pipes from corrosion. An increase in DBPs can also occur, either cumulatively or due to source interactions among multiple disinfectant types. The blending of groundwater and surfacewater must consider the combined water chemistry in the utility distribution system. Ultimately, potable water standards must be met in the blended water.

Each utility's source water and distribution system characteristics will be different. Therefore, it will be the responsibility of the utility to blend the water within their system and distribute water to their respective customers, and the determination of costs and the distribution infrastructure needed to properly blend groundwater and surfacewater falls with the individual utility. The method of blending and associated treatment processes to meet primary and secondary drinking water standards must also be determined by each utility.

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8.10 Conceptual Cost Estimate

The configuration of each supply facility was used to develop individual conceptual cost estimates according the methodology established in CH2M Hill (2004). The cost estimates are presented in this section.

8.10.1 Cost Definitions

The following elements are included in the cost estimates:

- Construction cost is the total amount expected to be paid to a qualified contractor to build the required facility.
- Non-construction capital cost is an allowance for construction contingency, engineering design, permitting and administration for the facility.
- Land cost is the market value of the land required for the facility.
- Land acquisition cost is the estimated cost of acquiring the land, exclusive of the land cost.
- Operation and maintenance cost is the estimated annual cost of operating and maintaining the facility when operated at average day capacity.
- Capital cost is the sum of construction cost, non-construction capital cost, land cost, and land acquisition cost.

[9] This assumes conventional rather than membrane treatment for surfacewater. Membrane processes are becoming increasingly common in the treatment of surfacewaters and may be considered during design as water quality data is gathered.

- Unit production cost is the annual lifecycle cost of the facility divided by the annual water production rate.
- Interest or discount rate is the time value of money criteria for the facility
- Equivalent annual cost is the annual lifecycle cost of the facility based on service life and time value of money criteria

8.10.2 Capital Cost Estimates

A summary of the conceptual capital cost for each water supply project option is presented in Tables 8-9 through 8-11, according to methodology and values established in CH2M Hill (2004). The non-construction capital cost was applied at 45 percent of the construction cost. This includes a 20% allowance for construction contingency (unknown conditions and/or changed field conditions) and a 25% allowance for engineering design, permitting, and administration. Easement acquisition costs of $0.75 per square foot (e.g., $32,760 per acre) are included in the capital cost. Land costs of $5,000 per acre are included for a 20-acre footprint for each water treatment facility, plus 18% acquisition cost.

Table 8-9. North Sumter Surfacewater: 10 mgd Capital Cost Estimate.

Item No.	Description	Total Cost (2009 dollars)
1	Raw Water Intake, Pump Station and Transmission	$7,916,000
2	Water Treatment and Storage Facility	$30,780,000
3	Transmission System	$22,902,000
4	Land and Easement Acquisition	$2,758,000
	Subtotal construction capital cost	$64,356,000
	Non-construction capital cost (45%)	$28,960,000
	Total:	**$93,316,000**

Table 8-10. Holder Surfacewater: 25 mgd Capital Cost Estimate.

Item No.	Description	Total Cost (2009 dollars)
1	Raw Water Intake, Pump Station and Transmission	$18,222,000
2	Raw Water Storage Reservoir	$93,081,000
3	Water Treatment and Storage Facility	$61,425,000
4	Transmission System	$64,877,000
5	Land and Easement Acquisition	$8,810,000
	Subtotal construction capital cost	$246,415,000
	Non-construction capital cost (45%)	$110,887,000
	Total:	**$357,302,000**

Notes:
1) The construction cost assumes the reservoir will be lined.
2) Actual MFL adoption and consideration of supplemental sources will affect reservoir costs.

Table 8-11. Lake Rousseau Surfacewater: 25 mgd Capital Cost Estimate.

Item No.	Description	Total Cost (2009 dollars)
1	Raw Water Intake and Pump Station	$16,682,000
2	Raw Water Transmission	$8,725,000
3	Water Treatment and Storage Facility	$61,425,000
4	Transmission System	$80,993,000
5	Land and Easement Acquisition	$8,025,000
	Subtotal construction capital cost	$175,850,000
	Non-construction capital cost (45%)	$79,132,000
	Total:	**$254,982,000**

8.10.3 Operation and Maintenance Cost Estimates

O&M include labor, power, and chemical costs necessary for operation; and R&R for equipment maintenance and membrane replacement. Labor costs were based on an estimated workforce needed to operate the facility. Chemical costs were based on estimated usage and vendor quotes. Power costs were estimated based on current rates and equipment operation needs. R&R were based on a combination of annual needs and project lifecycle of 30 years. For purposes of this report this is estimated to be 1% of the construction cost for the water treatment and storage facilities, and 0.5% of the construction cost for the transmission system. 0.5% is used for the reservoir facilities. The operating costs for this desalination process are considerable due to high power consumption and periodic membrane replacements. Tables 8-12 through 8-14 provide a summary of the O&M costs for the water supply project options.

Table 8-12. North Sumter Surfacewater: 10 mgd Operation and Maintenance Estimate.

Item No.	Description	Estimated Annual Costs
1	Labor	$850,000
2	Chemicals	$1,000,000
3	Power	$750,000
4	Equipment Renewal & Replacement	$337,000
5	Transmission Renewal & Replacement	$115,000
	Total:	**$3,052,000**

Notes:

1) *O&M costs assume continuous operation; however, the facility is expected to provide conjunctive supply. Actual MFL adoption will determine whether this facility can be a year-round or conjunctive supply.*

Table 8-13. Holder Surfacewater: 25 mgd Operation and Maintenance Estimate.

Item No.	Description	Estimated Annual Costs
1	Labor	$1,250,000
2	Chemicals	$2,400,000
3	Power	$1,110,000
4	Equipment Renewal & Replacement	$1,261,000
5	Transmission Renewal & Replacement	$449,000
	Total:	**$6,470,000**

Notes:
1) O&M costs include %0.5 renewal and replacement for the raw water storage reservoir.

Table 8-14. Lake Rousseau Surfacewater: 25 mgd Operation and Maintenance Estimate.

Item No.	Description	Estimated Annual Costs
1	Labor	$1,250,000
2	Chemicals	$2,400,000
3	Power	$1,110,000
4	Equipment Renewal & Replacement	$781,000
5	Transmission Renewal & Replacement	$324,000
	Total:	**$5,865,000**

8.10.4 Unit Production Cost Estimates

Unit production cost is a function of the capital costs, debt service, annual O&M costs and the amount of water produced. For this analysis, the debt service is estimated based on a 30-year project lifecycle at 4.625% interest (2009 federal discount rate for water resource projects). Tables 8-15 through 8-17 provide a summary of these costs for each water supply project option.

Table 8-15. North Sumter: 10 mgd Unit Production Cost Estimate.

Item No.	Description	Total Cost
1	Total Capital Cost	$93,316,000
2	Annual O&M Cost	$3,052,000
	Equivalent Annual Cost:	**$8,864,126**
	Unit Production Cost ($/kgal)	**$2.43**

Notes:
1) Unit production costs assume continuous operation; however, the facility is expected to provide conjunctive supply. Actual MFL adoption will determine whether this facility can be a year-round or conjunctive supply.
2) The construction cost within the total capital cost includes a 20% contingency.
3) 30-year amortization at 4.625%.

Table 8-16. Holder: 25 mgd Unit Production Cost Estimate.

Item No.	Description	Total Cost
1	Total Capital Cost	$357,302,000
2	Annual O&M Cost	$6,470,000
	Equivalent Annual Cost:	**$28,724,319**
	Unit Production Cost ($/kgal)	**$3.15**

Notes:
1) *The construction cost within the total capital cost includes a 20% contingency.*
2) *30-year amortization at 4.625%.*

Table 8-17. Lake Rousseau: 25 mgd Unit Production Cost Estimate.

Item No.	Description	Total Cost
1	Total Capital Cost	$254,982,000
2	Annual O&M Cost	$5,865,000
	Equivalent Annual Cost:	**$21,746,386**
	Unit Production Cost ($/kgal)	**$2.38**

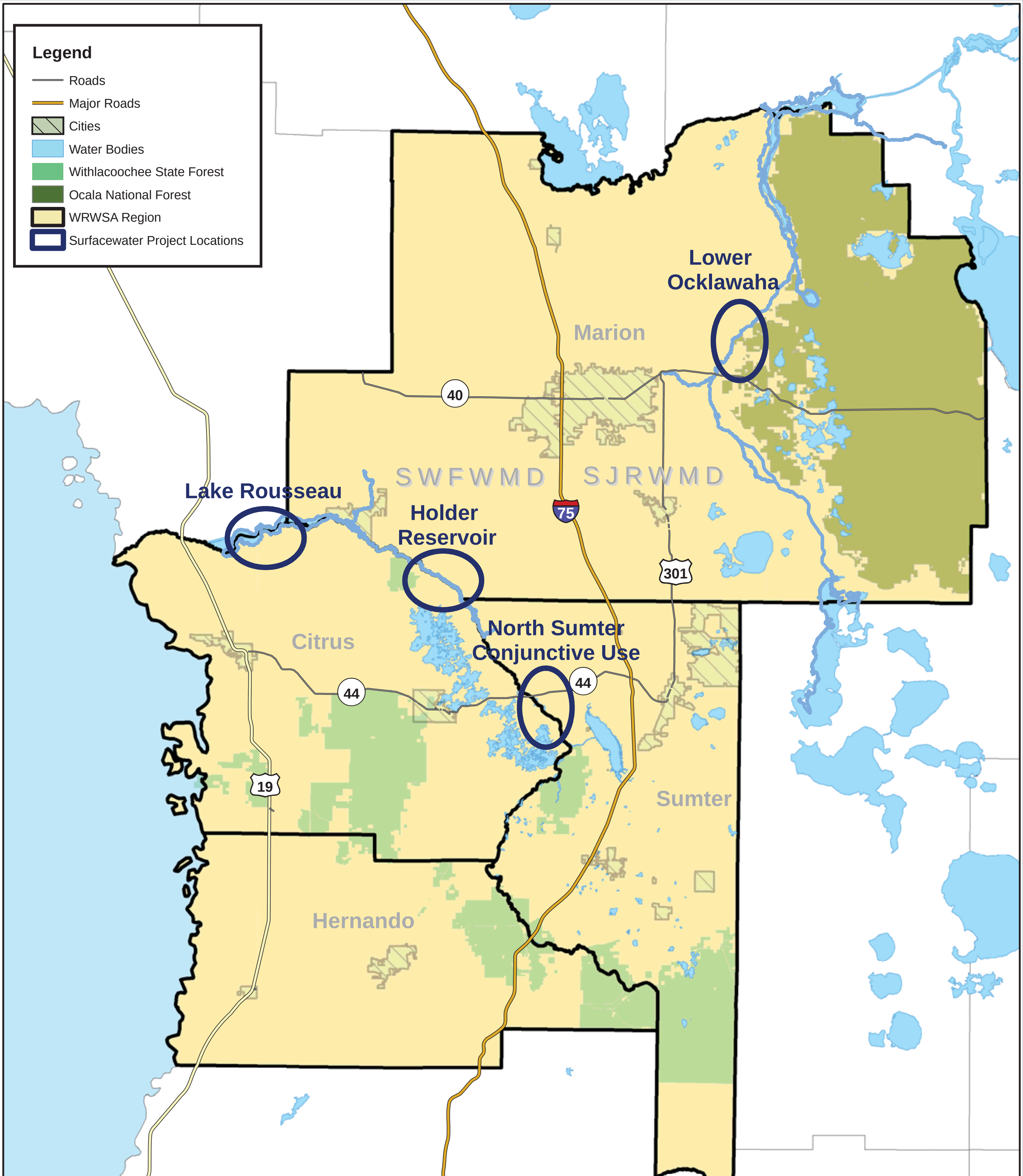
Notes:
1) *The construction cost within the total capital cost includes a 20% contingency.*
2) *30-year amortization at 4.625%.*

Unit production costs for the Lower Ocklawaha River project in Marion County were estimated at $3.04 per kgal for an 83.85 mgd project serving multiple counties (SJRWMD, 2009). Shorter transmission distances for a smaller Lower Ocklawaha concept serving members in Marion County would likely reduce this unit production cost.

8.11 Long-Range Planning Considerations

Long transmission distances exist between most of these locations and the projected demand areas. The length of transmission in some cases is such that economies of scale associated with service to multiple users will be diminished by the need for transmission. For example, a small or conjunctive withdrawal from the Withlacoochee River reach upstream of Holder is likely to prove more cost-effective for northeastern Sumter County utilities than a similar withdrawal from Lake Rousseau, which would require about 15 miles of additional transmission and regional-scale participation.

Fresh groundwater sources have been identified in the vicinity of the river systems, as discussed in Chapter 6. The identification of these groundwater sources provides opportunities for members to deal with the transmission distances to alternative sources in an incremental manner; the dispersed groundwater projects could transmit future river supplies through their transmission systems. Therefore, long-range planning for surfacewater development should consider dispersed groundwater development in the vicinity of the river systems.



Note: The Lower Ocklawaha River is a SJRWMD project.

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PROJECT: 0468 - Withlacoochee Phase II

Figure 8-1
Potable Surfacewater Project Options

ORIGINAL DATE: 01-06-10
REVISION DATE: NA
JOB NUMBER: 0468
FILE NAME: Figure 8-1...mxd
GIS OPERATOR: DR

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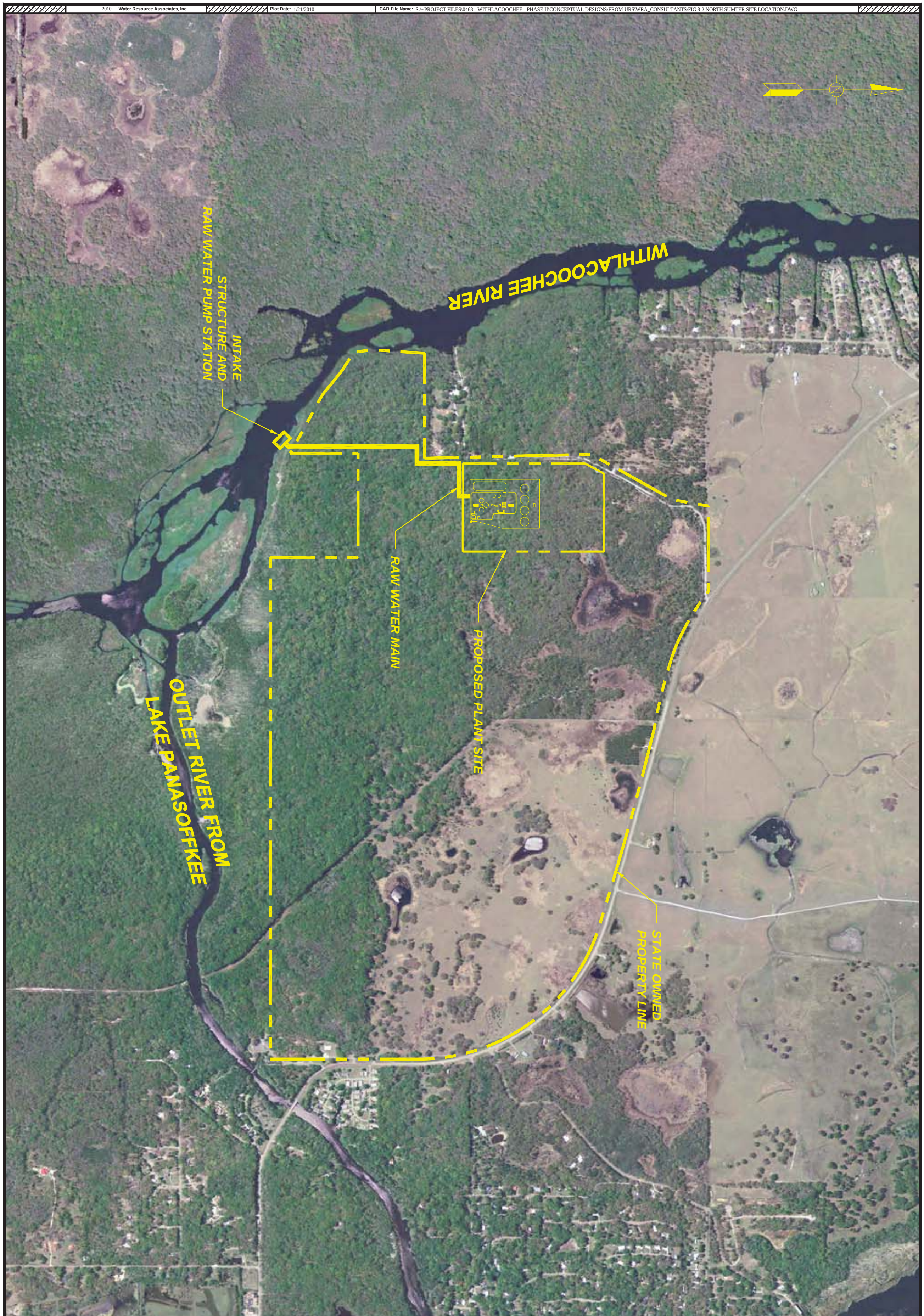
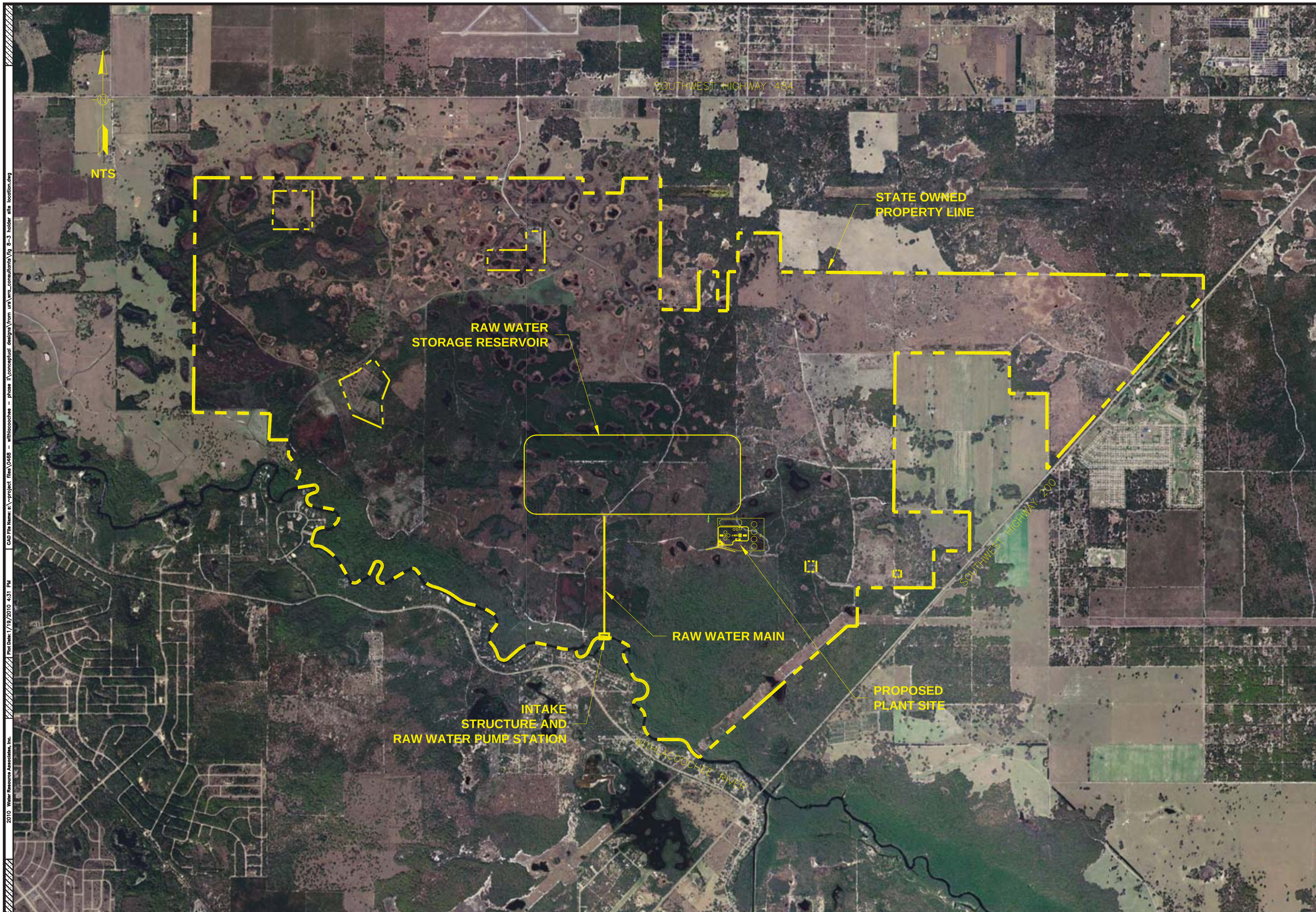
1 inch equals 10 miles



PROJECT	DRAWING	REVISIONS	GENERAL
WRWSA PHASE II WATER SUPPLY PLAN	NORTH SUMTER PROJECT LOCATION	1. 2. 3. 4. 5. 6. 7.	Original Date: 07/16/07 Last Modified: 01/19/10 Scale: NTS FIGURE 8-2
JOB #: 0468 SEC:____ TWN:____ RNG:____	DESIGNED:____ DRAWN:____ APPROVED:____		FL P.E. #____



PROJECT	DRAWING	REVISIONS	GENERAL
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JOB #:0488/12007377	DESIGNED: JRS DRAWN: TS APPROVED: DW		FIGURE 8-3

FL P.E. #

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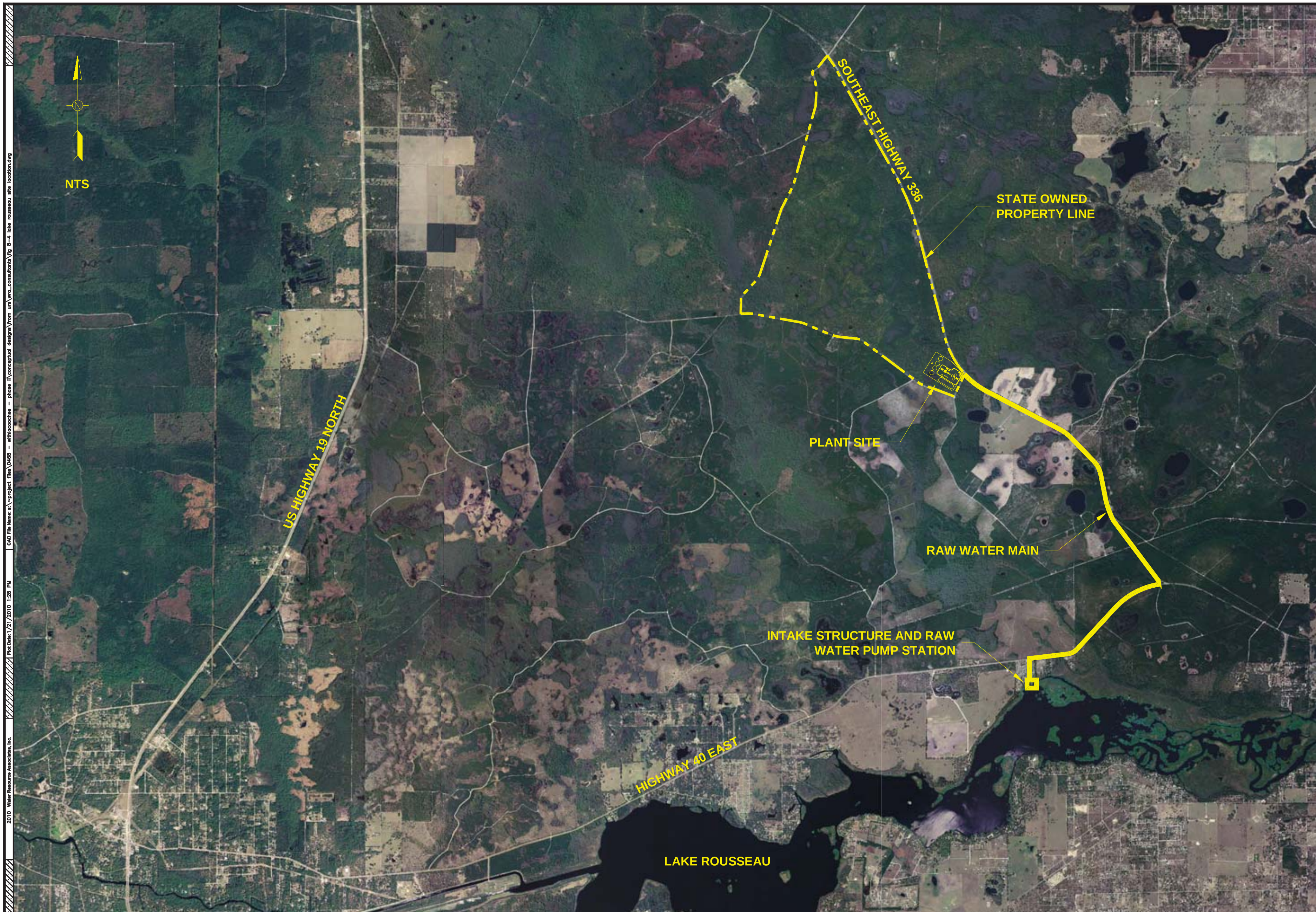

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PROJECT

WRWSA PHASE II
WATER SUPPLY PLAN

JOB #0468/12007377

DRAWING

LAKE ROUSSEAU
PROJECT LOCATION

DESIGNED: JRS DRAWN: TS APPROVED: DW

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Original Date: 7/16/07
Last Modified: 1/19/10
Scale: NTS

FIGURE
8-4

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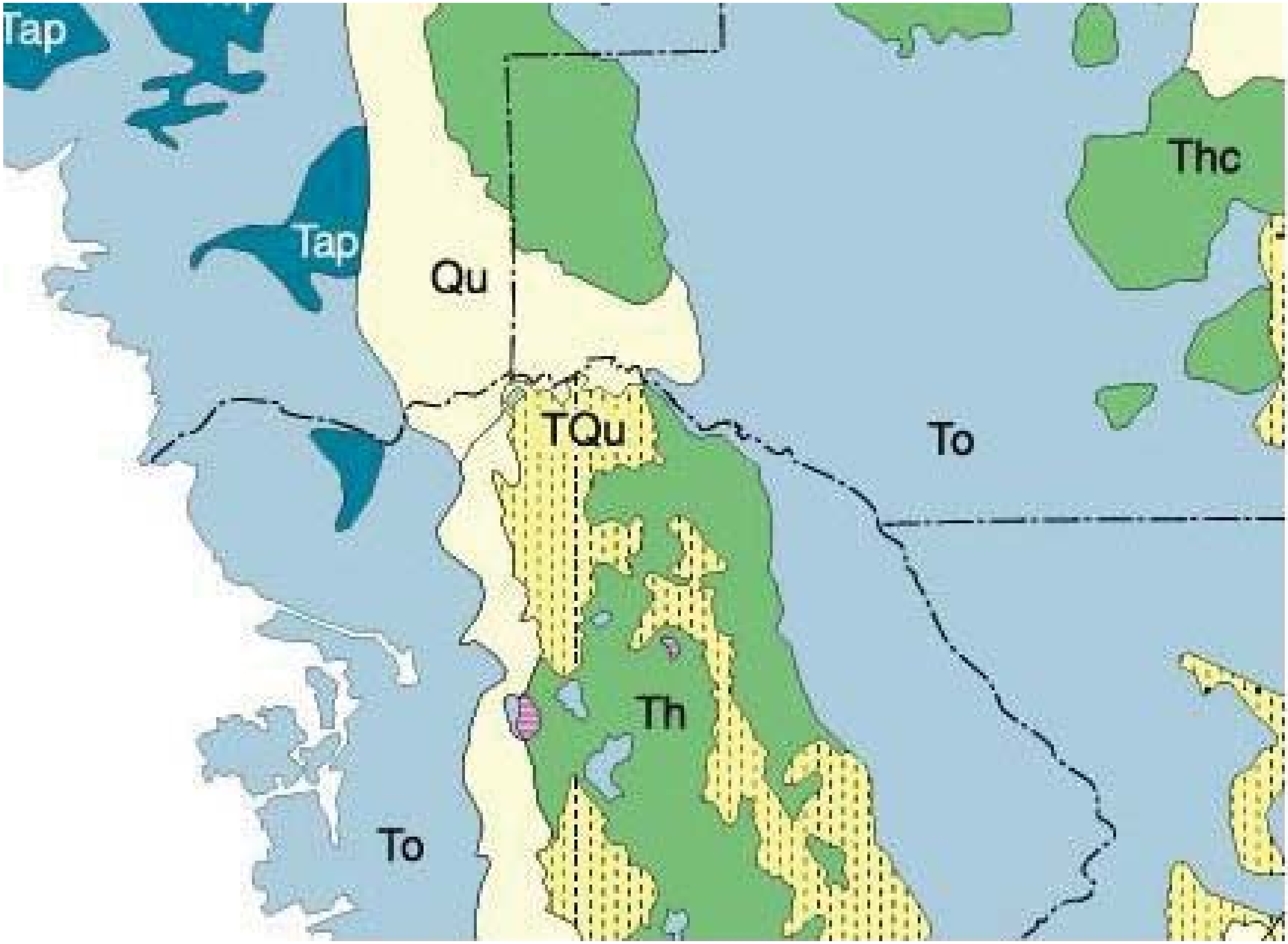

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PROJECT: 0468 – Withlacoochee Phase II

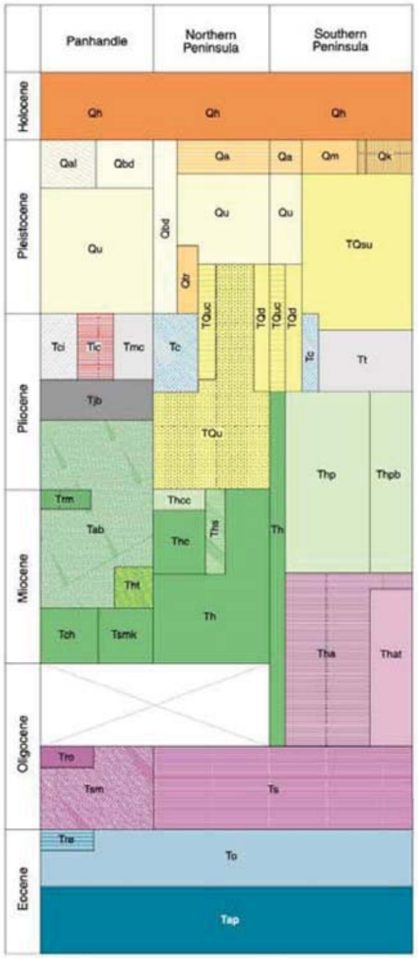
Figure 8-5
Holder Vicinity
Surface Geology

ORIGINAL DATE: 01-19-10
REVISION DATE: N/A
JOB NUMBER: 0468
GIS OPERATOR: DR

SCALE ON MAP

by

Thomas M. Scott, P.G.#99, Kenneth M. Campbell, Frank R. Rupert
Jonathan D. Arthur, Thomas M. Missimer
Jacqueline M. Lloyd, J. William Yon, and Joel G. Duncan



Quaternary

Holocene

- Qh — Holocene sediments

Pleistocene/Holocene

- Qal — Alluvium
- Qbd — Beach ridge and dune
- Qu — Undifferentiated sediments

Pleistocene

- Qa — Anastasia Formation
- Qk — Key Largo Limestone
- Qm — Miami Limestone
- Qtr — Trail Ridge sands

Tertiary/Quaternary

Pliocene/Pleistocene

- TQsu — Shelly sediments of Plio-Pleistocene age
- TQu — Undifferentiated sediments
- TQd — Dunes
- TQuc — Reworked Cypresshead sediments

Tertiary

Pliocene

- Tc — Cypresshead Formation
- Tci — Citronelle Formation
- Tmc — Miccosukee Formation
- Tic — Intracoastal Formation
- Tt — Tamiami Formation
- Tjb — Jackson Bluff Formation

Miocene/Pliocene

- Thcc — Hawthorn Group, Coosawhatchie Formation, Charlton Member
- Thp — Hawthorn Group, Peace River Formation
- Thpb — Hawthorn Group, Peace River Formation, Bone Valley Member

Miocene

- Trm — Residuum on Miocene sediments
- Tab — Alum Bluff Group
- Th — Hawthorn Group
- Thc — Hawthorn Group, Coosawhatchie Formation
- Ths — Hawthorn Group, Statenville Formation
- Tht — Hawthorn Group, Torreya Formation
- Tch — Chatahoochee Formation
- Tsmk — St. Marks Formation

Oligocene/Miocene

- Tha — Hawthorn Group, Arcadia Formation
- That — Hawthorn Group, Arcadia Formation, Tampa Member

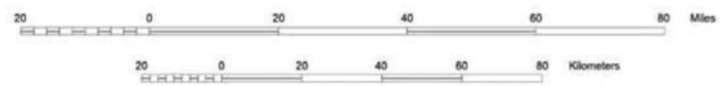
Oligocene

- Tro — Residuum on Oligocene sediments
- Ts — Suwannee Limestone
- Tsm — Suwannee Limestone - Marianna Limestone undifferentiated

Eocene

- Tre — Residuum on Eocene sediments
- To — Ocala Limestone
- Tap — Avon Park Formation

See OFR- 80 for description of mapped units.



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Albers Conic Equal-Area Projection

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PROJECT: 0468 – Withlacoochee Phase II

Figure 8-6
Holder Vicinity
Geologic Legend

ORIGINAL DATE: 01-19-10
REVISION DATE: N/A
JOB NUMBER: 0468
GIS OPERATOR: DR

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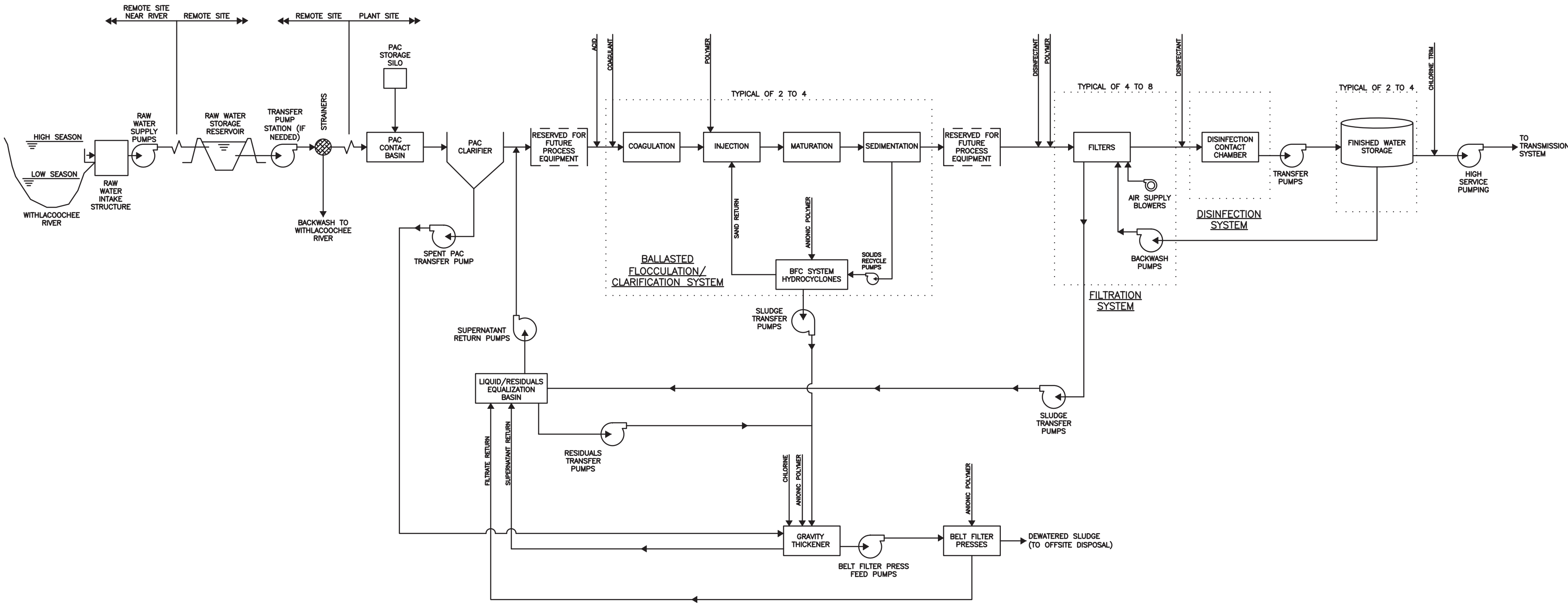
SCALE ON MAP



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GENERAL	REVISIONS	DRAWING	PROJECT
Original Date: 7/16/07 Last Modified: 1/20/10 Scale: NTS FIGURE 8-7	1. 2. 3. 4. 5. 6. 7.	SURFACE WATER TREATMENT PROCESS FLOW DIAGRAM DESIGNED: BH DRAWN: TS APPROVED: DW	WRWSA PHASE II WATER SUPPLY PLAN JOB #:0468/12007377

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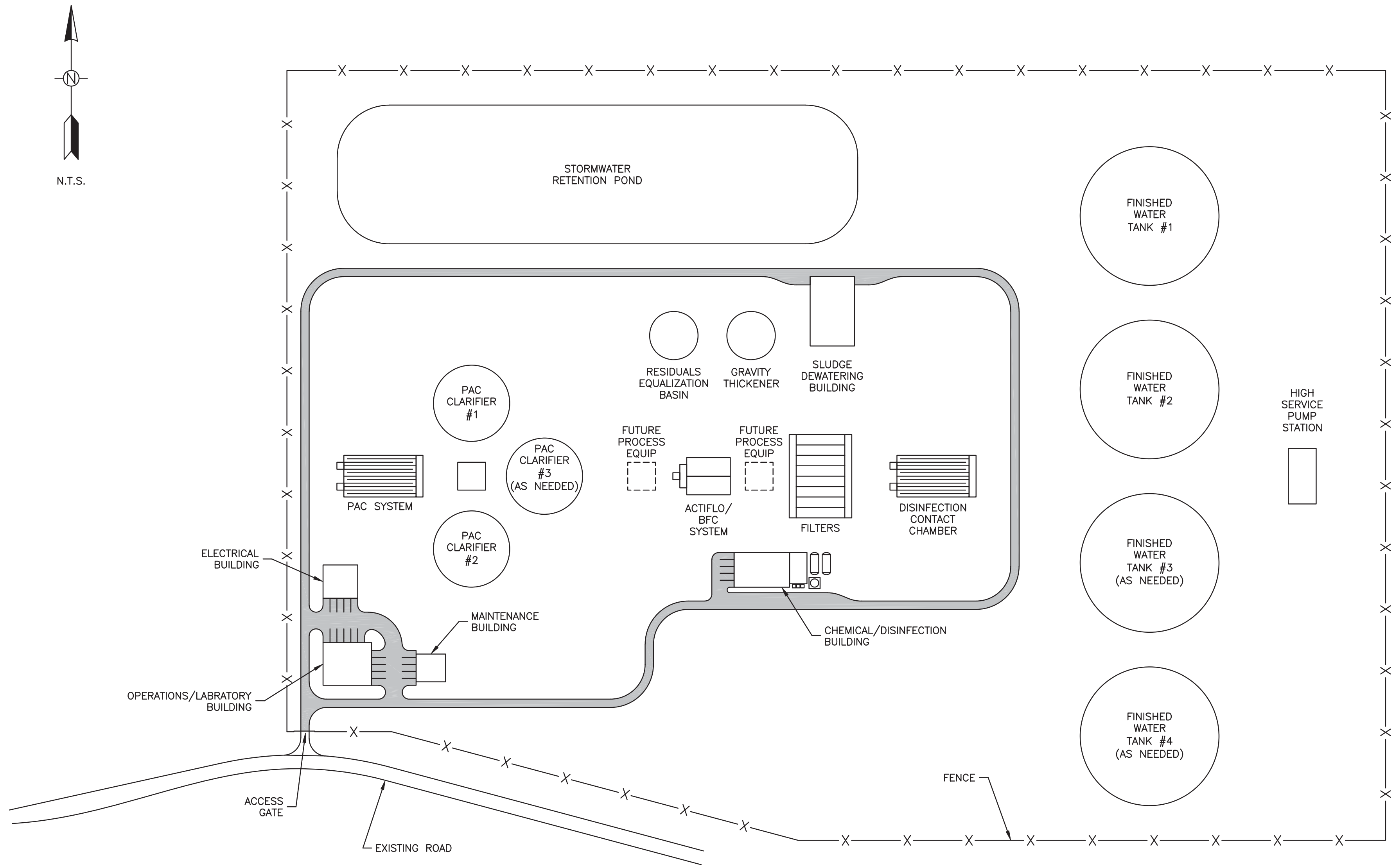

PROJECT: WRWSA PHASE II WATER SUPPLY PLAN

JOB #:0468/12007377

DRAWING: SURFACE WATER TREATMENT FACILITY LAYOUT

DESIGNED: JRS DRAWN: TS APPROVED: DW

REVISIONS: 1. 2. 3. 4. 5. 6. 7.

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FIGURE 8-8

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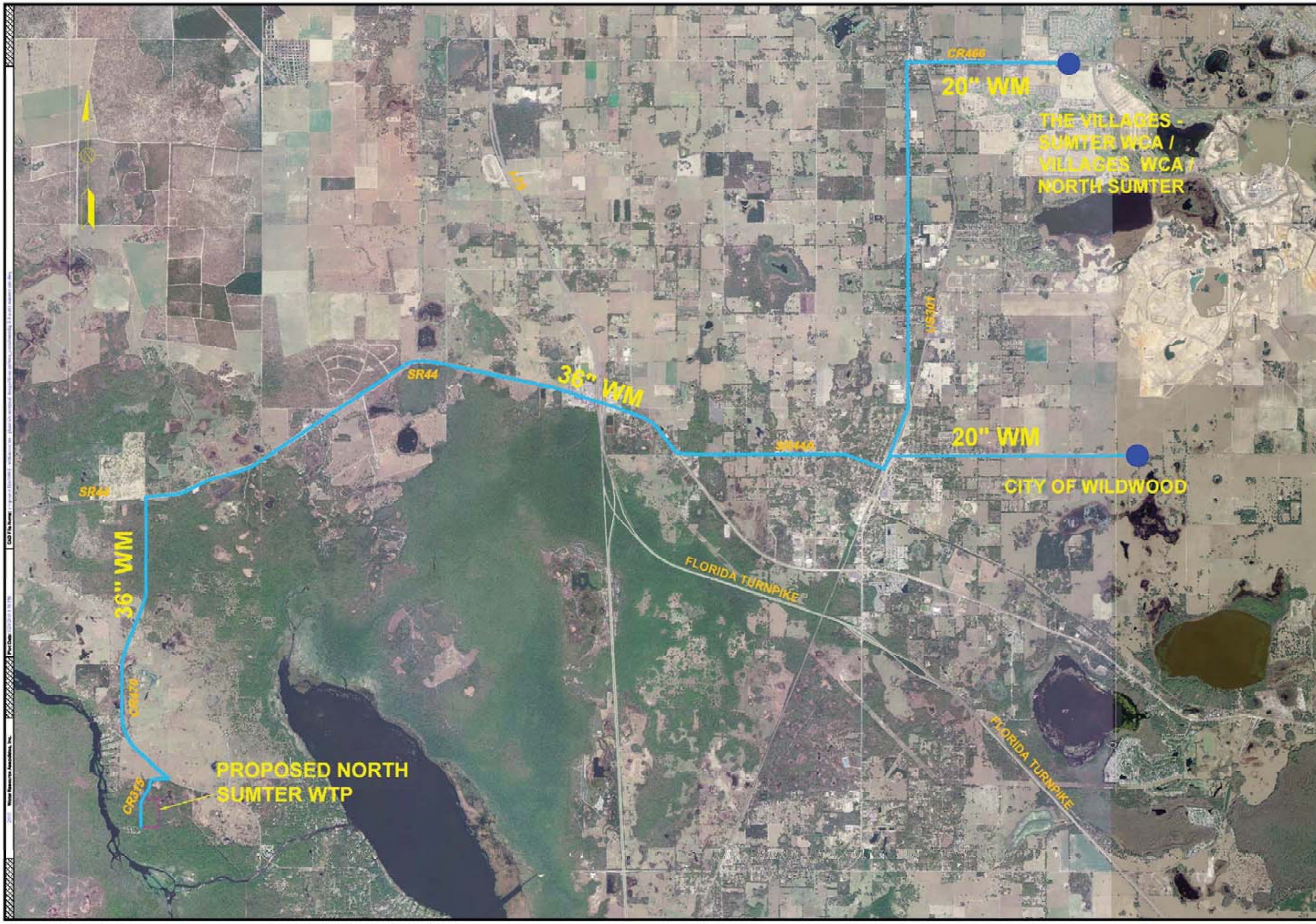

PROJECT
WRWSA PHASE II
WATER SUPPLY PLAN

DRAWING
CONCEPTUAL NORTH SUMTER
TRANSMISSION ROUTE

REVISIONS

GENERAL

FIGURE

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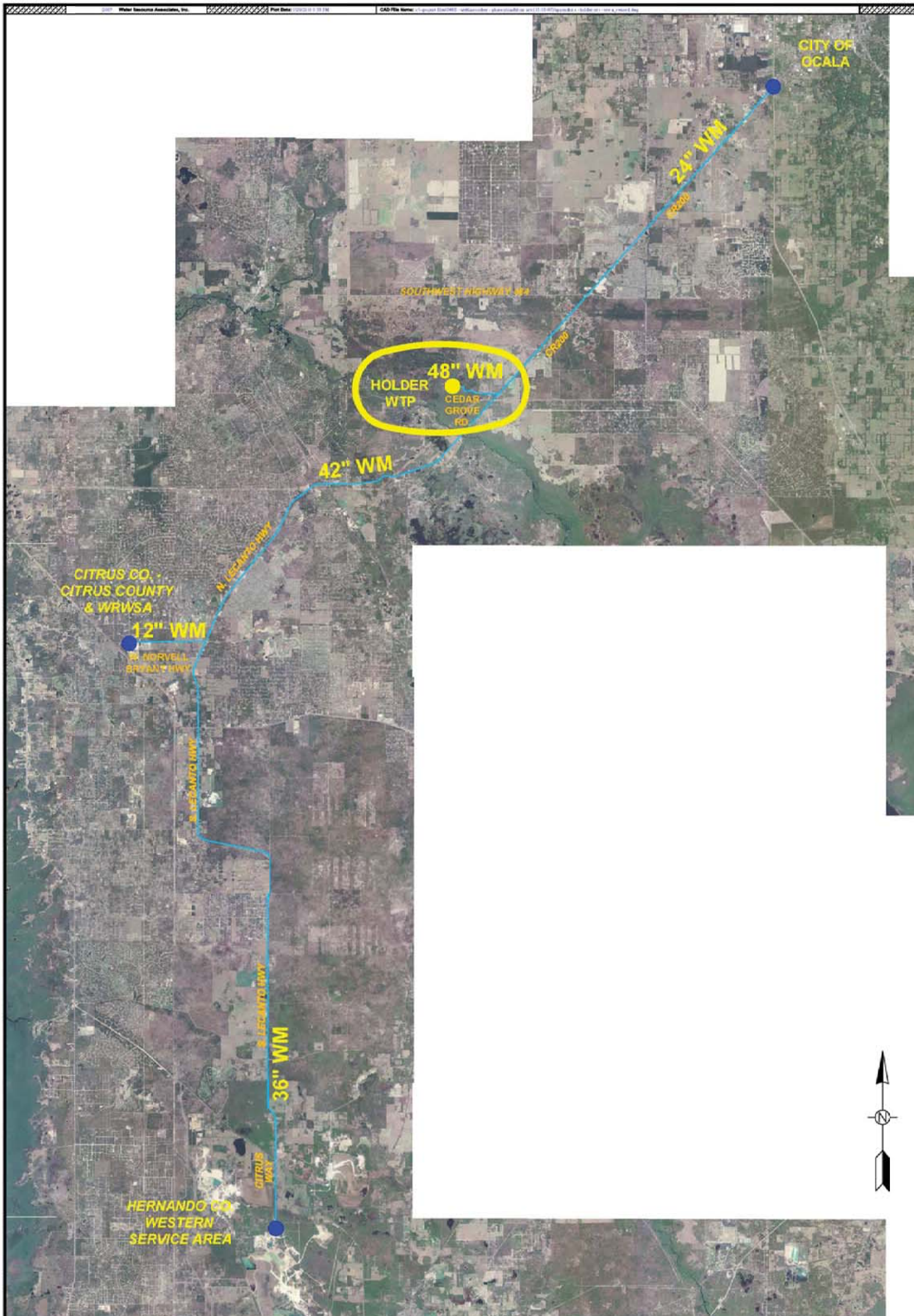

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		WRWSA PHASE II WATER SUPPLY PLAN	CONCEPTUAL HOLDER TRANSMISSION ROUTE	1. 2. 3. 4. 5. 6. 7.	Original Date: 07/16/07
					Last Modified: 01/08/10
					Scale: NTS
		JOB #: 0468	DESIGN: JBS DRWN: JBS CHKD: EH APVD: DAW		FIGURE 8-10

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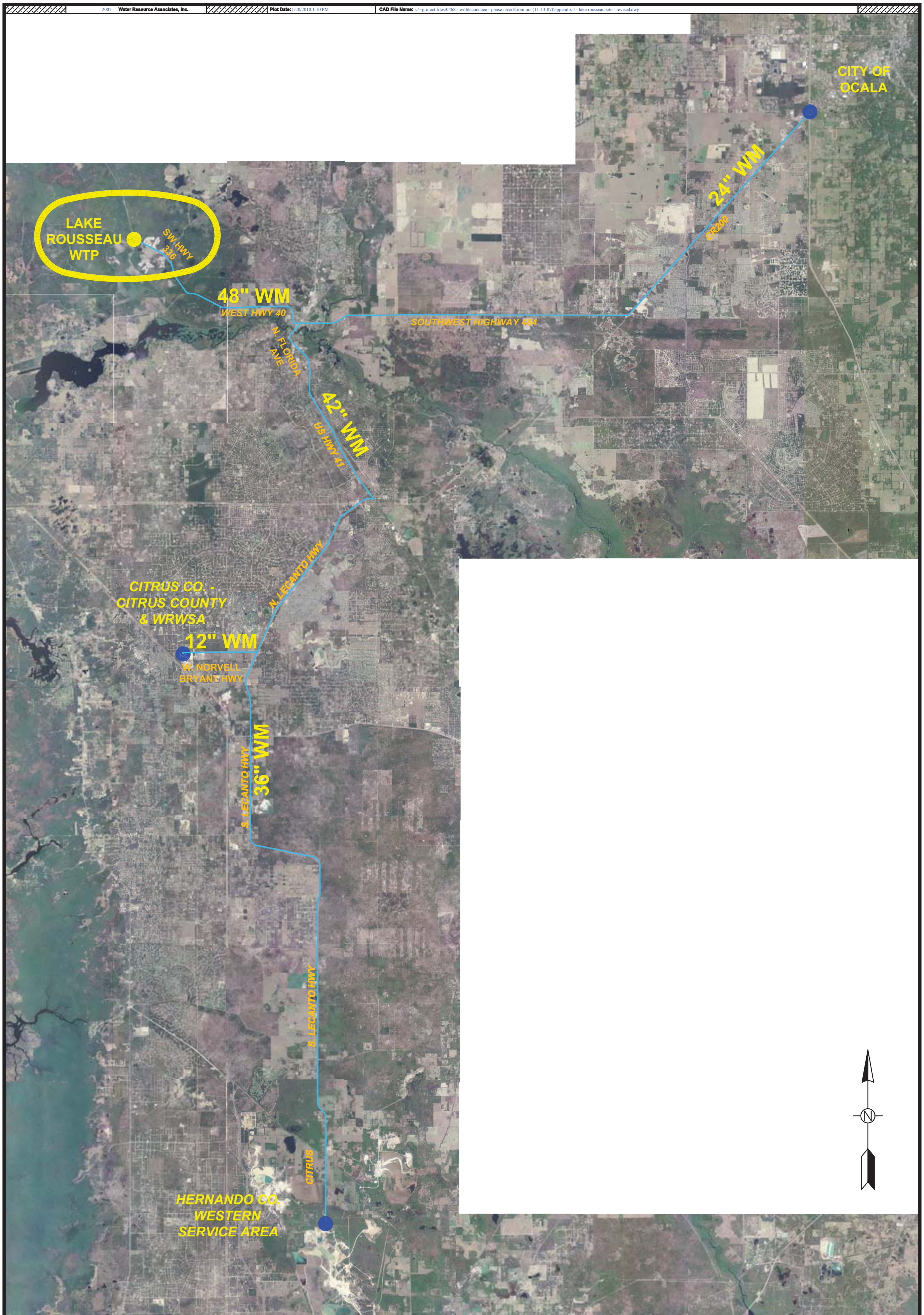

PROJECT	DRAWING	REVISIONS	GENERAL
WRWSA PHASE II WATER SUPPLY PLAN	CONCEPTUAL LAKE ROUSSEAU TRANSMISSION ROUTE	1. 2. 3. 4. 5. 6. 7.	Original Date: 07/16/07 Last Modified: 01/08/10 Scale: NTS
JOB #: 0468	DESIGN: JRS DRWN: JRS CHKD: BH APVD: DAW		FIGURE 8-11

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Chapter 9 – Seawater Desalination Project Option

9.0 Key Points

Key Points

- Seawater is a stable and drought-resistant water supply source that is becoming increasingly attractive as the availability of traditional supplies diminishes. Seawater contains high concentrations of minerals (salts) that must be removed prior to its use for water supply.
- A seawater desalination project option is located near the Progress Energy Crystal River Power Plant (Power Plant) in Citrus County. The project would provide 15 mgd of potable alternative water supply to potential users located in Citrus and Hernando Counties.
- The project would withdraw raw seawater from the Cross Florida Barge Canal about 4 miles north of the Power Plant.
- The concentrate removed from the seawater would be mixed with the Power Plant cooling water discharge for disposal. The cooling flow will dilute the concentrate discharge to environmentally acceptable levels.
- The desalination process would utilize pressurized reverse osmosis membranes to remove the salts from the seawater. The operating costs for this process are considerable due to high power consumption and periodic membrane replacements.
- The conceptual water production cost for the project is $4.27 per thousand gallons. The conceptual transmission distance is 37 miles.
- Operating and transmission costs account for over 75% of the water production cost for this option.

9.1 The Role of Potable Alternative Water Supply in the WRWSA Region

Chapters 1, 3, 4 and 5 demonstrated that existing permitting allocations and available local groundwater resources, conservation and reclaimed water will be sufficient to serve portions of the projected 2030 groundwater demand in the WRWSA. Significant adjustments to these projected demands are also anticipated region, due to regulatory and incentive measures which have been proactively implemented by the SWFWMD and the SJRWMD in order to extend the lifetime of fresh groundwater. These measures are detailed in Chapter 4 for water conservation and Chapter 5 for beneficial reuse in the WRWSA.

Dispersed fresh groundwater project options were presented in Chapter 6 as opportunities for utilities facing local groundwater resource limitations to continue to rely on groundwater for potable supply. A number of the wellfield options have capacities that exceed identified demands so it is unlikely that all of those projects will be implemented within the 2030 planning horizon.

Water conservation, beneficial reuse, and dispersed groundwater all provide more cost-effective approaches to water supply in the WRWSA region than potable alternative water supplies. There are significant cost and implementation challenges associated with these strategies, but those hurdles pale in comparison to the costs and challenges of developing potable alternative water supplies. The rural character of the region and relative abundance of water resources

suggests that smaller communities in the region will likely be able to rely on conservation, beneficial reuse, and planned groundwater for the long haul. The individual strategies will depend on the resources available to each specific utility and the actual rate of population growth.

Growth rates can change quickly and dramatically in rural areas such as the WRWSA region. Flexible strategies are needed within the 20-year planning horizon and beyond, because potable alternative water supplies can take an extremely long time (10-12 years) and are very costly to implement. For the purposes of this plan, potable alternative water supply strategies target larger population centers in the WRWSA where conservation, beneficial reuse, and dispersed groundwater may not meet water needs for the long haul. This strategy can be adjusted over time as growth occurs and additional data is gathered.

There are three service areas in the WRWSA with permitted water allocations exceeding 15 mgd:

- The Villages
- Hernando County (Western Service Area)
- City of Ocala

Of these, The Villages is projected to build out prior to 2030. The City of Ocala's long range water demand will depend on the rate of infill and commercial development and whether the utility service area expands. The capacity of the dispersed groundwater projects generally exceeds the projected water demands of these two utilities in 2030 and both of these communities are located closer to the Lower Ocklawaha River and Withlacoochee River system than they are to the Gulf of Mexico. Hernando County (Western Service Area) remains as a logical service area for a seawater desalination project if and when one is needed.

When a potable alternative water supply is developed, smaller communities in close proximity to the source may elect to be served. For this reason, Citrus County is included in the alternative water supply strategy for the seawater desalination project.

9.2 Seawater Desalination Project Description

Seawater is a stable and drought-resistant water supply source that is becoming increasingly attractive as the availability of traditional supplies diminishes. The concept of locating a seawater desalination facility with a once through coastal power plant was evaluated and proposed by the SWFWMD in 1995. Since that time, the SJRWMD and the SFWMD have also investigated and recommended the feasibility of locating a seawater desalination facility with a once through power plant. The synergy of this combined operation is the ability to utilize the existing in-place discharge system (used for cooling purposes) employed by the power plant to meet the discharge needs for the desalination facility. The result is a more cost-efficient and environmentally acceptable seawater desalination process.

The Crystal River Power Plant, a Progress Energy facility, is located on the Gulf of Mexico in Citrus County. Figure 9-1 shows an aerial photograph of the Power Plant. It includes four coal-fired generating units with a combined generating ability of 2,302 MW, and a nuclear unit with a capability of 825 MW. The Power Plant is currently undergoing an expansion to upgrade its generating facilities.

The major seawater flows associated with the Plant are once-through cooling flow for the two coal-fired units (Units 1 and 2) and the nuclear unit (Unit 3), which have a combined maximum permitted discharge flow of 1,898 mgd. Units 1 and 2, totaling 865 MW, were constructed in the 1960's and the nuclear unit was constructed in 1977. These units that utilize a common seawater intake and discharge system through a canal that discharges the cooling flow beyond the shoreline. This cooling flow would be essential to dilution of concentrate discharge from any desalination facility.

Florida Progress (now Progress Energy) was actively involved in the research and development of a co-located desalination facility as part of the SWFWMD feasibility work in the 1990's and subsequently was a qualified bidder to Tampa Bay Water in the development of the first large scale seawater desalination facility in Florida. Any project in the vicinity of the Power Plant would of course require their cooperation and participation.

The desalination facility would be located near the Power Plant site. The concentrate removed from the seawater would be mixed with the Power Plant cooling water discharge for disposal. The cooling flow will dilute the concentrate discharge to environmentally acceptable levels.

9.3 Design Capacity

It is anticipated at this time that the WTP may serve communities located in Citrus and Hernando Counties; however, more cost-effective water sources than seawater are likely to be sufficient to meet water supply needs in the WRWSA region to 2030. Water demands have not been projected for this region on a utility-by-utility basis beyond 2030,[1] so general long-range planning values are used to determine a possible design capacity for the seawater desalination project. These long-range values are roughly proportional to the permitted allocation in each service area. Table 9-1 below provides a summary of these potential consumers and the long-range planning demands.

Table 9-1. Potential Users for Seawater Desalination Facility.

#	Permitted Service Area	ADF mgd
1	Citrus County – Citrus County / WRWSA	2.50
2	Hernando County Utilities – West Hernando	10.00
3	Reserve Capacity	2.50
	Total:	**15.00**

9.4. Seawater Source and Intake Location

The raw water source will be seawater taken from the Gulf of Mexico. Seawater contains high concentrations of minerals (salts) that must be removed prior to its use for water supply, creating a concentrate which must be safely disposed of. The Power Plant discharge canal will serve to dilute the concentrate discharge. The amount of this source will likely only be limited by the utility demands that the project will serve. Assuming a 16:1 dilution ration for the concentrate

[1] Reference water demand projections to 2055 were included in Phase I, but they were developed on a county-by-county basis.

effluent, as required by FDEP for the Tampa Bay facility, the total capacity of the desalination facility could be as high as 85 mgd of potable water production.

Seawater, as a source water, does not require a water use permit from the SWFWMD at this time and is not limited by any regulatory limitations other than the concentrate disposal regulations imposed by the FDEP. At this time, the withdrawal location is anticipated to be the Cross Florida Barge Canal seaward of the Inglis Dam. Since this location receives large freshwater discharges from Lake Rousseau, water quality data in the barge canal were reviewed to identify potential issues associated with this location.

Salinity (total dissolved solids measured in ppt) is the most significant water quality parameter for desalination, due to the high operating pressures needed to drive saltwater through the membranes.[2] The power needed to achieve the pressures drives the high cost of desalination. The salinity in the Barge Canal usually runs at 15 to 20 ppt, as shown on Figure 9-2. It can vary from completely fresh (0 ppt) to full strength seawater (35 ppt). This is due to the regulation schedule of the Inglis Dam which routes freshwater discharges from Lake Rousseau to the Barge Canal. When discharges occur, they reduce salinity in the Barge Canal. They also create a wedge effect in the Barge Canal where the saltier water remains at depth and the fresher water flows at the surface.

The usual range of 15 to 20 ppt that occurs in the Canal is highly desirable in comparison to full seawater, because fresher estuarine (mesohaline) waters reduce operating costs. The proposed Gulf Coast desalination project drawing from the Lower Anclote River (JEI, 2008) takes advantage of fresher estuarine waters than those in the Gulf of Mexico. However, addressing the variability in Barge Canal source water will be a design and operational challenge. There are few if any operating desalination facilities in the world that draw such variable source water (MWH, pers. comm. 2008). The vast amount of freshwater discharging from Lake Rousseau affects Gulf of Mexico salinities well beyond the Power Plant and Barge Canal (JEI, 2007). Additional evaluation will be needed to determine the implications of the source variability and evaluate secondary intake options if needed.[3] There are submerged springs in the Barge Canal which could be considered as intake locations. Design assumptions for the project which are relevant to raw water quality are mentioned in subsequent sections of the chapter.

9.5 Conceptual Facility Design

This section presents the conceptual facility design for the seawater desalination project. The facility will include treatment operations and processes to efficiently and cost effectively convert seawater into potable (finished) water with quality meeting all requisite local, state, and federal regulations. The design and permitting for the facility will identify and evaluate potential project specific issues, including raw and discharge water quality. Site specific considerations related

[2] Water quality constituents requiring pre-treatment to avoid fouling the operating membranes are also significant parameters for desalination. These constituents include dissolved organic material, algae, and suspended solids. Raw water concentrations of these parameters normally increase dramatically during freshwater discharge events into estuarine waters. Water quality for these constituents in the Barge Canal was not reviewed for this chapter, but could have a dramatic impact on pre-treatment needs for the facility.

[3] See Note 2.

to land acquisition, requisite permitting issues of the F.A.C., the SWFWMD, and local ordinances and regulations are not addressed herein.

9.5.1 Basis of Design

In Florida, FDEP has jurisdiction over the drinking water standards described in Chapter 62-520 and 62-550, F.A.C. The primary drinking water standards, which are health-based and include the control of pathogens, are described in Rule 62-550.310, F.A.C., while the Secondary Drinking Water Standards are contained in Rule 62-550.320. Secondary standards generally apply to the aesthetic qualities of water (appearance, taste, and odor) that are typically desired for public acceptance and use. No known health effects are currently associated with the secondary standards. All primary and secondary standards are enforced for potable water supplies and, as such, compliance with all standards will occur when planning for and designing the new water supply facility.

Minimum capacity criteria for water supply facilities are described in Chapter 62-550, F.A.C. FDEP has jurisdiction over these criteria, which include design requirements for supply capacity, high service pumping capacity, stand-by power, and storage. The new water supply facility will meet all capacity criteria as well as the Ten State Standards. Key criteria are discussed in the applicable sections below.

9.5.2 Water Treatment Facility

9.5.2.1 Water Treatment Plant

The desalination treatment plant and appurtenant facilities would require an approximate 10 acre site. The general location of the plant and Barge Canal adjacent to the Power Plant is shown on Figure 9-3. The plant will not be located on the Power Plant property; however, its location would be coordinated with Progress Energy to ensure that the cooling flows can be used for dilution of concentrate discharge. The process selection will be a membrane RO type process that will meet potable water standards, as shown in the process flow diagram on Figure 9-4. The major elements of the facility are discussed below.

9.5.2.2 Raw Water Intake

A detailed study of the effect of the Barge Canal intake on the natural environment in the area will need to be performed during design and permitting in order to determine the location and design of the intake structure. For the purposes of this section, a concrete intake structure is proposed to be on the south bank of the Barge Canal, approximately 4 miles north of the Power Plant.

The intake will consist of a submerged reinforced concrete weir structure. The weir would be set at an elevation equal to the water elevation, below which no withdrawals can occur. A floating barrier and screens will be installed to prevent entry into the structure. The design of the structure will address FDEP criteria for impingement and entrainment of aquatic organisms. Generally, an intake velocity of less than 2.0 feet per second will be developed and the screen design will prevent access by listed species such as manatees and sea turtles.

9.5.2.3 Raw Water Pump Station and Transmission

The raw water pump station will be constructed next to the intake structure. Water would flow from the intake structure through a culvert or large diameter pipe to the wet well of the raw water pump station. A small building housing the MCC and an emergency generator will be constructed. The pump station would include two or more vertical turbine pumps with an estimated total capacity of 10,400 gpm (15 mgd) to pump raw water from the wet well to the head of the WTP. Standby pump capacity would be provided in accordance with the Ten State Standards and Chapter 62-550, F.A.C. The wet well would meet the hydraulic needs of the pumps but would not provide storage since adequate year-round flow is available in the Barge Canal. The raw water pump station would pump the raw seawater to the desalination plant through a large diameter high density polyethylene (HDPE) pipe.

9.5.2.4 Pretreatment

Raw water pretreatment will be designed based upon a comprehensive pilot plant study program concerning the full range of raw water quality that may be experienced. The goal of pretreatment is to remove compounds (such as dissolved organic material and suspended solids) that could prematurely clog the RO membranes. The pretreatment system will be based upon pilot plant studies, and will consider the dust generated by the existing limerock back hauling operation at the Power Plant. For the purposes of this section, a chemical fed coagulation-flocculation-filtration pretreatment system similar to the Tampa Bay Seawater Desalination Plant is assumed. The residuals from the pretreatment stage would be disposed of offsite.

As discussed above, raw water quality in the Barge Canal when Lake Rousseau is discharging could mean that a more extensive pre-treatment system will be needed. Potential pretreatment process options that could be considered include adding a sedimentation stage, ballasted flocculation (eg, ACTIFLO), and a dissolved air flotation (DAF) stage. The reader is referred to Chapter 8 for more information on treatment processes for fresh surfacewater.

9.5.2.5 Membrane RO Treatment

Removal of dissolved solids (salts) and other constituents remaining after pre-treatment will be performed by a pressurized RO system. Multiple passes through RO membranes are normally required to maintain reasonable operating pressures across the membranes. Design criteria for potable water are 250 mg/l total dissolved solids, but this value will vary depending on the configuration of the end user(s). 250 mg/l assumes that the desalinated product can be blended with treated waters from other sources prior to distribution by the receiving utility to consumers. If the desalinated product is to provide dedicated service (not be blended), a higher level of treatment to 100 mg/l would likely be required. This report assumes a product TDS level of 250 mg/l will be needed, achieved by a partial 2nd stage membrane. A full 2nd RO stage can be added if needed. Saline concentrate from the RO process would be fed into the Power Plant cooling canal for dilution and disposal.

9.5.2.6 Disinfection and Stabilization

Product water from the RO system will be highly aggressive as nearly all of its constituents will have been removed. The post membrane product water will require addition of chemicals for

pH stability, corrosion inhibition, and scale control in the transmission system. This could involve additions of lime, caustic soda, orthophosphates, or others. The final configuration of post membrane chemical addition will be affected by the selection of disinfectant method, the transmission line material and feasible blending considerations identified in preliminary design. However, end users would be responsible for blending the finished water with the water in their distribution system(s). Post membrane product water will likely be disinfected with a hypochlorite solution prior to entering the storage tank and transmission line.

9.5.2.7 Finished Water Storage

The water supply facility will typically be a new supply for member utilities. Storage for the product water would be provided in case of transmission interruption or other conflicts with the delivery and use system. Two storage tanks would be provided on site for plant downtime and transmission system interruptions. FDEP requirements for minimum storage stipulate that the total storage capacity of the facility meet at least 25% of the maximum daily demand of the system. For conceptual design, it is assumed that 50% of the projected average daily demand is sufficient storage to meet the storage requirements. The maximum daily demand and storage requirements will be determined during design and permitting through coordination with utility end users.

Storage will be provided by circular prestressed concrete storage tanks, constructed in accordance with AWWA D-110 (e.g., a composite similar to a CROM tank). The site will be developed with enough area to install a future storage tank to meet expansion needs beyond the horizon of this study.

9.5.2.8 Finished Water Pump Station

In order to transfer water from the treatment facility to the communities served, a dedicated finished water pumping system would be installed. This system would consist of three or more horizontal split-case pumping units (possibly with variable speed drives) and would be controlled using pressure levels in the downstream transmission/distribution system, water levels in downstream storage tanks, or both. Results from the hydraulic modeling of the finished water transmission system should be used to establish sizing and selection requirements for the finished water pumping system.

9.5.3 Support Facilities

Additional facilities required for the seawater desalination operations will include the following:

- Concentrate line connecting the desalination plant to the Power Plant cooling flow;
- Chemical storage tank facilities;
- Parking;
- Electrical feed and distribution system;
- Stormwater management system;
- Landscaping and buffer zones; and
- Lighting.

An operations/maintenance/administration building will be constructed to support the overall operations of the water treatment plant and the staff who will work there. The building will have an area from which the various plant operations can be monitored and controlled, a work space with tables, cabinets, tools and spare parts, a file storage and reference area, on-site laboratory, meeting rooms, and a bathroom. Operation and maintenance needs for the facility are anticipated to be staffed by participating utilities. The design of the support facilities will be closely coordinated with the needs of the participating utilities.

9.5.4 Environmental Monitoring

Monitoring of the plant concentrate discharge will be required in accordance with the NPDES criteria and the FDEP NDPES permit. Monitoring will be required downstream of the mixing zone which will likely be at the end of the Power Plant cooling water discharge. Additional monitoring may be needed in the Barge Canal or Power Plant cooling water depending on site specific conditions.

9.6 Transmission Systems

In order to deliver finished water produced by the new water supply facility to the users, a finished water transmission system will need to be evaluated, designed, and constructed. A conceptual transmission system was prepared for this element of the project. The transmission route typically assumes that water will be provided water to utilities at an approximate location within the respective service area, via easements acquired along public rights-of-way. Proposed pipe routes run along county or state roads for the purposes of this section.

For this project, a raw water transmission system would also be required to deliver raw water from the intake location to the treatment plant.

Since a proposed facility would be a major water supply facility for the area, careful planning and consideration should be given to the location where the raw and finished water should be routed and connected. Actual pipeline routes and points of connection will be identified during design and permitting through coordination with the participating utility and the Power Plant.

9.6.1 Conceptual Transmission Design

The conceptual design of the transmission piping is approximately based on the average day demands presented above and the overall capacity of the project. Since raw water storage would not be provided at the intake structure, the raw and finished water transmission systems would be designed on the same basis. Hydraulic modeling and coordination with participating utilities will be performed during design and permitting to determine the actual transmission requirements. Actual transmission sizes will be based on maximum daily flows determined by participating utilities.

Typical flow velocities for average daily flows for large transmission systems are in the range of 5-5.5 feet per second. Maximum daily flows may increase the flow velocities to the range of 6-8 feet per second assuming a typical peaking factor of 1.5. The transmission design assumes that the existing local supply facilities will support peak needs for participating utilities, with limited support for peak flows provided by the new facility.

For the purposes of this section, the raw water pipeline material is assumed to be a large diameter concrete pipe. Other alternatives such as specially coated DIP, fiberglass, and HDPE could be considered during design.

DIP is assumed as the finished water pipeline material for the purposes of this report; other pipeline materials including cement-lined reinforced concrete and PVC may be evaluated during preliminary design. The pipe routes and sizes are presented in Tables 9-2 and 9-3 for the conceptual transmission system.
Since the proposed pipe routes primarily run along county or state roads, consideration should be given to potential road upgrades in the future. In order to avoid future pipe relocation, easement along the pipeline corridors should be acquired. Easement width will be 30 feet for pipes 16 inch or larger and 20 feet for smaller pipes. Figure 9-5 illustrates the conceptual transmission system for the project.

Table 9-2. Conceptual Seawater Desalination Raw Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	**feet**	**miles**	**acres**
42	19,708	3.7	13.6
Total:	**19,708**	**3.7**	**13.6**

Table 9-3. Conceptual Seawater Desalination Finished Water Transmission System.

Pipeline Size	Pipeline Length		Easement Area
inches	**feet**	**miles**	**acres**
42	67,665	12.0	46.6
36	115,320	21.8	79.4
12	2,125	0.4	1.0
Total:	**185,110**	**34.2**	**127.0**

9.6.2 Blending Water with Utility Distribution Systems

If finished water will not provide dedicated service, the differences in the water chemistry between treated groundwater and treated seawater present potential issues that must be considered by utilities in the planning process. This will require review of the treated seawater supply characteristics, existing groundwater supply of the end user, the construction materials of the distribution system, and the disinfection and corrosion issues associated with blending potable water from different sources.

The primary issues with blending are water quality as it relates to the disinfectant residual, DBP formation, and pipeline corrosion. Post membrane seawater is highly aggressive water that must be chemically stabilized prior to introduction to a transmission system. In addition, the choice of disinfectant will affect byproduct formation – for example, hypochlorite will tend to decay to chlorate, which is a regulated parameter. Potable water standards must be met in the transmission system in accordance with Rule 62-550.310, F.A.C, and meeting the disinfection and corrosion control needs in the desalination plant's transmission system will affect the design of the blending facility.

After treated water from one source mixes with that from another source, changes in distribution system water chemistry can affect the stability of pipe coatings and disrupt the biofilms that protect pipes from corrosion. An increase in DBP's can also occur, either cumulatively or due to source interactions among multiple disinfectant types. The blending of groundwater and seawater must consider the combined water chemistry in the utility distribution system. Ultimately, potable water standards must be met in the blended water.

Feasible blending considerations will be evaluated during the desalination plant's preliminary design, but each utility's source water and distribution system characteristics will be different. Therefore, it will be the responsibility of the utility to blend the water within their system and distribute water to their respective customers, and the determination of costs and the distribution infrastructure needed to properly blend falls with the individual utility. The method of blending and associated treatment processes to meet primary and secondary drinking water standards must also be determined by each utility.

9.7 Conceptual Cost Estimate

The configuration of the facility was used to develop an individual conceptual cost estimate according the methodology established in CH2M Hill (2004). The cost estimate is presented in this section.

9.7.1 Cost Definitions

The following elements are included in the cost estimate:

- Construction cost is the total amount expected to be paid to a qualified contractor to build the required facility.
- Non-construction capital cost is an allowance for construction contingency, engineering design, permitting and administration for the facility.
- Land cost is the market value of the land required for the facility.
- Land acquisition cost is the estimated cost of acquiring the land, exclusive of the land cost.
- Operation and maintenance cost is the estimated annual cost of operating and maintaining the facility when operated at average day capacity.
- Capital cost is the sum of construction cost, non-construction capital cost, land cost, and land acquisition cost.
- Unit production cost is the annual lifecycle cost of the facility divided by the annual water production rate.
- Interest or discount rate is the time value of money criteria for the facility.
- Equivalent annual cost is the annual lifecycle cost of the facility based on service life and time value of money criteria.

9.7.2 Capital Cost Estimates

A summary of the conceptual capital cost for the water supply project option is presented in Table 9-4, according to methodology and values established in CH2M Hill (2004). The non-construction capital cost was applied at 45 percent of the construction cost. This includes a 20%

allowance for construction contingency (unknown conditions and/or changed field conditions) and a 25% allowance for engineering design, permitting, and administration. Easement acquisition costs of $0.75 per square foot (e.g., $32,760 per acre) are included in the capital cost. Land costs of $5,000 per acre are included for the 10-acre footprint of the supply facility, plus 18% acquisition cost. The capital cost estimate for each facility is detailed in the Appendices.

Table 9-4. Seawater Desalination: 15 mgd Capital Cost Estimate.

Item No.	Description	Total Cost (2009 dollars)
1	Raw Water Intake and Pump Station	$8,285,000
2	Raw Water Transmission	$4,498,000
3	Water Treatment and Storage Facility	$48,301,000
4	Finished Water Transmission	$51,727,000
5	Land and Easement Acquisition	$4,652,000
	Subtotal construction capital cost	$117,463,000
	Non-construction capital cost (45%)	$52,858,000
	Total:	**$170,321,000**

9.7.3 Operation and Maintenance Cost Estimate

O&M include labor, power, and chemical costs necessary for operation; and R&R for equipment maintenance and membrane replacement. Labor costs were based on an estimated workforce needed to operate the facility. Chemical costs were based on estimated usage and vendor quotes. Power costs were estimated based on current rates and equipment operation needs. R&R were based on a combination of annual needs and project lifecycle of 30 years. For purposes of this report this is estimated to be 2.5% of the construction cost for the water treatment and storage facilities (due to periodic costs for membrane replacement), and 0.5% of the construction cost for the transmission system. The operating costs for this desalination process are considerable due to high power consumption and periodic membrane replacements. Table 9-5 provides a summary of the O&M costs for the water supply project option.

Table 9-5. Seawater Desalination: 15 mgd Operation and Maintenance Estimate.

Item No.	Description	Estimated Annual Costs
1	Labor	$750,000
2	Chemicals	$2,150,000
3	Power	$8,500,000
4	Equipment Renewal & Replacement	$1,115,000
5	Transmission Renewal & Replacement	$281,000
	Total:	**$12,796,000**

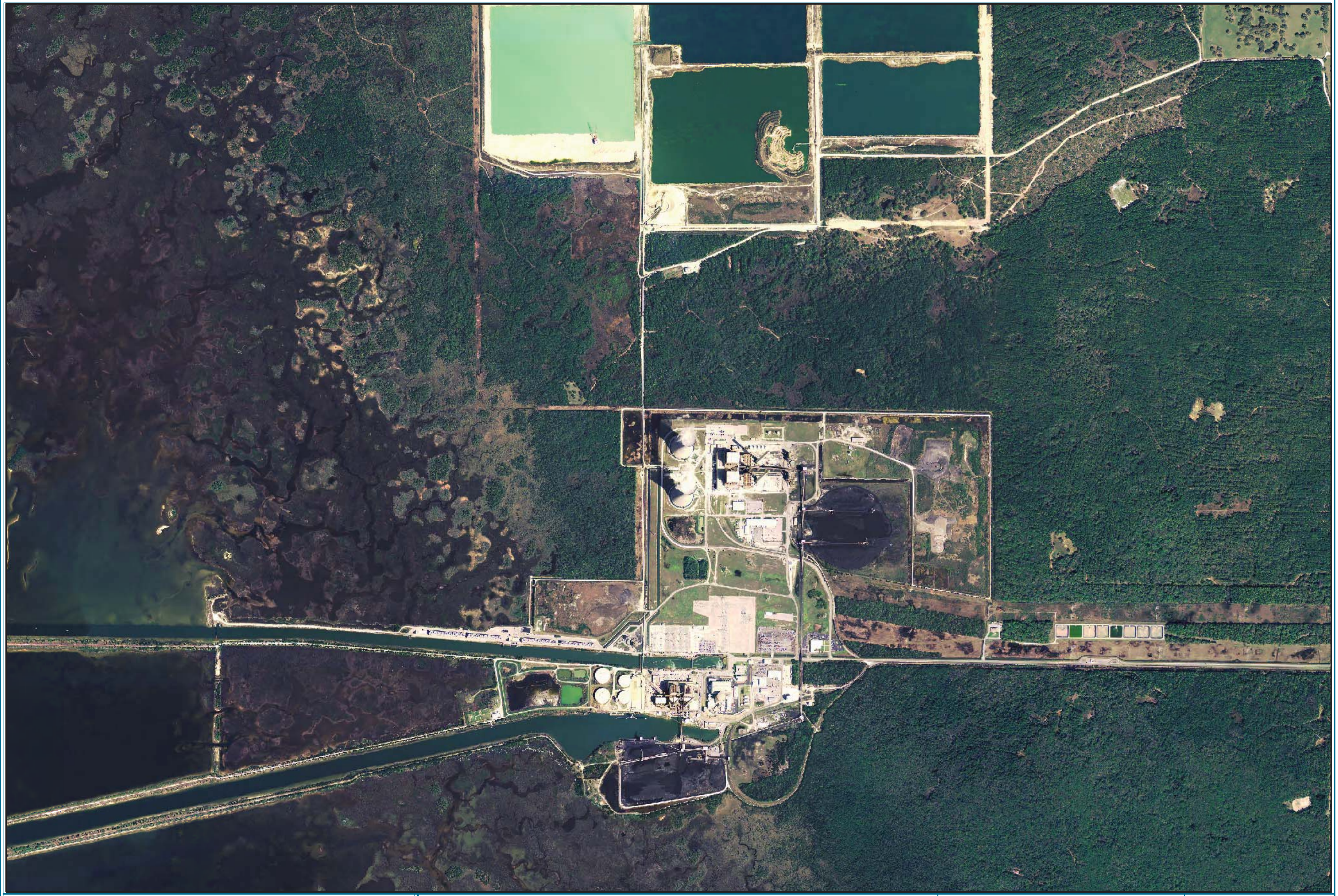
9.7.4 Unit Production Cost

Unit production cost is a function of the capital costs, debt service, annual O&M costs and the amount of water produced. For this analysis, the debt service is estimated based on a 30-year project lifecycle at 4.625% interest (2009 federal discount rate for water resource projects). Table 9-6 provides a summary of these costs for each water supply project.

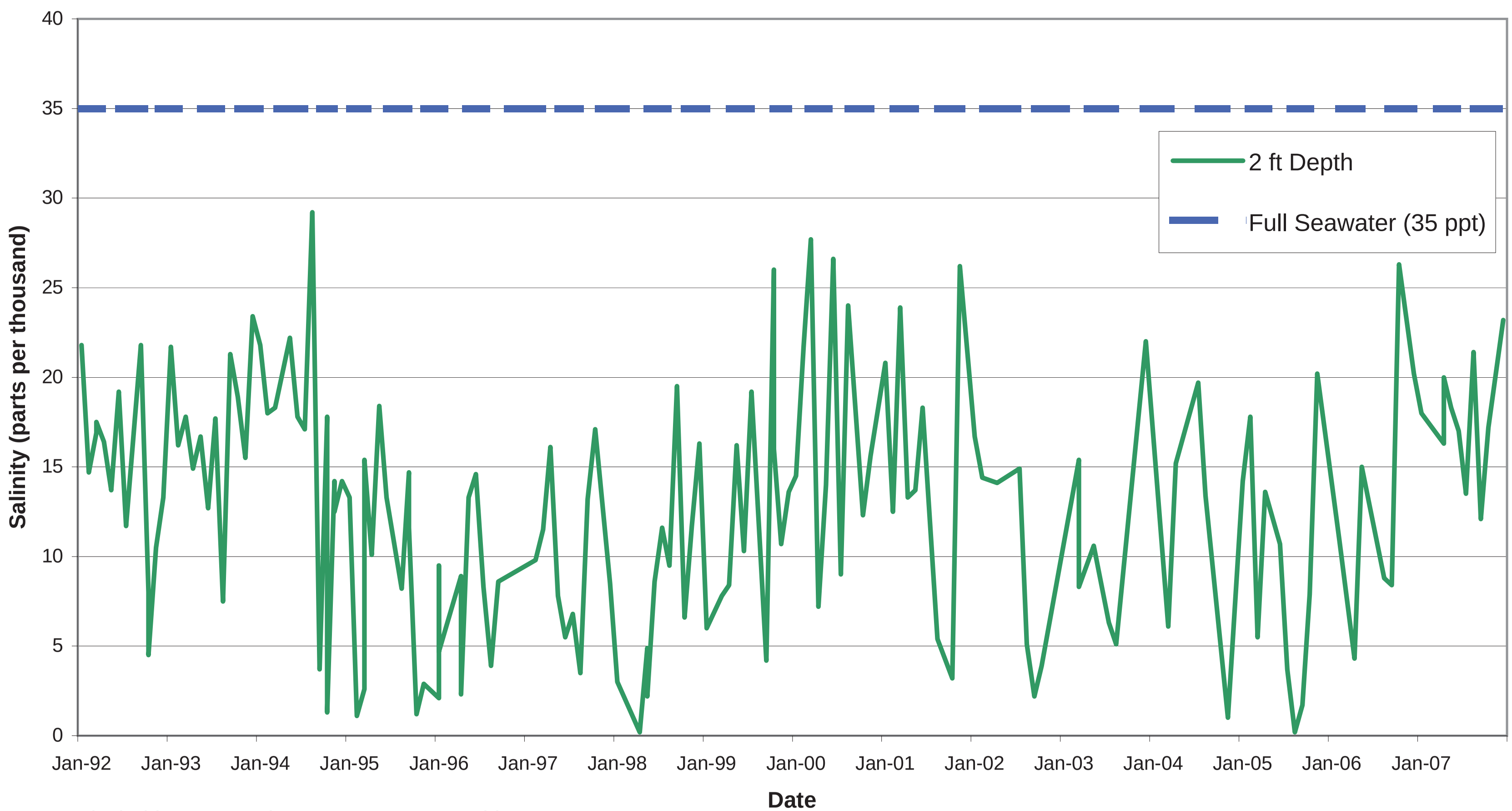
Table 9-6. Seawater Desalination: 15 mgd Unit Production Cost Estimate.

Item No.	Description	Total Cost
1	Total Capital Cost	$170,321,000
2	Annual O&M Cost	$12,796,000
	Equivalent Annual Cost:	**$23,404,331**
	Unit Production Cost ($/kgal)	**$4.27**

Notes:
1) *The construction cost within the total capital cost includes a 20% contingency.*
2) *30-year amortization at 4.625%.*

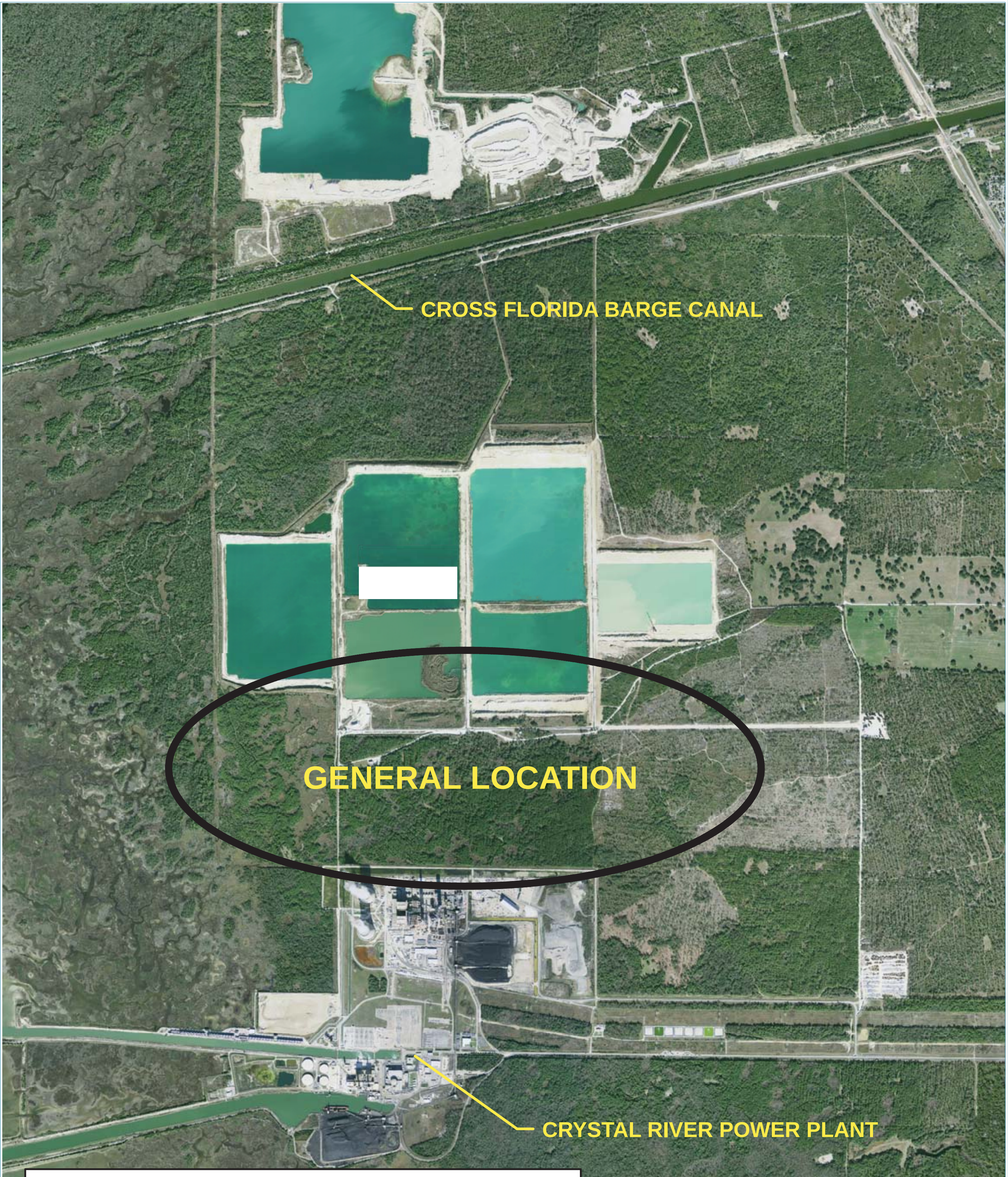


Water Resource Associates, Inc. *Engineering ~ Planning ~ Environmental Science* 4260 West Linebaugh Avenue Phone: 813-265-3130 Fax: 813-265-6610 www.wraconsultants.com	PROJECT: 0468 - Withlacoochee Phase II Figure 9-1 Progress Energy Crystal River Power Plant	ORIGINAL DATE: 08-08-06 REVISION DATE: 01-10-10 JOB NUMBER: 0468 FILE NAME: Crystal River Aerial.mxd GIS OPERATOR: DR	1 inch equals 2,000 feet



1) Florida DEP Station 34SEAS090. Monthly Data.

Figure 9-2. Cross Florida Barge Canal Salinity at Mouth[(1)]



NOTE: Raw water will be withdrawn from Barge Canal.
Concentrate discharge will be diluted by Power Plant cooling flow.

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FIGURE 9-3
GENERAL LOCATION OF SEAWATER DESALINATION FACILITY

ORIGINAL DATE: 10-28-09
REVISION DATE: 01-11-10
JOB NUMBER: 0468
FILE NAME: Desal location.mxd
GIS OPERATOR: LEF

N W E S

1 inch equals 1 miles

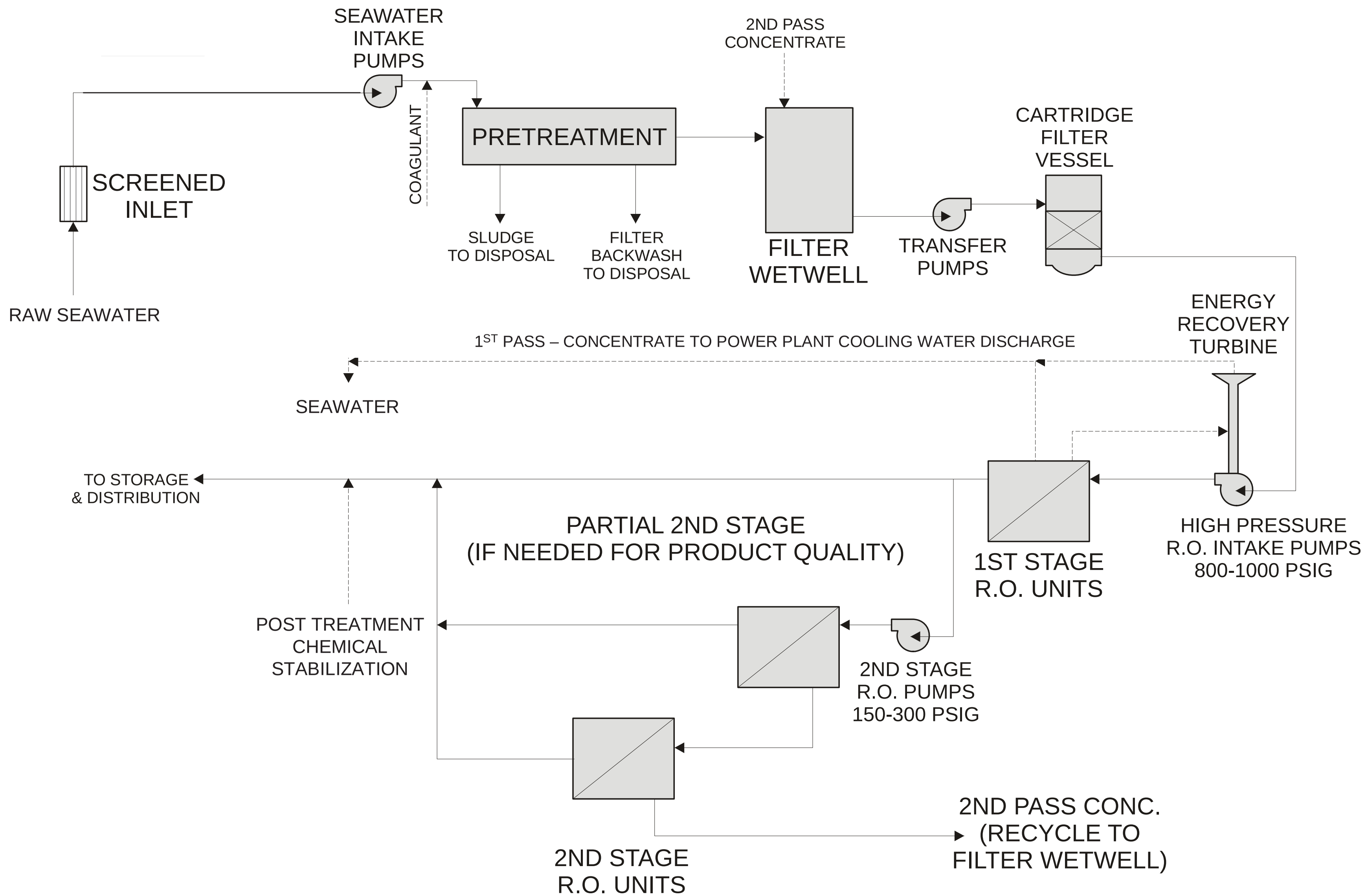

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FIGURE 9-4
SEAWATER DESALINATION
PROCESS FLOW DIAGRAM

Prepared For:

Withlacoochee Regional Water Supply Authority
Water Supply Feasibility Analyses

2007 Water Resource Associates, Inc. | Plot Date: 1/12/2010 5:09 PM | CAD File Name: s:\~project files\0468 - withlacoochee - phase ii\cad\from urs (11-13-07)\appendix g - crystal river site - revised.dwg

PROJECT	DRAWING	REVISIONS	GENERAL
WRWSA PHASE II WATER SUPPLY PLAN	FIGURE 9–5 CONCEPTUAL SEAWATER DESALINATION TRANSMISSION ROUTE	1. 2. 3. 4. 5. 6. 7.	Original Date: 07/16/07 Last Modified: 01/08/10 Scale: NTS FIGURE 9-5
JOB #: 0468	DESIGN: JRS DRWN: JRS CHKD BH APVD: DAW		

WRA URS

FL P.E. #