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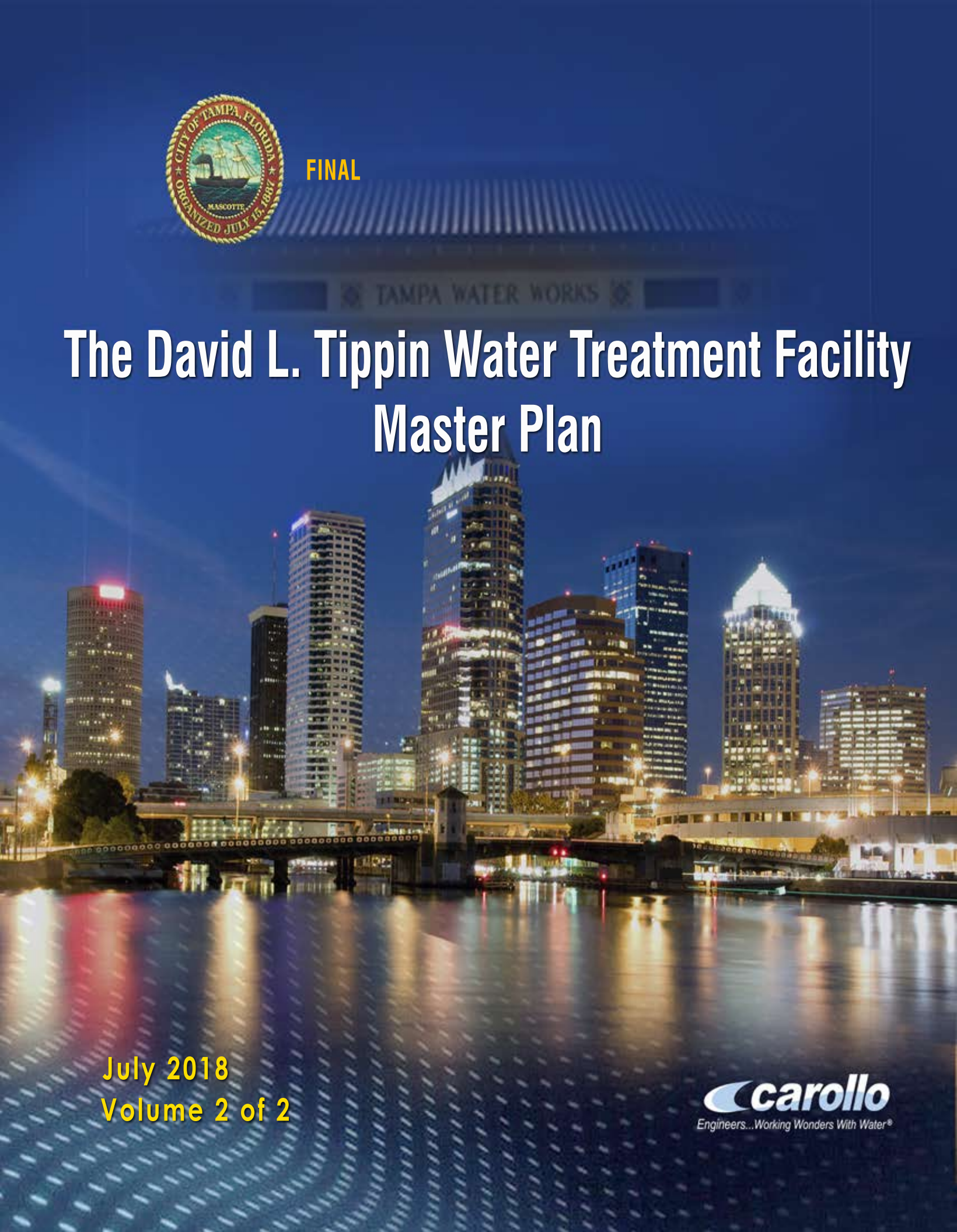
The David L. Tippin Water Treatment Facility Master Plan



July 2018
Volume 2 of 2

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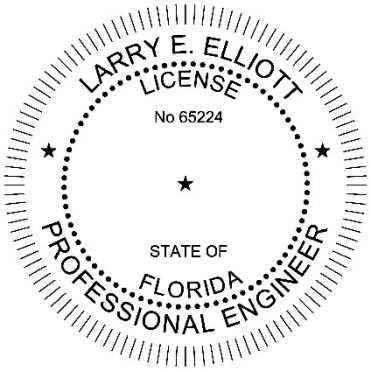
CITY OF TAMPA, FLORIDA

DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN

July 2018

FINAL



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DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN

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HYDRAULIC EVALUATION

4.1 CHAPTER SUMMARY

A hydraulic evaluation was conducted to develop the existing David L. Tippin Water Treatment Facility (DLTWTF)'s hydraulic profile at flows of 80, 120, and 140 mgd and to identify bottlenecks that hinder facility optimization and expansion. A field survey was also conducted to verify water surface and top-of-concrete elevations. The hydraulic profile model and discussions with City and plant staff identified a number of hydraulic concerns plant-wide, including:

- Uneven flow splitting to filters.
- Pressurization of the filtered water flume.
- Pressurization of the blend chamber.
- Overflow of the Hawkey box (120 mgd condition).
- Overflow of the pre-filter junction box (140 mgd condition).
- Overflow of Trains 5 and 6 if the low lift pump station level is not low (140 mgd condition).
- Sludge accumulation in line feeding GT 3 from Actiflo™ system.
- Ability to use only GT 3 for Actiflo™ due to overflow of Splitter Box 1/2.
- Overflow of Junction Box 4 when ASR, supernatant, and stormwater flows are high.

Recommendations to correct these hydraulic bottlenecks and issues are detailed herein, and intend to bring the DLTWTF hydraulic capacity to 140 mgd for future flows. Additionally, improvement with filter performance may be seen as a results of implementing these recommendations. The scopes of these items are captured in the 15-year CIP in their respective and appropriate projects.

4.2 INTRODUCTION

Originally built in the 1920s, the DLTWTF has been expanded numerous times, experiencing hydraulic changes in which flow paths have been reversed, rerouted, or split. This chapter will relay the results of the comprehensive hydraulic evaluation and offer a hydraulic profile for the following flow scenarios, each assuming 10 mgd from the ASR system:

- Current average daily demand--80 mgd.
- Current peak hour demand--120 mgd.
- Anticipated future peak hour demand--140 mgd.

Additional tasks included a field survey to verify water surface and top of concrete elevations, identification of hydraulic deficiencies and bottlenecks within the DLTWTF, and development of recommendations for mitigating the identified hydraulic constraints.

4.3 EXISTING PROCESS FLOW AND TREATMENT FACILITIES

4.3.1 Liquid Stream Flow

As described previously, flow enters the DLTWTF via the raw water intake structure from the Hillsborough River. Raw water pumps move it to the conventional treatment Trains 5 through 8 or Actiflo™ treatment Trains 1 and 2. A modulating valve controls flow to the Actiflo™ system.

On average, the Actiflo™ system receives 20 to 30 percent of the flow, and the conventional treatment trains receive 70 to 80 percent (Kim et al 2009). For the purposes of this evaluation, the conventional/Actiflo™ split was assumed to be 70/30 (this was modified slightly at 140 mgd based on train capacity).

There is one rapid mix basin before Trains 5 and 6 and one before Trains 7 and 8. Settled water from sedimentation basins 5, 6, 7, and 8 flows through a 60-inch gravity line that feeds into the low lift pump station. Just before it enters the pump station this line combines with a 54-inch gravity line from the Actiflo™ system into two 60-inch lines that connect to the low lift pump station.

The low lift pump station pumps into a head box, from which flow is gravity fed via an 84-inch pipe to the splitter box at the ozone contactors. The splitter box splits the flow through two 60-inch valves into the ozone contactors. Flow from the contactors exits over effluent weirs into a common box and is then discharged by gravity through an 84-inch line to the pre-filter (as known as settled water) junction box.

The pre-filter junction box receives 100 percent of the flow and distributes it through three gravity lines, which then split into numerous flow paths to the filters. Of the thirty filters, fourteen are "large" (Filters 1 through 14), with surface areas of 1,024 square feet (512 square feet each bay); sixteen are "small" (Filters 15 through 30), each with a surface area of 600 square feet (300 square feet each bay).

The typical filter loading rate is 2.5 gpm/sf. Filter run time is controlled based on head loss (6 feet), runtime (96 hours), or turbidity (0.15 NTU). Terminal head loss is typically the first parameter to reach its control point and is therefore the most common backwash control.

A valve on the effluent line uses three methods to control the flow rate through the filters:

- A specific flow rate is selected for each filter.
- A high and low water surface elevation are selected for the filters.
- An equal flow split (gpm/sf) among the filters based on the raw water flow rate.

The third control option is the typical operational mode for all the filters. In this mode, the flow rate through a filter can change over time as the raw water flow changes. High and low water levels are also selected under the third option, and the filter must operate within the selected range.

From the filter underdrains, filtered flow is fed to a filter effluent flume located below the filter gallery. The effluent flume for Filters 1 through 8 feeds into a 48-inch gravity line, for Filters 9 through 14 feed into a 48-inch gravity line, and the effluent flume for Filters 15 through 30 feeds into a 42-inch gravity line that all feed the Hawkey box.

The full facility flow from the filters is directed to the Hawkey box and then flows to the serpentine blend chamber. The water is dosed with chlorine at the start of the chamber followed by ammonia at the end of the chamber. The water then enters the clear wells. There are two gates (4-feet by 5-feet) at the end of the blend chamber that feed into the clear wells (one to the 7.5 million gallon (MG) and one to the east 5 MG clear well).

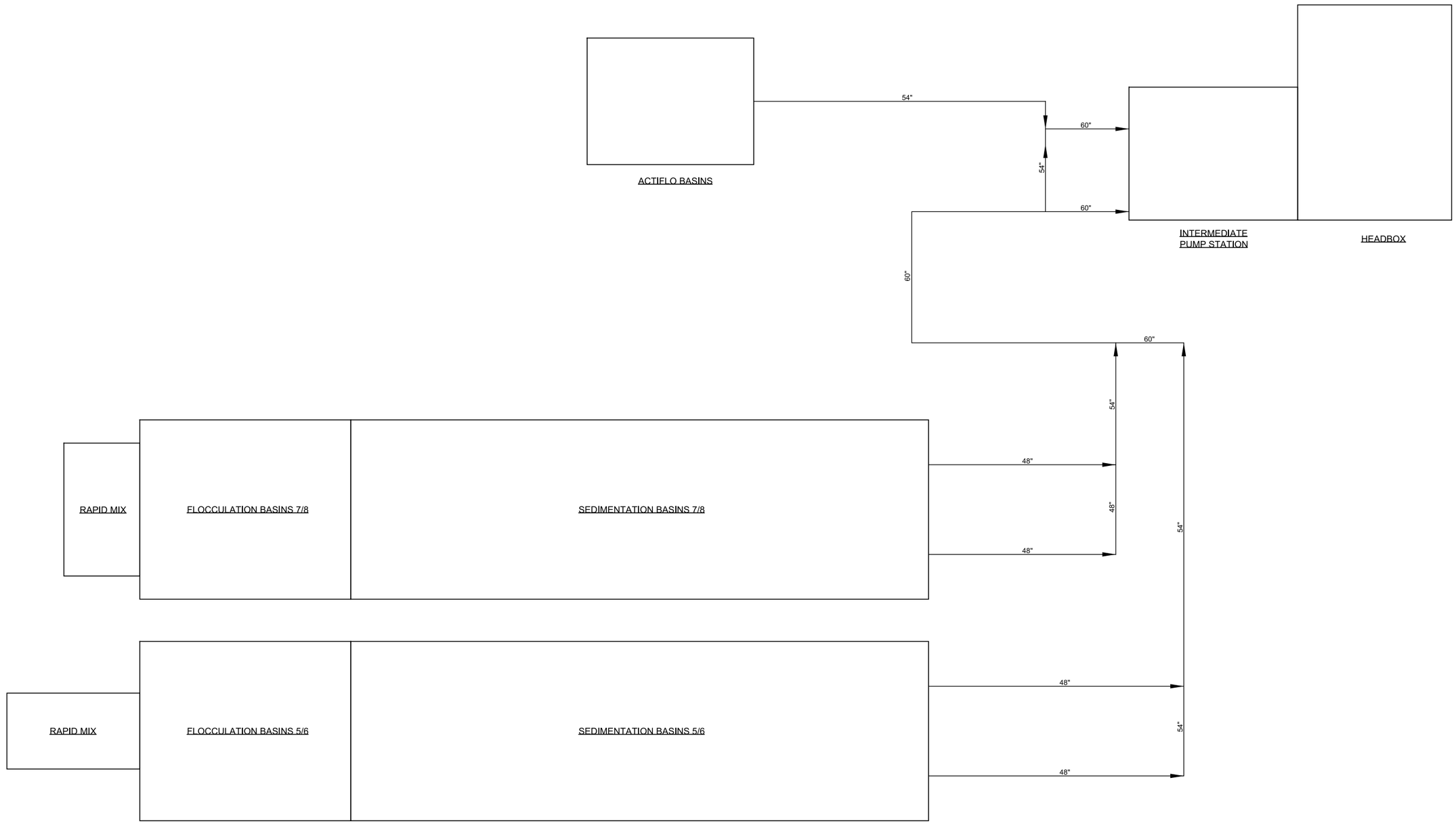
After the blend chamber, the flow is routed through each of the five clear wells, typically in this order:

7.5 MG Clearwell → west 5.0 MG Clearwell → east 5.0 MG Clearwell → 0.5 MG Clearwell → 2.0 MG Clearwell

Gates at the end of the blend chamber open to allow 70 percent of the flow to take the typical course through the 7.5 MG clear well, while 30 percent is diverted to the east 5.0 MG clear well. From the 7.5 MG clear well, flow enters the west 5.0 MG clear well via a 54-inch pipe and then through two 54-inch pipes to the east 5.0 MG clear well.

Eight high service pumps (HSPs) direct finished water to the distribution system. HSPs 5 through 8 pull from the east 5.0 MG clear well. Any additional flow that is not pumped by HSPs 5 through 8 is directed through 60-inch and 36-inch connections to the 0.5 MG clear well and then to the 2.0 MG clear well. HSPs 1 through 4 draw from these clear wells via a manifold.

Figure 4.1A and Figure 4.1B show the liquid process flow diagram.



LIQUID PROCESS FLOW DIAGRAM (PART A)

FIGURE 4.1A

CITY OF TAMPA

DAVID L. TIPPIN WTF MASTER PLAN

carollo

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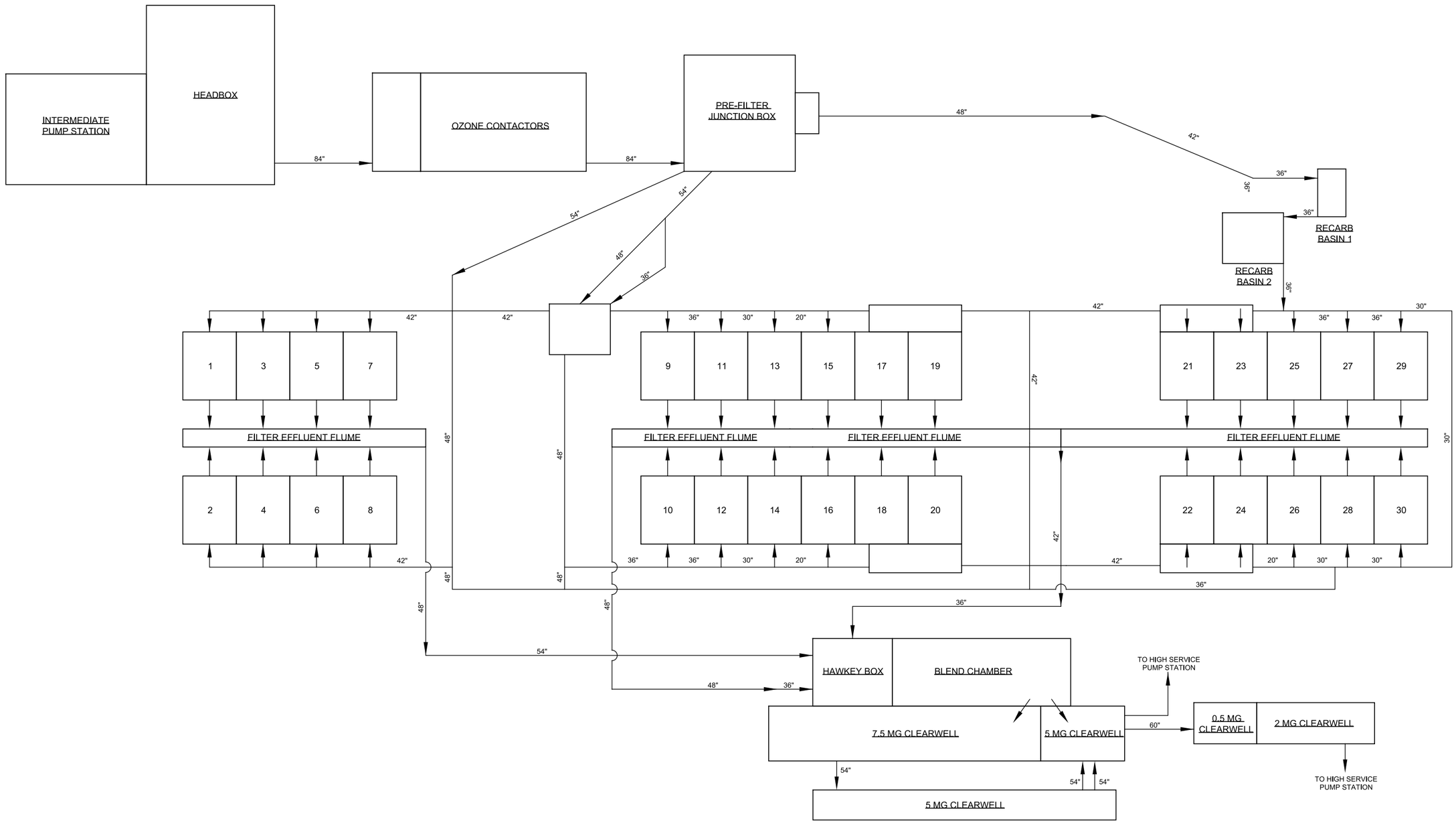

LIQUID PROCESS FLOW DIAGRAM (PART B)

FIGURE 4.1B

CITY OF TAMPA

DAVID L. TIPPIN WTF MASTER PLAN

carollo

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4.3.2 Solids Stream Flow

The DLTWTF processes sludge from the conventional and Actiflo™ treatment trains as well as the washwater from the filtration system onsite using gravity thickeners (GT). The thickened sludge is then pumped off site for further processing.

The filter waste washwater (FWW) from the filter backwashing system is directed to the surge tank via a 30-inch gravity line with two junction boxes. The washwater is then pumped from the surge tank to the gravity thickeners.

Sludge from the Actiflo™ system is gravity fed through a pipeline to Splitter Box 3 and then to GT 3. Sludge from the conventional treatment trains flows to Splitter Box 1/2, which diverts it to GTs 1 and 2. The belt filter press return water is directed to a line between Splitter Box 1/2 and Splitter Box 3. Typically, filtrate is returned to the thickeners via Splitter Box 1/2. The line between Splitter Box 1/2 and Splitter Box 3 can be used to transfer sludge between the two boxes, but valving at Splitter Box 3 is typically kept closed.

Supernatant from the thickeners is directed to Junction Box 3 just east of GT 3 and then back to the head of the DLTWTF via a 36-inch gravity line that splits into two 20-inch lines at Junction Box 4. According to the City, since the ASR system and stormwater system use this same route, when all systems operate, the two 20-inch gravity lines feeding the reservoir experience flows of up to approximately 18 mgd. The DLTWTF currently does not monitor or record flow to the junction boxes, splitter boxes, or gravity thickeners.

Figure 4.2 shows the DLTWTF solids process flow diagram.

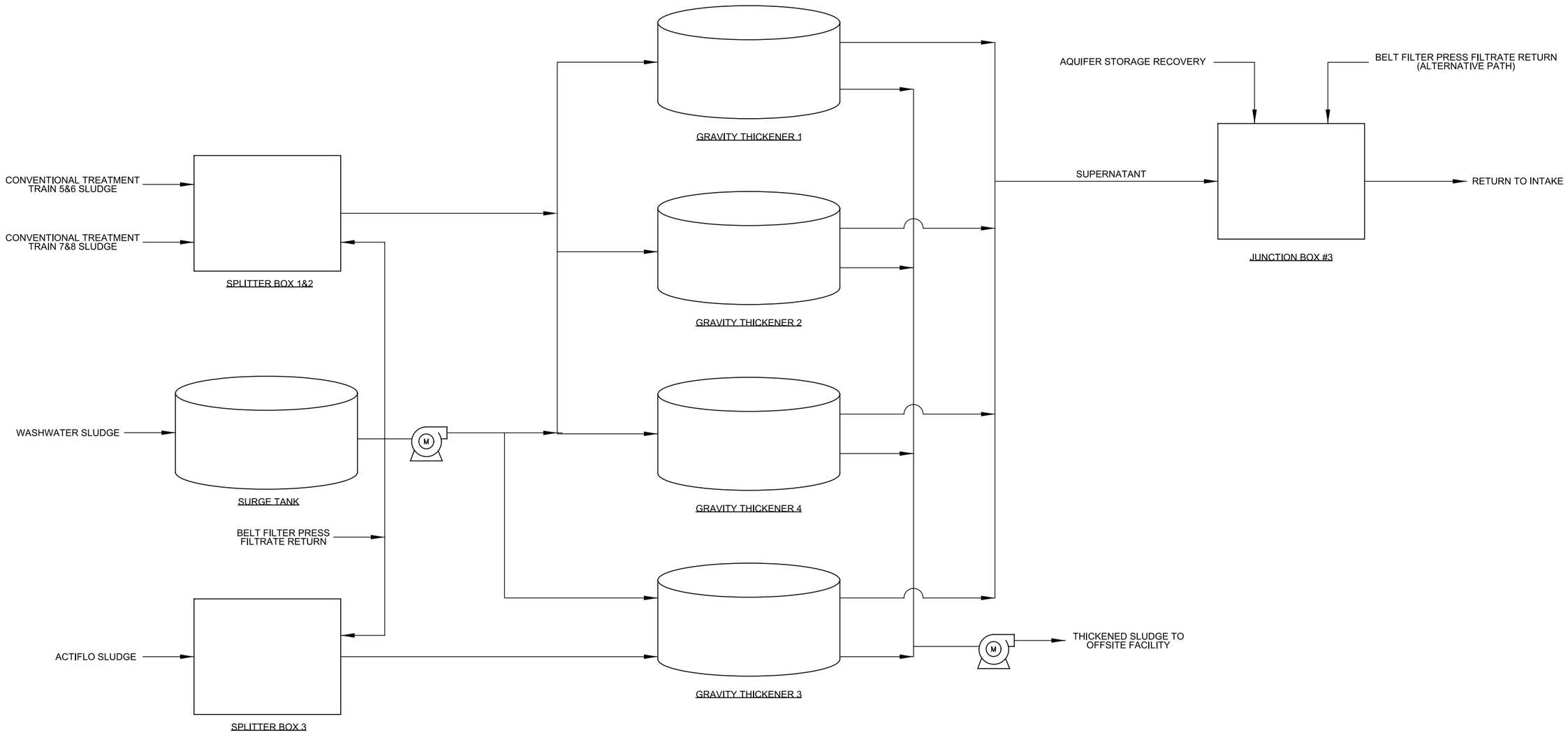
4.3.3 Hydraulic Issues Noted by Staff

In a meeting between the City and Carollo on July 26, 2016, to discuss the current hydraulic issues, DLTWTF staff shared observations and concerns about the liquid stream and solids stream flows and are noted herein.

4.3.3.1 <u>Liquid Stream Flow</u>

4.3.3.1.1 *Raw Water Intake*

The DLTWTF intake has nine raw water pumps. City staff did not note any major issues about the intake; however, they said that when the level in the reservoir is below 16-feet, Raw Water Pumps 6 through 9 (split-case pumps) are not functional. Since this issue is not recurring, it is not a major concern to plant operations and hydraulics.



SOLIDS PROCESS FLOW DIAGRAM
FIGURE 4.2
CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

carollo

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4.3.3.1.2 *Influent to Treatment Trains*

Due to the piping arrangement and design of the rapid mix, Trains 5 and 6 experience different flow and mixing than Trains 7 and 8. Short-circuiting may be occurring in Flocculation Basins 5 and 6 because the flow enters the influent box at high rates. Trains 7 and 8 may not experience this degree of short-circuiting because of friction loss and serpentine rapid mix configuration. Chapter 3 discussed this issue in greater detail.

The pipe configuration for the flow split between the conventional treatment and Actiflo™ treatment was noted to be an issue due to inability to isolate flow to Trains 7 and 8. This was noted a point source of failure due to lack of redundancy in feed piping to the Actiflo™ system.

4.3.3.1.3 *Treatment Trains*

The tops of Trains 7 and 8 were constructed 1 foot higher in elevation than Trains 5 and 6, and as a result, the full capacity of 7 and 8 cannot be utilized without overflowing 5 and 6. Staff noted that at 100 mgd, the low lift pump station must be operated at a low water surface level in order to prevent Trains 5 and 6 from overflowing.

4.3.3.1.4 *Effluent from Trains to Low Lift Pump Station*

Settled water from Trains 5 through 8 flows to a 60-inch gravity line that feeds the low lift pump station. City staff suspected this was a hydraulic bottleneck. Plant staff keeps the level in the pump station low at times of high flow to generate sufficient hydraulic gradient for the water to flow from the sedimentation basins to the pump station.

Functionally, these sedimentation basins serve as an extended and very large wet well to this low lift pump station. As a result, when the flow in the basins exceeds the flow rate of the pump station, the flow backs up, and the water level in the basins rises. Conversely, when the pump station flow rate exceeds that of the basins, the basin water level declines.

4.3.3.1.5 *Low Lift Pump Station and Head Box*

During high flow conditions, the head box nearly reaches capacity. Due to the turbulent conditions and the limited freeboard, water will splash over the top of the head box wall back into the lift station.

4.3.3.1.6 *Ozone Contactors and Effluent to Pre-filtration Junction Box*

The splitter box located prior to the contactors has been noted to overflow at high flow conditions, and is thought to be too small of a structure by staff. Staff noted that the influent box needs to be built up due to overflow issues at higher flow rates. Alternatively, eliminating downstream hydraulic bottlenecks or hydraulic constraints could also alleviate this concern.

At the ozone effluent channel there is a noticeable drop of approximately 5-feet following a free-flowing weir wall. A drop this substantial is not necessary to maintain a hydraulic grade across the ozone contactors for them to function properly and may present an opportunity

to increase the downstream process capacity or potentially remedy adverse upstream conditions, as long as a minimum water level in the contact basins is maintained.

4.3.3.1.7 *Pre-Filtration Junction Box*

Although staff did not note any hydraulic issues, this junction box receives 100 percent of the flow through the DLTWTF and could be a single point of failure because it lacks redundancy. Additionally, to achieve increased flows as high as 140 mgd, this basin may not have adequate freeboard, requiring modifications downstream or to the box itself to prevent overflow.

4.3.3.1.8 *Filtered Effluent to Hawkey Box and Blend Chamber*

The Hawkey box and blend chamber receive the full flow from the DLTWTF. They are both possible single points of failure due to lack of redundancy. It was noted that the size of one or both of these structures may be inadequate. Additionally, the gates at the end of the blend chamber are suspected to be too small.

Velocities through each gate should be under 12 feet per second (fps) and even below 8 fps, depending on the gate manufacturer's recommendations. However, the gates exhibit excessive headloss due to flow velocities as high as 19 fps through them to the clear wells. This substantial head loss leads to pressurization of the upstream filtered water flume. Pressurization of the flume, in turn, results in flooding of the filter gallery, cracking of the filter gallery floor, and manhole separation in the flume, which allows for a potential cross connection between untreated and treated water.

4.3.3.2 <u>Solids Stream Flow</u>

4.3.3.2.1 *Actiflo™ Sludge to Gravity Thickener #3*

The pipeline carrying Actiflo™ sludge is a gravity line routed uphill to Splitter Box 3, which feeds GT 3. The uphill route causes solids to build up in the pipe. Additionally, when flow is directed from Splitter Box 3 to Splitter Box 1/2 to be distributed to Thickeners 1, 2, and/or 4, it begins to back up, overflowing the Actiflo™ basins and Splitter Box 1/2. Therefore, Actiflo™ sludge can be processed only at GT 3, which represents a potential single point of failure for the Actiflo™ system.

4.3.3.2.2 *Junction Box 4 Overflow*

Junction Boxes 3 and 4 serve multiple systems, including ASR, thickeners, and stormwater. These junction boxes can also serve the belt filter presses, but the filtrate is currently directed to Splitter Box 1/2 instead.

Flow from these systems enters Junction Box 3 and flows by gravity via a 36-inch line to Junction Box 4. The flow then splits between two 20-inch pipelines and is directed to the raw water intake. Junction Box 4 has been observed to overflow when the ASR system is in use. Flows at the raw water intake, which represent the total flow through Junction Box 4 minus the amount of water that overflows, are approximately 18 mgd when the ASR system is utilized.

The overflow from Junction Box 4 eventually reaches the raw water intake as it flows over the ground into the river, but this flow is not accounted for by the flow meters on the 20-inch pipelines. Flow from Junction Box 3 is not currently measured.

4.3.3.2.3 *Washwater Surge Tank/Reclaim Tank*

The bottom elevation of the surge tank varies but on average is approximately 22.25 feet. The top of the surge tank is at an elevation of 43 feet. However, the high water level is restricted by the elevation of the channels in the filter gallery, which have overflowed when the surge tank water surface elevation is greater than 34.5 feet (top of the filter gallery floor).

An overflow weir (elevation 33 feet) in the junction box between the filters and the surge tank prevents flooding of the filter gallery. Therefore, as noted in the drawings, the high water level in the surge tank is 32 feet, which corresponds to a volume of 366,586 gallons.

Additional issues further restrict the usable volume of the surge tank. Under typical operation, the surge tank is unable to convey more water to the thickeners than it is receiving from the washwater system for two reasons: due to the redirection of much of the water to the recirculation system, and/or performance of the surge tank pumps. The three pumps that are used to recirculate water within the surge tank and to direct sludge to the thickeners are controlled by the water level in the tank. As the water level rises, additional pumps turn on. However, the amount of water directed to recirculation versus the amount of water being pumped to the thickeners cannot be controlled. To address this, the level within the tank is kept low (approximately one foot above the bottom elevation), which keeps recirculation at a lower level and also prevents the build-up of solids within the surge tank.

4.4 HYDRAULIC MODEL DEVELOPMENT

Hydraulic models of the DLTWTF were developed using Carollo Engineers' *Hydraulix*® modeling software to evaluate the hydraulic capacity of the DLTWTF's liquid treatment process.

A hydraulic profile was computed through each treatment process to determine the required hydraulic grade line to convey water through the DLTWTF at varying flow rates. This hydraulic evaluation did not include treatment process limitations, such as basin detention times or filtration rate criteria.

4.4.1 Hydraulic Model Inputs and Operations

The hydraulic model was built based on head loss calculations as water flows through each of the physical facilities in the treatment process. The model included an initial water surface elevation (WSE) at the clear wells upstream of the pumps. From this initial point, head loss was calculated through the upstream processes to the rapid mix and Actiflo™ basins.

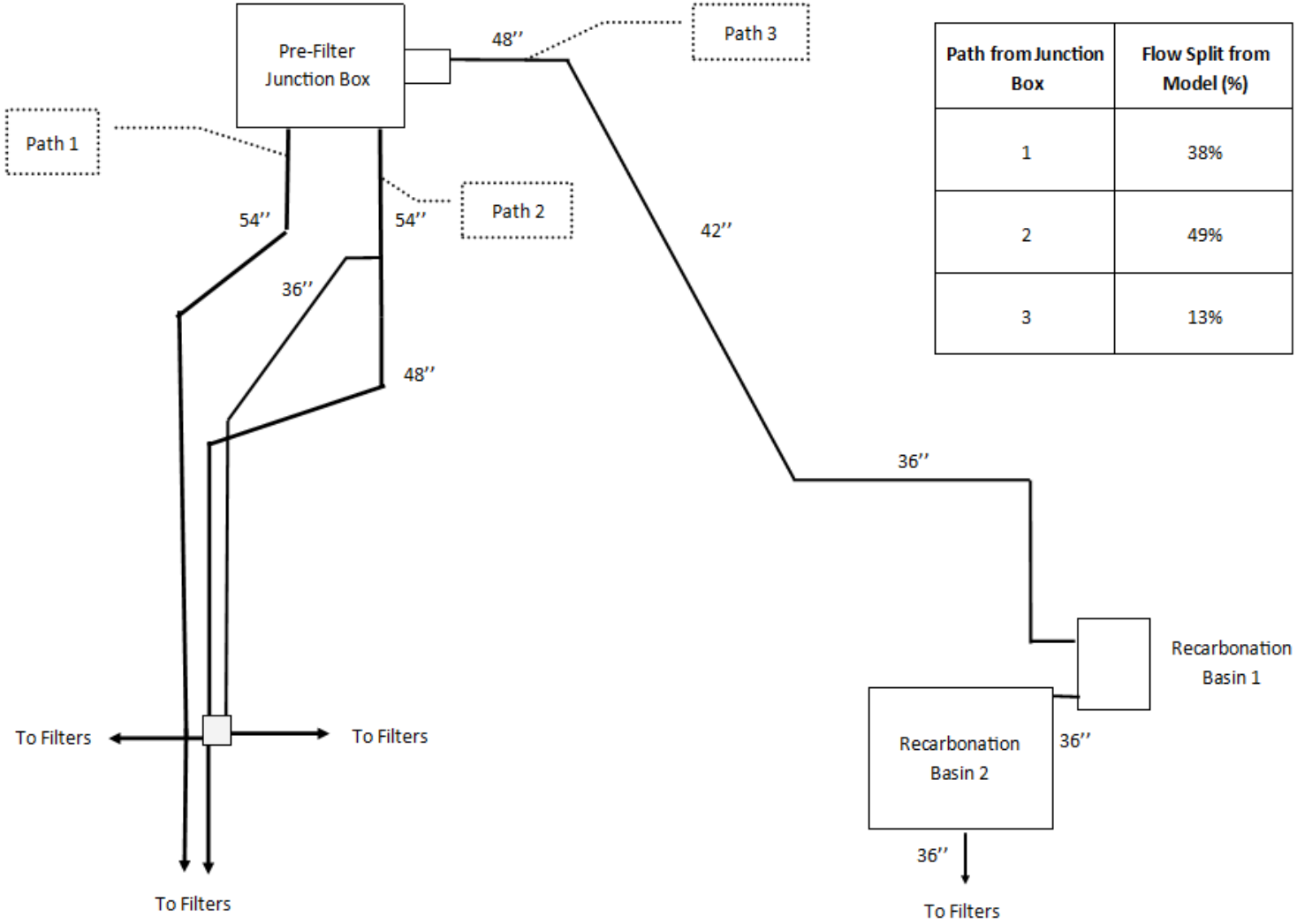
Hydraulix® model inputs and operational assumptions include the following:

- The hydraulic profile was prepared at flow rates of 80 mgd, 120 mgd, and 140 mgd. These flows include 10 mgd from the ASR system.

- A flow split of 30 percent through the Actiflo™ and 70 percent through the conventional treatment trains was assumed. This flow split was based on the 2009 Florida Water Resources Journal article (Kim et al 2009), which said that the conventional/Actiflo™ flow split ranged from 70/30 to 80/20. 70/30 was selected because it was closest to the split recorded during the calibration (64/36), as noted in Section 4.3.2. Under the 140 mgd flow conditions, the 70/30 flow split was modified based on the design flows of the treatment trains.
- The water surface elevation in the low lift pump station was assumed to be 46 feet at 80 mgd, based on the results of the model calibration. At 120 mgd, the pump station water surface elevation was assumed to be 44 feet based on feedback from staff that the water level is kept lower at higher flow rates. Similarly, at 140 mgd, the pump station water surface elevation was assumed to be 41.5 feet. According to the Hydraulic Institute standards, based on the minimum submergence of the pumps at the low lift pump station of 9.17 feet, the lowest elevation the pump station can be drawn down to is 39.17 feet.
- The bypass piping from the sedimentation basins to the pre-filter junction box (to bypass the ozone contactor) was not in service.
- Both ozone contactors were in operation.
- Two large filters were out of service. Figure 4.3 shows flow paths to the filters in the flow schematic. The piping configurations were modeled using WaterCAD to estimate the flow split from the junction box. A consistent filter loading rate among the filters was assumed at each flow rate, which resulted in the following flows:
  - 80 mgd: 4 mgd (large filters) and 2 mgd (small filters).
  - 120 mgd: 6 mgd (large filters) and 3 mgd (small filters).
  - 140 mgd: 7 mgd (large filters) and 3.5 mgd (small filters).

Figure 4.3 shows the three outlets from the pre-filter junction box and the flow percentages based on the WaterCAD results.

- The filter water surface elevations were assumed to be 45.5 feet, based on 2.5 feet of media and 6 feet of terminal head loss (worst-case scenario). In practice, the filters' water surface elevations will vary depending on the amount of flow reaching the filters from the junction box. The expected variation of the water surface elevations based on the WaterCAD results is discussed in Section 4.5.1.2.
- At 80 mgd, 0.5 feet of head loss was assumed for the recarbonation basins between the junction box and the filters. Head losses at 120 mgd and 140 mgd were then calculated using the model.
- Figure 4.1B shows flow splits from the filter flumes to Hawkey box in the flow schematic.
- Flow splits via two gates at the end of the blend chamber are as follows: 70 percent to the 7.5 MG clear well and 30 percent to the east 5 MG clear well.



Path from Junction Box	Flow Split from Model (%)
1	38%
2	49%
3	13%

FLOW PATHS FROM THE PRE-FILTER JUNCTION BOX

FIGURE 4.3

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pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH04/Graphics\Fig 4.3

The WSE was assumed to be at 30-foot elevation at the clear wells.

4.4.2 Hydraulic Model Calibration

On August 16, 2016, Carollo performed a site visit to measure WSEs and record flow rates to calibrate the calculated model based on the DLTWTF conditions. Measurements were taken at several units throughout the DLTWTF. The following conditions were documented:

- Total flow through the DLTWTF was 74.9 mgd.
- Total flow to the Actiflo™ system was 26.9 mgd.
- Total flow to Trains 5 through 8 was 48 mgd.

WSEs were measured at all accessible points and with known flow rates. Table 4.1 presents the calibrated hydraulic modeling results with the hydraulic elevations surveyed in the field. Table 4.2 summarizes the modeled and measured head loss across the Actiflo™ and conventional treatment train basins. Overall, the model results were similar to the field measurements. Notable variations between the model and the field measurements are as follows:

- Discrepancies of 0.27 feet and 0.24 feet were noted between the model and the field measurements at the sedimentation basin effluent channels for Trains 5 and 6 and Trains 7 and 8, respectively (Table 4.1). The WSE at the end of these basins is determined by the WSE in the intermediate pump station and the head loss accrued in the piping between the basins and the pump station.

  The level in the pump station is variable and determined by the flow through the DLTWTF and the pump station flow rate. If the flow rate into the DLTWTF drops slightly and the pumping rate remains the same, the level in the pump station will drop. Similarly, if the flow increases with no change in the pumping rate, the level will increase. Therefore, the level measured at the pump station during the field calibration (45.83 feet) would not have been exactly the same at the time the sedimentation basin water surface elevations were measured.

  Discrepancies between the modeled and measured sedimentation basin water surface elevations are likely due to slight variations at the pump station and are considered acceptable for this evaluation.

  In addition, the build-up of calcium carbonate detailed in Chapter 3 may have reduced the effective size of the piping between the sedimentation basins and the pump station. Calcium carbonate build-up would increase the head loss between the sedimentation basins and the pump station and may be partially responsible for the discrepancy between the model and field measurements.

- The model indicated lower head loss across the rapid mix basin for Trains 7 and 8 compared with the head loss measured in the field. As Table 4.2 shows, losses

measured in the field across the rapid mix basins were 0.03 feet and 0.27 feet greater for Trains 5 and 6 and Trains 7 and 8, respectively, compared to the modeled losses. It is suspected that the rapid mix basin configuration for Trains 7 and 8 used to develop the model does not exactly match the actual basin configuration. The Train 5 and 6 rapid mix basin was dewatered during the development of the model and the configuration was corrected accordingly, resulting in more closely matched losses.

- The model originally predicted higher head loss between the ozone contactors, and the head box of the low lift pump station than was observed in the field. However, the measured head loss between the head box and the contactors was negative, meaning the water surface elevation measured at the head box was lower than the elevation at the contactors. This suggests a measurement error.

  The turbulence within the pump station head box likely caused an inaccurate field measurement. Therefore, it was assumed that the water surface elevation at the head box was the same as the elevation at the ozone splitter box. This assumption is considered acceptable given that the pipe size between the head box and the ozone splitter box is 84 inches and is not expected to contribute substantial head loss. The resulting difference between the model and the field conditions is 0.06 feet.

Table 4.1 Calibrated Hydraulix® Results Compared to Field Measurements David L. Tippin WTF Master Plan City of Tampa

DLTWTF Location[1]	Water Surface Elevation (feet)		
	Field Measurement (ft)	Hydraulix® Output (ft)	Variation (ft)
Actiflo™ Entrance	47.92	48.04	0.12
Actiflo™ Exit	46.50	46.32	0.18
Rapid Mix 7/8 Entrance	47.88	47.80	0.07
Rapid Mix 7/8 Exit	47.50	47.69	0.19
Flocculation Basins 7 and 8 Entrance	47.40	47.34	0.06
Distribution Channel 7 and 8	47.42	47.25	0.17
Sedimentation Basin 7 and 8 Exit	47.35	47.11	0.24(1)
Rapid Mix 5/6 Entrance	48.46	47.36	0.10
Rapid Mix 5/6 Exit	47.92	47.86	0.06
Flocculation Basin 5 and 6 Entrance	47.67	47.5	0.17

Table 4.1 Calibrated Hydraulix® Results Compared to Field Measurements David L. Tippin WTF Master Plan City of Tampa			
	Water Surface Elevation (feet)		
DLTWTF Location[1]	Field Measurement (ft)	Hydraulix® Output (ft)	Variation (ft)
Distribution Channel 5 and 6	47.63	47.44	0.18
Sedimentation Basin 5 and 6 Exit	47.58	47.32	0.27(1)
Pump Station head box	55.00(2)	54.94	0.06
Ozone Splitter Box	55.00	54.78	0.22
Ozone Exit	47.17	47.31	0.14
Pre-Filter Junction Box	46.63	46.66	0.04

Notes:

(1) Trains 5 and 6 have one rapid mix basin (referred to as Rapid Mix 5/6), as do Trains 7 and 8 (referred to as Rapid Mix 7/8). For the flocculation and sedimentation basins, WSEs were measured in basins 5 and 7. The WSEs in 6 and 8 were assumed to be equal to the WSEs in 5 and 7, respectively.

(2) Assumes the water surface elevation at the head box is the same as the water surface elevation at the ozone contactor splitter box.

Table 4.2 shows the losses across each basin according to the field measurements and the model. The Actiflo™ losses differ by 0.3 feet between the model and the field measurements. However, since the model is more conservative, the discrepancy was considered acceptable for this evaluation.

Table 4.2 Calibrated Hydraulix® Results Compared to Field Measurements: Head Loss Across Basins David L. Tippin WTF Master Plan City of Tampa			
	Head Loss		
Process	Field Measurement (ft)	Hydraulix® Output (ft)	Variation (ft)
Actiflo™	1.42	1.72	0.30
Basins 7/8 (Rapid Mix)	0.38	0.11	0.27
Basins 7 and 8 (Flocculation through Sedimentation)	0.04	0.22	0.18
Basins 5/6 (Rapid Mix)	0.54	0.51	0.03
Basins 5/6 (Flocculation through Sedimentation)	0.08	0.18	0.1

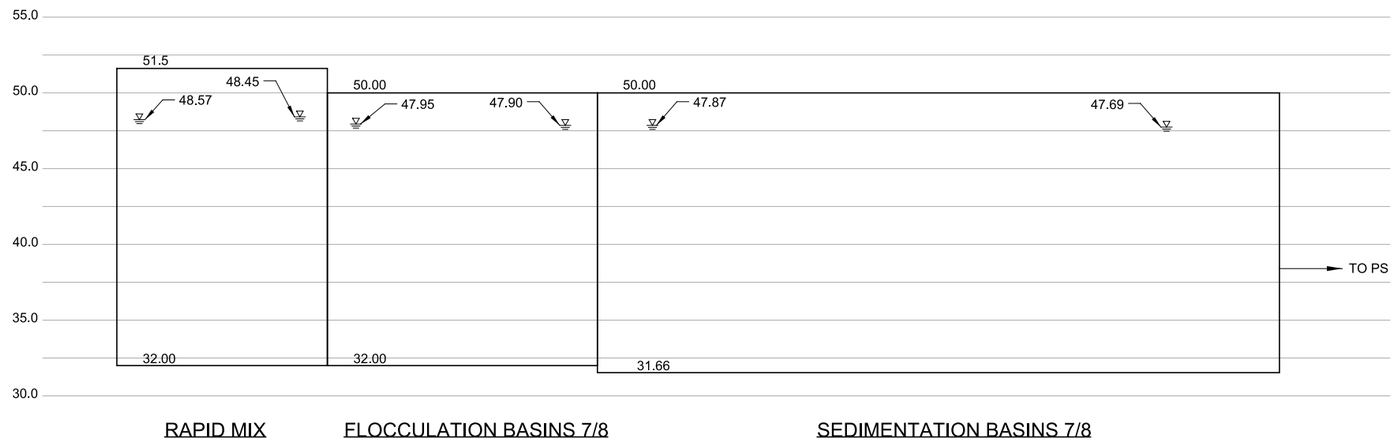
4.5 HYDRAULIC MODEL RESULTS

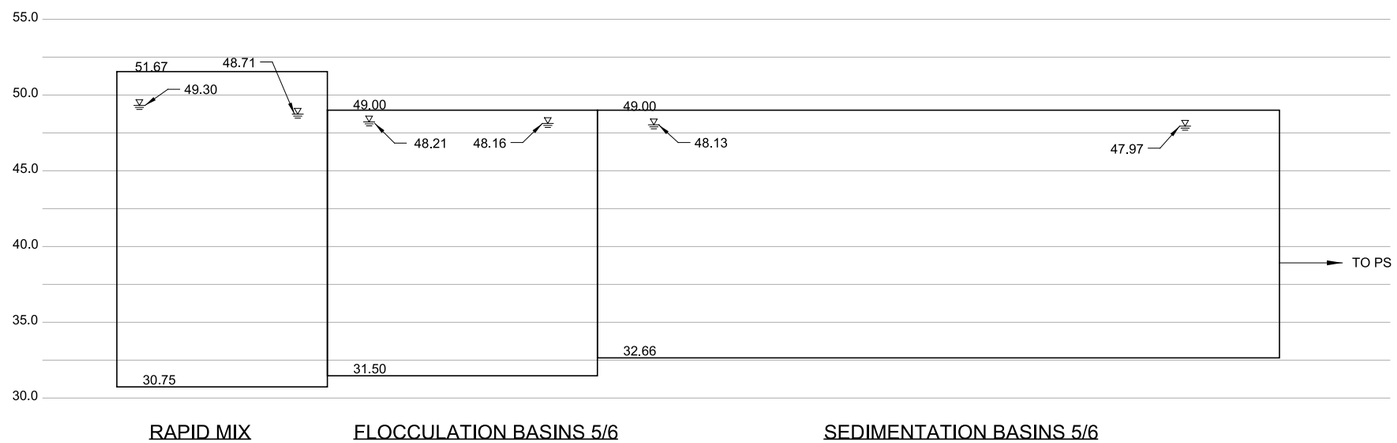
Flow was modeled through the DLTWTF at the current average daily demand of 80 mgd, the current permitted treatment capacity of 120 mgd, and an estimated future demand and facility capacity of 140 mgd. Additionally, simple models were developed to assess the suspected hydraulic bottlenecks between the low lift pump station and the ozone contactors, the Actiflo™ basins and Splitter Box 3, and at Junction Box 4 between Junction Box 3 and the DLTWTF raw water intake. The results produced by the *Hydraulix*® software is included electronically as Appendices E, F, and G.

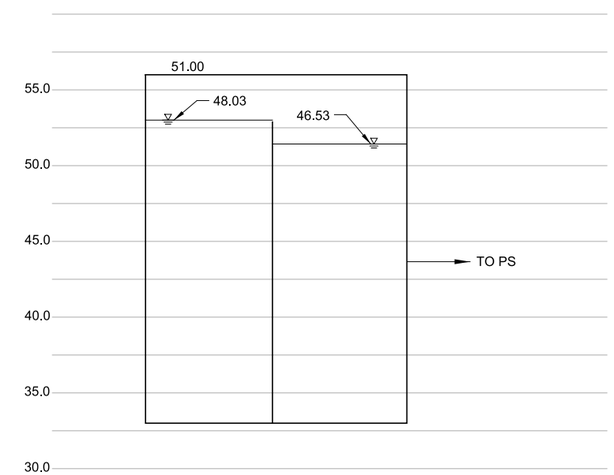
4.5.1 Current Average Daily Flows

This scenario assumes an average flow of 70 mgd with an additional 10 mgd of ASR water for a total flow of 80 mgd through the DLTWTF. Figure 4.4A, and Figure 4.4B show the hydraulic profile. Appendix E contains the *Hydraulix*® model results for this flow senario. Under the conditions and assumptions specified in Section 4.3.1, the WaterCAD model identified the following hydraulic limitations:

- Flow rates among each of the paths from the junction box to the filters were determined. Though the flow splits the model identified did not cause any hydraulic bottlenecks at 80 mgd, the uneven flow splitting limits the capacity of the system. As Figure 4.3 shows, the model indicates that 87 percent of the flow from the junction box exits through the two 54-inch pipelines (Paths 1 and 2), while only 13 percent leaves through the 48-inch pipeline (Path 3). At 80 mgd, this corresponds to velocities of 3.85 fps (Path 2) and 2.93 fps (Path 1) in the two 54-inch pipes and a velocity of 1.27 fps in the 48-inch pipe (Path 3). The reduced flow through Path 3 is due to additional losses along this piping route, which includes a more elaborate path through the bottom of the junction box and a baffled basin before the filters.
- The water surface elevation at the filter effluent flumes exceeds the top elevation of the filter effluent flume (33.33 feet) by more than a foot and a half at each filter bay. The highest water surface elevation (36.37 feet), and therefore the most substantial pressurization, occurs at the filter effluent flumes for Filters 15 through 30. This situation was observed during the site visit when the DLTWTF was processing approximately 70 mgd. Leaking and cracking of the filter gallery floor over the flume were observed, indicating that the flumes are pressurized.
- At 80 mgd the blend chamber top elevation of 33.5 feet was overtopped in the model. Located underground, the blend chamber was not observed to be overflowing at the access hatches. It is possible that the basin is pressurized.







HYDRAULIC PROFILE CURRENT
AVERAGE DAILY FLOW (80 MGD) (PART A)

FIGURE 4.4A

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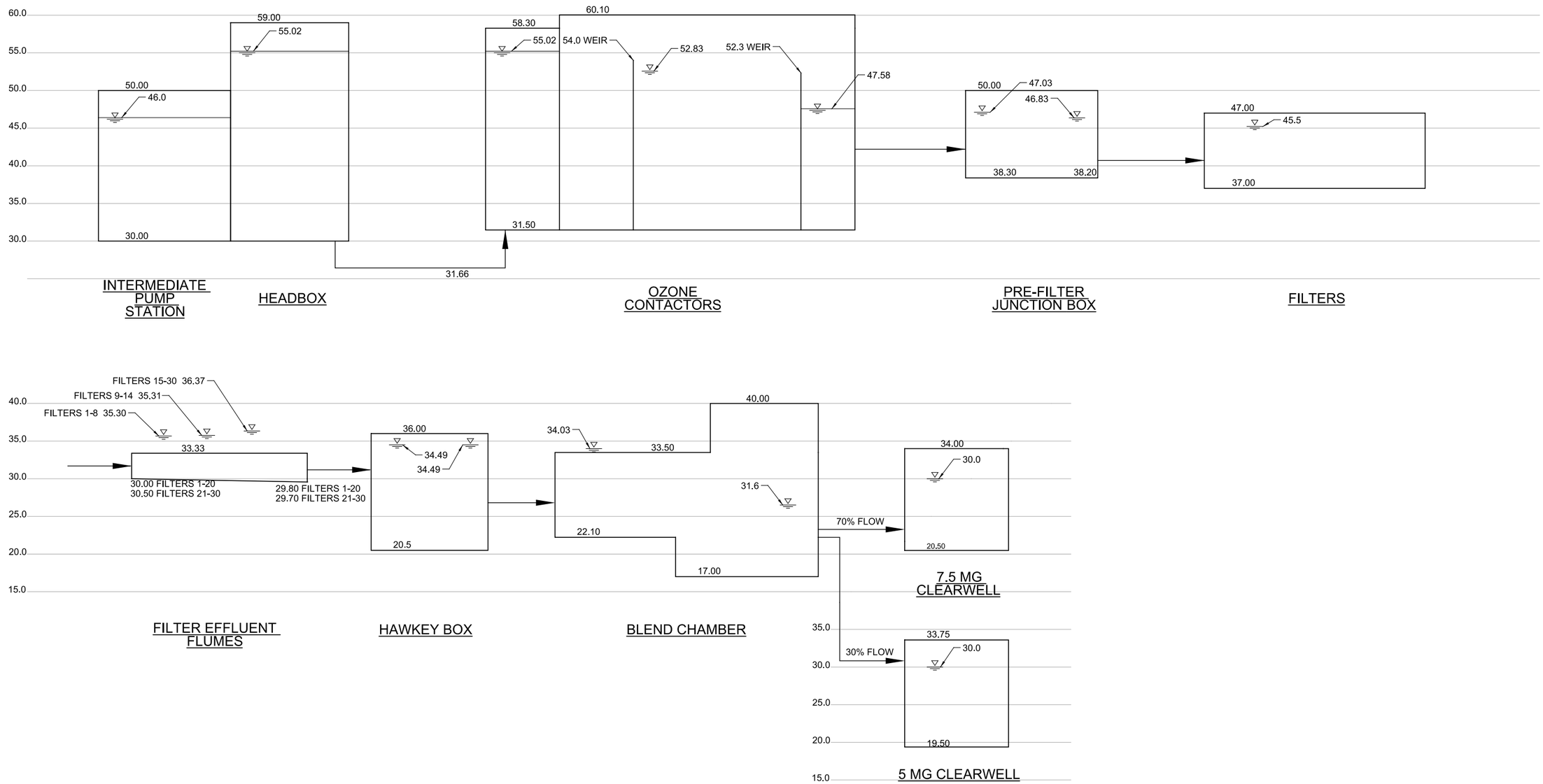

HYDRAULIC PROFILE CURRENT
AVERAGE DAILY FLOW (80 MGD) (PART B)

FIGURE 4.4B

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4.5.2 Current Permitted Treatment Capacity

This scenario assumes a flow of 110 mgd with an additional ASR flow of 10 mgd for a total flow of 120 mgd. Figure 4.5A and Figure 4.5B show the hydraulic profile for this scenario. The *Hydraulix*® model is included in Appendix F for this flow scenario. Under the conditions and assumptions described in Section 2.1, the following hydraulic limitations were noted:

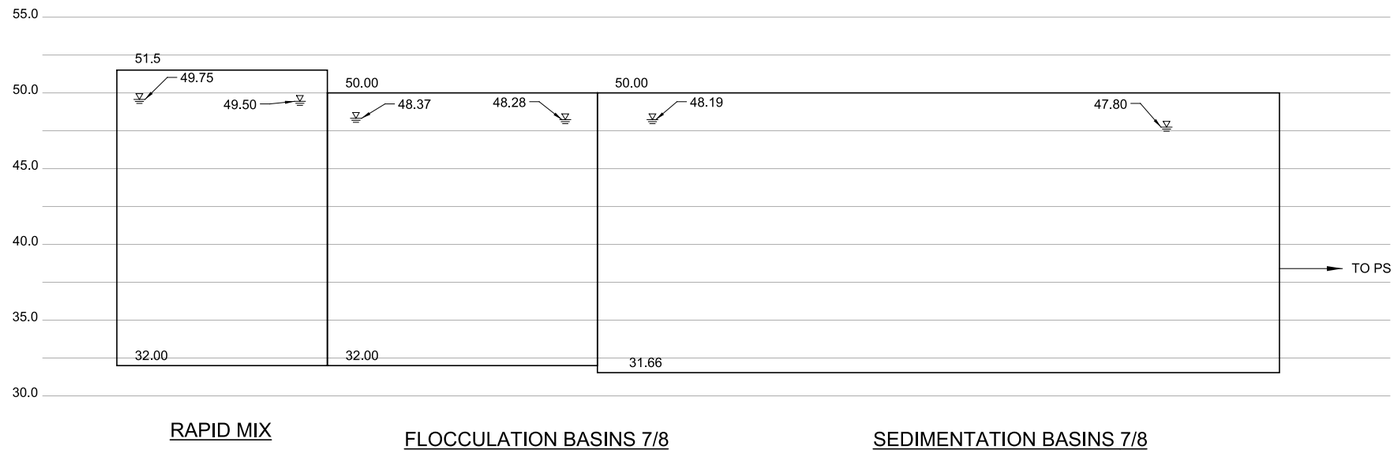
- Flow splitting from the junction box remains an issue. However, hydraulic bottlenecks were not observed between the filters and the junction box at 120 mgd.
- In this scenario, the elevation of the filter flume is over 37 feet, which affects the filter operation as detailed in Chapter 3.
- The issues cited in Section 4.1 at the filter effluent flume worsen under this scenario. The water surface elevation exceeds the top of the flumes by more than 6 feet. Under these conditions, leaking and cracking in the filter galleries would worsen.
- The top of concrete elevation in the Hawkey box (36-feet) is exceeded by more than 2 feet under these conditions.
- The hydraulic issues at the blend chamber mentioned in Section 4.2 are worsened under the 120 mgd flow conditions. The water surface elevation exceeds the top of concrete elevation by almost 4 feet under these conditions.

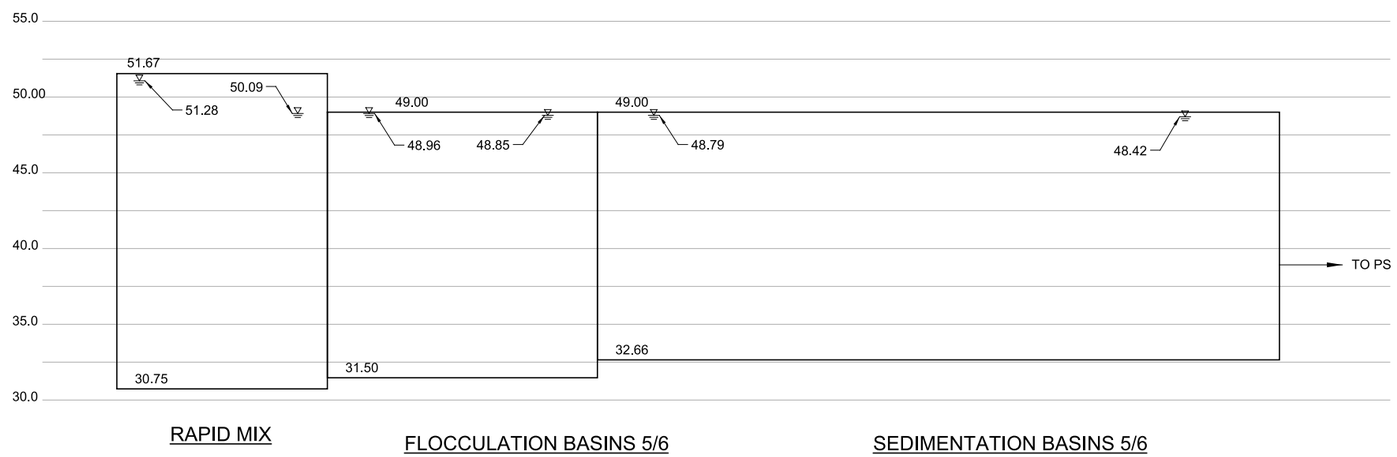
4.5.3 Future Demand and Facility Capacity Expansion

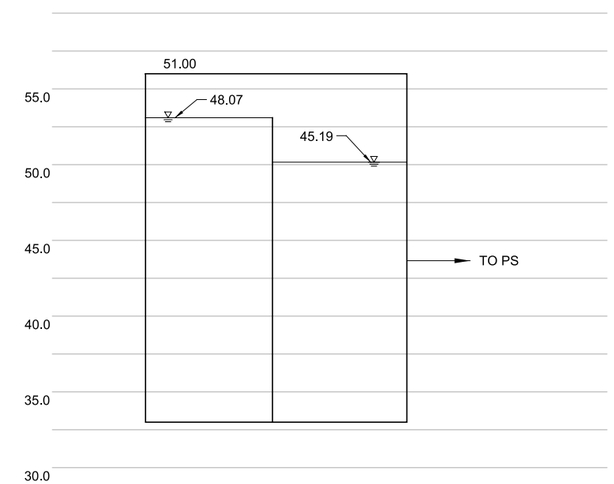
This scenario assumes a total flow of 140 mgd, which includes 10 mgd from the ASR system. The original 70/30 split to the conventional and Actiflo™ systems was revised to send 40 mgd (maximum design flow) into the Actiflo™ system and 100 mgd into the conventional system. Figure 4.6A and Figure 4.6B show the hydraulic profile for this scenario. Appendix G includes the *Hydraulix*® model for this flow scenario.

Under this scenario, all the issues discussed in Section 4.2 were present and had worsened. Additional issues that arose under the future demand conditions include:

- The pre-filter junction box overflows. The uneven flow splitting identified at 80 mgd has worsened to the point that the capacity of the junction becomes a hydraulic bottleneck.
- Operating levels of the filters receiving water from the under-utilized pre-filter junction box line may drop to the point that the head losses among the filters vary substantially. This could potentially lead to filter starvation and uneven flow splitting.
- The model indicates that the low lift pump station can be kept at a low level to prevent the overflow of Trains 5 and 6. However, DLTWTF staff cited this as a concern. It is discussed in the recommendations section.







HYDRAULIC PROFILE CURRENT PERMITTED TREATMENT CAPACITY (120 MGD) (PART A)

FIGURE 4.5A

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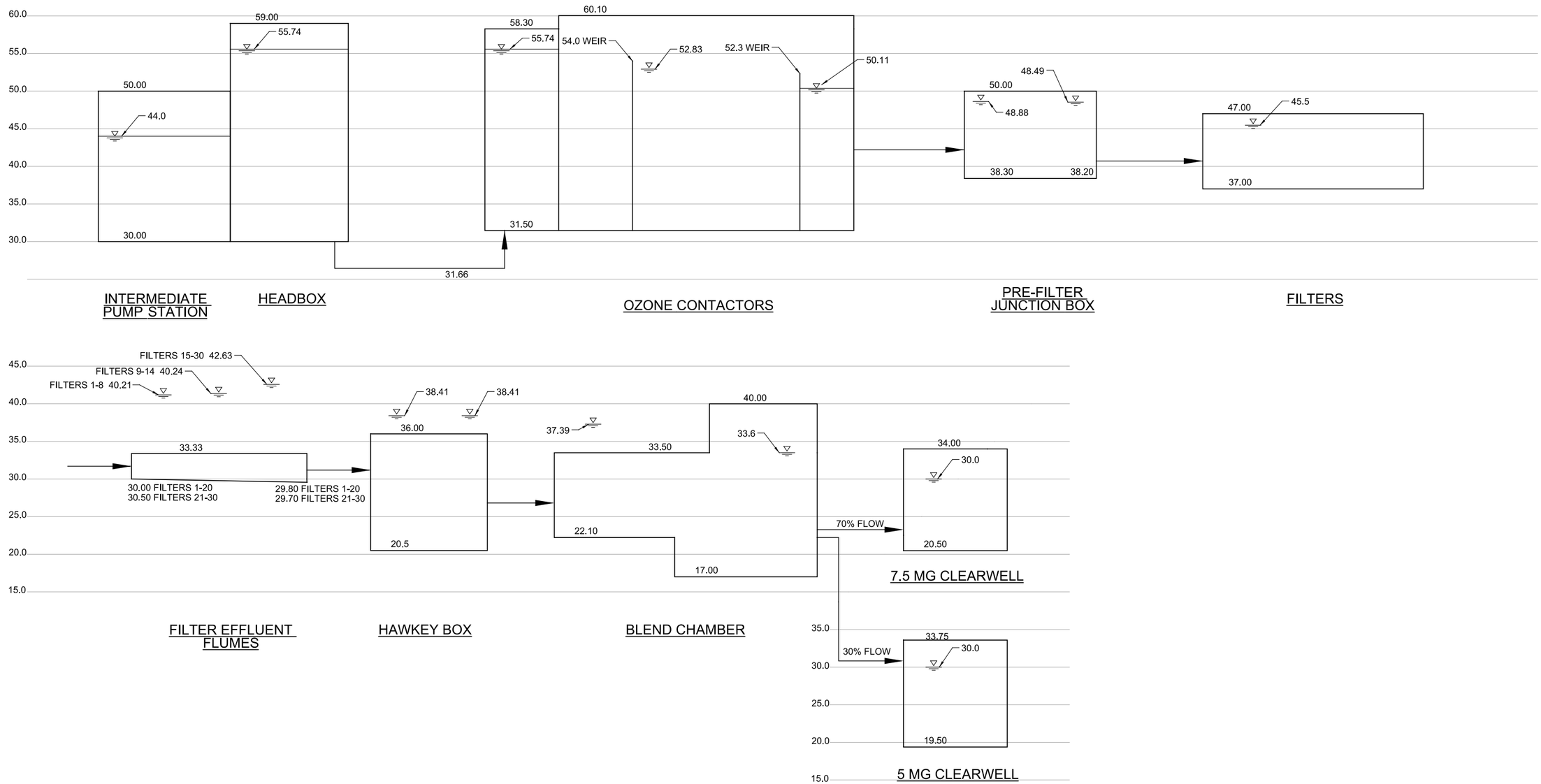

HYDRAULIC PROFILE CURRENT PERMITTED TREATMENT CAPACITY (120 MGD) (PART B)

FIGURE 4.5B

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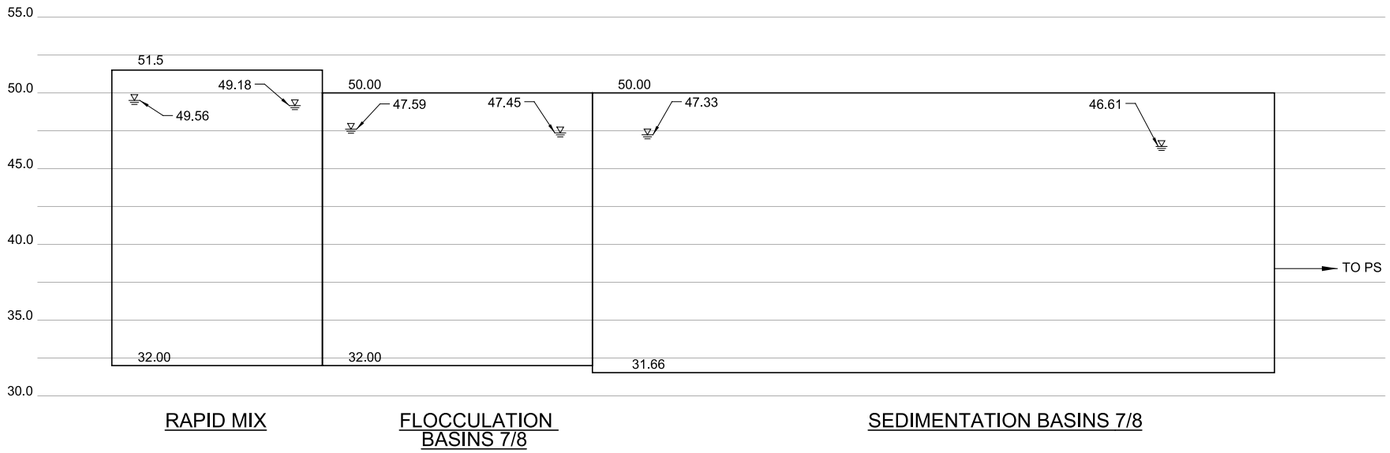

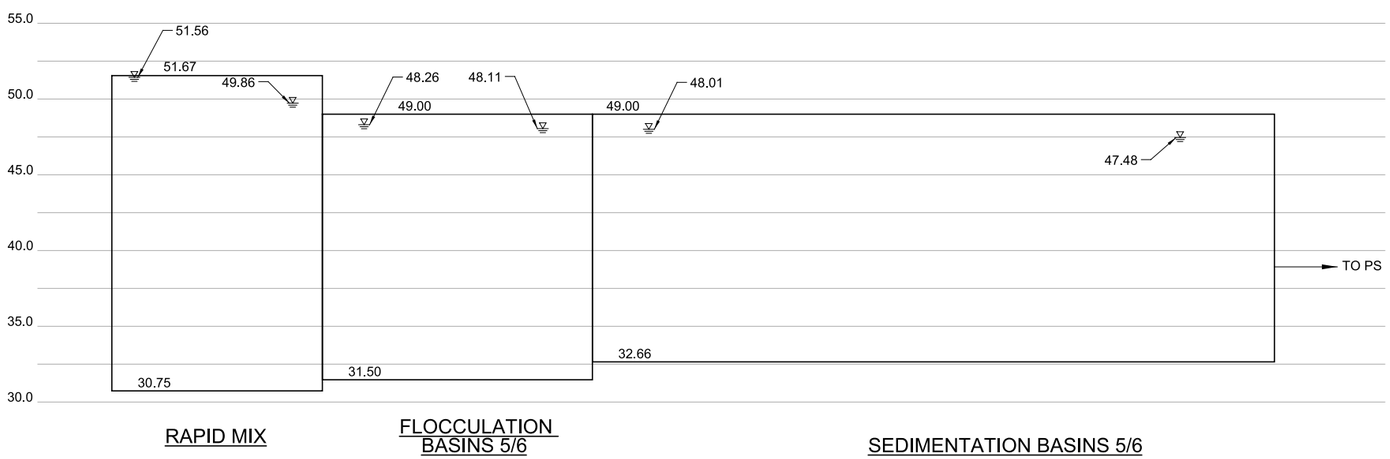

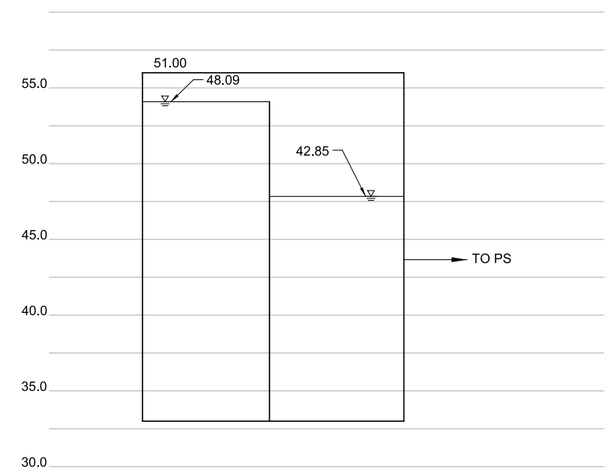

HYDRAULIC PROFILE FUTURE DEMAND FLOW (140 MGD) (PART A)

FIGURE 4.6A

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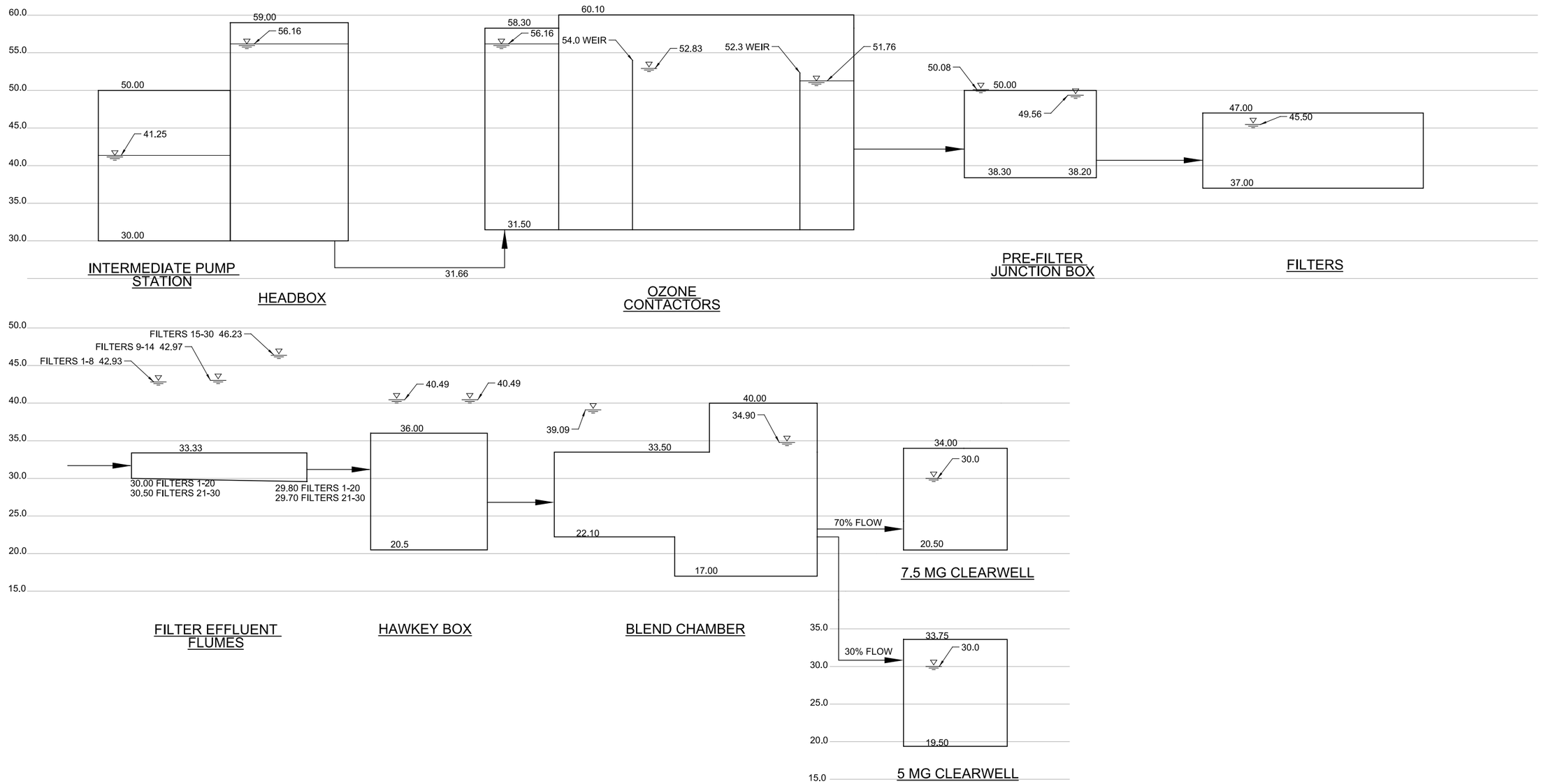

HYDRAULIC PROFILE FUTURE DEMAND FLOW (140 MGD) (PART B)

FIGURE 4.6B

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4.5.4 Ozone Influent Box Overflow

The staff noted overflow conditions at the ozone splitter box and the pump station head box, which the model did not capture, even at flows of 140 mgd. It is suspected that the valves between the ozone splitter box and the contactors may be partially closed and/or precipitation of calcium carbonate has occurred restricting the flow through the valves. The model was modified to mimic these conditions, and the diameter of the valve was reduced to 38-inches, at which point overflow of the ozone splitter box was shown at 120 mgd.

At these flows, the freeboard at the pump station head box was reduced to approximately 0.5 feet. At this level, splashing from the pumps could overflow the box, which plant staff observed at flow rates in this range. However, the restriction at the valves is not likely to be this significant.

The model calibration did not show evidence of any restriction. Therefore, there may be another explanation for the discrepancy between the model and staff field observations. One possibility is that the pump station is being drawn down below the water level required based on the allowable submergence of the pumps. According to the hydraulic institute, the allowable submergence of the pumps is 89.91 inches (although the operation and maintenance manual states 60 inches), suggesting the minimum water level is 9.17 feet above the floor with the bowl 20 inches above the floor. At lower water levels, vortexing and air entrainment may occur, resulting in issues downstream of the pumps.

4.5.5 Actiflo™ to Thickener 3

Actiflo™ sludge is transferred by gravity to Splitter Box 3. The transfer line is 10 inches at the Actiflo™ basins and expands to 16-inches for most of the distance between the basins and the splitter box. The transfer line is directed up an incline of approximately three feet between the basins and the splitter box. From Splitter Box 3, the Actiflo™ sludge can be directed to GT 3 or to Splitter Box 1/2 and then to GT 1, 2, or 4. Two main problems are associated with the Actiflo™ sludge line:

1. Solids accumulate in the transfer line.
2. When the line to Splitter Box 1/2 is open, the Actiflo™ basins begin to overflow. Therefore, only GT 3 can be used for Actiflo™ sludge.

The transfer line between the Actiflo™ basins and Splitter Box 3 was modeled in *Hydraulix*® using Actiflo™ sludge flow data provided by the City for 2014 through 2016. Average flow rates were 705 gpm (1.02 mgd) and the maximum flow rate was 1,273 gpm (1.83 mgd). The results indicate that the velocity through the 16-inch line is 1.12 fps under average conditions and 2.03 fps under maximum conditions. Therefore, the 16-inch line is likely oversized because ideally the velocity should be 3.5 fps or greater to prevent solids deposition.

4.5.6 Junction Box 4 Overflow

Supernatant from a number of systems is directed to the raw water intake. When the ASR system is in use, supernatant is also directed to the raw water intake along the same flow path. Junction Box 3 receives supernatant flows from the thickeners via gravity lines, while the ASR flows are pumped to the box when the ASR system is in use.

Stormwater flows can also be directed to Junction Box 3. Flows then travel via a 36-inch line to Junction Box 4, where the flow splits and is directed via two 20-inch lines to the raw water intake. A *Hydraulix*® model identified a hydraulic bottleneck between Junction Box 4 and the raw water intake when flows through the box were 22 mgd or greater. DLTWTF staff observed that Junction Box 4 overflowed when the measured flow at the raw water intake was 18 mgd.

Based on these observations, an accurate model should show overflows beginning to occur at flows just over 18 mgd. Therefore, the model may have underestimated the losses between Junction Box 4 and the raw water intake. However, the amount of overflow cannot be quantified, so flows may possibly reach 21.75 mgd before overflowing.

4.6 RECOMMENDED HYDRAULIC IMPROVEMENTS

This section summarizes the hydraulic issues identified by this evaluation and related recommendations. The more notable issues, including the uneven flow split from the pre-filter junction box, pressurization of the filter effluent flumes, solids accumulation within the Actiflo™ sludge line, and the overflow of Junction Box 4, are discussed in more detail along with possible solutions.

4.6.1 Pre-Filter Junction Box and Filter Influent Flow Paths

Modifications to the pre-filter junction box and the flow paths from the junction box to the filters were considered to 1) reduce head loss between the junction box and the filters to eliminate the junction box bottleneck at 140 mgd and 2) improve the consistency of the operational levels among the filters.

4.6.1.1 <u>Flow Splitting from the Pre-Filter Junction Box</u>

As discussed previously, the modeled flow split from the pre-filter junction box shows that 87 percent of the flow exits through the two 54-inch pipes (38 percent via Path 1 and 49 percent via Path 2) and 13 percent exits through the 48-inch pipe (Path 3). This flow split results in a velocity through Path 1 more than double the velocity through Path 3, while the velocity through Path 2 is more than triple the velocity through Path 3.

To achieve a more even flow split, losses along Path 3 should be eliminated. The recarbonation basins, particularly the second basin, which is heavily baffled, contribute substantial head loss. Additionally, sediment has likely built up over time in these basins, further restricting flow.

Based on available drawings and *Hydraulix*® modeling, head loss of 0.5 feet was assumed for the recarbonation basins at 80 mgd. With this input, WaterCAD predicted that the head loss through these basins will reach 1.15 feet at 120 mgd and 1.56 feet at 140 mgd. The *Hydraulix*® model indicated that the junction box will overflow at 140 mgd, with a WSE 0.08 feet above the top of the box.

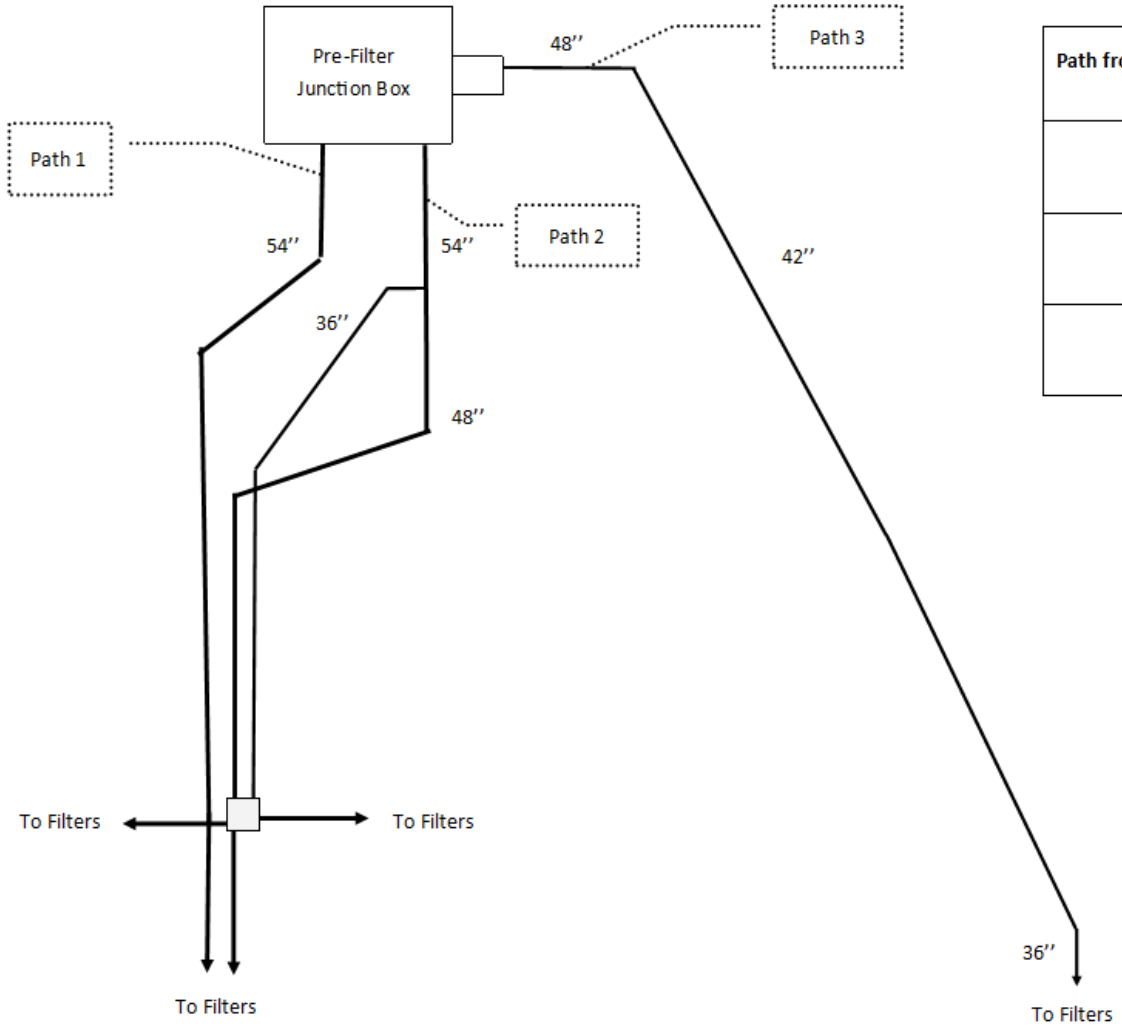
Since the recarbonation basins are no longer used for chemical addition, they can be bypassed to reduce head loss along the flow path. The WaterCAD model was updated to include a bypass of the recarbonation basins via a 42-inch pipeline (assumed to be 130 feet in length), as shown in Figure 4.7. The revised results indicate a flow split of 34 percent though Path 1, 45 percent through Path 2, and 22 percent through Path 3. Under these conditions, the velocities are more comparable through each of the three paths. Table 4.3 summarizes the flow splits and velocities through each path from the junction box.

Table 4.3 WaterCAD Flow Split Results
David L. Tippin WTF Master Plan
City of Tampa

Path from Junction Box	Existing Conditions		Recarbonation Basin Bypass	
	Flow Split (%)	Velocity in First Pipe Segment at 80 mgd (fps)	Flow Split (%)	Velocity in First Pipe Segment at 80 mgd (fps)
Path 1	38	2.93	34	2.66
Path 2	49	3.85	45	3.47
Path 3	13	1.27	21	2.09

Under the modified conditions, the model indicated that at 140 mgd, the junction box will no longer over flow, and 0.17 feet of freeboard will be available. Therefore, bypassing the recarbonation basins would eliminate the hydraulic bottleneck at the junction box.

Either in addition to or instead of bypassing the recarbonation basins, depending on feasibility and costs, another alternative is to modify the junction box itself. Currently, flow to all three flow paths enters the junction box from the ozone contactors and passes over a submerged weir. The flow then splits, with flow to Paths 1 and 2 passing through a large opening (25 feet by 4.5 feet) and flow to Path 3 entering channels under the junction box floor through 26 orifices that are 10 inches in diameter. From these channels, flow to Path 3 passes through two open gates into a smaller effluent box on the side of the junction box before reaching the 48-inch pipeline.



Path from Junction Box	Flow Split from Model (%)
1	34%
2	45%
3	21%

RECOMMENDED FLOW PATH MODIFICATIONS FROM PRE-FILTER JUNCTION BOX

FIGURE 4.7

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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At 80 mgd, the *Hydraulix*® model indicated that the opening prior to Paths 1 and 2 contributes 0.04 feet of head loss, while the orifices and gates along Path 3 contribute 0.12 feet of head loss. At 140 mgd, the orifices and gates along Path 3 contribute 0.37 feet of head loss. Therefore, eliminating the orifices and gates could help eliminate the bottleneck at the junction box.

Additionally, if the recarbonation basins are bypassed and the flow rate to Path 3 increases, the head loss at the orifices and gates will increase to approximately one foot. As Table 4.3 shows, modifying the junction box to reduce the head loss along Path 3 would further improve the flow splits. If the recarbonation basins were bypassed and the junction boxes modified, the WSE in the junction box could be brought down to 48.85 feet at 140 mgd, providing 1.15 feet of freeboard.

4.6.1.2 Filter Influent Flows

Typically, DLTWTF filter operation uses the filter effluent valve to send the desired flow rate through the filter. Opening the valve increases the flow rate through the filter, provided the influent rate is sufficient. If flows to a filter are low, the valve can be opened more slowly to increase the driving head. Though head loss is variable across the filter bed, the valve allows for the driving head to be controlled.

According to DLTWTF staff, flow rates among the filters during a filter run are generally consistent at and below 120 mgd under similar valve operation, and the influent flow for all filters provides sufficient driving head. Before adding Path 3 to the filters, however, the driving head generated by the influent flow rates among the filters changed so dramatically that the filters farthest away from the junction box (Filters 21, 23, 25, 27, and 29) were typically starved because they received much lower flows than the other filters. These conditions may become an issue again when flows to the DLTWTF increase.

At higher flows, the uneven distribution from the junction box will have a greater effect at the filters. Flow from the underutilized lines will be slower, which will eventually force the effluent valves of the filters downstream from these lines to open more to provide the process flow set by plant staff. As a result, the operating filter levels will begin to vary, and some filters will experience lower driving head than others. If the driving head differential becomes extreme enough, uneven flow splitting at the filters can result.

The WaterCAD model estimated the water surface elevations, which represent head loss, at each of the filters. At 80 mgd, the largest head loss differential was one foot, with Filters 17, 19, 21 and 23 at the lowest operating levels. The head loss differential increases to 1.54 feet and 2.08 feet at 120 mgd and 140 mgd, respectively. At 140 mgd, filters 17, 19, 21 and 23 have the lowest operating level of 42.33 feet, which is below the trough level in the filters. In fact, at 140 mgd, the filter levels in all but four of the operating filters (assuming two large filters are out of service) are below the trough level of 43.75 feet. At 120 mgd, sixteen of the operating filters have water surface elevations below the top of the troughs.

As expected, the filters operating at the lowest level are those in the filter bay served primarily by the line from the junction box that passes through the recarbonation basins (Figure 4.3 Path 3). The WaterCAD model was modified to reflect the elimination of the recarbonation basins. Under these conditions, the water surface elevations remain above the troughs until 140 mgd, at which point the lowest water surface elevation in the filters is 43.39 feet.

Head loss between the junction box and the filters must be low enough to keep the filter operating levels do not need to be altered. Improved distribution from the junction box as discussed in Section 5.1.1 (remove recarbonation basins and reconfigure junction box) should allow staff to maintain consistent operations at the expected higher flow rates. However, this should be confirmed if these recommendations are to be considered further. In addition to the changes discussed in Section 5.1.1, filter influent piping could be added to further improve distribution among the filters.

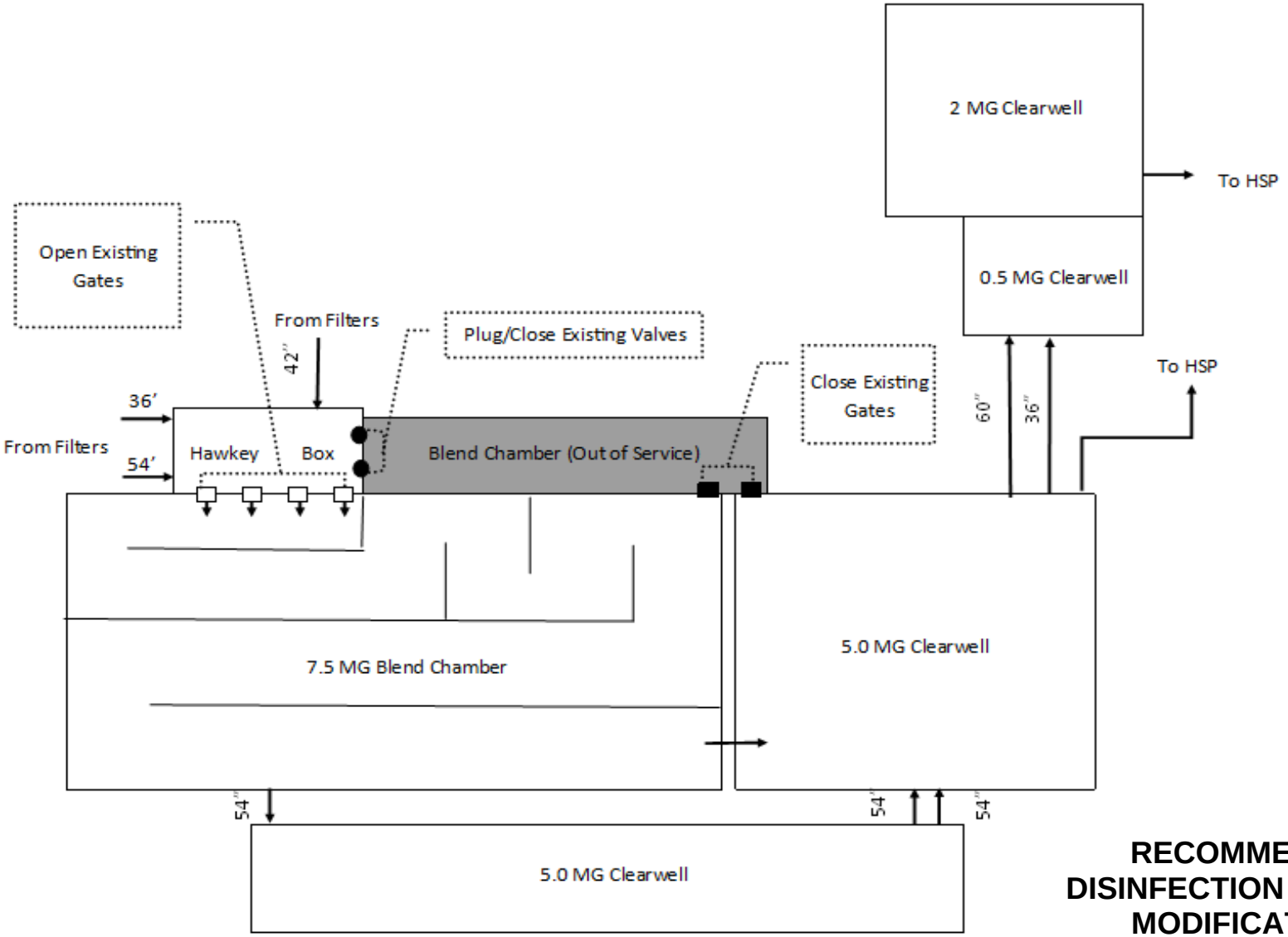
4.6.2 Pressurization of the Filter Effluent Flumes

Pressurization of the filter effluent flumes is a known issue at the DLTWTF and occurs even at flows of 70 mgd. Pressurization causes leaking in the filter gallery, which presents a cross connection issue, and structural damage to the filter galley floor. It also reduces the available head loss though the filters.

Based on the model results, pressurization issues begin at the blend chamber, which was pressurized at 80 mgd, 120 mgd, and 140 mgd. Within the blend chamber, high losses occur at the two small gates between the blend chamber and the clear well. The gates contribute 1.6 feet, 3.6 feet, and 4.9 feet of head loss at 80 mgd, 120 mgd, and 140 mgd, respectively. The baffling within the blend chamber contributes additional head loss-- 2.35 feet, 3.63 feet, and 4 feet at 80 mgd, 120 mgd, and 140 mgd, respectively. Flow enters the blend chamber from the Hawkey box via a 36-inch and 54-inch valve. The velocities though these valves were calculated to be 5.4 fps, 8.1 fps, and 9.5 fps at 80 mgd, 120 mgd, and 140 mgd, respectively. Losses through the valves varied from 0.45 feet at 80 mgd to 1.03 feet at 120 mgd and 1.4 feet at 140 mgd. Therefore, total losses associated with the use of the blend chamber range from 4.4 feet at 80 mgd to 8.26 feet at 120 mgd and 10.3 feet at 140 mgd.

To eliminate or reduce pressurization at the filter effluent flumes, bypassing the existing blend chamber and repurposing the 7.5 MG clear well to a blend chamber are recommended. Figure 4.8 includes the resulting new flow path that would utilize the 7.5 MG clear well for blending and bypass the existing blend chamber.

The Hawkey box was installed with four gates to direct flow into 7.5 MG clear well. These gates can be opened while the valves to the blend chamber are closed. The gates are expected to contribute 0.07 feet, 0.16 feet, and 0.21 feet of head loss at 80 mgd, 120 mgd, and 140 mgd, respectively. Baffling would be added to the clear well to achieve the required contact time. Baffles would not need to be water bearing, reducing the cost of these modifications.



RECOMMENDED DISINFECTION PROCESS MODIFICATIONS

FIGURE 4.8

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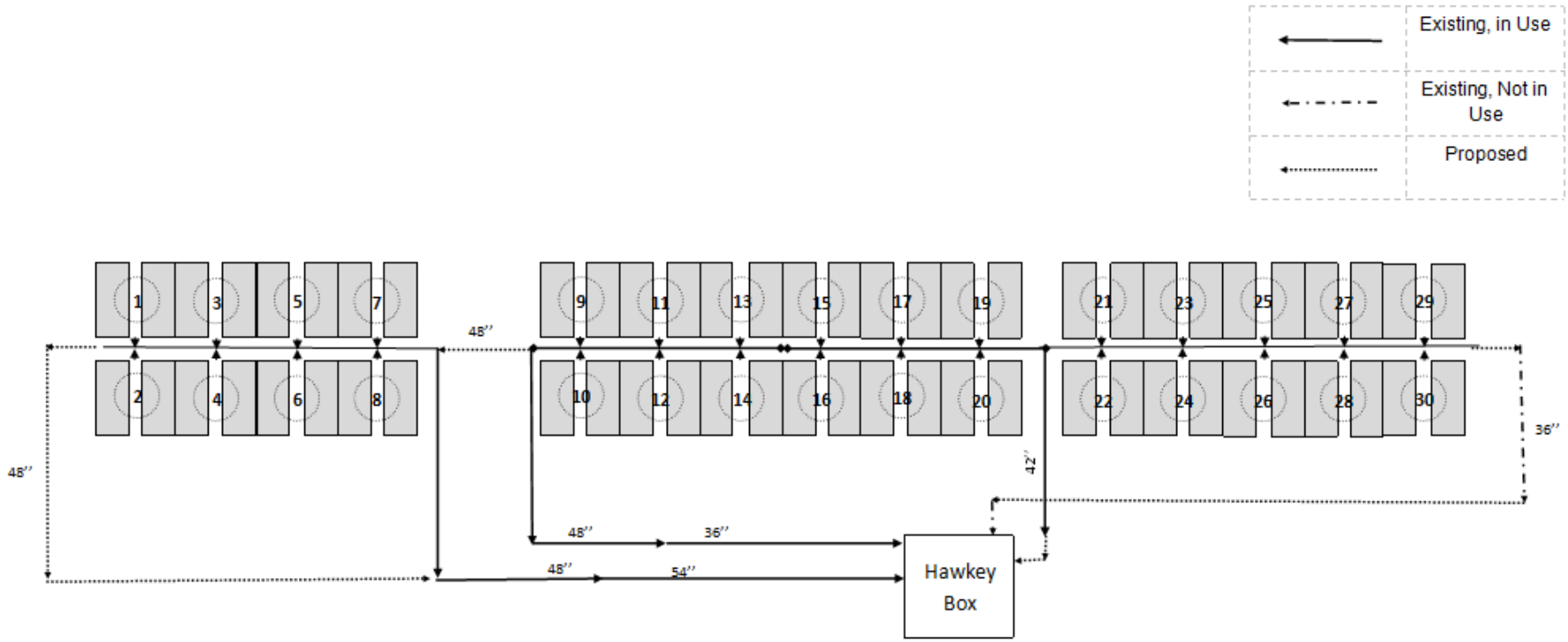
The modifications shown in Figure 4.8 were modeled using *Hydraulix*®. Results indicated that effluent flume pressurization was eliminated at 80 mgd. However, head losses were still substantial enough at 120 mgd and 140 mgd for flumes to remain pressurized. Additional flow paths from the filter effluent flumes to the Hawkey box may further alleviate head loss when flows are 120 mgd and 140 mgd.

One possible option, shown in Figure 4.9, was evaluated using WaterCAD and the *Hydraulix*® model. The option involves utilizing existing (but currently unused) piping and new piping to redirect flows along additional flow paths from the effluent flumes to the Hawkey box. The models indicate that using the additional flow paths can reduce the water surface elevations at the filter effluent flumes to below the top of flumes, eliminating the pressurization, even at 140 mgd.

Table 4.4 summarizes the approximate water surface elevations at the filter effluent flumes under the existing conditions and after modifying the 7.5 MG clear well. As Table 4.4 shows, the recommended alternative includes adding additional flow paths, eliminating the current blend chamber, and modifying the existing 7.5 MG clear well.

This recommendation is expected to resolve the filter flume pressurization issue at flows at or below 140 mgd. With these modifications, not only will filtered water flume pressurization be alleviated, but the terminal head loss available to the filters will be increased, improving the length of the filter run times, especially at high plant flow rates.



RECOMMENDED FILTER EFFLUENT FLOW PATH MODIFICATIONS

FIGURE 4.9

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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Table 4.4	Filter Effluent Flume WSE Comparison David L. Tippin WTF Master Plan City of Tampa			
	Existing Conditions		Filter Effluent Flow Modifications(1)(2)(3)	
Flow Rate	Filter Effluent Flume WSE (ft)(4)	Freeboard (ft)(5)	Filter Effluent Flume WSE (ft)(4)	Freeboard (ft)
80 mgd	36.37	-3.04	32.11	1.22
120 mgd	42.63	-9.30	32.84	0.49
140 mgd	46.23	-12.90	32.69	0.64

Notes:

(1) Alternative includes the following: Modified filter effluent piping. Existing 7.5 MG clear well baffled and used as a blend chamber. Flow enters clear well via four existing gates between clear well and Hawkey box.

(2) Calculations include losses through the modified piping routes, the existing Hawkey Box gates, and modified 7.5 MG clear well.

(3) As stated in Section 3.1, the WSE at the clear wells was originally assumed to be 30 feet. However, when the model was modified to reflect the effluent flow modifications, the filter effluent flume was nearly empty at 80 mgd and 120 mgd, which caused calculation issues in the model. To correct this, a modified scenario assumed a WSE of 31 feet in the clear wells at 80 mgd and 120 mgd. As a result, the WSE at 120 mgd is artificially higher than the WSE at 140 mgd. This change was required only because of limitations in the model and does not mean that the clear wells would need to be operated differently.

(4) Effluent flumes were modeled separately based on flow paths. Under the existing conditions, the WSEs were modeled for 1) Filters 1 to 8, 2) Filters 9 to 14, and 3) Filters 15 to 30. Under the modified conditions WSEs were modeled for 1) Filters 1 to 8, 2) Filters 9 to 14, 3) Filters 15 to 20, and 4) Filters 21 to 30. The highest WSEs, which were associated with Filters 15 to 30 (existing) and Filters 21 to 30 (modified), are reported in this table.

(5) Negative freeboard indicates overflow, which is associated with pressurization at the filter effluent flumes.

Benefits of alleviating the pressurization of the filter effluent flumes include:

- Increasing capacity through the DLTWTF.
- Eliminating a cross connection in filter gallery.
- Increasing the amount of terminal head loss available across the filters and therefore increasing filter run times.

4.6.3 Actiflo™ to Thickener 3

The velocity of the sludge in the existing Actiflo™ line was modeled and found to be below 3.5 fps. At these low velocities (1.12 fps on average, 2.03 fps maximum), solids are expected to accumulate. The velocity could be increased by replacing the 16-inch line with a smaller diameter pipe. At average flows, an 8-inch line would increase flow velocity to greater than 3.5 fps.

The additional head loss associated with a line of this size, however, would cause overflow conditions at the Actiflo™ basins. To accommodate the additional head loss, the Actiflo™ sludge could be piped to a new junction box at a lower elevation and then pumped to the thickener. However, space in the vicinity of the thickeners is limited, and this alternative could be costly.

Alternatively, a new sludge wet well and pump station could be installed downhill near the Actiflo™ basins where sludge would be pumped up to GT3. Finally, as another option, a sand separator could be added along the Actiflo™ sludge line near the basins. Accumulated solids could be easily removed without having to pig the line. The design engineer should investigated these alternatives to determine the most appropriate solution.

4.6.4 Junction Box 4 Overflow

The hydraulic model indicated that Junction Box 4 will overflow at flow rates of 22 mgd or greater. When it overflows, the amount of overflow water cannot be quantified because the flow is only measured downstream of the overflow point. Therefore, the required capacity of the junction box cannot be determined to make appropriate recommendations for modifying it and/or piping.

Additionally, without understanding the amount of flow currently entering the junction box from various sources (ASR, stormwater, thickener supernatant), it is not possible to accurately project how these flows may increase when DLTWTF flows increase. For example, though the ASR flows may be consistent, stormwater flows depend on rainfall while the supernatant flows relate to DLTWTF flows.

To recommend appropriate modifications to the junction box, such as increasing concrete height, supernatant flows would need to be projected based on expected DLTWTF flow increases. Additionally, varying stormwater flows throughout the year would be required to account for seasonal differences in flows.

Therefore, it is recommended that the City begin monitoring flows to Junction Box 3 including stormwater flows. A flow meter could also be placed upstream of Junction Box 4 along the 36-inch line. Supernatant flows could then be calculated by subtracting stormwater and ASR flows from the total flow to Junction Box 4. During times of overflow, the flows from Junction Box 4 measured at the intake could be subtracted to determine the amount of overflow. A better understanding of the situation as it is now is needed before specific recommendations can be made for preventing the overflow.

4.7 SUMMARY OF RECOMMENDATIONS

The DLTWTF experiences few limitations at 80 mgd, most notably the pressurization of the filter effluent flumes. As flows increase, hydraulic limitations begin to worsen and/or develop at the filter effluent channels and pre-filter junction box. Table 4.5 shows hydraulic issues in the liquid stream flow identified in this evaluation and recommendations for resolving them.

Table 4.5 Recommendations for Liquid Stream Flow
David L. Tippin WTF Master Plan
City of Tampa

Hydraulic Limitation	Description/Recommendation
Pre-Filter Junction Box Capacity and Filter Operations	As discussed in Section 5.1, the capacity of the junction box is currently limited due to the under-utilization of Path 3 (Figure 4.3). Flow through Path 3 includes unnecessary flow restrictions caused by the configuration of the junction box and flow routing through two old recarbonation basins. Under the existing conditions, the Hydraulix® model indicated the junction box does not have the capacity to meet future demands of 140 mgd. At these high flows, the junction box was shown to overflow, with a calculated water surface elevation 0.08 feet above the top of the box. The resulting uneven flow split can also result in inconsistent filter influent flows, which may result in starved filters at higher flow rates. To encourage more flow through Path 3 to the filters, is recommended that the recarbonation basins be bypassed (as shown in Figure 4.8) and that the junction box be modified (e.g. direct flow over a weir instead of through 10-inch orifices and small gates) to encourage more flow through Path 3 to the filters. When the recommended modifications were incorporated in the model, losses were reduced substantially and, at 140 mgd, 1.15-feet of freeboard remained in the junction box. While WaterCAD indicated that filter operation would be more consistent following these modifications based on flows to the filters, the flows through the filters could not be modeled accurately, so further evaluation is required to confirm if flow splitting among the filters will be improved by these modifications.
Filter Effluent Flume Pressurization	As discussed in Section 5.2, the blend chamber was identified as a source of substantial head loss accumulation and contributed to pressurization at the filter effluent flumes. It is recommended that the City consider eliminating the existing blend chamber and retrofitting the 7.5 MG clear well to function as a blend chamber. The revised flow path is expected to allow the filter effluent flumes to flow freely at 80 mgd. At 120 mgd and 140 mgd, additional flow paths from the filters are required to further reduce head loss and prevent filter effluent flume pressurization.

Table 4.5 Recommendations for Liquid Stream Flow David L. Tippin WTF Master Plan City of Tampa	
Hydraulic Limitation	**Description/Recommendation**
Incomplete Filtration	At 120 mgd and 140 mgd, the hydraulic model indicated that filter effluent water level exceeds 37 feet, which interferes with filtration. The modifications described in Section 4.6.2, which alleviate the pressurization of the effluent flumes, would also eliminate this issue.
Hawkey Box and Blend Chamber Pressurization	These limitations result from the size and configuration of the existing blend chamber, which would be eliminated by the modifications described in Section 4.6.2.
Overflow of Trains 5 and 6	When flows through the conventional treatment Trains 5 through 8 reach 100 mgd, which occurs under the future condition scenario (140 mgd through the DLTWTF), the hydraulic model indicated that overflow of Trains 5 and 6 may occur if the low lift pump station is not kept at 41.25 feet or below. At 41.25 feet, the WSE is 11.25 feet above the bottom of the basin. The WSE is required to be greater than 9.17 feet based on the minimum submergence of the pumps. Therefore, continuing to operate the pump station in this manner is acceptable at times of high flow. Modifications to the sedimentation basins, such as the elimination of the launders within the sedimentation basins, would eliminate some of the head loss and somewhat reduce the operational restrictions at the pump station. It is possible that solids have built up over time in the lines between the sedimentation basins and the pump station, contributing additional head loss. Visual inspection of these lines is recommended to determine the extent of this accumulation.
Ozone Influent Box and Pump Station Head Box Overflow	These limitations were not captured in the hydraulic model until it was modified to reflect a restriction at the valves between the influent box and the contactors. The issue was demonstrated in the model when the 60-inch valves were given a 38-inch restriction. Though restricting this level is unlikely and does not match the calibrated model, plant staff should investigate the valves to determine if they are partially closed or if calcium carbonate has accumulated over time. Additionally, operating the pump station at high flows may be causing air entrainment, resulting in issues downstream. The pump station should be operated at a depth no lower than 39.17 feet.

The solids process was evaluated by reviewing relevant plans and other documentation provided by the City. Table 4.6 summarizes the hydraulic limitations in the solids process as noted by staff and the recommendations to address them.

Table 4.6 Recommendations for Solids Stream Flow David L. Tippin WTF Master Plan City of Tampa	
Hydraulic Limitation	**Description/Recommendation**
Actiflo™ Solids Accumulating in Process Piping to Thickener 3	Solids from the Actiflo™ system flow by gravity to GT 3. DLTWTF staff noted that solids accumulate in this line over time. Options for addressing this issue are limited, as only GT 3 can receive sludge from the Actiflo™ basins, and the elevations at the thickener cannot be changed. A *Hydraulix*® model indicated that the line, which increases from 10 inches to 16 inches, is oversized. Based on historical flow data, this creates flows that are too low to push the solids through to the thickener at average and maximum flows. However, the head losses associated with a smaller line would result in overflow issues at the Actiflo™ basins, even at average flows. Installing a pump station or sand separator at the start of the Actiflo™ sludge line could reduce solids accumulation along the line and reduce or prevent the need for pigging the line.
Junction Box 4 Overflow	Junction Box 3 receives supernatant from the four thickeners, storm water flows, and flow from the ASR system. From Junction Box 3, water flows by gravity to Junction Box 4 and then to the DLTWTF intake. Staff has observed that Junction Box 4 overflows when the ASR is in use. Currently, however, flows are only measured downstream of Junction Box 4, so the amount of overflow cannot be quantified. Junction Box 4 has a top elevation of 44 feet, which is 5 feet lower than the upstream Junction Box 3. Hydraulic modeling suggests increasing the height of Junction Box 4 would eliminate the bottleneck. To determine the appropriate height for the junction box or identify other alternatives, the maximum and expected future flow rates through the box need to be identified. Therefore, the City is advised to begin monitoring flows to Junction Box 3, including the storm water flows, and to install a flow meter between Junction Box 3 and Junction Box 4.

Table 4.6 Recommendations for Solids Stream Flow David L. Tippin WTF Master Plan City of Tampa	
Hydraulic Limitation	**Description/Recommendation**
Washwater Surge Tank	Operational changes at the DLTWTF have largely addressed issues with the washwater surge tank. Before these changes, much of the flow from the surge tank was recirculated instead of being directed to the thickeners. The number of pumps recirculating water is controlled by the water surface level in the tank. To reduce the amount of water being used for recirculation, plant staff keeps the water surface level in the surge tank low, allowing more flow to reach the thickeners. Larger holes were cut into the recirculation nozzles to help maintain appropriate recirculation with fewer pumps in operation. To address this issue without continuing to operate the tank at a low volume, submersible mixers could be installed to replace the recirculation system.

The recommendations in this report are expected to greatly improve the DLTWTF's capacity by addressing hydraulic limitations. However, further evaluations are necessary to develop additional process improvements to meet demands of 120 mgd, and especially 140 mgd.

ALTERNATIVES DEVELOPMENT, ANALYSIS, AND RANKING

5.1 CHAPTER SUMMARY

Five alternatives to optimize and/or replace the enhanced coagulation (EC) treatment process at the David L. Tippin Water Treatment Facility (DLTWTF) were developed and evaluated, and their advantages and disadvantages, costs, and overall viability were studied.

By 2032, the DLTWTF needs to be expanded to a process capacity of 140 million gallons per day (mgd), assuming 96 percent efficiency (the percent of influent flow that reaches the distribution system after subtracting in-plant use/losses). The DLTWTF must also continue to meet water quality goals, improve efficiencies, and be prepared to treat up to 50 mgd of alternative water supply from the Tampa Augmentation Project (TAP).

Viable process alternatives vetted at a two-day workshop with Carollo and City staff resulted in the following alternatives.

- Alternative 1 - Baseline: Maintain existing Actiflo™ and conventional EC treatment processes.
  - Alternative 1A - Use the existing conventional trains to treat 60 mgd, retain the existing 40 mgd of Actiflo™ trains, and install 40 mgd of new Actiflo™ trains to meet 140 mgd future demands.
  - Alternative 1B - Improve the conventional trains to treat 100 mgd and retain the existing 40 mgd Actiflo™ train.
- Alternative 2 - Add ion exchange as a pretreatment step to the existing processes.
  - Alternative 2A - Include fluidized bed ion exchange (MIEX).
  - Alternative 2B - Include fixed bed ion exchange.
- Alternative 3 - Split treatment with micro/ultrafiltration membranes to blend with existing treatment processes.
- Alternative 4 - Replace existing treatment processes with micro/ultrafiltration and nanofiltration treatment with biological roughing filters.

Alternative 4 was deemed unsuitable because of the disadvantages detailed herein. Thus, it was not included in the alternatives analysis and ranking exercise.

Capital, operating, and life-cycle costs were developed for each alternative. Items common and equivalent for all alternatives were not included in the cost, therefore life-cycle costs are for comparative purposes only. Anticipated capital and construction costs are included in Chapter 9 that details the capital improvement projects.

Although cost is an important consideration in determining the final recommendation, it is not the only consideration. The alternatives were also ranked and scored based on the following critical success factors developed with City staff during the project kickoff meeting:

1. Optimization or replacement of processes, process controls, and monitoring to provide reduced treatment costs. (6 votes)
2. Provide the best water quality possible (including finished water stability), in the most efficient way possible. (5 votes)
3. Select alternatives based on cost effectiveness, considering operations and maintenance costs as well as capital costs. (4 votes)
4. Optimize operations and improve plant hydraulics to provide maximum (and efficient) utilization of existing facilities (2 votes)
5. Maximize reliability and provide suitable redundancy. (2 votes)
6. Provide the safest and most secure facility within reasonable costs. (1 vote)
7. Meet future regulations. (1 vote)

The cost evaluation, ranking analysis, and in-depth discussions with the City resulted in the recommendation that the City proceed with the implementation of Alternative 2A (improving/expanding the conventional treatment trains and retaining the existing Actiflo™ treatment trains and implementing a new 140 mgd MIEX pretreatment system) based on the results of a pilot study conducted from October 2017 – April 2018. These results are summarized in the Final Pilot Study Report completed by Carollo Engineers, Inc. in June 2018. The findings from the pilot study were utilized to update the draft version of this master plan and finalize capital, operating, and life cycle costs in this chapter and the final Capital Improvement Plan (CIP). Although, Alternative 2A has a 20-year life cycle cost 10.2 percent higher than Alternative 1B, this cost difference is substantially lessened with longer life cycle times which take greater advantage of the annual cost savings of Alternative 2A. Additionally, the primary disadvantage of proceeding with Alternative 1B over Alternative 2A is the non-monetary consideration of continuing to store and handle strong acids and bases onsite (sulfuric acid, caustic soda, lime, etc.).

Implementation of Alternative 2A will correct the conventional treatment trains' existing limitations and expand the overall plant capacity to 140 mgd in the existing infrastructure in additional to a new MIEX system as a pre-treatment step. As part of this master plan, additional filters are proposed to allow treatment of the full 140 mgd, though full scale testing after certain hydraulic improvements are made upstream may allow for refinement (reduction) of the number of filters required.

5.2 INTRODUCTION

TOC removal is the primary driver for the enhanced coagulation (EC) process now used at the DLTWTF to produce high-quality, potable water. This process uses high doses (approximately 120 milligrams per liter (mg/L) on average) of metal coagulant, in this case ferric sulfate, to form a precipitate on which organic matter is adsorbed and removed from the influent flow stream. The TOC-laden precipitate (floc) then settles in the conventional basins or Actiflo™ system.

To maximize the effectiveness of this EC process, the pH is depressed with sulfuric acid and later restored using a combination of caustic soda and lime. The process not only requires a vast amount chemicals for pH control and coagulation, it also produces significant residuals that require thickening, dewatering, and ultimate disposal.

The high operational cost of the EC process, detrimental effects low pH and aggressive water have on the basins and submerged equipment, and safety concerns about storing and handling high volumes of hazardous chemicals prompted the DLTWTF to consider alternative processes.

Five alternatives were evaluated to optimize and/or replace the enhanced EC treatment and solids handling processes. This chapter summarizes the alternatives and the advantages and disadvantages, costs, and viability of each. Additionally, each alternative was examined to see if they could replace the EC process (such as ion exchange) while still achieving the City's goals for TOC removal and overall finished water quality.

The alternatives evaluation included the following:

- Preparation and refinement of candidate technologies in collaboration with the City
- Development of pros and cons for each alternative
- Development of capital, operating, and life-cycle costs for each alternative presented
- Bench scale testing focused on optimizing existing systems and on alternative treatment technologies

With consideration of the results from previous tasks and the findings of this task, each alternative was analyzed and ranked to determine the most feasible alternative that provides equal or improved finished water quality, while reducing operating and treatment costs.

5.2.1 Methods, Assumptions, and Approach

For each alternative, the size, type, and location (within the process train) of the alternative treatment systems were considered. Also analyzed were conceptual level cost estimates, existing DLTWTF demolition requirements, and the pros and cons of the alternative

systems. The following sections describe additional considerations and the approach for ranking and determination of the final recommendation.

5.2.1.1 Raw and Finished Water Quality

Current raw and finished water quality at the DLTWTF was considered to develop the alternatives. TOC, taste and odor, and color removal are major treatment objectives, since, compared to others, the DLTWTF sees high concentrations of each in the rainy season.

The purpose of the master plan is to recommend ways the DLTWTF can continue to supply safe drinking water and meet established finished water quality goals while also considering any need for expansion. The intent of this task is to maintain or improve the finished water quality.

5.2.1.2 Facility Expansion and Future Treatment Considerations

As previously noted, the DLTWTF must be able to meet a max day demand of 134 mgd by 2032. Assuming 96 percent efficiency (the percent of influent flow that reaches the distribution system after subtracting in-plant uses and losses), the DLTWTF must expand to a process capacity of 140 mgd.

Additionally, the DLTWTF may be required to process and treat up to 50 mgd of alternative water supply as part of the Tampa Augmentation Project (TAP). There is potential that a indirect or direct potable reuse (I/DPR) connection option will become a viable alternative water supply for the City in the future. Therefore, each alternative was also assessed for its ability to remove contaminants of emerging concern (CECs) and personal care products (PCPs).

5.2.1.3 Alternative Analysis, Ranking, and Selection

Each treatment alternative was analyzed and ranked according to critical success factors developed with the City at the project kickoff meeting. Each critical success factor was weighted based on the number of votes received; those with many votes signify the importance of that factor to the success of the project. The critical success factors and their respective votes were as follows:

1. Optimization or replacement of processes, process controls, and monitoring to provide reduced treatment costs. (6 votes)
2. Provide the best water quality possible (including finished water stability), in the most efficient way possible. (5 votes)
3. Select alternatives based on cost effectiveness, considering operations and maintenance costs as well as capital costs. (4 votes)
4. Optimize operations and improve plant hydraulics to provide maximum (and efficient) utilization of existing facilities (2 votes)
5. Maximize reliability and provide suitable redundancy. (2 votes)

6. Provide the safest and most secure facility within reasonable costs. (1 vote)

7. Meet future regulations. (1 vote)

Each alternative received a score from 1 to 10 for each factor, where a 10 best satisfied the critical success factor. The score was then multiplied by the corresponding weighting factor (number of votes) for a composite score for that factor. The sum of the composite scores for each factor was considered for each alternative, allowing alternatives to then be compared to one another.

The alternative with the highest score out of 200 was selected as the best treatment alternative that satisfies the City's needs most effectively and efficiently. Section 5.5 of this chapter shows the results of this analysis.

5.2.1.4 Development of Capital and Operating Costs

Capital costs were determined for each alternative. They incorporate direct and indirect costs, including, but not limited to, materials, labor, installation, contractor overhead and profit, and contingency. They were developed in accordance with a Class IV opinion of probable cost of construction as defined by the Association of Advancement for Cost Engineering (AACE).The expected accuracy range is from +30 percent to -15 percent. Class IV budget estimates are typically prepared for master planning and based on preliminary process flow diagrams, main process systems, plant schematic layouts, and major equipment.

Operating and maintenance costs were developed based on knowledge of the DLTWTF's existing power and chemical costs in addition to annual costs for each alternative. It was assumed that chemicals and power costs will increase at a rate of 3 percent per year, while sludge disposal costs will increase at a rate of 6 percent per year due to the reduction in available land as the population grows. Operating costs were evaluated at average annual daily flows for each year based on the flow projections from the Transmission and Distribution System Master Plan completed by B&V.

After detailed operating costs were developed, a life-cycle cost was calculated to determine the 20-year net present value of each alternative. Operating costs were discounted at a rate of 3 percent to net present value.

To develop capital and O&M costs, only costs that differed among the alternatives were used. Items or systems not included in the alternative costs that will be implemented regardless of the alternative chosen include, but are not limited to, the following:

- Conventional sludge collection mechanism retrofit.
- Pump stations rehab or replacement, including raw, low lift, and high service.
- Additional clearwell storage.
- Recommendations addressing hydraulic and/or redundancy limitations at the facility.

- Recommendations for meeting condition assessment needs.
- New and expanded ozone system.
- New intake screening systems.
- Onsite solids handling and gravity thickening modifications.

Costs of recommendations made from evaluating the regulatory evaluation, condition assessment, hydraulic evaluation, and process evaluation are not included in the life-cycle cost for each alternative. Instead, they are captured in each of their respective chapters and included in the 15-year CIP. Any deviations from these recommendations, although seldom, are noted in this chapter.

Conversely, if one alternative required more chemical use than another, the difference was captured in the O&M cost (and subsequently the life-cycle cost) for all alternatives. As a result, life-cycle costs shown in this chapter are for comparing alternatives only. They do not reflect the entire CIP budget.

5.3 ALTERNATIVES DEVELOPMENT

5.3.1 Treatment Objectives and Applicable Technologies

Table 5.1 shows the City's finished water quality goals. These goals and the future impacts of TAP were assessed for this alternatives evaluation.

Table 5.1 Finished Water Quality Goals
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Parameter	Units	Value (Min - Max)
pH	units	7.80 - 8.00
Turbidity	NTU	< 0.08
Total Organic Carbon	mg/L	1.00 - 3.00
True Color	pcu	< 10
Geosmin	ng/L	< 3.0
MIB	ng/L	< 3.0
Free Ammonia	ppm	0.10 - 0.18
Fluoride	mg/L	0.65 - 0.75
Chlorine Residual	ppm	4.25 - 4.75

Carollo and City staff vetted viable process alternatives at a two-day workshop which resulted in the following alternatives and options:

- Alternative 1 - Baseline: Maintain existing EC treatment processes (Actiflo™ and conventional).

- Alternative 1A - Use the existing conventional trains to treat 60 mgd, retain the existing 40 mgd of Actiflo™ trains, and install 40 mgd of new Actiflo™ trains to meet 140 mgd future demands.
- Alternative 1B - Improve the conventional trains to treat 100 mgd and retain the existing 40 mgd of Actiflo™ trains.

- Alternative 2 - Add ion exchange as a pretreatment step to the existing processes.
  - Alternative 2A - Include fluidized bed ion exchange (MIEX).
  - Alternative 2B - Include fixed bed ion exchange.
- Alternative 3 - Split treatment with micro/ultrafiltration membranes to blend with existing treatment processes
- Alternative 4 - Replace existing treatment processes with micro/ultrafiltration and nanofiltration treatment with biological roughing filters.

5.3.2 Alternative 1 - Baseline

5.3.2.1 Overview

The existing treatment process removes TOC and color through EC and taste and odor compounds through ozone followed by BAF. Considering the implementation of TAP, the process is also well suited for removing microconstituents with the ozone and BAF unit processes.

Two options were considered. Both maintain enhanced coagulation and allow expansion of facilities to meet 140 mgd maximum day demands. The first alternative includes treating 60 mgd through the existing conventional coagulation/flocculation systems (Trains 5 through 8), retaining the existing Actiflo™ trains at the current capacity of 40 mgd, and implementing new 40 mgd of Actiflo™ trains. The second alternative involves upgrading the existing conventional system (Trains 5 through 8) to 100 mgd capacity and retaining the 40 mgd Actiflo™ trains. The following sections discuss each option and its associated capital and operating costs in more detail.

5.3.2.2 Alternative 1A - New Actiflo™

5.3.2.2.1 *Description and Process Flow*

As detailed in Chapter 3, the existing conventional treatment system capacity is limited by:

- Insufficient rapid mixing.
- Insufficient flocculation detention time and mixing.
- Short-circuiting within flocculation stages.
- High port and orifice velocities likely causing floc shearing in flocculation and sedimentation stages.

- High horizontal velocity within the sedimentation basins.

For these reasons, this alternative maintains the existing design and uses the conventional trains to treat 60 mgd instead of 80 mgd (from 20 mgd to 15 mgd per train). Operating each train at 15 mgd would allow for more favorable flocculation and settling conditions without modifying the basins. To expand the DLTWTF capacity to meet 140 mgd demands by 2032, two new 20 mgd Actiflo™ trains will be installed, for a total of 80 mgd of Actiflo™ capacity and 60 mgd conventional capacity. EC would remain the primary treatment process.

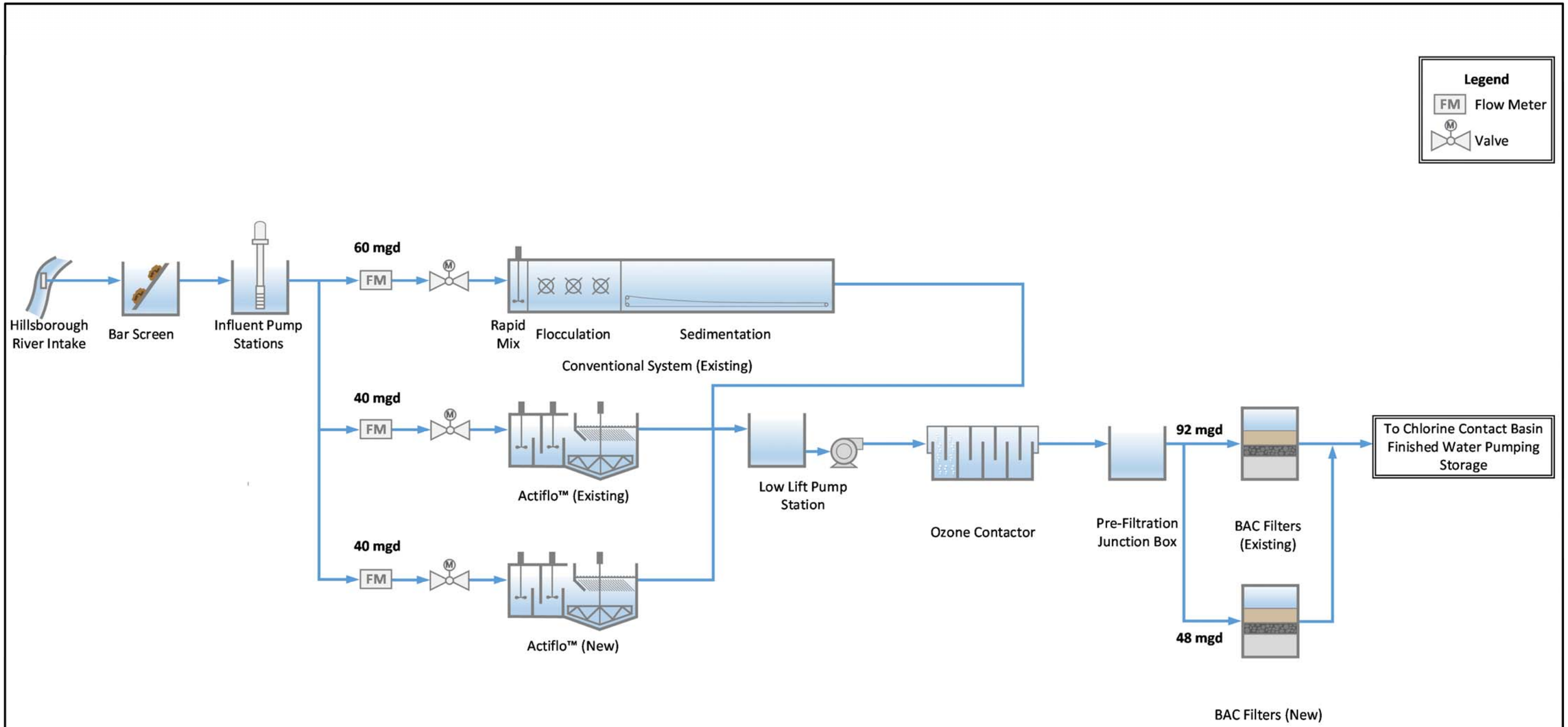
The benefits of this alternative include:

- Minimized additional infrastructure and facilities.
- Minimized changes in plant operating protocols and staff training requirements.
- Lowest capital cost.
- Improved performance of conventional trains without modification due to reduced treatment capacity.
- Improved overall plant water quality due to higher Actiflo™ capacity, which has historically better performance than conventional.
- Ability to add ion exchange or other pretreatment process in the future.
- Minimized impacts to plant operation during construction.

Conversely, the drawbacks are:

- High operating costs (chemical).
- Need for chemical system expansion.
- High solids production, solids handling requirements, and sludge hauling costs.
- Recurring rehabilitation required of infrastructure (concrete, metals, etc.) due to low pH process water.
- No mechanical improvements to conventional system.

Figure 5.1 shows the proposed process flow diagram for this alternative.



ALTERNATIVE 1A PROCESS FLOW DIAGRAM

FIGURE 5.1

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

carollo

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5.3.2.2.2 *CFD Modeling Results*

Carollo performed computation fluid dynamic (CFD) modeling of the existing conventional rapid mix, flocculation, and sedimentation basins to further identify the flow paths and restrictions of the existing design at the proposed flow of 15 mgd per train. Figure 5.2 shows several cross sections of the CFD model.

As the figure shows, at 15 mgd per train, uneven distribution of mixing energy and flow velocities still occurs, signifying extreme short-circuiting in the floc and sedimentation basins, non-laminar flow, and likely re-suspension of floc in the sedimentation basins.

However, compared to the 20 mgd per train model shown in Chapter 3, some issues in the existing system can be mitigated by reducing the flow to 15 mgd per train. Fully optimizing the conventional system to remedy these issues would require modifying the system as described in Alternative 1B.

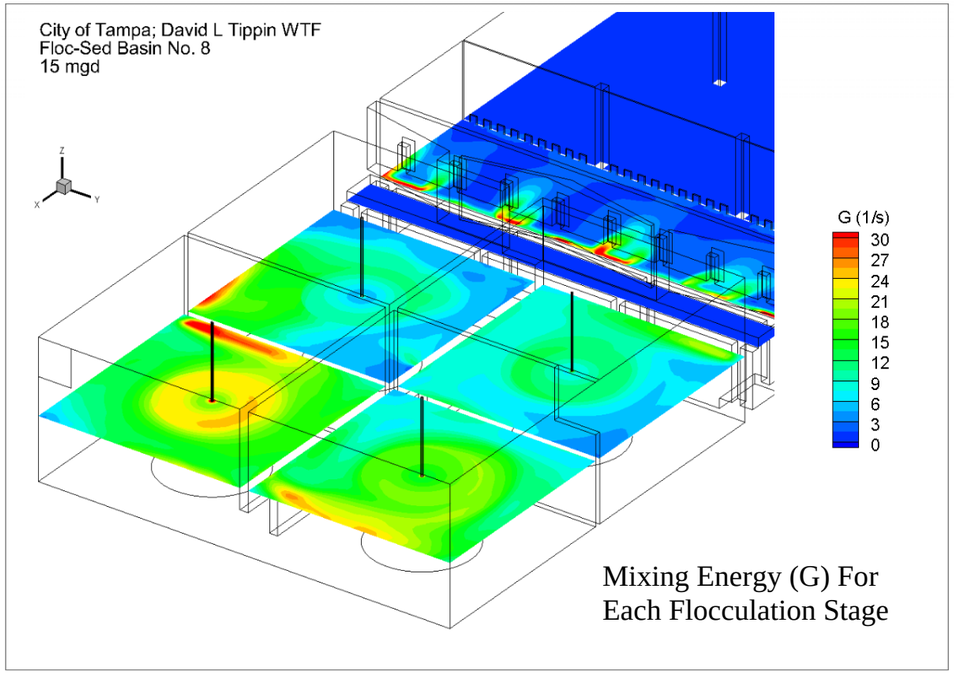
5.3.2.2.3 *Required Facility Modifications*

This alternative requires modifications to the existing DLTWTF, including, but not limited, to:

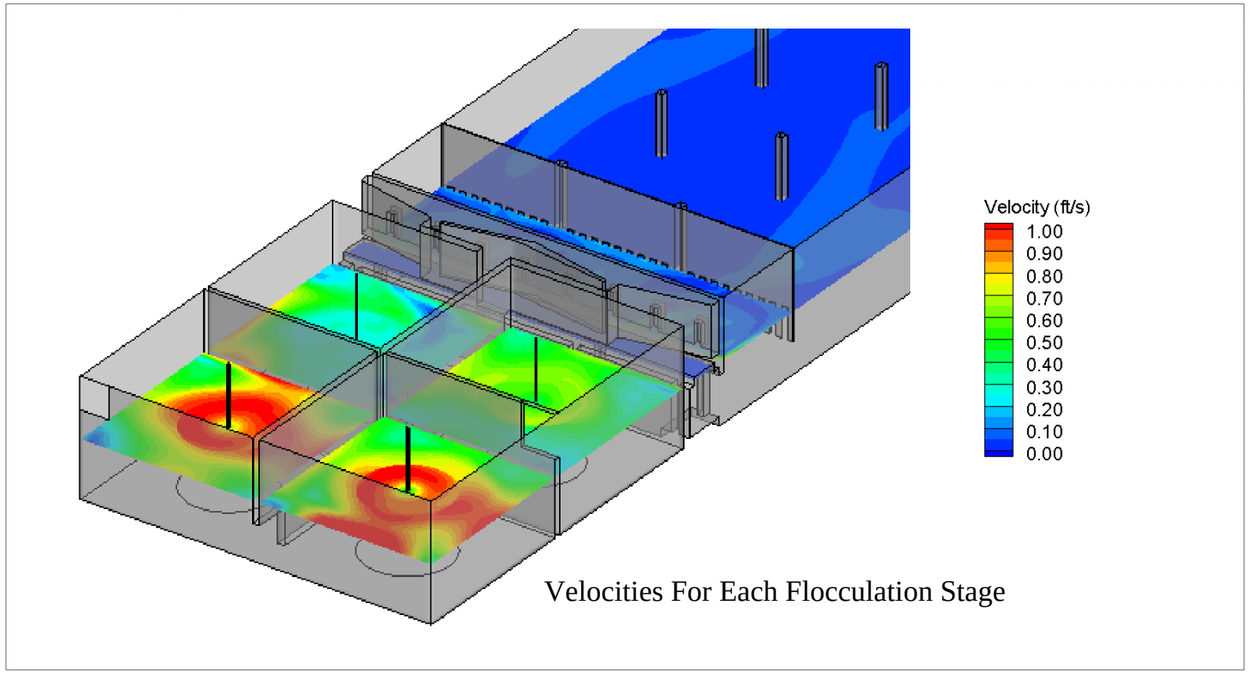
- Limiting flow to the conventional trains to 60 mgd, or 15 mgd per train.
- Adding new Actiflo™ system with two trains, each capable of processing 20 mgd.
  - Ancillary equipment needed, such as new sand silo, sand pumps, and hydrocyclones, etc
  - New sludge pumps needed to transfer sludge to existing gravity thickener.
- Supporting yard piping, including influent, effluent, and sludge piping to new Actiflo™ system.
- Supporting electrical components.
- Adding 48 mgd of new filters.
- Expanding the existing chemical systems

Other than items related to condition or hydraulics noted in previous chapters, no modifications to the existing conventional system would be made since it would retain its current configuration and equipment. To satisfy conditional assessment needs for the existing conventional basins, rehabilitating mechanical equipment for year 15 of the CIP and rehabilitating the interior of basins for year 10 of the CIP are included in this alternative's life-cycle cost. Other alternatives address this need immediately and are therefore included in their respective capital costs.

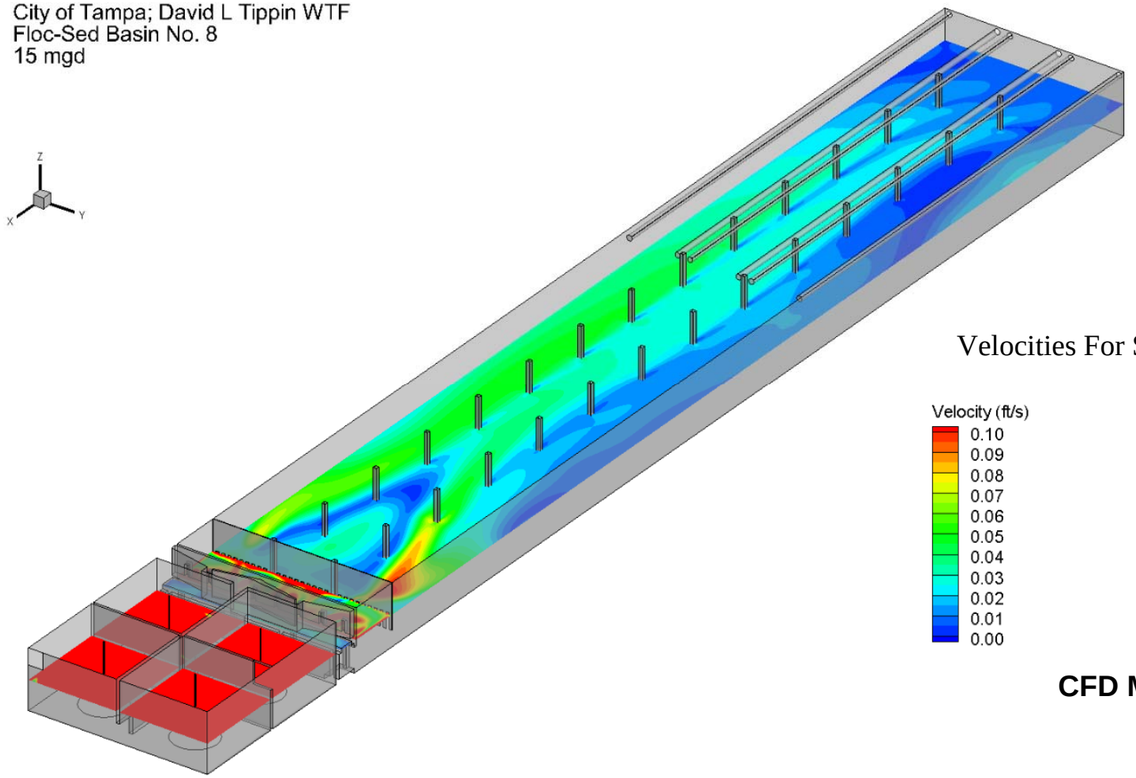
Figure 5.3 shows a proposed site layout and the location of the new Actiflo™.



Mixing Energy (G) For Each Flocculation Stage



Velocities For Each Flocculation Stage



Velocities For Sedimentation Basin

CFD MODEL RESULTS AT 15 MGD PER TRAIN

FIGURE 5.2

CITY OF TAMPA

DAVID L. TIPPIN WTF MASTER PLAN

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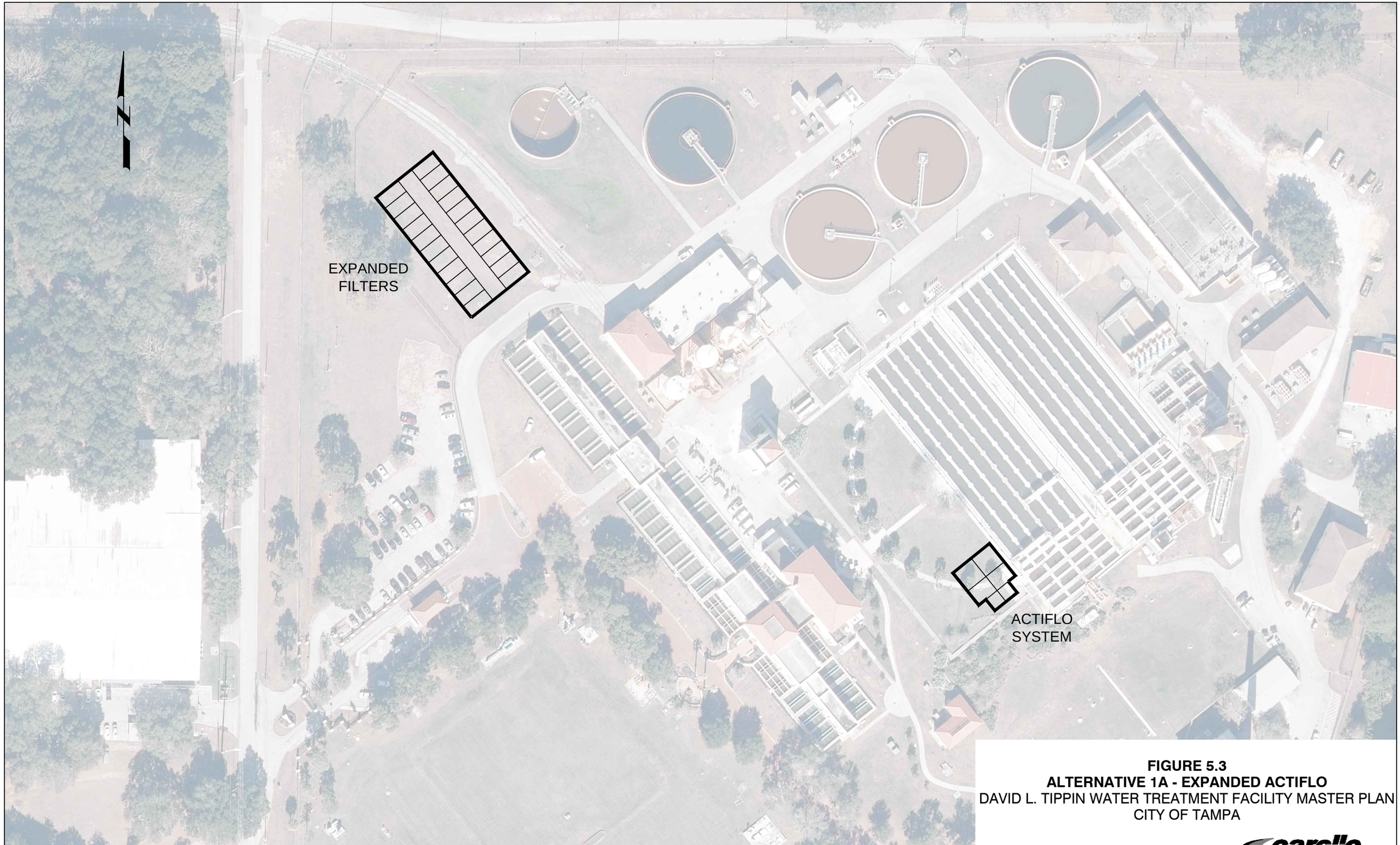


FIGURE 5.3
ALTERNATIVE 1A - EXPANDED ACTIFLO
DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN
CITY OF TAMPA

carollo

C:\Users\qrobertson\Desktop_JOBS\Tippin\FIGURE_5.3_ALT-1A 3-16-17 04:02pm qrobertson XREFS: AERIAL; 11x17_FIG-BDR HIL2016_469704; HIL2016_469705;

Chapter 3 describes evaluations of the conventional system according to the Ten States Standards at total conventional treatment flows of 80 mgd and above. For this alternative, Table 5.2 shows the conventional systems design criteria for 60 mgd (15 mgd per train) versus the Standards.

Since the design and construction of Trains 5 and 6 differ slightly from Trains 7 and 8, two values are shown. Values in red, blue, or green font reflect a significant variance, minor variance, or insignificant variance from the recommended value, respectively.

Table 5.2 Alternative 1A Conventional System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa

Parameter	Units	Value at Proposed Flow (15 mgd/Train)		Ten States Standard or Recommended Value
		Trains 5 and 6	Trains 7 and 8	
Rapid Mix				
Detention Time	sec	14.7	14.0	1 - 10
Mixing Energy	sec^{-1}	157	114	600 - 1000
Flocculation				
Detention Time	min	28.7	27.9	30
Mixing Energy (at 25° C)				
Stage 1	sec^{-1}	78.0	79.2	60
Stage 2		77.3	78.5	35
Stage 3		71.9	73.0	20
Stage 4		70.7	71.8	10
Velocity Through Ports in Flocculation Stage	fps	0.73 - 1.15		<1.5
Velocity Through Port to Sedimentation	fps	0.86		<0.8
Sedimentation				
Detention Time	hours	2.58	2.75	1.5 - 4.0
Surface Overflow Rate	gpm/sq ft	0.69	0.69	0.25 - 1.00
Reynold's Number	-	31,700	30,900	<20,000
Froude Number	-	3.6×10^{-6}	3.0×10^{-6}	$>10^{-5}$
Horizontal Velocity	ft/min	1.94	1.82	<1.00

As shown, at the lower flows of 15 mgd per train, more of the design parameters fall within the recommended range when compared to flows of 20 mgd per train. As stated in

Chapter 3, when employing enhanced coagulation (ferric sulfate dose of 120 mg/L on average) at a low pH (4.5), rapid mix and flocculation mixing energies become less vital in the formation of a settleable floc because of the sweep coagulation mechanism. For this reason, and due to the additional Actiflo™ system, this alternative does not include improvements to the rapid mix or flocculation basins or to operations.

Table 5.3 shows the proposed 40 mgd Actiflo™ system design criteria.

Table 5.3 Alternative 1A New Actiflo™ System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa

Parameter	Units	Value
Actiflo™ Capacity per Train	mgd	20
Number of Actiflo™ Trains	-	2
Total New Actiflo™ Capacity	mgd	40
Target Effluent Turbidity	NTU	< 2.0
Target TOC Removal	mg/L	> 70 percent removal
Target Effluent Color	PCU	< 10
Coagulation Tank Detention Time	min	3.0
Maturation Tank Detention Time	min	6.0
Settling Tank Hydraulic Loading Rate	gpm/sf	25
Sand Recirculation Flow (per pump)	gpm	435
Estimated Total Sludge Waste Flow	gpm	696
Estimated Sludge Concentration	percent	0.01 - 0.5

As shown in Table 5.3, the new Actiflo™ system design would be similar to the existing system. Since operating procedures and equipment are similar, operator training would be minimal.

This alternative also includes costs for expanding the existing filtration system to 140 mgd by adding 22 new filters at a total capacity of 48 mgd. As discussed in Chapter 3, the existing filters are limited to 92 mgd of capacity with an empty bed contact time (EBCT) of five minutes. The additional filter backwash will require a second surge tank, but this cost is not included because it is part of the solids handling upgrades discussed in Chapter 3 and is common among all alternatives.

This alternative allows for more flow to be processed through the Actiflo™ process, which has historically outperformed the conventional system in TOC removal. Therefore, the overall plant treated water quality could improve with this alternative. The primary disadvantage of this alternative is that is does not decrease chemical usage. EC would be maintained, meaning ferric sulfate, polymer, sulfuric acid, lime, and caustic soda doses

would not be reduced. Also, solids handling and disposal costs would continue to increase with increasing plant flows. The submerged infrastructure, such as concrete and metals, will require continuous maintenance due to degradation from the low pH water.

Finally, this alternative does not address all the modifications needed to fully optimize and rehabilitate Trains 5 through 8 flocculation systems. Issues with short-circuiting, sludge accumulation, marginal floc times, and aging equipment remain. The consequences of not retrofitting the floc systems are lessened by the fact that slightly more than half the plant flow will be processed with Actiflo™.

If lower coagulant doses were used (conventional versus EC treatment), there would be even more need to optimize floc basin performance with a major retrofit because of the lack of a sweep floc. The retrofit would promote inter-particle collisions of the floc and, in turn, build a settleable floc particle. This is true whether ferric sulfate or polymer alone was used for coagulation.

Because of these reasons, Alternative 1B incorporates provisions and costs to retrofit the floc systems in all four conventional basins.

5.3.2.2.4 *Future Pilot- and Bench-Scale Testing*

Pilot testing of basin modifications (pre-filter improvements) before detailed design is not essential because this alternative does not exceed what the DLTWTF has successfully demonstrated via full-scale treatment. To determine the number of new filters needed, if any, piloting (full scale and bench scale) of higher loading rates using optimized pilot filter columns (with hydraulic and process improvements recommended for the filters in Chapters 3 and 4) could be completed.

This filter column testing and full-scale testing, is recommended for all process alternatives in this chapter except for Alternative 3. This alternative does not include filter system expansion due to the new MF/UF complex associated with that option.

5.3.2.2.5 *Capital and Operating Costs*

The opinion of probable capital costs for this alternative are listed in Table 5.4. They include equipment and basins for the new Actiflo™ trains, expansion of the existing chemical systems, demolition basins 1 through 4 in the location of the new Actiflo™ trains, expansion of the existing filters, and scheduled improvements at the sludge processing facility.

Table 5.4 Alternative 1A Capital Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa	
Item	**Capital Cost**
Actiflo Equipment (2, 20 mgd Trains)	$3,110,000
VFDs (Mixers, Scrapers)	$36,000
Metals (4 percent)	$104,000
Concrete	$808,000
Concrete Coating	$2,525,000
Chemical Systems Expansion	$2,104,000
New Filters (22 new, 48 mgd)	$24,960,000
Sludge Processing Facility Improvements	$1,656,000
Demolition of Basins 1-4	$805,000
Site Work (10 percent)	$3,610,000
Piping, Valves, Appurtenances (20 percent)	$7,220,000
EI&C (25 percent)	$9,030,000
Total Direct Cost	**$55,968,000**
Contingency (25 percent)	$13,990,000
GC OH&P (12 percent)	$6,720,000
Sales Tax (7 percent of 50 percent of the total direct cost)	$1,960,000
Total	**$76,638,000**

Table 5.5 shows operating costs for this alternative. They include power, chemicals, recurring replacement costs, and infrastructure maintenance.

Table 5.5 Alternative 1A Operating and Maintenance Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa	
Item	**Average Annual Cost**
Power	**$114,000**
Coagulation Tank Mixers	$21,000
Maturation Tank Mixers	$21,000
Settling Tank Equipment	$3,000
Microsand Pumps	$42,000
Backwash Pumps	$18,000
Backwash Blowers	$9,000
Chemical Usage	**$5,820,000**
Ferric Sulfate	$2,548,000
Sulfuric Acid	$932,000
Polymer	$59,000
Lime	$1,064,000
Caustic	$1,217,000
Other	**$875,000**
Actiflo™ Sand Replacement	2,300
Sludge Disposal	$485,000
Sludge Processing Facility	$387,000
Total Average Annual Cost	**$6,855,000**
Basin Rehabilitation - Mechanical (2027)	**$556,000**
Basin Rehabilitation - Structural (2032)	**$3,531,000**

5.3.2.3 Alternative 1B - Retrofit Trains 5 through 8 for Increased Capacity and Efficiency

5.3.2.3.1 *Description and Process Flow*

Unlike Alternative 1A, this alternative would correct the conventional system's limitations and increase process capacity from 80 to 100 mgd, or 20 to 25 mgd per train. The existing 40 mgd Actiflo™ system would be retained to bring the total plant capacity to 140 mgd. This alternative also maintains enhanced coagulation in both treatment systems.

The benefits of this alternative include:

- No additional infrastructure or facilities.
- Minimized changes in plant operating protocols and staff training requirements.
- Low capital cost.
- Improved performance and capacity of conventional trains.

- Additional volume in existing sedimentation basins available for future use.
- Ability to add ion exchange or other pretreatment process in the future.

The drawbacks for this alternative (as with Alternative 1A) include:

- High operating costs (chemical).
- Need for chemical system expansion.
- High costs for solids production, solids handling requirements, and sludge hauling.
- Recurring rehabilitation of infrastructure such as concrete and metals due to low pH process water.

Figure 5.4 shows the proposed process flow diagram.

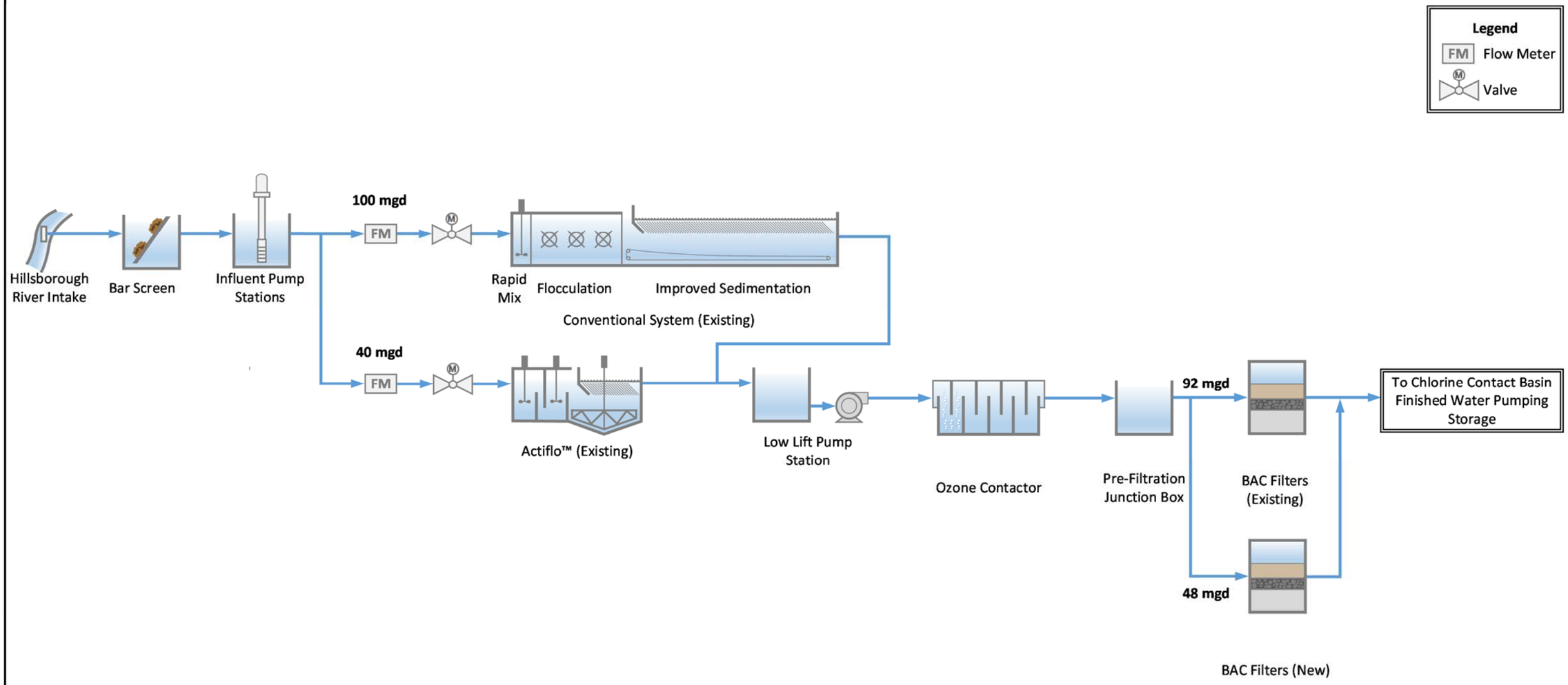
5.3.2.3.2 *Jar Testing and CFD Modeling Results*

Jar testing was conducted in February 2017 with the purpose of testing for TOC removal at varying coagulation conditions as well as for further MIEX testing. At the time, the DLTWTF was experiencing low TOC and color concentrations from the dry season. Nevertheless, the tests concluded that current operations require high coagulation doses with pH adjustment in order to remove more than 60 percent of the influent TOC, confirming the suitability of existing treatment protocol at the plant.

Additionally, the use of polymer as a flocculant aid results in better TOC removal. Unlike existing operations, the results showed that tapered flocculation would result in approximately 8 to 10 percent more TOC removal, as shown in Figure 5.5. Additional jar testing in the rainy season was recommended to more properly determine the impacts of each coagulation condition and the effects of MIEX as a pretreatment step and was completed as a part of the Pilot Study.

In addition to jar testing onsite, Carollo performed CFD modeling of the existing conventional rapid mix, flocculation, and sedimentation basins to further identify the flow paths and restrictions of the existing design. Retrofitting the existing rapid mix and flocculation basins is clearly valuable, as seen by examining the high degree of short-circuiting now occurring in the basins at the proposed flow of 25 mgd per train. See Figure 5.6 for CFD modeling results of the existing basin at 25 mgd per train.

Further evidence of this problem is the high degree of solids buildup annually. This build-up further compromises (shortens) the effective time of flocculation to less (perhaps far less) than half the recommended value of 30 minutes.



ALTERNATIVE 1B PROCESS FLOW DIAGRAM

FIGURE 5.4

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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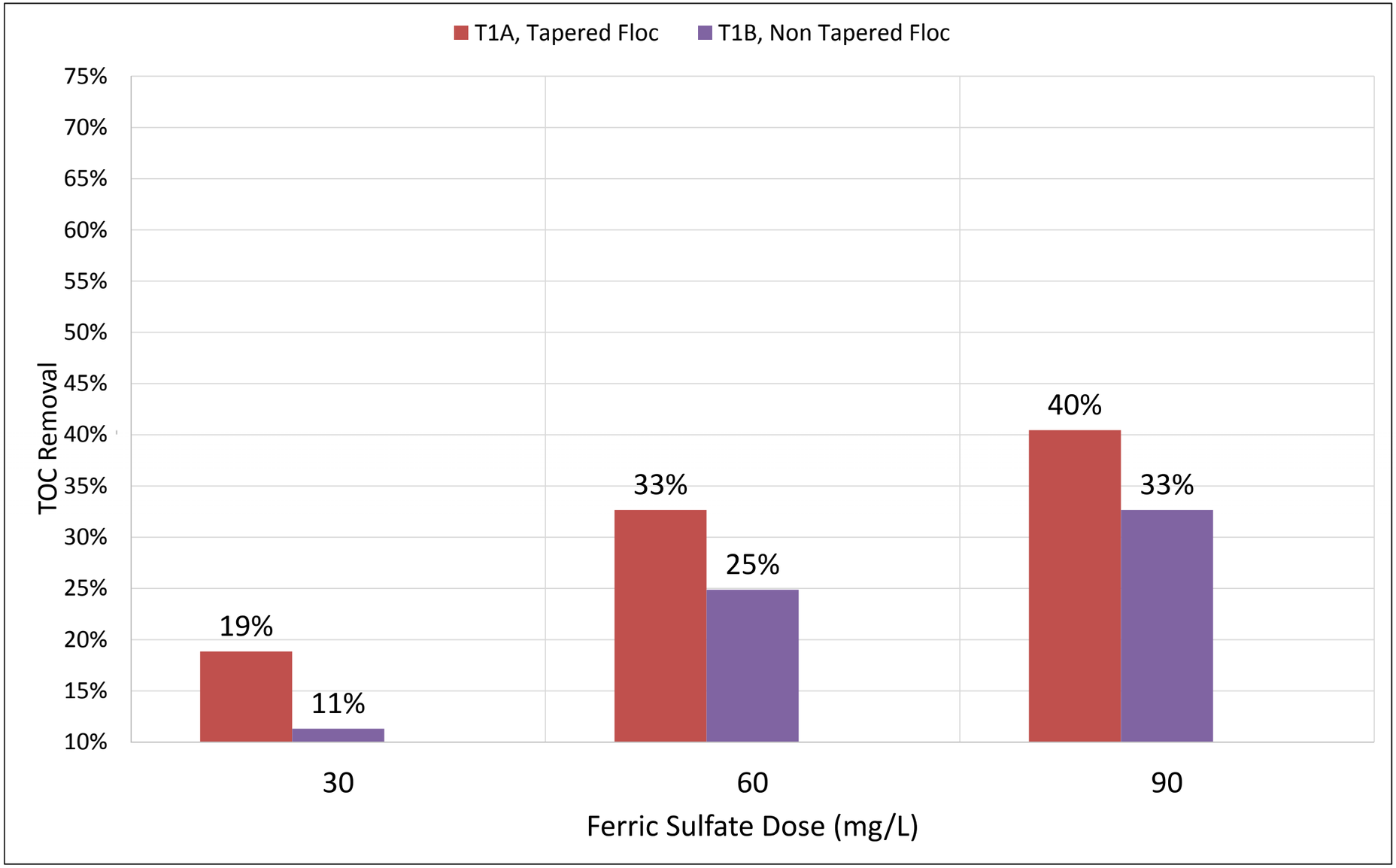

TAPERED VERSES NON-TAPERED FLOCCULATION JAR TESTS

FIGURE 5.5

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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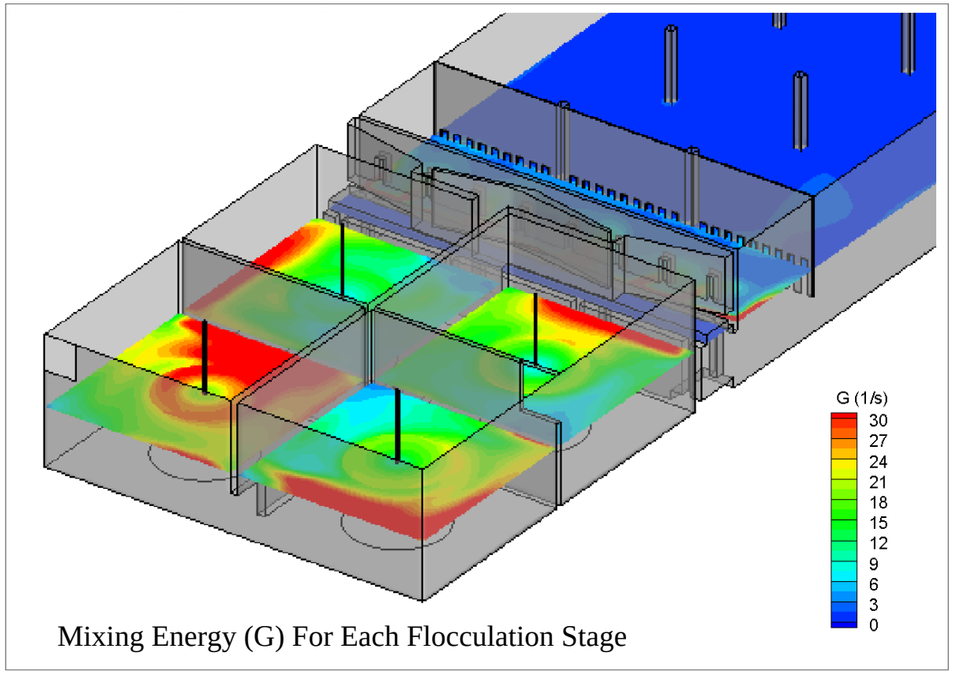
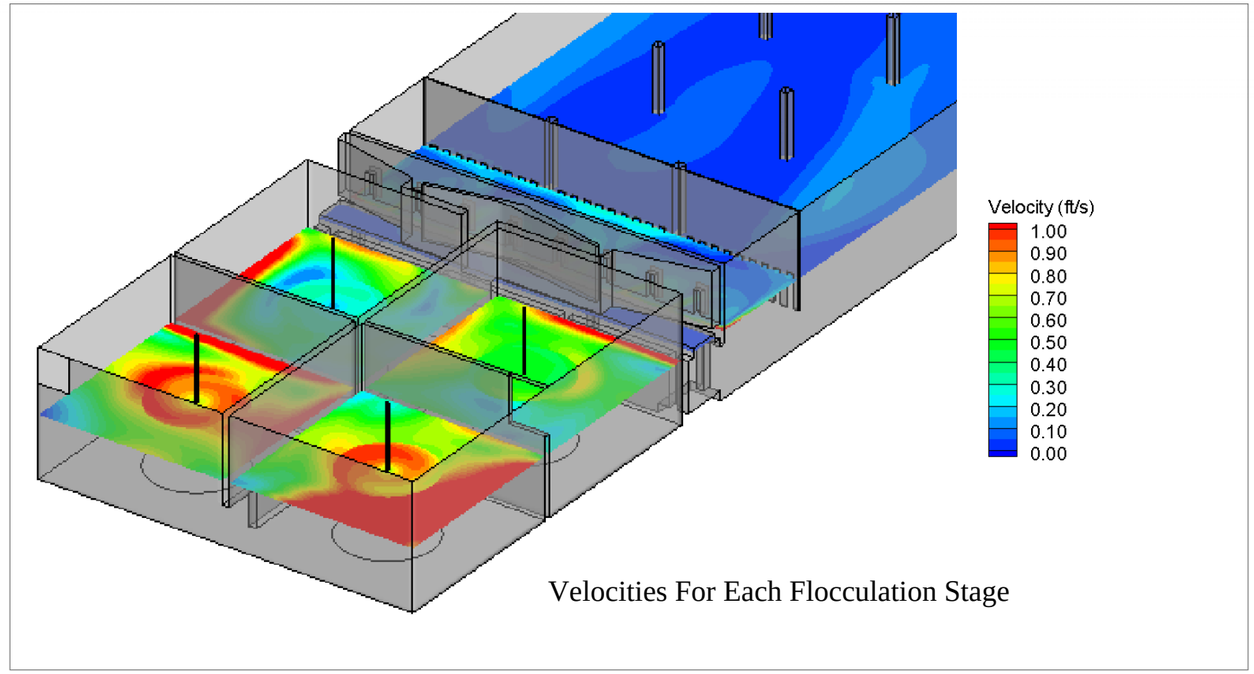
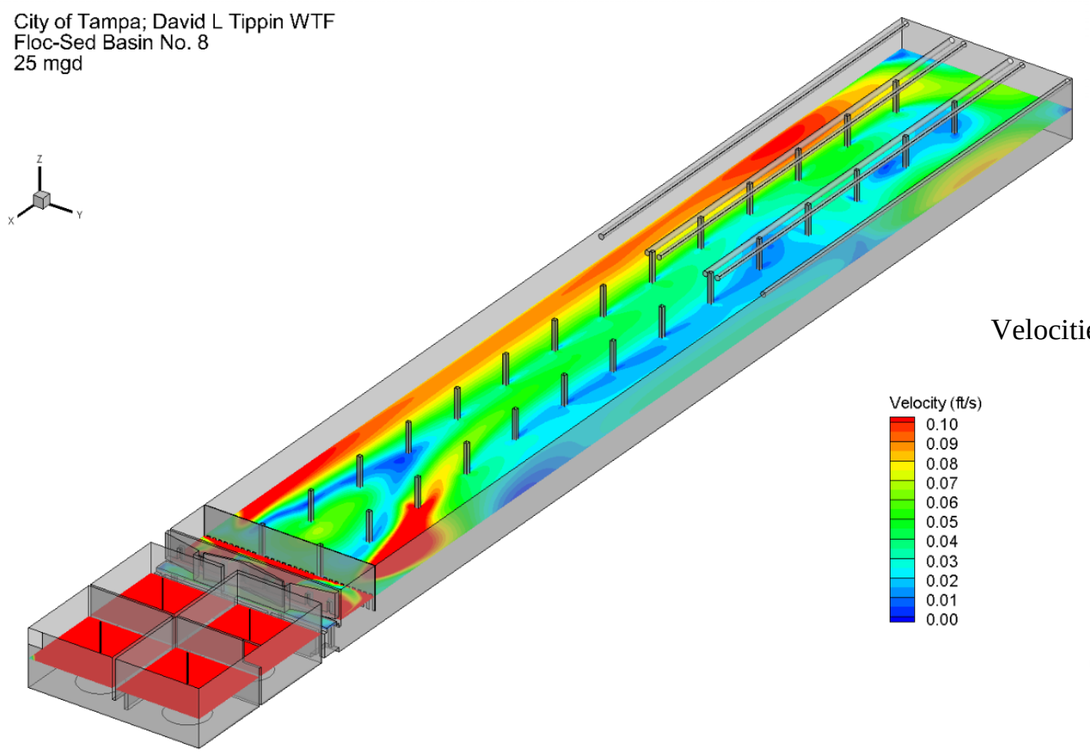


Mixing Energy (G) For Each Flocculation Stage



Velocities For Each Flocculation Stage



Velocities For Sedimentation Basin

CFD MODEL RESULTS AT 25 MGD PER TRAIN

FIGURE 5.6

CITY OF TAMPA

DAVID L. TIPPIN WTF MASTER PLAN

carollo

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Figure 5.7 shows the CFD model after implementing the proposed modifications. It shows the improved flow distribution, reduced short-circuiting, and laminar flow conditions in the sedimentation basin.

The proposed modifications to the conventional treatment basins will improve short-circuiting and detention time issues noted in the CFD modeling and process evaluations in Chapter 3. In turn, this will improve the efficiency of the treatment system as a whole.

As noted during jar testing, raw water TOC concentrations were less than 4 mg/L. Preliminary jar testing showed that under optimal conditions, the TOC in the raw water could still be reduced by 30 percent, with half of the typical ferric dose used at the plant and no acid added. During limited times of the year when TOC is very low in the raw water, operating the retrofitted conventional trains could be modified to use conventional coagulation rather than EC.

This switch could save on chemical costs and reduce sludge production. Although this savings is likely limited to a month or so each year, the financial benefit could add up to approximately $400,000 per year.

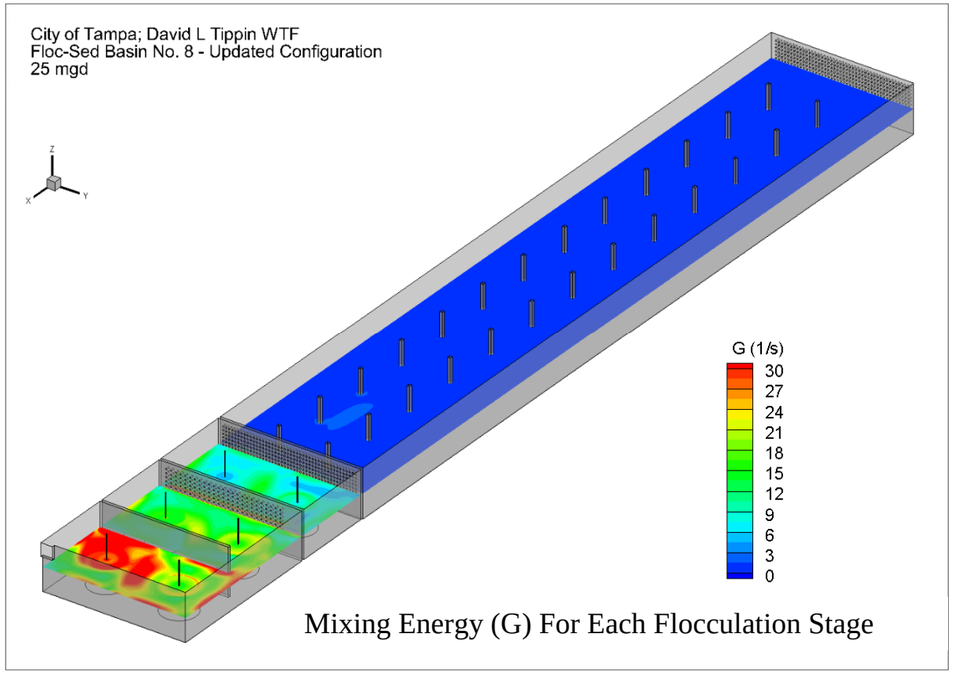
5.3.2.3.3 *Required Facility Modifications*

In the conventional system, the modifications necessary to increase process capacity include but are not limited to:

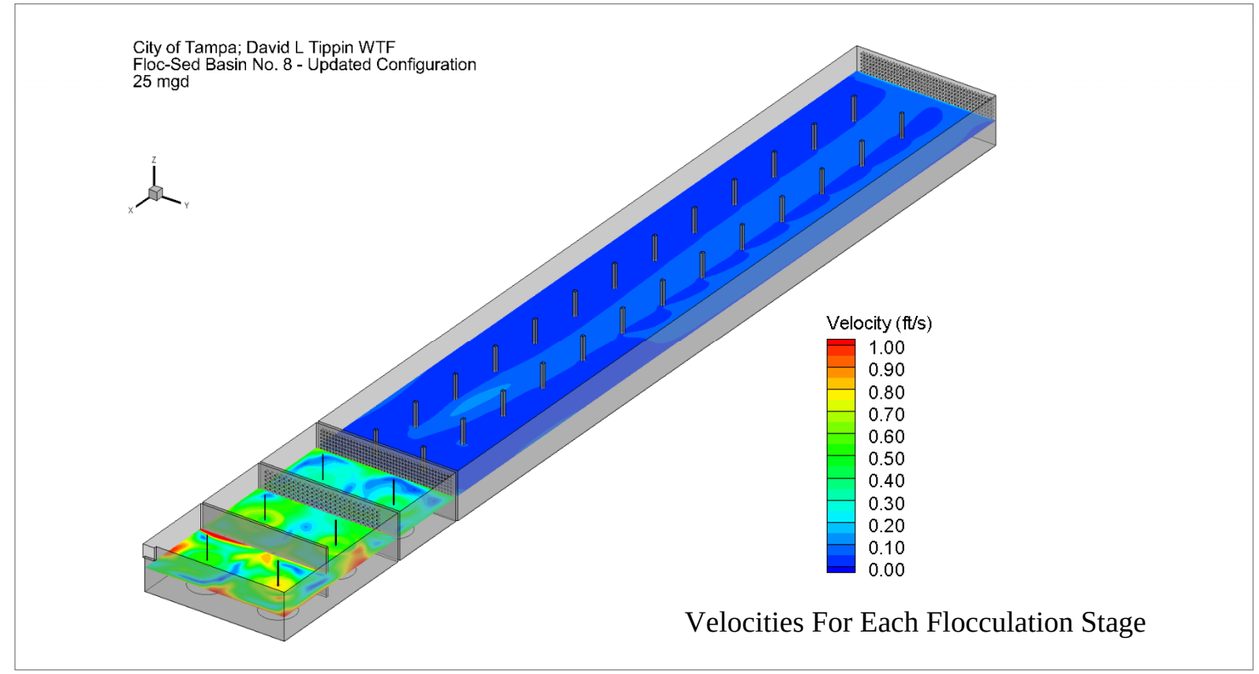
- Modifying the rapid mix basins.
- Reconfiguring flocculation basins and sedimentation basins.
- Replacing flocculators.
- Installing plate settlers in sedimentation basins, pending results of full-scale testing.
- Adding 48 mgd of new filters.
- Expanding existing chemical systems.

No major modifications would be made to the Actiflo™ system other than the condition assessment needs presented in Chapter 2, such as replacing mixers and drives.

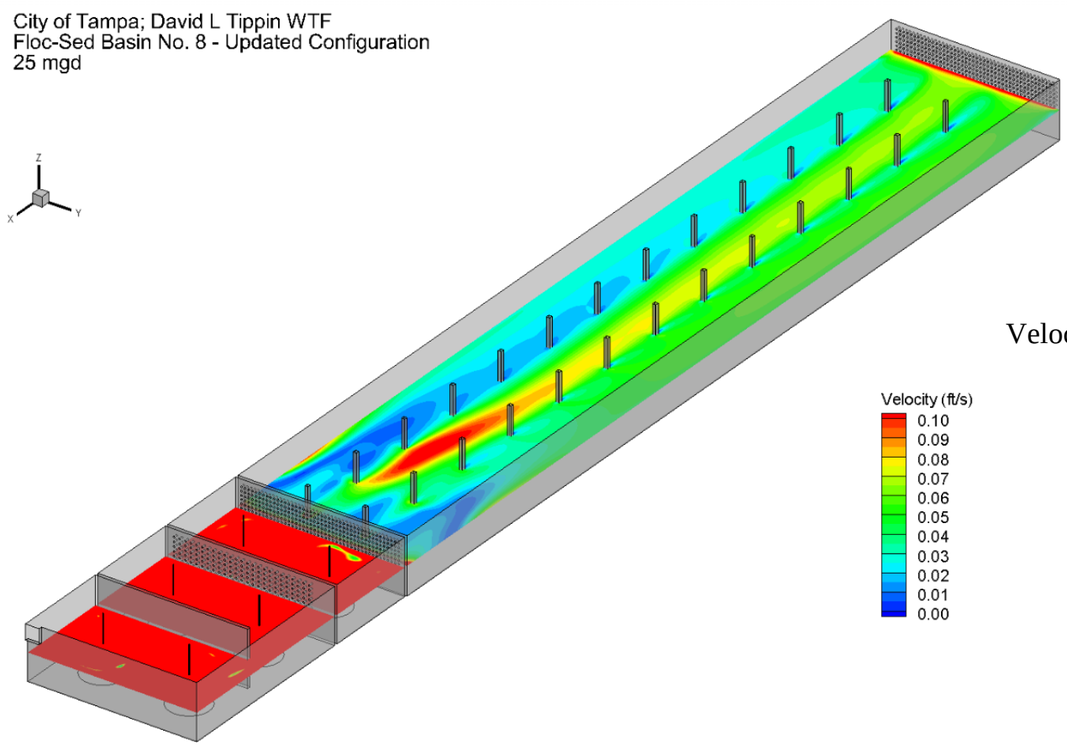
Although the existing rapid mix basin is sized to handle future flows of 100 mgd, this alternative includes modifying these basins and adding a mechanical mixer or a flash mix pump system to allow more ideal flash mix conditions. Additionally, this configuration minimizes short-circuiting and floc shear through the flocculation basins. Short circuiting and floc sheer are reduced by gradually lowering the mixing energy from stage to stage and minimizing the velocities and head losses from each stage and into the sedimentation basins. To achieve this, some existing sedimentation basin length must be used and reconfigured for flocculation. Adequate space is available in the sedimentation basin to reduce its length without significantly affecting detention times for settling.



Mixing Energy (G) For Each Flocculation Stage



Velocities For Each Flocculation Stage



Velocities For Sedimentation Basin

CFD MODEL RESULTS WITH PROPOSED MODIFICATIONS

FIGURE 5.7

CITY OF TAMPA

DAVID L. TIPPIN WTF MASTER PLAN

carollo

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Table 5.6 details the design criteria against the standards for this configuration at a 100 mgd flow rate (25 mgd per train).

Table 5.6 Alternative 1B Conventional System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa

Parameter	Units	Value for Reconfigured Trains 5 - 8 (25 mgd each)		Ten States Standard or Recommended Value
Rapid Mix				
Detention Time	sec	2.8		1 - 10
Proposed Mixer	hp	10		-
Mixing Energy	sec^{-1}	1164		600 - 1000
Flocculation				
Detention Time	min	30		30
Mixing Energy	sec^{-1}			
Stage 1		40		60
Stage 2		30		35
Stage 3		15		20
Stage 4		None Proposed		10
Velocity Through Ports in Flocculation Stage	fps	<1.5(1)		<1.5
Velocity Through Port to Sedimentation	fps	<0.8(1)		<0.8
Sedimentation (without Plate Settlers)				
Detention Time	hours	1.37	1.47	1.5 - 4.0
Surface Overflow Rate	gpm/sq ft	1.31	1.31	0.25 - 1.00
Reynold's Number	-	52,866	51,556	<20,000
Froude Number	-	9.9×10^{-6}	8.3×10^{-6}	$>10^{-5}$
Horizontal Velocity	ft/min	3.24	3.03	<1.00
Sedimentation - Proposed Plate Settlers				
Design Loading Rate	gpm/sq ft	0.30		≤0.50
Efficiency Factor	percent	90		-
Angle of Plate Inclination	degrees	55		-
Width of Plate	ft	4.5		-

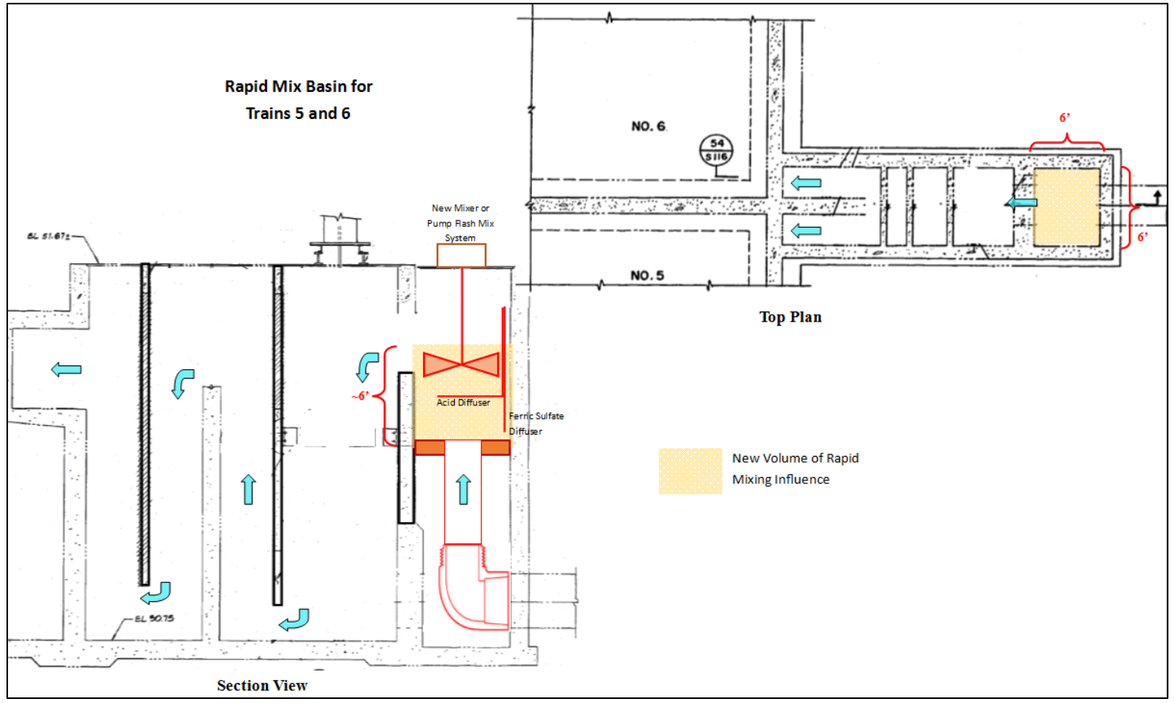
Table 5.6 Alternative 1B Conventional System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa			
Parameter	**Units**	**Value for Reconfigured Trains 5 - 8 (25 mgd each)**	**Ten States Standard or Recommended Value**
Length of Plate	ft	10	-
Flow per Plate	gpm	6.89	-
Total Plates per Train	each	2520	-
Effective Settling Area	sq ft	57,870	-
Notes: (1) Dependent on the number and size of ports to be determined by the design engineer			

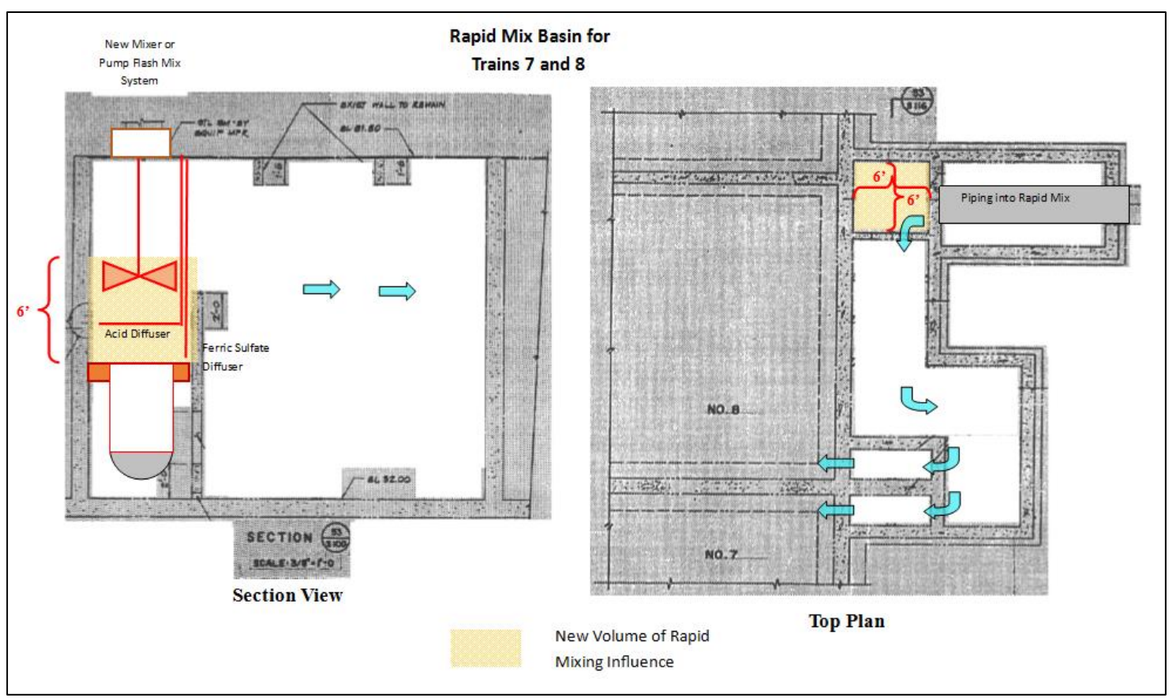
These improvements would allow the conventional system to process up to 100 mgd while maximizing use of the existing infrastructure onsite. As shown, without plate settlers, many of the Ten States Standards would be outside of their recommended range. With the additional of plate settlers, these standards are not applicable.

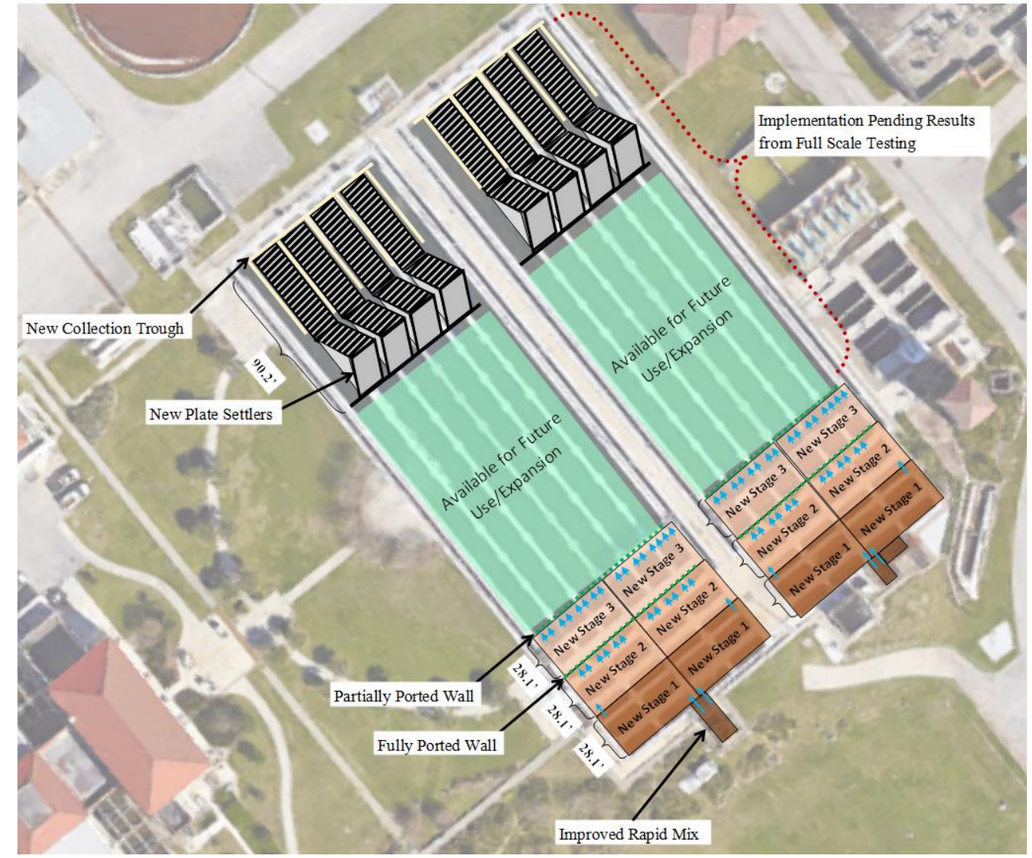
However, conducting full-scale testing with and without plate settlers is recommended in one renovated train before implementing it in all trains. Stainless steel plates should be used and various concrete basin coatings tested because of the low pH process water. Additionally, approval from regulatory agencies may be required to use this technology since there are no industry guidelines for this equipment.

Assuming the plate settlers are implemented and because the improved system requires less area, approximately 160 feet in length of the existing sedimentation basin infrastructure would be available for future use and possible future expansion.

If plate settlers are not feasible, installing a ported wall at the end of the sedimentation basins is recommended to replace the existing effluent launders. This modification is included for all other alternatives. Figure 5.8 shows the proposed reconfiguration of the rapid mix, flocculation system, and sedimentation basins.







Flocculation and Sedimentation Modifications

ALTERNATIVE 1B PROPOSED MODIFICATIONS TO EXISTING FACILITIES

FIGURE 5.8

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

carollo

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5.3.2.3.4 *Future Pilot and/or Bench-Scale Testing*

As discussed above, additional jar testing at the plant is recommended during the rainy season to confirm the tapered flocculation design in this alternative as well as optimized chemical doses for the retrofitted basins. In addition, full-scale testing of plate settlers, stainless steel material used in the plates, and various concrete coatings would be prudent before implementing these recommendations on all trains.

As with Alternative 1A, piloting of higher loading rates using optimized pilot filter columns and/or a full scale filter (after implementation of recommended hydraulic improvements noted in Chapters 3 and 4) is recommended to determine the number of new filters that could be scaled back.

This filter column/full-scale testing, is recommended for all process alternatives in this chapter, except for Alternative 3, which does not include filter system expansion due to its new MF/UF complex.

5.3.2.3.5 *Capital and Operating Costs*

The opinion of probable capital costs for this alternative are listed in Table 5.7. Capital costs include new plate settlers for the sedimentation basins, new flocculators for the existing and expanded flocculation tanks, concrete removal, and addition for modifying the flocculation zones, a concrete coating system for the conventional and Actiflo™ trains, expansion of the existing chemical systems, and scheduled improvements at the sludge processing facility.

Table 5.7 Alternative 1B Capital Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Cost
Plate Settlers	$5,400,000
Flocculators	$720,000
Concrete Demolition	$725,000
Concrete	$1,545,000
Concrete Coating	$1,907,000
Chemical Systems Expansion	$2,104,000
New Filters (22 new, 48 mgd)	$24,960,000
Sludge Processing Facility Upgrades	$1,656,000
Site Work (5 percent)	$1,950,000
Piping, Valves, Appurtenances (15 percent)	$5,850,000
EI&C (20 percent)	$7,800,000
Total Direct Cost	**$54,617,000**
Contingency (25 percent)	$13,660,000
GC OH&P (12 percent)	$6,555,000
Sales Tax (7 percent)	$1,910,000
Total	**$76,742,000**

Operating costs were developed for this alternative and are presented in Table 5.8. Operating costs for this alternative were updated based on more recent chemical usage from the Pilot Study conducted and therefore total chemical costs differ from Alternative 1A, even though actual chemical usage could be the same. Alternative 1A chemical costs were not updated to reflect the new data since this alternative is not recommended for implementation in the master plan. Operating costs include power, chemicals, solids handling, and infrastructure maintenance.

Table 5.8 Alternative 1B Operating and Maintenance Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa

Item	Cost
Power	**$33,000**
Additional Mixers (8)	$6,000
Backwash Pumps	$18,000
Backwash Blowers	$9,000
Chemical Usage (Total)(1)	**$7,556,000**
Other	**$873,000**
Sludge Disposal	$485,000
Sludge Processing Facility	$387,000
Total	**$8,156,000**
Basin Rehabilitation - Structural (2032)	**$2,913,000**

Notes:
(1) Includes ferric sulfate, lime, sulfuric acid, caustic, and polymer.

5.3.3 Alternative 2 - Ion Exchange

5.3.3.1 Overview

The primary goal of an ion exchange (IX) treatment system is to reduce the DOC, TOC, and color in the water supply in a more cost-effective and operator-friendly manner than the current practice of enhanced coagulation. A key secondary goal is to drastically reduce production, handling, processing, and disposing of the ferric-based solids generated by enhanced coagulation.

In theory, both fluidized bed and fixed bed systems can be installed as a pretreatment system prior to rapid mix/flocculation/sedimentation, as a mid-step treatment system prior to ozonation, and as a final post filtration polishing step (fixed bed only). Albeit, several of these specific application points in the process train have limited or no demonstrated experience in the waterworks industry.

Factors to consider when locating an IX system in the process train are as follows:

- Minimizing the need for re-pumping or adding additional pump stations that would be required, for instance, if the fixed bed system were to be used post-filtration.
- Avoiding biological fouling of the IX resin, a concern when placed immediately downstream of ozonation.
- For fluidized bed, avoiding carryover of lost resin into the clearwell/distribution system (post treatment application) or the ozone contact basin (mid-step application).
- For fixed bed, avoiding the IX bed serving as a filter for particulate removal as opposed to organics removal. To minimize this occurrence, pre-filtration or screening would be required.

After considering the merits of each alternative, it is recommended, for the purposes of this master plan, that IX should be employed solely as a pretreatment system. This location is consistent with manufacturers' recommendations (Tonka and IXOM) and may avoid the need for an additional pump station and other supporting infrastructure. The pretreatment location also serves to best reduce chemical use, reduce solids production, and meet finished water TOC goals in the most cost-effective and logical way.

For direct comparison of IX alternatives, each alternative was based on a treatment capacity of 140 mgd, treating 100 percent of the plant influent flows. IX could be employed on the conventional train only and blended with water (treated with enhanced coagulation) from the Actiflo™ train. However, such a treatment option would use two different treatment methodologies while still requiring significant storage and chemical use for pH reduction, enhanced coagulation (for TOC adsorption), and pH restoration. Maintaining enhanced coagulation on any portion of the plant still results in sludge production and handling needs that could be largely avoided with an IX pretreatment scheme.

Table 5.9 shows the advantages and disadvantages for fluidized versus fixed bed ion exchange alternatives.

Table 5.9 Comparison of Ion Exchange Alternatives
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Parameter	Fluidized Bed Ion Exchange	Fixed Bed Ion Exchange(1)	Comments
Reported Removal Efficiencies	Moderate	Moderate to Good	
Proprietary System	Yes (MIEX)	No	Fixed bed requires customized design
Proprietary Resin	Yes	No	

Table 5.9 Comparison of Ion Exchange Alternatives David L. Tippin Water Treatment Facility Master Plan City of Tampa			
Parameter	**Fluidized Bed Ion Exchange**	**Fixed Bed Ion Exchange(1)**	**Comments**
Requires Pre-Filtration or Pre-Screening	No	Yes	Filtration or screening for particulate removal
Operating installations at facilities 20 mgd or greater	Yes	No	
Ease of Operation	More Complex	Less Complex	
Parameter	**Fluidized Bed Ion Exchange**	**Fixed Bed Ion Exchange(1)**	**Comments**
Operating Costs	Higher	Lower	Higher costs due largely to resin loss
Capital Costs	Lower	Higher	
Requires Piloting	Yes	Yes	
Notes: (1) Assumes gravity IX cells using common wall construction similar to a filter complex.			

5.3.3.1.1 *Brine Disposal*

Sodium chloride is the typical regenerant chemical for both IX systems considered. The salt usage for both systems is also similar, in the range of 350 pounds of salt per million gallons (MG) of water treated. Several waste brine disposal options are available, including sewer disposal, onsite treatment, and deep well injection.

Based on the size of this system, waste brine flow is estimated to be about 100,000 gallons per day, depending on the frequency of regeneration. Although discussions with the City indicated that deep well injection was a favorable method, deep well injection is not recommended due to its capital and O&M costs as well as permitting requirements. Instead a waste brine treatment and salt recovery system is recommended for both the fixed and fluidized bed IX alternatives. This system could consist of an industrial nano-filtration membrane that allows chloride ions to pass through the membrane, providing a clean permeate stream to be reused in the MIEX system. IXOM also provides a vibratory shear-enhanced process (VSEP) unit for brine treatment. These systems can save on overall salt consumption and reduce the volume of waste disposed. The waste stream can be trucked to an off-site disposal facility that can reuse the organics concentrated brine as a valuable fertilizer product. This salt and waste brine recovery system and storage facility is included in the capital cost for both Alternative 2A and 2B. This eliminates the capital and operating costs for a deep injection well.

5.3.3.2 Alternative 2A – Fluidized Bed Ion Exchange

5.3.3.2.1 *Description and Process Flow*

Used in several locations in Florida, Magnetic Ion Exchange (MIEX) is a fluidized bed ion exchange process that removes TOC using a patented process developed by IXOM Watercare, Inc. (formerly Orica Watercare, Inc.). As the only manufacturer of fluidized bed systems with full-scale installations and a track record in the waterworks industry, MIEX was selected for consideration of this alternative. Its installations include Palm Beach County Water Utilities Department (Plant 2; 17 mgd), Boynton Beach Utilities (20 mgd), and the C.B. Collier Water Treatment Plant in Gadsden, Alabama (24 mgd).

MIEX uses a strong base anion exchange resin that was developed to remove DOC and has magnetic properties to facilitate settling. The MIEX resin has a mean particle size of 180 to 200 micrometers (µm). The small particle size increases the available surface area compared to traditional ion exchange resins.

In the treatment process, chlorides are exchanged for DOC on the resin particle surface. The resin is later regenerated with a sodium chloride (salt) solution (regenerant) to reverse the exchange of DOC and chlorides on the resin surface, resulting in a refreshed resin, which is then reused. The resulting waste stream is a brine solution rich in TOC and requiring treatment via the waste brine and salt recovery system. Because the MIEX process is minimally affected by suspended solids, low concentrations of sand, or other colloidal particles, it is ideally suited as a pretreatment process for surface waters prior to conventional treatment. The MIEX technology removes additional TOC and can significantly reduce chemical, sludge processing, and operating costs. Consequently, expanding certain chemical systems may not be required even at the future flow of 140 mgd.

It should be noted that MIEX is the proprietary resin currently offered and is manufactured in Australia and shipped to the U.S. water plants employing this technology. IXOM has suggested in meetings that they would commit to constructing a U.S.-based manufacturing facility should the City of Tampa move forward with design of a MIEX treatment system. They acknowledged a two-year time frame for completion of the DLTWTF. Other agreements up front would need to include guarantees on resin cost (perhaps over a 20-year life), resin loss, and overall system performance.

Incidentally, the MIEX resin previous to the one now marketed, noted as their DOC resin, is no longer protected by patent. Any piloting should include the DOC resin as well, assuming that a minimal decrease in performance with that resin could mean significant savings in operating costs if another manufacturer entered the market.

The benefits of this alternative include:

- Reduced TOC load to existing treatment systems.
- Reduced chemical costs. (Reduced ferric dose plus eliminated need for pH adjustment with sulfuric acid and lime.)

- Reduced solids production, handling, and hauling costs.
- Expansion of existing chemical systems may not be required.
- Minimal impacts to MIEX system from suspended solids in raw water.
- Less corrosive water in contact with the basins and submerged equipment, avoiding the cost for future basin coating and basin refurbishment.

Conversely, the drawbacks for this alternative include:

- High capital cost.
- Currently, largest installation is 24 mgd.
- Proprietary system.
- Proprietary resin manufactured in Australia (though patent expired).
- Resin must be replaced continuously.
- Annual cost of salt and resin replacement can be substantial.
- New technology to the City.

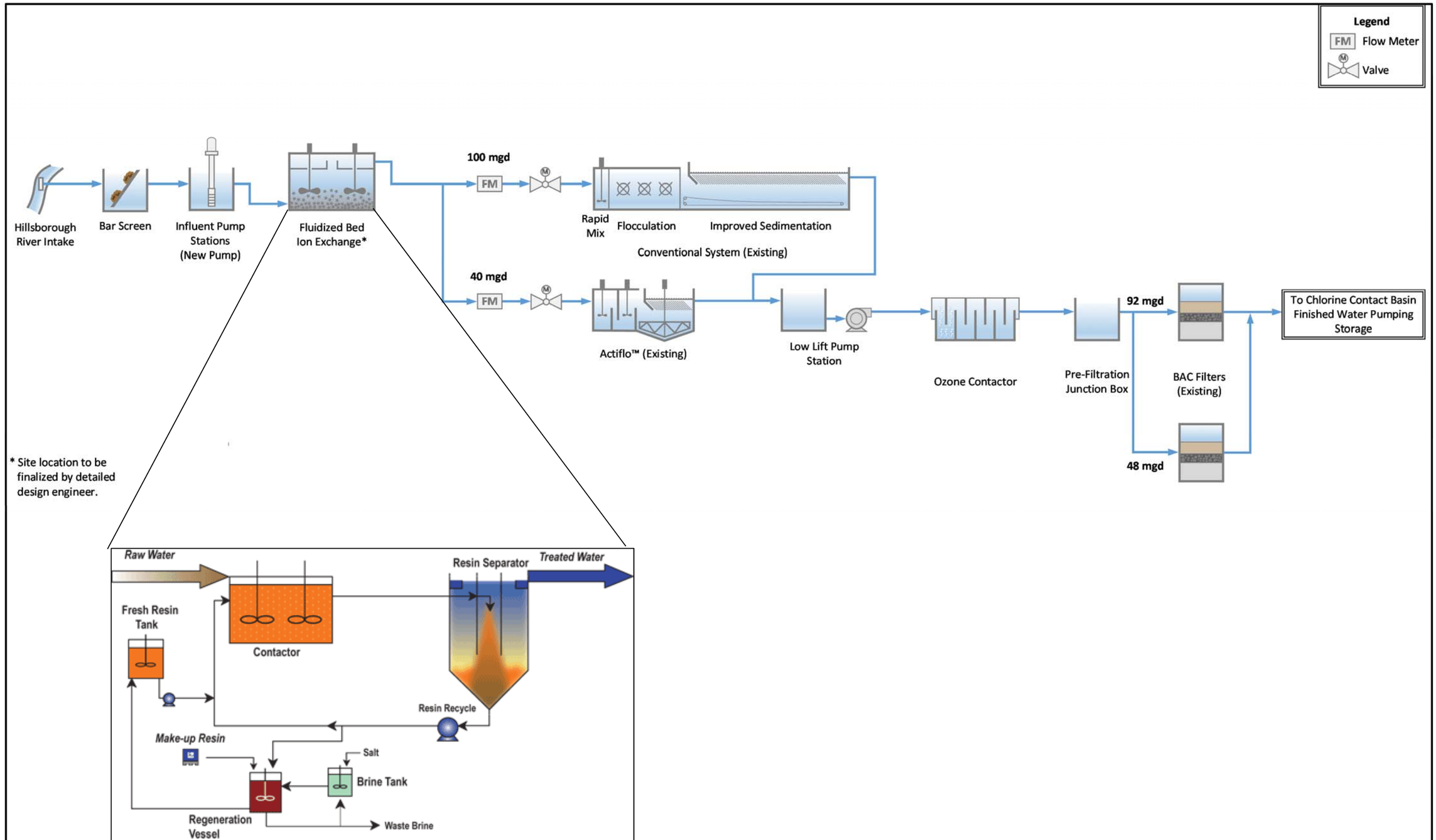
To mitigate salt costs, groundwater or saltwater could potentially be used for resin regeneration. Since this has been seldom practiced, additional research and piloting is recommended if this option is considered. In addition, waste brine recovery alternatives should be piloted. The waste brine recovery option was included in the capital cost for this alternative.

See Figure 5.9 for a process flow diagram showing the proposed orientation of this treatment scheme as it relates to the existing WTP facilities.

5.3.3.2.2 *Required Facility Modifications*

A few facility modifications are required for this alternative, including multiple features of the MIEX system shown in Figure 5.9:

- New 140 mgd MIEX system
  - Fluidized bed contactor in which water flows continuously through a vessel and is in contact with the ion exchange resin where DOC and other negatively charged ions are exchanged for chloride ions)
  - Regeneration system in which resin is accumulated and sent for regeneration, which is the process of exposing the resin to saturated sodium chloride solution to reverse the ion exchange process and concentrate the waste brine/organics for discharge.
  - Salt saturator for producing a saturated sodium chloride solution used for regeneration.
  - Recycled brine storage and feed.
  - Virgin resin storage and feed for storage of the bulk virgin resin and incorporation of make-up resin into the process.
  - Waste brine treatment and salt recovery system, which consists of the VSEP system and storage tanks for concentrate.



MIEX PROCESS FLOW DIAGRAM

FIGURE 5.9

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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- Expanded conventional system as required by Alternative 1B.
- One less belt filter press at off-site SPF due to reduced solids handling.
- 48 mgd of new filters.

Figure 5.10 shows a potential site layout that includes the MIEX contactors and new filters. The regeneration and VSEP systems would require the additional available area next to the MIEX contactors.

The modifications proposed in Alternative 1B are recommended for this alternative because they can expand capacity to 140 mgd (100 mgd of conventional and 40 Actiflo™). Additionally, since MIEX allows for a lower coagulant dose in the existing system (due to the significant TOC removed within MIEX), the inefficiencies of the conventional system must be corrected to maximize TOC removal. With lower ferric doses and process water between 5 - 7 pH, proper rapid mixing and flocculation become integral to floc formation and subsequent TOC removal. For these reasons, Alternative 1A was not considered for inclusion in this alternative.

Another possible location of the MIEX system is within the existing conventional basins. However, this option should be evaluated by the design engineer and should include a feasibility and hydraulic analysis. Since a portion of the basins would be used, the improvements in Alternative 1B would still be required to continue the use of coagulation after the MIEX system. The high-rate MIEX treatment system was considered for the DLTWTF. Table 5.10 shows typical design parameters for the high-rate system.

Table 5.10. Alternative 2A MIEX System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa

Parameter	Units	Value
MIEX Resin Mean Particle Size	µm	180-200
Hydraulic Loading Rate		
All in Service	gpm/sq ft	7.7
One Out of Service	gpm/sq ft	8.3
Hydraulic Detention Time	min	21
Capacity per train	mgd	10
Number of new trains	No.	14
Total capacity	mgd	140
Resin regeneration rate	BV	600
Regeneration vessels	No.	16
Recycle brine tanks	No.	3
Salt saturator tanks	No.	6
Max waste brine generated at 600 BV	gpd	105,000

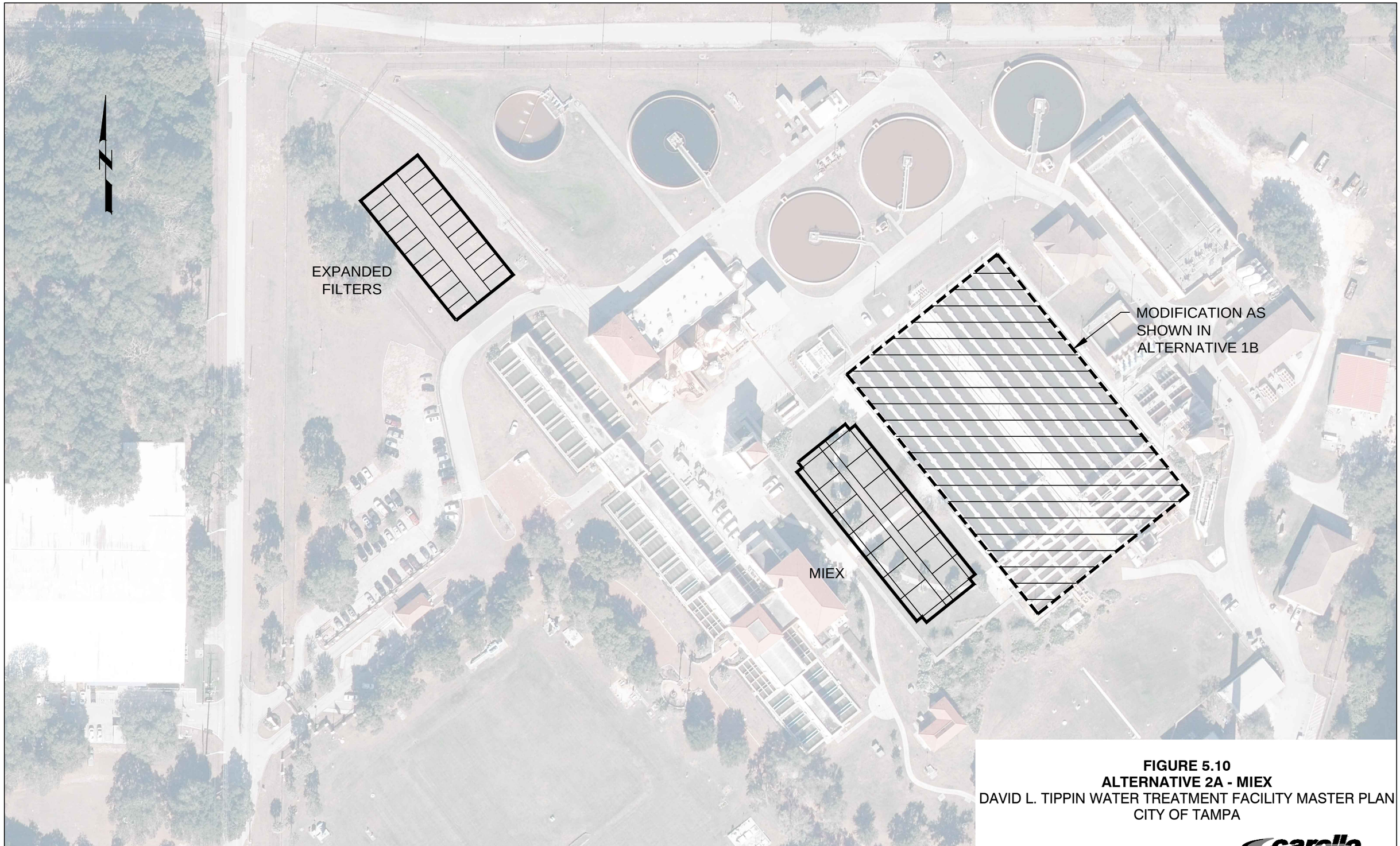


FIGURE 5.10
ALTERNATIVE 2A - MIEX
DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN
CITY OF TAMPA

carollo

C:\Users\qrobertson\Desktop_JOBS\Tippin\FIGURE_5.10_ALT-2A 3-27-17 10:30am qrobertson XREFS: AERIAL; 11x17_FIG-BDR HIL2016_469704; HIL2016_469705;

5.3.3.2.3 *Bench Scale and Pilot Testing Results*

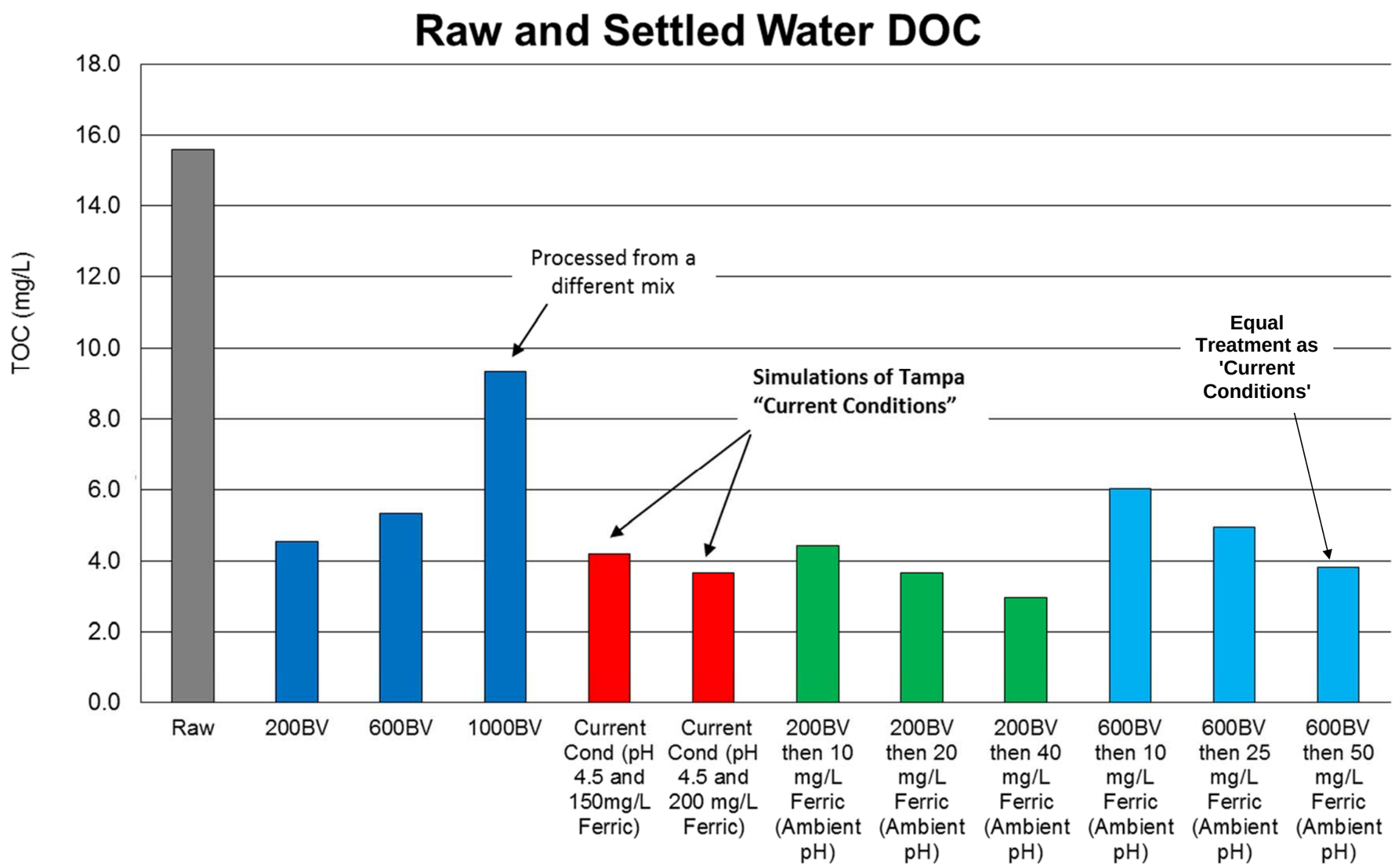
Testing by Orica of the MIEX process was conducted at the DLTWTF in 2009. That study, performed at raw water TOC concentrations of 27 mg/L, concluded that the use of a split stream MIEX process blended with raw water could significantly reduce the plant's operational costs. A coagulant dose reduction of 43 percent was estimated based on the bench-scale work during this study.

Figure 5.11 shows results of Carollo's initial testing, which treated 100 percent of the influent flow with IX. At raw water TOC values of approximately 15.5 mg/L, MIEX treated water at 600 bed volumes and using a subsequent rapid mix, flocculation, sedimentation step with approximately 50 mg/L ferric sulfate, would produce water slightly better than current operations (red vertical bars). A bed volume (BV) is defined as the volumetric ratio of treated water to resin. Of note, MIEX treatment alone at 600 BV (middle blue bar) produced water nearly as good (approximately 5 mg/L TOC) as the DLTWTF's current enhanced coagulation process.

Pilot testing of the entire full-scale treatment process (starting with MIEX through to the filters) was completed in April 2018. Pilot testing included working closely with the manufacturer, to verify removal efficiencies can be maintained long-term, when the resin has aged and differing raw water quality conditions were encountered.

The piloting included testing a range of chemical dosing strategies within the improved conventional system (post MIEX), including use of metal coagulants and polymers individually and in combination (at periods of high and low TOC). Pilot testing may showed dramatically reduced chemical usage and costs, and sludge handling/disposal costs.

Pilot testing also included testing to optimize treatment downstream of the conventional treatment basins, including ozonation and optimized biological active filtration to determine the full range of product water TOC, turbidity, and other water quality parameters that could be expected during seasonal changes in raw water quality. Additionally, pilot testing included the waste brine recovery system and alternative regeneration salt sources (seawater). The Final Pilot Report completed by Carollo Engineers, Inc. in June 2018 details the full and complete results of the study. In summary, the MIEX system was able to provide better or similar finished water TOC as the existing full scale EC system but at a significantly lower chemical costs, with complete elimination for the need of lime and sulfuric acid usage. Included in the overall recommendations was additional piloting of just the MIEX unit to quantify data gaps during preliminary design of the full scale system.



MIEX BENCH SCALE TESTING RESULTS

FIGURE 5.11

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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5.3.3.2.4 *Capital and Operating Costs*

Table 5.11 shows the opinion of probable costs for this alternative. Capital costs include the MIEX equipment and tanks, expansion of the conventional treatment trains as described in Alternative 1B, expansion of the existing filter, reduced upgrades to the sludge processing facility, demolition of the existing Basins 1 through 4 at the proposed location of the new MIEX tanks, and piloting. The cost for MIEX equipment includes the initial resin inventory.

Table 5.11 Alternative 2A Capital Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Cost
MIEX Equip (140 mgd)	$40,944,000
MIEX Tanks (Concrete)	$6,220,000
Demolition of Existing Basins 1-4	$805,000
Expand Conventional Train (100 mgd)	$8,390,000
New Filters (48 mgd)	$24,960,000
Sludge Processing Facility Upgrades	$1,161,000
Additional Piloting	$800,000
Site Work (10 percent)	$8,330,000
Piping, Valves, Appurtenances (20 percent)	$16,660,000
EI&C (12 percent)	$9,990,000
Total Direct Cost	**$118,260,000**
Contingency (25 percent)	$29,560,000
GC OH&P (12 percent)	$14,200,000
Sales Tax (7 percent)	$4,140,000
Total	**$166,160,000**

Operating costs were developed and are shown in Table 5.12. This estimate includes costs for power, chemicals, and major replacement items such as continuous makeup resin for the MIEX system based on anticipated resin loss (estimated at 1.2 gallons of resin per million gallons of water treated); however, it does not include minor maintenance or labor costs. Like Alterative 1B, chemical costs were updated based on the results of the pilot study. Power estimates and costs for the MIEX system were also updated based on information provided by IXOM.

Table 5.12 Alternative 2A Operating and Maintenance Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa	
Item	**Cost**
Power	**$148,000**
MIEX	$115,000
Additional Mixers (8)	$6,000
Backwash Pumps	$18,000
Backwash Blowers	$9,000
Chemical (total)(1)	**$4,580,000**
Other	**$652,000**
Waste Brine Treatment and Disposal	$202,000
Sludge Disposal	$261,000
Sludge Processing Facility	$189,000
Total	**$5,142,000**
Notes: (1) Includes MIEX resin, salt, ferric sulfate, caustic, and polymer	

5.3.3.3 Alternative 2B – Fixed Bed Vessel Ion Exchange

5.3.3.3.1 *Description and Process Flow*

Unlike fluidized bed ion exchange, this type of ion exchange technology does not require patented equipment, nor is the resin proprietary to one manufacturer. This system uses a strong base anion exchange resin developed for removing dissolved organic carbon (DOC). Similar to the fluidized bed IX process, chlorides are exchanged for DOC on the resin particle surface. The resin is later regenerated with a sodium chloride solution (regenerant) to reverse the exchange of DOC and chlorides on the resin surface. The result is a refreshed resin, which is then reused in place.

Hydraulic loading rates for this type of system generally range from 8 to 9 gpm/sq ft. Loading rates as high as 10 gpm/sq ft can be achieved, but treatment performance is compromised. Unlike the fluidized bed ion exchange system, no normal resin attrition occurs in this system.

The resin is used continuously through multiple regeneration cycles for approximately 10 years. At the end of its useful life, the resin is replaced entirely, which might be a significant capital cost at that time.

The benefits of this alternative include:

- Reduced TOC load to existing treatment systems.

- Not a patented process or resin.
- Reduction in chemical costs (reduced ferric dose plus eliminate need for pH adjustment with sulfuric acid and lime).
- Reduction in solids production, handling, and hauling costs.
- Expansion of existing chemical systems may not be required.

Conversely, the drawbacks for this alternative include:

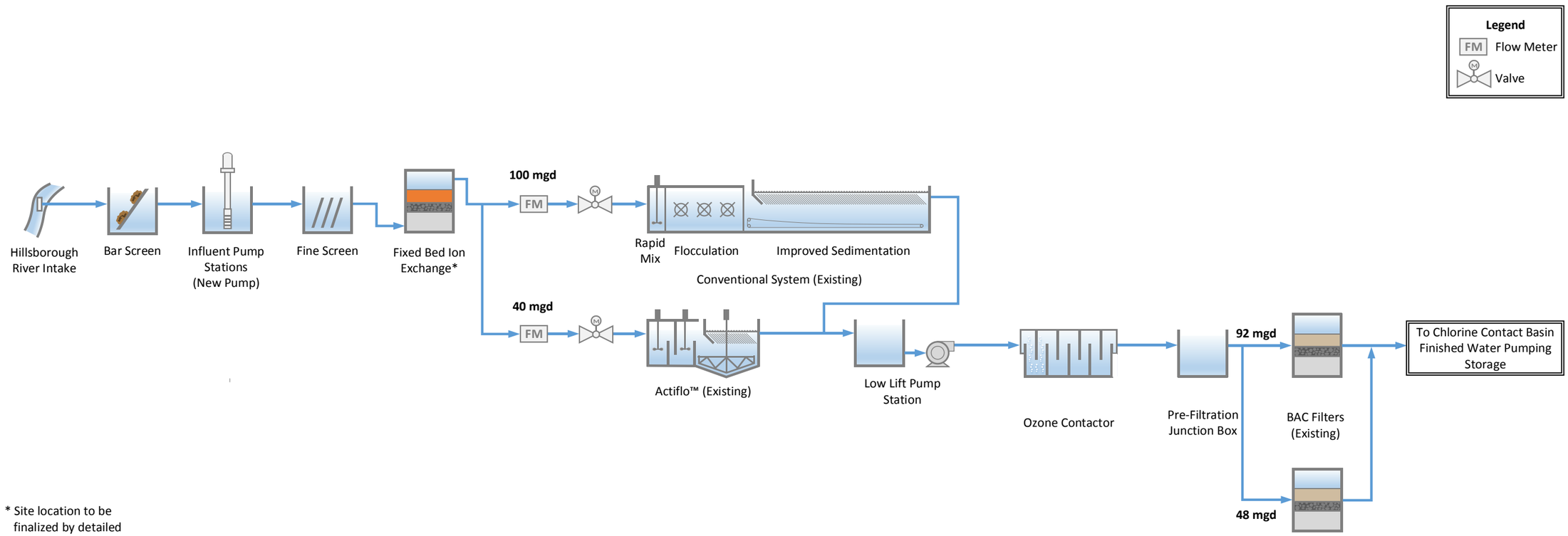
- Disposal of waste brine stream required via deep injection well.
- High capital cost.
- Ion exchange in this configuration largely unknown.
- Fine screens required upstream.
- Resin regenerated in-place causing the unit to be out of service for periods of time.
- Large capital cost for resin replacement every 10 years.
- New technology to the City.

Figure 5.12 shows a proposed process flow diagram for this alternative.

5.3.3.3.2 *Required Facility Modifications*

A fixed bed IX system would require approximately 105 large, vertical, cylindrical treatment pressure vessels and an area of approximately 320 feet by 100 feet. Integral to the pressure vessels are supporting underdrains with a gravel and sand support system for the resin and approximately 48 inches of resin. Since the size of the vessel does not change appreciably (foregoing an "economy of scale" cost decrease) and since an overwhelming number of vessels and land area would be required, this alternative is not considered practical or cost-effective for the DLTWTF.

As an alternative to prefabricated the pressure vessels manufactured by Tonka (and others), fixed bed ion exchange media is recommended to be installed in resin contact cells, similar to conventional high rate filters typically installed in newer state-of-the-art water plants. The concern is that this has not been done before in the industry, particularly for such a large-scale project.



* Site location to be finalized by detailed design engineer.

ALTERNATIVE 2B PROCESS FLOW DIAGRAM

FIGURE 5.12

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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DLTWTF modifications required for this alternative include:

- New 140 mgd fixed bed ion exchange system.
  - Twenty-eight, 5 mgd resin contact trains.
- Fine screening facility on raw water.
- One less belt filter press at off-site SPF due to reduced solids handling.
- Expanded conventional system as required by Alternative 1B.
- 48 mgd of new filters.
- Waste brine treatment and salt recovery system

With the amount of available space due to the reconfigured conventional basins, there is an option to move the expanded conventional system back toward the end of the basins and then retrofit the first half of the basins with these resin contact trains. This would present hydraulic challenges, since water flows down through the media and is collected at the bottom of the train. In addition, this option would require an intermediate pump station at each conventional train. If this alternative were selected, these items could be further explored and vetted.

The first and only determined ion exchange plant using IX media in gravity cells was the Weymouth Water Treatment plant in La Verne, CA. Weymouth was built to include the capability for softening with both lime precipitation, as well as ion exchange. During its peak, between 1946 and 1970, the plant operated as a 200 mgd ion exchange softening facility. The ion exchange process used polystyrene strong acid cation gel resin of the same type still used today. The resin was placed in concrete gravity flow filters, each containing approximately 4,000 ft^3 of resin. Historical records do not show how long this ion exchange system operated.

Therefore, before moving forward with a customized IX system that employs gravity IX beds for contact time between the resin and flow stream, vetting the pros/cons of such an installation with the City is recommended.

Figure 5.13 illustrates the proposed site layout for the IX basin additions and the pre-screening facility for particulate removal.

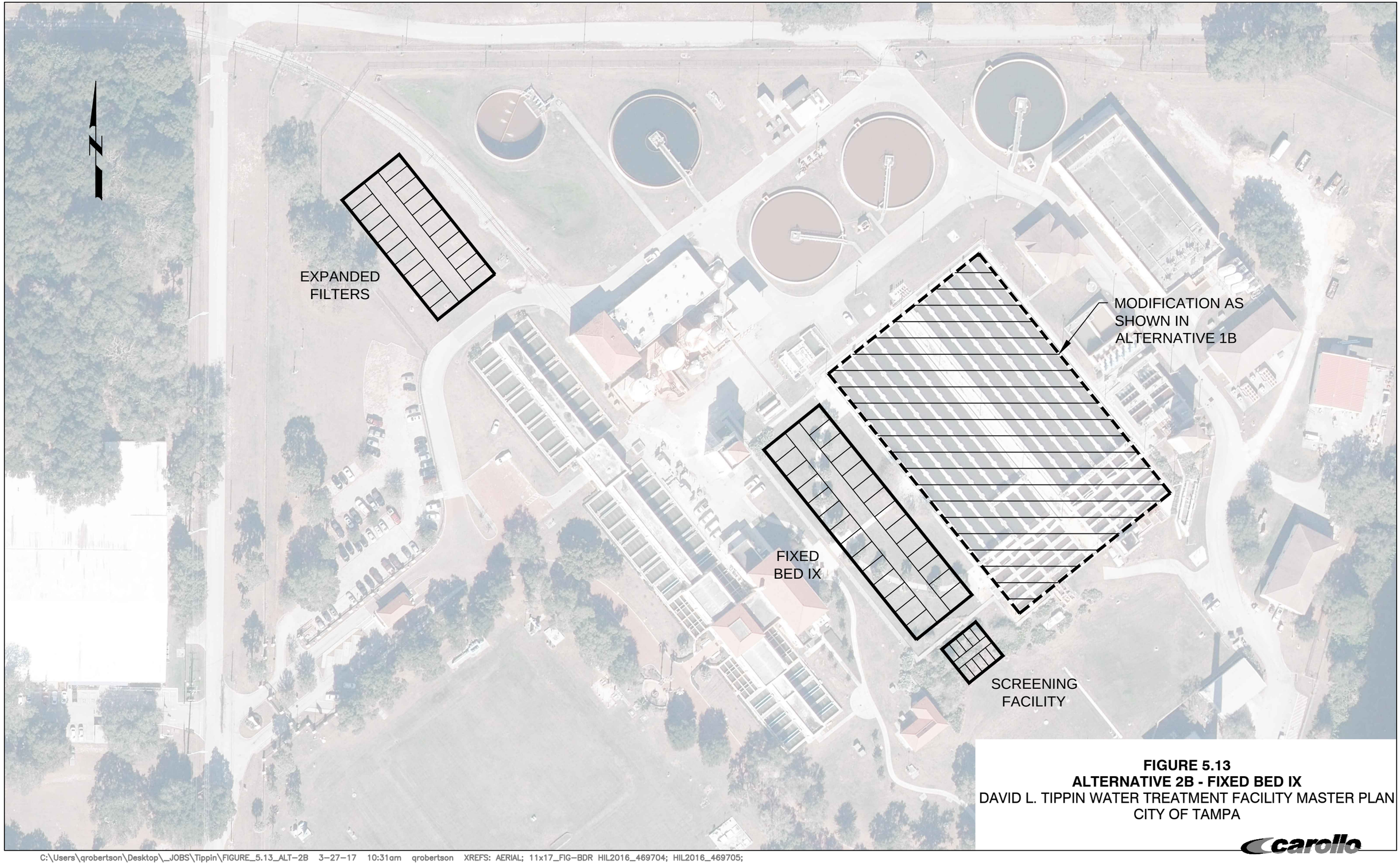

FIGURE 5.13
ALTERNATIVE 2B - FIXED BED IX
DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN
CITY OF TAMPA

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C:\Users\qrobertson\Desktop_JOBS\Tippin\FIGURE_5.13_ALT-2B 3-27-17 10:31am qrobertson XREFS: AERIAL; 11x17_FIG-BDR HIL2016_469704; HIL2016_469705;

Table 5.13 provides design criteria for the fixed bed IX alternative.

Table 5.13 Alternative 2B Fixed Bed IX System Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa		
Parameter	**Units**	**Value**
Hydraulic Loading Rate		
All in Service	gpm/sq ft	5.6
Two Out of Service	gpm/sq ft	6.0
Capacity per train	mgd	5.3
Width of Train	ft	28
Width of Train	ft	31
Surface per Train	sq ft	620
Number of new trains	-	28
Total Surface Area	sq ft	17,360
Total capacity	mgd	140
Resin depth	in	48
Resin volume, each	cf	2,480
Resin volume, total	cf	69,440

5.3.3.3.3 *Future Pilot and or Bench-Scale Testing*

Because IX in this configuration is largely unknown, a pilot gravity IX bed would be prudent to test the efficiency and operating characteristics of such a system, including:

- TOC removal efficiency.
- Ability to regenerate the resin without excess resin shearing.
- Ability to repeatedly and reliably return the system to service.

This could be integrated into the aforementioned study for a fluidized bed IX system pilot study along with piloting higher loading rates using optimized pilot filter columns, simulating the hydraulic and process improvements recommended for the filters.

5.3.3.3.4 *Capital and Operating Costs*

The opinion of probable costs for the alternative are listed in Table 5.14. There are at least two equipment manufacturers that regularly engage in fabricating fixed bed vessel ion exchange and there are numerous resin manufacturers. System costs include a fine screen pretreatment facility ahead of the fixed bed IX. As opposed to ion exchange, this would protect the IX bed from defaulting as a filtering media. Table 5.14 also includes the cost for initial resin inventory.

Table 5.14 Alternative 2B Capital Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa	
Item	**Cost**
IX Resin	$24,998,000
IX Equipment	$15,100,000
IX Tanks (Concrete & Coating)	$8,791,000
Screening Facility	$2,848,000
Demolish Existing Basins 1-4	$805,000
Deep Injection Wells	$5,000,000
Expand Conventional Train (100 mgd)	$8,390,000
New Filters (48 mgd)	$24,960,000
Sludge Processing Facility Upgrades	$1,161,000
Piloting	$800,000
Site Work (10 percent)	$9,285,000
Piping, Valves, Appurtenances (20 percent)	$18,570,000
EI&C (25 percent)	$23,210,000
Total Direct Cost	**$143,918,000**
Contingency (35 percent)	$50,370,000
GC OH&P (12 percent)	$17,270,000
Sales Tax (7 percent)	$5,040,000
Total	**$216,600,000**

Table 5.15 shows estimated operating costs for power, chemicals, and major replacement items, such as 10-year replacement of the fixed bed vessel ion exchange resin. Minor maintenance or labor costs are not included.

Table 5.15 Alternative 2B Operating and Maintenance Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa	
Item	**Cost**
Power	**$53,000**
IX Pumps	$7,000
IX Blowers	$13,000
Additional Mixers (8)	$6,000
Backwash Pumps	$18,000
Backwash Blowers	$9,000
Chemical	**$1,740,000**
Regeneration Salt	$683,000
Ferric Sulfate	$1,057,000
Other	**$450,000**
Sludge Disposal	$212,000
Sludge Processing Facility	$189,000
Deep Injection Well	$50,000
Total	**$2,244,000**
Resin Replacement (every 10 years)	**$20,832,000**

5.3.4 Alternative 3 - Split Treatment with MF/UF Treatment Train

5.3.4.1 Overview

As another alternative option, microfiltration/ultrafiltration (MF/UF) treatment train was evaluated in addition to the existing enhanced coagulation process. UF is a pressure-driven membrane process that uses microporous membranes to remove particulate matter. Used in conjunction with IX for TOC removal, biological roughing filters (BRF) for taste and odor removal, and an ultraviolet disinfection/advanced oxidation process (UV/AOP) for disinfection and future microconstituent removal, this alternative would meet the expansion needs and treatment goals for T&O, TOC, and microconstituent removal.

The following sections provide an overview on the details of each treatment process required in this alternative. Details on the IX process are provided in sections 5.3.3 for Alternative 2 and, therefore, are not repeated here

5.3.4.1.1 *Biological Roughing Filters*

Similar to the existing biological filters in place downstream of the ozone system, a BRF is proposed at the head of the new treatment train to remove taste and odor compounds (MIB and Geosmin) and act as a filter upstream of the fixed bed IX.

BRFs rely on bacteria gaining energy and reproducing by mediating the transfer electrons from reduced compounds (compounds that readily donate electrons) to oxidized compounds (those that readily accept electrons). As compounds gain or lose electrons, they are converted to different, often innocuous, forms that are thermodynamically more stable than the original compounds.

Biological drinking water treatment processes are based on the growth of bacterial communities able to mediate oxidation-reduction reactions involving at least one target contaminant. Among the biofilm systems, the fixed bed biofilm system is most commonly used. It includes a biogrowth support medium such as sand, anthracite, and/or GAC on which microbial communities attach and grow. Biofilters can be housed in pressure vessels or open basins. In pressure-vessel systems, water is pumped up-flow or down-flow across the biological bed, while gravity feeds water down-flow across open basin systems.

An open basin system is proposed here. As water is treated, biofilm growth and solids accumulation restrict flow and cause head loss across the bed. Biofilters are routinely taken offline for backwashing to manage head loss build-up and maintain uniform hydraulic flow. The backwash water can be routed back to the head of the process.

5.3.4.1.2 *Microfiltration or Ultrafiltration*

MF and UF are membrane processes typically used to remove particulate and microbial contaminants. Contaminants are physically separated using synthetic porous membranes with sub-micron pore sizes. These low-pressure membrane processes operate at pressures less than 40 psig, while vacuum-driven MF/UF systems operate at less than 10 psi equivalent pressure.

Standard operating configurations for membranes include direct (or dead-end) filtration. In this filtration method, all the feed water flow to the membrane module is converted to filtrate. Crossflow filtration converts a portion of the feed water to filtrate. The balance becomes concentrate.

Depending on the application, the concentrate is either wasted from the system or recycled and blended with the feed water. MF/UF membrane also require regular chemical cleaning, which produces a waste stream that must be neutralized and wasted with the IX regeneration brine.

5.3.4.1.3 *UV/AOP*

Ultraviolet (UV) light and advanced oxidation process (UV/AOP) will handle primary disinfection and microconstituent removal. UV disinfection is a physical process that uses no toxic chemicals and produces no known residuals or byproducts. UV lamps with a peak wavelength of 254 nanometers (nm) can inactivate pathogenic organisms, including Cryptosporidium oocysts, which are resistant to chlorine. The transference of electromagnetic energy (wavelength of 254 nanometers, or 254-nm) from a UV lamp to the

genetic material damages or destroys an organism's genetic material. The organism can no longer replicate.

The effectiveness of the radiation is a direct function of the quantity or dose of energy absorbed by the organism. Dose is defined as the product of the rate at which the energy is delivered (intensity) and the time over which the organism is exposed to this intensity (duration). Intensity is largely limited by the percent transmission of the UV light to the organism. Water with a high turbidity is thus more resistant to UV disinfection than an effluent with a low turbidity. High UV doses also can degrade certain organic compounds such as NDMA by direct photolysis.

Although UV light can be effectively used for disinfection, it provides no disinfectant residual required for drinking water applications. Chlorine or chloramines must be added to drinking water to maintain the appropriate residual in the distribution system. It is important to note that although disinfecting an effluent with UV light is very effective, it does not reduce the chorine demand of the disinfected water. Therefore, chlorine must be fed as if the water had not been disinfected with UV.

Advanced oxidation processes (AOPs) include components such as ultraviolet (UV) light, ozone (O_3), hydrogen peroxide (H_2O_2), and/or catalysts. AOPs may be used to degrade organic compounds or for disinfection. Using an AOP to degrade organic compounds may yield a series of intermediate byproducts until the compounds are completely oxidized (mineralized). The intermediate products may be more toxic than the parent compounds; therefore, the relative toxicity must be assessed.

Upon mineralization, inert byproducts are formed, such as carbon dioxide, water, and dilute mineral acids (in the case of halogenated compounds). The AOP proposed for this treatment train is UV with H_2O_2. The combination of UV and H_2O_2 is generally more efficient than UV or H_2O_2 alone for oxidation and disinfection. UV light increases the efficiency of H_2O_2 by promoting its decay to the hydroxyl radical (OH*), which is the neutral form of the hydroxide ion (OH^-) and a more powerful oxidant than H_2O_2. Adding H_2O_2 can significantly reduce the UV dose required for oxidation and disinfection.

Although the combination of UV and H_2O_2 can decrease the intensity of UV required, a continuous chemical feed of H_2O_2 is required. The effectiveness of UV alone for the degradation of soluble organic compounds is governed by the absorption spectra of the compound and the quantum yield. Since the hydroxyl radical is nonspecific in its reaction with organic compounds, the degradation efficiency of these compounds can be increased by using H_2O_2 in conjunction with UV.

5.3.4.2 Description and Process Flow

BRF is required upstream of ion exchange because it can act as a filter for fixed bed IX, if needed. A fixed bed IX system is proposed as the next unit process in the new treatment train for removal of TOC, which is a necessary pretreatment step for a MF/UF system to prevent fouling the membranes.

The next unit process proposed is MF or UF membranes. For this application, the dead-end filtration mode is recommended, and the membrane backwash water can be processed with the filter backwash water in place at the facility. The UV/AOP system follows the membranes.

Due to the nature of UV, for this application, the product water for this treatment train is proposed to be blended with the existing treatment facility water following BAC filtration. A new separate disinfection system is proposed rather than reusing the existing ozone system for this train because it is a highly treated water that should not be blended with the settled water from the conventional and Actiflo™ treatment trains. Both the existing and proposed trains are combined after filtration for final disinfection residual, storage, and pumping.

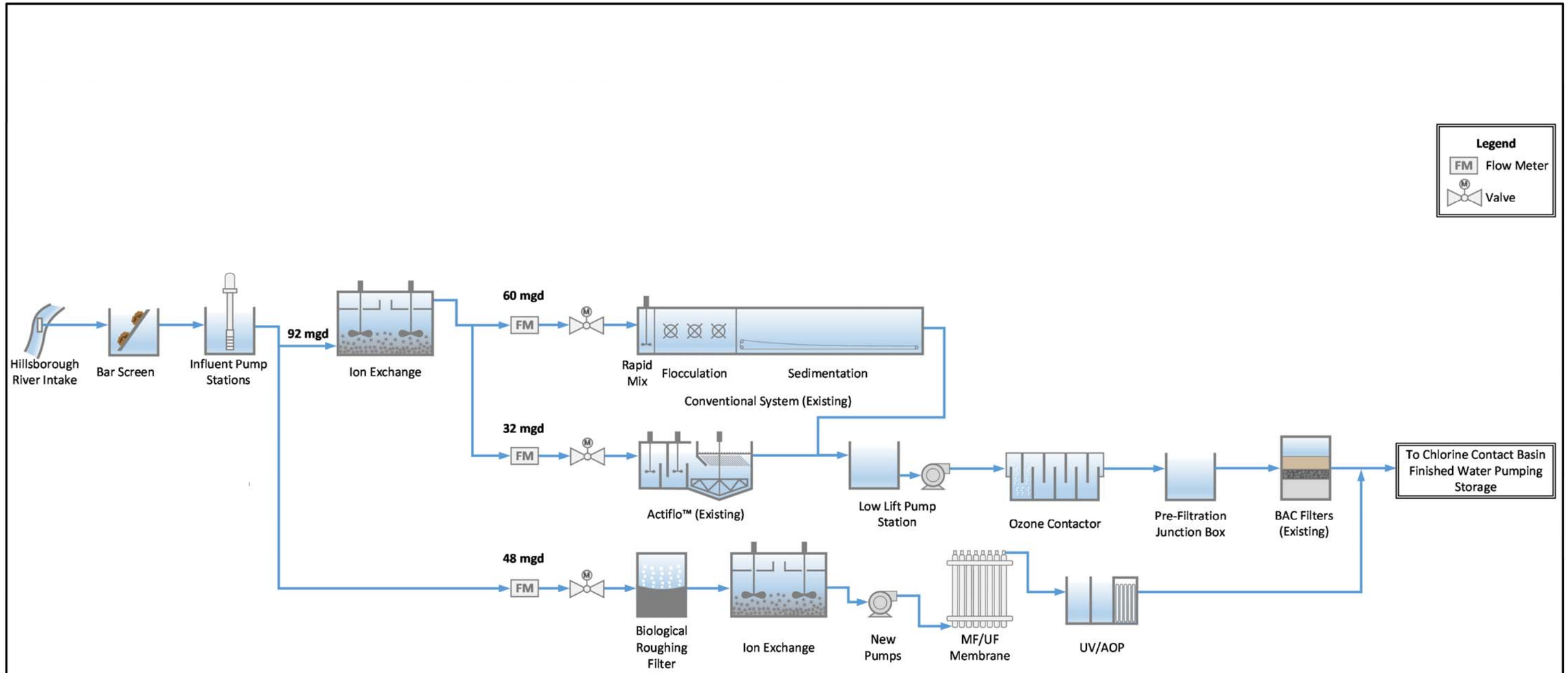
The benefits of this alternative include:

- Reduced TOC load to existing treatment systems.
- Reduced chemical costs-- lower ferric dose and no need for pH adjustment with sulfuric acid and lime.
- Reduced solids production, handling, and hauling costs.
- Expansion of existing chemical systems may not be required.
- Expansion of existing filters not required.
- Flexibility to treat TAP water in separate treatment train in future DPR scenario.
- Highly automated system.

Conversely, the drawbacks for this alternative include:

- Disposal of waste brine stream required via deep injection well.
- New and complex technology for the City.
- Energy intensive UV system.
- Intermediate pumping required for MF/UF membranes.
- Cleaning and neutralization chemicals required onsite for MF/UF membranes.
- High capital and operating costs.
- UV/AOP less effective than ozone at CEC removal in reclaimed water. (WERF.)
- UV/AOP system very large due to expected water quality.

Figure 5.14 shows the proposed process flow diagram.



ALTERNATIVE 3 PROCESS FLOW DIAGRAM

FIGURE 5.14

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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5.3.4.3 Required Facility Modifications

Modifications and additions to the existing DLTWTF would include, but are not limited to:

- A new 92 mgd fluidized IX system installed as pretreatment to the existing conventional treatment train.
- A new 48 mgd UF treatment train--
  - New 48 mgd fixed bed IX system.
  - New 48 mgd BRF system.
  - New 48 mgd MF/UF system.
  - New 48 mgd UV/AOP system.
- Modifications to the existing treatment processes, including derating the conventional system to 60 mgd, and processing a maximum of 32 mgd through existing Actiflo™ system.
- Required yard piping for raw water to new treatment train and finished water piping to clearwells.
- No additional filters.

While a fixed bed IX system is proposed for pretreatment of the MF/UF membranes, a fluidized bed IX pretreatment to the existing conventional system is proposed due to site constraints and previously mentioned associated disadvantages of the large-scale fixed bed IX system in that application.

Table 5.16 shows the proposed design criteria for the existing treatment train, and Figure 5.15 shows the proposed site layout. As shown, allowing the conventional basins to process 60 mgd results in the same design criteria as Alternative 1A.

Table 5.16 Alternative 3 Existing Treatment Train Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa			
		Value at Proposed Flow (15 mgd/Train)	
Parameter	**Units**	**Trains 5 and 6**	**Trains 7 and 8**
Rapid Mix (60 mgd)			
Detention Time	sec	14.7	14.0
Mixing Energy	sec^{-1}	157	114
Flocculation			
Detention Time	min	28.7	27.9
Mixing Energy	sec^{-1}		
Stage 1		11.7	11.8
Stage 2		11.5	11.7
Stage 3		10.7	10.9
Stage 4		10.6	10.72
Velocity Through Ports in Flocculation Stage	fps	0.73 - 1.15	
Velocity Through Port to Sedimentation	fps	0.86	
Sedimentation			
Detention Time	hours	2.58	2.75
Surface Overflow Rate	gpm/sq ft	0.69	0.69
Reynold's Number	-	31,700	30,900
Froude Number	-	3.6×10^{-6}	3.0×10^{-6}
Horizontal Velocity	ft/min	1.94	1.82
Actiflo™ (32 mgd)			
Parameter	**Units**	**Value**	
Actiflo™ Capacity per Train	mgd	20	
Number of Actiflo™ Trains	-	2	
Total Actiflo™ Capacity	mgd	40	
Target Effluent Turbidity	NTU	< 2.0	
Target TOC Removal	mg/L	> 70 percent removal	
Target Effluent Color	PCU	< 10	

Table 5.16 Alternative 3 Existing Treatment Train Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa			
		Value at Proposed Flow (15 mgd/Train)	
Parameter	**Units**	**Trains 5 and 6**	**Trains 7 and 8**
Rapid Mix (60 mgd)			
Coagulation Tank Detention Time	min	3.0	
Maturation Tank Detention Time	min	6.0	
Settling Tank Hydraulic Loading Rate	gpm/sq ft	25	
Sand Recirculation Flow (per pump)	gpm	435	
Estimated Total Sludge Waste Flow	gpm	696	
Estimated Sludge Concentration	percent	0.01 - 0.5	

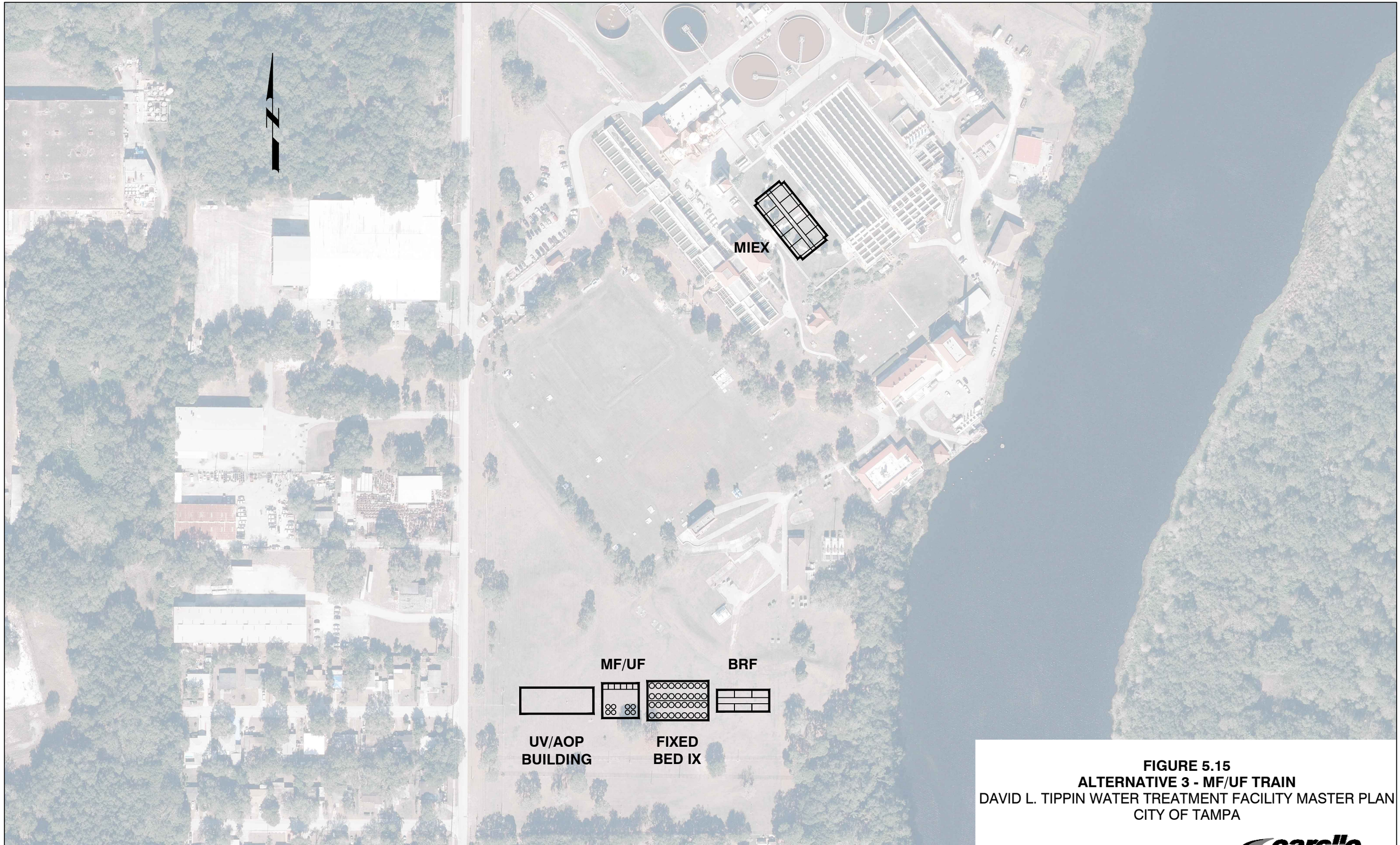

FIGURE 5.15
ALTERNATIVE 3 - MF/UF TRAIN
DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN
CITY OF TAMPA

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C:\Users\qrobertson\Desktop_JOBS\Tippin\FIGURE_5.15_ALT-3 3-27-17 10:32am qrobertson XREFS: AERIAL; 11x17_FIG-BDR HIL2016_469704; HIL2016_469705;

Table 5.17 shows the design criteria for the new treatment train and summarizes the proposed sizing for each of the unit processes. This train is designed to be able to treat the existing surface water, future TAP water blended in the Hillsborough River, or future DPR water from the Howard F. Curren WRF (up to 50 mgd).

Table 5.17 Alternative 3 New Treatment Train Design Criteria
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Parameter	Units	Value
Biological Roughing Filter		
Type	-	Gravity
Purpose	-	T&O Removal
Number of units	No.	6
Filter dimensions	ft x ft	40 x 18
Surface area per filter	sf	720
Total surface area	sf	4,320
Filtration rate (all in service)	gpm/sq ft	8.0
Filtration rate (one out of service)	gpm/sq ft	9.6
EBCT (all in service)	min	3.7
EBCT (one out of service)	min	3.1
Filter media type	-	GAC
Depth	in	48
Effective Size	-	1.0 - 1.2
Uniformity Coefficient	-	< 1.5
Backwashes per day (maximum)	No.	3
Air scour rate	scfm/sq ft	3 - 4
Backwash rate	gpm/sq ft	8 - 20
Total volume per backwash	gal	201,600
Ion Exchange		
Type	-	Fixed bed, pressure vessel
Purpose	-	TOC removal
Number of units	No.	36
Unit diameter	ft	12
Surface area per unit	sq ft	113
Total surface area	sq ft	4,072
Treatment rate (all in service)	gpm/sq ft	8.5

Table 5.17 Alternative 3 New Treatment Train Design Criteria David L. Tippin Water Treatment Facility Master Plan City of Tampa		
Parameter	**Units**	**Value**
Treatment rate (two out of service)	gpm/sq ft	9.0
Resin type	-	Strong-base anion exchange
Resin depth	in	48
Resin volume per unit	cf	452
Total resin volume	cf	16,286
Regeneration frequency	days	6
Regeneration chemical	-	NaCl
Salt consumption (max)	lb/MG	600
MF/UF		
Type	-	Polymeric hollow fiber
Purpose	-	Turbidity removal
Number of treatment trains	No.	6
Surface area per train	sf	264,000
Total surface area	sf	1,584,000
Flux (all in service)	gfd	32
Flux (one out of service)	gfd	38
Backwash interval	min	60
Integrity test interval	hr	24
CIP interval	days	30
UV/AOP		
Type		UV + H_2O_2
Purpose		Disinfection / 1-log removal NDMA and 0.5-log removal 1,4-dioxane
Number of trains	No.	12
Design UVT	percent	75
Lamp power	W	600
Rows	No.	28
No. of lamps	No.	4,032
H_2O_2 dose	mg/L	26

5.3.4.4 Future Pilot and/or Bench-Scale Testing

Piloting the complex and highly technical process train proposed for this alternative is recommended to develop design criteria for each unit process and its ancillary support systems. In addition, the various feed water conditions should be pilot-tested scale including the existing Hillsborough River water, future TAP water blended in the Hillsborough River, and the alternative water supply from the Howard F. Curren WRF to monitor the impacts on the unit processes and monitor the resulting product water for meeting water quality goals.

5.3.4.5 Capital and Operating Costs

The opinion of probable costs for this alternative are listed in Table 5.18. The capital cost includes each of the unit processes and ancillary support systems for the new MF/UF process train as well as a MIEX pretreatment system on the existing conventional treatment trains.

Table 5.18 Alternative 3 Capital Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Cost
BRF (50 mgd)	$8,470,000
Fixed Bed IX (50 mgd)	$20,980,000
MF/UF (50 mgd)	$23,284,000
UV/AOP (50 mgd)	$32,760,000
Deep Injection Well	$5,000,000
MIEX (92 mgd)	$38,443,000
Sludge Processing Facility Upgrades	$1,161,000
Piloting	$800,000
Site Work (10 percent)	$13,090,000
Piping, Valves, Appurtenances (20 percent)	$26,180,000
EI&C (25 percent)	$32,724,000
Total Direct Cost	**$202,892,000**
Contingency (25 percent)	$50,723,000
GC OH&P (12 percent)	$24,347,000
Sales Tax (7 percent)	$7,101,000
Total	**$285,063,000**

Table 5.19 shows the operating costs developed for this alternative. Operating costs include power, chemicals, and recurring replacement costs. The MF/UF system requires

chemicals for regular cleaning of the membranes and chemicals for neutralizing the waste cleaning solution. In addition, membranes are assumed to be replaced every 7 years. The IX resin is assume to be replaced every 10 years and requires salt for regeneration.

Table 5.19 Alternative 3 Operating and Maintenance Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa

Item	Cost
Power	**$1,986,000**
BRF Backwash Pumps & Blowers	$24,000
MF/UF Pumping & Ancillary	$122,000
UV Lamps	$1,695,000
MIEX	$145,000
Chemical	**$5,666,000**
MIEX Regeneration Salt	$391,000
MIEX Resin	$1,211,000
Ferric Sulfate	$738,000
UF Train Chemicals	$3,326,000
Other	**$2,208,000**
Sludge Disposal	$148,000
Sludge Processing Facility	$189,000
Deep Injection Well	$50,000
UV Lamps, Sleeves, Ballasts	$1,821,000
Total	**$9,860,000**
IX Resin Replacement (every 10 years)	**$5,374,000**
Membrane Replacement (every 7 years)	**$2,088,000**

5.3.5 Alternative 4 - MF/UF/NF with Biological Roughing Filters

Primarily used for softening and organics removal, nanofiltration (NF) is a well-established process that uses semi-permeable membranes and a driving force of hydraulic pressure. Able to reject contaminants as small as 0.001 um, NF membranes can remove almost all divalent ions (hardness) but monovalent ions (salinity) only to a limited extent. NF can also remove synthetic organic compounds (SOCs) and disinfection byproduct (DBP) precursors. Therefore, NF is primarily used for softening and organics removal.

To use NF membranes, operation requires addition of an antiscalant to mitigate the precipitation of less soluble salts on the membrane. Alternative 4 considers replacement of the existing conventional and Actiflo™ trains with a new 140 mgd treatment train of

biological roughing filters followed by MF/UF membranes and nanofiltration (NF) membranes.

In the aerobic environment of a surface water such as at DLTWTF, unless it is pretreated for biodegradable organic carbon, NF is prone to biological fouling, compromising its viability. In addition, the molecular weight cutoff of NF is too large to effectively remove taste and odor compounds, notably MIB and geosmin. Therefore, a biological roughing filter is recommended upstream of the NF process.

NF is also very sensitive to fouling by particles (turbidity) and requires pretreatment to remove turbidity. Common NF pretreatment technologies include MF/UF membranes and granular media filtration. Thus, an MF/UF system is proposed upstream of NF.

As mentioned previously, the MF/UF system must be preceded by effective coagulation and TOC removal to work properly and prevent excessive fouling and unacceptable cleaning intervals. If granular media filtration were selected as the pretreatment to NF, coagulation would still be required to meet the coagulant demand of TOC for particle destabilization and turbidity removal in the granular media filters.

Consequently, a coagulation or ion exchange step for TOC removal is recommended upstream of the MF/UF system. Since NF at the DLTWTF would be used for TOC and color removal and those constituents are removed through the pretreatment required, NF does not provide any particular benefits.

Implementing TAP in the future brings up another disadvantage of an NF system. TAP will require microconstituent removal, which in turn would require reverse osmosis membranes due to molecular weight cut off and percent removals. Because of its disadvantages and the high costs of treating 140 mgd with each of the proposed unit processes, this alternative was not considered further.

5.4 CAPITAL, OPERATING, AND LIFE-CYCLE COSTS

In accordance with the methods and approach described in Section 5.2, capital, operating, and life-cycle costs were developed for each alternative (excluding Alternative 4). Items common and equivalent for all alternatives were not included in the cost.

Table 5.20 summarizes the results of these cost developments for each alternative. Alternative 1B resulted in the lowest calculated life-cycle costs, and Alternative 3 had the highest. Although cost is important for determining the final recommendation, it is not the only consideration. The alternatives and their respective rankings of critical success factors are compared in subsequent sections.

Table 5.20 Alternatives Life-Cycle Cost Comparison David L. Tippin Water Treatment Facility Master Plan City of Tampa					
Cost Component	**Alternative 1A: New Actiflo™**	**Alternative 1B: Expanded Conventional**	**Alternative 2A: Fluidized Bed IX**	**Alternative 2B: Fixed Bed IX**	**Alternative 3: MF/UF Split Treatment**
Capital ($1,000)	$78,600	$76,700	$166,200	$216,600	$285,100
Amortized O&M ($1,000/yr)	$6,900	$8,200	$5,100	$5,100	$11,700
Life-Cycle Cost (Net Present Value) ($1,000)	$218,400	$242,900	$269,000	$288,500	$492,300

When comparing longer life-cycles the net present value for Alternative 1B and 2A get much closer. There is a 10.2 percent difference in Alternative 1B and 2A over a 20 year life-cycle and no significant difference (2.7 percent) over a 30 year life-cycle. Since much of the equipment is likely to last longer than 20 years, especially at the DLTWTF, an average 30 year equipment life is reasonable. Additionally, there is a possibility that chemical costs could increase more than 3 percent per year over the span of 30 years. Since Alternative 1B relies heavily on chemical usage, increases in costs above 3 percent annually would make Alternative 2A more attractive in regard to life-cycle costs.

5.5 COMPARISON OF ALTERNATIVES AND RANKINGS

As stated earlier, each alternative received a score from 1 to 10 for each critical success factor, with a 10 best satisfying that criterion. The score was then multiplied by the corresponding weighting factor (number of votes) for a composite score for that factor. The sum of the composite scores for each factor was then considered for each alternative, and alternatives were compared to one another.

The alternative with the highest score out of a possible 200 was selected as the best treatment alternative to satisfy the City's needs most effectively and efficiently. Table 5.21 compares the pros and cons of each alternative to aid in scoring the alternatives relative to each critical success factor.

Table 5.21 Comparison of Alternatives David L. Tippin Water Treatment Facility Master Plan City of Tampa					
Parameter	**1A**	**1B**	**2A**	**2B**	**3**
Familiar technology	Yes	Yes	No	No	No
Reduces treatment costs (chemicals, solids handling)	No	No	Yes	Yes	Yes
Uses existing infrastructure	Yes	Yes	No	No	No
Meets WQ goals	Yes	Yes	Yes	Yes	Yes
Able to treat future TAP water	Yes	Yes	Yes	Yes	Yes
Able to treat future TAP water in separate treatment train	No	No	No	No	Yes
Chemical system expansion required	Yes	Yes	No	No	No
Filter expansion required	Yes	Yes	Yes	Yes	No
Reduces infrastructure maintenance from low pH	No	No	Yes	Yes	Yes
Produces waste stream requiring DIW	No	No	No	Yes	Yes
Proven technology	Yes	Yes	Limited	No	Limited
Proprietary technology	No	No	Yes	No	No

Table 5.22 shows the scoring matrix for the five alternatives with the total composite score. The raw scores were developed with input from City staff at a workshop about results of the alternatives evaluation.

As the table shows, except for Alternative 3, scores for all alternatives were very close to one another, though Alternative 2A received the highest score. The following discusses the ranking of each alternative in terms of scores for each critical success factor (CSF).

1. **Optimizes process and reduces treatment cost** - Alternatives 2A and 2B (the IX alternatives) received the highest score for their ability to reduce ferric sulfate doses and eliminate the need for pH adjustment chemicals in the conventional and Actiflo™ treatment systems. Alternatives 1A and 1B include upgrades to optimize the existing treatment process but do not reduce treatment costs significantly. Alternative 3 includes additional unit processes with high treatment costs.

2. **Provides superior water quality in most efficient manner** - All the alternatives were expected to provide water quality approximately equivalent to current operations. The difference in the scores is due to efficiencies from chemical and power requirements. Alternatives 1A and 1B require EC chemicals. Alternatives 2A and 2B reduce the chemical requirements compared to current operations. Alternative 3 requires cleaning chemicals and significant power compared to the others.
3. **Minimizes capital costs** - The alternatives were ranked according to their respective capital costs.
4. **Minimizes O&M costs** - The alternatives were ranked according to their respective O&M costs.
5. **Solves hydraulic bottlenecks with maximum use of existing infrastructure** - All alternatives include corrections to the existing hydraulic bottlenecks. Alternative 1B was given the highest score because it does not require any additional infrastructure. Alternative 1A was given the next highest score because it requires a comparatively small structure for the additional Actiflo™ treatment capacity. Alternative 2A requires new basins for the MIEX treatment system. Alternative 2B requires new, large basins for the fixed bed IX contact cells and the new fine screening facility. Alternative 3 requires many new structures for the new unit processes in the MF/UF treatment train.
6. **Maximizes reliability and redundancy** - Reliability and redundancy are best achieved with Alternative 1B with correction to the existing conventional treatment train flocculation and sedimentation zones. Alternative 1A is a close second due to the use of additional Actiflo™ treatment capacity, but it does not improve the reliability of the conventional treatment trains. Alternative 2A has some inherent risk due to the unknowns of seasonal impacts to the resin and the proprietary nature of the MIEX system. Alternative 2B is a higher risk because the proposed configuration is largely unknown in the industry. Alternative 3 is high risk because of the unconventional nature of the technology.
7. **Best meets drinking water regulations and safety and security concerns** - All alternatives would comply with drinking water regulations. Safety and security concerns from the storage of strong chemicals onsite were considered when scoring the alternatives. Alternative 2A and 2B reduce the chemicals required onsite, but the others do not.

Table 5.22 Alternatives Evaluation Matrix
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Critical Success Factor	Optimizes Process & Reduces Treatment Costs (1)		Provides Superior Water Quality in Most Efficient Manner (2)		Minimizes Capital Costs (3)		Minimizes O&M Costs (3)		Solves Hydraulic Bottlenecks with Max Use of Existing Infrastructure (4)		Maximizes Reliability & Redundancy (5)		Best Meets Drinking Water Regs & Safety / Security Concerns (6&7)		Total Score
Weighting Factor	**6**		**5**		**2**		**2**		**2**		**2**		**1**		
Alternative	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	
1A	7	42	7	35	9	18	7	14	9	18	8	16	4	4	**147**
1B	7	42	7	35	9	18	7	14	10	20	9	18	4	4	**151**
2A	9	54	8	40	5	10	9	18	6	12	6	12	8	8	**154**
2B	9	54	8	40	3	6	9	18	5	10	4	8	8	8	**144**
3	1	6	6	30	1	2	1	2	3	6	3	6	4	4	**56**

5.6 SUMMARY AND RECOMMENDATIONS

This chapter summarized the results of the alternatives evaluation including advantages, disadvantages, costs, and an overall comparison and ranking for selection of the optimal alternative for expansion and improving or replacing the existing EC treatment process.

In a two-day workshop with City staff, many treatment process alternatives were vetted, and five alternatives were ultimately evaluated in depth. These five alternatives were selected for their ability to (1) meet the City's capacity goals of 140 mgd maximum per day by 2032, (2) meet water quality goals, (3) achieve treatment objectives for reducing chemical and solids handling expenditures, and (4) treat water from the future implementation of TAP. The five alternatives evaluated are:

- Alternative 1A - Expand Actiflo™
- Alternative 1B - Expand conventional
- Alternative 2A - MIEX
- Alternative 2B - Fixed bed IX
- Alternative 3 - Split treatment with MF/UF membranes

The cost evaluation, ranking analysis, and in depth discussions with the City resulted in the recommendation that the City proceed with the implementation of Alternative 2A (improving/expanding the conventional treatment trains and retaining the existing Actiflo™ treatment trains and implementing a new 140 mgd MIEX pretreatment system), based on the results of a pilot study conducted from October 2017 – April 2018. These results are summarized in the Final Pilot Study Report completed by Carollo Engineers, Inc. in June 2018, but the recommendations are reiterated here.

With respect to the filters, based on the pilot plant results (unit filter run volumes, solids loading rates, runtimes, and clean bed head losses), it is believed even with MIEX pretreatment the existing filters can only reliably and efficiently treat at a max loading rate of 2.9 gpm/sq ft (~92 mgd assuming two large filters out of service) as originally noted in Chapter 3. At this rate, the expansion project would include 48 mgd of new filters. Therefore, it is recommended that the City take a phased approach to filter expansion as to not unnecessarily construct new filters. The City, first, should implement the hydraulic improvement recommendations, as specified in Chapter 4 of the Master Plan, first and then proceed with full scale demonstration and testing to witness any impacts to increased filter loading rates, runtimes, and UFRVs. This can be completed independently of MIEX implementation since this pilot study did not find MIEX pretreatment to significantly impact or improve filter operations. Filtration optimization with the new implemented hydraulic and process improvements could then help determine the new max loading rates and

subsequent finalization of exact quantity of additional filters needed to meet 140 mgd capacity.

Based on the collective results and observations of the pilot study on water quality, capital and O&M costs, and qualitative considerations, it is recommended that the City implement Alternative 2A that includes MIEX as a pretreatment system for the DLTWTF; however, with a caveat that the City include the cost of an additional extended (one year) pilot study with MIEX pretreatment in operation the entire duration. Additionally, mitigation and resolution of the risks identified and presented herein should be wholly resolved through piloting before the MIEX full scale system is constructed. This pilot would be operated in conjunction with the conceptual engineering design of the full scale MIEX system.

This recommendation is partly based on the water quality and economic considerations of MIEX. Water quality and overall process performance for the pilot and full scale systems were very similar, with MIEX at times providing lower finished water TOC concentrations. Additionally, the economic analysis showed that both alternatives have essentially the same net present values at 30 years, with MIEX being 2.7 percent less in NPV life-cycle. Considering this, MIEX is a viable and promising treatment option for the DLTWTF. However, due to the qualitative considerations and intermittent gaps in data, it is recommended to fully capture an entire year of data, not only in regard to TOC removal, but more specifically to include:

- Resin condition monitoring (RCM) analysis and organics desorption during the regeneration process throughout the year to understand degradation and decrease in organics removal performance over time.
- VSEP treatment runs multiple times per month to gather additional data to fully understand potential salt savings, in addition to multiple sample set deliveries to the third party vendor for confirmation of viable concrete stream usage.
- Collection of ozone dose and demand data, and bromate data (can be completed at bench scale), and consideration of various bromate control techniques. Testing should include blends of raw water from various DLTWTF supply sources including the reservoir and ASR recovery wells.
- Collection of DBP data to determine the impacts of prechlorination prior to MIEX (can be completed at bench scale)
- Evaluation and mitigation of air entrainment issues associated with the original pilot.
- Operation of the MIEX system at 600 bed volumes throughout the study to determine the impacts on TOC treatment, ozone demand, and filter runs.
- Piloting of the SIX process simultaneously with the MIEX process (for the last 6 months).

Additionally, IXOM should provide a performance guarantee for TOC removal as well as documentation supporting their intent to construct a resin manufacturing facility in the United States.

Without full understanding and mitigation of the identified risks, MIEX cannot be confidently recommended. By conducting additional piloting to confirm risk mitigation approaches in conjunction with the conceptual design, the City and their consultant could better understand the needed customized design of this complex system to fully meet the needs of the DLTWTF while minimizing risks and unknowns.

The following modifications are recommended under this alternative:

- Construct a new 140 MIEX system implemented as a pretreatment step to the conventional and Actiflo™ systems
- Modify the rapid mix basins with a flash mix pump system.
- Reconfigure the conventional basin flocculation basins to a plug flow configuration with three stages of tapered energy flocculation with new flocculators, or, as a minimum, dramatically improve the hydraulic and mixing deficiencies of the existing floc systems
- Installation of plate setters in the sedimentation basins, pending results of full-scale testing.
- Installation of 22 new filters, unless piloting demonstrates fewer filters without compromised treatment.
- Expansion of select existing chemical systems to accommodate the additional capacity pending and based on results from piloting (once future chemical dosage needs are understood).
- Re-evaluate the needed capacity of the off-site sludge processing facility based on the reduced volumes of solids generated with an ion exchange pretreatment system.

As discussed in Chapter 6 on benchmarking, the DLTWTF performs well compared to peer facilities. Its existing processes treat Hillsborough River water and produce high-quality potable water. Expenditures for chemical per gallon of water treated and per mg/L of TOC removed are at or below that of many other utilities around Florida. Further reducing chemical and solids handling costs while maintaining similar TOC removal to current operations would be realized with implementation of Alternative 2A.

BENCHMARKING STUDY

6.1 CHAPTER SUMMARY

The benchmarking study compared the David L. Tippin Water Treatment Facility (DLTWTF's) operations and performance to six utilities with like-sized plants of similar complexity, process systems, and raw water quality. These comparisons helped identify opportunities to improve the DLTWTF. The benchmarked plants are:

- Tampa Bay Regional Surface Water Treatment Plant (WTP), Tampa, FL
- Peace River Water Treatment Facility (WTF), FL
- Lake Manatee Water WTP, Bradenton, FL
- Preston & Hialeah WTPs, Miami, FL
- Santan Vista WTP, Gilbert AZ
- Palm Beach County WTP 2, Palm Beach County, FL

Data gathered from customized questionnaires and follow-up communications were compiled and analyzed.

Process performance benchmarking involved analyzing the performance of treatment processes (individually and collectively) to determine opportunities for enhanced performance.

For operational components, non-cost data, such as power use, chemical use, number of labor staff, and sludge disposal, were examined. Cost data also was studied to eliminate biases caused by geographical differences in unit costs.

Cost parameters included power, chemicals, residuals, labor, maintenance, laboratory, and other costs for plant operations and maintenance. To remove the discrepancy of scale among the plants, usage and costs were normalized to annual average production and, where pertinent, to total organic carbon (TOC) removed).

In terms of water quality, the DLTWTF is characterized by TOC, color, and turbidity levels comparable to corresponding levels seen in other benchmarked plants, especially the surface in Florida. The DLTWTF has the highest relative TOC removal compared to the other plants: 80 percent versus 60 to 75 percent. Additionally, TOC removal by the pretreatment process is highest for the DLTWTF compared to the other plants. However, the other plants do not target an absolute TOC level in the finished water. Rather, they remove TOC based on regulatory requirements, which in turn are based on source water TOC and alkalinity requirements. Color and turbidity removal at the DLTWTF were comparable to those of the other benchmarked plants.

In terms of normalized energy, chemical use and overall operation and maintenance (O&M) cost, usage and costs for the DLTWTF are comparable to those for the benchmarked plants, especially the surface WTPs in Florida. Also, the overall O&M cost was comparable to or slightly lower than the overall cost for the Palm Beach County WTP which uses the magnetic ion exchange (MIEX) process.

6.2 INTRODUCTION

As part of the master plan, a benchmarking study was performed to compare operations and performance of the plant to utilities to like-sized plants with similar complexity, process systems, and raw water quality. This chapter details the methodology and results of the benchmarking study.

The goal of the benchmarking study was to compare various criteria associated with the processes and cost of operations of the DLTWTF to other treatment plants that are similar in one or more aspects (i.e., size, raw water quality, complexity of operations, treatment processes .employed, etc.). These comparisons would help to identify opportunities where the DLTWTF can be improved.

6.3 APPROACH

The approach to benchmarking included selection of similar plants, and comparison of performance and O&M cost components.

Six (6) treatment plants of interest that were similar in one or more aspects (i.e., size, raw water quality, complexity of operations, treatment processes employed, etc.) to DLTWTF were identified by the team and discussed with the City. The candidate plants were contacted and asked to participate in the benchmarking study. A customized questionnaire was developed and submitted to the participating plants to facilitate the data collection. Table 6.1 summarizes the plants and the reasons for their selection.

Each treatment plant received a customized questionnaire. Responses were reviewed for consistency and completeness. Follow-up communications with plant personnel clarified data and filled in any information gaps in the initial responses. Raw data were then compiled into spreadsheets and analyzed to compare performance of treatment processes and the costs of O&M components.

To benchmark process performance and identify improvements, the performance of treatment processes (individually and collectively) was analyzed.

To benchmark operational components, both the non-cost and cost data were analyzed.

Table 6.1 Facilities Benchmarked
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

#	Water Treatment Plant	Owner	Capacity (mgd)	WQ Challenge	Treatment Process	Reasons for Benchmarking
-	David L. Tippin WTF Tampa, FL	City of Tampa	120	TOC, Color	(Conventional C/F/S(1) + Actiflo), Ozone, BAF	Baseline Plant
1	Tampa Bay Regional Surface WTP Tampa, FL	Tampa Bay Water	120	TOC, Color	Actiflo, Ozone, BAF	Similar processes, water quality and capacity to DLTWTF, FL location
2	Peace River WTF, FL	Peace River Authority	51	TOC, T&O	PAC, Conventional C/F/S, Media Filters	Similar process (conv. C/F/S), water quality, and FL location
3	Lake Manatee WTP Bradenton, FL	Manatee County	54	TOC, T&O	PAC, Conventional C/F/S, Media Filters	Similar process (conv. C/F/S), water quality, and FL location
4	Preston & Hialeah WTPs Miami, FL	Miami-Dade WASD	235	TOC, Color	Lime softening/precipitation , Rapid sand filters	Similar TOC, magnitude of capacity, FL location
5	Santan Vista WTP, Gilbert AZ	Town of Gilbert & City of Chandler	24	TOC	Actiflo, Ozone, BAF	Similar processes (Actiflo, Ozone, BAF)
6	Palm Beach County WTP 2 Palm Beach County, FL	Palm Beach County	17	TOC, Color, Hardness, high DBPs	MIEX, Softening, BAF	Similar TOC, FL location; comparison with MIEX

Notes:
(1) C/F/S = Coagulation/Flocculation/Sedimentation.
(2) MIEX = Magnetic Ion Exchange

Non-cost parameters included power use, chemical use, number of labor staff, and tons of sludge disposed, among other parameters. Cost parameters included power, chemicals, residuals, labor, maintenance, laboratory, and other costs for plant operations and maintenance and were compared to eliminate biases from geographical differences in unit costs. In addition to comparing these parameters individually, the total annual costs and cost per 1,000 gallons of water produced for plant operations were developed and compared.

To remove the discrepancy of scale among the plants, usage and costs were normalized to annual average production and, where pertinent, to total organic carbon (TOC) removed).

6.3.1 Plants Benchmarked

6.3.1.1 Tampa Bay Regional Surface WTP

The Tampa Bay Regional Surface WTP is a 120 million gallons per day (mgd) facility in Tampa, Florida, owned by Tampa Bay Water. Treatment is Actiflo™ high-rate ballasted flocculation/sedimentation, ozonation, and biological active filtration (BAF). The treatment process is identical to the one used at the DLTWTF, except that Tampa Bay WTP uses only Actiflo™, unlike DLTWTF which comprises parallel trains - one with Actiflo™ and another with conventional coagulation/flocculation/ sedimentation (C/F/S) process.

The primary water quality challenges the Tampa plant faces are Total Organic Carbon (TOC) and color removal. This plant was selected for the benchmarking study due to its proximity to the DLTWTF and the strong similarities in capacity and treatment processes.

6.3.1.2 Peace River WTF

Located in DeSoto County, Florida, this facility is operated as a regional partnership by the Peace River Manasota Regional Water Supply Authority. With a capacity of 51 mgd, it was selected for the benchmarking study because it is in Florida and has water quality and treatment processes similar to those at the DLTWTF.

As with the DLTWTF, TOC is a primary water quality challenge for the Peace River facility, as well as taste and odor (T&O). Conventional C/F/S and media filters are used for treatment, while powdered activated carbon (PAC) addresses T&O issues. Peace River's C/F/S system uses alum as a coagulant and features both conventional rectangular flocculation/sedimentation trains and solid contact units (SCUs).

6.3.1.3 Lake Manatee WTP

Located in Bradenton, Florida, this facility is owned and operated by Manatee County Utilities. It is unique among those selected for benchmarking because it draws from both surface and groundwater sources, operating two surface water treatment trains and one groundwater treatment train at the same facility. The two surface water treatment trains are

rated for 54 mgd and account for approximately 60 percent of the plant's total finished water production.

For comparison with the DLTWTF, only the surface water trains were evaluated. Where combined cost data were available for the surface and groundwater treatment (for example, maintenance and laboratory costs), the cost was proportioned to surface water treatment based on either design capacity or annual production.

The primary water quality challenges of the raw surface water are TOC and T&O. Similar to the Peace River facility, these are treated with PAC, C/F/S, and media filtration. Alum is used as the coagulant in the C/F/S pretreatment. Finished water from surface water treatment is blended with finished water from the groundwater train for disinfection and distribution.

6.3.1.4 Preston and Hialeah WTPs

Preston and Hialeah WTPs are two separate facilities, both withdrawing raw water from the Biscayne aquifer using a well system of four well-fields in Miami-Dade County. Because they are close to each other and share raw water sources, they have been combined for this benchmarking study. The combined capacity of these two plants is 235 mgd.

Preston WTP is a 165-mgd plant that uses lime softening with caustic soda and polymer, recarbonation, dual-media filtration, and air stripping. Hialeah WTP is a 70-mgd plant that uses lime softening with activated sodium silicate, recarbonation, mono-media filtration, and air stripping. Hialeah WTP also hosts major pumping facilities that serve both itself and the Preston WTP.

Due to the varied locations of the source well-fields, raw water quality for both Preston and Hialeah plants is highly variable. The primary parameters of concern are TOC and color. These plants were selected for the benchmarking study because of their similar raw water TOC, the magnitude of their combined capacity, and their location in Florida.

A unique characteristic of these facilities is recalcination to generate lime for the treatment process. Recalcination involves burning natural gas to heat lime sludge from the treatment process. It requires significant energy from the system but reduces net process solids production to a negligible amount.

6.3.1.5 Santan Vista WTP

Located in Gilbert, Arizona, Santan Vista WTP is a 24-mgd facility owned by the town of Gilbert and the City of Chandler. Like the DLTWTF, it operates an Actiflo™ ballasted flocculation system, ozonation, and BAF.

Rated at 24 mgd, the facility is expanding to 48 mgd by adding a new Actiflo™ system and ozone contactor, additional filters, finished water storage, and sludge handling capacity. The primary water quality challenge is TOC reduction.

6.3.1.6 Palm Beach County WTP No. 2

This 17-mgd facility is operated by Palm Beach County Water Utilities Department in Palm Beach County, Florida. Raw water for the plant comes from two well-fields near the plant site. The primary treatment challenge for this facility is removing TOC, color, and hardness and limiting disinfection byproducts.

Of the facilities in this benchmarking study, Palm Beach County is unique because it uses the Magnetic Ion Exchange (MIEX) process to treat TOC and color. The chemical/media costs for a MIEX system include the purchase of the resin media and salt to regenerate the resin. Since MIEX is being evaluated as a possible treatment alternative at the DLTWTF, the performance of the WTP No. 2 system is of particular interest.

Post-MIEX, water is directed to lime softening and media filtration systems. Lime removes hardness and generates high residuals.

To better compare with the DLTWTF and simulate a potential alternative, two scenarios were considered for the Palm Beach County plant when comparing chemical use and costs or total O&M costs. The first scenario reflects actual plant operation. In the second scenario, an adjustment was made to the Palm Beach County WTP data to remove the use/cost of lime and associated residuals, while adding cost for 50 mg/L of ferric chloride and associated residuals. Accordingly, where presented in figures or tables, the data for the second scenario are labeled 'Palm Beach County WTP (adjusted)'.

6.4 WATER QUALITY AND PROCESS PERFORMANCE

Process performance is compared in terms of levels and removal of turbidity, color, and TOC. Water quality data show the average and range for one recent year.

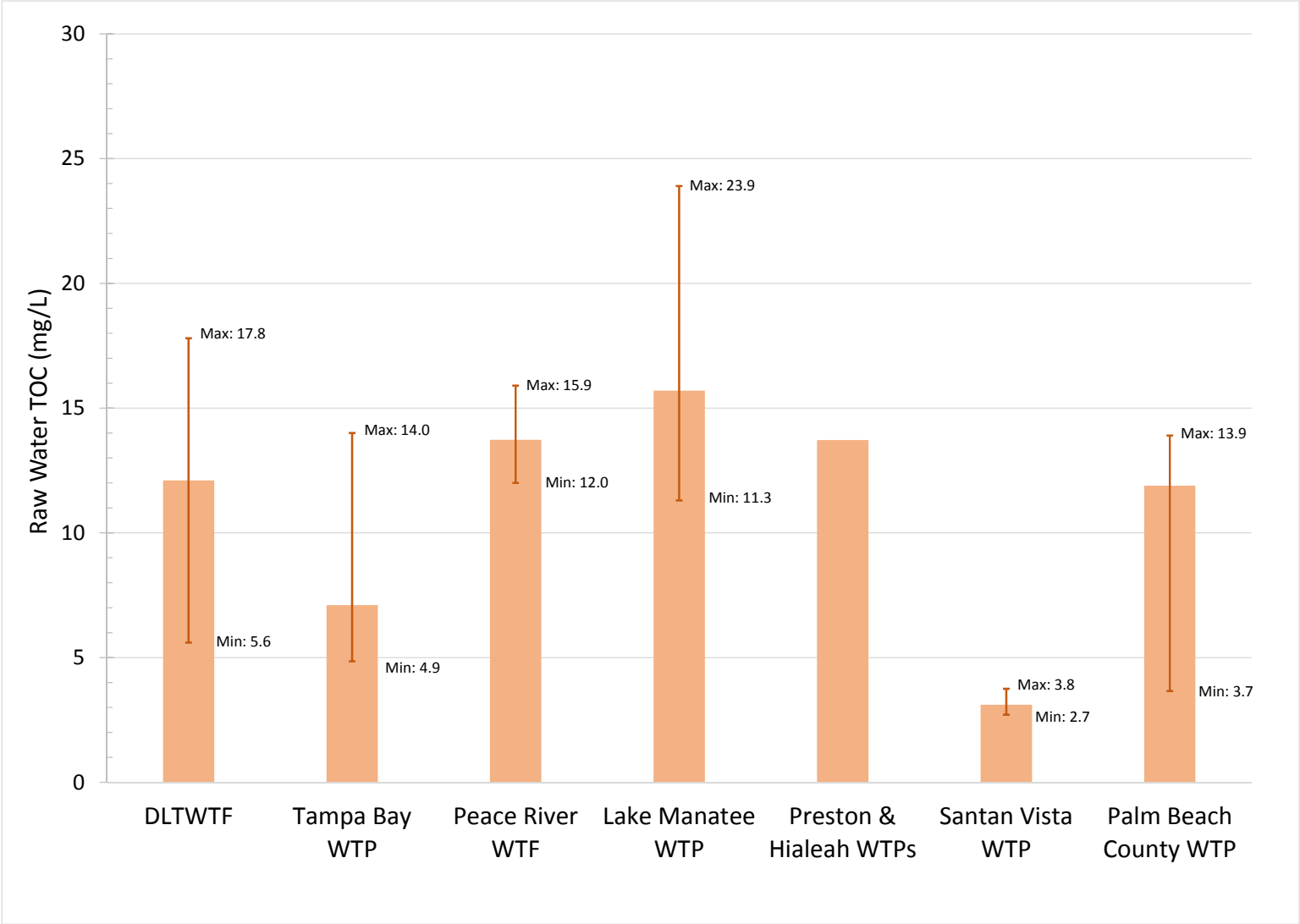
6.4.1 TOC

Given the DLTWTF raw water characteristics, TOC was identified as a primary water quality parameter in the study. The benchmarked plants supplied data for their raw and finished water TOC concentrations and for settled water or pre-filtered water, where available.

However, the benchmarked plants reported that they do not target an absolute TOC level in the finished water. Rather, these plants remove TOC based on the regulatory requirements that are in turn based on source water TOC and alkalinity requirements.

Figure 6.1 shows the average raw water TOC for each plant and the annual maximum and minimum. TOC levels are comparable among the five plants located in Florida: DLTWTF, Peace River, Palm Beach County, and Preston & Hialeah. Santan Vista in Arizona demonstrates the lowest TOC.

Figure 6.2 shows for each plant the overall TOC removal (for example, from raw water to finished water), and, where available, the TOC removed by the pretreatment process (from the raw water to the settled or pre-filtered water).



RAW WATER TOC

FIGURE 6.1

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

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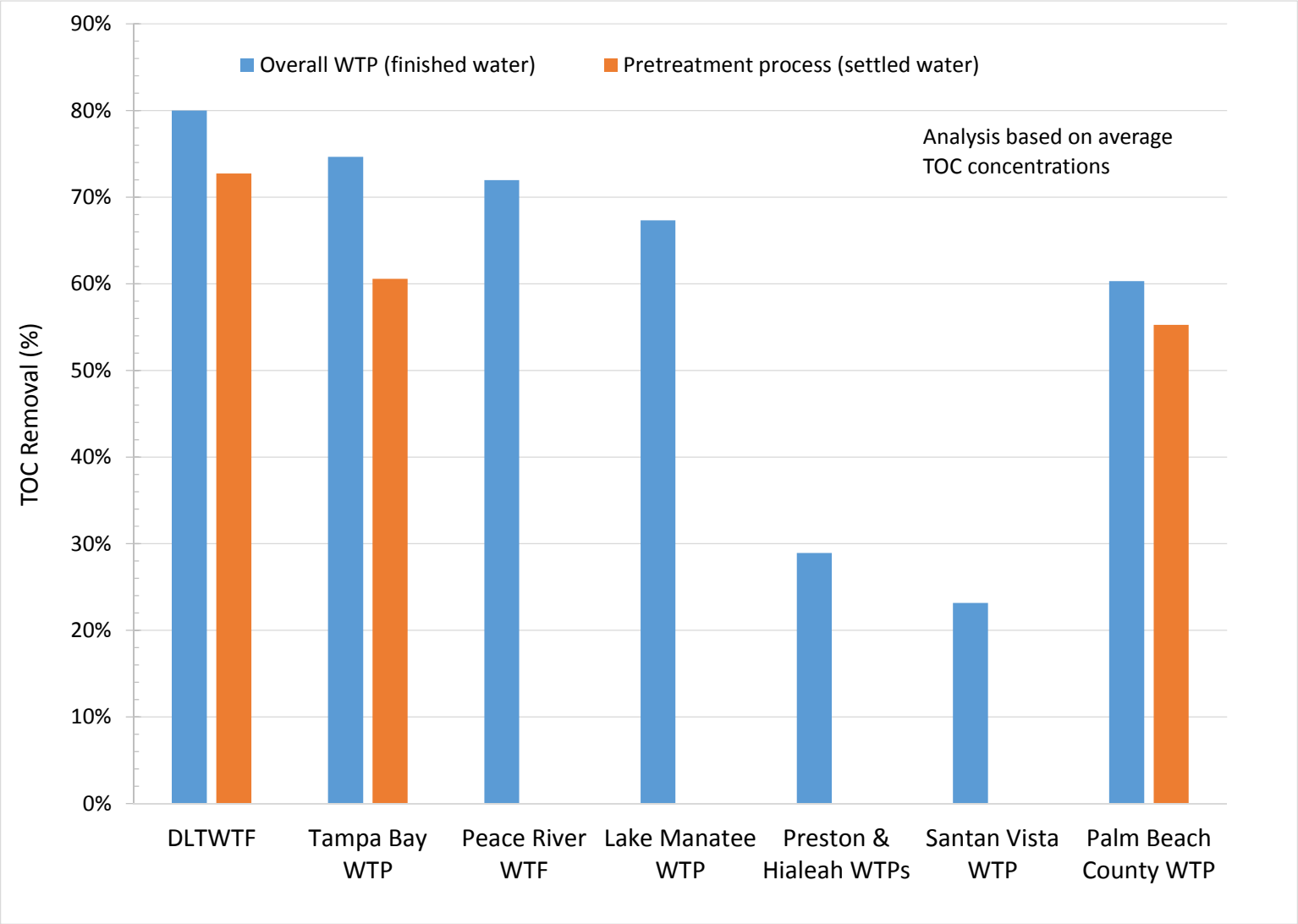

TOC REMOVAL

FIGURE 6.2

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

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The DLTWTF demonstrates the highest relative TOC removal: 80 percent overall TOC removal versus 60 to 75 percent demonstrated by the other plants. Additionally, TOC removal by the pretreatment process is the highest for the DLTWTF compared to the other plants. Of particular interest in this graph is the difference between the DLTWTF and the Palm Beach County WTP, which receive comparable average influent TOC but show an approximately 20 percent difference in removal.

6.4.2 Color

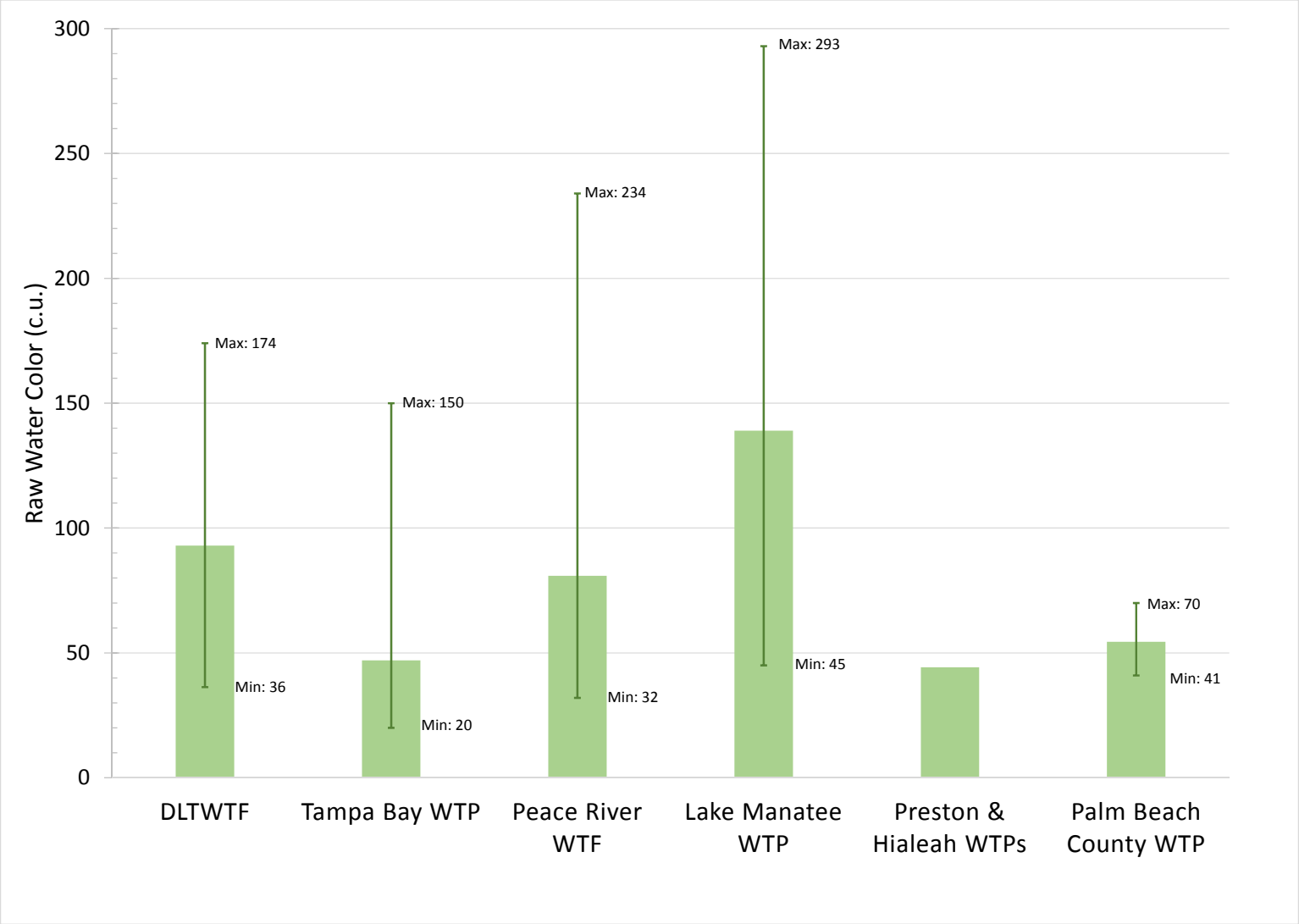
Figure 6.3 shows the average raw water color for each plant and the annual maximum and minimum. As with TOC, the Manatee County WTP showed both highest average (and range) of raw water color among the Florida plants, and the DLTWTF showed the second highest average raw water color. Color data were not available for the Santan Vista WTP. Figure 6.4 shows color removal in the benchmarked plants.

Except for the Preston-Hialeah system in Miami, which practices only lime softening, all the benchmarked plants showed greater than 90 percent color removal. The Tampa Bay Water WTP showed relatively higher color removal in the Actiflo™ system compared to the DLTWTF and Peace River WTF, both of which use conventional C/F/S for some or all of their treatment.

6.4.3 Turbidity

Since turbidity is primarily handled by filtration, pretreatment turbidity data were not considered for this comparison. Figure 6.5 shows the raw water turbidity for the benchmarked plants. As is apparent from the raw water data, significant variability exists in raw water turbidity for several of the Florida plants, particularly Lake Manatee WTP. While maximum and minimum data for the Preston & Hialeah WTPs were not available, the average raw water turbidity was significantly higher than the other plants. Due to the variety of raw water sources used by this system, raw water quality is typically quite variable. On the other hand, the Palm Beach County WTP No. 2 and Santan Vista WTP, have notably low raw water turbidity. Figure 6.6 shows the turbidity removal at each plant.

In terms of percent removal of raw water turbidity, most of the benchmarked plants are comparable, removing more than 90 percent. Although the Santan Vista WTP and Palm Beach County WTP No. 2 both fall short of this mark, they also had the lowest raw water turbidity of all the benchmarked plants. This lower percentage of removal is likely not indicative of any deficiencies in their filtration systems, but rather follows from their much lower influent turbidity values.



RAW WATER COLOR

FIGURE 6.3

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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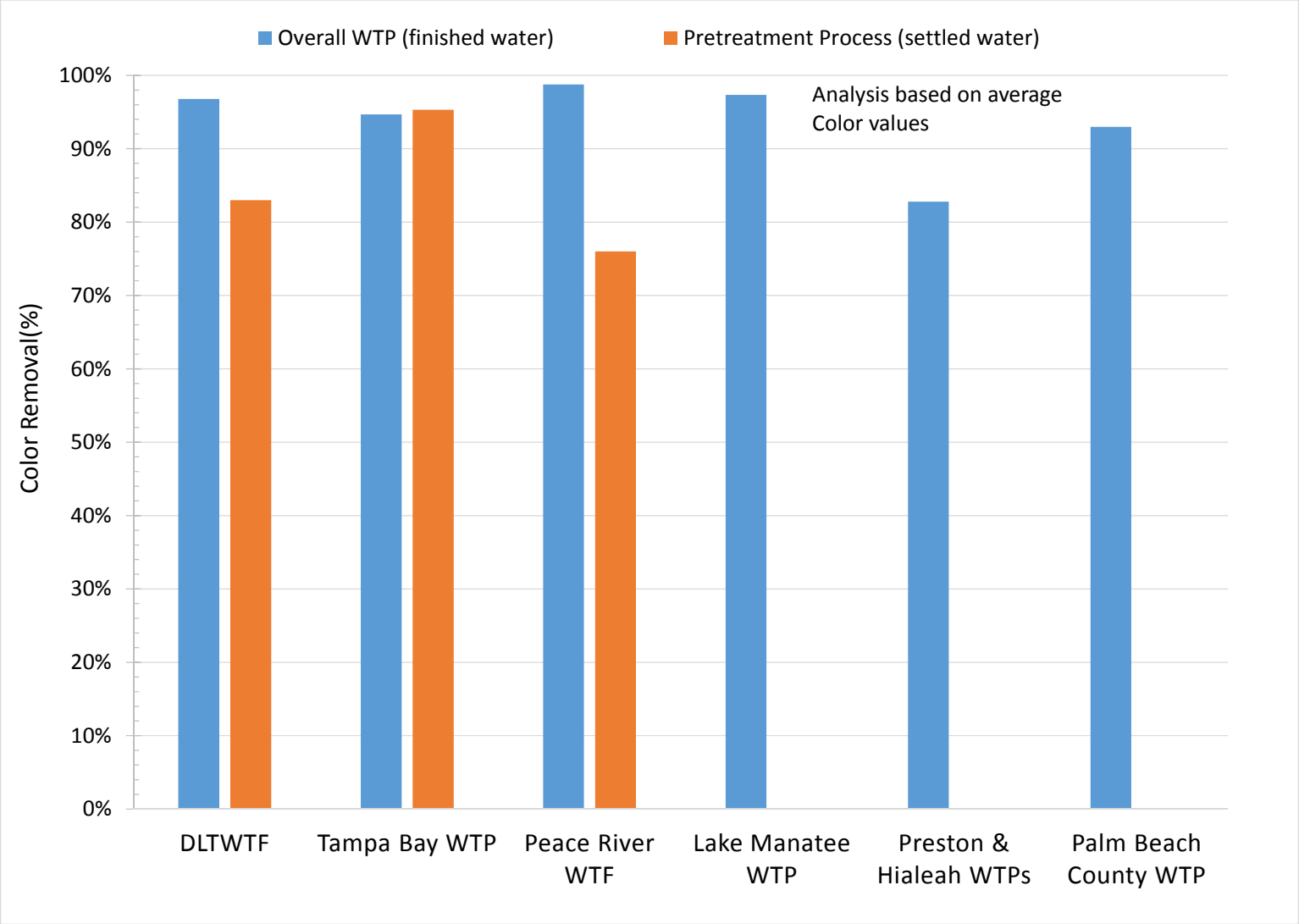

COLOR REMOVAL

FIGURE 6.4

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

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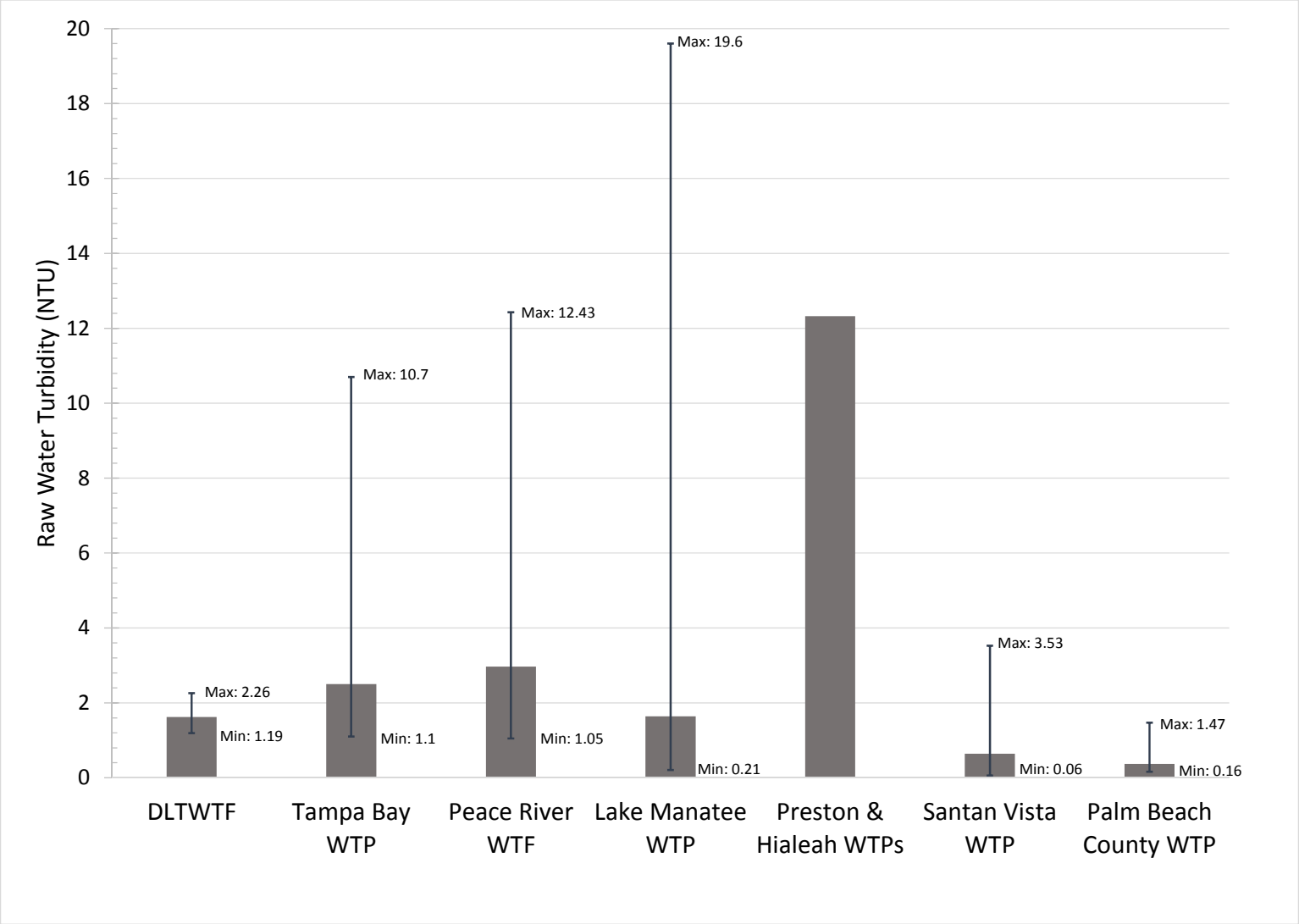

RAW WATER TURBIDITY

FIGURE 6.5

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

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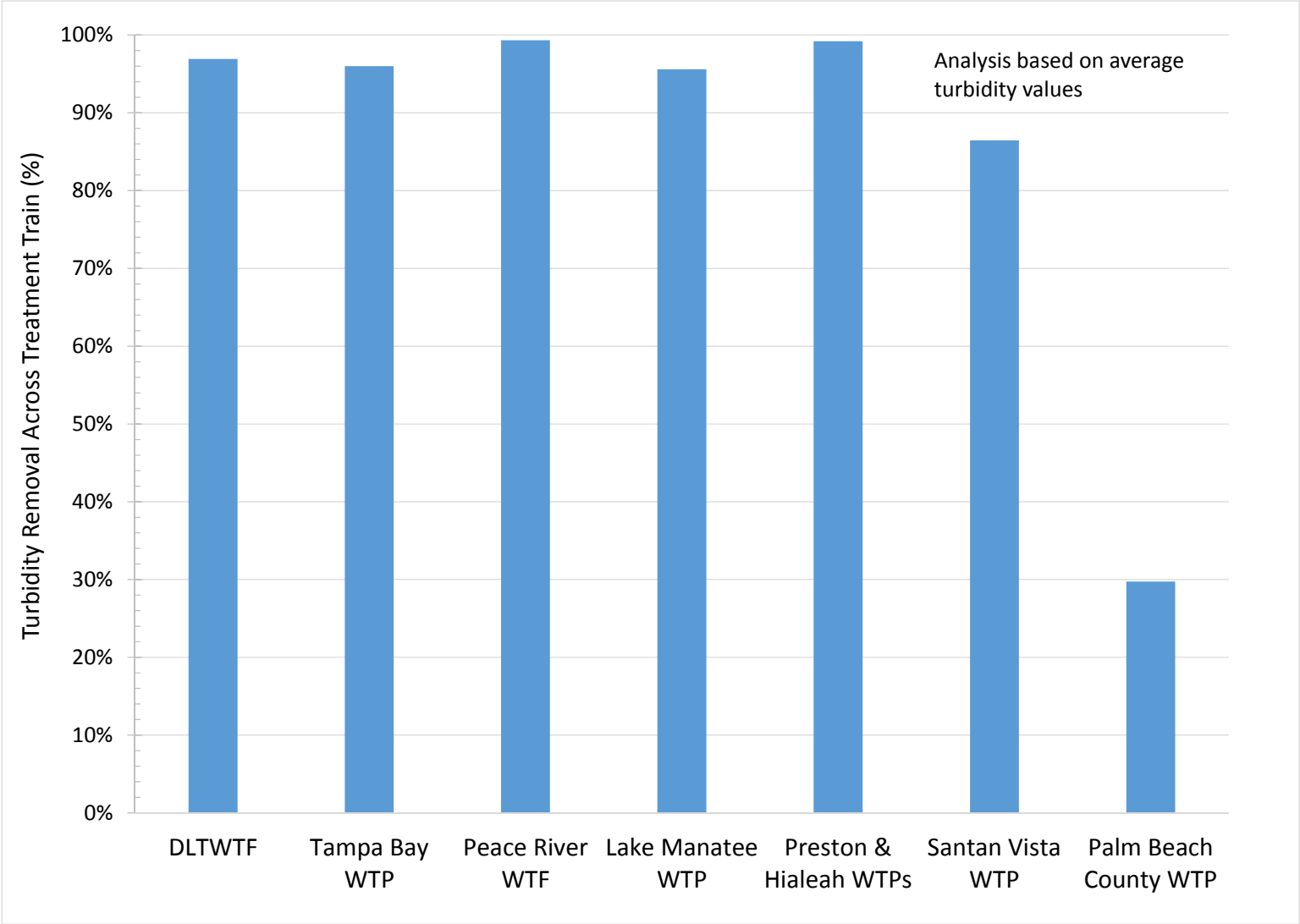

TURBIDITY REMOVAL

FIGURE 6.6

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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6.5 COMPARISON OF OPERATIONS COMPONENTS

Operational components were compared in terms of corresponding use and cost. To remove the discrepancy of scale among the plants, usage and costs were normalized to annual average production. Table 6.2 summarizes annual plant production (million gallons per year) and the corresponding annual average daily flow (AADF) for all the plants.

Table 6.2 Annual Total Production and Average Annual Daily Flow David L. Tippin Water Treatment Facility Master Plan City of Tampa

Cost Component	DLTWTF	Tampa Bay WTP	Peace River WTF	Lake Manatee WTP	Preston & Hialeah WTP	Santan Vista WTP	Palm Beach County WTP
Permitted Design Capacity (mgd)	120	120	51	54	235	24	17
Annual Average Daily Flow (mgd)	73.8	57.4	38.4	22.5	150.2	22.7	8.9
Annual Production (MG/yr)	26,025	21,018	14,005	8,220	54,838	8,289	3,238

6.5.1 Energy

Table 6.3 summarizes the total annual energy use and normalized energy use and cost for all plants. Figures 6.7 and 6.8 illustrate the variation of normalized energy use and cost. Energy use includes electrical power for all plants except the Preston & Hialeah WTPs, for which it also includes natural gas (see Section 3.1.5). In that case, natural gas use was converted to equivalent kilowatt hours and added to the energy use to calculate the total equivalent energy.

Energy use includes high service pumping. Some variation in normalized energy use is thus expected even with plants using similar treatment processes, because different plants target different discharge pressures with the high service pumping.

Normalized energy use for the DLTWTF is comparable to the Lake Manatee and Palm Beach County WTPs. The Tampa Bay Water WTP demonstrates the lowest relative energy use. The Preston and Hialeah WTPs exhibit the highest normalized energy use; kilns use the additional energy for lime processing (see Section 3.1.5).

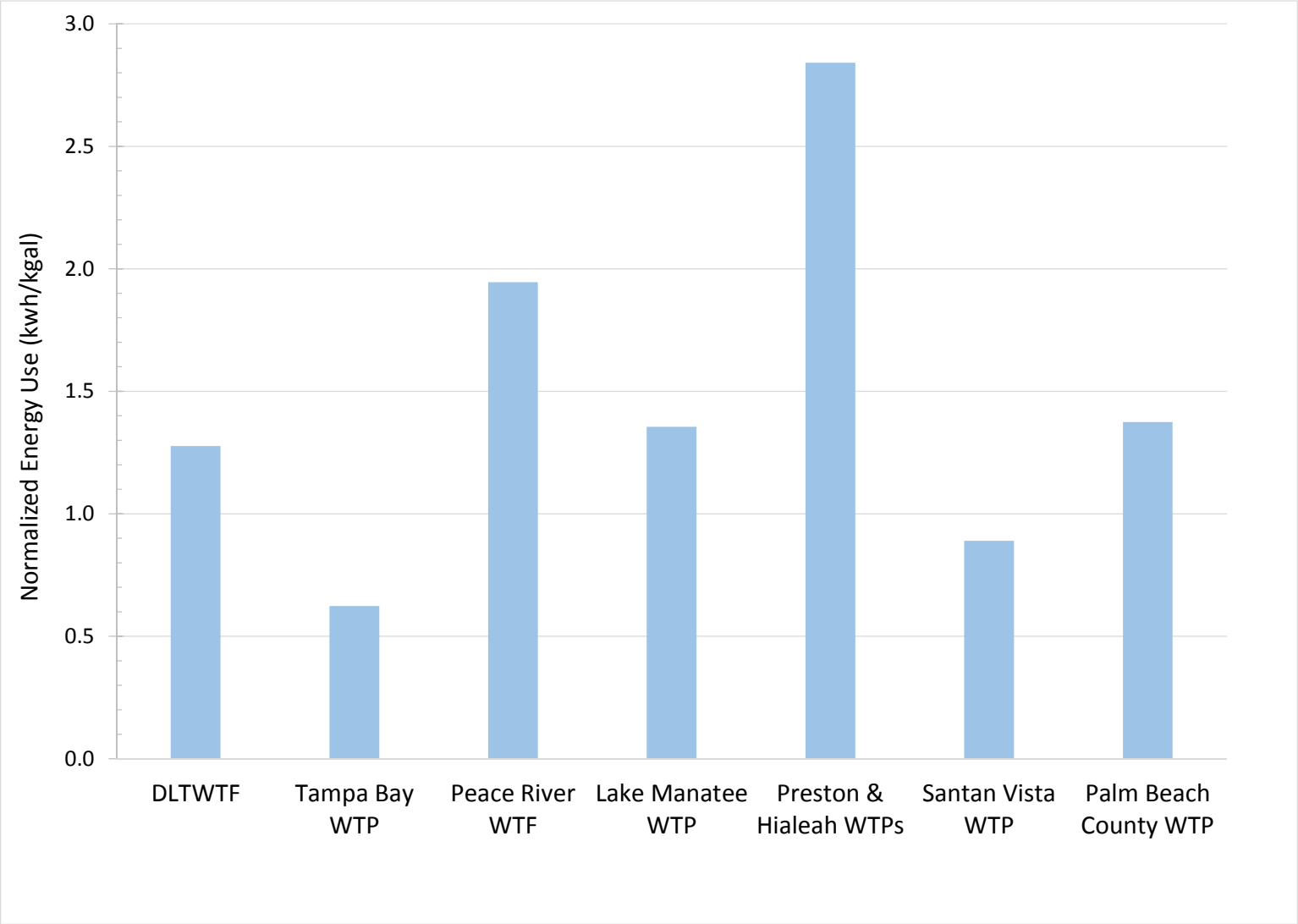
Table 6.3	Energy Use and Cost David L. Tippin Water Treatment Facility Master Plan City of Tampa						
Cost Component	**DLTWTF**	**Tampa Bay WTP**	**Peace River WTF**	**Lake Manatee WTP**	**Preston & Hialeah WTP**	**Santan Vista WTP**	**Palm Beach County WTP**
Energy Use (MWh/yr)	33,231	13,111	27,250	11,137	155,363	7,378	4,450
Normalized Energy Use (kwh/kgal)	1.3	0.6	1.9	1.4	2.8	0.9	1.4
Normalized Energy Cost ($/kgal)	0.099	0.048	0.146	0.081	0.203	0.069	0.102

6.5.2 Chemicals

Figure 6.9 shows the coagulant use normalized to production rate and amount of TOC removed across the treatment plant. The Palm Beach County WTP and Preston & Hialeah WTPs do not use a coagulant. Normalized coagulant use for Santan Vista appears much higher due to the relatively low TOC in the raw water, which in turn results in a low TOC removal requirement (Figure 6.1). The normalized coagulant use at the DLTWTF is comparable to other Florida plants (and slightly lower than all but one other plant) that use the coagulation process and are characterized by similar levels of raw water TOC (Figure 6.1). The Lake Manatee WTP demonstrates the lowest normalized coagulant use, while Tampa Bay Water WTP demonstrates the highest. Some variation in normalized coagulant use is expected due to the different raw water chemistries among the plants.

Figure 6.10 shows the total annual chemical and media cost normalized to production rate and the amount of TOC removed. In general, the trend is similar to the coagulant use data (Figure 6.9). Both Palm Beach County WTP scenarios, unadjusted and adjusted, (see Section 3.1.7) are also included. Chemicals/media cost for the DLTWTF are comparable and slightly lower than the adjusted Palm Beach County WTP scenario, which uses the MIEX process. Similar to the normalized coagulant use, the Lake Manatee WTP demonstrates the lowest normalized chemicals/media cost, while the Tampa Bay Water WTP demonstrates the highest among the Florida plants (not considering the unadjusted Palm Beach County WTP).

Appendix H is the detailed chemical cost breakdown for each plant normalized to the AADF ($/mgd). Appendix I shows a chemical cost breakdown for each plant expressed as a percent of total annual chemical cost.



ENERGY USE NORMALIZED TO PRODUCTION FLOW

FIGURE 6.7

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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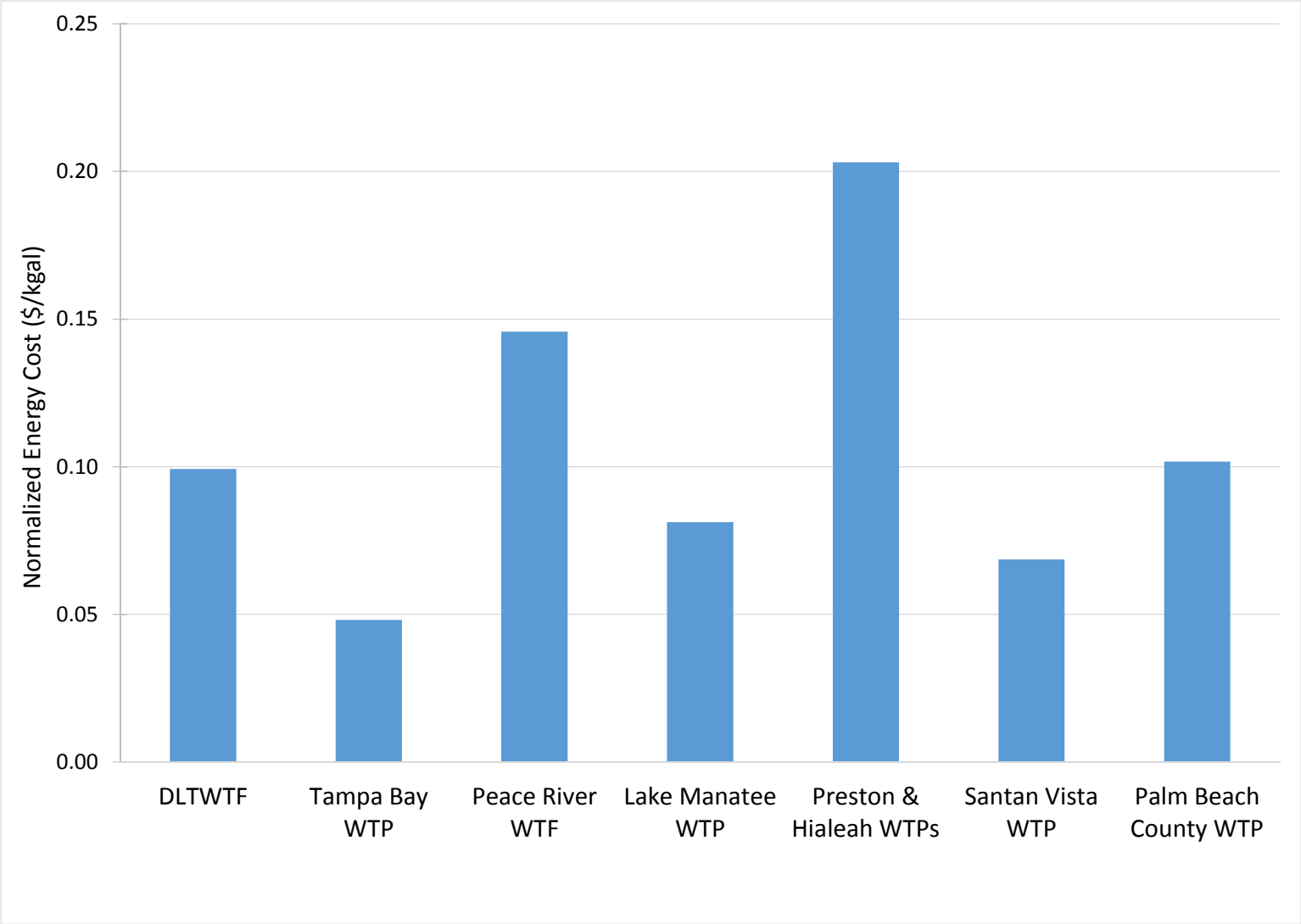

ENERGY COST NORMALIZED TO PRODUCTION FLOW

FIGURE 6.8

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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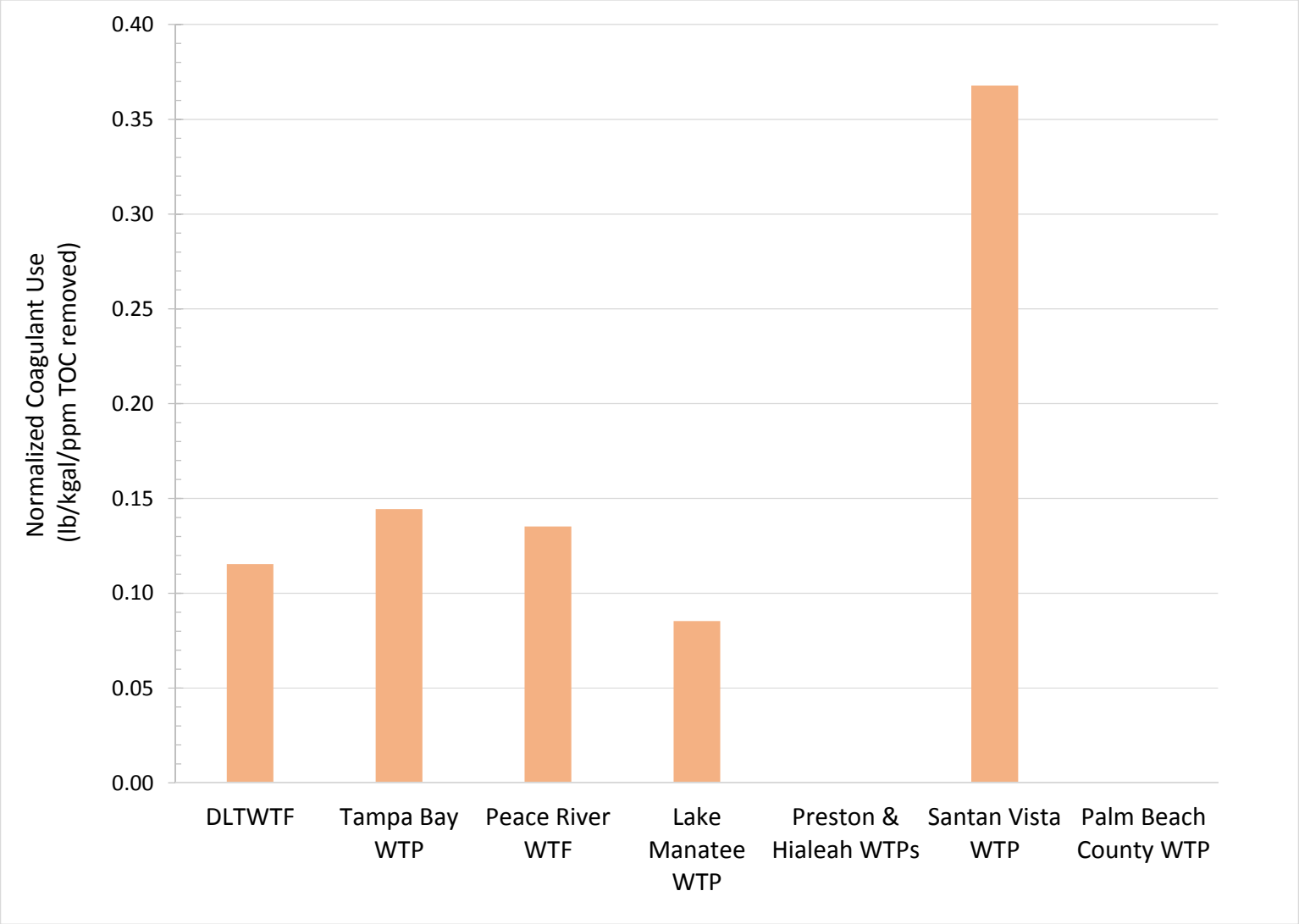

COAGULANT USE NORMALIZED TO PRODUCTION FLOW AND AMOUNT OF TOC REMOVED

FIGURE 6.9

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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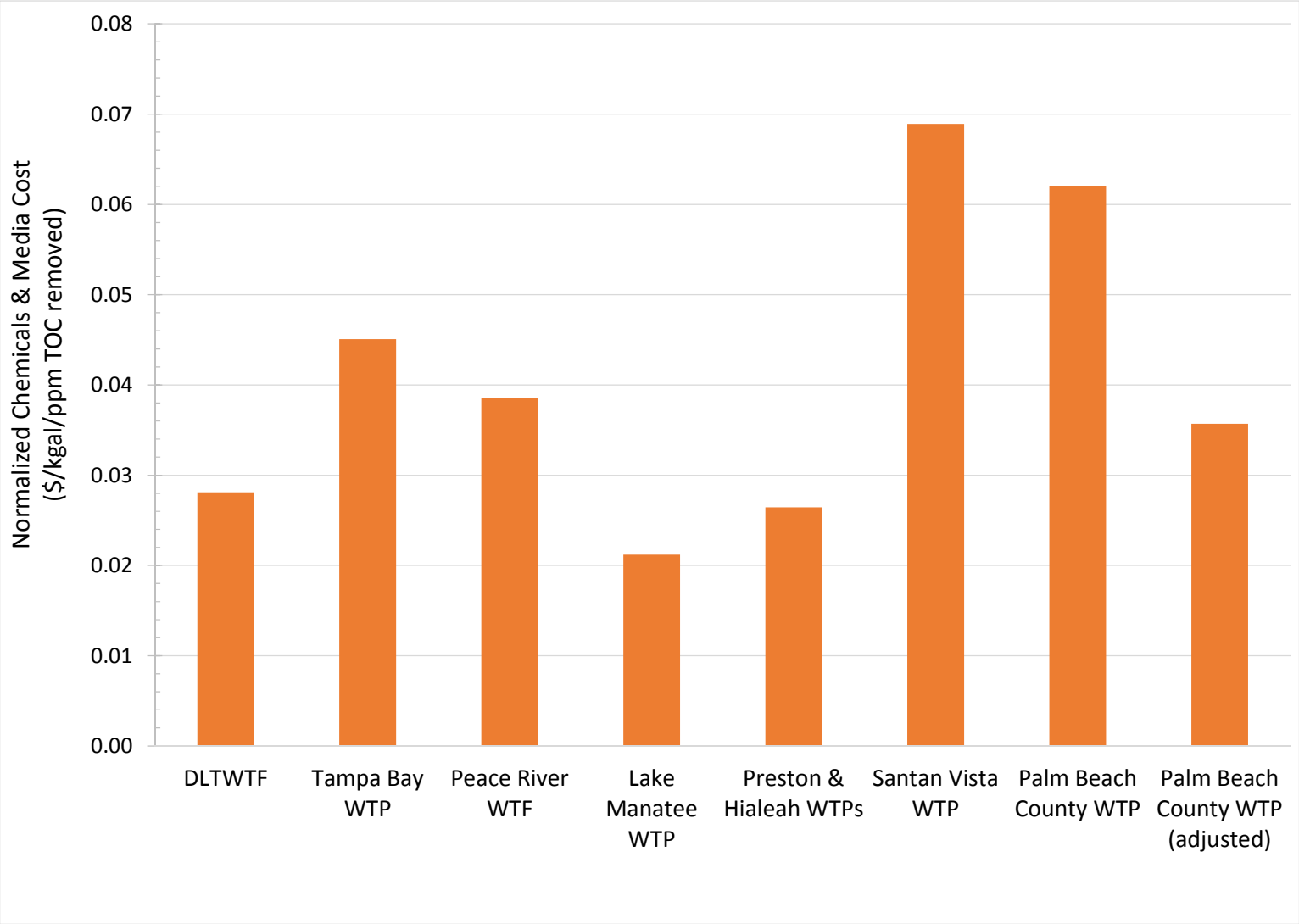

CHEMICALS AND MEDIA COST NORMALIZED TO PRODUCTION FLOW AND AMOUNT OF TOC REMOVED

FIGURE 6.10

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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6.5.3 Residuals Disposal

Figure 6.11 shows the residuals generated by each treatment plant, normalized to production rate and amount of TOC removed. The Palm Beach County WTP does not use a coagulant, and residuals generated were not available for the Santan Vista WTP.

The trend in residuals generated (or disposed) is consistent with the trends demonstrated in the normalized coagulant use (Figure 6.9) or normalized chemicals/media cost (Figure 6.10). Normalized residuals for the DLTWTF are comparable to both other Florida plants using C/F/S and to the Palm Beach County WTP (adjusted). Manatee County residuals are estimated, assuming a solids content of 2 percent. (This has not been measured by the plant.)

Figure 6.12 illustrates the annual residual disposal cost normalized to the production flow and amount of TOC removed.

6.5.4 Labor

Table 6.4 summarizes the number of people staffing each plant who work with water production, as both absolute numbers and normalized to capacity and production flows.

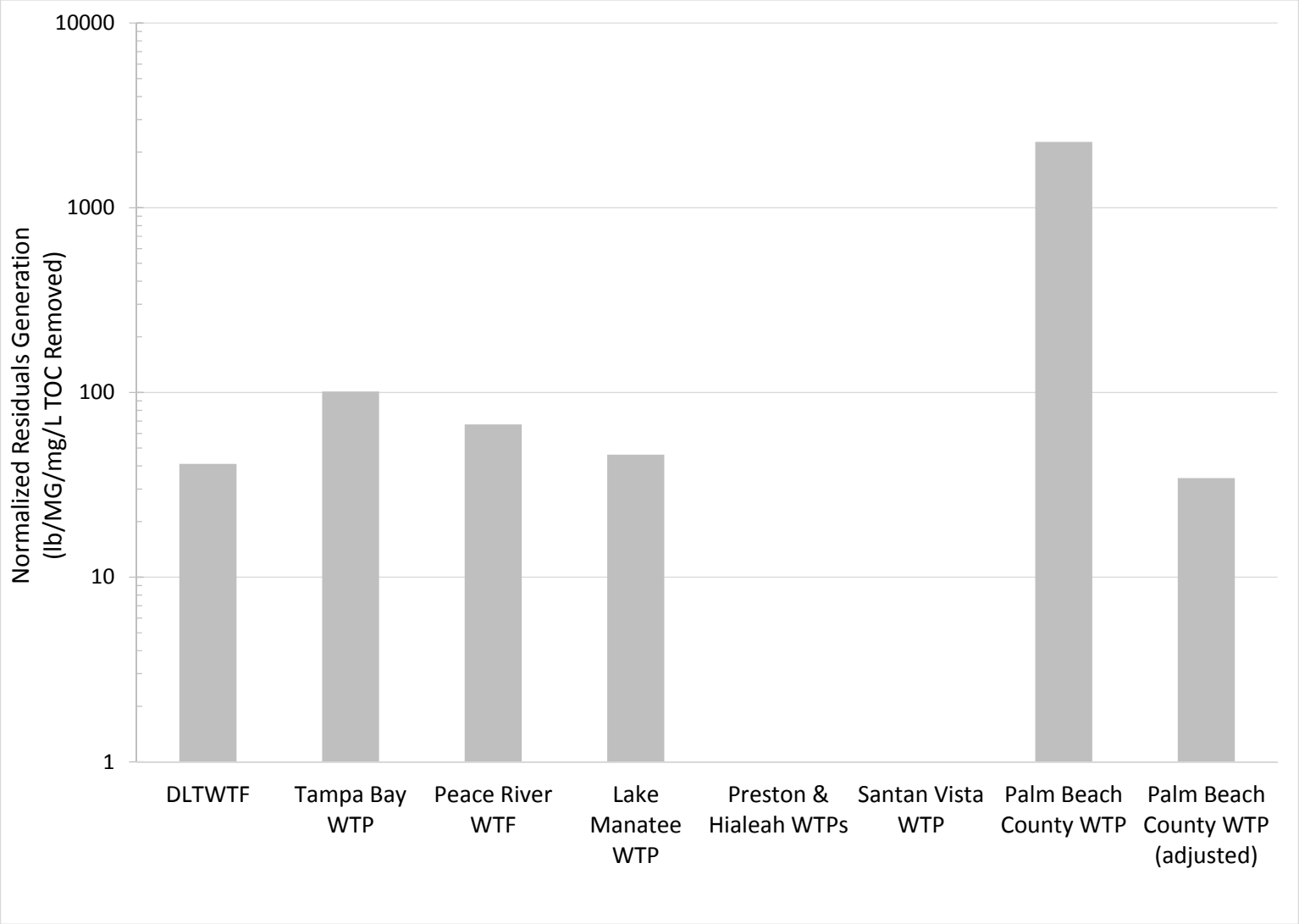
Even after normalization to capacity and production flows, the number of personnel among plants varies widely. This might be due to several factors: some plants use contract operations, while others share staff among multiple functions or divisions (such as production and distribution) and cannot easily determine the number of staff limited to production. Thus, despite clarifications with the plants, these inherent differences make it difficult to compare plant staff numbers.

Figure 6.13 shows labor costs normalized to the production flow.

Table 6.4 Labor Staff
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Cost Component	DLTWTF	Tampa Bay WTP	Peace River WTF	Lake Manatee WTP	Preston & Hialeah WTP	Santan Vista WTP	Palm Beach County WTP
Total O&M Staff	88	27	29	25	ND	11	18
Normalized Staff (#/MGD Capacity)	0.73	0.23	0.57	0.46	ND	0.46	1.10
Normalized Staff (#/MGD AADF)	1.19	0.47	0.76	1.11	ND	0.48	2.03

Notes:
ND = data not available.



RESIDUALS NORMALIZED TO PRODUCTION FLOW AND AMOUNT OF TOC REMOVED

FIGURE 6.11

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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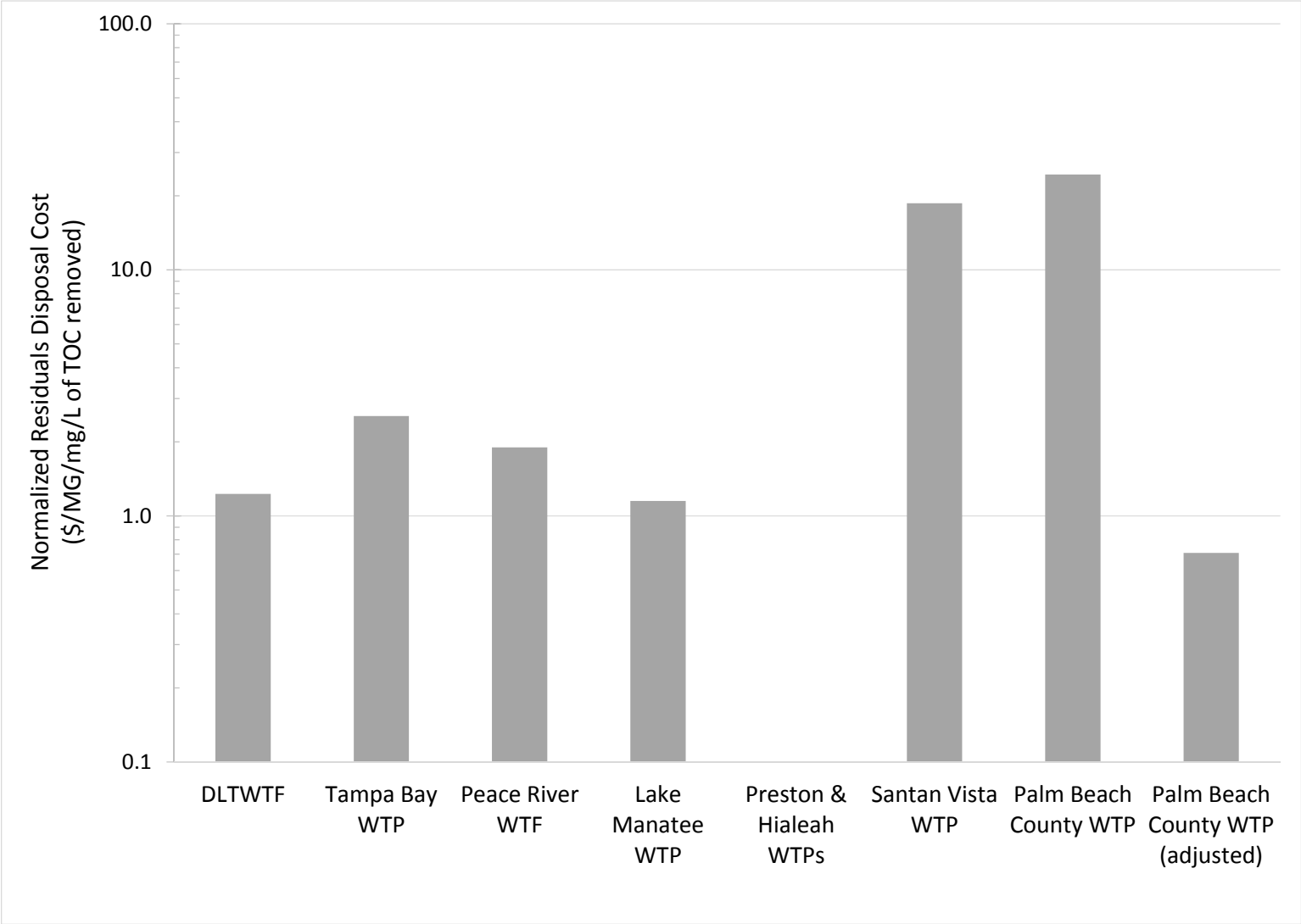

RESIDUALS DISPOSAL COST NORMALIZED TO PRODUCTION FLOW AND AMOUNT OF TOC REMOVED

FIGURE 6.12

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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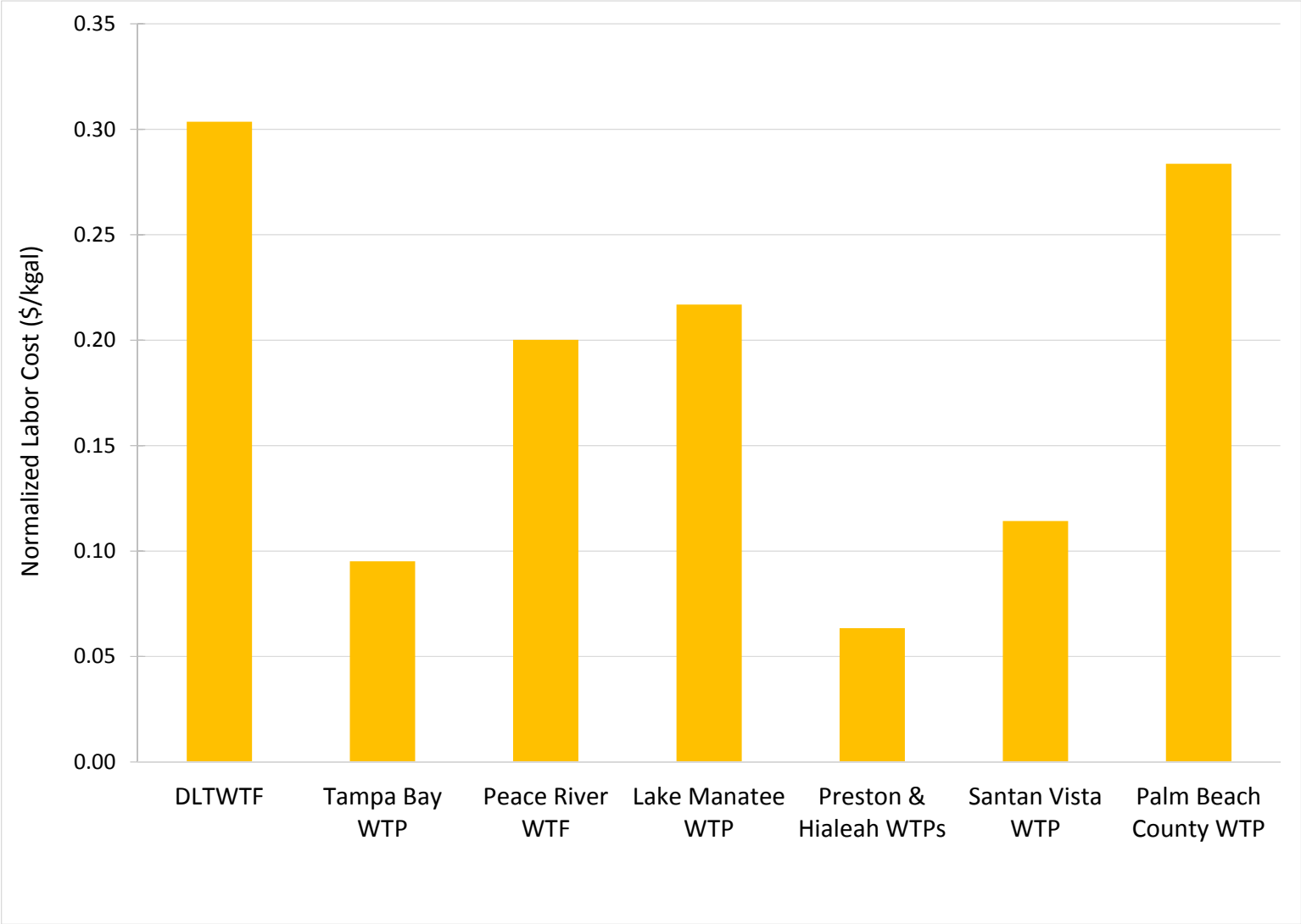

LABOR COST NORMALIZED TO PRODUCTION FLOW

FIGURE 6.13

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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6.5.5 Maintenance

Figure 6.14 shows maintenance costs normalized to production flow. This includes the costs for repairing or maintaining equipment and buildings, repair and replacement projects, and miscellaneous costs, such as inspecting and maintaining storage tanks and cleaning backwash ponds where pertinent.

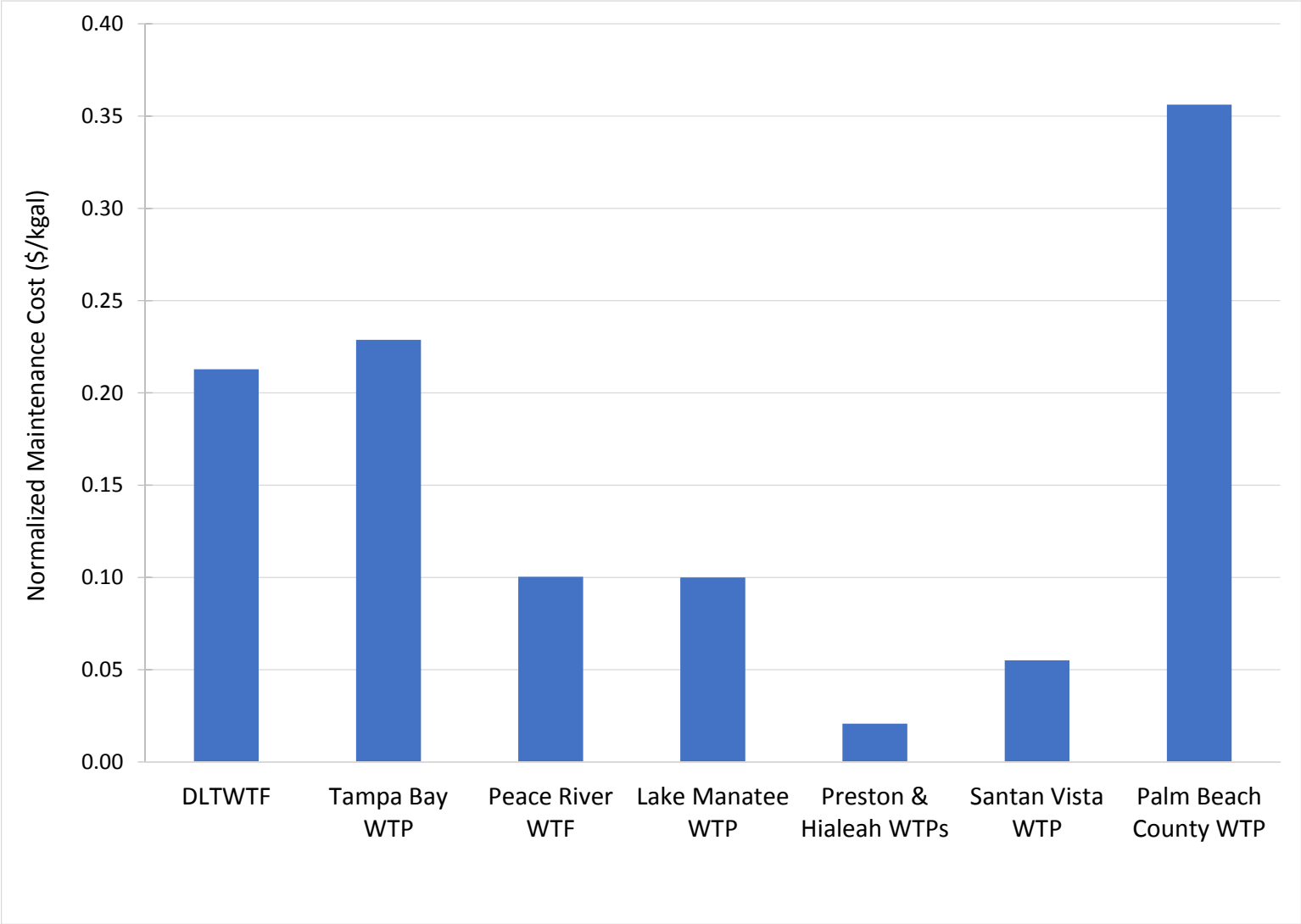
Normalized costs for maintenance for the DLTWTF are comparable to those for the Tampa Bay WTF, which is closest in location and capacity to DLTWTF. The Lake Manatee WTP and Peace River WTP are also closely located, similar in capacity, and demonstrate comparable costs. This implies that even after normalization to production capacity, factors such as location and design capacity influence maintenance costs.

6.5.6 Laboratory

Table 6.5 summarizes the laboratory costs at each plant, both as absolute costs and normalized to production flows. These costs are for laboratory supplies and contract laboratory services only. Plant laboratory staff is included under the labor category (see Section 5.4). Laboratory costs (exclusive of labor) as derived for the DLTWTF appear to be the highest; however plant staff needs to clarify that this derived cost accurately reflects DLTWTF laboratory costs.

Table 6.5 Laboratory Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Cost Component	DLTWTF	Tampa Bay WTP	Peace River WTF	Lake Manatee WTP	Preston & Hialeah WTP	Santan Vista WTP	Palm Beach County WTP
Annual Cost ($/yr)	1,100,000	61,546	170,000	18,711	83,908	46,000	47,287
Normalized Cost ($/kgal)	0.042	0.003	0.012	0.002	0.002	0.006	0.015



MAINTENANCE COST NORMALIZED TO PRODUCTION FLOW

FIGURE 6.14

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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6.5.7 Overall O&M Cost

Table 6.6 summarizes and Figure 6.15 illustrates the normalized overall O&M cost, including costs for all categories discussed in previous sections. The overall O&M cost for the DLTWTF is generally comparable to or slightly higher than the other plants using similar treatment processes and slightly lower than the overall cost for the Palm Beach County WTP (adjusted), which uses the MIEX process.

Note that the overall O&M cost is normalized only to production flow and not to the amount of TOC removed. As noted in previous sections, when certain components, such as chemical/media cost or residuals disposal cost, are also normalized to the amount of TOC removed, the cost component for the DLTWTF is comparable or slightly lower compared to the corresponding costs for other Florida plants using a similar treatment process.

Table 6.7 and Figure 6.16 illustrate the percent of each O&M cost component (normalized to production flow) across the different plants.

Table 6.6 Normalized Overall O&M Cost and Components
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Cost Component	DLTWTF	Tampa Bay WTP	Peace River WTF	Lake Manatee WTP	Preston & Hialeah WTP	Santan Vista WTP	Palm Beach County WTP	Palm Beach County WTP (adjusted)(1)
Energy ($/kgal)	0.099	0.048	0.146	0.081	0.203	0.069	0.102	0.102
Chemicals & Media ($/kgal)	0.272	0.239	0.381	0.224	0.105	0.050	0.445	0.256
Residual Mgt. ($/kgal)	0.012	0.014	0.019	0.012	ND	0.013	0.175	0.005
Labor ($/kgal)	0.304	0.095	0.200	0.217	0.063	0.114	0.284	0.284
Maintenance ($/kgal)	0.213	0.229	0.100	0.100	0.013	0.055	0.356	0.356
Laboratory ($/kgal)	0.042	0.003	0.012	0.002	0.000	0.006	0.015	0.015
TOTAL O&M ($/kgal)	**0.942**	**0.627**	**0.858**	**0.637**	**0.384**	**0.307**	**1.376**	**1.017**

Notes:

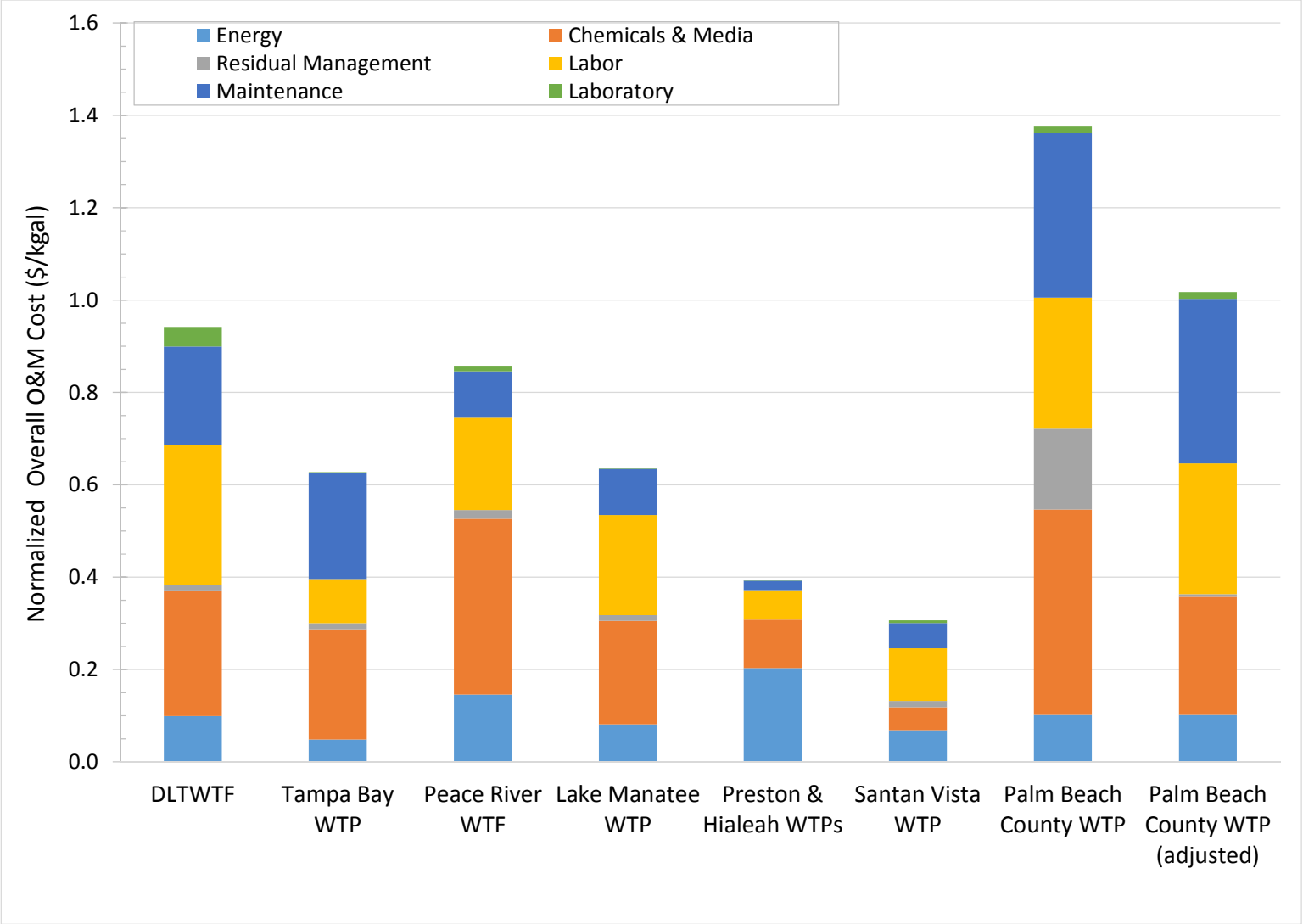
(1) See Section 3.1.7. In this scenario, an adjustment was made to the Palm Beach County WTP data to remove the use/cost of lime and associated residuals, while adding cost for 50 mg/L of ferric chloride and associated residuals to better compare with the DLTWTF and simulate one potential alternative being considered in this study.

Table 6.7 Percent of Overall Normalized O&M Cost and Components
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Cost Component	DLTWTF	Tampa Bay WTP	Peace River WTF	Lake Manatee WTP	Preston & Hialeah WTP	Santan Vista WTP	Palm Beach County WTP	Palm Beach County WTP (adjusted)(1)
Energy (%)	10.5%	7.7%	17.0%	12.8%	52.7%	22.4%	7.4%	10.0%
Chemicals & Media (%)	28.9%	38.1%	44.4%	35.2%	27.2%	16.2%	32.3%	25.2%
Residual Mgt. (%)	1.3%	2.2%	2.2%	1.9%	ND	4.4%	12.7%	0.5%
Labor (%)	32.2%	15.2%	23.3%	34.1%	16.5%	37.3%	20.6%	27.9%
Maintenance (%)	22.6%	36.5%	11.7%	15.7%	3.5%	18.0%	25.9%	35.0%
Laboratory (%)	4.5%	0.5%	1.4%	0.4%	0.0%	1.8%	1.1%	1.4%
TOTAL O&M (%)	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%

Notes:

(1) See Section 3.1.7. In this scenario, an adjustment was made to the Palm Beach County WTP data to remove the use/cost of lime and associated residuals, while adding cost for 50 mg/L of ferric chloride and associated residuals to better compare with the DLTWTF and simulate one potential alternative being considered in this study.



OVERALL O&M COST NORMALIZED TO PRODUCTION FLOW

FIGURE 6.15

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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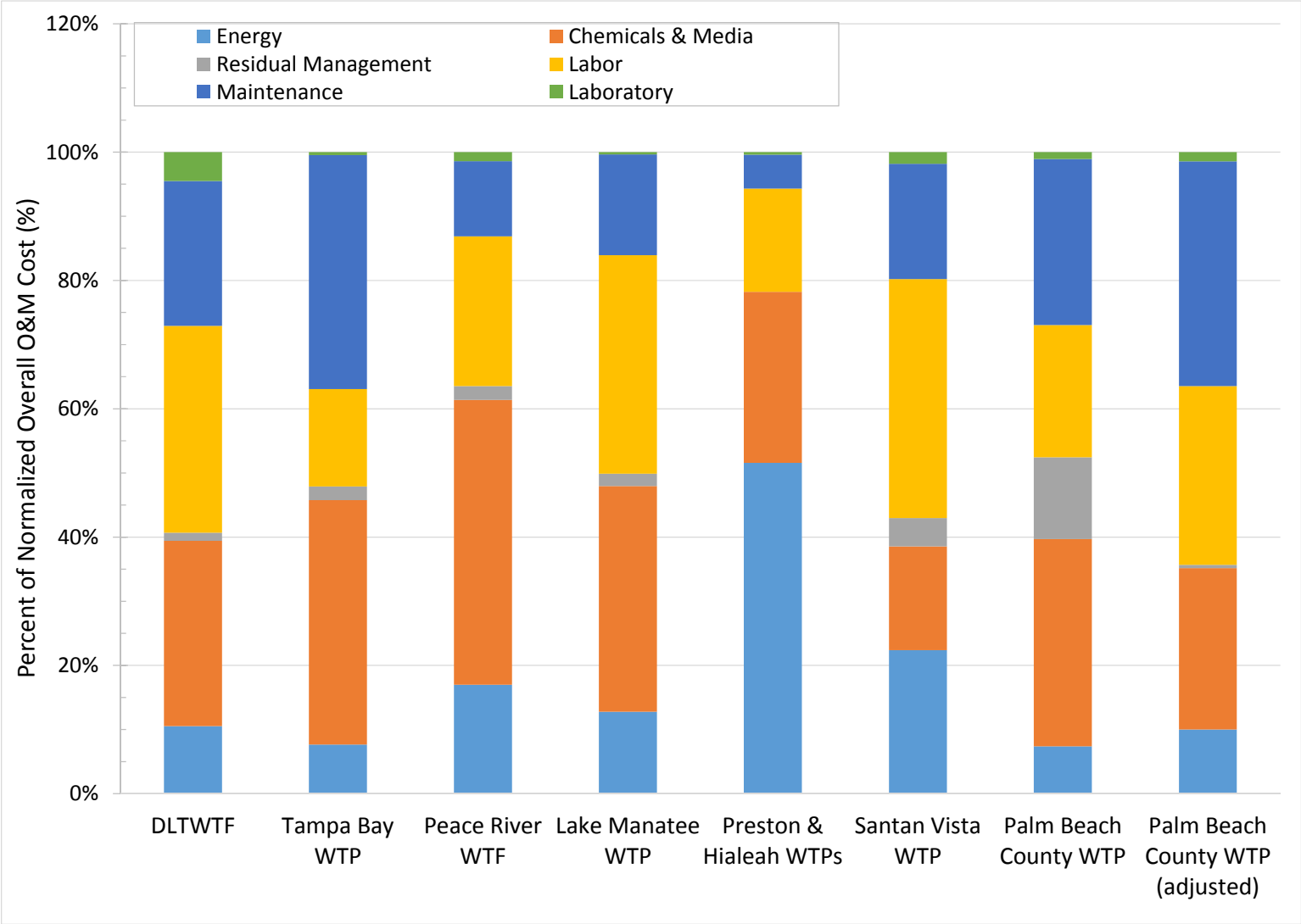

PERCENT OF NORMALIZED OVERALL O&M COST

FIGURE 6.16

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

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6.6 SUMMARY AND CONCLUSIONS

The benchmarking study compared operations and performance of the DLTWTF to utilities with plants of comparable size and similar complexity, process systems, and raw water quality.

In terms of water quality, DLTWTF is characterized by TOC, color, and turbidity levels comparable to levels of similar plants benchmarked in this study, especially the surface water treatment plants in Florida. Among these plants, the DLTWTF demonstrates the highest relative TOC removal (80 percent versus 60 to 75 percent) of the other plants.

However, benchmarked plants reported that they do not target an absolute TOC level in the finished water. Rather, they base TOC removal on the regulatory requirements, which are in turn based on source water TOC and alkalinity requirements. The DLTWTF's color and turbidity removals were comparable to those of the other benchmarked plants.

In terms of normalized energy, chemical use, and overall O&M cost, the usage and costs of the DLTWTF are generally comparable to the usage and costs of the plants benchmarked in this study, especially the surface water treatment plants in Florida. The DLTWTF overall O&M cost was also found to be comparable or slightly lower than the overall cost for the Palm Beach County WTP (adjusted), which uses the MIEX process.

In summary, it can be concluded that the City is indeed performing at the head of the pack, or nearly so, in every evaluated category, which is a positive testament to the experience and dedication of plant O&M staff. At the same time, the benchmarking effort did unveil some opportunities for optimization and for a close examination of alternative TOC/color removing technologies.

ALTERNATIVE DELIVERY METHODS

7.1 INTRODUCTION

This chapter details four delivery methods the City of Tampa (City) can use to execute design, construction, and commissioning of every project in the 15-year Capital Improvement Program (CIP). These delivery methods are design-bid-build (DBB), construction manager at risk (CMAR), design-build (D/B), and progressive design-build (PDB). The delivery methods vary by market sector, type of project, funding options, the City's operational and maintenance preferences, and other drivers as described herein. The proposed delivery methods for each CIP project and reasoning for each is included in Chapter 9.

7.2 PROJECT DELIVERY ANALYSIS

7.2.1 Common Drivers for Method Selection

Common factors that drive the alternative delivery method selection include cost, level of control, technical coordination, schedule, risk, contractual, and regulatory considerations. Each driver is described in subsequent sections.

7.2.1.1 Cost

Cost competitiveness is an important factor to consider for project delivery methods. The procurement method should produce a project cost similar to the costs of other delivery methods. Market and industry viability should also be evaluated.

The certainty of the project cost is also important. With some alternative delivery methods, a fixed project cost can be established early in the project to protect the owner from inflation and fluctuations in market conditions and budget. The procurement method selected should be evaluated for the cost certainty that can be determined early in the project's development.

7.2.1.2 Level of Control

The City's level of control over the project is also important, since it should allow the City to implement its preferred O&M methodology. Some delivery methods restrict the client's input on specific project components and how the City may have to operate and maintain after the project is completed.

The delivery method can also affect how much control the City has over the procurement process and the project's overall quality. This includes opportunities for innovation in design, construction, and project delivery.

Finally, the City's staffing resources should be considered. Some alternatives require more client involvement than others.

7.2.1.3 Technical Coordination

Some delivery methods will require the City or the City's in-house engineer to coordinate between the teams for the project completion. For example, a project requiring a proprietary equipment supplier will need coordination between that supplier, the design engineer, and the contractor.

7.2.1.4 Schedule

Understanding how each delivery method affects the project implementation schedule is critical. For example, some procurement methods may not readily allow for changes to the project if operational or construction improvements are identified. Furthermore, with some delivery methods, various project phases can overlap, shortening the overall project schedule.

7.2.1.5 Risk Allocation

Each delivery method allocates risk to different parties. It should be evaluated if the project's size and complexity provide an opportunity to realize the advantages associated with a specific procurement method. In addition, evaluation of the risk should determine if the procurement method will result in a project that will reduce or minimize impact to the public and if the risk is allocated in a manner acceptable to the City.

7.2.1.6 Contractual and Regulatory

Contractual drivers include contractual obligations to complete the project under a specific time frame, delivery method, etc. Regulatory drivers include regulatory bodies that require completing the project within specific constraints. The selected procurement method should be analyzed for its impact on the City's compliance with regulatory and permitting requirements imposed on the project.

7.3 ALTERNATIVE DELIVERY METHODS

7.3.1 Design-Bid-Build

Design-bid-build (DBB) is the conventional delivery method for most major construction projects in the water and wastewater industry in the United States. In a DBB delivered project, the owner (City) contracts with a design professional (engineer) who produces contract documents (drawings and specifications). Contractors then bid on the construction, and the lowest bidder is awarded the project and contracts with the City for delivery of the finished product.

7.3.1.1 Timeline and Schedule Considerations

The typical sequence for a DBB delivered project consists of design, bidding, and construction. These phases are distinct and sequential and do not overlap; the bidding phase does not begin until the design phase is complete, and the construction phase does not begin until the bidding phase is complete. After construction, the City operates the facility.

The first step in a DBB project is to retain the engineer. After that, the design phase begins, in which the facilities are designed, and contract documents, including drawings and specifications, are prepared for bidding. In the bidding phase, bids are tendered from contractors. The project is then constructed in compliance with the construction documents during the construction phase.

The working relationship between the contractor and the engineer is not established until the design is 100 percent complete and the contractor is selected. Therefore, constructability reviews, operability reviews, and value engineering are often incorporated into the design phase before construction.

Additionally, the contractor does not contribute to resolve constructability issues with the design during the design phase. Changes that must be made to construct the design after the design is complete can affect the schedule.

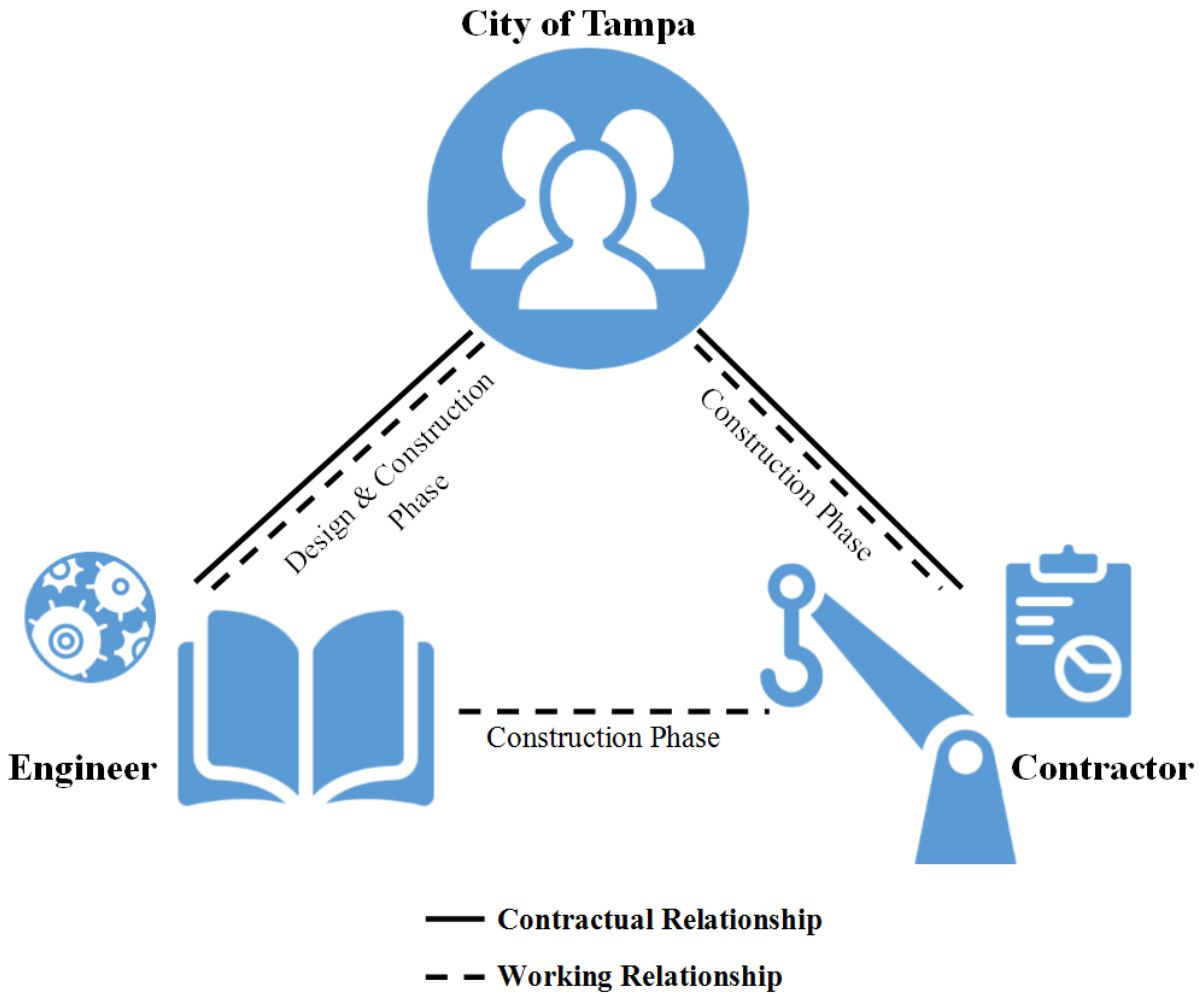
7.3.1.2 Teams

Retaining an engineer is typically done through a qualifications-based selection process. First, the City issues a request for qualifications (RFQ) or request for proposals (RFP), and the engineering team with best qualifications is awarded the contract. Qualifications can include experience with similar projects, project approach, references, team members' experience, and others.

The contractor is selected through a competitive bidding process. The contractor that responds with the lowest cost is selected without negotiations. The contractor's qualifications for the specific project are not considered.

Variations in the traditional DBB method exist to overcome some of the shortfalls. One option is to prequalify contracts by leaving project bidding open only to contractors the City has already prequalified, with previous experience on similar projects. Other options include prequalifying suppliers and preselecting or pre-purchasing equipment, which gives the City control to standardize equipment.

In a DBB delivered project, the City has two contractual relationships: one with the engineer and one with the contractor. While the engineer and the contractor maintain a working relationship, there is no contract between them, as Figure 7.1 shows.



CONTRACTUAL AND WORKING RELATIONSHIP FOR DBB PROJECT

FIGURE 7.1

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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The roles for each of these teams are as follows:

- City:
  - Issues the RFP for selection of an engineer and coordinates with the engineer during the design phase.
  - Tenders the bids from prospective contractors during the bidding phase.
  - After construction, owns and operates the completed facility
- Engineer:
  - Determines the facility requirements for the project and explicitly defines many of the project's risk elements.
  - Completes the project's preliminary and final engineering design.
  - Develops the contract documents, including drawings and specifications, for the City to bid competitively.
  - Oversees whether the builder complies with the contract documents and resolves any conflicts with the design's constructability during construction.
- Contractor:
  - Construction and delivery of the completed facility.
- Others:
  - The City or the City's representative can take on the responsibility for ensuring the builder's performance is in compliance with the contract documents and resolve any conflicts with the engineer's design.

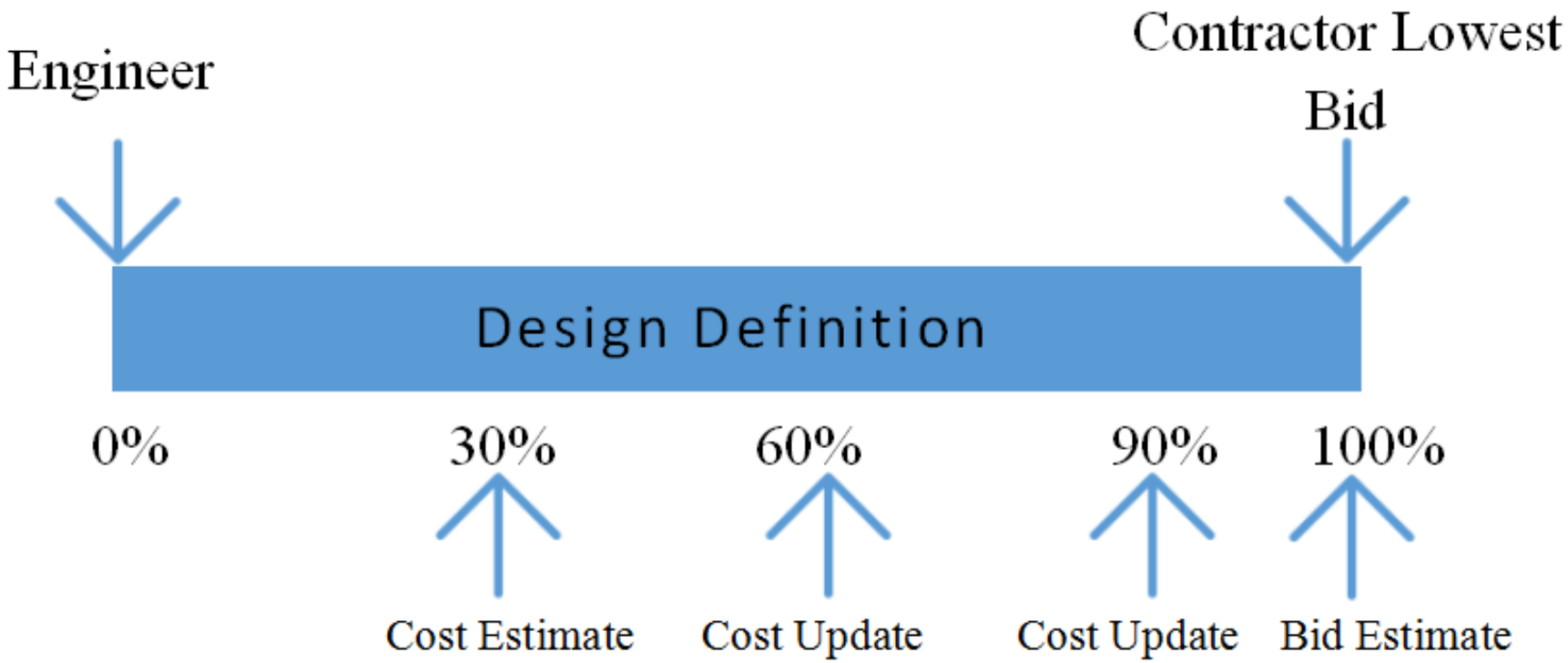
7.3.1.3 Risks

In a DBB project, the City assumes the majority of the risk for the project's design and constructability of the project. The City gives contract documents to the contractor and are not guaranteed to be without errors or omissions by the engineer.

Because they are separate entities, neither the engineer nor the contractor will guarantee the performance of the constructed facility. The engineer cannot guarantee the contractor's construction performance, and the contractor cannot guarantee that the design in the contract documents will perform as stated. Therefore, the liability remains with the City.

7.3.1.4 Cost Model

The pricing structure for a DBB project is either a fixed bid price or a lump sum. The engineer estimates a project cost at 30, 60, 90, and 100 percent completion of the design. However, the true cost of the project is determined by the lowest bidder after the design is complete and the bids have been tendered. Figure 7.2 shows a timeline of this process.



COST MODEL TIMELINE FOR DBB PROJECT

FIGURE 7.2

CITY OF TAMPA
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Similar to the schedule considerations for selecting a contractor after design, the lack of contractor's input during design can also affect cost. At this stage, contractors often charge a higher cost for changes because they affect the schedule, procurement, and construction of related elements.

7.3.1.5 Quality Considerations

Contractors are selected through a competitive bidding process that awards the contract to the lowest responsive bidder. Through this process, a general contractor can be awarded even though they are not qualified for the work, solely because they offered the lowest bid. Variations, however, such as prequalifying contractors, can help alleviate this issue.

7.3.1.6 Advantages and Disadvantages

Advantages to this delivery method are as follows:

- Typical method used and high familiarity with City.
- The City has a high level of control over design elements.
- The project scope is fully defined at the start of construction.
- Contractors and subcontractors can be prequalified.
- Equipment can be preselected and pre-purchased.
- It encourages a competitive bidding environment.

Disadvantages are as follows:

- The cost of the project is not established until the Contractor's bid is awarded.
- The Contractor is selected based on the lowest bidder if prequalification is not used.
- Competition decreases for contractors if prequalification is used.
- The Contractor does not have input during the design phase.
- The City assumes the majority of the risk for design and construction.

7.3.2 Construction Manager at Risk

This method overcomes two major disadvantages of traditional delivery methods: low-bid contractors and the uncertainty of project costs. In a CMAR project, the City contracts with an engineer and a Construction Manager/Contractor (CM) through qualification-based selection. Early in the process, the CM and the City negotiate a guaranteed maximum price (GMP) to which the CM is contractually bound.

The CMAR delivery method is best suited for projects that are schedule driven, are difficult to define, require critical construction input during the design phase, or where a fixed budget has to be met.

7.3.2.1 Timeline and Schedule Considerations

With this method, the design phase and construction phase may overlap. A CMAR project begins with the City retaining an engineer for the design services and selecting a CM shortly after. Since the CM is selected early in the design phase, they can provide input on constructability of the design while major decisions are still being made. This input is valuable in reducing delays during the construction phase caused by changes made in the design.

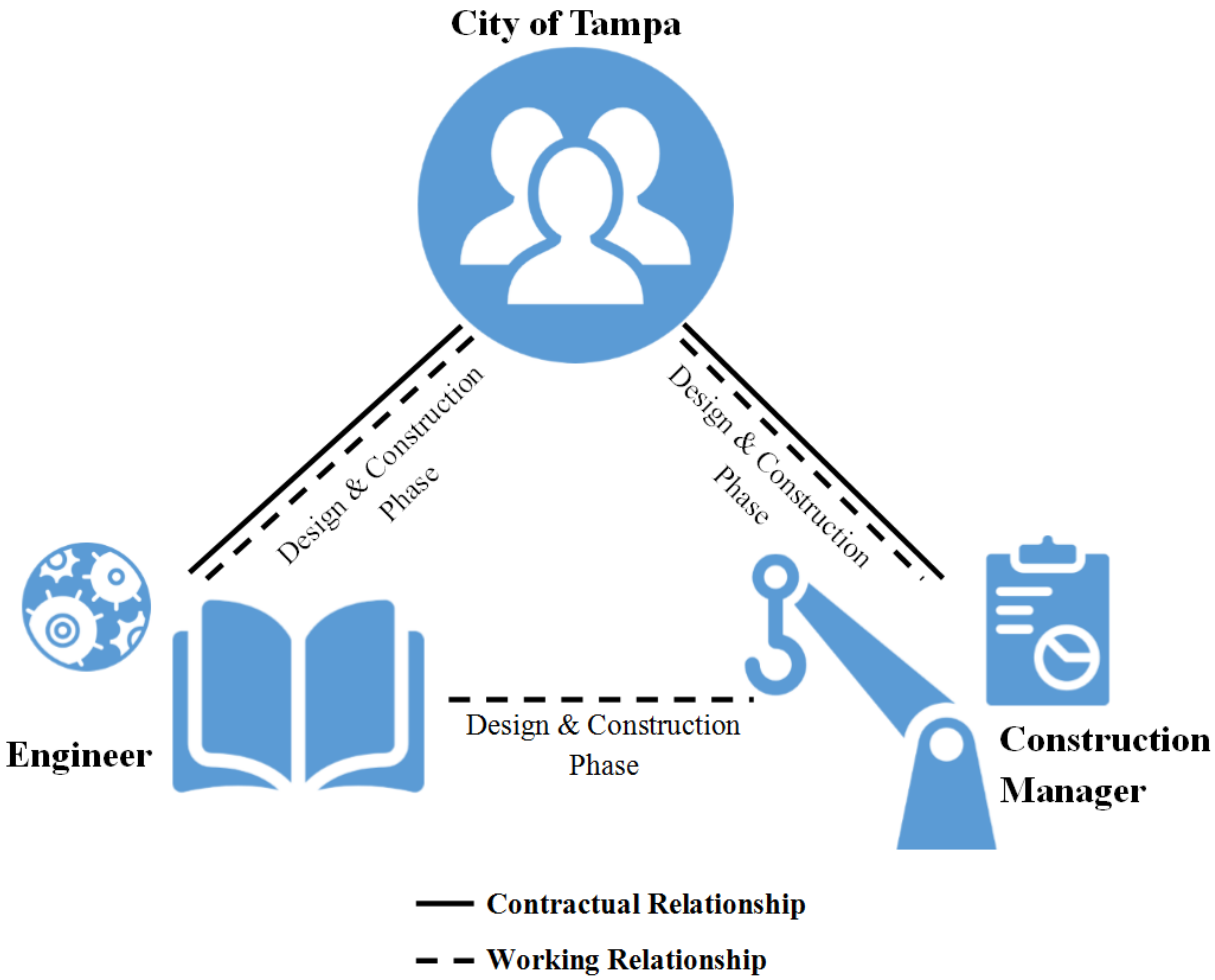
When the design phase reaches 75 to 100 percent complete, the CM negotiates the project's GMP with the City. If the City and CM cannot agree on a GMP, the City can have the engineer complete the design and proceed with the DBB procurement process. Subcontractors for construction are typically selected after the design phase is essentially complete. The final phase of the project is constructing the facility.

In a CMAR project, the bidding phase of the traditional delivery method is eliminated. This can shorten the project's duration and allow the construction phase to start before design is 100 percent complete.

7.3.2.2 Teams

As stated, the City typically selects the engineer and the CM through a qualifications-based selection process. The City starts by issuing an RFQ or RFP for design services and then awards the contract to the engineering team with the best qualifications. The same is done for construction management and general contractor services. Qualifications can include experience with similar projects, project approach, references, team members' experience, etc. In addition, prequalifying and preselecting discipline-specific subcontractors is typical during the later stages of the design phase.

As Figure 7.3 shows, a CMAR project has two major contracts with the City: one with the engineer and one with the CM. Since the engineer and CM are selected early in the project, the engineer, CM, and City work together to prepare the contract documents, which include drawings and specifications. Although the engineer and CM have no contractual relationship, they maintain a working relationship throughout the design phase.



CONTRACTUAL AND WORKING RELATIONSHIP FOR CMAR PROJECT

FIGURE 7.3

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The roles for each of these teams are as follows:

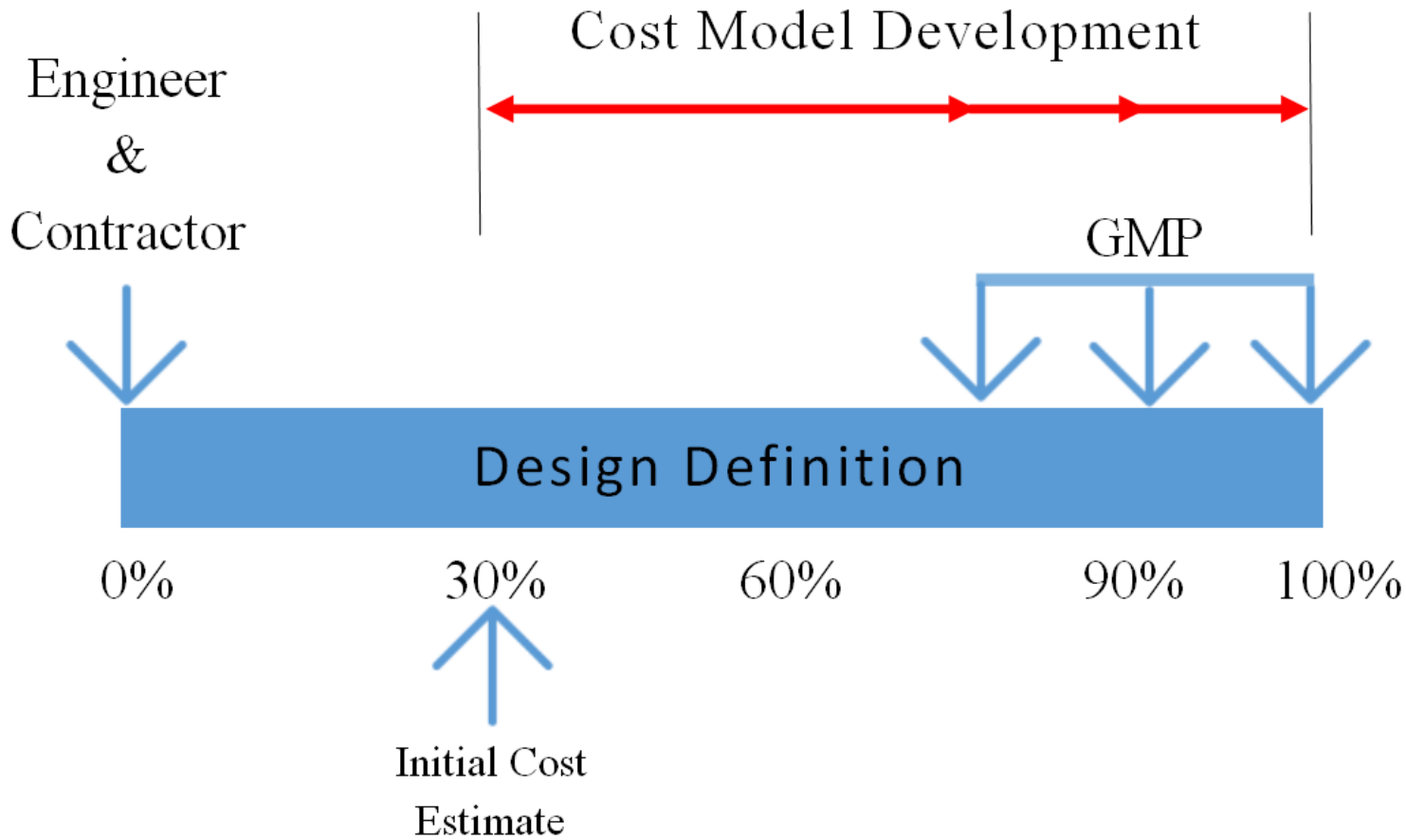
- City:
  - Issues the RFPs for selecting the engineer and CM early in the project.
  - Coordinates with the engineer and CM during the design phase by bringing operations expertise to project.
  - Owns and operates the completed facility after construction.
- Engineer:
  - Responsible for the design of the project
  - Provides the conceptual and detailed requirements for use in developing the GMP.
- Construction Manager/Contractor:
  - Delivers the work and coordinates the project's construction.
  - Helps provide design input and assistance to the engineer during the design phase by providing consultation in evaluating costs, schedule, implications of alternative designs and systems, and materials.
  - Performs portions of the work during construction.
  - Selects and manages qualified subcontractors for the remaining portions to complete the work.
- Others:
  - Subcontractors are typically selected near the end of the design phase and are not involved in the project's design phase.

7.3.2.3 Risks

The CM is designated at-risk early in the project to deliver the work within a GMP. The CM and the City establish the GMP, and the CM is contractually bound to that price. The CM then assumes the risk of construction after establishing the GMP. However, because the CM and engineer are separate entities, the City remains liable for the design and provides the design to the CM with the risk of design errors.

7.3.2.4 Cost Model

A CMAR's pricing structure is a negotiated GMP, often established when the design is between 75 to 100 percent complete, as shown in Figure 7.4. Once the GMP is established, any changes to the project scope may affect the GMP. This helps the project meet a fixed budget without waiting for the bidding phase to determine the project cost.



COST MODEL TIMELINE FOR CMAR PROJECT

FIGURE 7.4

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As with the schedule considerations, input from the CM during the design phase on constructability of the design while design decisions are still being made helps reduce costs. Increased costs caused by design changes in the construction phase can be reduced or eliminated.

7.3.2.5 Quality Considerations

A project in which both the engineer and contractor are selected based on qualifications should be of superior quality. Both teams are qualified to perform the work for this specific project and bring experience to deliver a quality project. In addition, selection of the CM early in the design phase allows for reduction of constructability issues and more "ownership" in the project from all teams.

7.3.2.6 Advantages and Disadvantages

Advantages of this method are as follows:

- CM is involved early in the design phase to allow for collaborative relationships and teamwork.
- CM has input in the project's design phase.
- The cost is established earlier in the project.
- The schedule could be shorted with no bidding phase.
- The project can be designed and delivered on a fixed budget.

Disadvantages of the method are as follows:

- The CM's involvement during design does not relieve the City of risk for design errors.
- The project cost could be higher than DBB or D/B, due to lack of competitive pressure on the CM after selection.

7.3.3 Design-Build

In recent years, the D/B delivery method has become more popular to overcome traditional project delivery methods' constraints of speed, quality, and cost. In the D/B method, the D/B team receives a description of the desired end product or project outcome. The team then develops detailed design and specifications, selects material and equipment, constructs the facility, and meets the performance criteria.

The focus is on measurable performance criteria or objectives for operation rather than on specific design approaches to achieve those objectives. D/B projects are best suited when the City has a clear vision of how the facility must perform and is open to the specific method of achieving the required performance.

7.3.3.1 Timeline and Schedule Considerations

The City issues an RFQ or RFP containing standard construction specifications to establish minimum quality standards. Based on these specifications and standards, the cost of the proposed facility is established via a lump sum proposal provided by the D/B team. The D/B team with the proposal of greatest value to the City is selected and proceeds with design and construction.

The RFP process can take longer with this method, but it is generally reduced if the RFP contains only performance criteria. In addition, because the engineer and contractor are on the same team, communications are improved during the design phase, allowing for procurement and construction to begin before the design is complete. A completion date can also be established early in the process because the contractor and designer can communicate during the conceptual development stages.

7.3.3.2 Teams

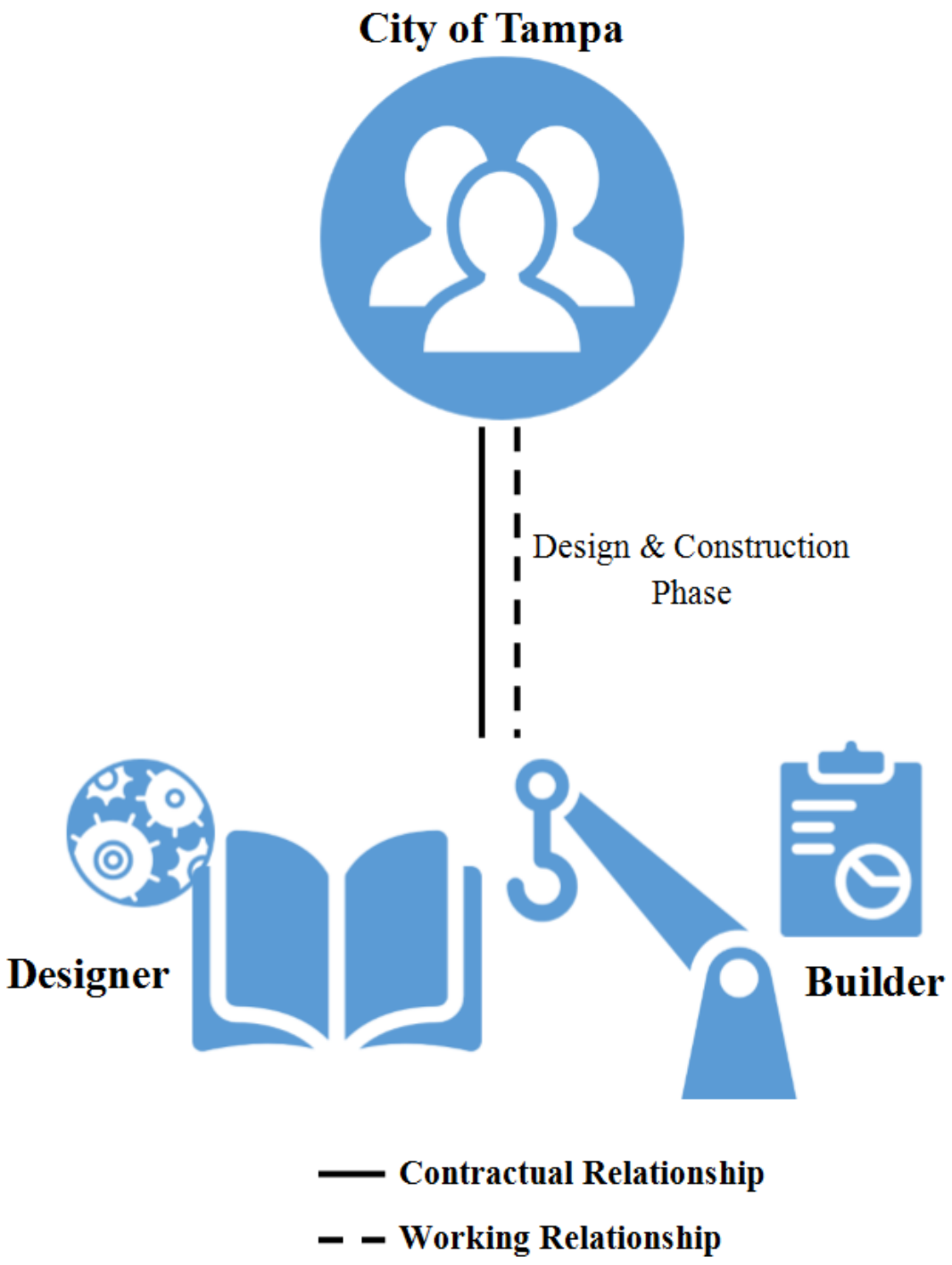
The D/B team is selected by the City through development of performance requirements for the team that offers the best value to the City. The value is based on qualifications, technical and business merit, and/or project costs. The contract is then negotiated based on the formal submitted proposal. The RFP may or may not include design drawings.

Variations of the D/B method differ in the degree of project design given to the D/B team during the RFQ or RFP process. These variations include performance-based D/B, prescriptive-based D/B, and progressive D/B. The prescriptive-based D/B differs in that the RFP is issued with 20 to 30 percent preliminary design drawings, giving a specific design approach the D/B team must follow. Progressive D/B project is discussed in subsequent sections.

In a D/B delivered project, the City contracts with a single entity to perform the design and construction services. Figure 7.5 shows the contractual and working relationship between the City and the D/B team.

The roles for each of these teams are as follows:

- City:
  - Issues an RFP for a D/B team and selects the team offering the greatest value.
  - Contracts with that single entity to design and construct the project, but has limited input during design.
- D/B Team (Engineer/Contractor):
  - Responsible for the delivery and performance of the project.
  - Works together to produce the design and to construct the final product.



CONTRACTUAL AND WORKING RELATIONSHIP FOR D/B PROJECT

FIGURE 7.5

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- Others:
  - Subcontractors contract directly with the D/B team. The City may have independent technical, legal, and/or financial consultants to serve as agents in managing the procurement process, establishing performance criteria, and monitoring performance. A conceptual to preliminary design (10 to 30 percent) may be prepared with the City's direction by a separate design professional to detail the prescriptive and performance requirements through an RFQ or RFP process.

7.3.3.3 Risks

The design and construction liability remain with the D/B team, as does the design-related performance risk. Since the D/B team combines the design and construction liability into one entity, they are responsible for the overall project and can warrant successful performance.

A detailed RFP and contract document should give the City their desired project. Under a prescriptive D/B method, the City would retain some of the design risk because they provide the preliminary design in the RFP. Also, the financial and schedule impacts of change orders for uncontrollable circumstances during construction rest with the City.

7.3.3.4 Cost Model

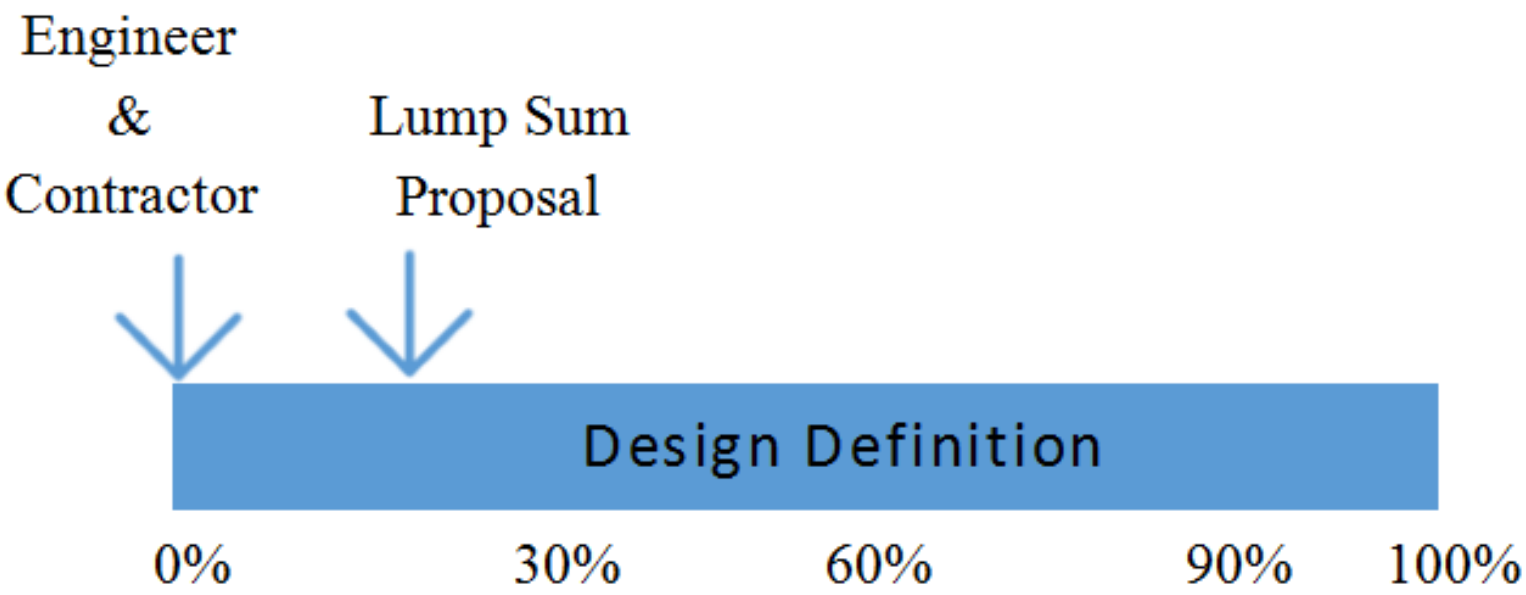
The pricing structure for a D/B project is a fixed bid price or a lump sum. The D/B team establishes the lump sum cost in their proposal at the beginning of the project, as Figure 7.6 shows. The team is bound to this cost early in the process, and it cannot be changed since the engineer and contractor are on the same team and coordinate throughout the process.

The D/B method's competitive procurement setting offers flexibility in developing innovative and cost-saving designs. In addition, the cost of developing an RFP during pre-design is reduced, since only performance criteria are issued. However, the D/B teams can encounter less competition because of the cost and risk of preparing a proposal for a D/B project.

7.3.3.5 Quality Considerations

The quality of a D/B project should be superior because the D/B team is selected based on the greatest value offered to the City. The D/B team also works as one entity to reduce constructability issues during the design phase.

However, the high costs of preparing a proposal for the D/B project may decrease the competition between the various D/B teams submitting proposals. Therefore, the quality of D/B projects tends to vary.



COST MODEL TIMELINE FOR D/B PROJECT

FIGURE 7.6

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7.3.3.6 Advantages and Disadvantages

Advantages of this delivery method are as follows:

- Single team and single point of responsibility.
- Shortened scheduled by overlapping design and construction phases.
- Design-related performance risks on the D/B team, not the City.
- Innovation from D/B allowing potential cost savings.

Disadvantages of this method are as follows:

- The City has minimal control over design.
- O&M and life-cycle costs are potentially higher.
- The proposal evaluation and selection process are complex.
- Competition is decreased, leading to higher project costs.

7.3.4 Progressive Design-Build

The PDB delivery method incorporates many of the advantages of other delivery methods mentioned and eliminates many disadvantages. In a PDB project, the D/B team is selected based on qualifications and completes the design to about 75 percent. Then, a GMP is established similar to a CMAR project, providing greater control over the design definition than the other D/B methods.

7.3.4.1 Timeline and Schedule Considerations

With this method, the PDB team is procured at about a 30 percent design level that incorporates the owner's preferences and input into essential design decisions. Once the PDB firm is selected, they take control of the design effort and work with the owner and owner's advisor to complete the design. The owner can accept the GMP or interim GMPs at any point in the design, even at a 100 percent level of completion.

Nonetheless, the PDB must produce a set of 100 percent biddable drawings and specifications. If the owner is not satisfied with the GMP at 100 percent, they may then bid it out under a conventional DBB approach.

7.3.4.2 Teams

The City selects the PDB team based on qualifications at around 30 percent design completion. The PDB team could be a single entity with both design and construction capabilities, a designer-led team or a contractor-led team, or a joint venture company in which the designer and the contractor work together as a new entity.

In contrast to CMAR, the PDB team forms independent of the owner because they have experience working together and/or their teaming provides advantages to the owner; the PDB team chooses to work together because they want to, not because they have to.

In a PDB delivered project, the City contracts with a single entity to perform design and construction services. Figure 7.7 shows the contractual and working relationship between the City and the PDB team.

The roles for each of these teams are as follows:

- City:
  - Issues an RFP at 30 percent design for a PDB team and selects the team that offers the greatest value.
  - Contracts with the single entity to design and construct the project and has input in the design.
- PDB Team (Engineer/Contractor):
  - Responsible for the delivery and performance of the project.
  - Works together to produce the design and to construct the final product
- Others: Subcontractors contract directly with the PDB team. The City may have independent technical, legal, and/or financial consultants to serve as agents in managing the procurement process, establishing performance criteria, and monitoring performance.

7.3.4.3 Risks

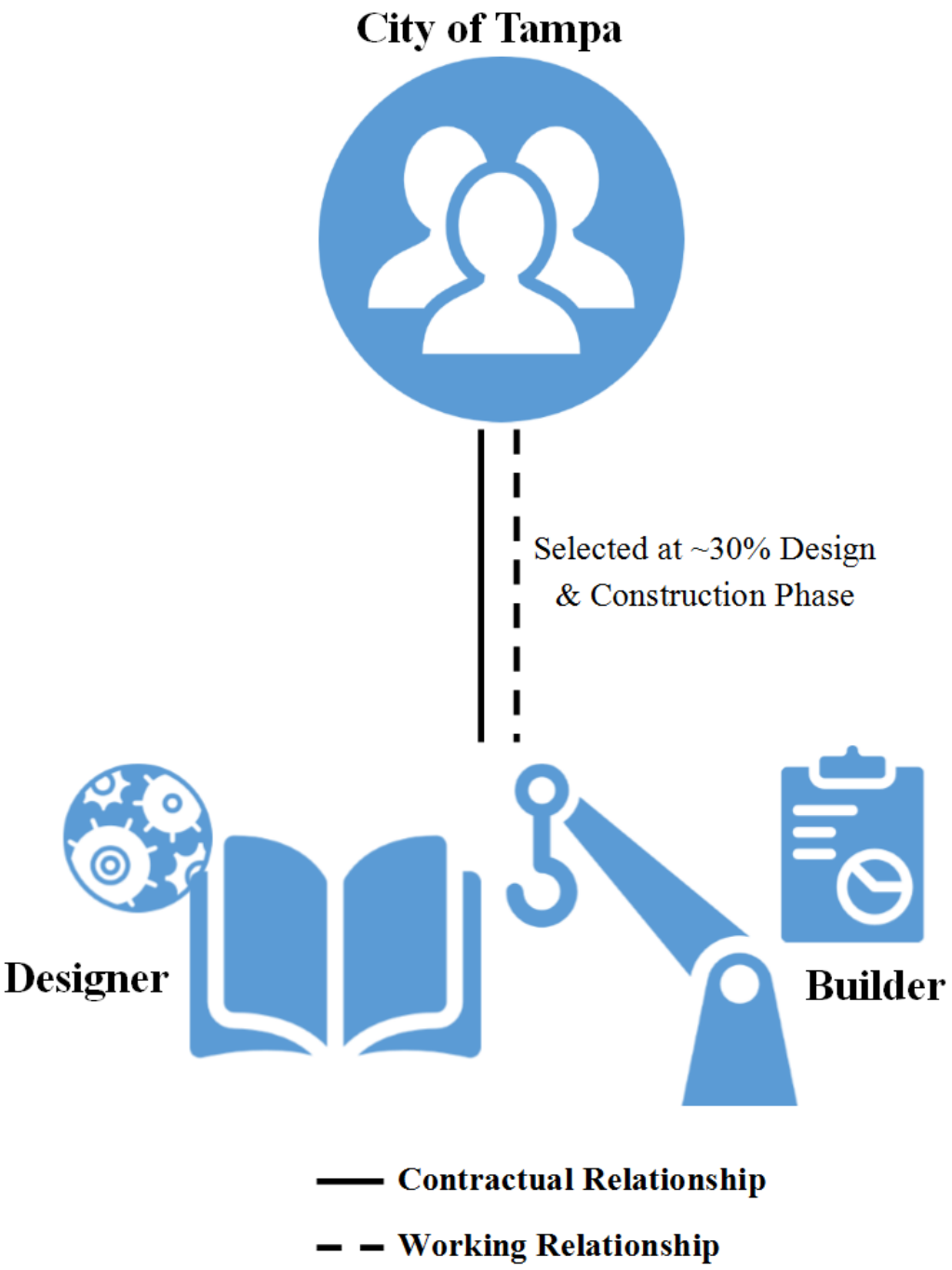
The design and construction liability remain with the PDB team. Once the PDB team is selected, the design-related performance risk also rests with the PDB team. Since the PDB team combines the design and construction liability into one entity, they are responsible for the overall project and can warrant successful performance.

With this method, the City retains some risk of the design because they provide the 30 percent (or preliminary) design in the RFP. Also, the financial and schedule impacts of change orders for uncontrollable circumstances during construction rest with the City.

7.3.4.4 Cost Model

The PDB method has no fixed bid price or GMP when the designer/contractor is selected. The PDB establishes the opinion of probable construction cost (OPCC) in their proposal at the beginning of the project, but the GMP is determined at 100 percent design, as Figure 7.8 shows.

The cost model is developed and progressed with official OPCCs at the nominal 60 percent, 90 percent, and 100 percent design levels. With the PDB's contract terms, they can perform construction work on their own, but all work must be bid in packages in an open-book fashion. The contractor is paid on a cost-plus, fixed-fee basis, and the owner and PDB firm share the cost savings from a construction cost below the GMP under a prescribed formula.



CONTRACTUAL AND WORKING RELATIONSHIP FOR PDB PROJECT

FIGURE 7.7

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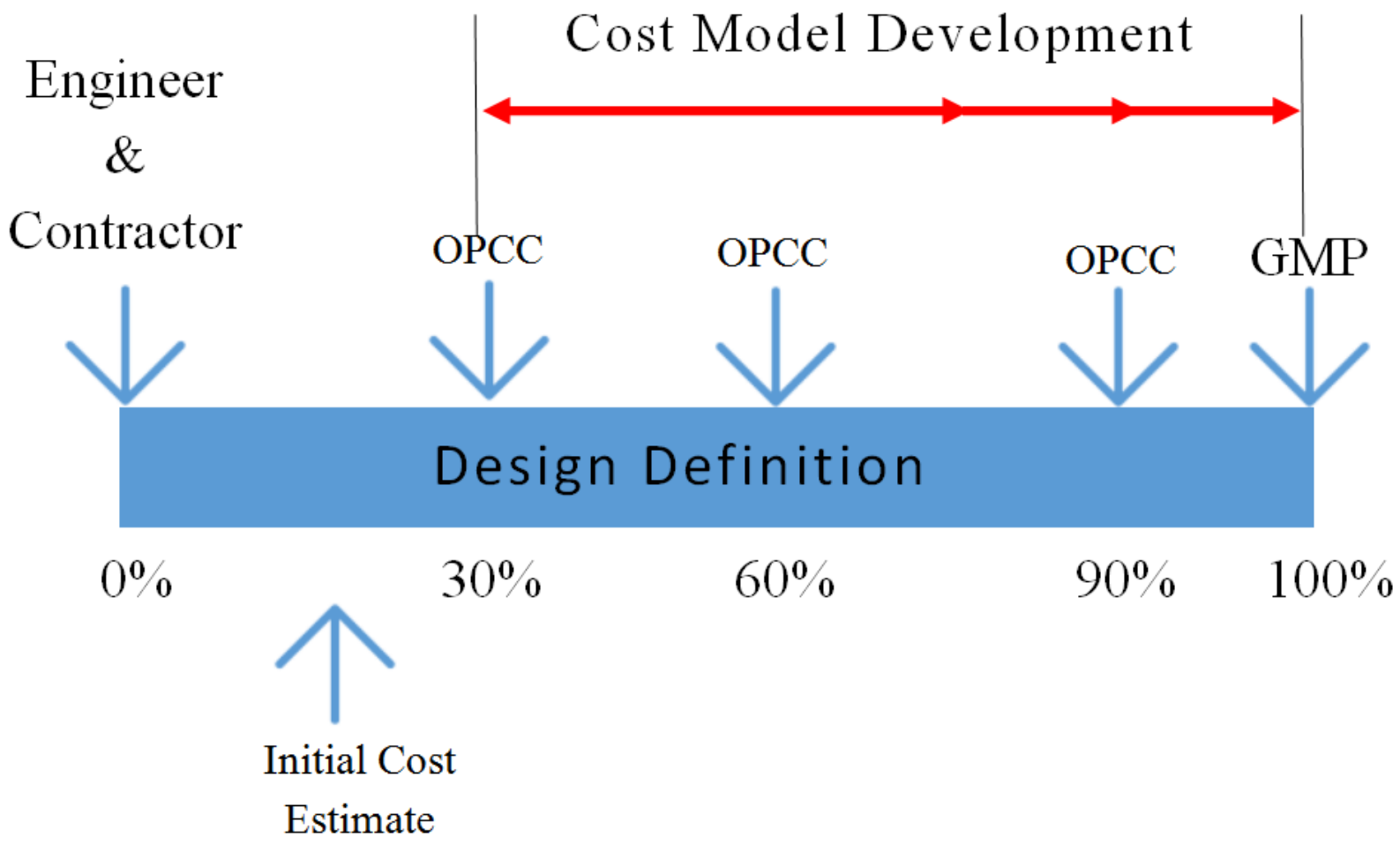

COST MODEL TIMELINE FOR PDB PROJECT

FIGURE 7.8

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7.3.4.5 Quality Considerations

A PDB project's quality should be superior because the PDB team is selected for its ability to provide the greatest value to the City. In addition, the PDB team works as one entity to reduce constructability issues during the design phase. However, the high costs of preparing a proposal (and preliminary design) for the PDB project may decrease the competition between the various PDB teams submitting proposals.

7.3.4.6 Advantages and Disadvantages

Advantages of this delivery method are as follows:

- Single team and single point of responsibility.
- Shortened schedule with overlapping design and construction phases.
- Simpler procurement process.
- Ability to design and deliver project to a fixed budget if needed.
- Higher level of control for City.
- Designer and contractor pick each other.

Disadvantages of this method are as follows:

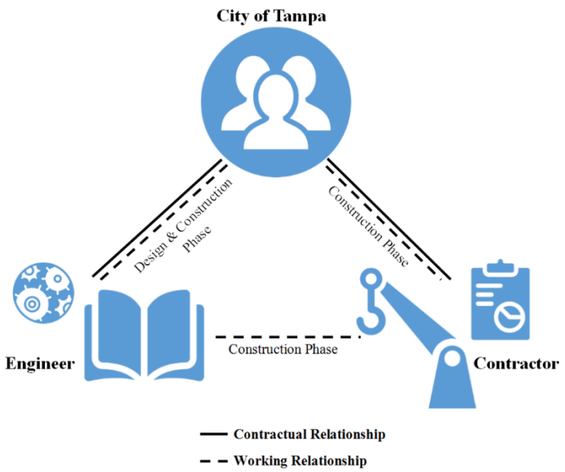
- No fixed price or GMP when PDB team is selected.
- Decreased competition leading to higher project costs.

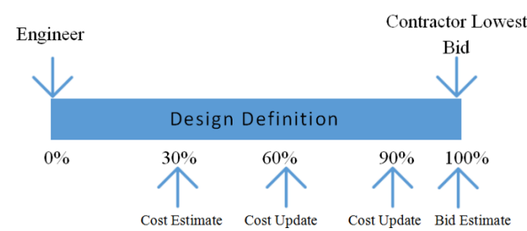
7.4 SUMMARY AND CONCLUSIONS

While many factors influence a project's implementation, this chapter summarizes four common delivery methods. Table 7.1 details pros and cons and Figure 7.9 shows a comparative summary including information on the teams, cost models, and overall advantages and disadvantages for each method. Each project was assigned a delivery method using the information provided herein in conjunction with in-depth discussions with City staff and with understanding of the DLTWTF's immediate and future needs. The proposed delivery methods for each CIP project and reasoning for each is included in Chapter 9.

Table 7.1 Comparison of Delivery Methods David L. Tippin Water Treatment Facility Master Plan City of Tampa				
Parameter	**DBB**	**CMAR**	**DB**	**PDB**
City has control over design elements	Yes	Yes	No	Yes
Project scope defined at start of construction	Yes	Yes	No	Yes
Competitive bidding environment	Yes	No	Some	Some
Cost established early in project	No	Yes	Yes	No
Design and delivery on a fixed budget	No	Yes	Yes	Yes
Contractor selected based on lowest bid	Yes	No	No	No
Contractor input during design phase	No	Yes	Yes	Yes
City assumes risk for design	Yes	Yes	No	No
City assumes risk for construction	Yes	No	No	No
Schedule could be shortened	No	Yes	Yes	Yes

DESIGN—BID– BUILD (DBB) METHOD

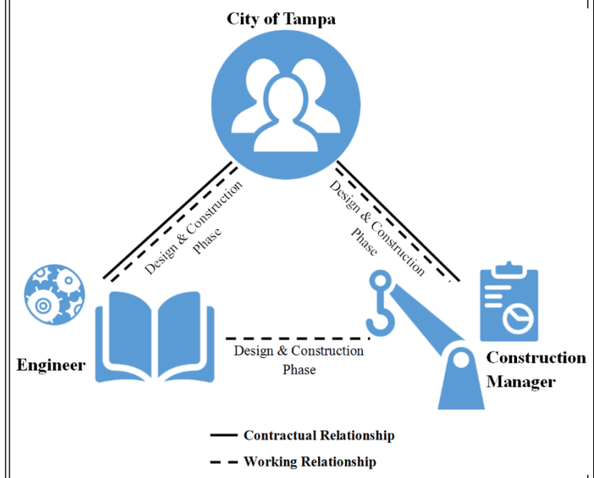


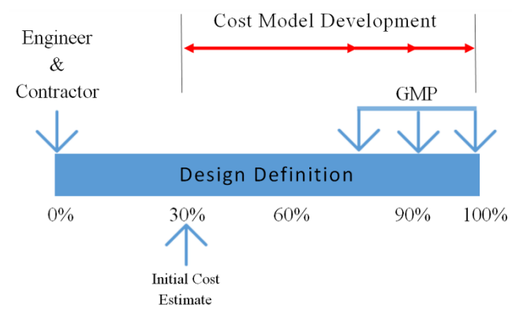


Advantages:
Typical method used and high familiarity with City.
The City has a high level of control over design elements.
The project scope is fully defined at the start of construction.
Contractors and subcontractors can be prequalified.
Equipment can be preselected and pre-purchased.
It encourages a competitive bidding environment.

Disadvantages:
Cost of the project not established until the contractor's bid awarded.
Contractor selected based on the lowest bidder if prequalification is not used.
Competition decreases for contractors if prequalification is used.
Contractor does not have input during the design phase.
The City assumes the majority of the risk for design and construction.

CONSTRUCTION MANAGER AT RISK (CMAR) METHOD

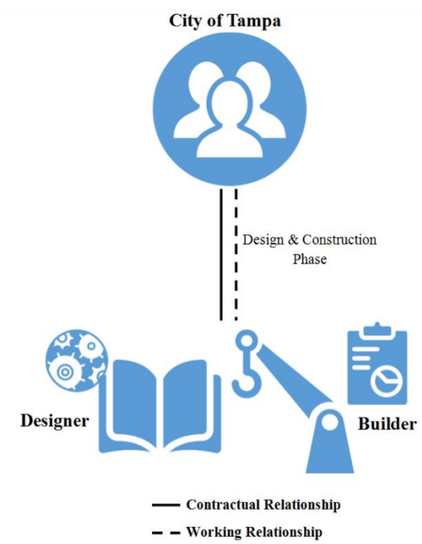


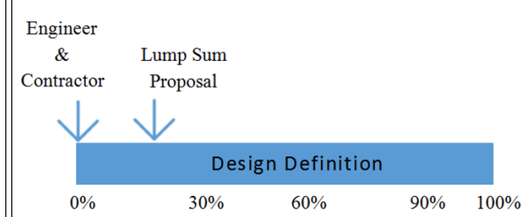


Advantages:
CM involved early in the design phase to allow for collaborative relationships and teamwork.
CM has input in the project's design phase.
The cost is established earlier in the project.
The schedule could be shorted with no bidding phase.
The project can be designed and delivered on a fixed budget.

Disadvantages:
The CM's involvement during design does not relieve the City of risk for design errors.
The project cost could be higher than DBB or D/B, due to lack of competitive pressure on the CM after selection.

DESIGN-BUILD (DB) METHOD

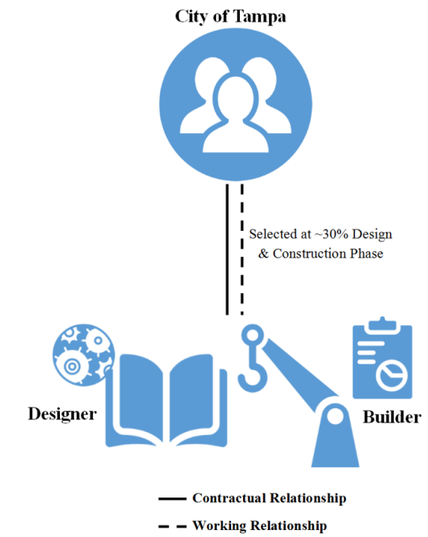


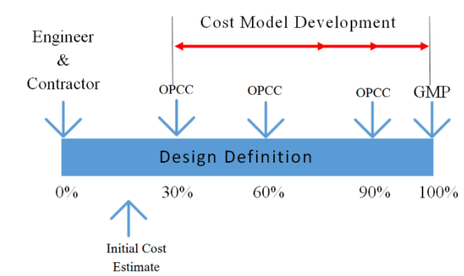


Advantages:
Single team and single point of responsibility.
Shortened scheduled by overlapping design and construction phases.
Design-related performance risks on the D/B team, not the City.
Innovation from D/B allowing potential cost savings.

Disadvantages:
The City has minimal control over design.
O&M and life-cycle costs are potentially higher.
The proposal evaluation and selection process are complex.
Competition is decreased, leading to higher project costs.

PROGRESSIVE-DESIGN-BUILD (PDB) METHOD





Advantages:
Single team and single point of responsibility.
Shortened schedule with overlapping design & construction phases.
Simpler procurement process.
Ability to design & deliver project to fixed budget if needed.
Higher level of control for City.
Designer and contractor pick each other.

Disadvantages:
No fixed price or GMP when PDB team is selected.
Decreased competition leading to higher project costs.

ALTERNATIVE DELIVERY METHOD SUMMARY

FIGURE 7.9

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SUMMARY AND RECOMMENDATIONS

8.1 REGULATORY EVALUATION

A review of the current regulations and data from the David L. Tippin Water Treatment Facility (DLTWTF) show the facility to be in compliance with existing regulations. Additionally, the evaluation provided a review of the anticipated water quality regulations that influence the selection of future treatment technologies. In summary, it is anticipated that a number of new contaminants and changes to existing regulations will be forthcoming that may impact the DLTWTF. Considerations and recommendations for these contaminants are included in Table 8.1.

Table 8.1 Regulatory Evaluation Recommendations David L. Tippin Water Treatment Facility Master Plan City of Tampa

Contaminant	Health Advisory Level or MCL	Comments	Recommendation
Strontium	1.5 mg/L	The EPA postponed the decision to regulate while collecting additional occurrence data. Strontium removal via coagulation with ferric sulfate is limited, so alternative technologies may be required.	Continue to monitor strontium levels at DLTWTF and EPA's regulatory developments.
Perchlorate	15 µg/L	In June 2016, EPA issued a request for nominations to peer review the approach for deriving a MCLG for perchlorate. Biofiltration via granular activated carbon has been shown to remove perchlorate at pilot scale.	Continue to monitor the regulatory development for perchlorate and begin monitoring concentrations at DLTWTF with bench-scale testing to measure removal as needed.
Nitrosamines	-	Anticipated that Regulatory Determination 4 will address nitrosamines as a group (including NDMA). Chloraminated systems, and ozonation may increase NDMA formation potential. However, Biofiltration has been shown to remove more than 80 percent of NDMA	Continue to monitor the regulatory development of nitrosamines and begin monitoring concentrations at DLTWTF with bench-scale testing to measure removal as needed.

Table 8.1	**Regulatory Evaluation Recommendations David L. Tippin Water Treatment Facility Master Plan City of Tampa**		
Contaminant	**Health Advisory Level or MCL**	**Comments**	**Recommendation**
PFOA/PFOS	70 ppt combined	Currently under investigation by the EPA through its Contaminant Candidate List 4 (CCL). Given the source of PFOA and PFOS is typically industrial, it is not expected that DLTWTF will identify significant levels.	Monitor PFOA and PFOS in the source water and TAP is warranted to confirm occurrence since removal via existing treatment processes is unlikely.
Cyanotoxins	0.3 / 0.7 µg/L(1)	The existing treatment processes at DLTWTF are capable of removing cyanotoxins, though oxidants may also lyse the cells and release toxins.	Continue to monitor the regulatory development for cyanotoxins and optimize existing processes for cyanotoxin removal as needed.
Fluoride	0.7 mg/L	EPA will likely revise the existing fluoride regulations at the conclusion of the current six-year review.	Continue to monitor this regulatory development. Treatment protocols at DLTWTF may need to be adjusted accordingly.
Hexavalent Chromium	-	EPA is working toward developing a final risk assessment and Maximum Contaminant Level (MCL) for Cr-6, most likely during the current six-year review.	Continue to monitor the Cr 6 regulatory development.
TOrCs	-	EPA has not yet regulated any of these products, but may as further research is developed. TOrCs are of particular concern in direct and indirect potable reuse applications.	Continue to monitor the regulatory development over the coming years and consider any impacts resulting from investigations during the concurrent TAP.

Notes:

(1) Health advisory for children published as 0.3 and 0.7 µg/L microcystins and cylindrospermopsin, respectively.

Impacts of the water quality changes being evaluated in the concurrent Tampa Augmentation Project (TAP) should also be considered when monitoring and bench testing is conducted for these parameters.

8.2 FACILITIES CONDITION ASSESSMENT

A visual inspection of the DLTWTF's aboveground assets was conducted to determine their current condition. This assessment included, but was not limited to, structures, electrical components, mechanical equipment, aboveground piping, pump systems, chemical systems, and assets that are generally valued over $5,000 or perform a critical function. Items not included in this assessment were instrumentation and controls (flow meters, sensors, SCADA, etc.), small sample pumps, piping and valves less than 6 inches in diameter, and buried/inaccessible piping, structures, or mechanical equipment.

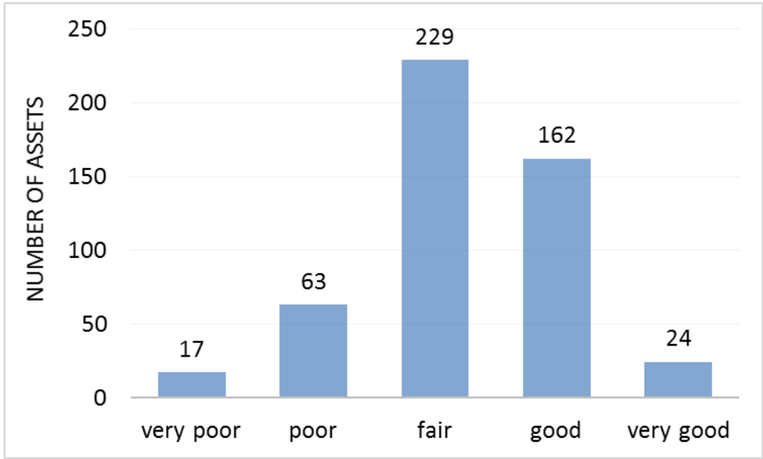
Asset replacement was determined based on the condition and criticality, and replacement timing was based on the calculated EvRUL and calculated risk which are a function of the two. Criticality was determined based on an asset's impact on treatment, plant capacity, plant reliability, safety, and meeting the City's established level of service if it were to partially or completely fail.

Figure 8.1 shows a number of figures that detail the condition of the DLTWTF's assets in relation to overall condition, criticality, and breakdown by discipline and major unit processes. The DLTWTF was found to be in fair condition overall.

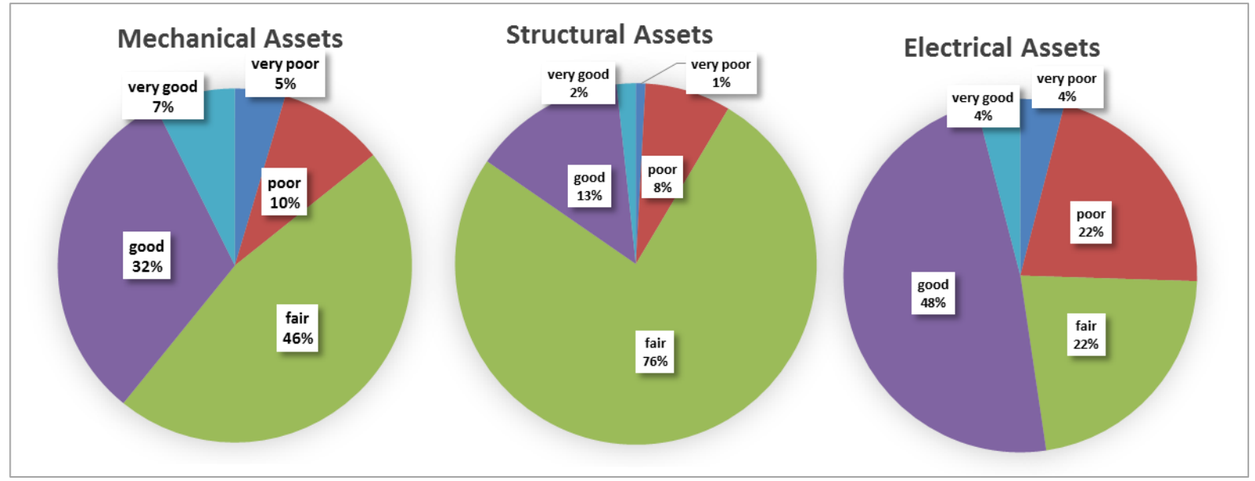
The condition and rehabilitation or replacement of an asset based on condition was considered in the development of the Capital Improvement Program (CIP). Specific scope items for these assets are included in the next chapter. Appendix A includes the condition, EvRUL, risk, and criticality for each asset evaluated in the assessment. Appendix B includes photos of all assets evaluated and Appendix C includes the Water Infrastructure Manager Software database.

8.3 PROCESS EVALUATION

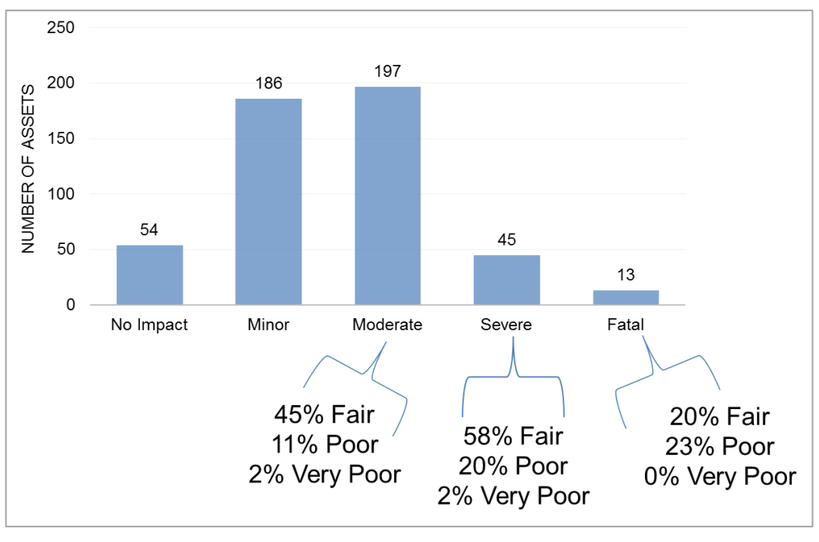
To determine current process capacity and performance at the DLTWTF, the process evaluation included assessment of the existing unit operations and processes. Additionally, limited bench scale testing was completed by City staff and Carollo, with additional testing and piloting completed during times of high color and total organic carbon (TOC) (rainy season).



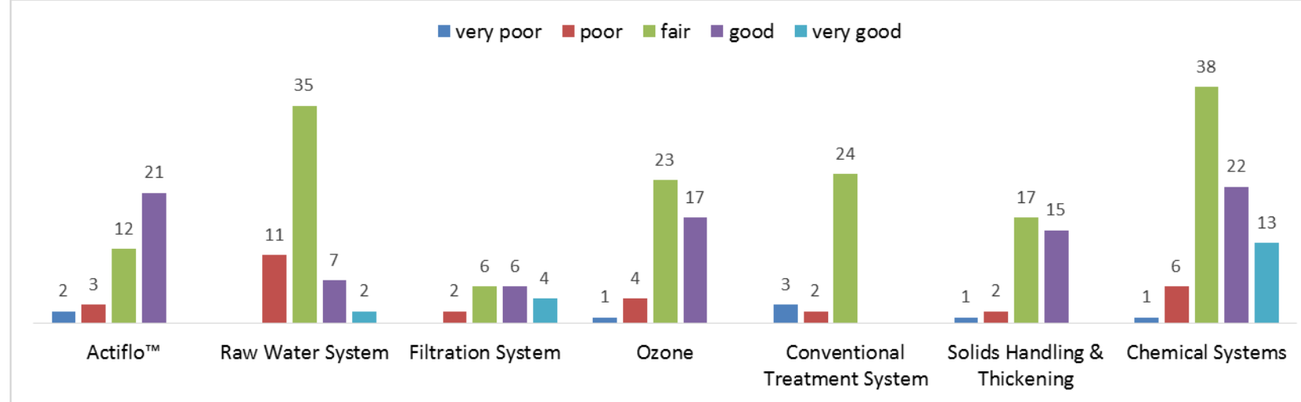
Asset Condition Summary - All Assets



Asset Condition Summary - Breakdown by Discipline



Asset Criticality Summary with Percent Condition Noted for Moderate to Fatal Assets



Asset Condition Summary - Breakdown by Major Unit Process

FACILITIES CONDITION ASSESSMENT SUMMARY

FIGURE 8.1

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH08/Graphics\Fig 8.1

In considering the Ten States Standard and Carollo's industry experience, there were a number of process issues found within the existing conventional system with the majority of these issues addressed in Chapter 5. Additionally, a number of process issues subsequent recommendations that were separate from the process alternatives are detailed in Table 8.2.

Table 8.2 Process Evaluation Recommendations
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Process Area	Limitation	Recommendation
Ozone	Inability of ozone generation and production system to meet existing and future production rate demands	Replace with new system capable of meeting 6,700 lb/day max day demand
Chemicals	Storage of 15 days not met at current or future flows based on existing chemical dosing strategy	Expand ammonia, sulfuric acid, lime, polymer, and hydrogen peroxide systems to meet 15 days of storage(1)
Filtration	Suspected system performance and hydraulic issues due to calcium carbonated precipitation. Short run times and low available head loss, Non-ideal filter geometry dimensions, Unmatched media and non-fluidization of media bed during backwash events, Filter to waste cross connection issue, Inability to meet future demands	Modify lime dosing strategy to prevent localized softening in conventional and Actiflo system. Change filter media with better match Sand and GAC. Raise filter troughs by 1 foot and inspect condition of IMS caps. Implement simultaneous air/water backwash mode. Conduct study to determine if filter to waste could be replaced with rinse to waste.
Onsite sludge handling	Inconsistent sludges between GTs, TOC desorption due to pH differences in sludges and possible inability to meet future demands	Confirm sludge flows to GTs, confirm supernatant flows to GTs to understand if current capacities are sufficient. Employ two new GTs and new surge tank, direct Actiflo™ sludge to GT3, direct belt filter press filtrate to existing surge tank.

Notes:

(1) If average chemical doses change then these systems need to be re-evaluated. Additionally, if a new aqueous ammonia system is implemented as plan by the City then expansion may not be required. Due to the recommendations in Chapter 5 (new proposed process flow) and for purposes the master plan and CIP, it is assumed only hydrogen peroxide will require expansion (if average dose levels are greater than 1.00 mg/L).

8.4 HYDRAULIC EVALUATION

The hydraulic evaluation was conducted for current and future flows of 80, 120 and 140 million gallons per day (mgd), and determined a number of bottlenecks within the DLTWTF that can/are hindering process optimization and ability for expansion. Table 8.3 details the considerations and recommendations as a result of this evaluation.

Table 8.3 Hydraulic Evaluation Recommendations
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Hydraulic Area	Limitation	Recommendation
Pre-Filter Junction Box Capacity and Filter Operations	Capacity of the junction box is currently limited due to the under-utilization of Path 3. Flow through Path 3 includes unnecessary flow restrictions caused by the configuration of the junction box and flow routing through two old recarbonation basins. Under the existing conditions, the Hydraulix® model indicated the junction box does not have the capacity to meet future demands of 140 mgd. The resulting uneven flow split can also result in inconsistent filter influent flows, which may result in starved filters at higher flow rates.	To encourage more flow through Path 3 to the filters, the recarbonation basins should be bypassed and the junction box modified, for example, by direct flow over a weir instead of through 10-inch orifices and small gates. Further evaluation is required to confirm if flow splitting among the filters will be improved by these modifications.
Post Filtration	The blend chamber is a source of substantial head loss accumulation and contributed to pressurization at the filter effluent flumes and Hawkey Box, and incomplete filtration.	Eliminate the existing blend chamber and retrofit the 7.5 MG clearwell to function as a chlorine contact tank. At 120 mgd and 140 mgd, additional flow paths from the filters are recommended to further reduce head loss and prevent filter effluent flume pressurization.

Table 8.3 Hydraulic Evaluation Recommendations David L. Tippin Water Treatment Facility Master Plan City of Tampa		
Hydraulic Area	**Limitation**	**Recommendation**
Conventional Trains 5 and 6	When flows through the conventional treatment Trains 5 through 8 reach 100 mgd, the overflow of Trains 5 and 6 may occur if the low lift pump station is not kept at 41.25 feet or below. Solids have built up over time in the lines between the sedimentation basins and the pump station, contributing additional head loss.	Continue to operate the pump station between 39.2 feet and 41.25 feet at times of high flow. Elimination of the launders within the sedimentation basins, reduce head loss and the operational restrictions at the pump station. Visually inspect piping between low lift and conventional systems to determine the extent of solids accumulation.
Actiflo™ Solids Accumulating in Process Piping to Thickener 3	Currently, only GT 3 can receive sludge from the Actiflo™ basins, which are at elevations that don't encourage consistent solids flow.	Install a pump station or sand separator at the start of the Actiflo™ sludge line to reduce solids accumulation along the line and reduce or prevent the need for pigging the line.
Junction Box 4 Overflow	Junction Box 4 overflows when the ASR is in use.	Confirm flows to this junction box to quantify amount of overflow so the appropriate recommendation on the required top of concrete elevation.
Washwater Surge Tank	Before recent operational changes, much of the flow from the surge tank was recirculated instead of being directed to the thickeners. The number of pumps recirculating water is controlled by the water surface level in the tank. To reduce the amount of water being used for recirculation, plant staff keeps the water surface level in the surge tank low, allowing more flow to reach the thickeners.	To address this issue without continuing to operate the tank at a low volume, submersible mixers could be installed to replace the recirculation system.

Appendix E through G contain the computed hydraulic profiles for each flow scenario.

8.5 ALTERNATIVES EVALUATION

A number of alternative options were developed and evaluated to optimize and/or replace the existing treatment processes at the DLTWTF for this evaluation which included:

- Alternative 1 - Baseline maintaining existing EC treatment processes (Actiflo™ and Conventional)
  - Alternative 1A
    - Existing conventional trains to treat 60 mgd
    - Retain existing 40 mgd of Actiflo™ trains
    - Install 40 mgd of new Actiflo™ trains
  - Alternative 1B
    - Improve conventional trains to treat 100 mgd
    - Retaining existing 40 mgd Actiflo™ train
- Alternative 2 - Ion exchange as a pretreatment step to the existing processes
  - Alternative 2A
    - New 140 mgd fluidized bed ion exchange (i.e. MIEX) system
    - Alternative 1B improvements
  - Alternative 2B
    - New 140 mgd fixed bed ion exchange.
    - Alternative 1B improvements
- Alternative 3 - Split treatment with micro/ultrafiltration membranes to blend with existing treatment processes.
- Alternative 4 - Micro/ultrafiltration and nanofiltration treatment scheme with biological roughing filters to replace existing treatment processes.

Each alternative was assessed for its ability to improve process performance, allow for expansion to 140 mgd, and treat up to 50 mgd of reclaimed water from TAP, among other considerations.

The capital, operating, and life cycle costs were developed for each alternative. Table 8.4 summarizes the capital, operating, and 20-year life-cycle costs for each of the five alternatives under consideration. Items common and equivalent for all alternatives were not included in the cost. As shown, Alternative 1B resulted in the lowest calculated 20-year life-cycle costs, where Alternative 3 had the highest.

When comparing longer life-cycles, however, the net present value of Alternative 1B and 2A get closer. There is a 10.2 percent difference between Alternative 1B and 2A for a 20-year life-cycle, and no significant difference (2.7 percent) in net present value for a 30-year life-cycle. Since much of the equipment is likely to last longer than 20-years, especially at the DLTWTF, an average 30-year equipment life is reasonable. A similar trend is seen when

considering increases in chemical costs above 3 percent annually due to Alternative 1B's heavy reliance on chemical usage when compared to Alternative 2A.

Table 8.4 Alternatives Life-Cycle Cost Comparison
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Cost Component	Alternative 1A: New Actiflo™	Alternative 1B: Expanded Conventional	Alternative 2A: Fluidized Bed IX	Alternative 2B: Fixed Bed IX	Alternative 3: MF/UF Split Treatment
Capital ($1,000)	$78,600	$76,700	$166,200	$216,600	$285,100
Amortized O&M ($1,000/yr)	$6,900	$8,200	$5,100	$5,100	$11,700
20 Year Life-Cycle Cost (Net Present Value) ($1,000)	$218,400	$242,900	$269,000	$288,500	$492,300

Table 8.5 compares the pros and cons of each alternative to aid in scoring the alternatives relative to each critical success factor. The alternative with the highest score out of a possible 200 was selected as the best treatment alternative to satisfy the City's needs most effectively and efficiently. Table 8.6 shows the scoring matrix for the five alternatives with the total composite score. As the table shows, except for Alternative 3, scores for all alternatives were very close to one another, though Alternative 2A received the highest score. Details on how scores were applied and reasons why are included in Chapter 5.

Table 8.5 Comparison of Alternatives
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Parameter	1A	1B	2A	2B	3
Familiar technology	Yes	Yes	No	No	No
Reduces treatment costs (chemicals, solids handling)	No	No	Yes	Yes	Yes
Uses existing infrastructure	Yes	Yes	No	No	No
Meets WQ goals	Yes	Yes	Yes	Yes	Yes
Ability to treat future TAP water	Yes	Yes	Yes	Yes	Yes
Ability to treat future TAP water in separate treatment train	No	No	No	No	Yes
Chemical system expansion required	Yes	Yes	No	No	No
Filter expansion required	Yes	Yes	Yes	Yes	No

Table 8.5 Comparison of Alternatives David L. Tippin Water Treatment Facility Master Plan City of Tampa					
Parameter	**1A**	**1B**	**2A**	**2B**	**3**
Reduces infrastructure maintenance from low pH	No	No	Yes	Yes	Yes
Produces waste stream requiring DIW	No	No	Yes	Yes	Yes
Proven technology	Yes	Yes	Limited	No	Limited
Proprietary technology	No	No	Yes	No	No

The cost evaluation, ranking analysis, pilot study results, and in depth discussions with the City resulted in the recommendation that the City proceed with the implementation of Alternative 2A (improving/expanding the conventional treatment trains and retaining the existing Actiflo™ treatment trains and implementing a new 140 mgd MIEX pretreatment system) Implementation of Alternative 2A will correct the conventional treatment trains' existing limitations, expand the overall plant capacity to 140 mgd, and provide for reduced chemical and solids handling costs (associated with use, storage, hauling, etc.). The results of the pilot study are summarized in the Final Pilot Study Report completed by Carollo Engineers, Inc. in June 2018. Water quality and overall process performance for the pilot and full scale systems were very similar, with MIEX at times providing lower finished water TOC concentrations. Considering this, MIEX is a viable and promising treatment option for the DLTWTF. The findings from the pilot study were utilized to update the draft version of this master plan and finalize capital, operating, and life cycle costs previously shown and the final CIP.

The following modifications are recommended under this alternative:

- Refurbish the existing on-site pilot plant and add MIEX pretreatment pilot module(s)
- Construct a new 140 MIEX system implemented as a pretreatment step to the conventional and Actiflo™ systems
- Modify the rapid mix basins with a flash mix pump system
- Reconfigure the conventional basin flocculation basins to a plug flow configuration with three stages of tapered energy flocculation with new flocculators, or, as a minimum, dramatically improve the hydraulic and mixing deficiencies of the existing floc systems
- Install plate setters in the sedimentation basins (pending results of full-scale testing)

Table 8.6 Alternatives Evaluation Matrix
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Critical Success Factor	Optimizes Process & Reduces Treatment Costs (1)		Provides Superior Water Quality in Most Efficient Manner (2)		Minimizes Capital Costs (3)		Minimizes O&M Costs (3)		Solves Hydraulic Bottlenecks with Max Use of Existing Infrastructure (4)		Maximizes Reliability & Redundancy (5)		Best Meets Drinking Water Regs & Safety / Security Concerns (6&7)		
Weighting Factor	6		5		2		2		2		2		1		
Alternative	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Raw Score**	**Wt. Score**	**Total Score**
1A	7	42	7	35	9	18	7	14	9	18	8	16	4	4	**147**
1B	7	42	7	35	9	18	7	14	10	20	9	18	4	4	**151**
2A	9	54	8	40	5	10	9	18	6	12	6	12	8	8	**154**
2B	9	54	8	40	3	6	9	18	5	10	4	8	8	8	**144**
3	1	6	6	30	1	2	1	2	3	6	3	6	4	4	**56**

- Construct 22 new filters (unless hydraulic improvements can demonstrate fewer filters without compromised treatment)
- Re-evaluate the needed capacity of the off-site sludge processing facility based on the reduced volumes of solids generated with an ion exchange pretreatment system.

However, it is also recommended that the City include an additional extended (one-year) pilot study with MIEX pretreatment in operation the entire duration. Additionally, mitigation and resolution of the risks identified and presented in the Pilot Plant Study report should be wholly resolved through piloting before the MIEX full scale system is constructed. This pilot would be operated in conjunction with the conceptual engineering design of the full scale MIEX system. As such, it is recommended to fully capture an entire year of data, not only in regard to TOC removal, but more specifically to include:

- Resin condition monitoring (RCM) analysis and organics desorption during the regeneration process throughout the year to understand degradation and decrease in organics removal performance over time.
- VSEP treatment runs multiple times per month to gather additional data to fully understand potential salt savings, in addition to multiple sample set deliveries to the third party vendor for confirmation of viable concrete stream usage.
- Collection of ozone dose and demand data, and bromate data (can be completed at bench scale), and consideration of various bromate control techniques. Testing should include blends of raw water from various DLTWTF supply sources including the reservoir and ASR recovery wells.
- Collection of DBP data to determine the impacts of prechlorination prior to MIEX (can be completed at bench scale)
- Evaluation and mitigation of air entrainment issues associated with the original pilot.
- Operation of the MIEX system at 600 bed volumes throughout the study to determine the impacts on TOC treatment, ozone demand, and filter runs.
- Piloting of the SIX process simultaneously with the MIEX process (for the last 6 months).

Additionally, IXOM should provide a performance guarantee for TOC removal as well as documentation supporting their intent to construct a resin manufacturing facility in the United States.

Without full understanding and mitigation of the identified risks, MIEX cannot be confidently recommended. By conducting additional piloting to confirm risk mitigation approaches in conjunction with the conceptual design, the City and their consultant could better

understand the needed customized design of this complex system to fully meet the needs of the DLTWTF while minimizing risks and unknowns.

Additional testing recommended prior to final design of these improvements should also include:

- Pilot or bench scale testing of various stainless steel passivation processes in existing basins to determine impacts of low-pH water on corrosion of future plate settlers
- Full-scale stress testing of a retrofitted conventional treatment basin to determine the need for plate settlers to achieve targeted settled water quality goals.

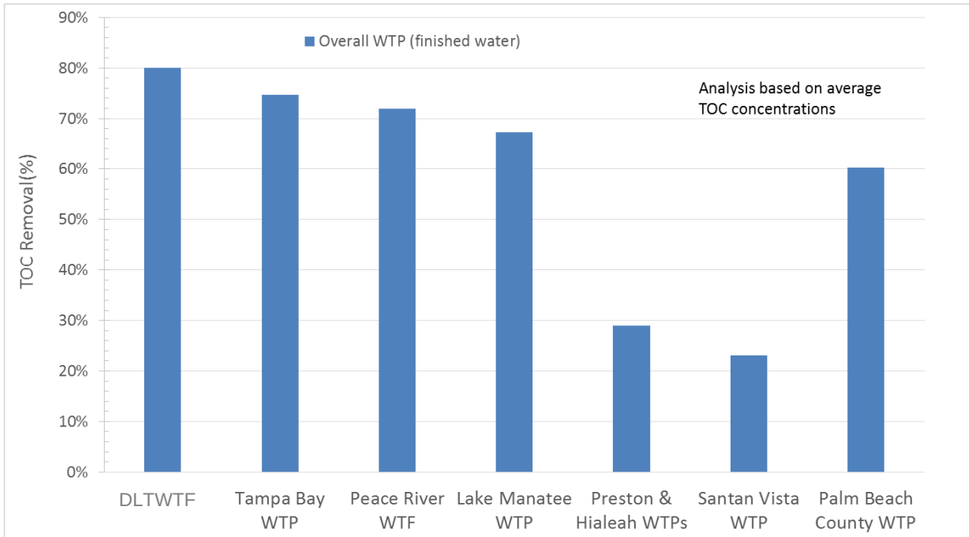
The implementation of this project, in addition to needs summarized in the condition assessment, hydraulic evaluation, and process evaluations are all noted in the 15-year CIP in the next chapter.

8.6 BENCHMARKING EVALUATION

To adequately compare operations and performance of the DLTWTF to utilities with similarly sized plants with similar complexity, process systems, and raw water quality, a benchmarking evaluation was conducted.

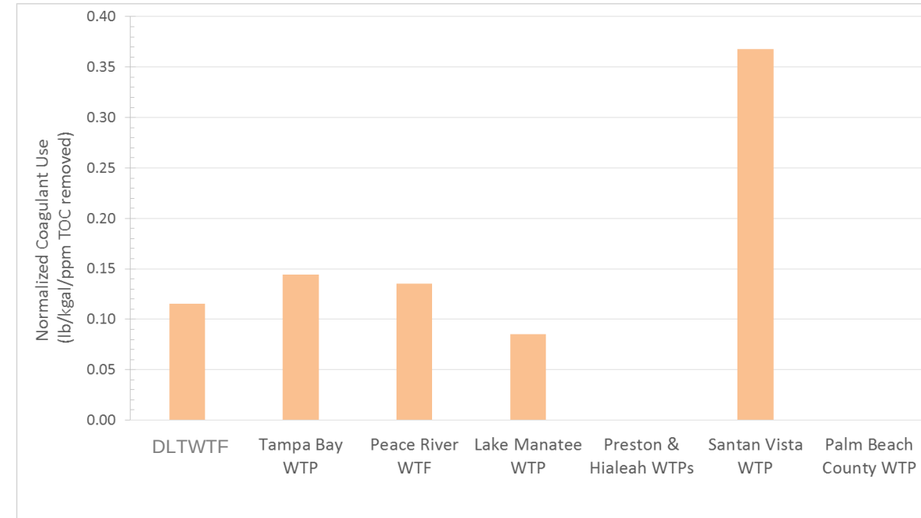
Figure 8.2 shows some of a key results of this effort. As shown, with regard to water quality parameters, the DLTWTF demonstrated the highest relative TOC removal (80 percent vs 60 to 75 percent) demonstrated by the other plants. Additionally, the normalized (to production flow) of TOC removed based on coagulant use is below (meaning most efficient) or comparable to the other plants. Color and turbidity removals for DLTWTF were comparable to those of the other benchmarked plants.

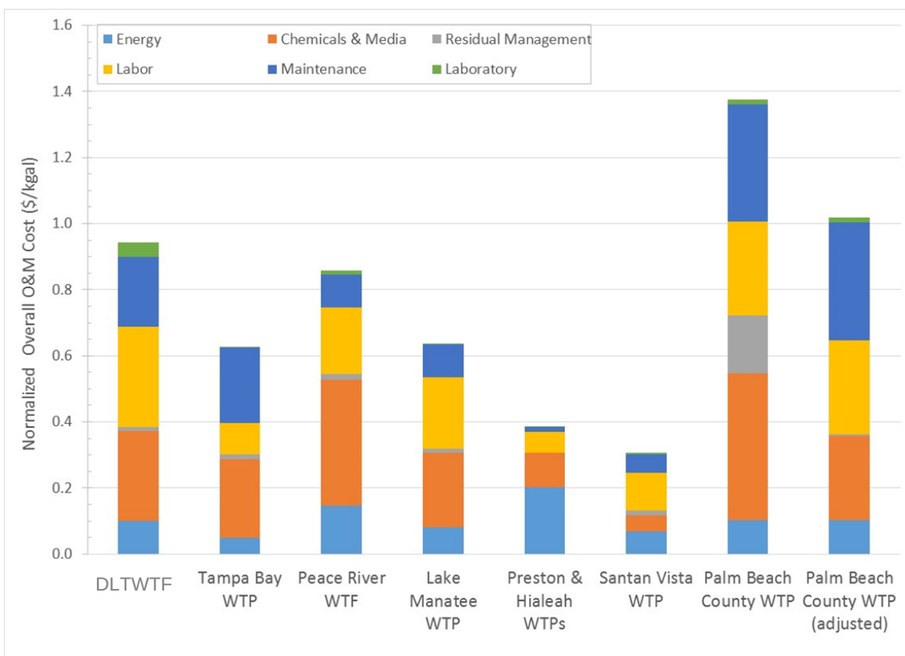
In terms of normalized energy, chemical use, and overall O&M cost, the DLTWTF is comparable to the similar plants benchmarked in this study. Overall O&M cost was also found to be comparable or slightly lower than the overall cost for the Palm Beach County WTP (adjusted) that uses the MIEX process.



TOC Removal

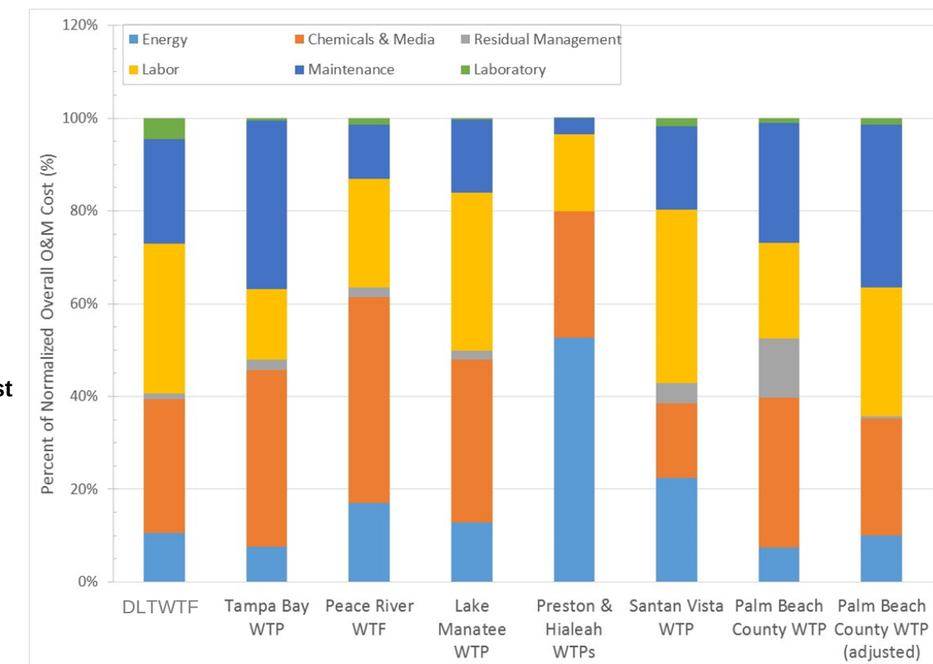
Coagulant Use Normalized to Production Flow and Amount of TOC Removed





Overall O&M Cost Normalized to Production Flow

Percent of Normalized Overall O&M Cost



BENCHMARKING EVALUATION SUMMARY

FIGURE 8.2

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

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PRIORITIZED CAPITAL IMPROVEMENT PLAN AND IMPLEMENTATION SCHEDULE

As a comprehensive approach to develop this master plan, recommendations from the previously completed tasks were prioritized, grouped, and scheduled over the 15-year planning horizon (2018 through 2032) are detailed in this chapter. Additionally, defined scopes, yearly and cumulative CIP expenditures, recommended alternative delivery methods, and an implementation plan are included herein.

9.1 PRIORITIZED CAPITAL IMPROVEMENT PLAN

Projects were prioritized considering construction sequencing, minimizing disruptions in operating the facility, maintaining balanced expenditures year to year, considering alternative funding and delivery options, and development of construction packages for projects with similar scopes in similar areas of the facility.

Upon completion of project groupings, the year of implementation was determined based on the needs of the projects individual scope items, their key drivers, and based on discussions with the City. For example, some projects are needed earlier in the 15-year plan in order to expand the plant capacity (i.e., high service pump station). Alternatively, some projects are not required until later in the plan due to their current fair or good condition asset scope items (i.e., building improvements). The following sections detail each project's individual scope and cost.

9.1.1 Project Definition and Costs

There are a total of 16 projects delegated for implementation in the 15-year planning horizon. Each of these projects are discussed in detail herein. Recommendations on alternative delivery methods are noted and references to fixed price design build are in reference to traditional design bid as discussed in Chapter 7. Hard bid signifies projects where the City should utilize contractors with whom they already have working under current contracts.

The key drivers for each project scope item were defined based on the primary reason for implementation. The five key drivers included:

1. Improve process performance or reliability (process)
2. Improve hydraulics or redundancy (hydraulic)
3. Satisfy condition needs (condition)
4. Allow for plant expansion (expansion)
5. Safety or other reasons like improving operational flexibility (other)

Capital costs were estimated for each CIP line item based on vendor equipment quotes plus installation, materials estimates, and similarly completed projects. Markup was then added to obtain an estimated overall project construction costs. These markups included, where applicable and appropriate based on project scope, site work (5 to 10 percent); piping, valves, appurtenances (15 to 20 percent); electrical and instrumentation and controls (12 to 25 percent); contractor overhead and profit (12 percent), sales tax (7 percent), and contingency (30 percent). Capital costs were developed in accordance with a Class IV opinion of probable cost of construction as defined by the Association of Advancement for Cost Engineering (AACE) in which the expected accuracy range is within +30 percent to -15 percent. Recommendations that call for the rehabilitation or replacement of an asset include costs for complete replacement for conservatism in the CIP.

The project total CIP budgets shown include engineering design and construction phase services, which are typically 15 percent of the construction costs.

9.1.1.1 Project 1 – On-site Sodium Hypochlorite Generation

9.1.1.1.1 *Scope:*

The scope includes a new on-site sodium hypochlorite generation system to replace the existing gas chlorine system, in addition to the following chemical system improvements:

- Replace piping associated with ferric sulfate system within the chemical trenches with stainless steel. Additionally, replace any steel anchor bolts at all ferric tanks if not addressed in existing tank rehabilitation project.
- Replace chlorination ventilation fans with new(1).
- Replace the building MCC (previously damaged by chlorine leak) and its associated Transformers TRA03-1 and TRC03-1.
- Provide additional 3,000 gallons of chemical storage for hydrogen peroxide system(2).
- Sandblast, structurally repair with shotcrete, and repaint interior of both ozone contactors.

Key Drivers:	Process, Condition, Expansion, Other (Safety)
Construction Costs:	$11,270,000
Engineering Design and Construction Services:	**$**1,677,000
Total CIP Budget:	**$12,947,000**
Proposed Delivery Method:	**Fixed Price DB**

This project is currently in place as design-build since there are schedule constraints, undefined scope items (requiring studies/evaluations), complex equipment, and well thought out construction sequencing that would benefit from a design-build team.

Notes:

(1) Replacement of the chlorination ventilation fans are required for safety reasons until decommissioning and removal of the existing chlorine storage and feed system.
(2) Additionally hydrogen peroxide system storage may not be required if average doses remain below 1.00 mg/L. Design engineer should re-evaluate this need at time of design.

9.1.1.2 Project 2 – High Service Pump Station Upgrades and Expansion

9.1.1.2.1 *Scope:*

The scope includes a new high service pump station to replace the existing HS Pumps No. 1 through 6 with five new vertical turbine pumps with support pump station building, in addition to a 5.0 MG clearwell, and retaining HS Pumps No. 7 and No. 8. This project includes retaining existing blend chamber, add additional blend chamber (for redundancy), construction of a new junction chamber wet well between new pump station and clearwells, and retrofit of the existing 7.5 MG clearwell with baffling and further modification (additional piping routes to Hawkey box or 7.5 MG clearwell from filters) to avoid filtered flume pressurization. Additionally, a design for a new backwash pump system to allow for utilization of non-chlorinated water will be included.

Key Drivers:	Hydraulic, Condition, Expansion
Construction Costs:	$43,800,000
Engineering Design and Construction Services:	$6,530,000
Total CIP Budget:	**$50,330,000**
Proposed Delivery Method:	**PDB**

This project is under schedule pressure and is complex involving multiple sites/pump stations and with multiple pieces of equipment to be installed while maintaining operations; therefore, sequencing is critical as it will be vital to have a construction team integrated with the design time as a progressive design build delivery.

Notes:

The final design should consider required head, flows, and most appropriate site location for the new pump station, as well as construction sequencing. These improvements should be completed before completion of the expansion project to fully realize the potential of the existing filters. Additionally, the engineer should confirm current Filter Backwash Pump No. 1 and No. 2 design to confirm adequate flows and pressure.

9.1.1.3 Project 3 - Intake Improvements and Raw Water Pump Station Upgrades

9.1.1.3.1 *Scope:*

The scope includes replacement of the intake bar rack, grass rake structures, hopper, and equipment. The new bar rack should be stainless steel and of the same bar spacing size. The grass rake system should be supplied with a new automated rake equipment, hopper, and rail system. Additionally, the existing screenings collection channel that services the mechanical rotating screens should be modified to allow screenings to flow in the southwest direction where a dumpster can be placed for collection.(1)

This project will also include reassessment of the three mechanical rotating screens at the intake and replacement of the screens, if required.(2)

Replacement of Raw Water Pumps No. 1 through 9 in addition to replacement of check valves for pumps No. 6 through 9, new reduced voltage starters for pumps No. 6 and No. 7, and a new masonry building to house the new electrical equipment for the new pumps is included.(3)

Based on the discussions with the City, a new boat storage facility (to replace the Old Raw Water Pump Building) and new floating dock system was also included in this project.

Key Drivers:	Hydraulic, Condition, Expansion, Safety, Other (operational flexibility and preference)
Construction Costs:	**$15,780,000**
Engineering Design and Construction Services:	**$2,370,000**
Total CIP Budget:	**$18,150,000**
Proposed Delivery Method:	**DBB**

This project impacts the entire raw water system, combining two systems into one which requires a holistic design approach, but there are no specific schedule constraints (as the existing system can handle future flows and is in fair condition) or early work sequencing required with construction, and therefore this project is suited for design-bid-build.

Notes:

(1) It is recommended that these modifications be conducted during periods of low flow and times during minimal algal issues to minimize impacts to raw water pumps and treatment systems.
(2) This item was originally required later in the CIP since the City was implementing a refurbishment project on these screens during completion on this master plan; however, it was preferred that this item be included since it is in the same process area that will be highly affected by the other intake and pump improvements.
(3) The new masonry building should be completed before completion of the pump change out in order to house the electrical equipment once the pumps are installed. The final design should consider pump type, the required head (due to the new MIEX system), and required future flow when selecting new pumps. Construction sequencing should also be considered to minimize plant downtime and meet demands.

9.1.1.4 Project 4 - Ozone Improvements 1

9.1.1.4.1 *Scope:*

The scope includes a new ozone system with a higher firm capacity to meet 6,700 pounds per day (lb/day) demand at 140 million gallons per day (mgd), with the replacement of the generators, PSUs, shells, and PSU HVAC systems. New LOX storage and vaporizers will be included to address demand at 140 mgd, if ozone product gas concentration is 5 percent. If the design is for 6,700 lb/day at 10 percent, then additional LOX may not be needed (to be confirmed during final design).

Additional scope items include:

- Replace ozone diffusers with gasketless type.
- Replace ozone chillers No. 1 and No. 2.
- Replace inlet valves and actuators on ozone destruct units No. 1 and No. 3.
- Replace Watlow single loop controllers for ozone destruct unit preheaters.
- Clean steel with power tools and wire brush, and repaint corroded steel members on LOX vaporizers and tanks, and repair spalled concrete in area.
- Inspect condition of two 60-inch lines into ozone contactors to determine amount of calcium carbonate precipitation (that would be contributing to ozone splitter box overflows).
- Replace by-pass vapor cooling system with new.
- Re-mount lightning rods atop ozone contractor to safer location.

Key Drivers:	Process, Condition, Expansion, Safety
Construction Costs:	$12,100,000
Engineering Design and Construction Services:	$1,730,000
Total CIP Budget:	**$13,830,000**
Proposed Delivery Method:	**CMAR**

This project involves working with equipment supplier, there is good scope definition with known sizing and process technology. However, the existing ozone system is not reliable, and therefore CMAR will allow the option of starting early-work packages (such as long-lead equipment procurement) in order to advance schedule and reduce the risk of severe impact in case of equipment failure.

Notes:

This project should be given careful consideration on the best construction sequencing to eliminate the need for major plant shut-down. The project should be implemented early in the 15-year plan while water demands are low and to address immediate issues with the current generation system.

9.1.1.5 Project 5 - Yard Piping Inspection and Improvements

9.1.1.5.1 *Scope:*

The scope includes inspection of all buried yard piping onsite, including CCTV, manhole, and lift station inspection. Costs for rehabilitation and replacement of up to 30 percent of the total yard piping (estimated to be 25,000 linear feet of various sized piping) was included; 70 percent was assumed to required cleaning, and 30 percent of the total yard piping was assumed to be less than 18 inches in diameter. A total of 15 manholes were assumed to be on-site.

Key Driver:	Condition
Construction Costs:	$2,300,000
Engineering Design and Construction Services:	$348,000
Total CIP Budget:	**$2,648,000**
Proposed Delivery Method:	**CMAR**

While design is not complex, there are funding phases and sequencing issues that require close coordination between designer and construction team; due to the ground-truthing and sequencing issues, it is preferable for the Contractor to retain the risk so a CMAR method is preferable for this project.

Notes:

Yard piping sizes, lengths, and types and number of manholes and lift stations should be confirmed by the design engineer.

9.1.1.6 Project 6 - Facility Expansion

9.1.1.6.1 *Scope:*

The scope of the facility expansion project includes the addition of a new 140 mgd MIEX system and its supporting equipment, as well as upgrades to the conventional system, the filtration system, and the solids handling systems. Rehabilitation and expansion of the facility's pilot plant and testing for MIEX and conventional system optimization are included in addition to as-built drawings for the facility upon project completion. Specific scope items are as follows:

- MIEX equipment, tanks, foundations
- Demolition of Basins 1-4(1)
- Conventional System
  - Rapid mix improvements
    - New rapid mixers
    - Structural modifications to rapid mix area to reduce flash mix volume
  - Replace conventional flocculators with new, in addition to two more mixers per train.
  - Structural reconfiguration of the flocculation stages.
  - Replace effluent launders with plate settlers, pending the results of pilot and (likely) full-scale testing demonstrating their need.(2)
  - Add 1' concrete to Basins 5 and 6 to increase capacity.(3)
  - New Conventional basin sludge collection mechanisms (i.e. chain and flight, superscraper, etc.).(4)

- Actiflo™ System:
  - Replace tube settlers with new.
  - Reinstall coagulation tank mixers in Actiflo™.
  - Wirebrush and recoat Actiflo™ sand silo and tank where coating failure has occurred within structure.
  - Evaluate condition of Actiflo™ Maturation Tank Mixer No. 1 blade to determine replacement.
  - Pressure inject Sikadur35 into crack in top slab of Actiflo™ Train No. 2 structure.
  - Replacement butterfly valve and shaft with new (or different type) in Low Lift Pump Station.
- Pre-filtration:
  - Bypass existing carbonation basins and modify junction box (new weir) to encourage more flow through Path 3 to the filters.
  - Inspect piping between ozone and pre-filter junction box and clear build up as needed.
  - Inspect condition of settled water junction box gates and replace with new if required.
- Filtration:
  - Install 22 new filters (number of filters need to be confirmed by design engineer) with supporting backwash pumps and air scour blowers(5).
  - Evaluate and rebuild or replace Filter Backwash Air Scour Blowers No. 1 and No. 2 with new.
  - Inspect condition of Filter Underdrain IMS Caps to determine if replacement is needed.
  - Raise filter troughs one foot(6).
  - Replace media in existing filters with matching GAC and Sand (6).
  - Replace filter gallery exhaust fans.
  - Consider replacement of tilting disc valve with alternative valve for filter backwash pumps.
  - Reline/Repair filtered water flume(7).
  - Pressure inject Sikadur 35 into crack in top slab of settled water junction box.
  - Sand/water blast and repaint filtration system piping located in filter gallery.
- Solids Handling:
  - Construct two GTs, one clarifier, new surge tank(8).
  - Install submersible mixers for suspension of solids in surge tank.
  - New sludge pumps with new wetwell for existing Actiflo™ system.

- Increase height of junction box 4 by 1 foot to help with overflow when ASR, BFP and supernatant are combined[9].
- Sand blast and recoat steel rake arms systems and replace select members for Gravity Thickeners and hard ware that are significantly corroded and unable to restore by sand blasting.
- Sand/water blast and repaint exterior of above ground sludge piping onsite.

Key Drivers:	Process, Hydraulic, Condition, Expansion, Safety, Operational Flexibility
Construction Costs:	$230,400,000
Engineering Design and Construction Services:	$34,524,000
Total CIP Budget:	**$264,924,000**
Proposed Delivery Method:	**PBD**

This project is large and complex, with multiple components that must be carefully sequenced to maintain existing operations and allow smooth startup of the new systems. It will be imperative to have an integrated design and construction team to minimize the risk of missed coordination and schedule slippage, and related potential for claims. Progressive design-build is the preferred option with such a complex project, allowing the Owner to have one point of contact/contract holder to coordinate all issues.

Notes:

Refurbishment and addition to the existing pilot plant occur within the first year prior to start of the design. Pilot testing should be conducted during rainy and dry seasons to refine the optimized treatment strategy to be considered in full scale design. Additional recommendations on piloting are included in Chapter 5.

(1) The demolition of Basins 1 through 4 is required due to the proposed location of the new MIEX system.
(2) Replacing the effluent launders with plate settlers is contingent upon piloting (and likely full scale testing of one train) demonstrating their need in order to produce satisfactory settled water quality. If plate settlers are not required, then a new ported wall should replace the existing effluent launders.
(3) The additional foot of concrete for Basins 5 and 6 may not be required if operation of the low lift pump station remains at elevations noted in the hydraulic evaluation.
(4) The design engineer should determine the most appropriate sludge collection mechanism based on anticipated solids loadings, the type of effluent collection and discussions with City staff.
(5) The number of new filters needed should be evaluated in piloting and full scale testing after implementation of upstream and downstream hydraulic improvements.
(6) Pilot and full scale testing on filter capabilities with process and hydraulic improvements should be realized before implementation of these items.
(7) Survey and interior investigation of this flume to determine extent of concentrate deterioration and determination of access limitations for repairs is recommended since the interior was not accessible during the assessment.
(8) The design engineer should confirm influent, effluent, supernatant flows to confirm sizing and number of new GTs/clarifiers. The number of new filters should also be understood before implementation.
(9) Flows to this junction box should be confirmed and understood to confirm the additional elevation needed to prevent overflow.

9.1.1.7 Project 7 - Electrical Transformer Upgrades

9.1.1.7.1 *Scope:*

The scope includes replacing the 240 V transformers with the standard wye configuration with 120/208V, 3 phase, 4 wire for selected transformers plant wide with the exception of TRA03-1 and TRC03-1 (addressed in Project 2).

Key Drivers:	Condition, Electrical Improvement
Construction Costs:	$1,400,000
Engineering Design and Construction Services:	$210,000
Total CIP Budget:	**$1,610,000**
Proposed Delivery Method:	**DBB**

This work will be bid by specialty (electrical) contractors, and could have been a possible candidate for Fixed Price DB, but there are no schedule constraints driving it towards an alternative delivery model and therefore design-bid-build is recommended.

9.1.1.8 Project 8 - Clearwell Expansion

9.1.1.8.1 *Scope:*

New 4.5 million gallon clearwell

Key Drivers:	Expansion
Construction Costs:	$12,070,000
Engineering Design and Construction Services:	$1,930,000
Total CIP Budget:	**$14,000,000**
Proposed Delivery Method:	**DBB**

The project scope and design are relatively simple with greenfield construction and likely only one major tie-in to existing wet well. There would be minimal to no sequencing issues and no schedule constraints, making this project a good fit for traditional delivery of design-bid-build.

Notes:
None

9.1.1.9 Project 9 - Buildings Improvements 1

9.1.1.9.1 *Scope:*

The scope includes upgrades to a number of building systems, as follows:

- Replace roof and decking systems in Old Raw Water Pump Building to match existing clay barrel tile roof and wood deck(1)

- Replace flat roof and inject areas where concrete cracking has occurred in Remote High Service Pump Building
- Replace outdated light fixtures with new LED lighting in Old Raw Water Pump Building(1)
- Replace interior lighting and switches with LED lighting in Remote High Service Pump Building
- Replace lighting in Basin Control Building with LED type. Replace MCC-S and lightning transformer within building
- Replace lighting in Maintenance building on the first floor at time of renovation
- Replace electrical panel boards and lighting in Utility building
- Replace main panel PP1 and outdated light fixtures with LED in lab building
- Load bank test the 35 kW generator in the filter building and gallery
- Perform Interior and Exterior survey of Accelator building structure

Key Driver:	Condition
Construction Costs:	$392,000
Engineering Design and Construction Services:	$60,000
Total CIP Budget:	**$452,000**
Proposed Delivery Method:	**Hard Bid**

The size and scope of this project is likely best suited for hard bid; in other words, the City should utilize already contracted companies and contractors to complete the work as opposed to other delivery methods since the project is small and not complex.

Notes:

(1) If not demolished as a part of construction of new boat storage facility in Project 3.

9.1.1.10 Project 10 - Ozone Improvement 2

9.1.1.10.1 *Scope:*

This project includes replacement of the ozone nitrogen boost system equipment and the ozone destruct system off gas blowers.

Key Driver:	Condition
Construction Costs:	$1,982,000
Engineering Design and Construction Services:	$299,000
Total CIP Budget:	**$2,281,000**

Proposed Delivery Method: **DBB**

There is no schedule driver as the components are sized for future capacity and currently in fair condition, but there is moderate design complexity with ozone systems, and the project size is relatively small so traditional design-bid-build delivery is preferable.

Notes:

These items were found to be in fair to good condition and are adequately sized for expansion; therefore, they were deemed to be separate from Project 3, and will be implemented later in the CIP.

9.1.1.11 Project 11 - Chemical Systems Upgrades

9.1.1.11.1 *Scope:*

This project includes upgrades to various chemical systems, as follows:

- Re-design or Repair/rehabilitate existing dry polymer feed system[(1)].
  - Re-evaluate structural condition of FRP tanks and replace (if needed) Polymer Tanks No. 1 and No. 2 and Dry Polymer Bins No. 1 and No. 2, Polymer Pumps, and Polymer Mixers.
- Patch areas where concrete spalling has occurred in Sulfuric Acid Unloading and Containment area.
- Replace chlorine booster pump, hydrogen peroxide feed pumps, polymer water booster pump with new.
- Evaluate structural condition of Hydrogen Peroxide Tanks 1 and 2 to determine if replacement is needed.
- Replace ferric feed pumps, sulfuric acid feed pumps.
- Replace steel anchor bolts at concrete pad for Ferric Tanks 1 – 3.
- Demolish and remove carbon silo.
- Sandblast and repaint ferric tanks 1 and 3, and replace steel anchor bolts.

Key Driver: Condition

Construction Costs: $1,660,000

Engineering Design and Construction Services: $250,000

Total CIP Budget: **$1,910,000**

Proposed Delivery Method: **DBB**

There is no schedule driver, but there is design complexity, and project size is relatively small so it is not likely to attract larger DB firms, therefore traditional delivery of design-bid-build is preferable.

Notes:
(1) Re-design of the polymer system should be considered at the implementation of this project.

9.1.1.12 Project 12 - Actiflo™ System Improvements

9.1.1.12.1 *Scope:*

The scope includes:

- Replace the VFDs and evaluate the condition of Actiflo™ Settling Tank Scrapers No. 1 and No. 2 blades with consideration of torque, noise and any corrosion.
- Evaluate internal condition of Actiflo™ Sand Pumps No. 1 through 8 and replace or rebuild accordingly.
- Evaluate condition of Actiflo™ Maturation Tank Mixer No. 2 blade to determine replacement. Replace motor and supporting appurtenances.
- Replace Hydrocyclone Structures No. 1 and No. 2 with new.
- Evaluate condition of Actiflo™ Injection Tank Mixer No. 2 blade to determine replacement.
  - Replace motor and supporting appurtenances, and rebuild or replace gearbox.

Key Driver:	Condition
Construction Costs:	$205,000
Engineering Design and Construction Services:	$30,800
Total CIP Budget:	**$235,800**
Proposed Delivery Method:	**Fixed Price DB**

A Fixed-Price DB method would be advantageous here, as the scope is able to be clearly defined and priced, and this would allow a turnkey approach with only one contract for the Owner to administer.

Notes:
This project focuses on items requiring attention based on condition later in the CIP.

9.1.1.13 Project 13 - Low Lift Pump Station Upgrades

9.1.1.13.1 *Scope:*

This scope includes replacing all the low lift pumps in addition to replacement of VFDs (Pump No. 1 and No. 4) and auto-transformers (Pumps No. 2 and No. 3), while also addressing structural crack repairs.

Key Driver:	Condition
Construction Costs:	$5,220,000
Engineering Design and Construction Services:	$781,000

Total CIP Budget:	**$6,001,000**
Proposed Delivery Method:	**DBB**

There is no schedule constraint and minimal anticipated sequencing issues. The existing system is already sized for future flows and wet wells already exist therefore project is a good candidate for traditional delivery of design-bid-build.

Notes: The design engineer should re-evaluate the condition of these pumps to determine if rehabilitation is more feasible than replacement.

9.1.1.14 Project 14 - Buildings Improvements 2

9.1.1.14.1 *Scope:*

The scope of this project includes the following:

- Renovate Utility Building Lavatory.
- Renovate lab building lavatory and potable water system.
- Replace Chemical building HVAC.
- Replace Maintenance building HVAC.
- Repair spalled concrete and remove lime build up and inspect condition to determine if repair or replacement is needed for structural members in Lime House Structure.
- Replace Ozone Building HVAC and lighting.
- Replace exhaust fans in High Service/Lower Floor Room Building.
- Replace polymer building's exterior and interior light fixtures not previously replaced.
- Replace the Material storage facility and building exterior attached light fixtures, interior light fixtures, and light switches/controls.
- Material Storage Building: Replace electrical systems.
- Material Storage Building: Replace Exhaust Fans.
- Perform interior and exterior survey of Maintenance Building Smoke Stack Structure and structural repairs as needed.

Key Driver:	Condition
Construction Costs:	$6,040,000
Engineering Design and Construction Services:	$902,000
Total CIP Budget:	**$6,942,000**
Proposed Delivery Method:	**DBB or Fixed Price DB**

There are no schedule constraints, medium sized job, the scope is clearly defined and therefore either traditional design-bid-build or fixed design-build delivery is recommended.

Notes:

These improvements are required later in the CIP, hence their separation from Project 9.

9.1.1.15 Project 15 - Solids Handling Improvements

9.1.1.15.1 *Scope:*

The scope of this project includes solids handling improvements to meet condition needs, as follows:

- Evaluate internal condition of Surge Tank Pump No. 3 and replace or rebuild accordingly.
- Rebuild gearboxes and/or motors for GT No. 1 and No. 2 clarifier mechanisms.
- Evaluate internal condition GT Sludge Pumps No. 1 and No. 2 and replace or rebuild accordingly.
- Replace surge tank valves with new.
- Evaluate condition of surge tank recirculating water jets with consideration of torque, efficiency, noise and any corrosion(1).
- Evaluate internal condition of Surge Tank Pumps No. 1 and No. 2 and replace or rebuild accordingly.
- Evaluate internal condition of Basin Sludge Transfer Pumps No. 1 through 4 and replace or rebuild accordingly.
- Evaluate condition of GT Sludge Pump No. 3, Surge Tank Pump No. 1 and Surge Tank Pump No. 2.
- Evaluate condition of gravity thickener sludge gates and replace if required.

Key Driver:	Condition
Construction Costs:	$1,270,000
Engineering Design and Construction Services:	$196,000
Total CIP Budget:	**$1,466,000**
Proposed Delivery Method:	**DBB**

There are minimal or no schedule or plant capacity constraints/impacts, small job size (not likely to attract larger DB firms), making this a good candidate for traditional delivery.

Notes:

These improvements are to satisfy deteriorated condition later in the CIP, the design engineer should re-evaluate these items are the time of project implementation.

(1) If not already replaced by new mixers as noted in Project 6.

9.1.1.16 Project 16 - General Rehabilitation and Repair

9.1.1.16.1 *Scope:*

This project is for general rehabilitation and repair items that arise yearly. Examples of these items include building systems, plant-wide HVAC/plumbing, miscellaneous piping repair, sitework/roadways maintenance and repair, painting, structural repairs, and all other R&R items not addressed by the above projects.

Key Driver:	Condition
Construction Costs:	$900,000 per year, plus 3 percent inflation per year until Year 15 ($16,740,000 total)
Engineering Design and Construction Services:	$63,000 per year, plus 3% inflation per year until year 15 ($1,172,000 total)
Total CIP Budget:	**$17,910,000**
	Proposed Delivery Method: Will depend on specific scopes for R&R items. Refer to Chapter 7 for details on each method to determine best applicable method for each R&R project.

Notes:
None.

9.1.2 15-Year Project Timeline

Project implementation timing was determined based on the condition and expansion needs of the facility. For example, if an asset had an EvRUL of five years, then that asset should be replaced by the year 2 or 3 of the CIP since that asset will have a high risk of failure near Year 5. Although maximum day demands of 134 mgd are not realized until 2032, the functional capacity (~100-105 mgd) of the existing facilities is below current and projected demands well before this time. Therefore the facility should be expanded early in the CIP with design to start in FY19. Purchased water may be required during max day demand events until completion on the expansion project.

Table 9.1 shows the timing for projects by fiscal year for the 15 year planning horizon. Project costs are included in addition to engineering design and construction service fees. The cumulative expenditures are shown for each year and the total CIP budget is shown as approximately $415 million for the 15 year planning horizon.

Table 9.1 15-Year Project Timeline
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Project No.	Project Title	Fiscal Year	FY 19	FY 20	FY 21	FY 22	FY 23	FY 24	FY 25	FY 26	FY 27	FY 28	FY 29	FY 30	FY 31	FY 32	FY 33	All Years
		Yearly Expenditure ($1,000s)	**$36,904**	**$68,184**	**$57,690**	**$58,872**	**$56,204**	**$51,388**	**$47,480**	**$14,915**	**$7,457**	**$5,947**	**$5,013**	**$1,333**	**$1,373**	**$1,414**	**$1,457**	**$415,630**
1	On-Site Sodium Hypo Generation	Construction	$5,000	$6,270														$11,270
		Engineering Design	$1,001															$1,001
		Construction Services	$300	$376														$676
2	High Service Pump Station Upgrades and Expansion	Construction	$13,000	$18,000	$12,800													$43,800
		Engineering Design	$3,900															$3,900
		Construction Services	$780	$1,080	$768													$2,628
3	Intake Improvements & Raw Water Pump Station Upgrade	Construction	$4,000	$11,780														$15,780
		Engineering Design	$1,420															$1,420
		Construction Services	$240	$706														$947
4	Ozone Improvements 1	Construction		$12,100														$12,100
		Engineering Design	$300	$701														$1,001
		Construction Services		$726														$726
5	Yard Piping Inspection and Improvements	Construction		$2,300														$2,300
		Engineering Design		$210														$210
		Construction Services		$138														$138
6	Facility Expansion	Construction		$4,900	$35,000	$52,000	$52,000	$47,000	$39,500									$230,400
		Engineering Design	$6,000	$6,000	$6,000	$2,700												$20,700
		Construction Services		294	$2,100	$3,120	$3,120	$2,820	$2,370									$13,824

Table 9.1 15-Year Project Timeline
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Project No.	Project Title	Fiscal Year	FY 19	FY 20	FY 21	FY 22	FY 23	FY 24	FY 25	FY 26	FY 27	FY 28	FY 29	FY 30	FY 31	FY 32	FY 33	All Years
		Yearly Expenditure ($1,000s)	**$36,904**	**$68,184**	**$57,690**	**$58,872**	**$56,204**	**$51,388**	**$47,480**	**$14,915**	**$7,457**	**$5,947**	**$5,013**	**$1,333**	**$1,373**	**$1,414**	**$1,457**	**$415,630**
7	Electrical Transformers Upgrades	Construction		$1,400														$1,400
		Engineering Design		$126														$126
		Construction Services		$84														$84
8	Clearwell Expansion	Construction							$3,070	$9,000								$12,070
		Engineering Design							$1,206									$1,206
		Construction Services							$184	$540								$724
9	Building Improvements 1	Construction						$392										$392
		Engineering Design						$36										$36
		Construction Services						$24										$24
10	Ozone Improvements 2	Construction								$1,982								$1,982
		Engineering Design								$180								$180
		Construction Services								$119								$119
11	Chemical Systems Upgrades	Construction								$1,660								$1,660
		Engineering Design								$150								$150
		Construction Services								$100								$100
12	Actiflo System Improvements	Construction									$205							$205
		Engineering Design									$19							$19
		Construction Services									$12							$12

Table 9.1 15-Year Project Timeline
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Project No.	Project Title	Fiscal Year	FY 19	FY 20	FY 21	FY 22	FY 23	FY 24	FY 25	FY 26	FY 27	FY 28	FY 29	FY 30	FY 31	FY 32	FY 33	All Years
		Yearly Expenditure ($1,000s)	**$36,904**	**$68,184**	**$57,690**	**$58,872**	**$56,204**	**$51,388**	**$47,480**	**$14,915**	**$7,457**	**$5,947**	**$5,013**	**$1,333**	**$1,373**	**$1,414**	**$1,457**	**$415,630**
13	Low Lift Pump Station Upgrades	Construction									$5,220							$5,220
		Engineering Design									$468							$468
		Construction Services									$313							$313
14	Building Improvements 2	Construction										$4,000	$2,040					$6,040
		Engineering Design										$450	$90					$540
		Construction Services										$240	$122					$362
15	Solids Handling Improvements	Construction											$1,270					$1,270
		Engineering Design											$120					$120
		Construction Services											$76					$76
16	General Rehabilitation and Repair	Construction	$900	$927	$955	$983	$1,013	$1,043	$1,075	$1,107	$1,140	$1,174	$1,210	$1,246	$1,283	$1,322	$1,361	$16,739
		Engineering Design	$36	$37	$38	$39	$41	$42	$43	$44	$46	$47	$48	$50	$51	$52	$54	$670
		Construction Services	$27	$28	$28	$30	$30	$31	$32	$33	$34	$35	$36	$37	$39	$40	$41	$502

9.2 IMPLEMENTATION SCHEDULE

An implementation schedule was developed in order to show the complete works of this master plan in a graphical manner. Figure 9.1 shows the resulting illustration.

The schedule shows the cumulative expenditure for each fiscal year in addition to which projects are implemented each year, the number of projects in design, and the number of projects in construction. The cumulative expenditures are shown and add up to the total proposed CIP budget of $415 million.

The expected plant capacity by year is shown as well as the max day and average demands to show periods where water may need to be purchased. As shown, water will only need to be purchased during periods when the demands exceed the capacity of the plant, which at times will be reduced to facilitate basin renovation. This reduced plant capacity is expected to fall in years 2020 through 2021 due to the modifications to conventional system basins. The plant will reach 140 mgd capacity upon completion of the filter modifications as the last scope item in the expansion project. Actual plant capacities should be confirmed by during final design once construction sequencing is better defined.

The project information summary contains project numbers, project titles, start and end dates, the total estimated cost (construction, design fees, and construction services), the key driver(s), and the recommended delivery method for each project.

David L. Tippin Water Treatment Facility Master Plan - Proposed Capital Improvement Plan

15-Year Implementation Schedule

Fiscal Year	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033
Project Number(s)	1,2,3,4,6,16	1,2,3,4,5,6,7,16	2,6,16	6,16	6,16	6,9,16	6,8,16	8,10,11,16	12,13,16	14,16	13,14,15,16	18	18	18	18
Yearly CIP Expenditure ($1,000,000)	$36.9	$68.2	$57.7	$58.9	$56.2	$51.4	$47.5	$14.91	$7.46	$5.95	$5.01	$1.33	$1.37	$1.41	$1.46
No. of Projects in Design	6	5	2	2	1	2	2	3	3	2	3	1	1	1	1
No. of Projects in Construction	3	6	5	3	3	3	3	4	3	3	3	2	1	1	1
Cumulative CIP Expenditures ($1,000,000)	$36.90	$105.09	$162.78	$221.65	$277.85	$329.24	$376.72	$391.64	$399.09	$405.04	$410.05	$411.39	$412.76	$414.17	$415.63

Plant Capacity

Project Information Summary

1 Process
2 Hydraulic
3 Condition
4 Expansion
5 Safety/Other

Project No.	Project Description	Start Date	End Date	Estimated Cost	Key Driver	Delivery Method
1	On-Site Sodium Hypo Generation	FY19	FY20	$12,947,200	1 2 3 4 5	DB
2	High Service Pump Station Upgrades and Expansion	FY19	FY21	$50,328,000	1 2 3 4 5	PDB
3	Intake Improvements & Raw Water Pump Station Upgrade	FY19	FY20	$18,147,000	1 2 3 4 5	DBB
4	Ozone Improvements 1	FY19	FY20	$13,827,000	1 2 3 4 5	CMAR
5	Yard Piping Inspection and Improvements	FY20	FY20	$2,648,000	1 2 3 4 5	CMAR
6	Facility Expansion[1]	FY19	FY25	$264,924,000	1 2 3 4 5	PDB
7	Electrical Transformers Upgrades	FY20	FY20	$1,610,000	1 2 3 4 5	DBB
8	Clearwell Expansion	FY25	FY26	$14,000,000	1 2 3 4 5	DBB
9	Building Improvements 1	FY24	FY24	$451,520	1 2 3 4 5	Hard Bid
10	Ozone Improvements 2	FY26	FY26	$2,280,920	1 2 3 4 5	DBB
11	Chemical Systems Upgrades	FY26	FY26	$1,909,600	1 2 3 4 5	DBB
12	Actiflo System Improvements	FY27	FY27	$235,800	1 2 3 4 5	DB
13	Low Lift Pump Station Upgrades	FY27	FY27	$6,001,200	1 2 3 4 5	DBB
14	Building Improvements 2	FY28	FY29	$6,942,400	1 2 3 4 5	DBB or DB
15	Solids Handling Improvements	FY29	FY29	$1,466,200	1 2 3 4 5	DBB
16	General Rehabilitation and Repair	FY19	FY33	$17,910,754	1 2 3 4 5	DBB/Hard Bid

Notes:
1. The costs associated with the additional flow of 50 mgd provided by Tampa Augmentation Project for this Facility Expansion project is equivalent to approximately $38,300,000.

15-YEAR IMPLEMENTATION SCHEDULE

FIGURE 9.1

CITY OF TAMPA
DAVID L. TIPPIN WTF MASTER PLAN

carollo

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APPENDIX A – COMPREHENSIVE LIST OF DLTWTF ASSET CONDITION SCORES

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX B – CONDITION ASSESSMENT PHOTOS

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX C – ASSET MANAGER DATABASE

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX D – APPLICABLE TEN STATES STANDARDS

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX E – HYDRAULIX® RESULTS: 80 MGD

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX F – HYDRAULIX® RESULTS: 120 MGD

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX G – HYDRAULIX® RESULTS: 140 MGD

(TO BE PROVIDED ELECTRONICALLY)

APPENDIX 5A – CHLORINE STORAGE AND FEED SYSTEM ALTERNATIVES EVALUATION

This document is released for the purpose of information exchange review and planning only under the authority of Larry E. Elliott February 22, 2017, Florida PE No. 65224.

CITY OF TAMPA

DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN

TECHNICAL MEMORANDUM NO. 5A CHLORINE STORAGE AND FEED SYSTEM ALTERNATIVES EVALUATION

DRAFT
February 2017

CITY OF TAMPA

DAVID L. TIPPIN WATER TREATMENT FACILITY MASTER PLAN

TECHNICAL MEMORANDUM
NO. 5A
CHLORINE STORAGE AND FEED SYSTEM ALTERNATIVES EVALUATION

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CHEMICAL STORAGE EVALUATION

1.0 INTRODUCTION

The purpose of this Technical Memorandum (TM) is to present the options regarding increased safety and security surrounding the existing 90-ton chlorine rail car storage and feed system located at the David L. Tippin Water Treatment Facility (DLTWTF). Most notably, this system is not equipped with a scrubber system to abate a leak of chlorine gas, particularly a catastrophic leak that could endanger plant staff, visitors, and the neighboring community. It is the City's desire, and the recommendation of Carollo Engineers, that the plant either be equipped with a state-of-the art scrubber system, or alternatively, the chlorination system be replaced with an on-site storage system of sodium hypochlorite (delivered in bulk to the facility), or a system capable of generating (on-site) sodium hypochlorite to be used for disinfection. These three options are noted herein as follows:

- Option A - Retain the existing 90-ton rail car storage system, along with the evaporators and chlorinators (suitable for plant flows of 140 mgd), and install a new 90-ton chlorine scrubber facility
- Option C - Replace the chlorine system in its entirety with a new sodium hypochlorite storage and feed system that relies on routine bulk delivery of sodium hypochlorite
- Option B - Replace the chlorine system in its entirety with a new on-site sodium hypochlorite generation and storage system, together with new chemical feed pumps

This TM summarizes the evaluation of each of these three options, in consideration with their costs, and provides a recommendation on the most appropriate option for the City moving forward. After review and confirmation of the selected option with the City, the costs and timing of this recommendation will be incorporated into the DLTWTF Master Plan.

2.0 BACKGROUND

2.1 Chlorine Chemical Hazard Assessment

There are a number of chemical hazard classifications and required protective measures for chlorine in the current regulations codes and standards, which are as follows:

- MSDS – Chlorine is a corrosive, poisonous, greenish-yellowish, suffocating gas which can cause burns to skin and eyes, and may be fatal if inhaled. Chlorine is typically stored under pressure as an amber colored liquid. Chlorine has a strong pungent irritating odor. Severe exposure to chlorine vapors may cause pneumonitis and pulmonary edema. Use of tepid water is recommended for flushing eyes and skin in the event a person comes in contact with chlorine. Use of dry chemical or CO_2 fire

suppression is recommended for small fires and water spray or foam for large fires; however care shall be taken to ensure that water is unable to get inside storage container. Water runoff should be contained due to potential contamination.

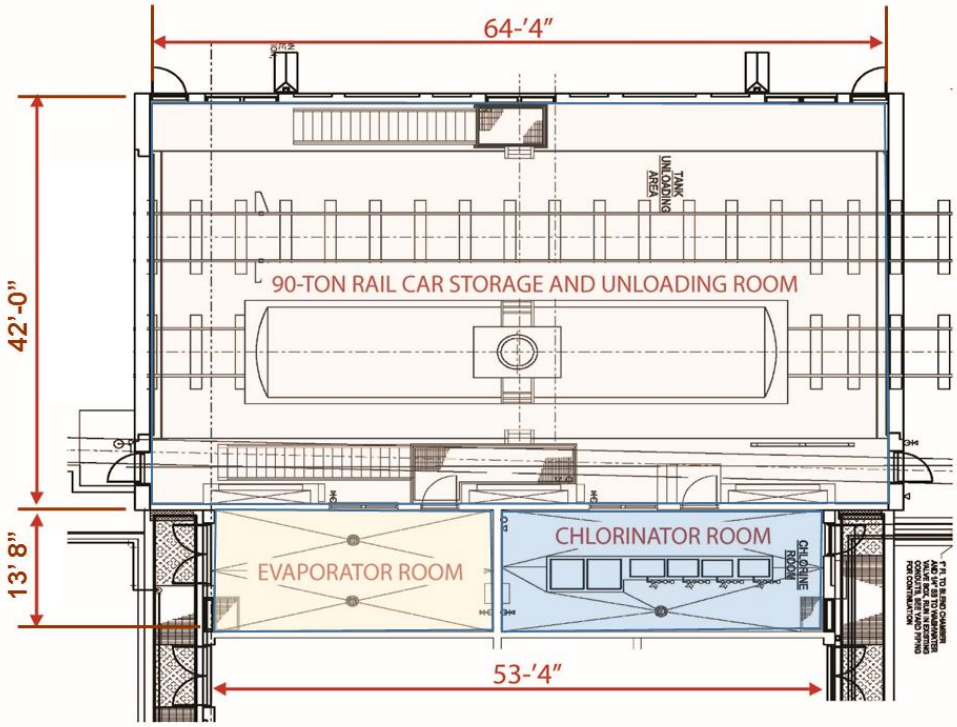
- Chlorine is listed as a regulated toxic substance by Title 40 Code of Federal Regulations (CFR) 68.130, as well as an Extremely Hazardous Substance (EHS) in Section 302 of the Emergency Planning and Community Right-to-Know Act (EPCRA).
- Chlorine is a heavier than air gas, and all ventilation control systems must be designed appropriately to capture chlorine at low points in the room where the chlorine gas can leak.
- NFPA Hazard Rating – Health Hazard – Chlorine has a Health Hazard Rating of 4 (Extreme Hazard – Highly Toxic); Fire Hazard – Chlorine has a Fire Hazard Rating of 0 (Minimal, will not burn under normal conditions).
- Building Code Rating – Chlorine is listed as a CORROSIVE and OXIDIZER chemical in the codes. Chlorine is also a TOXIC chemical per the building codes. The International Fire Code (IFC) defines a chemical as TOXIC whenever its median lethal concentration (LC 50) falls between 200 ppm and 2,000 ppm by volume of gas or vapor. Chlorine has LC50 = 293ppm (refer to MSDS) so it clearly falls within the code defined TOXIC range. Chlorine is a TOXIC chemical per the building codes.
- Other National Regulations – The national standards and regulations, such as OSHA and SARA, list chlorine as a hazardous chemical. Chlorine is listed as an extremely hazardous chemical per the SARA Title III.

Each of these regulations and standards must be taken into careful consideration when determining the most appropriate chlorine chemical storage and feed system for a facility.

2.2 Existing Storage and Feed System

Two 90-ton chlorine rail cars are housed side-by-side in the existing chlorine storage building which has a single rolling door that remains closed when rail car deliveries are not in progress. The general site layout of the chlorine building is shown in Figure 5.1A. Adjacent to the storage area is a chlorine evaporator room and a chlorinator room. Chlorine gas, which has been converted from liquid to gaseous chlorine (under pressure) via the evaporators, is in turn converted from a pressured gas to gas under vacuum using vacuum regulating valves before it enters the chlorinator room. As such, the areas that would require scrubbing would include the 90-ton rail car storage area as well as the evaporator room.

Currently, in the event of a leak, automated shut off valves on the feed lines from the rail car close via a signal from the leak detector(s) and the ventilation fan is turned off in order to contain a leak. This system is not adequate in ensuring the protection of plant staff in the event of a leak.



GENERAL SITE LAYOUT OF EXISTING CHLORINE BUILDING

FIGURE 5.1A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

pw:\\Carollo\Documents\Client\FL\Tampa\10194A00\Deliverables\Appendices\Fig 5.1A

3.0 CHLORINE STORAGE AND FEED SYSTEM ALTERNATIVES

3.1 Option A - Chlorine Scrubber System

The DLTWTF follows the Florida Fire Prevention Code (FFPC) and the National Fire Protection Association (NFPA) where, under the NFPA 55: Compressed Gases and Cryogenic Fluids Code, Section 7.9.3.3, is it required to have a system capable of the following:

- Treatment system capable of handling the maximum anticipated pressure of release.
- Treatment system capable of reducing the allowable discharge concentrations to one-half the immediately dangerous to Life or Health (IDLH) threshold at the point of discharge (10 ppm for Chlorine).
- Treatment system sized to process maximum worst case release of gas based on the maximum flow rate of release from the largest vessel.

The chlorine scrubber would be designed to handle a gaseous chlorine leak rate of 15,000 lbs/hr from the excess flow valve on the rail car and will operate at a ventilation rate of 5,000 cfm. A ventilation rate of 5,000 cfm is a standard rate among major equipment manufacturers and would provide a negative pressure in the room being scrubbed. The chlorine scrubber system would serve the chlorine storage room and the evaporator room.

When a chlorine leak is detected by the leak detector(s), motorized louvers would automatically shut to prevent chlorine from leaving the building. The scrubber system exhaust fan will then initiate, creating the negative pressure required to draw chlorine from the chlorine storage and evaporator rooms through the ventilation duct. Contaminated air would enter the scrubber unit where the recirculation pumps will provide caustic from the caustic storage tanks. The chlorine in the contaminated air would chemically react with the caustic and become neutralized. The system will continue to operate until chlorine is no longer detected by the detector.

Aqueous sodium hydroxide (NaOH), or caustic, solutions are typically manufactured at 50 percent and 70 percent concentrations. However, concentrations used for scrubbing solutions range from 15 percent to 25 percent, with 20 percent being the most common. At this concentration, caustic solution has its lowest freezing point of -17 degree Fahrenheit (F). The freezing points at 15 percent and 20 percent solution are 4 degree F and 0 degree F respectively. Using a 20 percent solution will therefore minimize heat tracing and insulation required for the system. In addition, at concentrations above 25 percent the solution becomes more viscous and would be too hot during neutralization, and the resulting salts would not be soluble in the solution.

Caustic reacts with chlorine to produce sodium hypochlorite (NaOCl), salt (NaCl) and water (H_2O). Two moles of caustic (80 pounds) are required to neutralize each mole of Cl_2

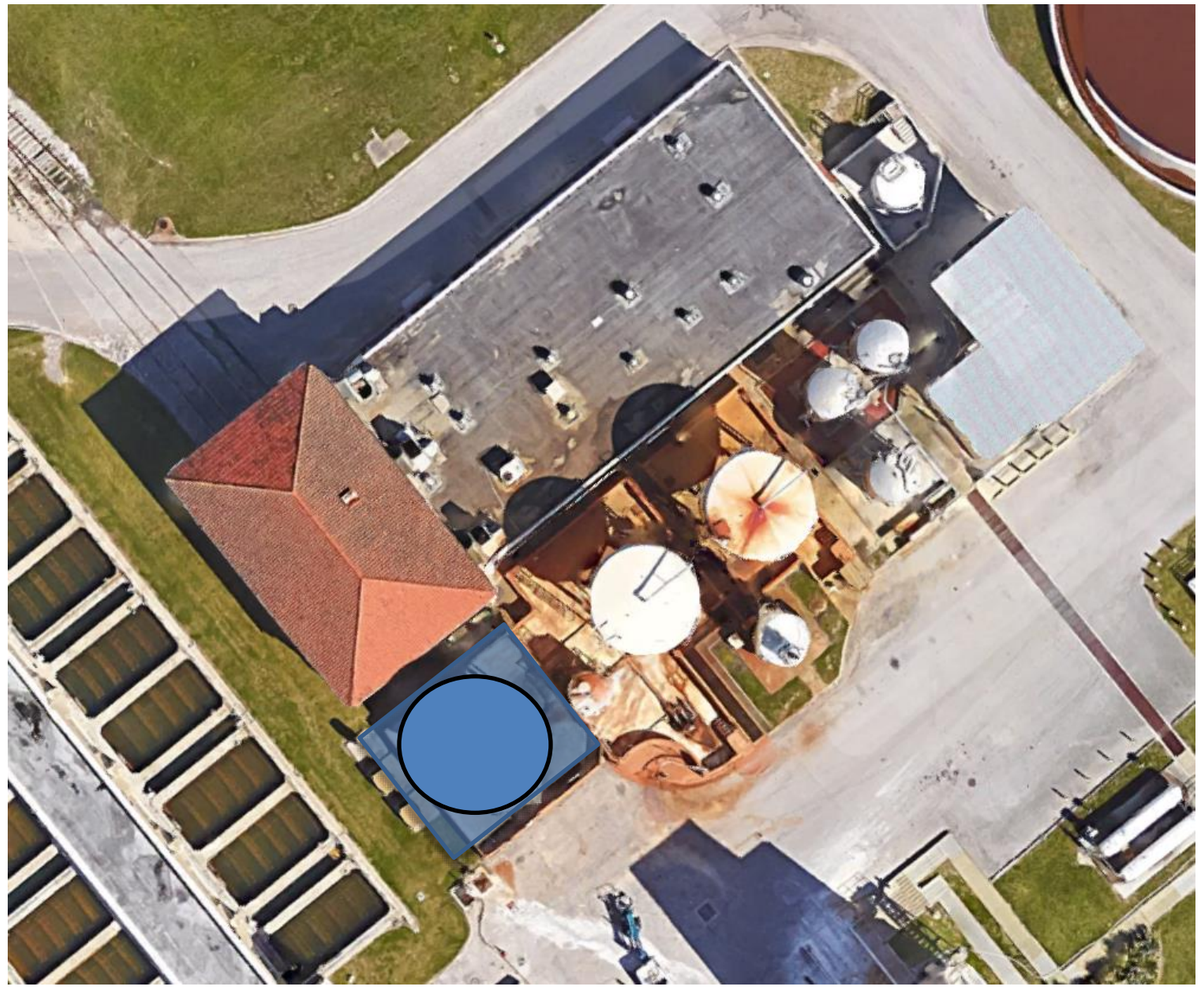
(70.9 pounds). Each gallon of 20 percent solution contains 2.04 lbs of NaOH. Additional caustic volume is provided so that the final caustic concentration will not drop below a minimum desired value of 4 percent. At concentrations lower than this, decomposition of the solution may occur, terminating the absorption of chlorine.

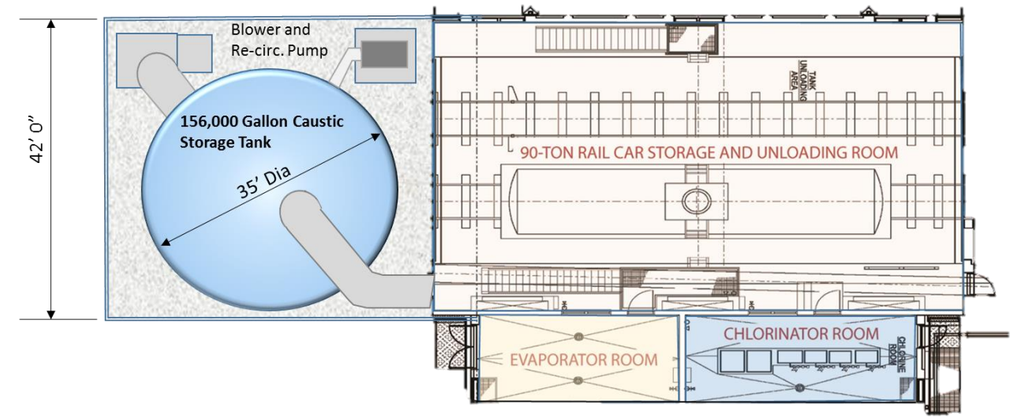
Also, excess caustic solution is typically added to the required quantity to account for fugitive leaks and for caustic depletion due to reaction with carbon dioxide (CO_2) in the atmosphere. If an adequate quantity of excess caustic is not supplied and is subsequently consumed for fugitive leaks and reaction with CO_2, the entire caustic inventory may be drained. The scrubber system would therefore be designed to hold a conservative quantity of excess caustic for the given leak event.

The scrubber system option evaluated provides for treating the entire contents of either rail car (90 tons) with ample caustic that is stored in a single tank and equipped with a recirculation pump and a blower. The tank would need to store approximately 156,600 gallons of 20 percent caustic to be stored in one storage tank, with the option to mount the scrubber unit on top of the tank or adjacent to the tank. Both mounting options are depicted in Figure 5.2A. The storage tank would be 35 ft in diameter and 22 ft tall. It should be noted that the size of the scrubber unit itself, 5,000 cfm, is the same whether caustic is stored in a single tank or multiple tanks. The decision to use multiple tanks versus one tank comes with inherent advantages and disadvantages related to system flexibility, handling and replacing spent or degraded caustic, and cost. This type of decision should be part of a more detailed preliminary design study should this option be implemented.

The proposed location of the scrubber is also shown in Figure 5.2A, and assumes that continued use of other chemicals in the area of the chemical building (lime, sulfuric acid, and sodium hydroxide) will be maintained (or increased in volume due to capacity expansion) in keeping with the concept of enhanced coagulation treatment. In the scenario shown, one of the ferric sulfate tanks would need to be relocated or replaced in an alternate location to avoid long runs and road crossings of duct from the building to the new caustic tank.

Should an alternate treatment method (other than enhanced coagulation) be adopted as part of this Master Plan, then the area now used for the bulk of these chemicals could be repurposed for the caustic storage for the scrubber system (Option A), or perhaps for sodium hypochlorite storage associated with Options B and C.





OPTION A MOUNTING OPTION AND PROPOSED LAYOUT

FIGURE 5.2A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

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3.1.1 Option A - Advantages and Disadvantages

The major drawback of the scrubber system is the need to maintain high volumes of caustic soda which has a limited shelf life and represents a chemical that is hazardous in and of itself, albeit it is a chemical plant staff are accustomed to handling given the plant's current treatment process that uses caustic as one of the two chemicals for pH adjustment. A second concern is the need to exercise the equipment regularly to ensure its reliability and functionality and the likelihood that this system would need a reliable source of backup power.

The obvious single advantage of the system is the chemical costs are far less than purchasing bulk sodium hypochlorite (Option B). A second advantage (over Option C) is that it is simpler to operate than an on-site sodium hypochlorite generation system.

3.1.2 Option A - Capital and Operating Costs

The estimated capital costs for a 90-ton chlorine scrubber system, as well as the average annual operating and maintenance (O&M) costs for chlorine and maintaining a fresh supply of caustic soda, is shown on Table 5.1A

Table 5.1A Option A Capital and Operating Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Capital Costs (x 1,000)	Annual O&M Costs (x 1,000)	Total Present Worth Costs (x 1,000)
Chlorine Scrubber System (includes installation)	$1,850	$50	$2,594
Electrical & Instrumentation	$225	$0	$225
Structural (tank foundation, spill containment)	$200		$200
Replacement Ferric Sulfate Tank	$80	$0	$80
Annual Costs (Chlorine)	$0	$485	$7,214
Demurrage on Rail Cars/Track Maintenance (??)	$0	$0	$0
Annual Costs for Refreshed Caustic Soda	$0	$162	$2,414
Contingency and Contractor Overhead and Profit	$470	$0	$470
Total	**$2,825**	**$697**	**$13,197**

3.2 Option B - On-Site Storage of Sodium Hypochlorite

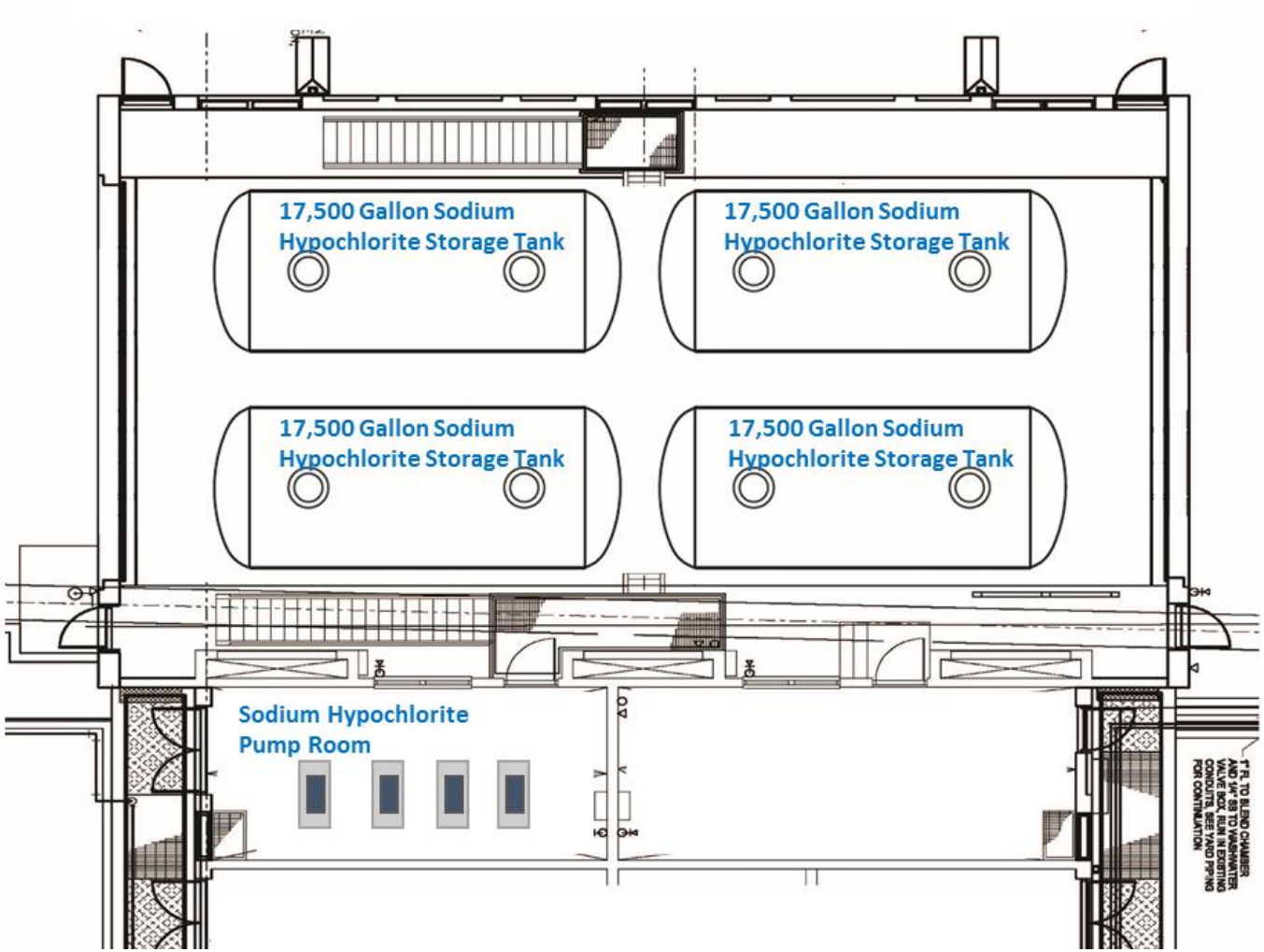
The option to purchase sodium hypochlorite in bulk and have regular deliveries to the plant is common in the industry, particularly for smaller treatment plants. Should this option be implemented, the sodium hypochlorite tanks could be located in the existing rail car storage facility, and the existing chlorinator and/or evaporator room converted to a pump room for the chemical feed pumps associated with this system. A conceptual layout of this option is shown on Figure 5.3A.

This option depicts four 17,500 gallon storage tanks in the location where the 90-ton rail cars now reside. This volume of storage (70,000 gallons) would provide approximately 12 days of available storage at 140 mgd using an average chlorine dosage of 6.5 mg/l and a 12.5 percent solution of sodium hypochlorite. The storage times at 2027 average plant flow rates (mid-point of 20-year life cycle) would be approximately 19 days.

3.2.1 Option B - Advantages and Disadvantages

The advantages of this option rest largely with its simplicity and improved safety from the standpoint of leaks of chlorine gas, although storing and feeding sodium hydroxide still must be approached with extreme caution as this chemical is hazardous, particularly at 12.5 percent concentration. The operation with simple feed pumps resembles many of the other types of feed systems, eliminating the two stage process with liquid chlorine where the chlorine is first vaporized with evaporators and then fed with gaseous chlorinators/injectors to the points of application.

The major disadvantage of this option is the cost of sodium hypochlorite, which increases the annual cost for disinfection as shown on the summary tables at the end of this TM.



OPTION B PROPOSED LAYOUT

FIGURE 5.3A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

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3.2.2 Option B - Capital and Operating Costs

The costs for a new on-site storage and feed system to accommodate bulk deliveries of sodium hypochlorite are shown in the Table 5.2A. These costs assume an initial $0.55/gallon for bulk sodium hypochlorite, which is the actual contracted costs (plus 5 percent) that Manatee County now pays for this same chemical, plus an escalation factor of 3 percent per year. Of note, this cost ($0.55/gallon) is much lower than most other areas of the U.S. which pay costs per gallon anywhere from $0.80 to well over $1.00.

Table 5.2A Option B Capital and Operating Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Capital Costs (x 1,000)	Annual O&M Costs (x 1,000)	Total Present Worth Costs (x 1,000)
Sodium Hypochlorite Storage Tanks (includes installation and containment)	$335	$0	$335
Sodium Hypochlorite Feed Pumps	$34	$0	$34
Chemical Building Renovation (three rooms)	$190	$0	$190
Annual Costs Sodium Hypochlorite	$0	$750	$11,162
Contingency and Contractor Overhead and Profit	$120	$0	$120
Total	**$679**	**$750**	**$11,841**

3.3 Option C - On-Site Generation of Sodium Hypochlorite (OSHG)

The option of generating sodium hypochlorite on-site is a proven technology, yet one that has had problems in the past largely due to designs that failed to properly vent the byproduct hydrogen gas from the generation system. Pockets of gas allowed to accumulate caused explosions at some installations. Some utilities, such as Palm Beach County WUD, decommissioned their on-site generation system over concerns of such an occurrence. Modern day designs however, have recognized this gas accumulation potential and incorporate provisions to properly vent this gas to the atmosphere.

Two types of systems are now commonly marketed and employed in the waterworks industry for on-site generation of sodium hypochlorite with the major difference being the concentration of sodium hypochlorite in the generated product; 12.5 percent solution or 0.8 percent solution.

3.3.1 On-Site Generation of 0.8 Percent Sodium Hypochlorite

On-site sodium hypochlorite generation involves use of salt, softened water, and electricity to generate chlorine equivalent (CE) solution on site, typically at 0.8 Solution strength of 0.8 percent is considered non-hazardous thus eliminating the need for OSHA risk management planning requirements associated with gaseous chlorine. A typical layout (sized for the DLTWTF) as offered by ClorTec® is shown in Figure 5.4A. The overall dimensions of this building are approximately 94 feet x 64 feet, or about 6,000 square feet (sq ft). The size of the existing rail car storage room, evaporator room, and chlorinator room combined is about 3,400 sq ft, slightly more than half of the needed space for the 0.8 percent solution alternative. As a result, a new on-site generation building would likely be necessary for this option.

3.3.2 On-Site Generation of 12.5 Percent Sodium Hypochlorite

Generation of high strength (12 percent) sodium hypochlorite is also an option. One example of a recent design by Carollo is the 15,000 ppd facility for Miami-Dade's new Northwest Wellfield 125 mgd (165 mgd ultimate) Nanofiltration WTP. This OSHG facility is nearly identical in size as what would be needed for the DLTWTF. Due to the additional complexity of the system (compared to the 0.8 percent option), the overall building dimensions are approximately 200 feet by 130 feet, hence, like the low strength option, a new building would be needed.

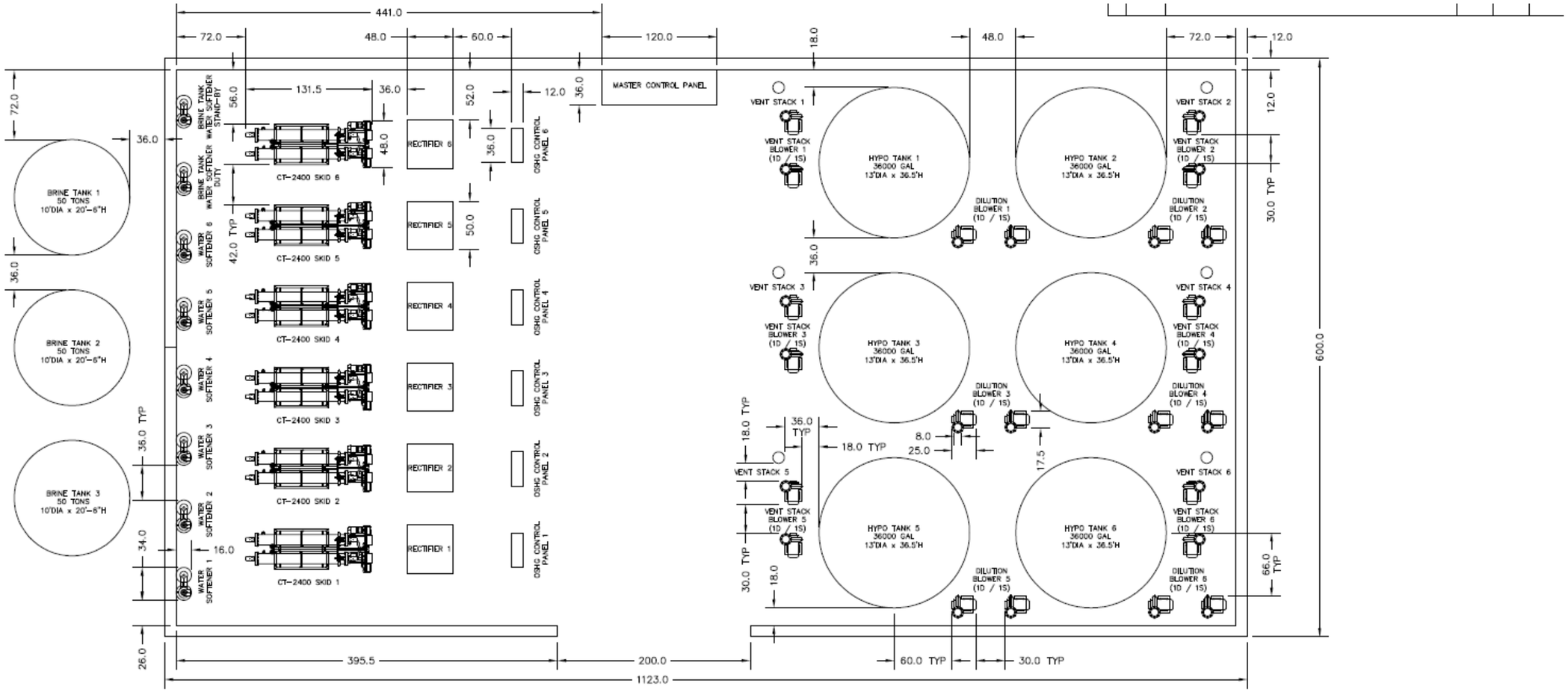
Figures 5.5A, and 5.6A are 3-D scalable drawings of this facility depicting the extensive amount of equipment and size of the building needed. Figure 5.6A depicts the building (less the roof) along with the housed equipment and tankage and one view shows predominantly the equipment without the walls, roof, and second floor slab in order to better visualize the system components.

3.3.3 Option C - Advantages and Disadvantages

Like the on-site storage of sodium hypochlorite delivered in bulk, this option also eliminates concerns regarding releases of chlorine gas from a catastrophic leak. Unlike the on-site storage option (Option B), however, the operation is not simple and both the low strength and high strength alternatives do have extensive equipment needs and require operator training and experience.

The key advantages rely almost entirely on reduced annual operating cost by eliminating the purchase of chemicals (liquid chlorine or sodium hypochlorite) in favor of the purchase of salt and electricity.

The major drawback of on-site generation is the capital cost of the equipment and building, along with the annual costs of salt and electricity.



NOTES (UNLESS OTHERWISE SPECIFIED)

DE NORA

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UNLESS OTHERWISE NOTED
ALL DIMENSIONS ARE IN INCHES
[MM] = REFERENCE

FRACTIONS	± 1/16"	±1.5mm
.X	± .1"	±2.5mm
.XX	± .01"	±0.51mm
.XXX	± .005"	±0.127mm
ANGLES	± 1°	

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SYTELINE ITEM NUMBER(S)

CLORTEC

CT-2400 (2400 LBS/DAY) RACK LAYOUT

SCALE:	DNWT PROJECT No:	SIZE:	REV:
NTS		D	0

DNWT DWG No: P-46997-ML-01-1

SHEET: 1 of 1

OPTION 3 CLORTEC LAYOUT

FIGURE 5.4A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

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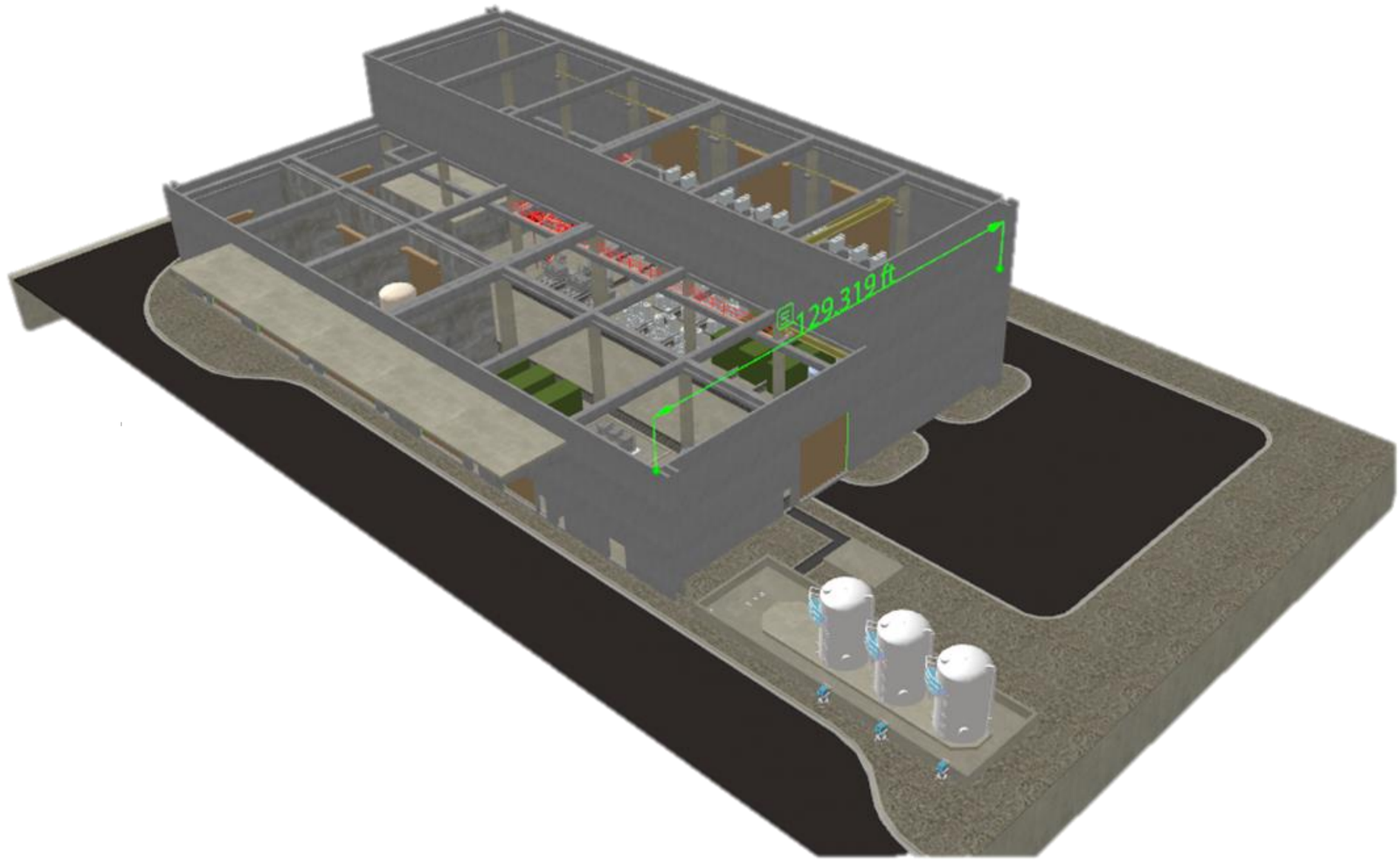

3D RENDERING OF OSHG FACILITY FOOTPRINT

FIGURE 5.5A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

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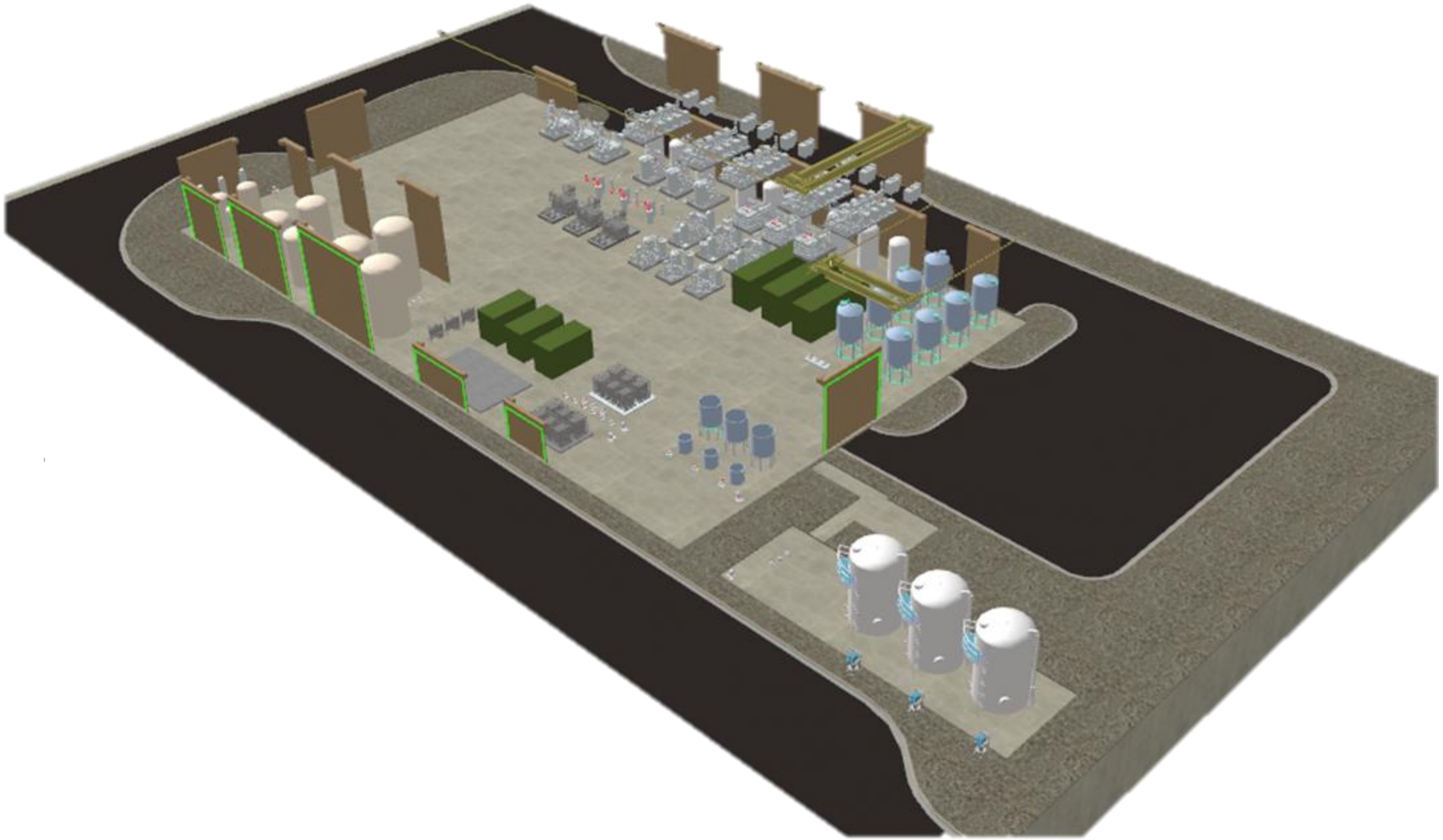

3D RENDERING OF OSHG FACILITY AND EQUIPMENT

FIGURE 5.6A

CITY OF TAMPA
DAVID L. TIPPIN MASTER PLAN

carollo

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3.3.4 Option C - Capital and Operating Costs

Table 5.3A, shown below, is based largely on the less expensive 0.8 percent OSHG option. This option is somewhat less efficient than the 12.5 percent OSHG system, but comes with a lower capital cost.

Table 5.3A Option C Capital and Operating Costs
David L. Tippin Water Treatment Facility Master Plan
City of Tampa

Item	Capital Costs (x 1,000)	Annual O&M Costs (x 1,000)	Total Present Worth Costs (x 1,000)
Decommission Existing Chlorine Storage and Feed System	$300	-	$300
New On-Site Sodium Hypochlorite Generation Building	$1,200	-	$1,200
Six 2400 lbs. /day on-site sodium hypochlorite generators and associated equipment and tankage (includes installation)*	$3,640	-	$3,640
Salt Usage	-	$284	$4,231
Power Consumption	-	$279	$4,154
Sitework, Electrical & Instrumentation	$500	-	$500
Contingency and Contractor Overhead and Profit	$1,300	-	$1,300
Total	**$6,940**	**$564**	**$15,326**

4.0 SUMMARY AND RECOMMENDATIONS

4.1 Comparison of Options and Costs

The comparative capital, annual O&M, and present worth costs of the three options described in this technical memorandum are included in Table 5.4A, along with comments regarding key advantages and disadvantages of each alternative.

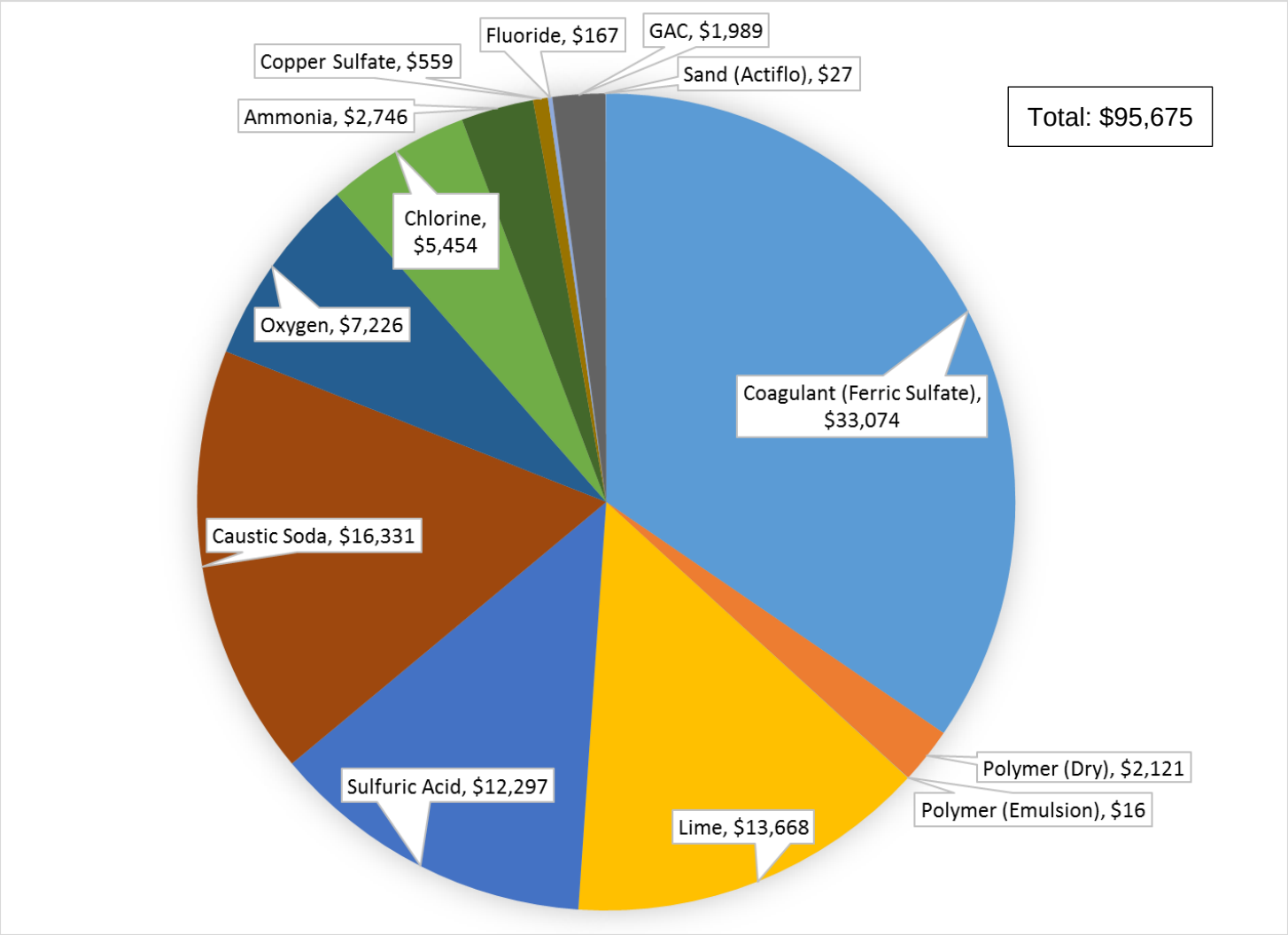
Table 5.4A Comparison of Options and Costs David L. Tippin Water Treatment Facility Master Plan City of Tampa				
Option & Description	**Capital Costs (x 1,000)**	**Annual O&M Costs (x 1,000)**	**Total Present Worth Costs (x 1,000)**	**Comments**
Option A - Chlorine Scrubber and Maintain the Existing Storage and Feed System	$2,825	$697	$13,197	Maintains existing method of chlorination Requires storage of large volumes of Caustic Soda that must be protected from degradation Risk of chlorine gas release still potential if the scrubber system fails or loses power
Option B - On-Site Storage of Sodium Hypochlorite (delivered in bulk)	$589	$750	$11,841	Simplest operation with bulk storage and feed pumps similar to other plant systems Price for Sod Hypo in Florida is much lower than most of the U.S. leading to concerns of future price escalation Size of tankage evaluated will require deliveries between two and three times each month
Option C - New On-Site Sodium Hypochlorite Generation System	$6,940	$564	$15,326	Most costly of all options on both capital and present worth costs; least costly on annual operating costs Most complex of all operating systems and additional O&M labor is not included in these costs.

4.2 Recommendations

From a cost standpoint given the current pricing of sodium hypochlorite, the least cost option is on-site storage of this chemical. However, the biggest risk that could impact the cost effectiveness of this option is the cost of the delivered chemical. For example, if costs for Sodium Hypochlorite were closer to that in other areas examined as part of this evaluation (i.e., Texas) where the costs were much higher ($0.85/gallon versus $0.55/gallon), then this option would have a total present worth cost of about $17 million; an increase of about 50 percent over the value shown in the table. Thus, over the long term, the attractiveness of this alternative is almost solely dependent upon future price indices of this chemical. At about $0.75/gallon, the present worth costs of storing this chemical versus on-site generation are essentially equal.

Both on-site storage of sodium hypochlorite (Option B) and on-site generation of sodium hypochlorite (Option C) are considered preferable to maintaining 90-ton rail cars of liquid chlorine, even with a scrubber system. This is based on both costs and exposure to risks. Either Option B or C are considered solid and viable options with Option C, on-site generation, offering the best life cycle costs should the price of sodium hypochlorite purchased in bulk approach the national norm. It is recommended that the City proceed with planning for an on-site generation system (Option C), but retain the option to revisit this recommendation based on price trends and cost projections of bulk sodium hypochlorite when this project is under conceptual design.

APPENDIX 6H – NORMALIZED CHEMICAL COMPONENT COST ($/MGD AADF)



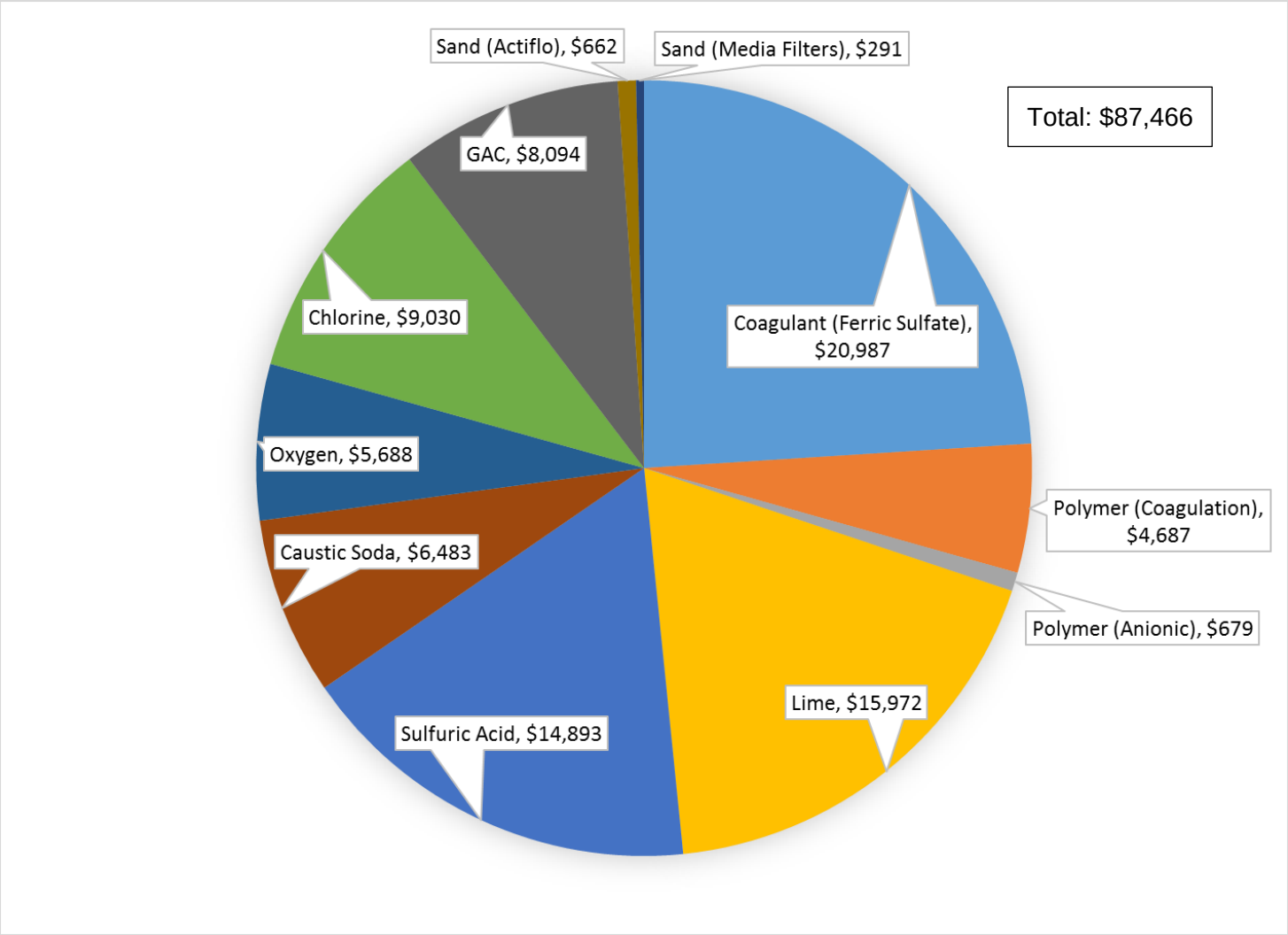
DLTWTF - NORMALIZED CHEMICAL COST

FIGURE 6H.1

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.1



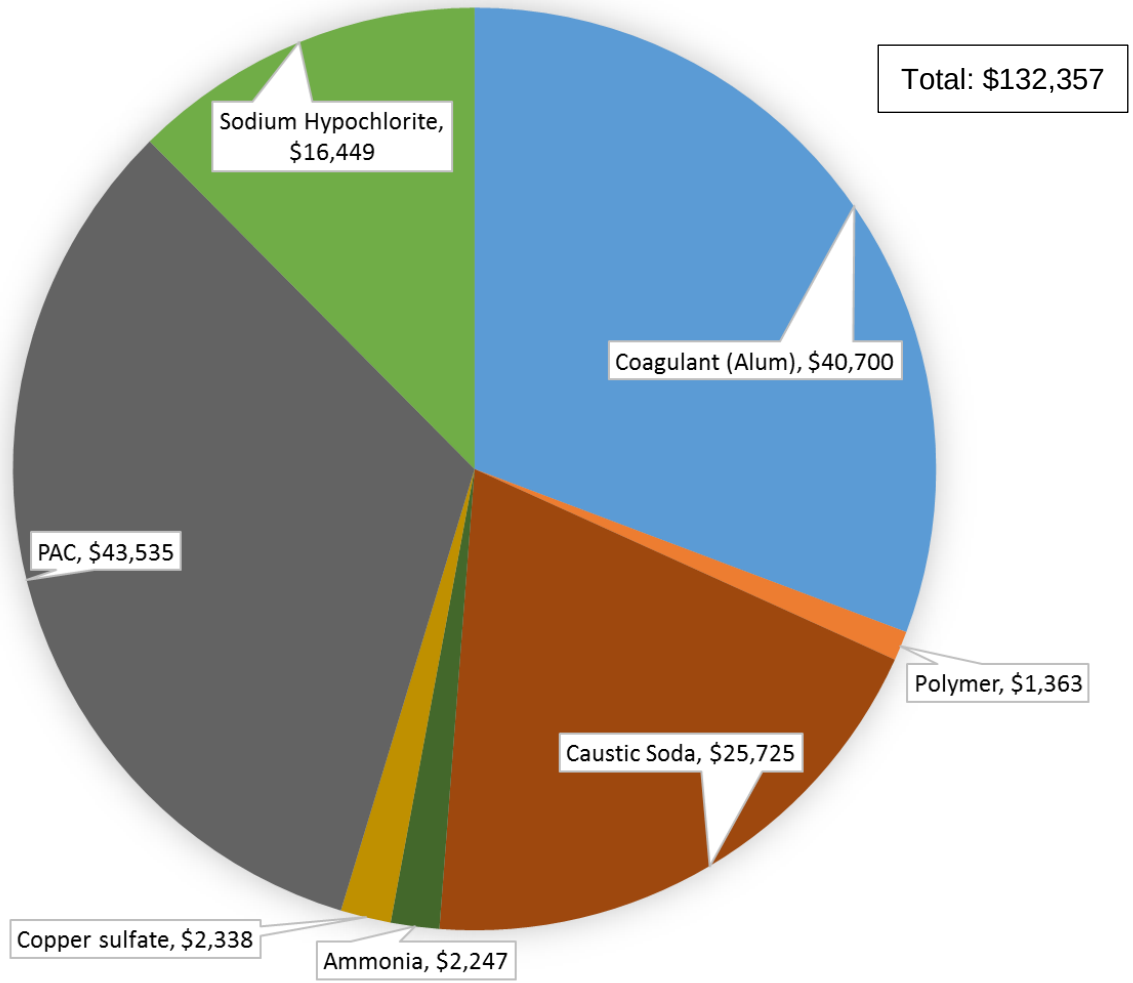
TAMPA BAY WTP - NORMALIZED CHEMICAL COST

FIGURE 6H.2

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.2



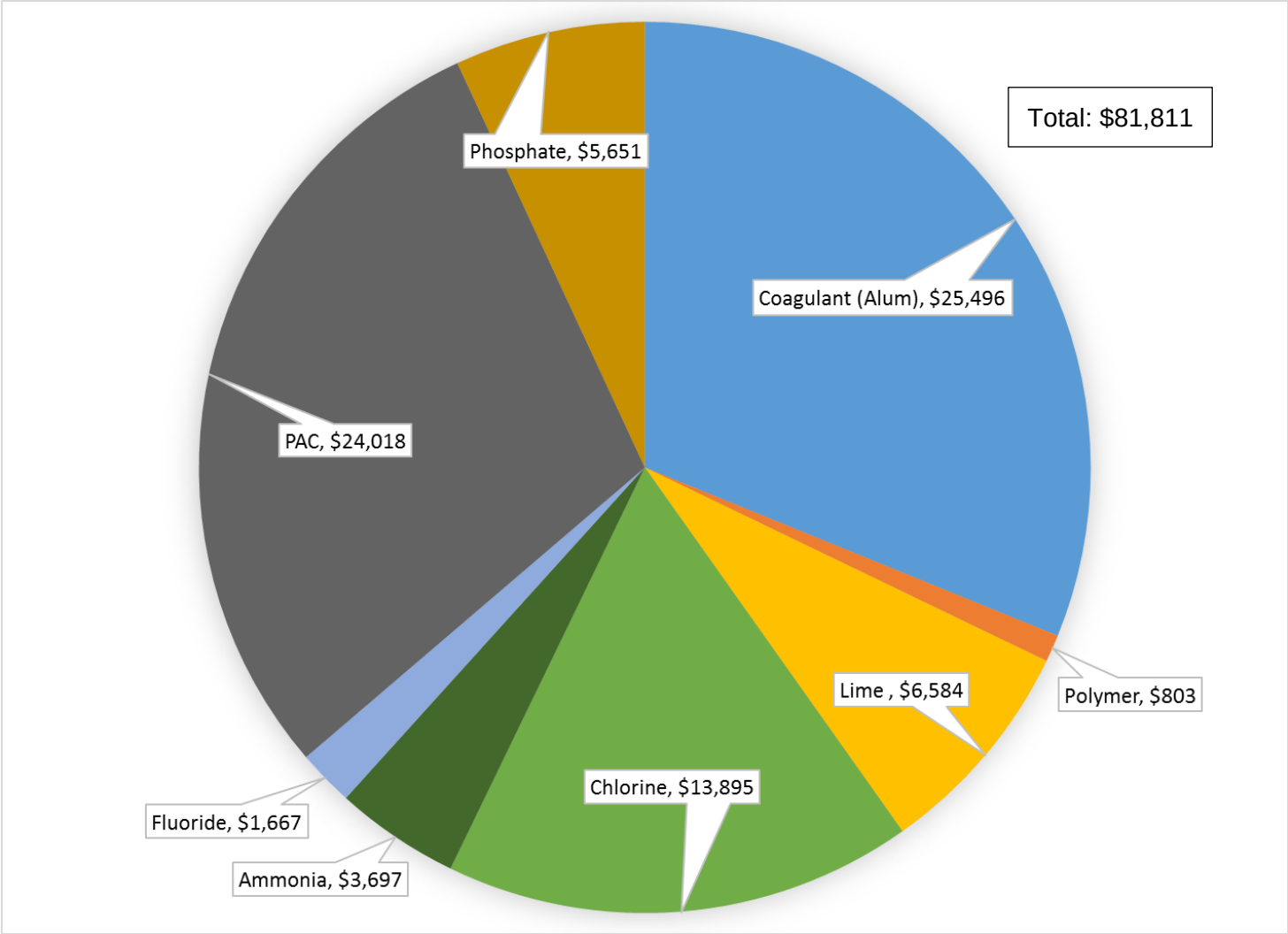
PEACE RIVER WTP - NORMALIZED CHEMICAL COST

FIGURE 6H.3

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.3



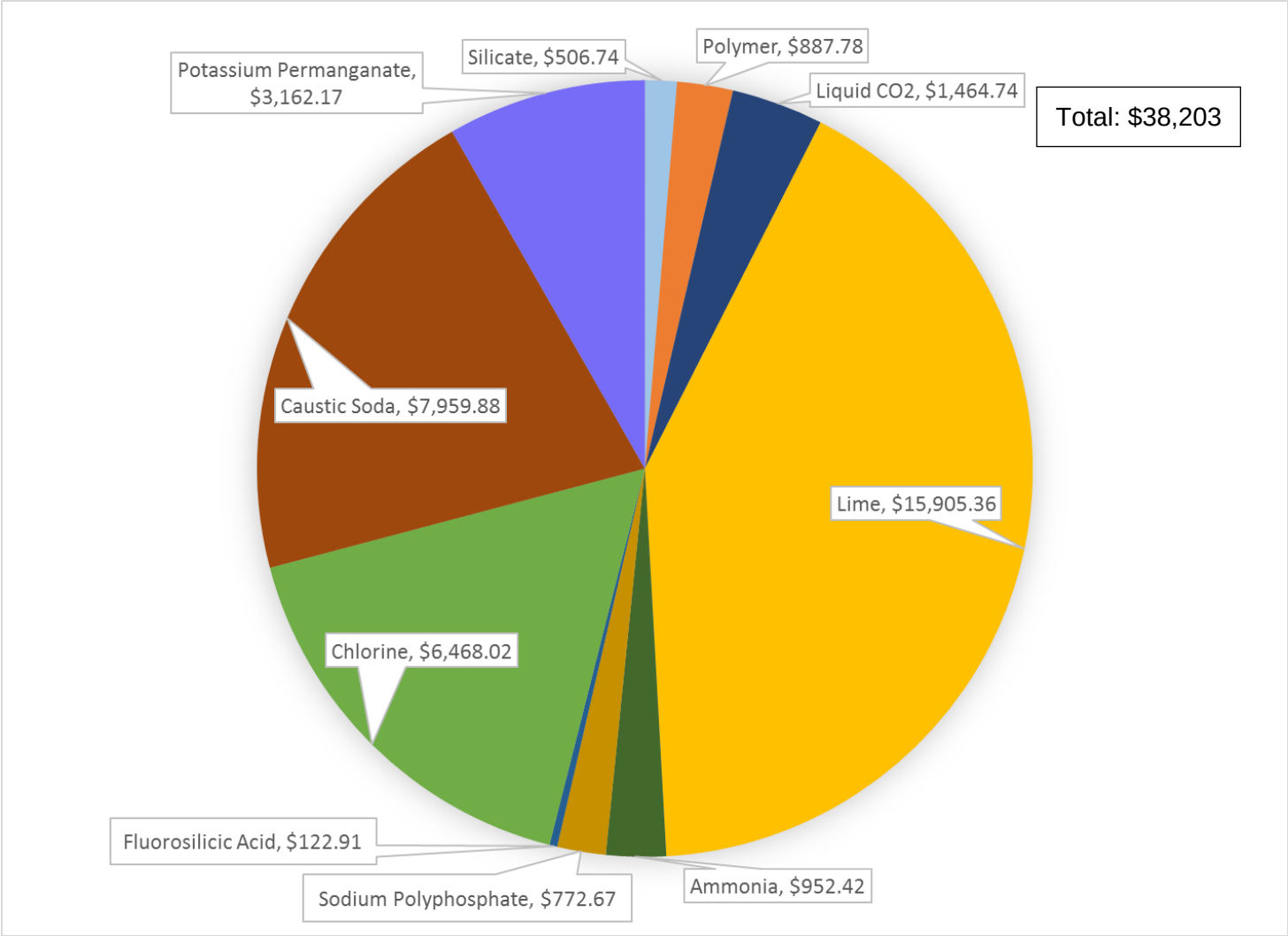
LAKE MANATEE WTP - NORMALIZED CHEMICAL COST

FIGURE 6H.4

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.4



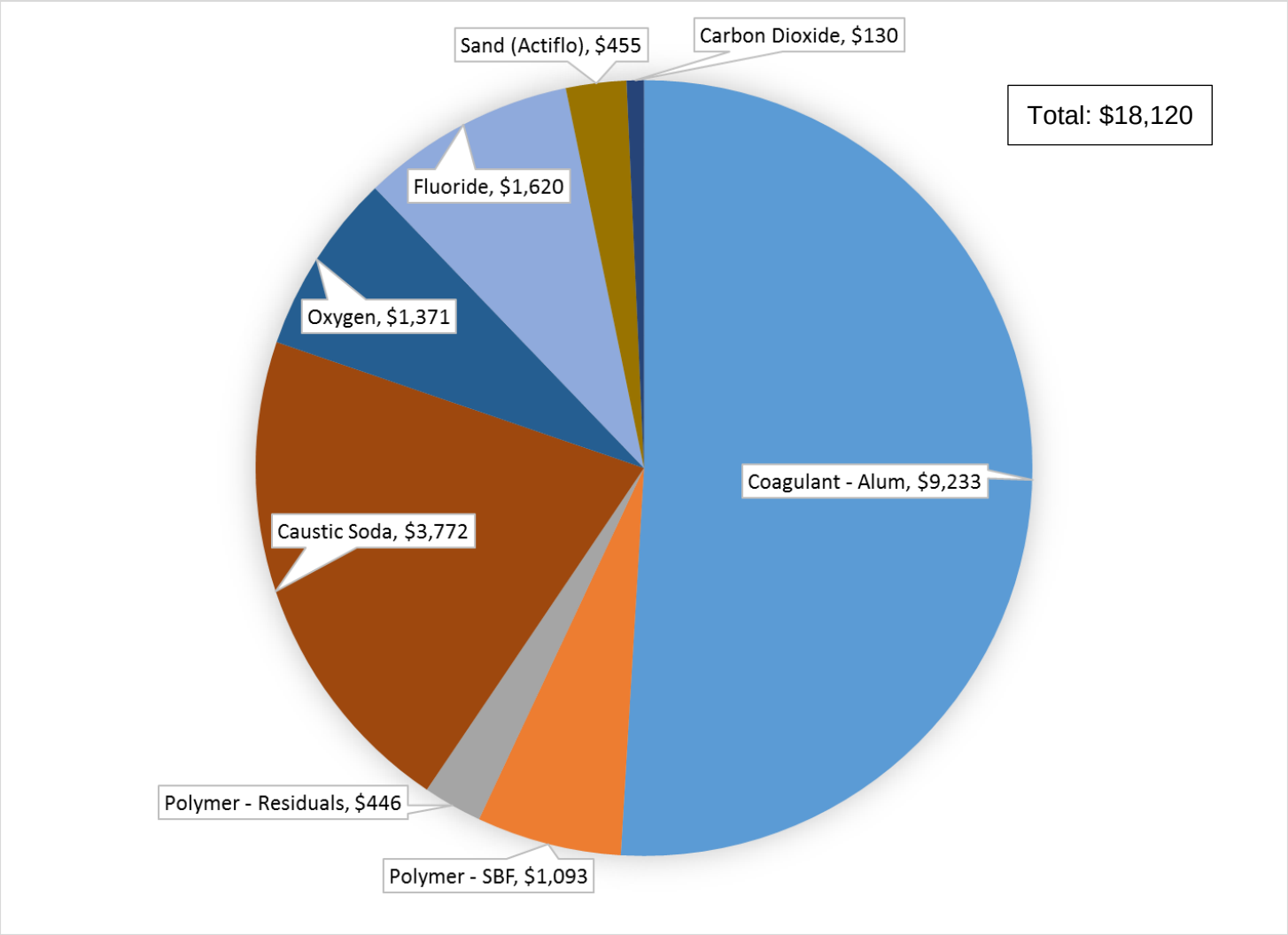
PRESTON AND HIALEAH WTPs - NORMALIZED CHEMICAL COST

FIGURE 6H.5

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.5



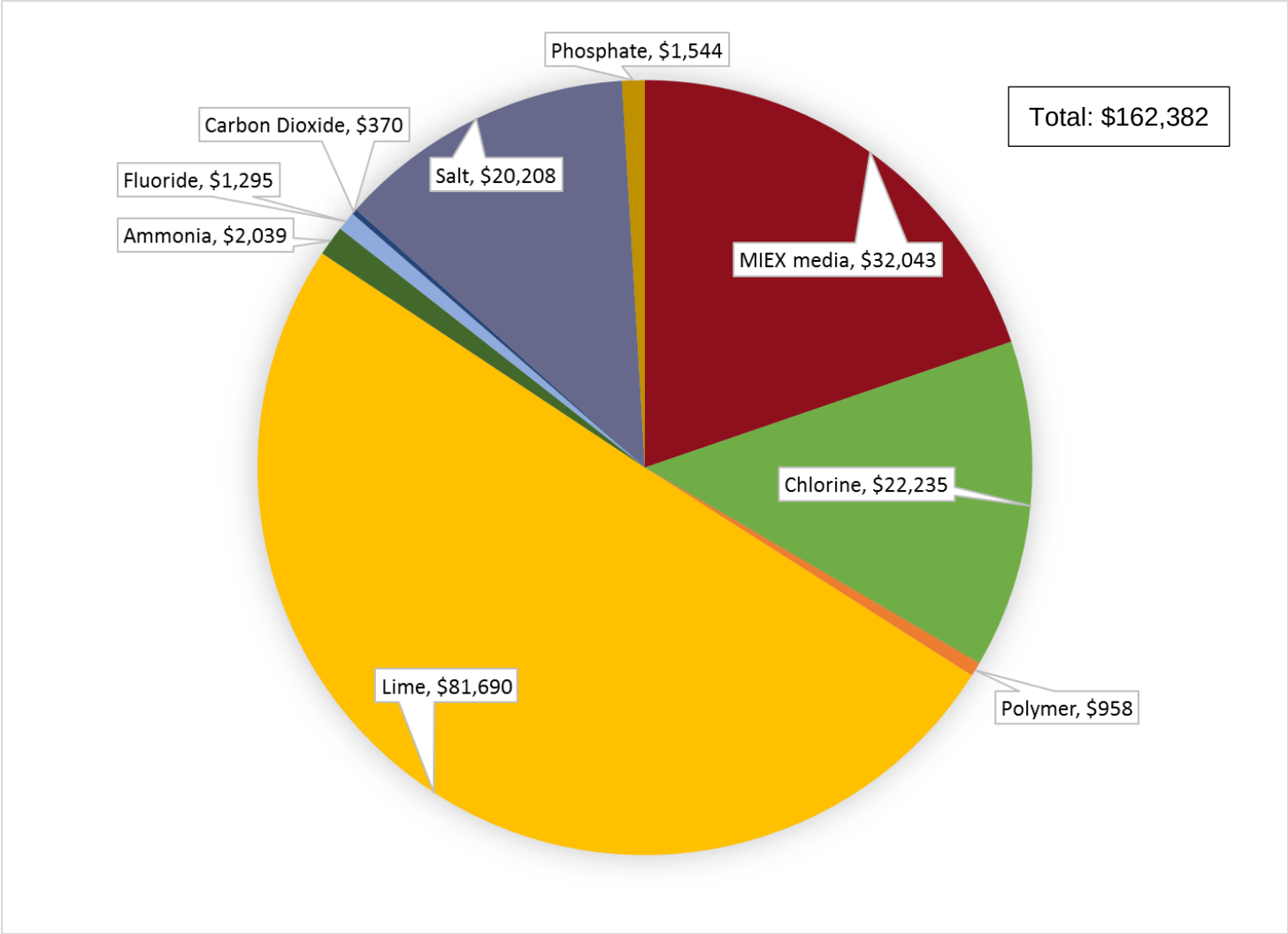
SANTAN VISTA WTF - NORMALIZED CHEMICAL COST

FIGURE 6H.6

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.6



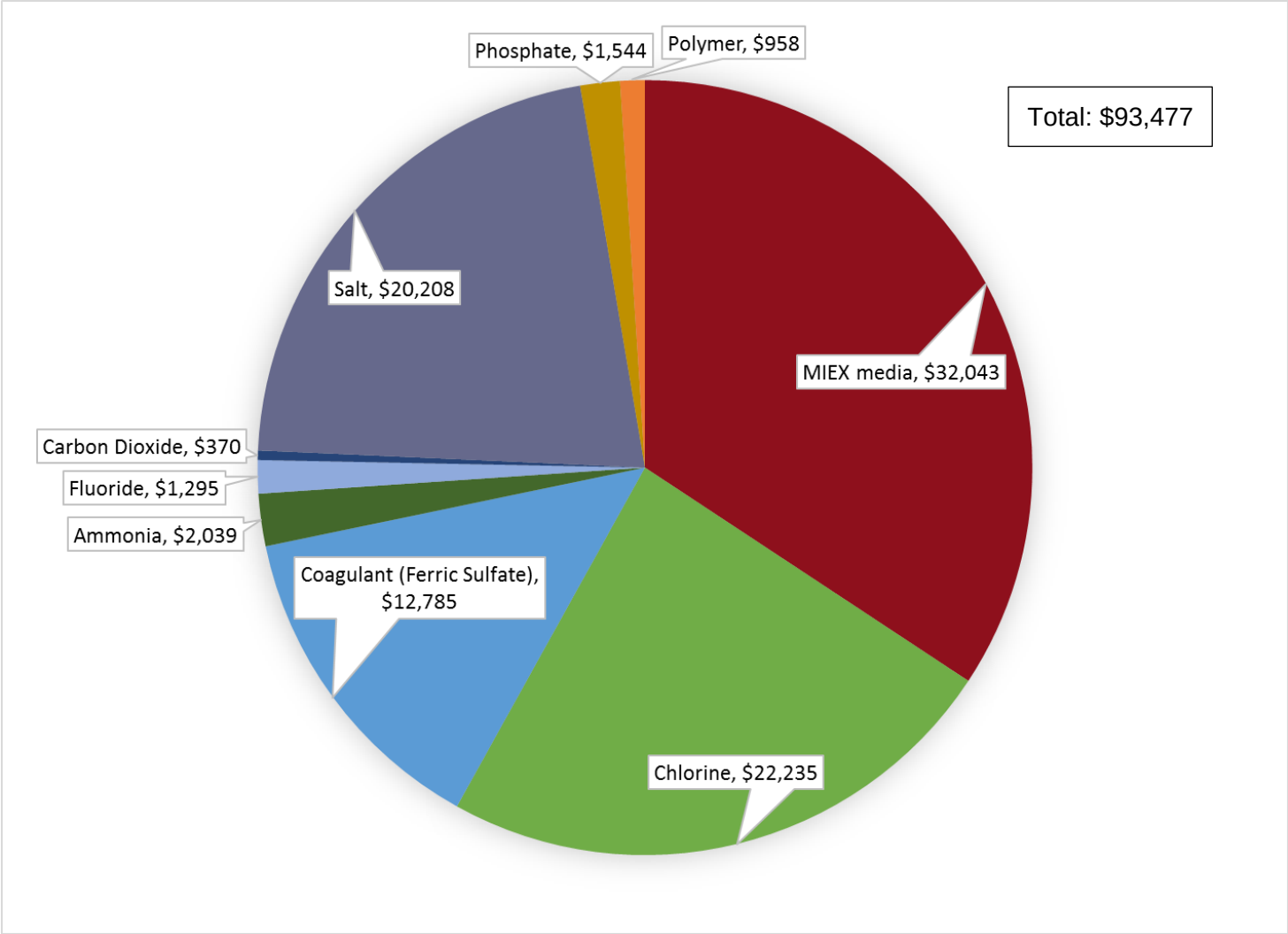
PALM BEACH COUNTY WTP - NORMALIZED CHEMICAL COST

FIGURE 6H.7

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.7



PALM BEACH COUNTY WTP - NORMALIZED CHEMICAL COST (ADJUSTED)

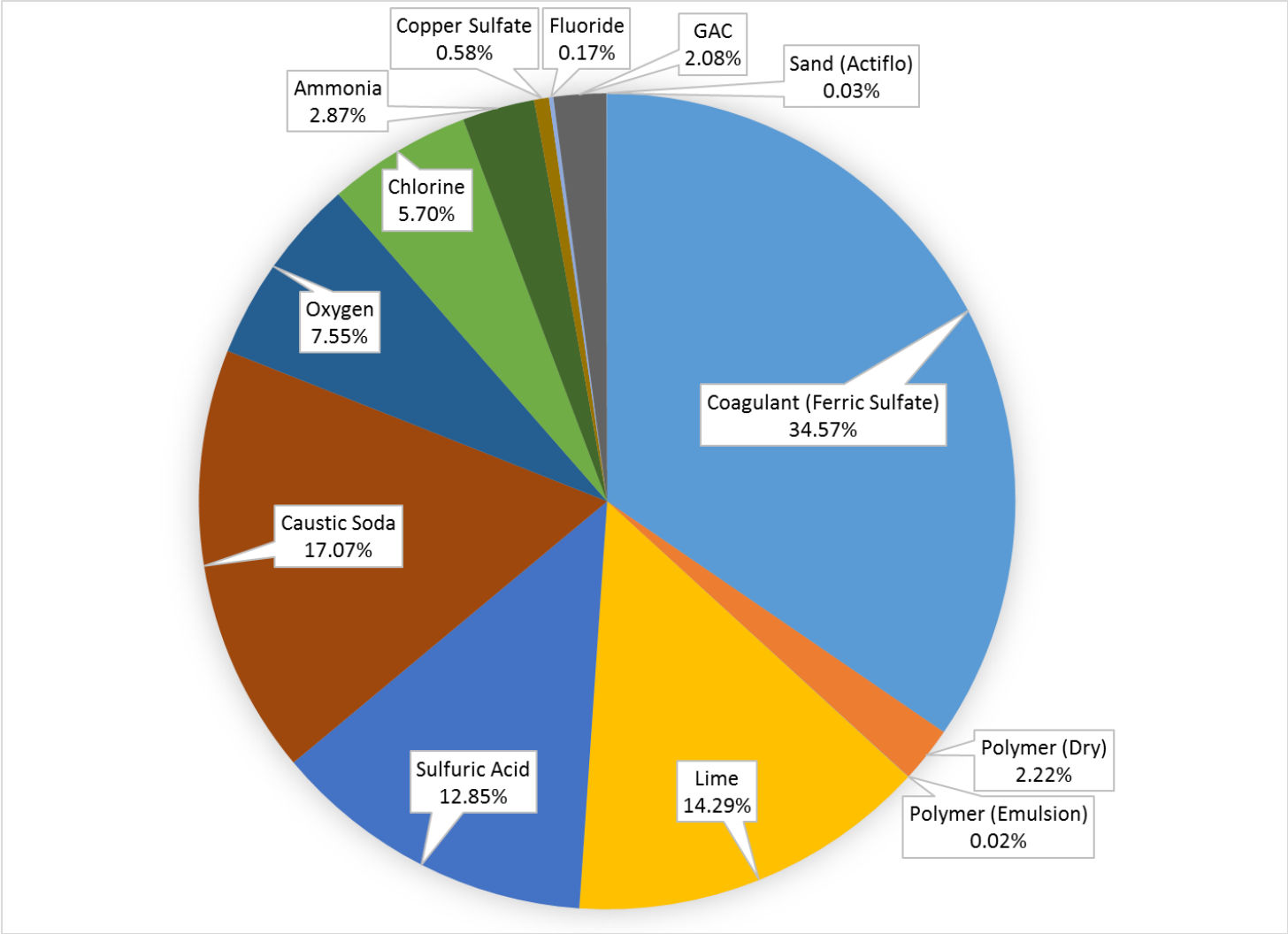
FIGURE 6H.8

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6A.8

APPENDIX 6I – PERCENT CHEMICAL COMPONENT COST



DLTWTF - CHEMICAL COST

FIGURE 6I.1

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

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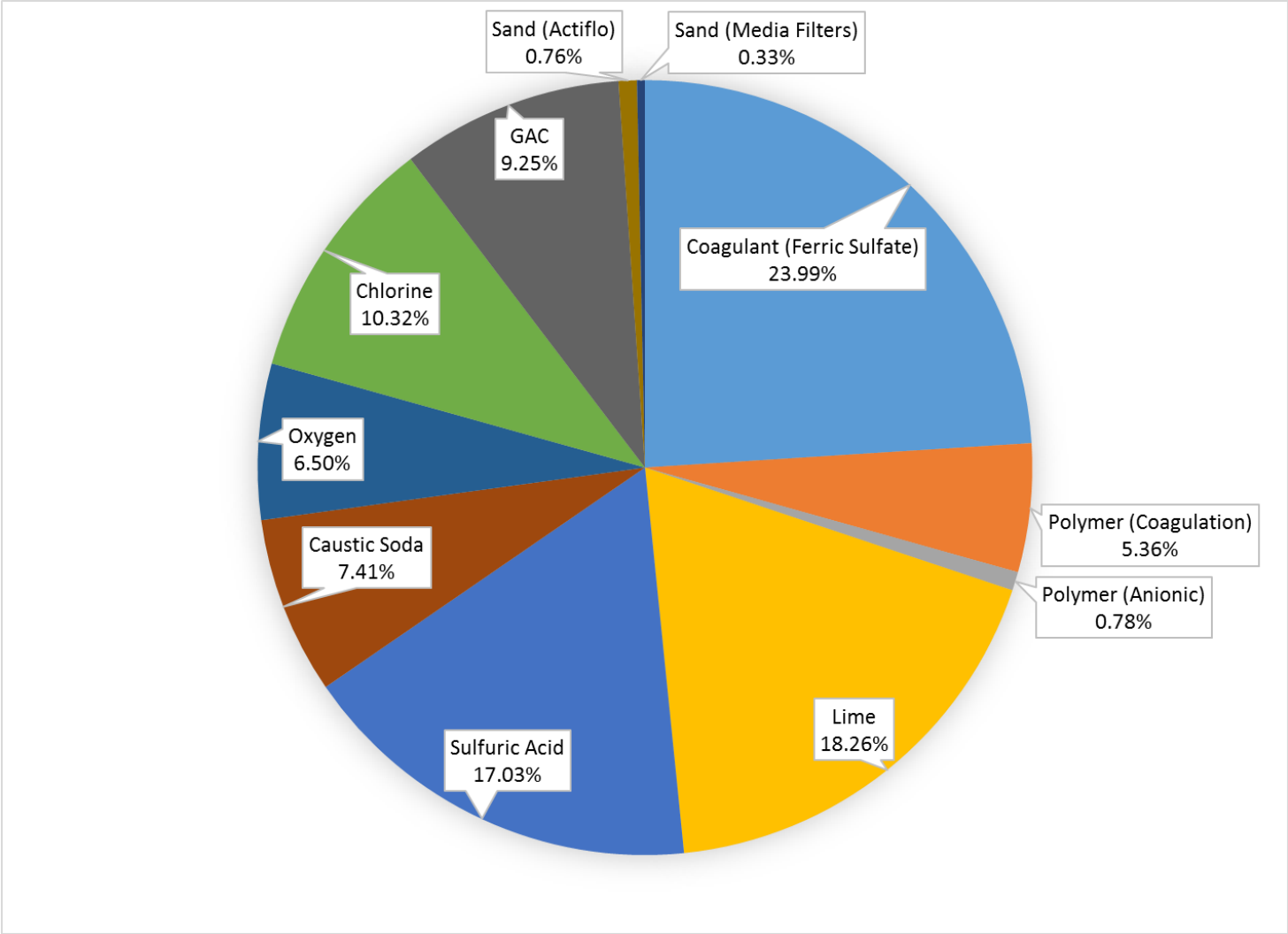


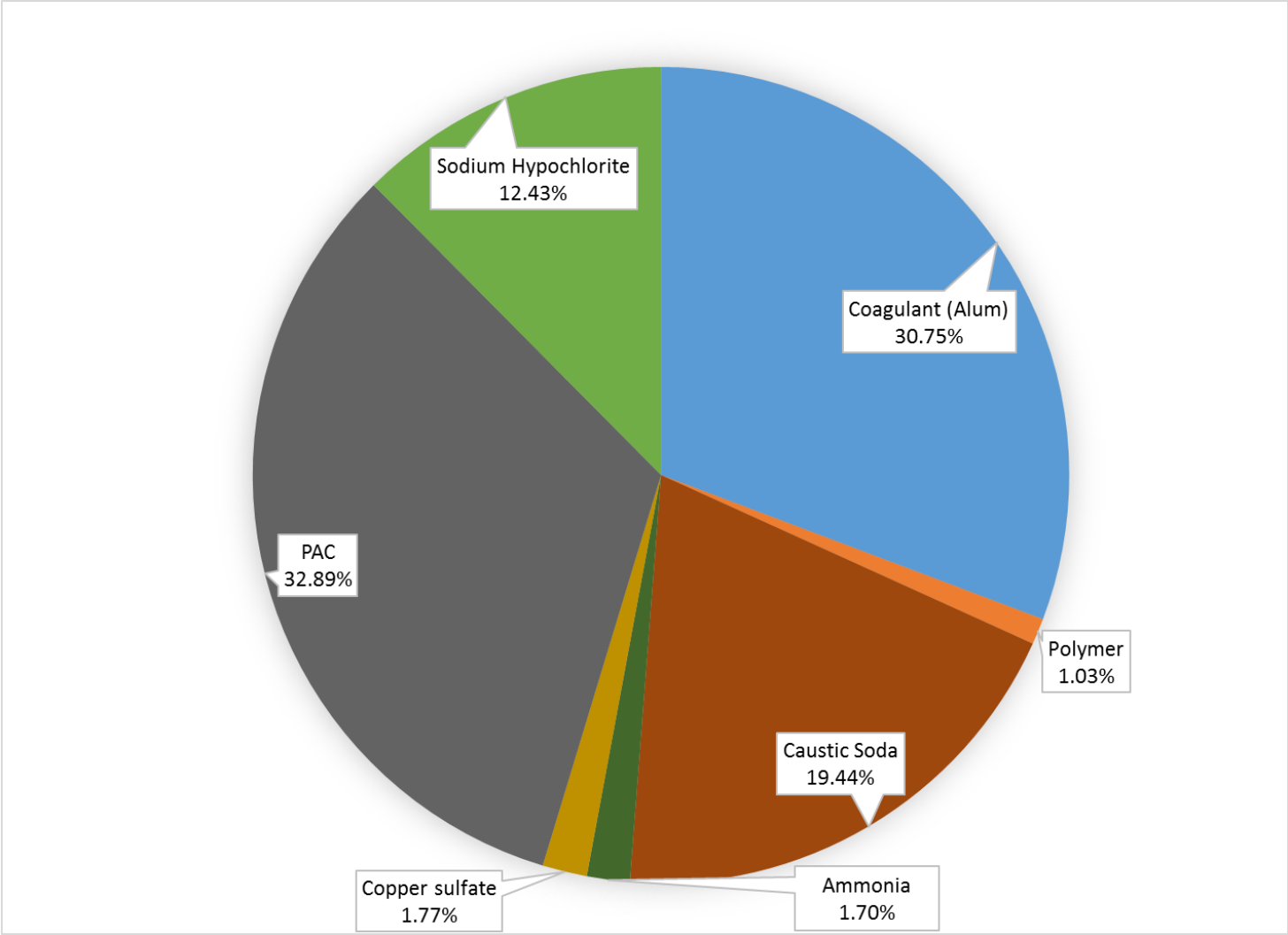
TAMPA BAY WTP - CHEMICAL COST

FIGURE 6I.2

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.2



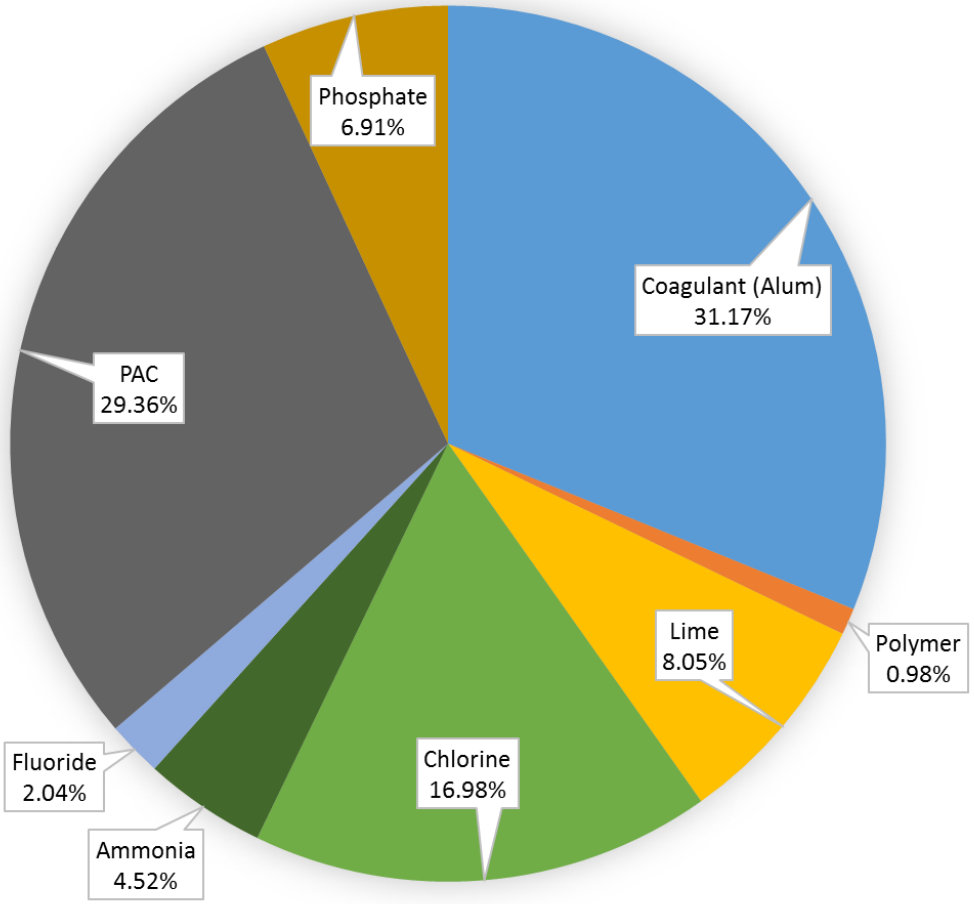
PEACE RIVER WTP - CHEMICAL COST

FIGURE 6I.3

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.3



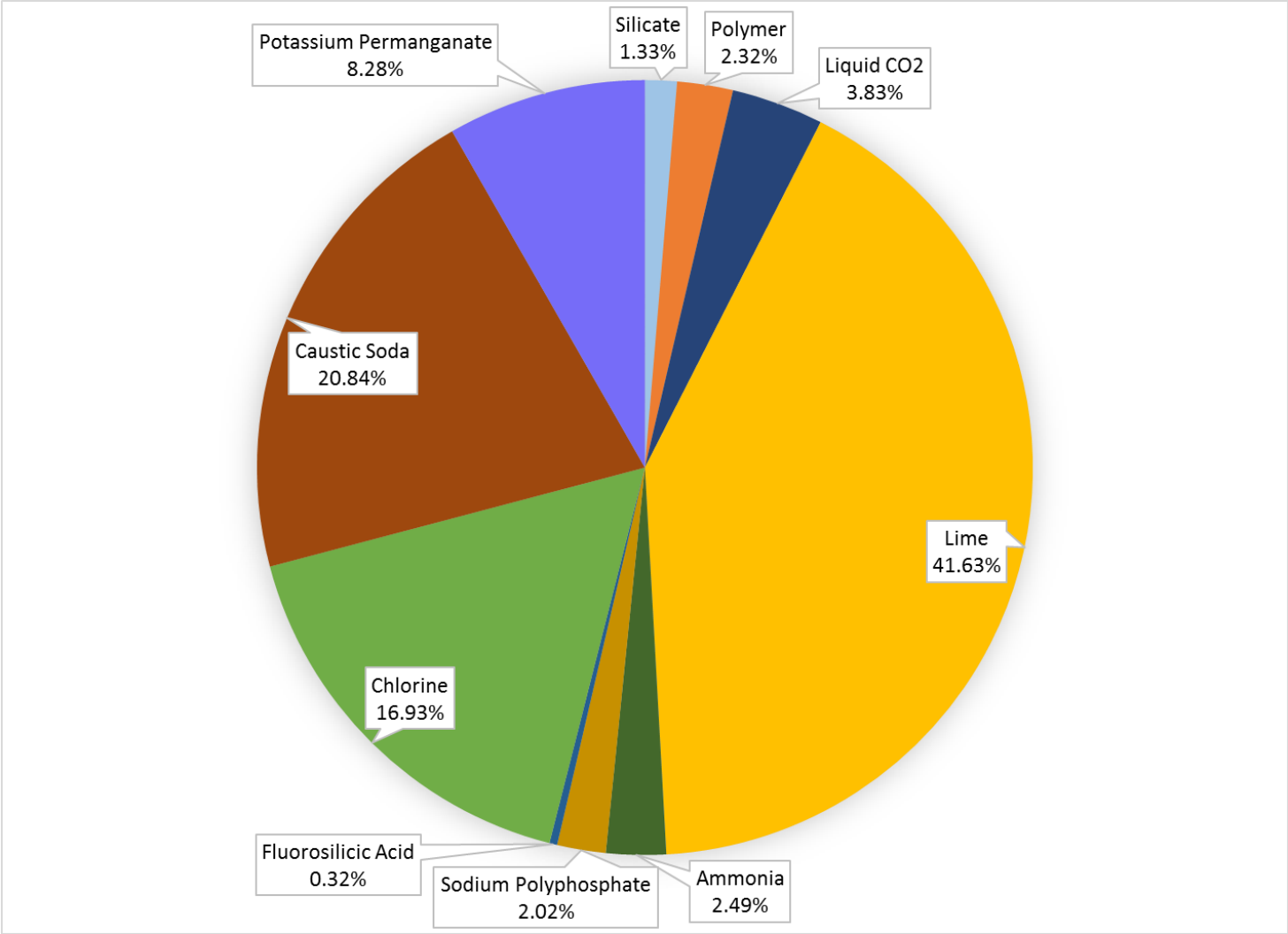
LAKE MANATEE WTP - CHEMICAL COST

FIGURE 6I.4

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.4



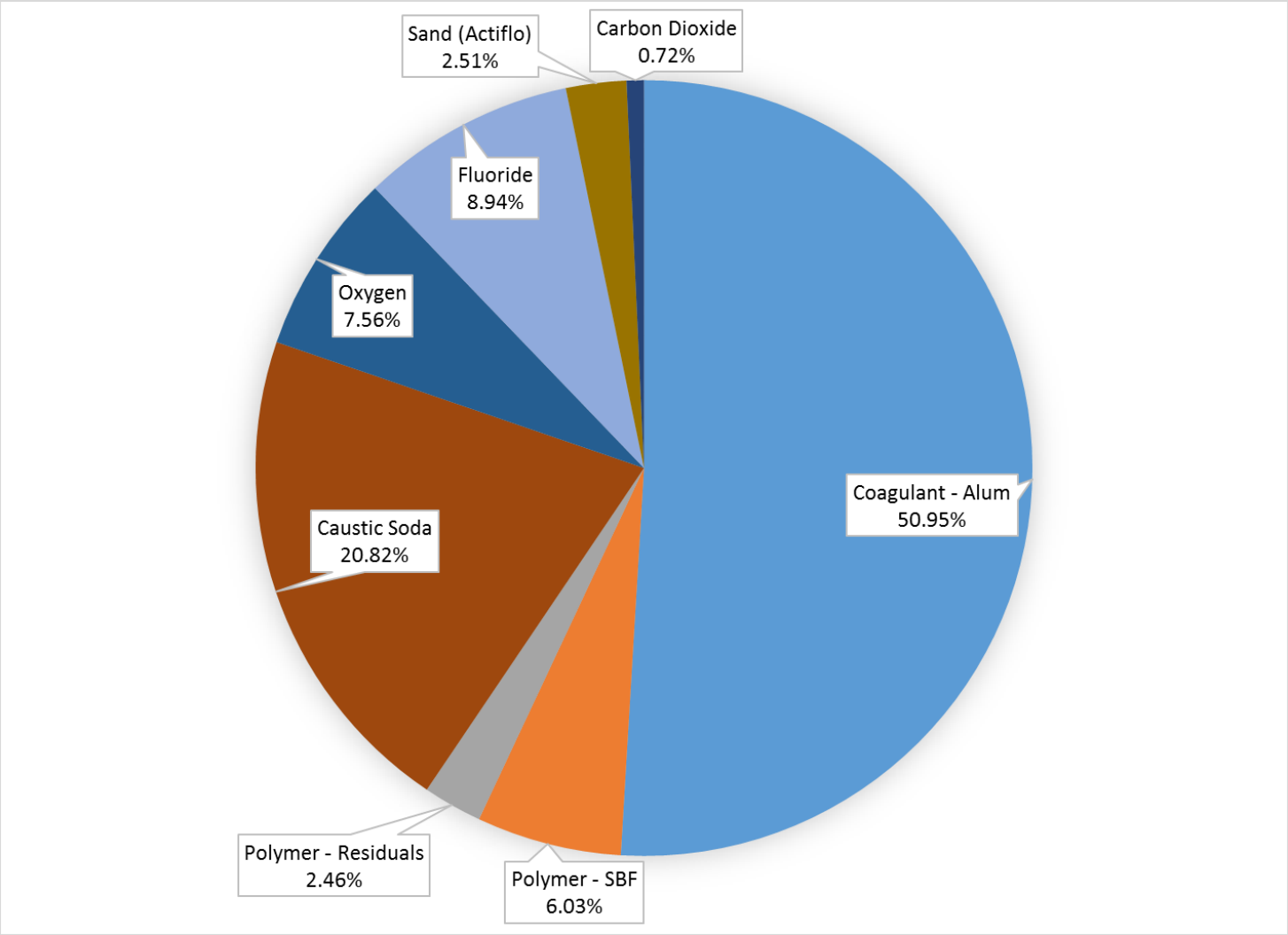
PRESTON AND HIALEAH WTPs - CHEMICAL COST

FIGURE 6I.5

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.5



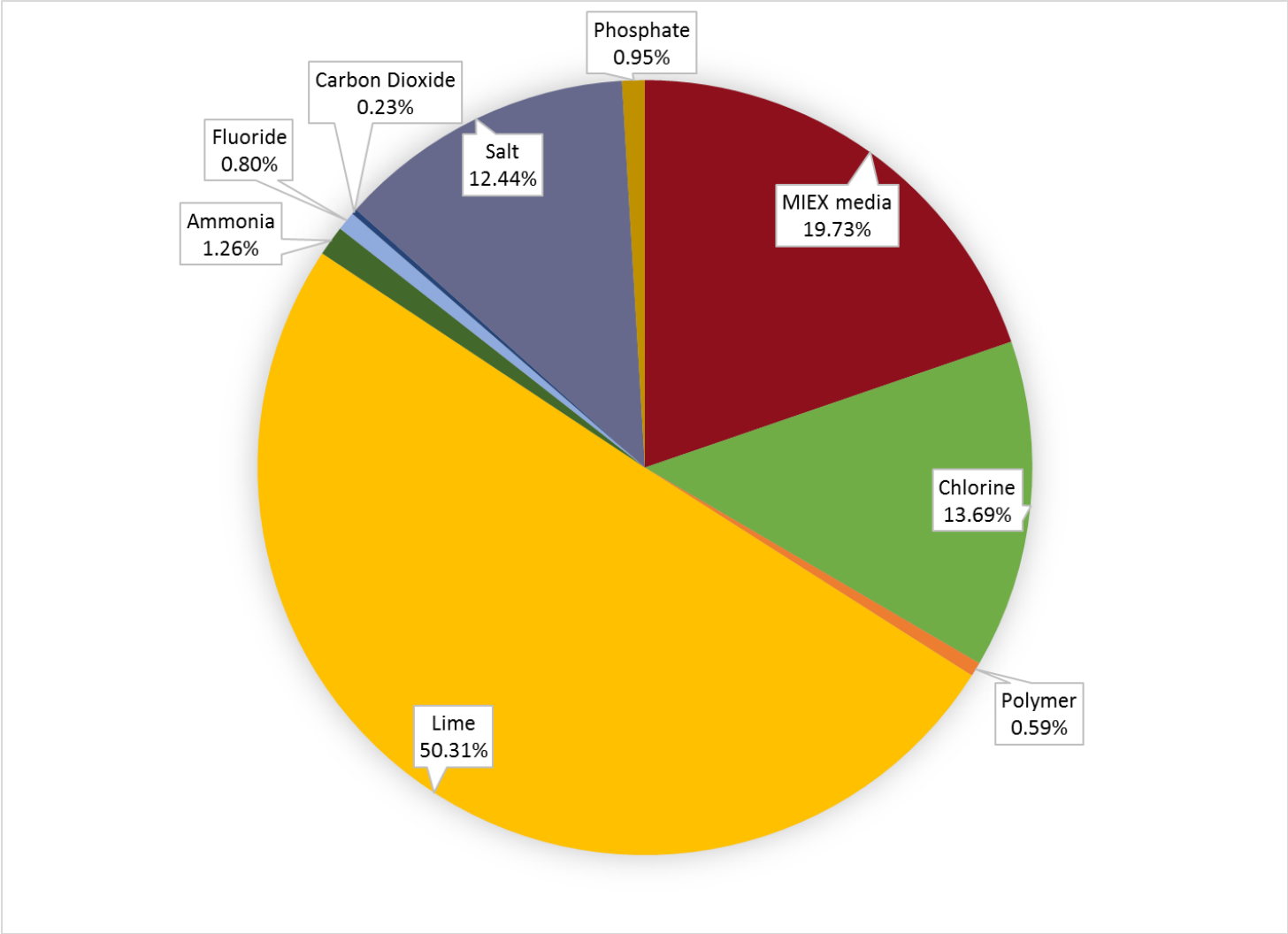
SANTAN VISTA WTF - CHEMICAL COST

FIGURE 6I.6

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.6



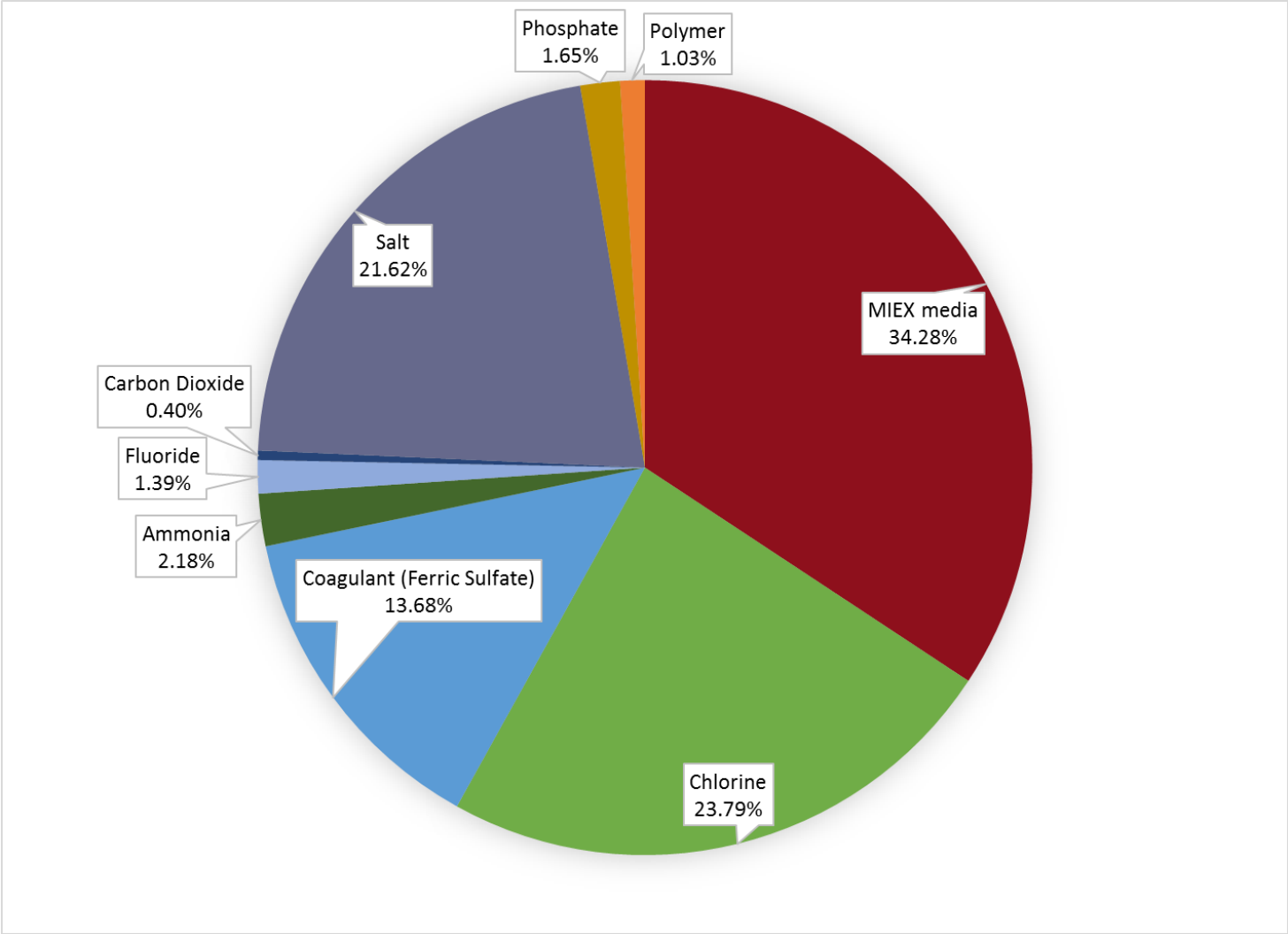
PALM BEACH COUNTY WTP - CHEMICAL COST

FIGURE 6I.7

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.7



PALM BEACH COUNTY WTP - CHEMICAL COST (ADJUSTED)

FIGURE 6I.8

CITY OF TAMPA
DAVID L TIPPIN WTF MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/FL/Tampa/10194A00/Deliverables/CH06/Graphics\Fig 6B.8