APTIM

SUBJECT

Fort Bend LID 19 – Phase 3 Recommendations

DATE

June 3, 2019

FROM

Hilary Thibodeaux, PE / Laura Barnes, PE

TO

Fort Bend County LID 19
c/o Nancy Carter

STATE OF TEXAS
LAURA L. BARNES
91094
6-3-19
LICENSED
PROFESSIONAL ENGINEER
Laura L. Barnes P.E.

1.0 EXECUTIVE SUMMARY

This memorandum represents Aptim Environmental & Infrastructure, LLC (APTIM)'s final review phase of a three-phased review process of Levee Improvement District No. 19 (LID 19) of Fort Bend County, Texas facilities, Phase 3 Recommendations. The purpose of this phase was to develop recommendations to address any findings noted during the Phase 1 review of the LID 19 facilities. During the Phase 1 review, it was determined that the Interconnected Channel and Pond Routing (ICPR) Model used to design the system had issues requiring further investigation. In addition, it was determined that a larger coincidental design storm event could be used to develop the required pump capacities. As a result, the focus in Phase 3 was to provide recommendations for the required pumping capacities for a coincidental event that met Fort Bend County Drainage Criteria Model (FBCDCM) criteria. As requested, recommendations for pumping capacities to exceed criteria is also provided. **It is important to note that the base conditions for all scenarios considered were as follows; non-wet (dry), all design storage was available, and the various design storms were modeled as single-occurrence events. As such, if repeated rainfall events were to occur before the system returned to the non-wet (dry) state, the required pumping capacities would be in excess of those listed in the summary table.**

During the Phase 3 process, the Atlas 14 rain data was released by The National Oceanic and Atmospheric Administration (NOAA) and is currently being evaluated by Fort Bend County Drainage District (FBCDD) to replace the current Technical Paper 40 (TP40) rain data. Although not yet adopted, we did included the Atlas 14 rain data in our analysis.

After completing the analysis, we ultimately recommend proceeding with implementation of a supplemental pump station just north of the current Steep Bank Creek pump station. This station should be designed to meet current criteria of a 10-year rain event utilizing TP40 data. The conceptual configuration of the supplemental pump station includes three (3) 30,000GPM pumps. The third 30,000GPM pump is required to satisfy the FBCDCM criteria of having a spare of the largest pump in the event one is out of service. The total pumping capacity of this combined station would be 170,000GPM, which includes the necessary spare pump. We also recommend incorporating into the design an upgrade option in anticipation of the acceptance of the Atlas 14 data or changes to the FBCDCM criteria.

With regard to pumping capacities exceeding criteria, we were able to demonstrate the magnitude of these increases as the design storm increased. Conceptually, a pump station located near Pond "O" can supplement the Steep Bank Creek pump station to exceed current FBCDCM criteria; however, more analysis on the effects of the outfall on the Flat Bank Diversion Canal will have to be performed. It is also recommended to utilize the completed 2D model analysis of the regional watershed to better understand all the inflow contributors to Steep Bank Creek.

Summary of Design Storm Analyses and Pumping Capacities

Coincidental Scenario	ICPR Model Requirements	Current Pumping Capacity	Supplemental Pumping Needs (includes spare pump)	New Total Pumping Capacity (includes spare pump)
10-Year TP40*	139,000	80,000 GPM	90,000 GPM (3 x 30,000 GPM pumps)	170,000 GPM
10- Year Atlas 14	157,000	80,000 GPM	120,000 GPM (4 x 30,000 GPM pumps)	200,000 GPM
25-Year TP40	257,000	80,000 GPM	240,000 GPM (4 x 60,000 GPM pumps)	320,000 GPM
100-Year TP40	450,000	80,000 GPM	460,000 GPM (3 x 30,000 GPM pumps, 6 x 60,000 GPM pumps and 1 x 10,000 GPM pump)	170,000 GPM + 370,000 GPM at Pond "O"
100-Year Atlas 14	690,000**	80,000 GPM	TBD with 2-D Regional Model	

* Recommended pumping improvements

** Does not account for all inflows

2.0 INTRODUCTION

APTIM was selected by LID 19 to provide a multi-phase review of the existing LID 19 facilities. Below, Figure 2.1 represents LID 19's drainage area and facilities.

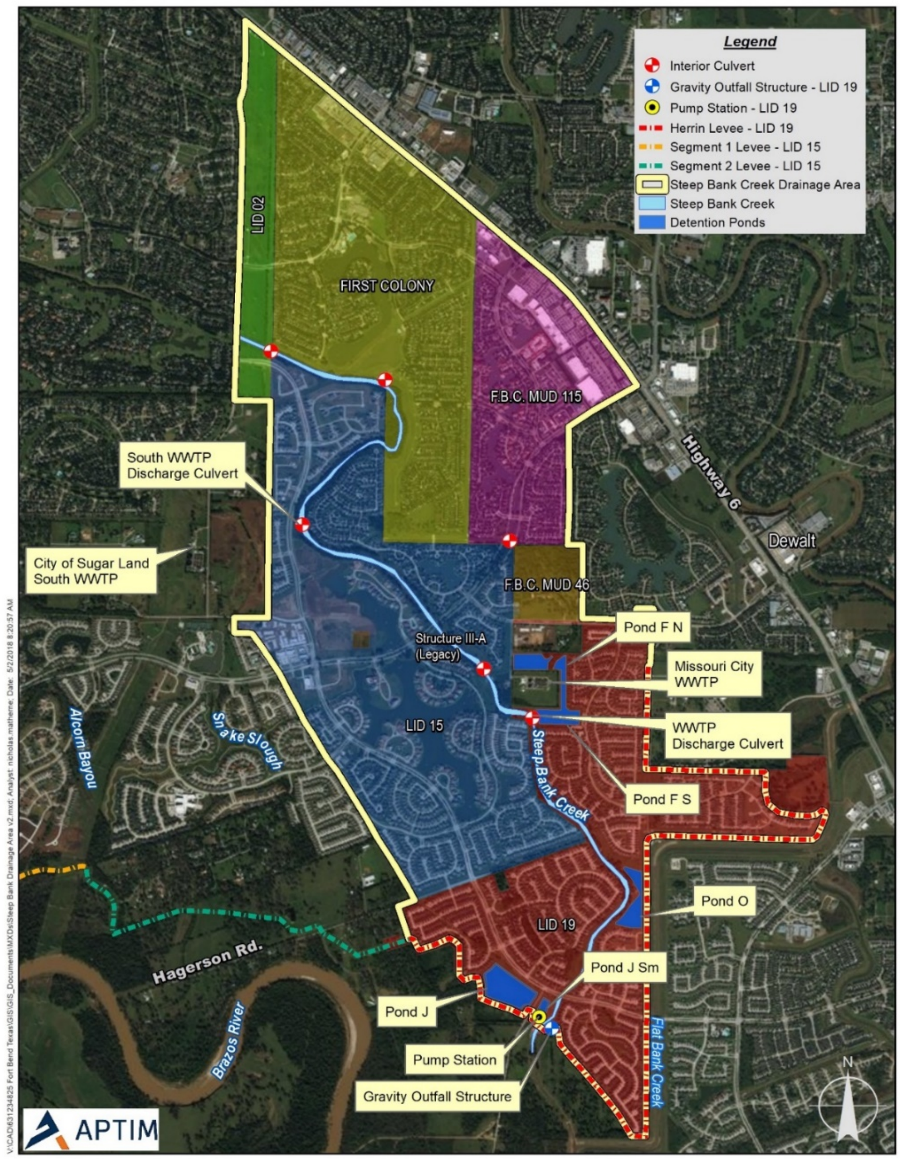

Figure 2.1 LID 19 Drainage Area

This review was divided into three phases: Phase 1 included a System Review; Phase 2 was a review of the Operations and Response during Hurricane Harvey; the current Phase 3 is the summation of Recommendations to address any findings noted in Phase 1. With Phases 1 & 2 complete, this memorandum serves as the final phase and provides the Recommendations to findings as tasked under Phase 3.

3.0 PURPOSE

The intent of these services is to provide LID 19 with recommendations to improve their facilities utilizing the revised ICPR model, with the focus on pumping capabilities. The recommendations to be provided include improvements to the existing pumping location at Steep Bank Creek, as well as accommodating larger coincidental rainstorm events. These include:

- Recommendations for improvements to LID 19's existing pumping system to meet the current criteria (TP40) as identified in APTIM's Phase I Report for coincidental events.
- Recommendations for improvements to LID 19's pumping system to exceed the current 2011 FBCDD criteria for a larger design rainstorm for a coincidental event, such as a 25-year (TP40) rainstorm event.
- Recommendations for improvements to LID 19's pumping system to a design rainstorm for a coincidental event equivalent to a 100-year (TP40) rainstorm event.
- Recommendations for improvements to LID 19's pumping system to a design rainstorm for a coincidental event equivalent to a 100-year storm, utilizing the Preliminary NOAA Atlas 14 Data that is currently being evaluated.

4.0 METHODOLOGY

The methodology for Phase 3 focuses on the ICPR modeling efforts. In Phase 1, we identified several issues with the existing model that would need to be adjusted or revised in order to provide stable results. APTIM utilized an ICPR expert, Singhofen & Associates, who initiated these revisions. Singhofen's memorandum that outlines these revisions is provided in Attachment 1. Figure 4.1, below, represents the revised ICPR model for LID 19's drainage area.

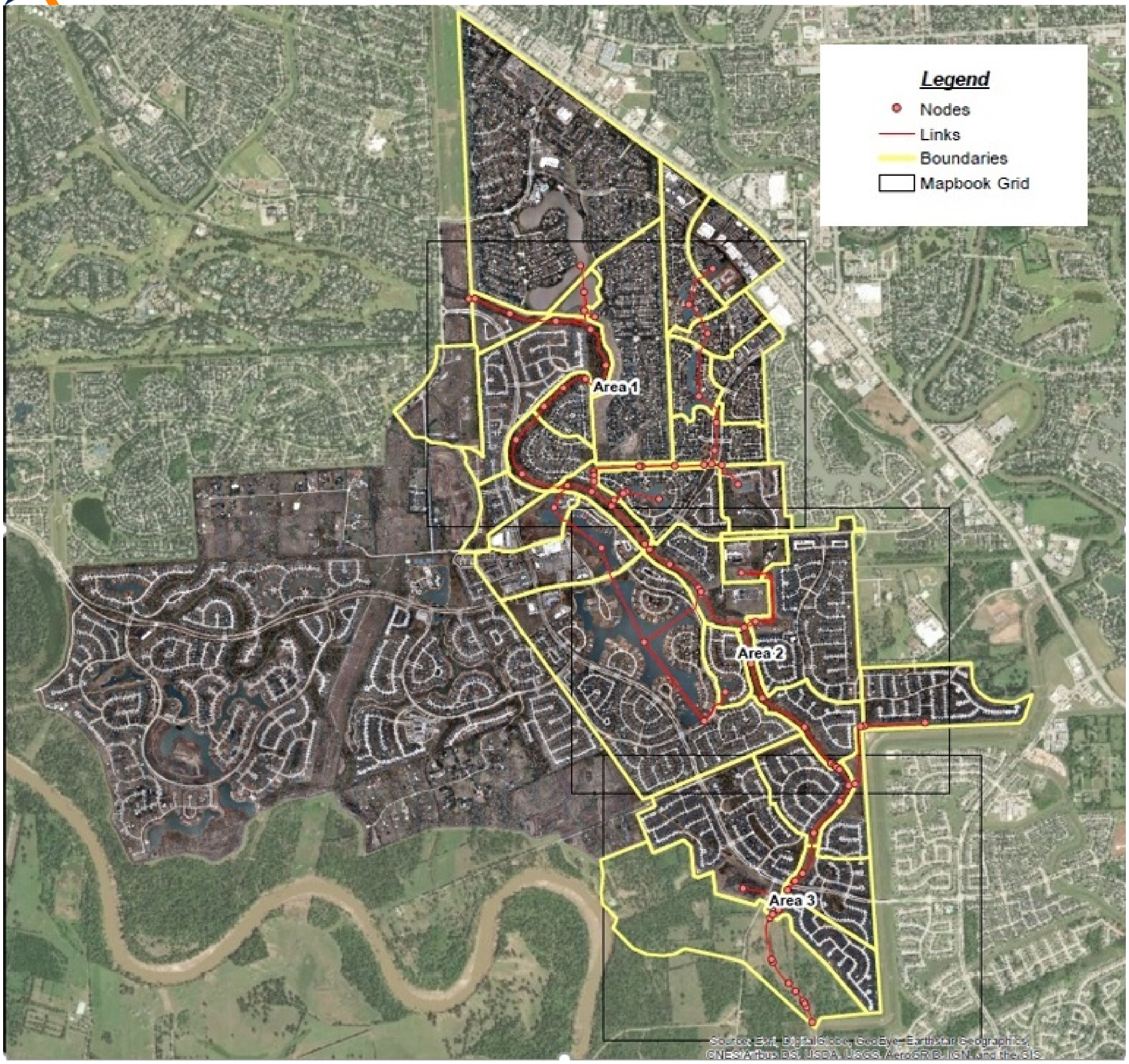

Figure 4.1 ICPR Model LID 19 Drainage Area

To represent the rainfall, hydrographs for each sub-basin were generated using the Technical Paper 40 (TP40) rain data table in the FBCDCM with the existing HEC-HMS model for each rain event to be analyzed. Also, hydrographs were created and included in the analysis for the same events using the new Atlas 14 rain data, which was released by NOAA during this phase and is still being evaluated by FBCDD. As per the FBCDCM, each analysis was run assuming the base conditions were non-wet and all design storage was available. In addition, each design storm was modeled as a single-event and does not include multiple events where cumulative storage affects could occur.

Since these recommendations will focus on coincidental events, a critical element that must be determined to obtain the pump station requirements is the maximum ponding elevation. The maximum ponding elevation is the internal water surface that cannot be exceeded. This water surface must be maintained at, or below, this elevation by either sufficient storage, or pumping. The maximum ponding elevation provided by the design engineer was 61.87 ft. NGVD29, which converts to 60.59 ft. NAVD88. This elevation was recently confirmed by survey to be sufficient. Attachment 2 is the survey report performed for the on-going 2D regional modeling effort. Table 2 in the report includes the surveyed finished-floor elevations of representative homes in the area. These elevations appear to meet the 1 ft. of freeboard as required in the FBCDCM.

In the ICPR, the maximum ponding model this elevation was applied in the southern portion of Steep Bank Creek near the pump station at node SB-02988, shown in Figure 4.2. To determine pumping capacity, multiple iterations of model runs were performed with the outflow being adjusted for each iteration until the maximum ponding elevation could be maintained at the selected node. When achieved, final elevation checks were performed throughout the model to verify that design stage elevations, such as detention lakes, were not exceeded.

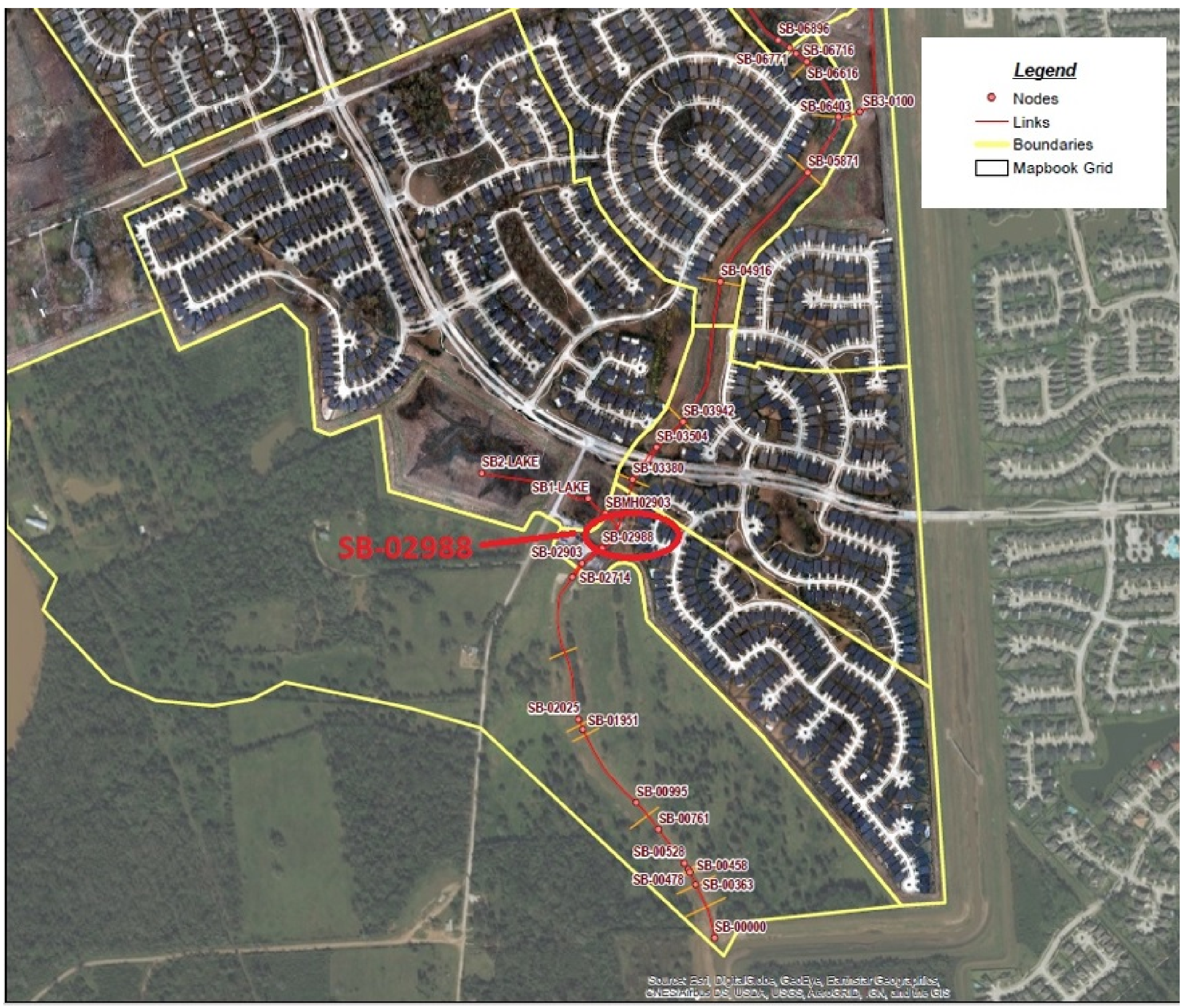

Figure 4.2 Node SB-02988 Location

5.0 STORM EVENT ANALYSIS

5.1 Coincidental 10-year event using TP40 (and Atlas 14)

This analysis has been the priority of this task with the objective to identify the pumping requirements needed to meet the current drainage criteria. For this analysis, a 10-year, 24hr. rain event utilizing TP40 data was used to create hydrographs for each sub-basins in the existing HEC-HMS model. These hydrographs were then imported into the ICPR model that was setup for coincidental events. This configuration assumes the Brazos River elevation at Steep Bank Creek is at 66.0 ft. NGVD29, which prevents gravity drainage; the pumps are set to "ON"; and rainwater can only be removed by pumping.

To determine the pumping capacities needed for the rain event, multiple iterations of model runs were necessary. In this case, we started with the existing pump station capacity of 80,000 Gallons Per Minute (GPM), which was not sufficient for a TP40 10-year rain event. In order to maintain the maximum ponding elevation of 60.59 ft. NAVD88, the GPM was increased. Ultimately, 139,000GPM was necessary to maintain the maximum ponding elevation near Steep Bank Creek pump station. It should be noted that this pump capacity DOES NOT include a spare pump, as required by FBCDCM.

In addition to the TP40 data, we were asked to analyze the preliminary Atlas 14 data. Using the 10-year, 24hr. data from Atlas 14, we created new hydrographs in HEC-HMS for each sub-basin and imported them into the ICPR model configured for coincidental events. In running the iterations, we observed more pumping capacity was required to maintain the maximum ponding elevation at the same ICPR node (SB-02988) near Steep Bank Creek pump station. The pumping requirements for a 10-year, 24hr. rain event using Atlas 14 data was determined to be 157,000GPM. This capacity also DOES NOT include a spare pump.

Understanding that the new Atlas 14 data is still under evaluation with a high probability of being adopted by FBCDD, APTIM provided an advance deliverable in the form of a letter advising of this finding. See Attachment 3.

To achieve the increased pumping needs under the TP40 results, and to provide minimal disruptions to the existing pump station, a new supplemental pump station is being proposed just north of the existing Steep Bank Creek pump station, as shown in Figure 5.1, utilizing the same outfall as the Steep Bank Creek pump station. The conceptual configuration of the supplemental pump station includes three (3) 30,000GPM pumps. The third 30,000GPM pump is required to satisfy the FBCDCM criteria of having a spare of the largest pump in case one is out of service. The result is a total pumping capacity of 140,000GPM + 30,000GPM, or 170,000GPM at Steep Bank Creek. These additions would meet the current criteria utilizing TP40 rain data. Refer to Table 6.1 Summary of Design Storm Analyses and Pumping Capacities.

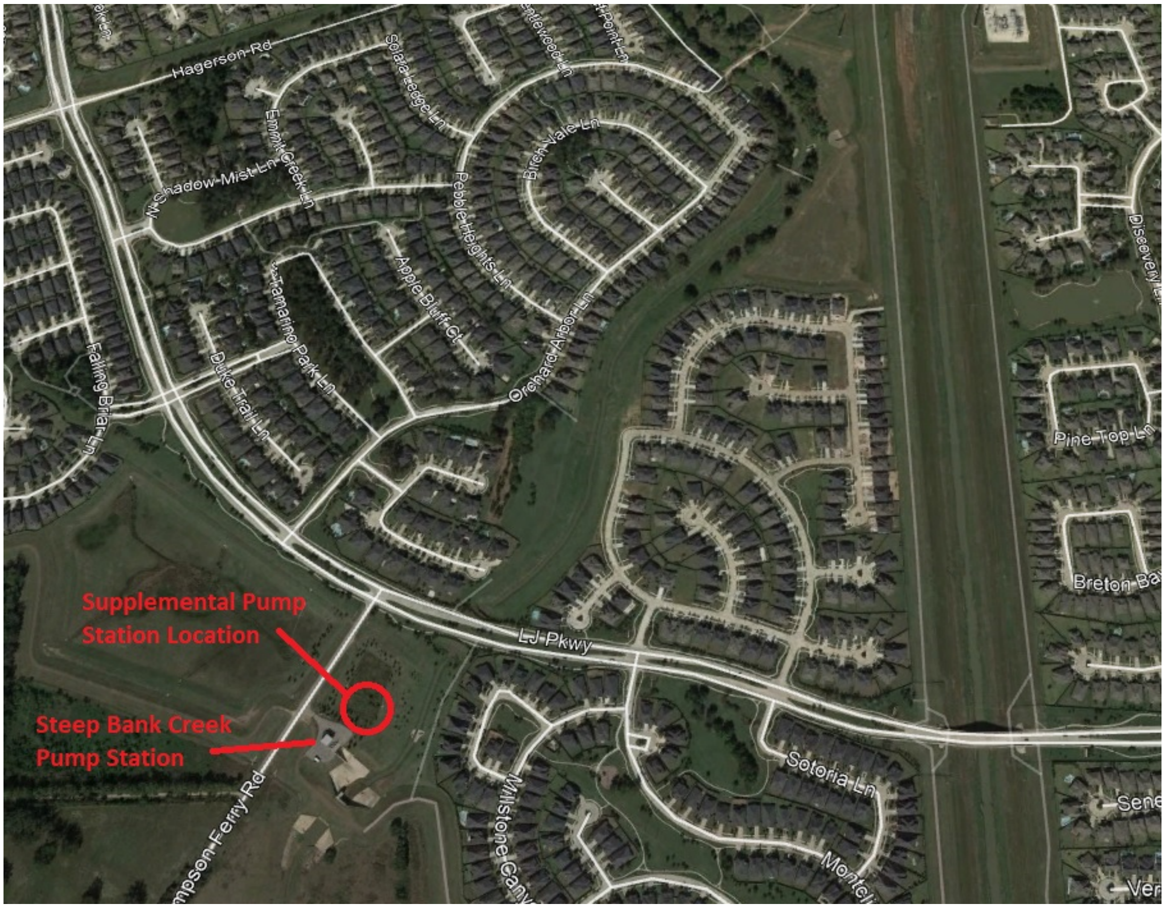

Figure 5.1 Supplemental Pump Station Location

Using the Atlas 14 results, a new supplemental pump station just north of the existing pump station would still be proposed, but with a different configuration. The conceptual pump configuration includes three (3) 40,000GPM pumps. The third 40,000GPM pump is to satisfy the FBCDCM criteria of having a spare of the largest pump. The new pumping capacity would then be 160,000GPM + 40,000GPM for a total of 200,000GPM at Steep Bank Creek. Refer to Table 6.1 Summary of Design Storm Analyses and Pumping Capacities.

When evaluating these concepts to develop a rough order of magnitude (ROM) cost, the supplemental pump stations can be viewed as new pump stations. Pump stations of this complexity, on average, are estimated at $15,000/CFS (Cubic Feet/Second) which converts to $33.42/GPM. Applying this cost to the conceptual supplemental pump stations, for both TP40 and Atlas 14, the estimated ROM costs are as follows:

- TP40 (10-year), Supplemental pump station: 90,000GPM X $33.42/GPM = $3,007,800
- Atlas 14 (10-year), Supplemental pump station: 120,000GPM X $33.42/GPM = $4,010,400

5.2 Coincidental 25-year event using TP40

In addition to the 10-year analysis, APTIM also analyzed the 25-year coincidental event to provide the pumping capacities necessary to exceed the current criteria. To accomplish this, we utilized 25-year hydrographs for each sub-basin generated in HEC-HMS utilizing the TP40 data, and imported them into the revised ICPR model configured for coincidental events. Iterations were run to determine the GPM necessary to maintain the maximum ponding elevation. The pumping capacity necessary was determined to be 257,000GPM. This does not include the spare pump requirement of the current FBCDCM.

To achieve this increased pumping capacity, a concept similar to that suggested for the 10-year event was considered. A new supplemental pump station could be constructed just north of the existing Steep Bank Creek pump station; however, due to its larger capacity, the existing outfall would require further review and a completely new outfall may be needed, independent of the existing culvert, that would still flow into Steep Bank Creek. The conceptual configuration of this supplemental pump station could be comprised of four (4) 60,000GPM pumps. The fourth 60,000GPM pump would be required to satisfy the FBCDCM criteria of providing a spare of the largest pump, in the event one was out of service. This would put the pumping capacity at 260,000GPM + 60,000GPM for a total of 320,000GPM at Steep Bank Creek. Refer to Table 6.1 Summary of Design Storm Analyses and Pumping Capacities.

Again evaluating conceptual costs, the supplemental pump station would be considered a new pump station. Using the previous $33.42/GPM, the ROM cost is:

- TP40 (25-year),Supplemental pump station: 240,000GPM X $33.42/GPM = $8,020,800

5.3 Coincidental 100-year event using TP40

The final analysis using TP40 data was conducted for the 100-year coincidental event to determine the pumping capacity needed to remove all the runoff for a 100-year event if gravity drainage was lost. To accomplish this, APTIM utilized 100-year hydrographs for each sub-basin generated in HEC-HMS utilizing the TP40 data and imported them into the revised ICPR model configured for coincidental events. Multiple iterations of the ICPR model determined the GPM necessary to maintain the maximum ponding elevation. The pumping capacity was determined to be 450,000GPM, not including the spare pump requirement of the current FBCDCM.

To achieve this pumping capacity, an additional pump station could be considered near “Pond “O” Location”, shown below. Because Steep Bank Creek’s slope is relatively flat in this area, and Pond “O” is adjacent to the Flat Bank Creek Diversion Channel, both locations can utilize this reach of Steep Bank Creek to effectively maintain the maximum ponding elevation. It should be noted that additional investigation will be required for the Flat Bank Creek Diversion Channel, to better understand the effects of adding additional flow to the channel, ensuring there aren't any impacts upstream of the Pond "O" location's outfall.

Figure 5.3 Pond "O" Location

This concept builds on the TP40 10-year supplemental pump station of three (3) 30,000GPM pumps. If this pump station expansion is built, then Pond "O" pump station could include six (6) 60,000GPM pumps, with the sixth pump as the required spare. Unfortunately having two separate stations requires a spare at each station, adding to the construction cost. However, this increased cost is relatively low compared to refurbishing the entire Steep Bank Creek pump station, as well as keeping the Steep Bank Creek pump station operational during construction.

With this configuration, Steep Bank Creek will have a total pumping capacity of 140,000GPM + 30,000GPM = 170,000GPM. The Pond "O" pump station total pumping capacity was determined to be 300,000GPM + 60,000GPM = 360,000GPM. This would still require a smaller 10,000GPM pump to be included as a stripper pump to achieve the GPM required. The capacity of both pump stations would total 540,000GPM. Refer to Table 6.1 Summary of Design Storm Analyses and Pumping Capacities.

When evaluating costs for this concept we used the TP40 10-year cost of:

- TP40 (10-year), Supplemental Pumps Station 90,000GPM X $33.42/GMP = $3,007,800

The Pond “O” pump station construction would be all new construction for the entire 370,000GPM. Using the same cost per GPM rate of $33.42 results in:

- TP40 (100-year), Pond “O” pump station 370,000GPM X $33.42/GPM = $12,365,400

The total ROM cost to achieve 100-year pumping capacity is $15,373,200.

5.4 Coincidental 100-year event using Atlas 14

As a final run, APTIM analyzed the 100-year Atlas 14 coincidental event, which is expected to generate 16.5” of rain in 24hrs. To accomplish this, we generated new hydrographs for each sub-basin in HEC-HMS utilizing the Atlas 14 data, and imported them into the revised ICPR model configured for coincidental events. The pumping capacity required to maintain the desired maximum ponding elevation was determined to be 690,000GPM, which does not include the spare requirements of the current FBCDCM. Because several detention lakes exceeded their designed water surface elevations and the ICPR model is not able to accurately account for water that exceeded these boundaries, we are not making a recommendation for this scenario. This analysis would be better performed under the 2D modeling currently underway. We will defer any recommendations with regards to this event until that model is complete.

6.0 RECOMMENDATIONS

With the understanding that FBCDD is currently evaluating the new Atlas 14 data and there is uncertainty on when it may be adopted, we recommend proceeding with implementing the supplemental pump station just north of the current Steep Bank Creek pump station. This station should be designed to meet current criteria of a 10-year rain event utilizing TP40 data, as discussed above. It is also our recommendation to incorporate into the design, an upgrade option in anticipation of the acceptance of the Atlas 14 data or changes to the FBCDCM. See the summary of results and recommendation below.

Table 6.1 Summary of Design Storm Analyses and Pumping Capacities

Coincidental Scenario	ICPR Model Requirements	Current Pumping Capacity	Supplemental Pumping Needs (includes spare pump)	New Total Pumping Capacity (includes spare pump)
10-Year TP40*	139,000	80,000 GPM	90,000 GPM (3 x 30,000 GPM pumps)	170,000 GPM
10- Year Atlas 14	157,000	80,000 GPM	120,000 GPM (4 x 30,000 GPM pumps)	200,000 GPM
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100-Year Atlas 14	690,000**	80,000 GPM	TBD with 2-D Regional Model	

* Recommended pumping improvements
** Does not account for all inflows

By analyzing the multiple rain events that exceeded current criteria, we were able to demonstrate the magnitude of GPM increases. Conceptually, Pond “O” pump station can supplement the Steep Bank Creek pump station to exceed FBCDCM criteria; however, more analysis on the effects of the outfall on the Flat Bank Diversion Channel will have to be performed. And finally, it is recommended that the 2D model analysis of the watershed region be completed to better understand all the inflow contributors to Steep Bank Creek.

ATTACHMENT 1
SINGHOFEN'S REPORT

Singhofen & Associates, Inc.
stormwater management and civil engineering

MEMO

To: Hilary J. Thibodeaux, PE
From: Mark X. Troilo, PE, CFM
Date: January 31, 2019
Re: The Grove at Riverstone Drainage Model Revisions and Modification -- APTIM

INTRODUCTION
Singhofen & Associates, Inc. (SAI) was contracted by APTIM to provide engineering consulting services for work related to the revision and update of an ICPRv3 stormwater model that was developed for a private development called The Grove at Riverstone (The Grove). The development is located within a levee control district (LCD) in Fort Bend County, Texas. Updates included revisions to select model parameters based on recommended defaults for the ICPR program. SAI also updated the existing model to include daily flows from two wastewater treatment plants (WWTP) resulting in small revisions to starting water elevations within the LCD. Finally, SAI made modifications to an existing pump station at the direction of staff at APTIM.

This technical memorandum provides a summary of the various revisions and updates that were made to the model.

Task 1: Model Revisions and Updates

1.1 **Data collection and field reconnaissance:** SAI was provided data including terrain data (DEM), aerial mapping and construction plans. These data were used to update the model. Staff from APTIM conducted field reconnaissance and provided photographs and additional information for this effort. Model revisions included changes to pipe geometry data, channel geometry and cross section data and culvert loss coefficients as explained below.

1.2 **Pipe Geometry data:** SAI reviewed and compared pipe geometry data in the original model to plans, DEM, aerial maps and photography. This included reviews of pipe size, length and invert elevations. **Appendix A** includes a comparison of the original model data and final, updated model data. Two significant changes were made including conversion of culvert **Link: 1UP-2N** to a control structure (i.e., an ICPR drop structure link) and elimination of culvert Link: SB83-Out. The latter was included in the original modeling but plans and a field visit by APTIM staff confirmed this culvert was never constructed. As a result, the culvert was set to No Flow in ICPR, effectively removing it from flood routing calculations. **This resulted in increased lake stages upstream and reduced stages in the receiving canal.**

1.3 **Entrance Loss Coefficients:** A significant number of entrance loss coefficients in the original model were not consistent with the assigned FHWA inlet edge descriptions and end treatments observed in the field or from aerial photography. SAI updated the pipe entrance loss coefficients using field photographs provided by APTIM, aerial mapping and plans. **Figure 1.1** shows an example of one such location. Culvert **Link: 8-9-LEVE**

includes mitered end sections. The original model had an entrance loss coefficient value set to 0.5, however, a value of 0.7 is more appropriate. as well as which is used for updating entrance loss. In addition, it was noted that most channels in the original model included a value of 0.5 for the entrance loss coefficient. Channels do not normally include entrance losses, unless an unusual situation is encountered which warrants additional hydraulic losses (e.g., aerial crossings, etc.). SAI reviewed all channels and reset entrance loss coefficients to 0.0. A total of 24 pipes and 38 channel entrance loss coefficients were revised with changes ranging from +/- 0.1 to 0.5. **Impacts from these changes will vary from location to location, depending upon the specific change.**

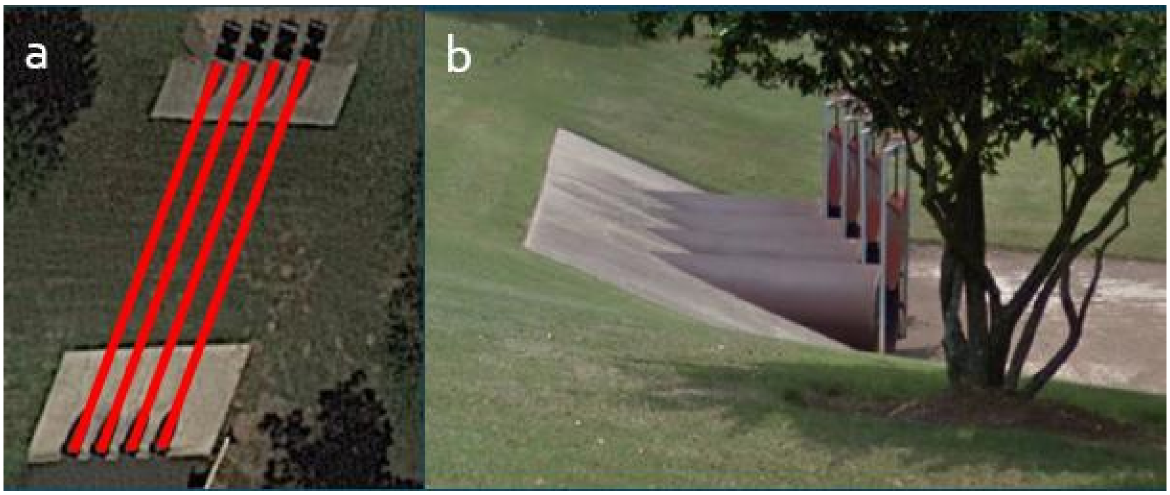

Figure 1.1 Aerial (a) and Field Photographs (b) at Culvert Link: 8-9-LEVE

1.4 **Exit Loss Coefficients:** The original model included exit loss values between 1.0 and 1.5 at most pipes, regardless of the type of condition at the pipe discharge point or the accompanying velocity change. Most channel exit loss coefficients were set to zero in the original model. SAI updated the pipe exit loss coefficients to be representative of the energy lost from velocity decreases in the pipe's receiving water. A total of 27 pipe and 2 channel exit loss coefficients were changed with changes ranging from +/- 0.1 to 1.05. **These changes will tend to reduce overall hydraulic losses at the affected locations.**

1.5 **Channel Cross Section:** The actual locations of channel cross sections were not available. To confirm the accuracy of the data, SAI drew channel centerlines and storage exclusion polygons using the channel link locations, provided by APTIM, the DEM and aerial mapping. Note that in locations with two unique cross sections specified at the same node location, the more appropriate cross section of the two was determined based on the DEM and plans. Using the resulting data, SAI noted some conflicts between the channel storage exclusion area and adjacent basin boundaries.

Figure 1.2 shows a conflict between a ditch cross section and an adjacent basin boundary and storage area. Storage in the basin adjacent to channel was originally approximated with the channel cross section and included in SB3-0100; This approach inaccurately included the pond area as effective flow area through its inclusion in the section in addition to "double counting" the storage. This problem was addressed by revising the channel cross section . **Figure 1.3** compares the original cross section and updated cross section for this location.

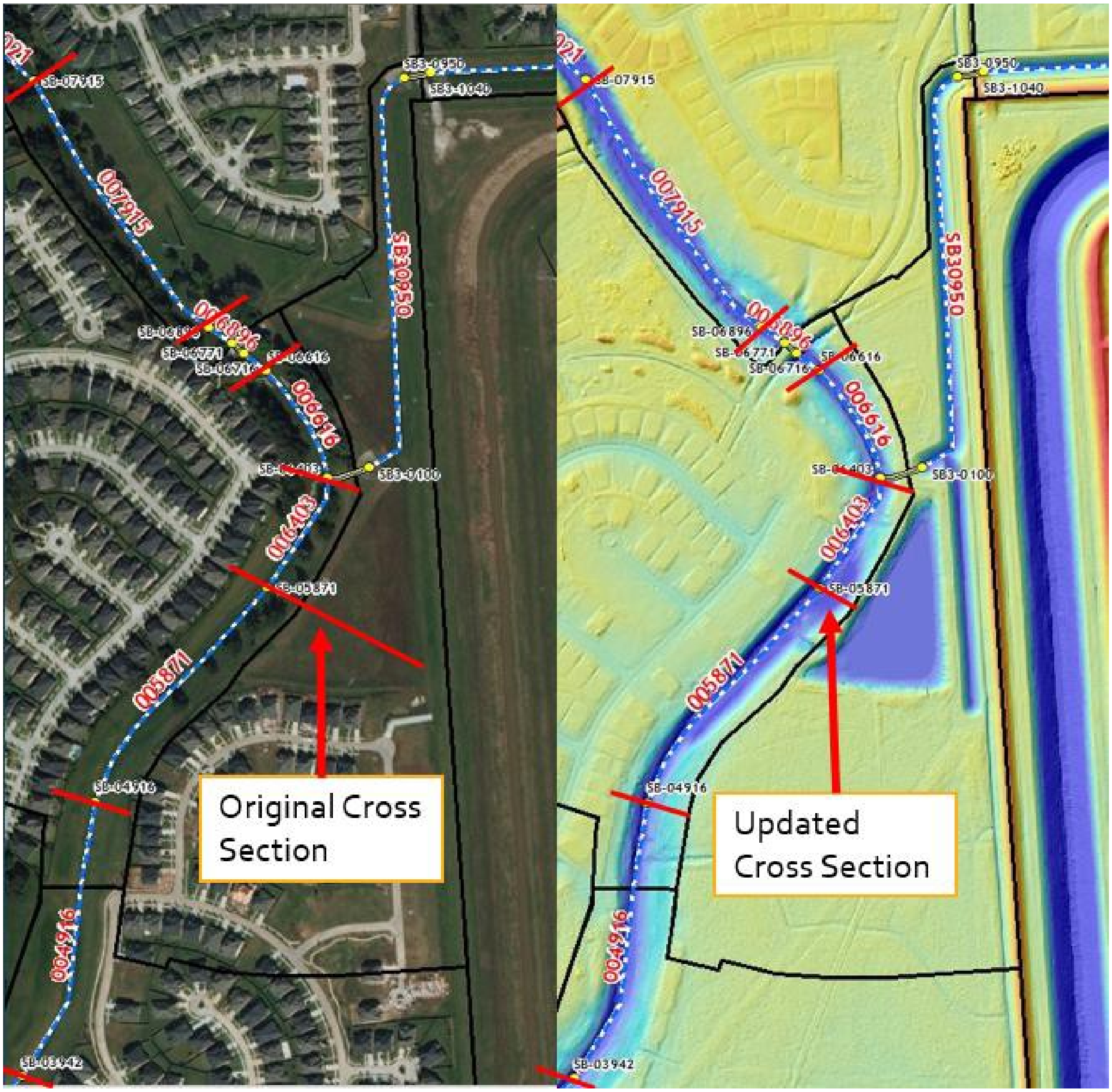

Figure 1.2: Correction to Offline Storage / Channel Cross Section

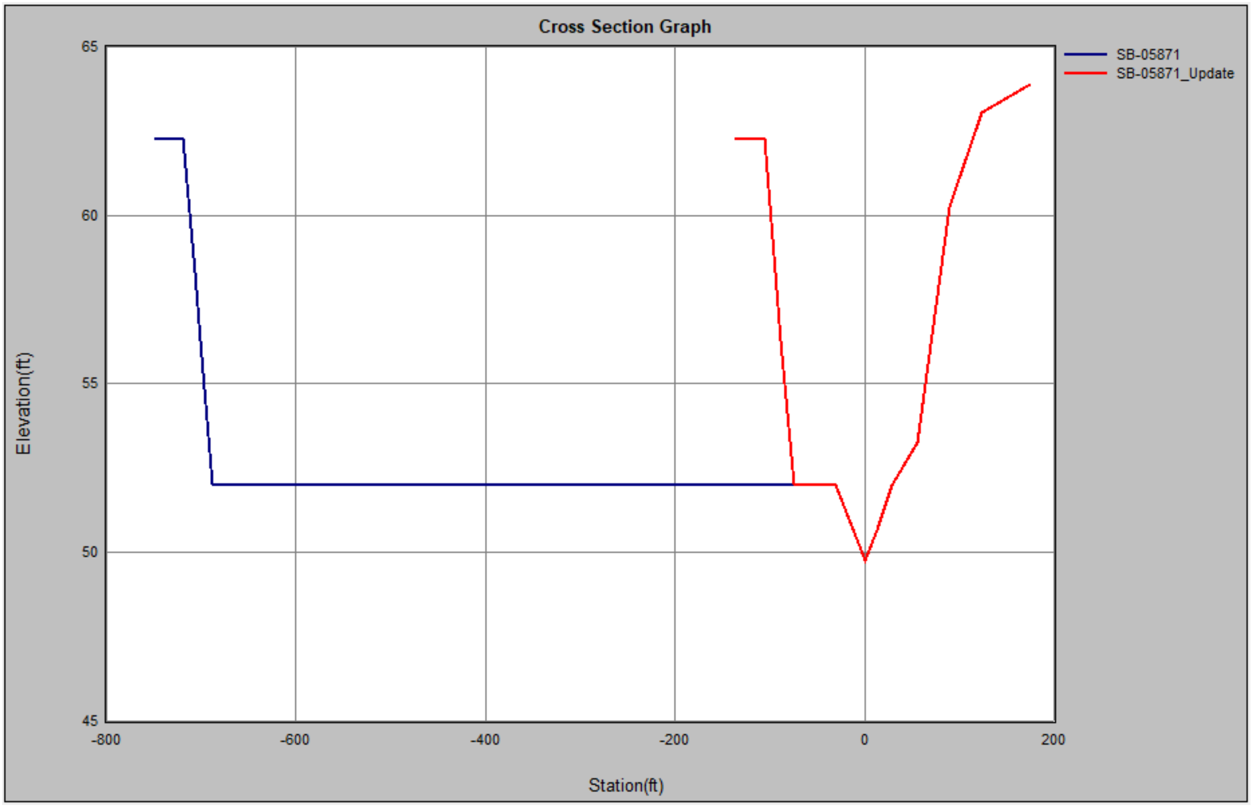

Figure 1.3: Original and Updated Cross Section

1.6 **Pond and stage area data:** SAI drew polygons around all stormwater ponds. Using these polygons and the DEM, SAI generated stage-area data for each pond node, omitting flood storage that may be available within the contributing basin (e.g., road flooding, etc.). APTIM provided areas as well as elevations at the pond top of bank and bottom from plans. Comparison of that information to the DEM showed there are significant discrepancies between the updated stage-area data and the original model data. Since the plans appear to agree with the DEM, all stage-area for the ponds were replaced with new information.

1.7 **Subbasin Boundary Adjustment:** Some subbasin boundaries associated with the original model were noted to include areas within the ditch system (i.e., on the ditch sidebank). These were revised to follow the DEM and avoid overlaps between the pond and channel control volumes. **Figure 1.4** shows an example one revision. It should be mentioned that the subbasin boundary changes were minor and did not have significant effects on basin area and runoff calculations.

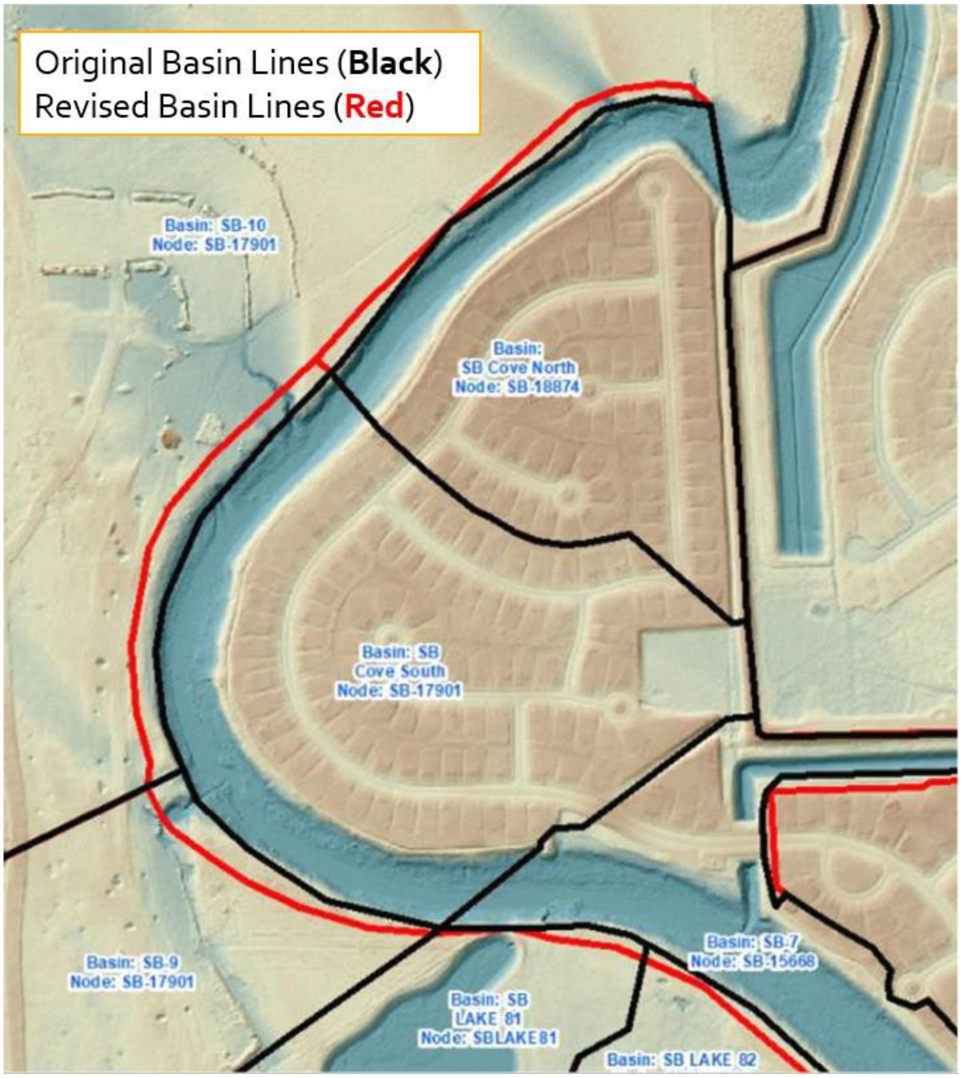

Figure 1.4 Subbasin Boundary Adjustments

1.8 Boundary flow data: For this study, HEC-HMS was used to perform runoff calculations and generate hydrographs. Those data were imported into ICPR which was then used for hydraulic routing. The runoff hydrographs from HEC-HMS model were assigned to nodes within each subbasin. SAI worked with staff at APTIM ensure the boundary flow data were correct. The HEC-HMS model results were provided in Excel file format. SAI imported the data into ICPRv4 using CSV import files. **Table 1.1** shows the name of the boundary flow data in ICPRv4 and the related HEC-HMS subbasin name.

Table 1.1 Boundary Runoff Hydrograph / Node Assignments

HEC-HMS Basin Name	Related Hydrograph in ICPR
D/S LA	7-UNIVDN#1
DVLPT	4-LOLAKE#1
H-1	SB-17901#1
HE-OUT	SB-01951#1
LID2	LID2#1
MTHWS	MTHW-Pond#1
N_LAKE	2-NLAKE#1
PONDA1	PONDA1#1
PONDA3	PONDA3#1
SB COVE NORTH	SB-20002#1
SB COVE SOUTH	SB-17901#3
SB LAKE 81	SBLAKE81#1
SB LAKE 82	SBLAKE82#1
SB LAKE 83	SBLAKE83#1
SB LAKE 84	SBLAKE84#1
SB POND FL	SBPONDFL#1
SB POND FU	SBPONDFU#1
SB-1	SB-02988#1
SB-10	SB-17901#2
SB-11	SB-23970#1
SB-3	SB3-2370#1
SB-4E	SB-03942#1
SB-4W	SB-06896#1
SB-5	SB-07915#1
SB-6	SB-12029#1
SB-7	SB-15668#1
SB-9	SBLAKE81#2
SB2 LAKE	SB2-LAKE#1
SB3 LAKE	SB3-0100#1
SB_CROSSING	DETSEC2#1
S_LAKE	3-SLAKE#1
TERRAC	5-TRLAKE#1
UPPER	1-UPLAKE#1

1.9 Rating curves and Operating Tables: SAI reviewed rating curves and operating tables for the pump station to verify they represent reasonable flow characteristics for the station. Minor modifications were made in preparation for later analyses of various pump station configurations.

1.10 Channel Extrapolation: SAI reviewed the irregular channels at locations where irregular section data were extrapolated or locations where significant vertical translation of cross sections occurs. SAI did not identify significant issues during this review.

1.11 **The original and updated model results discrepancies:** SAI finalized the ICPR model and executed model simulations of several storms. **Table 1.2** (located at the end of this memorandum) shows differences between the original model and the updated model.

Task 2: Incorporating Waste Water in Model

SAI incorporated WWTP discharges into the model. The WWTP discharge was assigned to two different locations: SB-17020 (4.48 MGD) and SB-10883 (1.9 MGD) as a base flow. The resulting model was then used to simulate the baseflow condition that results from the WWTP flows. **Table 2.1** (located at the end of this memorandum) shows the resulting peak stages in the ditch system with and without the WWTP flows. Note that model simulations for pump station evaluations discussed below were performed using the initial conditions resulting from the WWTP discharge rates.

Task 3: Modification and Evaluation of Steep Bank Pump Station

Model runs for this project considered two general conditions: a gravity outfall to the Brazos River without pump station operation and a pumped condition required when the Brazos River is at flood stage (66 FT-NGVD29) coincident with a storm within the LCD. Under the latter "coincident" scenario, the high tailwater in the river prevents function of the gravity outfall, requiring the pump station for discharge capacity. SAI evaluated the existing pump station under "coincident" scenarios using rainfall for an 8.2-year and 10-year 24-hour storm. Results are shown in **Table 3.1** (located at the end of this memorandum).

SAI also evaluated several pumping options under the "coincident" scenarios. This effort included rainfall volumes based on TP40 as well as NOAA's ATLAS 14. The goal of this evaluation was to determine pump rates required to maintain peak flood stage at 61.87 FT-NGVD29 at two different locations in the LCD: Node: SB-12029 and Node: SB-02988. **Table 3.2** shows the pump sizes required to meet this condition.

Table 3.2 Pump Station Rate Requirements

Rainfall based on TP 40			
SB-02988		SB-12029	
8.2 Yr	10 Yr	8.2 Yr	10 Yr
111 K GPM	139 K GPM	123 K GPM	154 K GPM

Rainfall based on NOAA Atlas 14			
SB-02988		SB-12029	
8.2 Yr	10 Yr	8.2 Yr	10 Yr
121 K GPM	157 K GPM	136 K GPM	177 K GPM

ATTACHMENT 2
2019 FORT BEND TOPOGRAPHIC SURVEY REPORT

2019
Topographic Survey Report for the Steep Bank Creek Regional Watershed Hydrologic and Hydraulic Modeling Study

Prepared for:

Fort Bend County LID No. 19, LID No. 15,
First Colony LID, MUD 46, MUD 115

Prepared by:

APTIM Environmental & Infrastructure, LLC

April 2019

1.0 Executive Summary

Aptim Environmental and Infrastructure Inc. (APTIM) was contracted by the Fort Bend Levee Improvement District to provide topographic survey services to supplement and ground truth the Light Detection and Ranging (LiDAR) elevation data collected by Fugro, Inc (Fugro).

APTIM independently verified survey control used by Fugro for the collection of the LiDAR data and found the control was stable and harmonized with horizontal and vertical values measured by APTIM. Further, it was found that the topographic data collected by APTIM validated the LiDAR Digital Elevation Model (DEM) provided by Fugro within the tolerances described within the Fugro LiDAR QA/QC Report dated January 17, 2019. Nine RTK GPS measurements collected on open roadways were used to validate the LiDAR DEM datum resulting in an average difference of -0.07' between the RTK GNSS and the LiDAR DEM. Further, a total of 296 RTK GNSS measurements were used to validate the LiDAR DEM relative accuracy on natural ground resulting in an average difference of -0.36' between the RTK GNSS and the LiDAR DEM.

In addition, APTIM collected fifteen finished floor elevations and data to document eight high water marks resulting from the Hurricane Harvey flood event. APTIM also collected 80 cross-sections of creeks and sloughs to supplement LiDAR data in submerged areas.

Data were collected on March 11, 2019 through March 15, 2019. All coordinates presented in this report are U.S. Survey Feet, relative to the North American Datum of 1983 (2011) (NAD83/2011), Texas State Plane Coordinate System, South Central Zone. Elevations are presented in U.S. Survey Feet, relative to the North American Vertical Datum of 1988 (NAVD88), using Geoid Model 12B.

2.0 Table of Contents

3.0 List of Figures

4.0 List of Tables

5.0 List of Appendices

6.0 Survey Methods and Results

Phase One: Control Reconnaissance/Establishment/Verification

Prior to the start of the survey, control monuments from the Nation Geodetic Survey were verified using Static Global Navigation Satellite Systems (GNSS) and Real Time Kinematic (RTK) GNSS measurement methods. Temporary benchmarks (TBMs) were also established using a combination of Static survey and RTK GNSS methods to augment the published control and provide bench marks for leveling loops. In addition, temporary benchmarks established by Fugro were located to confirm control stability and datum to ensure a valid comparison between the topographic data collection by APTIM and the DEM produced from Furgo LiDAR.

All control used for this project is presented below in **Table 1.** Recovered monuments and TBM solutions were processed and adjusted using the National Geodetic Survey (NGS) Online User Positioning Service (OPUS). The program utilizes statistical routines that employ satellite data that is continually logged at numerous GPS stations referred to as Continually Operating Reference Stations (CORS). The vertical and horizontal movement of the stations is well understood providing that the station has been operating for several years. It should be noted that TBMs are intended for short term project use and future use is contingent on reoccupation to verify stability of the TBMs.

NGS data sheets for the monuments used in this survey are presented in **Appendix A**. OPUS Solutions are presented in **Appendix B.** Static solutions were used for comparison purposes only and published monument values were held. Horizontal and vertical differences between published locations, static solutions, and RTK measurements averaged less than 0.2’ and are within the tolerances for GNSS methods and geophysical survey.

Table 1 – Published Control Monument Information

Control Monument Information Texas State Plane South Central NAD 83/2011 NAVD 88 US Survey Feet				
Monument Name	*Northing*	*Easting*	*Elevation*	*Source*
HGCSD 72	13748862.76	3078378.98	61.45	NGS
FUGRO 8805	13761774.20	3061001.36	63.53	Previous Survey
FUGRO 1105	13771175.72	3057343.32	65.13	Previous Survey
APTIM TBM 1	13757887.26	3061249.35	70.52	Established
APTIM TBM 2	13755955.87	3058992.43	71.223	Established

All vertical data were collected in the North American Vertical Datum of 1988 (NAVD88) relative to geoid model 12B. All horizontal data were collected in the Texas State Plane Coordinate System, South Central Zone, North American Datum of 1983(2011) (NAD83/2011). All horizontal and vertical data were collected in U.S. survey feet.

Phase Two: Finished Floor Elevations and High Water Marks
APTIM collected finished floor elevations (FFE) and approximate high-water marks using a combination of RTK GNSS and levelling techniques. Fifteen homes were included in the survey. Elevations are for information only and should not be implied to represent a flood certificate. Elevations for the each home surveyed is provided in **Table 2.**

Table 2 – Finished Floor Elevations

Address	Elevation (NAVD/FEET)
6619 Tara Creek Court ,Missouri City, TX 77459	62.06
4630 Millstone Canyon Lane, Sugar Land,TX 77479	62.65
4618 Millstone Canyon Lane, Sugar Land, TX 77479	62.17
4527 Marilee Christ Court, Sugar Land, TX 77479	62.95
4522 Millstone Canyon Lane, Sugar Land, TX 77479	62.26
4414 Piper Pass Lane, Sugar Land, TX 77479	63.40
4406 Piper Pass Lane, Sugar Land, TX 77479	63.73
6706 Fairwood Creek Lane, Sugar Land, TX 77479	63.27
6702 Marbrook Saddle Court, Sugar Land, TX 77479	63.52
4123 Abigail Way, Sugar Land, TX 77479	63.90
3939 Orchard Arbor Lane, Sugar Land, TX 77479	63.94
4031 Orchard Arbor Lane, Sugar Land, TX 77479	63.23
6122 Apple Bluff Court, Sugar Land, TX 77479	63.53
6123 Falling Briar Lane, Sugar Land, TX 77479	63.67
6114 Bristol Path Lane, Sugar Land, TX 77479	63.37

APTIM collected elevations of high water (HW) mark from the Hurricane Harvey Flood events where evidence was still available. Elevations data collected is provided in Table 2. An example of high water evidence is shown below in Figure 1.

Table 3 – High Water Mark Elevations

Address	Elevation (NAVD/FEET)
HW Mark near 4406 Piper Pass Lane	63.73
HW Mark near 4422 Piper Pass Lane	64.00
HW Mark near 4618 Millstone Canyon Lane	64.10
HW Mark near 4610 Millstone Canyon Lane	64.05
HW Mark near 4523 Marilee Christ Court	64.18
HW Mark near 4518 Montcliff Bend Lane	64.14
HW Mark near 6127 Falling Briar Lane	64.00
HW Mark near 6114 Bristol Path Lane	64.01
HW Mark near 4031 Orchard Arbor Lane	64.08

Figure 1 – High Water Mark Photograph

Phase Three: General Topographic Data Collection

APTIM collected approximately 1500 topographic data points to supplement the LiDAR data set collected by Fugro as well as ground truth the LiDAR Digital Elevation model. Data were collected at specific points as requested by the Client and APTIM's Project Engineer. An overview of the data collection location areas are shown in Figure 2.

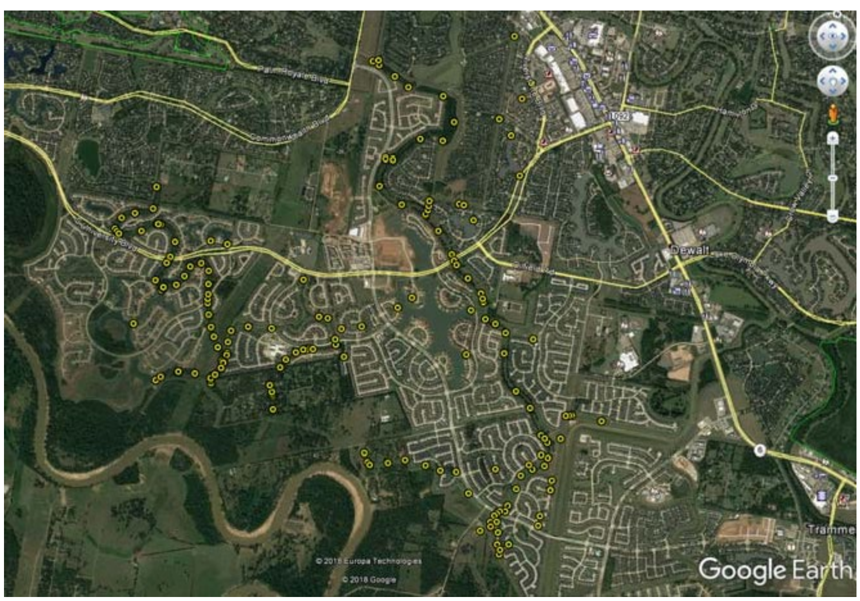

Figure 2 – Topographic Data Collection Overview

Multiple measurements and/or cross-sections were collection at each location show in Figure 2. Additional data were also collected to better define the creek embankments where needed. A total of 80 cross-sections were taken within the creeks to supplement the DEM in submerged areas where LiDAR cannot penetrate. The cross-section data is provided as an ASCII XYZ file and final data presented in plotted cross-section format in appendix C.

In additional to the cross-section data, RTK GNSS measurements were taken on hard structures, drainage structures, roads, and bridges to supplement the LiDAR DEM. Of these measurements, nine measurements taken on open roadways were used to validate the LiDAR datum.

296 RTK GNSS measurements taken on natural ground, including in vegetated and low vegetation areas were used for comparison to the LiDAR DEM resulting in an average difference of -0.36' between the LiDAR DEM and the RTK GNSS measurement. Overall, the LiDAR DEM values comport with measured GNSS values in the comparison areas and the LiDAR data are within the tolerances stated by Fugro, as well as general industry standards for these applications.

Survey maps showing locations of cross-sections collected to supplement LiDAR data in area of inundation and drainage structure inverts are presented in Appendix C. All raw RTK GNSS data and compiled levelling data are presented as digital ASCII files in Appendix D (digital files only).

7.0 Survey Certification

The content contained within represents an actual on the ground survey performed by me or under my supervision. The survey and associated repo1is are true and accurate to the best of my knowledge and belief.

Any revision made to this document or associated rep01is without the written consent of the undersigned will void the seal which has been placed hereon. Revisions shown hereon do not represent a "Field Survey Update" unless otherwise noted.

P. <2,h'ad Maxwell, RPLS
LS#6547

(IJ?oYJ(
\ Signature Date

ATTACHMENT 3
APTIM'S Preliminary Atlas 14 Findings

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February 4, 2019

Fort Bend County Levee Improvement District No. 19
Nancy Carter
Muller Law Group, PLLC
202 Century Square Blvd.
Sugar Land, Texas77478

RE: Phase III Recommendations - Coincidental Pumping Capacities

Dear Board of Directors:

APTIM is providing this letter, as per discussions during the LID 19 Joint Meeting on January 3, 2019, to provide the ICPR model coincidental results of the LID 19 drainage area to support the Steep Bank Creek Pump Station Expansion. These results are part of the Phase III review currently being performed by APTIM and will focus only on Fort Bend County Drainage District (FBCDD) Criteria for Coincidental Events. Results for larger events that exceed FBCDD Criteria will be presented in a supplemental memorandum.

The results are based on the original Costello provided ICPR model for this drainage area that has been revised per the findings from APTIM's ICPR model expert during the Phase I review. Also, the Costello provided HEC-HMS model was utilized to develop hydrographs for each sub-basin within the LID 19 drainage area. In APTIM's Phase I review it was determined by utilizing FBCDD Criteria that an 8.2-year rain event, or even as high as a 10-year rain event, should be considered for the coincidental design storm for this drainage area. After further review, APTIM is recommending a 10-year event to be used for sizing the pump station expansion based on the Brazos River's influence near Snake Slough area.

An important element to note in this analysis is the Maximum Ponding Level. The Maximum Ponding Level is the internal water level that must be maintained by the pump station to avoid structural flooding. The Maximum Ponding Level provided by Costello was 61.87 ft. NGVD 29, which converts to 60.59 ft. NAVD 88. After reviewing the recently collected 2018 LiDAR data, the Maximum Ponding Level of 60.59 ft. NAVD 88 appears to be appropriate. Utilizing this Maximum Ponding Level with the revised ICPR model, the pumping requirement for Steep Bank Creek Pump Station was determined to be **157,000 GPM for a 10-year Rain Event** utilizing the recently released **Atlas 14 rain data.**

Please note that the pumping capacity stated above is the total amount required, and is not in addition to the existing pumping capacities at Steep Bank Creek Pump Station. It should also be noted that this capacity does not account for the spare pump required by FBCDD Criteria.

If you have any questions regarding this information, please contact me or Hilary Thibodeaux at **985-868-3434.**

Sincerely,

Aptim Environmental & Infrastructure, LLC

Laura L. Barnes, P.E.
Operations Manager

HJT:llb