JAY MARTIN
CHAIRMAN

DANNY EARP
VICE-CHAIRMAN

CHUCK ENGELKEN
MEMBER

VACANT
ALTERNATE

CITY OF LA PORTE DRAINAGE AND FLOODING COMMITTEE MEETING AGENDA

Notice is hereby given of a meeting of the Drainage and Flooding Committee of the City Council of the City of La Porte, to be held July 9, 2018, in the City Hall Council Chambers, 604 W. Fairmont Parkway, La Porte, Texas, beginning at 5:00 PM to consider the following items of business.

1. Call to order

2. Authorizations

(a) Consider approval or other action regarding minutes of the Drainage and Flooding Committee Meeting held on June 11, 2018 - P. Fogarty

3. Staff Reports

(a) Receive report from Harris County Flood Control District and Harris County Precinct 2 regarding current and future plans relating to flooding in the City of La Porte - D. Pennell

(b) Brookglen Subdivision - L. Wingate/ D. Pennell

(c) Bob's Gully - L. Wingate/ D. Pennell

(d) Lomax F101-03-00 Drainage Channel Feasibility Study - L. Wingate/ D. Pennell

(e) Little Cedar Bayou F216 Phase III Improvements - L. Wingate/ D. Pennell

(f) Battleground Estates Improvements - L. Wingate/ D. Pennell

(g) North P Street Culvert Improvements - L. Wingate/D. Pennell

(h) Re-grading/De-silting H., L., P. Streets - L. Wingate/D. Pennell

(i) Bayside Terrace - L. Wingate/ D. Pennell

(j) Southside Neighborhood Storm Water Conveyance Improvements - L. Wingate/ D. Pennell

(k) 6th Street from Madison to W. Main Drainage Analysis - L. Wingate/ D. Pennell

(l) Grant Funding Options - L. Wingate/ D. Pennell

(m) Completed Drainage Reports - L. Wingate/ D. Pennell

4. Set date for next Drainage and Flooding Committee Meeting - L. Wingate/D. Pennell

5. Committee Member Comments - Matters appearing on agenda; Recognition of community members, city employees, and upcoming events; Inquiry of staff regarding specific factual information or existing policy

6. Adjourn

In compliance with the Americans with Disabilities Act, the City of La Porte will provide for reasonable accommodations for persons attending public meetings. To better serve attendees, requests should be received 24 hours prior to the meetings. Please contact Patrice Fogarty, City Secretary, at 281.470.5019.

Councilmembers may attend in numbers constituting a quorum. This is a Drainage and Flooding Committee Meeting at which there will be no deliberation or formal action taken by City Council as a governmental body.

CERTIFICATION

I certify that a copy of the July 9, 2018 , agenda of items to be considered by the Drainage and Flooding Committee was placed on the City Hall Bulletin Board on July 3, 2018.

_____

Patrice Fogarty, City Secretary

Chairman

JAY MARTIN
Vice-Chairman

CHUCK ENGELKEN
Member

DANNY EARP
Member

Alternate Member

MINUTES OF DRAINAGE AND FLOODING COMMITTEE MEETING HELD JUNE 11, 2018

The Drainage and Flooding Committee of the City of La Porte met on **Monday, June 11, 2018,** at the City Hall Council Chambers, 604 West Fairmont Parkway, La Porte, Texas, at **5:00 p.m**. to consider the following items of business:

1. Call to Order

Vice-Chairman Jay Martin called the meeting to order at 5:00 p.m. The following members of the Drainage and Flooding Committee were present: Committee Members Chuck Engelken and Danny Earp. Staff Present: Corby Alexander, Don Pennell, Lorenzo Wingate and Sharon Harris.

2. Authorizations

(a) Consider approval or other action regarding minutes of the Drainage and Flooding Committee Meeting held on June 29, 2015 – S. Harris

Committee Member Engelken moved to approve the minutes of the Drainage and Flooding Committee Meeting held on June 29, 2015, as presented. Committee Member Earp seconded. **MOTION PASSED 3/0.**

(b) Consider approval or other action appointing a Chairman and/or Vice-Chairman of the Drainage and Flooding Committee – S. Harris

Committee Member Engelken moved to nominate Jay Martin as Chairman and Danny Earp as Vice-Chairman of the Drainage and Flooding Committee. Committee Member Earp seconded. **MOTION PASSED 3/0.**

3. Staff Reports

(a) S. Shady Ln., Outfall Rehabilitation – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on S. Shady Ln., Outfall Rehabilitation.

There were no questions from the Committee.

(b) Bayside Terrace Drainage Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Bayside Terrace Drainage Improvements.

There were no questions from the Committee.

(c) Pine Bluff Subdivision Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Pine Bluff Subdivision Improvements.

There were no questions from the Committee.

(d) Drainage Interconnect Project at Lakes at Fairmont Green – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on the Drainage Interconnect Project at Lakes at Fairmont Green.

Chairman Martin asked which developer is doing the project. Mr. Wingate responded the Brownstone Group.

(e) Bob's Gully – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Bob's Gully. There were no questions from the Committee.

(f) 6th St., from Madison to W. Main Drainage Analysis – L. Wingate/D. Pennell.

City Engineer Lorenzo Wingate provided an update on 6th St., from Madison to W. Main Drainage Analysis. There were no questions from the Committee.

(g) 8th Street & D. Storm Water Conveyance Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on 8th Street & D. Storm Water Conveyance Improvements. There were no questions from the Committee.

(h) Little Cedar Bayou F216 Phase II Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Little Cedar Bayou F216 Phase II Improvements. There were no questions from the Committee.

(i) Little Cedar Bayou F216 Phase III Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Little Cedar Bayou F216 Phase **III** Improvements.

Vice-Chairman Earp asked if the improvements are in the upcoming budget. City Manager Corby Alexander responded yes.

(j) Browning & Golden Drainage Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Browning & Golden Drainage Improvements and reported the project has been completed. There were no questions from the Committee.

(k) Airport On-site Drainage Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update Airport On-site Drainage Improvements and reported the study has been completed**.** There were no questions from the Committee.

(I) Airport Off-site Drainage Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update Airport Off-site Drainage Improvements and reported the study has been completed**.** There were no questions from the Committee.

(m) Re-grading/Desilting H., L., & P., Streets – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Re-grading/Desilting H L & P Streets.

Vice-Chairman Earp asked how soon can the City began de-silting H Street. Director of Public Works Don Pennell responded it will be 2-3 weeks before a crew can be at the site.

Chairman Martin requested Staff to invite Harris County Flood Control District, Precinct 2, along with Commissioner Jack Morman to a City Council Meeting to provide an update on projects in the City.

(n) Lomax/F101-03-00 Drainage Channel Feasibility Study – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Lomax/F101-03-00 Drainage Channel Feasibility Study.

Vice-Chairman Earp requested Staff provide a timeline and cost estimate on all of the projects for the next meeting.

(o) North P Street Culvert Improvement – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on North P Street Culvert Improvement. There were no questions from the Committee.

(p) Battleground Estates Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Battleground Estates Improvements. There were no questions from the Committee.

(q) Coupland Drive Drainage Improvements – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Coupland Drive Drainage Improvements. There were no questions from the Committee.

(r) Brookglenn Flooding Mitigation Analysis – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on Brookglenn Flooding Mitigation Analysis.

Committee Member Engelken commented recent studies have not done anything for the flooding in the subdivision.

Mr. Wingate advised Staff met with Harris County Flood Control last week and they acknowledged drainage in Brookglenn is an issue and the issues are a priority.

Staff was directed again to have Harris County Flood Control District, Precinct 2, along with Commissioner Jack Morman to a City Council Meeting to provide an update on projects in the City.

(s) City-wide Drainage Model – L. Wingate/D. Pennell

City Engineer Lorenzo Wingate provided an update on the City-wide Drainage Model.

Vice-Chairman Earp informed he will not support new studies when there are items that need to be resolved from 2015 and any additional funds can be used to de-silt ditches.

Chairman Martin asked if there are any funds/grants available after Hurricane Harvey. Mr. Wingate responded yes, 12 applications have been submitted. Chairman Martin requested an update at the next meeting.

4. Set date for next Drainage and Flooding Committee Meeting

The Committee members agreed to set the next meeting date for July 9, 2018, at 5:00 p.m.

5. Committee Comments

There were no Committee comments.

6. ADJOURN

There being no further business, Committee Member Engelken made a motion to adjourn the meeting at 5:50 p.m. Vice-Chairman Earp seconded the motion. **MOTION PASSED UNANIMOUSLY 3/0.**

Respectfully submitted,

_____
Patrice Fogarty, City Secretary

Passed and approved on this 9th day of July 2018.

_____
Chairman, Jay Martin

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Don Pennell	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Harris County Flood Control District and Harris County Precinct 2 are present tonight to provide a report on current and future plans relating to flooding in the City of La Porte.

Action Required:

Receive report.

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITEE AGENDA ITEM

Agenda Date Requested: July 9, 2018

Requested By: Lorenzo Wingate

Department: Public Works

(•) Report ( ) Resolution ( ) Ordinance

Exhibits:

Appropriation	
Source of Funds:	
Account Number:	
Amount Budgeted:	
Amount Requested:	
Budgeted Item:	( ) Yes ( ) No

SUMMARY & RECOMMENDATION

The B112-02-00 channel segment of Willow Springs Bayou is a concrete lined trapezoidal channel that flows from east to west, where it merges with the B112-00-00 Channel (Willow Springs Bayou). Flooding in the Brookglen subdivision is due to backwater from B112-00-00 backing up into B112-02-00, per the 2009 Fairmont Park East & Brookglen Drainage Study, prepared by Klotz Associates. The B112-02-00 has about 200-acres of contributing drainage area and the channel is approximately 2,500 feet in length between B112-00-00 and Canada Rd. Willow Springs Bayou is a concrete line trapezoidal channel with a 4' deep low flow section. Upstream of the confluence, the low flow section is 12' wide with 16' benches and concrete riprap side slopes from Deer Park, upstream of Spencer Hwy., to the confluence with B112-02-00, or 1,200-feet upstream. Downstream of the confluence the low flow section expands to 20' wide with 14' benches and concrete riprap side slopes between B112-02-00 and about 920' downstream where the channel transitions to an earthen trapezoidal section with a 30' bottom width and 4H:1V side slopes. The Willow Springs Bayou continues south of Fairmont Pkwy, into Pasadena and ultimately converges into Armand Bayou.

A drainage study, prepared by Klotz Associates in May 2009, concluded that reductions in the water surface elevation within B112-02-00 can be achieved by reducing the backwater effects of B112-00-00 and by addressing flows coming into the City from upstream.

In August 2009, Klotz Associates prepared a hydrologic and hydraulic analysis to evaluate HCFCD Unit B112-02-00 and determined the benefits of redirecting the B112-02-00 channel to drain east to Unit B109-03-00.

There were two options modeled, with “Option 1” resulting in a maximum water surface reduction in B112-00-00 of -0.60 ft and an average of -0.23 ft, and a maximum flow reduction of -301 cfs. Maximum water surface reduction in B109-00-00 was -0.36 ft and occurs in B109-03-00. Average water surface reductions along B112-02-00 are -0.89 ft between B112-00-00 and Canada Street.

“Option 2” provides an average water surface reduction in B112-00-00 of -0.44 ft and average flow reduction of -269 cfs. Average water surface reduction in B109-00-00 is estimated at -0.13 ft and average flow increase around 65 cfs. Average water surface reductions in B112-02-00 are -1.09 ft between B112-00-00 and Canada Street with associated reduction in flow of 182 cfs.

Channel Reach	Average Changes in WS & Q					
	Option 1		Option 2			
			Phase 1		Phase 2	
	WS Change	Q Change	WS Change	Q Change	WS Change	Q Change
	(ft)	(cfs)	(ft)	(cfs)	(ft)	(cfs)
B112-00-00	-0.23	-140.58	-0.19	-94.37	-0.4	-269
B112-02-00	-0.89	0	-0.32	-242.2	-1.09	-182
B109-00-00	0.03	162.62	0	0	-0.13	65
Costs	**$3M - $4M**		**$1.2M**		**$2.5M**	

As recommended within the 2009 study, staff has been coordinating with HCFCD and neighboring municipalities to address the Brookglen Subdivision drainage issues on a regional level.

ACTION REQUIRED

Approved for Drainage Committee Agenda

_____ _____

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Administration	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

HCFCD Unit F210-00-00 is under the jurisdiction of the City of Morgan's Point and the HCFCD. The HCFCD has a 60-foot drainage easement along the channel from Bayridge Street to East Main Street. The City of Morgan's Point has indicated that HCFC Unit F210-00-00 has channel slope failures and erosion problems which have jeopardized private property and utilities. They have expressed their concerns to the Harris County Flood Control District and have requested a channel maintenance project to correct the channel erosion problems.

Staff meets with the City of Morgan's Point, Harris County, and HCFCD on July 12th to discuss future assessment of this channel.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Harris County Flood Control District's (HCFCD) F101-06-00 Channel system conveys storm water runoff from the Lomax area and ultimately outfalls into Lower San Jacinto Bay. The downstream section of the channel has been improved to ultimate capacity. An existing pipeline corridor, containing several pipelines located at depths ranging from approximately 2' to 18', cross the channel, limiting the depth of potential channel improvements. Moderate/heavy rain events, compounded with backwater conditions from this section of the channel, contributes to wide-spread flooding within the Lomax Area.

Harris County Flood Control staff has informed Public Works staff that survey data has been collected on this most upstream section. Construction drawings are being prepared in-house, by HCFCD staff, to provide direction on clearing overgrowth, trees, and obstructions; minor erosion control and slope stabilization; and desilting. Construction is projected to begin late 4th quarter 2018 or early 1st quarter 2019 (calendar year).

A high level delineation by KSA found the watershed to be about 310 acres, versus the 583 acres used in the 1987 masterplan. Unit F101-06-00 has a capacity of less than 200 cfs, currently, before it rises out of its banks.

The following options were included within the preliminary draft of the feasibility study prepared by KSA:

1) Full excavation of existing 60-ft ROW to construct a concrete channel that will maximize capacity of current ROW, providing approximately 700 cfs, without deepening the channel ($2.4M).

2) Construct concrete channel from Valley View to the existing drop structure; widen the channel beyond the ROW to convey the 100-year event (1,326 cfs) by maintaining existing flow line of the channel ($3.2M).

3) Construct concrete channel from Valley View to the pipeline corridor and deepen the existing grass lined channel, by one foot, from the pipeline corridor to the existing drop structure. ($1.5M).

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Littler Cedar Bayou exists as a narrow, winding, earthen channel between SH 146 and West Barbours Cut Boulevard. Downstream of SH 146, parts of the channel have been straightened and improved with concrete slope paving. Upstream of W Barbours Cut Boulevard, the bayou exists as a relatively straight earthen channel, resembling an oversized roadside ditch.

Improvements to Little Cedar Bayou, from Hwy 146 to Madison, have either been completed or are currently awarded for construction. Improvements include, but are not limited to, excavating and disposing off- site soil as required for the new channel alignment, clearing and grubbing, demolition of existing structures, erosion control, and site restoration for approximately 5,533 LF of channel. Approximately 4,680 LF of channel remains unimproved upstream, with those improvements slated to be included within Phase III. HCFCD has collected survey data on this most upstream section in effort to prepare construction drawings to provide direction for routine maintenance and clearing overgrowth, trees, and obstructions; minor erosion control and slope stabilization; and desilting. Those maintenance operations are projected to begin late 4th quarter 2018 or early 1st quarter 2019 (calendar year).

Estimated cost to regrade & desilt approximately 13,000 linear feet of roadside ditch is approximately $700,000.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

A proposed conditions hydraulic model was developed during the preliminary design for the N P Street Culvert Improvement Project, which recommended replacing the upstream portion of the existing composite structure at N P Street, with structures matching the downstream portion of the composite structure. That project allows for full utilization of the structure's capacity, as well as protect the channel during rainfall events that produce higher amounts of runoff. Per the recommendations of the Phase 1 report, a more detailed Phase 2 engineering analysis of segments F101-00-00 and F101-08-00, that are upstream of N P Street, as well as an evaluation of the roadside ditch drainage conduits within Battleground Estates should be performed to identify necessary improvements to address structural flooding within the area. The more detailed study would evaluate the impacts of culverts located upstream of N P Street, along F-101-00-00 resulting from the 10-, 50-, and 100-year rain event. Channel improvements, and increasing culvert sizes to provide sufficient capacity within culverts along the roadside ditches are the anticipated mitigation efforts.

Estimated cost to improve the channel conveyance upstream of N "P" Street is $565,000.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Klotz Associates, Inc. provided a report in 2011 analyzing flooding in the Battleground Estates, which indicated that flows within segments of the F101 Channel, north of N 'P' Street, rise to a level creating capacity limitations which produce frequent out of banks occurrences. The unimproved upstream channel flows into two forty-two inch (42") corrugated metal pipes (CMP), which feed into the improved two 8' x 10' reinforced concrete boxes downstream. LJA Engineering, Inc. provided Engineering Design Services for the recommended improvements of removing the two upstream forty-two inch (42") corrugated metal pipes and extending the dual 8' x 10' reinforced concrete boxes across N 'P' Street. An Inter-local Agreement with Harris County allows the City to become entitled to a total maximum compensation of $85,000.00, from Harris County, for the construction of drainage improvements across the N 'P' Street right-of-way, intersecting the HCFCD Channel Unit F101-06. Construction is expected to be completed by September 3, 2018.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Staff has been coordinating with Harris County on regarding and desilting roadside ditches along "H", "L", & "P" Streets. Harris County Staff has communicated to City staff that this responsibility would be assigned to their contractor for completion.

The estimated cost for the City to contract ditch re-grading the Harris County right-of-way ditches is $225,000 at $2.00 per foot, loose spoil haul off 2000 yard at $8.00 per yard, and $38,000 contingency.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Approximately 800 linear feet of RCP pipe, ranging in size from 15" to 24", exists within the Bayside Terrace Subdivision, which has not been properly maintained due to access issues attributed to limited access to infrastructure, provided within a five foot utility easement. Portions of Hamilton Street and Fondren Street utilize this system to convey stormwater to its outfall point of Galveston Bay. The system fails to function properly, causing flooding within the adjacent portion(s) of the subdivision. A proposed drainage study would discuss feasibility of rerouting this flows from the 800 feet of RCP, towards Bayside Dr. and utilizing the existing system within Bayside Dr. to convey the storm water within the existing system.

The FY19 budget is anticipated to include an allocation of $100K for preliminary engineering and design phase services. Staff intends to apply for grant funding through the Texas Department of Emergency Management, for the Hurricane Harvey DR-4332 Hazard Mitigation Grant and the Texas Water Development Board's (TWDB) Fiscal Years 2018 – 2019 Flood Protection Grants.

Proposed project cost to be developed as required during preparation of HMGP Grant Application.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

The City retained Civil Concepts in 2017 to perform a "Topographic Survey" of the project area to include the topography of the intersections of the described area and the S 8th Street centerline right-of-way including ditches, drainage structures, and pavement from W Main to W D Street.

Topographic survey determined the drainage flow and capacity of the drainage structures in the study area. Civil Concepts provided recommendations and plan views for improvements in the study area.

Proposed improvements include enclosing the existing roadside ditches and installing approximately 634 linear feet of 48-inch reinforced concrete pipe (RCP); 1,264 linear feet of 42-inch RCP; 484 linear feet 36-inch RCP; and 144 linear feet of 30" RCP; to provide conveyance capacity for at least a 5-year rain event.

Proposed improvements are planned to be completed within FY20. Proposed project cost to be developed as required during preparation of HMGP Grant Application.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

The segment of 6th St from W. Madison St to W. Main St is considered part of Old La Porte, which was generally noted in the City-Wide Drainage Study as not having sufficient storm sewer capacity due to undersized storm sewer, undersized storm inlets, and/or not enough storm inlets. RPS-Klotz provided an analysis of the existing storm sewer system with results indicating that 6th St, from W. Madison St to W. Main St, has significant ponding in storm events as low as 5-year event. Recommendations for improvements were provided, however, areas would still experience ponding depths of approximate 6" to 2'.

Thus, additional efforts are required to understand the root cause of flooding in the area and to recommend improvements to reduce flood risk within the area.

The GLO-CDR has allocated $325,775.30 to the City of La Porte through Harris County's initial MOD and MOD Amendments No. 1 and 2 from the 2015 Floods CDBG-DR funds. Now, to access the funding allocated to the City of La Porte, GLO requires submission of a Non-Housing Project Application detailing the City's proposed non-housing project to repair or mitigate damage that occurred during the Presidentially declared Federal Emergency Management Agency (FEMA) 4223-DR-TX (2015 Memorial Day) and FEMA 4245-DR-TX (2015 Halloween) disasters. Staff has been working with the GLO to ensure that these funds are captured, with plans to utilize the allocation to supplement the project budget.

Preliminary proposed project cost is estimated to be $2.5M.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018	Appropriation
Requested By: Lorenzo Wingate	Source of Funds:
Department: Public Works	Account Number:
Report: ◉ Resolution: ○ Ordinance: ○	Amount Budgeted:
	Amount Requested:
Exhibits:	Budgeted Item: ◉ YES ○ NO

SUMMARY

Texas Water Development Board FY 2018-2019 Flood Protection Grants

The Texas Water Development Board (TWDB) has posted a request for applications (RFA) for $1.8 million in grants for flood protection planning, flood early warning systems, and flood response strategies. In consideration of the significant and deadly flooding events in 2015 and 2016, as well as Hurricane Harvey in 2017, the TWDB is placing a high priority on projects with the merit and ability to protect the health, public safety, and economic prosperity of Texans residing in declared disaster areas dating from May 2015 to September 2017 to include all declared areas from the seven federally declared disasters as well state declared disasters. It is the intent of this program to award at least 50% of the available funding to communities with less than 50,000 in population. Fifty percent of the total cost of the project may be provided to applicants. Application are due by 2:00 PM on Jul 11, 2018. The anticipated award date is September 2018. Awarded projects must be completed no later than August 31, 2021.

TDEM Hurricane Harvey DR-4332 Hazard Mitigation Grant Program

The Hazard Mitigation Grant Program (HMGP) is funded by the Federal Emergency Management Agency (FEMA) and administered by the State of Texas through the Texas Division of Emergency Management. This program is a mitigation grant with a single mission to provide financial assistance to eligible applicants to prevent or reduce future losses to lives and property through the identification and funding of cost-effective mitigation measures; and minimize the costs of future disaster response and recovery. The funding for this program is based on a 75/% federal and 25% local cost share.

Staff submitted Notices of Intent (NOI's) to for the following eleven (11) projects: 6th Street (from Madison to Main); 8th and D; 25th Street Water Well Generator; Battleground Estates; Bayside Terrace; Brookglen Subdivision; Little Cedar Bayou Phase III; Lift Station #40 Generator; F101-06-00 Improvements; Rec Center Generator; Public Works Facility. Staff received notification on June 25, 2018 that the following five projects are considered application eligible: 8th and D; 25th Street Water Well Generator; Bayside Terrace; Brookglen Subdivision; Lift Station #40 Generator.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

REQUEST FOR DRAINAGE COMMITTEE AGENDA ITEM

Agenda Date Requested: July 9, 2018
Requested By: Lorenzo Wingate
Department: Public Works
Report: ◉ Resolution: ○ Ordinance: ○

Appropriation
Source of Funds: _____
Account Number: _____
Amount Budgeted: _____
Amount Requested: _____
Budgeted Item: ◉ YES ○ NO

Exhibits:

1. **Airport Offsite Drainage Study**
2. **Airport Onsite Drainage Study**
3. **6th Street Drainage Improvements**
4. **F101-006-00 Pipeline Corridor Feasibility Study**

SUMMARY

The above referenced and attached drainage studies have been completed.

Action Required:

Approved for Drainage Committee Agenda

Corby D. Alexander, City Manager **Date**

City of La Porte
La Porte Municipal Airport

Offsite Drainage Analysis Report

Prepared for:
City of La Porte

F-14087

STATE OF TEXAS
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1/10/2018

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1. Introduction

Project Overview

This report evaluates drainage interactions between the La Porte Municipal Airport and the neighborhoods to the west (Glen Meadows) and to the north of the airport property. HT&J has also conducted a detailed hydrology and hydraulic analysis of the La Porte Municipal Airport, including its drainage infrastructure and outfall conditions. The airport drains to a culvert along Spencer Highway which outfalls to Big Island Slough, a FEMA studied stream.

The La Porte Municipal Airport is an approximately 300-acre site in the City of La Porte, located to the north of Spencer Highway, west of Sens Road, south of State Highway 225, and east of Underwood Road. The airport site is mostly surrounded by residential development and is on the watershed divide of Armand Bayou watershed, San Jacinto and Galveston Bay watershed, and Clear Creek watershed. Figure 1 presents the location map of the airport property and the offsite areas being evaluated.

Figure 1 - Project Location

Purpose

This report was commissioned by the City of La Porte to investigate the occurrence of sheet flow runoff from the airport to neighborhoods to the west and north of the airport during significant storm events. The City of La Porte asked Civil PEs and HT&J to conduct this separate study alongside a TxDOT Aviation Division study of drainage within the airport. The purpose of this report is to evaluate any drainage impacts resulting from the airport draining into bordering neighborhoods to the north and west of the airport property. The problem areas along the north and west borders of the airport were identified, which coincide with the two major drainage systems of the airport. Overflow rates to these areas were estimated and potential improvements were evaluated to mitigate the severity of flooding to the neighboring offsite areas.

Assumptions and Constraints

For the evaluation of the airport drainage system, the Federal Emergency Management Agency (FEMA) effective models were used for evaluation of tailwater in the Big Island Slough channel. The HEC-RAS model for the channel (B106-00-00) and the HEC-HMS model for the Armand Bayou watershed were obtained from Harris County Flood Control District (HCFCD) and appropriate information was used.

For evaluating the drainage system of the offsite neighborhood to the north, tailwater conditions were assumed to be at the top of the natural channel. This was used as an estimate of the 25-year tailwater conditions that are recommended in the City's design standards. The numerous small culverts through the residential driveways were not modeled due to the instability this would create in the model.

When modeling natural channels, cross sections were taken at the mid-point of each channel and the channel was assumed to be uniform using that cross section. The width of cross sections varied from 50 ft to 400 ft.

Texas Department of Transportation (TXDOT) intensity duration parameters were used to calculate the rainfall intensity to use in rational method for runoff calculation from each of the sub-basins. The XPSWMM model is calibrated at the sub-basin level against the rational method to generate the similar peak flow. TxDOT parameters were used to be consistent with on-site and off-site evaluation.

Manning's Roughness Coefficient (n) values were assumed, and are displayed below in Table 1.

Table 1 – Manning's n Values

Material Type	Manning's n
Reinforced Concrete Pipe (RCP) Culvert	0.014
Concrete Pilot Channel	0.025
Natural Channel	0.035
Natural Channel Overbanks	0.040

Previous Studies

The most recent drainage analysis for the City of La Porte is the City Wide Drainage Study, prepared by Klotz Associates, Inc. in January of 2009. In this study, the La Porte Municipal Airport and bordering areas were not evaluated in detail.

The Spencer Highway drainage design plan (Record Drawings dated 1994) is used for the storm sewer along the Highway.

2. Existing Conditions

Location

The La Porte Municipal Airport is located in the City of La Porte, Texas, to the south of TX-225/Pasadena Highway and to the west of TX-146. It is located at the northeast corner of Spencer Highway and Farrington Drive.

The neighborhoods in question are located to the west and north of the airport site. The neighborhood to the west is located just west of Farrington Drive, north of Spencer Highway and east of the Big Island Slough channel, and for this report is known as Glen Meadows. The neighborhood to the north is located between the northern border of the airport and N Avenue H. Figure 1 shows these neighborhoods.

Floodplains

The corner of the Glen Meadows subdivision near Spencer Highway adjacent to the Big Island Slough is within the 100-year FEMA floodplain, with a larger portion of the southern and western areas within the 500-year floodplain. The neighborhood to the north of the airport site is outside the 100-year floodplain.

Exhibit 1 presents the effective FEMA floodplains at the airport site. The 100-year Base Flood Elevation (BFE) ranges from elevation 19.70 near Spencer Highway to elevation 21.0 near the north end of the Glen Meadows subdivision.

Topography and Drainage

Topography of the site and bordering areas was developed using a combination of a detailed survey provided by Gessner Engineering and LiDAR data downloaded from Houston-Galveston Area Council (HGAC). A Digital Terrain Model (DTM) was developed from the detailed survey and drainage areas were determined for each ditch and drainage structure using GIS tools. Overall, the airport property generally slopes north to south, with isolated high and low points. The ground elevations on site range from approximately 24 feet to 20 feet.

The neighborhood to the north of the airport slopes southwest to northeast along Northern Avenue H from approximately 25 feet to 20 feet. The neighborhood to the west of the airport generally slopes northeast to southwest toward Spencer Highway from approximately 23 feet to 19 feet.

Land Use

The majority of the 300 acre airport site is open area covered in native grasses. The impervious area on the site is made up of runways, taxiways, parking lots, storage lots, and some commercial area. Approximately 20 percent of the site is impervious.

The Glen Meadows neighborhood to the west is a single family residential development with an average lot size of approximately 0.2 acres. The neighborhood to the north along N Avenue H is made up of larger residential lots of approximately 2 acres. The lots along N Avenue H are made up of mostly open space covered in grass.

Existing Drainage Structures

The drainage infrastructure on the airport is made up of Reinforced Concrete Pipe (RCP) culverts and ditches in that outfalls into an underground storm sewer in Spencer Highway. The existing onsite RCP culverts range in diameter from 15-inch to 36-inch.

A roadside ditch along Farrington Drive receives runoff from approximately 70 acres. This roadside ditch, on average, is about 2.5 feet deep and top width varies to maximum 30 feet. Two culvert structures (See Exhibit 2) restrict flow to the system.

The drainage infrastructure in the neighborhood to the north of the airport site consists of grass-lined drainage ditches and RCP culverts to transfer runoff beneath driveways. The ditch along the north property boundary receives runoff from approximately 76 acres. Three ditch systems converge to a 1.6 ft deep, 35 ft wide ditch which drains south along the east edge of the airport property toward Spencer Highway (See Exhibit 3).

The drainage infrastructure in the neighborhood to the west of the airport site consists of an underground storm sewer system containing RCP ranging in size from 18-inch to 60-inch and streets that are curbed and guttered. The storm sewer flows west and has three separate outfalls to the Big Island Slough channel. Storm sewer information for the Glen Meadows subdivision was obtained from the City of La Porte online GIS system. The Glen Meadows Park along the Big Island Slough channel at the north end of the subdivision functions as a detention basin for part of the drainage system with a footprint of approximately 7.3 acres.

Exhibit 1 presents the Harris County watershed boundaries.
Exhibit 2 presents a detailed look at the existing drainage infrastructure for the Farrington Drive ditch.
Exhibit 3 presents the existing drainage infrastructure for the ditch along the northern boundary of the airport.

Existing Outfalls

The drainage system along North Avenue H to the north of the airport property outfalls into a roadside ditch system along Sens Road.

The drainage system in the neighborhood to the west of the airport property has three separate outfalls into the Big Island Slough. Figure 2 shows the location of these outfalls.

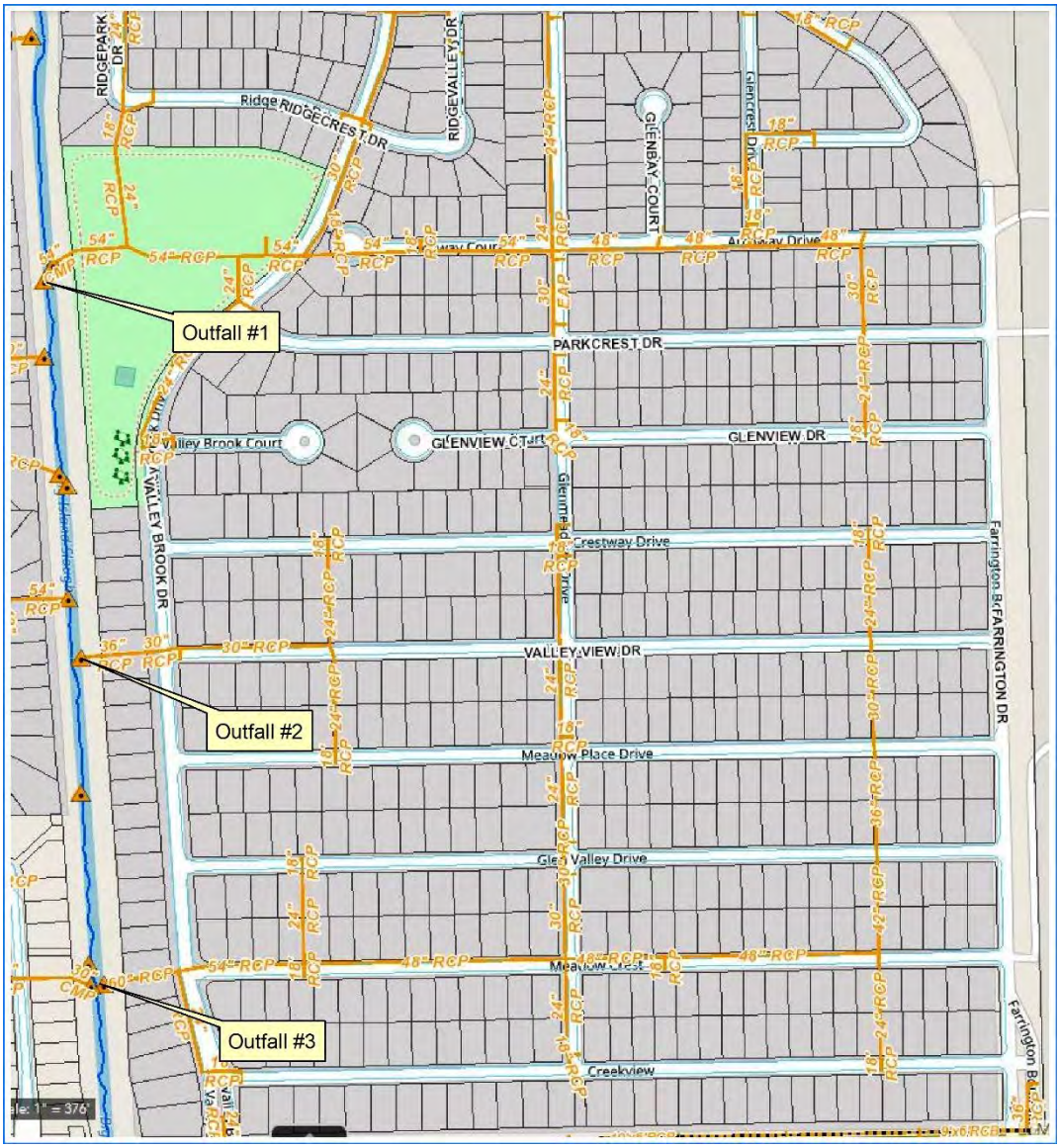


Figure 2 - Glen Meadows Outfall Locations

Both the Farrington Drive roadside ditch and northern airport ditch systems drain to Spencer Highway which outfalls to Big Island Slough channel.

3. Drainage Design Criteria

Texas Department of Transportation (TxDOT) Design Criteria

Table 2 presents the TxDOT intensity-duration-frequency coefficients for use in the rational method runoff calculations. TxDOT parameters were used to be consistent with the airport on-site study.

Table 2 – Rainfall Intensity Coefficients

Rain Event Frequency	b	d	e
2-year	68	7.9	0.800
5-year	70	7.7	0.749
10-year	81	7.7	0.753
25-year	81	7.7	0.724
50-year	91	7.7	0.728
100-year	91	7.9	0.706

The Kerby-Kirpich Method was used in adherence with the TxDOT Hydraulic Design Manual Chapter 4 Section 11 to calculate the Time of Concentration for each basin. The peak flow values were used to determine the storage coefficient for each individual drainage area on site.

City of La Porte Design Criteria

The City of La Porte Public Improvement Criteria Manual (PICM) Chapter 5, Storm Water Design Criteria, was reviewed. The design event for storm sewer systems is the 5-year storm event, with requirements to keep the hydraulic grade line below the gutter elevation.

4. Existing Conditions Drainage Analysis

Hydrologic Analysis

Hydrologic calculations were performed in accordance with the TxDOT Hydraulic Design Manual, in order to be consistent with on-site and off-site analysis. The Rational Method was used to calculate peak flows using the TxDOT rainfall intensity coefficients in Table 2. Contributing drainage area boundaries were established based on the LiDAR data from HGAC and topographic survey and drainage culverts detail data provided by the Gessner Engineering survey. Rainfall totals were obtained from the Harris County Flood Control District H&H Manual for the purpose of developing hydrograph and model simulation, and can be found in Table 3.

Table 3 – Harris County Hydrologic Region 3 Rainfall (inches)

Duration	Storm Frequency							
	2-yr	**5- yr**	**10- yr**	**25- yr**	**50- yr**	**100- yr**	**250- yr**	**500- yr**
5 Minutes	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
15 Minutes	1.1	1.4	1.5	1.7	1.9	2.1	2.3	2.5
30 Minutes	1.5	1.9	2.1	2.4	2.7	3.0	3.4	3.7
60 Minutes	2.0	2.5	2.9	3.4	3.8	4.3	5.0	5.5
2 Hours	2.4	3.1	3.7	4.4	5.0	5.7	6.8	7.7
3 Hours	2.7	3.5	4.2	5.1	5.9	6.8	8.2	9.4
6 Hours	3.2	4.4	5.3	6.6	7.7	9.1	11.2	13.1
12 Hours	3.8	5.3	6.4	8.0	9.5	11.1	13.6	15.9
24 Hours	4.5	6.4	7.8	9.8	11.6	13.5	16.6	19.3
2 Days	5.3	7.5	9.0	11.2	13.1	15.1	18.1	20.7
4 Days	6.2	8.7	10.5	12.9	14.8	16.9	19.8	22.3

The Rational Method Runoff Coefficient for each drainage area was calculated with land cover imperviousness data downloaded from the National Land Cover Database and verified with the latest aerial photographs. The average percent imperviousness was converted to a runoff coefficient (C) value decimal with 100 percent impervious equal to a 0.9 C-value and zero percent impervious equal to 0.2 C-value.

In order to fully integrate the system and define the overall drainage impacts, XPSWMM models were set up for the airport site and northern bordering neighborhood with links and nodes representing the drainage areas and drainage structures.

The Green-Ampt Method was used to calculate infiltration losses, and Harris County Region 3 loss parameters from HCFCD design manual were used. Table 4 presents the parameters.

Table 4 – Harris County Hydrologic Region 3 Loss Parameters

Parameter	Value
Initial Loss (in)	0.100
Moisture Deficit	0.385
Suction (in)	12.45
Conductivity (in/hr)	0.024

For each individual drainage area, the Clark Method was used to develop a hydrograph. Time of Concentration (TC) was calculated using the Kerby-Kirpich Method, per the TxDOT Hydraulic Design Manual Chapter 4 Section 11.

$$T_c = T_{ov} + T_{ch}$$

Where:

$$T_{ov} = overland\ flow\ time\ (min) = K(L\ x\ N)^{0.467} S^{-0.235}$$

$$K = 0.828$$

$$L = overland\ flow\ length\ (feet)$$

$$N = dimensionless\ retardance\ coefficient = 0.15$$

$$S = dimensionless\ slope\ of\ terrain$$

$$T_{ch} = channel\ flow\ time\ (min) = KL^{0.770}S^{-0.385}$$

$$K = 0.0078$$

$$L = channel\ flow\ length\ (feet)$$

$$S = dimensionless\ main\ channel\ slope$$

The Reservoir Storage Coefficient (R) is necessary to develop the runoff hydrograph from the sub-basins in a Clark Hydrograph method. The R value is adjusted to calibrate the XPSWMM model against the Rational Method, and the peak flow at the hydrograph is matched to the peak flow estimated from rational method.

Hydraulic Analysis – La Porte Airport Property

An XPSWMM model used to evaluate existing conditions of La Porte Municipal Airport was modified to analyze the two offsite areas separately.

Figure 3- XPSWMM Model Layout

For the 5-year and 10-year storm events, the tailwater was modeled with the water surface at the top of the outfall pipe (10 x 6 feet box outfall) into the Big Island Slough. For storm events larger

than the 10-year event, the tailwater conditions were modeled using a stage time series. The time series was constructed from the HEC-HMS hydrograph and the HEC-RAS flow vs. stage relationships in the Big Island Slough channel (B106 00-00). Flow vs. stage data for different storm events was obtained from FEMA effective HEC-HMS and HEC-RAS models.

The neighborhoods to the west of the airport property (both north and south of Spencer Highway) have their own storm sewer systems that outfall to the Big Island Slough separate from the Spencer Highway system.

Ditches that overbanked or culverts that were surcharged were identified and on-site ponding was delineated. Exhibits 4, 5, and 6 show the inundation conditions in the existing system for the 5-year, 10-year, and 100-year storm events.

There are two areas of interest that border the La Porte Municipal Airport property, one to the north along N Avenue H, and the other to the west of the airport, west of Farrington Drive. Both areas were evaluated to determine the following:

- sheet flow direction
- maximum channel conveyance capacity of the bordering airport ditches
- overflow rates for the 2-, 5-, 10-, 25-, 50- and 100-year storm events
- water surface elevations for the 2-, 5-, 10-, 25-, 50- and 100-year storm events

Exhibit 4 presents the ponding during the 5-year storm event for the existing conditions.
Exhibit 5 presents the ponding during the 10-year storm event for the existing conditions.
Exhibit 6 presents the ponding during the 100-year storm event for the existing conditions.

Farrington Drive (West of Airport) Hydraulic Analysis

The airport channel that runs along Farrington Drive slopes from an elevation of 21.5 ft to an elevation of 16.8 ft. The centerline of Farrington Drive serves as the high bank to the west. Figure 4 and Figure 5 below display the profile and typical cross section of the channel.

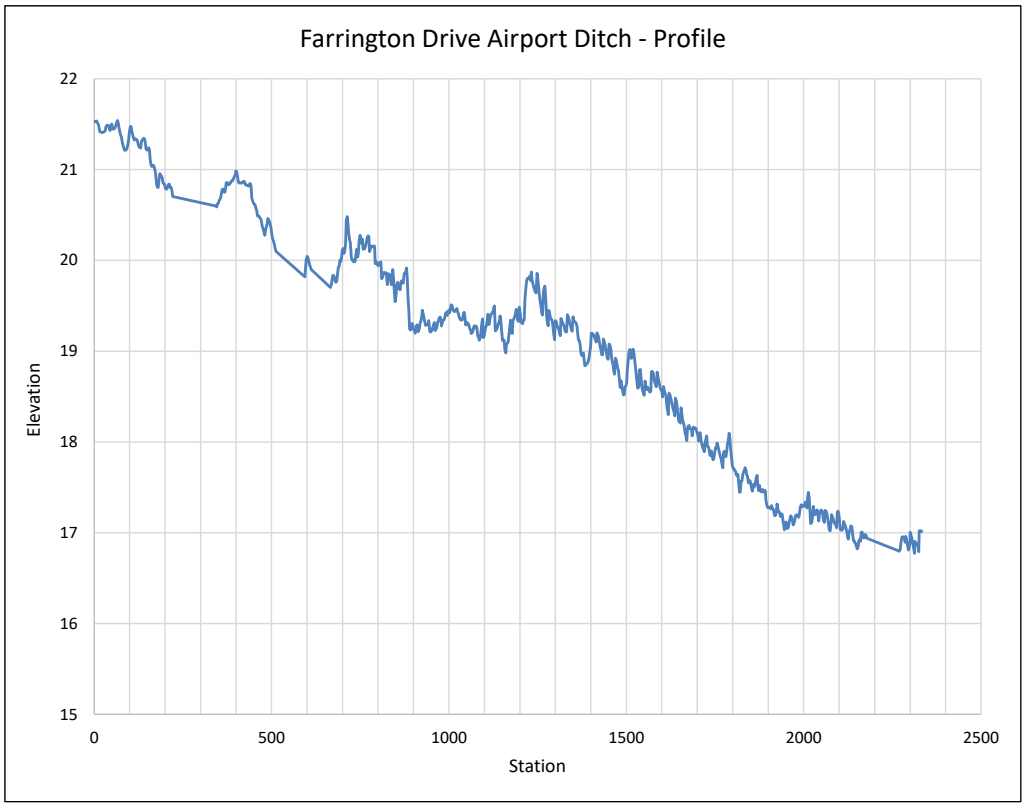


Figure 4 – Farrington Drive Roadside Ditch Profile

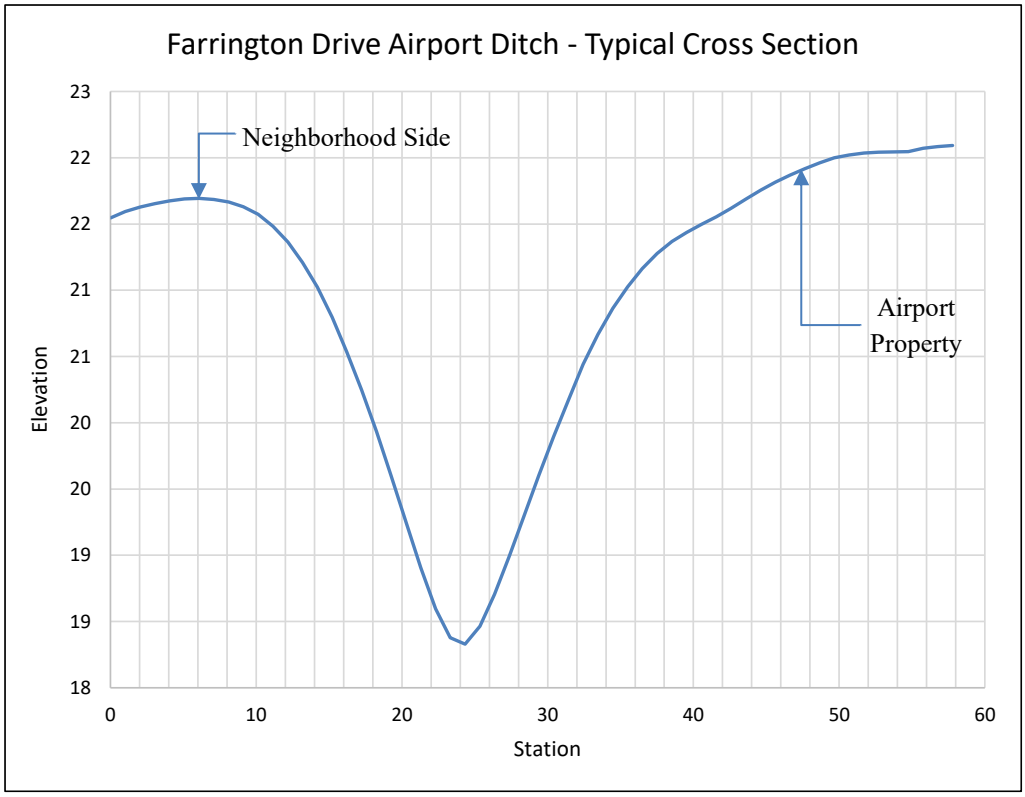


Figure 5 – Farrington Drive Roadside Ditch Typical Cross Section

The maximum conveyance was determined for the channel using two separate methods. The first was calculating flow at bankfull elevation using Manning's Equation with a roughness coefficient of 0.035. This method resulted in an approximate maximum conveyance of 155 CFS. However, this does not take into account the downstream control of the 30-inch RCP that conveys the flow from the channel into the Spencer Highway storm sewer system. Using the XPSWMM model of the existing airport drainage system, it was determined that the outfall to Spencer Highway controls the flow down to only approximately 34 CFS when the channel is flowing full.

Four study points were chosen to evaluate any overflow that occurs between the airport site and the neighborhood west of Farrington Drive. Figure 6 presents the location of the study points, located at Archway Drive, Crestway Drive, Meadow Place Drive, and Meadow Crest Street.

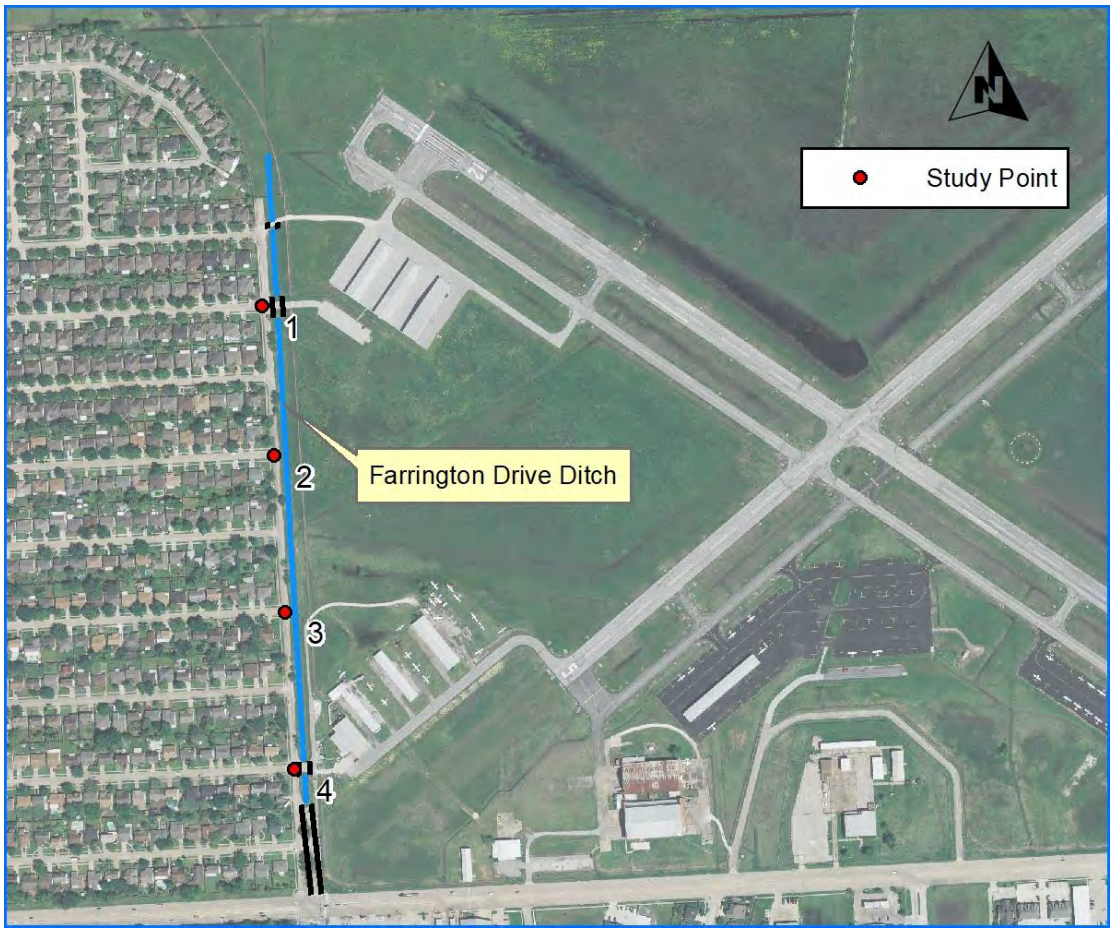


Figure 6 – Farrington Drive Study Points

The water surface elevation at different storm events was compared to the elevation at Farrington Road, and from that differential the overflow rate was calculated. The existing XPSWMM model was expanded to calculate the overflow rate at each study point. Links were added to simulate runoff leaving the channel and flowing over Farrington Drive. The Links were given a width matching the distance to the upstream study point, so that all overflow along Farrington Drive was accounted for in four stream segments. The following tables present the comparison of road

elevation, water surface elevation, and overflow rate for existing conditions at each of the four study points.

Table 5 – Farrington Drive Water Surface Elevations – Existing Conditions

Study Point	Road Elevation	WSE					
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	21.81	22.22	22.19	22.14	22.04	21.91	21.60
2	20.61	21.17	21.14	21.10	21.04	20.97	20.60
3	20.79	20.86	20.77	20.71	20.59	20.49	20.04
4	19.75	20.39	20.18	20.14	20.02	19.95	19.43
Study Point	**Road Elevation**	**Difference (WSE - Road El.) [ft]**					
		100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	21.81	0.41	0.38	0.33	0.23	0.10	-0.21
2	20.61	0.56	0.53	0.49	0.43	0.36	-0.02
3	20.79	0.07	-0.02	-0.08	-0.20	-0.30	-0.75
4	19.75	0.64	0.43	0.39	0.27	0.20	-0.32

Table 6 – Farrington Drive Overflow Rates – Existing Conditions

Study Point	Overflow Rate (cfs)					
	100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	14.41	10.39	6.13	1.93	0.17	0.00
2	36.28	30.36	24.47	17.32	10.76	0.00
3	0.07	0.00	0.00	0.00	0.00	0.00
4	35.37	8.29	5.20	2.16	0.82	0.00
Total:	86.12	49.04	35.80	21.41	11.75	0.00

Table 5 indicates the overtopping of roadside ditch along Farrington Drive at 5-year storm event, and the maximum depth is about 0.64 ft. near Creek View Drive. The City of La Porte design standards require the neighborhood storm sewer system to maintain an HGL below the street gutter elevation during the 5-year storm event. However, this is beyond the scope of this report and HT&J did not evaluate the neighborhood storm sewer system.

A total of approximately 86 cfs of sheet flow is introduced to the neighborhood during the 100-year storm event, and approximately 12 cfs is introduced during the 5-year storm. This likely results in roadway conveyance during this storm event. Total sheet flow introduced will result in an increase flooding in the neighborhood, but the extent is not uniform throughout. The resulting additional flooding depth varies from 0.07 ft. to 0.64 ft. in the existing conditions. We expect the neighborhood storm sewer system reaches its capacity when this additional sheet flow is introduced. Thus increasing the overall sheet flow depth within the neighborhood.

The overall runoff from each subbasin was estimated using the rational method using a C value of 0.45 and rainfall intensity values from Harris County Flood Control District. Table 7 presents the drainage area data and peak runoff for each outfall in the Glen Meadows subdivision.

Table 7 – Glen Meadows Rational Method Calculations

Outfall	Outfall Size (in)	Drainage Area (acres)	C	Time of Concentration (min)	Q, 2-YR (CFS)	Q, 5-YR (CFS)	Q, 10-YR (CFS)	Q, 25-YR (CFS)	Q, 100-YR (CFS)
1	54	76	0.45	36.44	95.03	123.66	143.06	167.24	203.43
2	36	14		30.92	19.20	24.79	28.59	33.33	40.38
3	60	70		36.13	87.96	114.41	132.34	154.68	188.12
Total	**-**	**160**	**-**	**-**	**202.19**	**262.85**	**303.99**	**355.25**	**431.93**

Though detailed analysis was not conducted for the neighborhood drainage system, we believe the neighborhood storm sewer system was designed for 2-year storm event.

North Avenue H (North of Airport) Hydraulic Analysis

There are two airport channels that run along the northern border of the airport property, that converge before turning to flow south. The typical cross section for the converged channel is shown below in Figure 7.

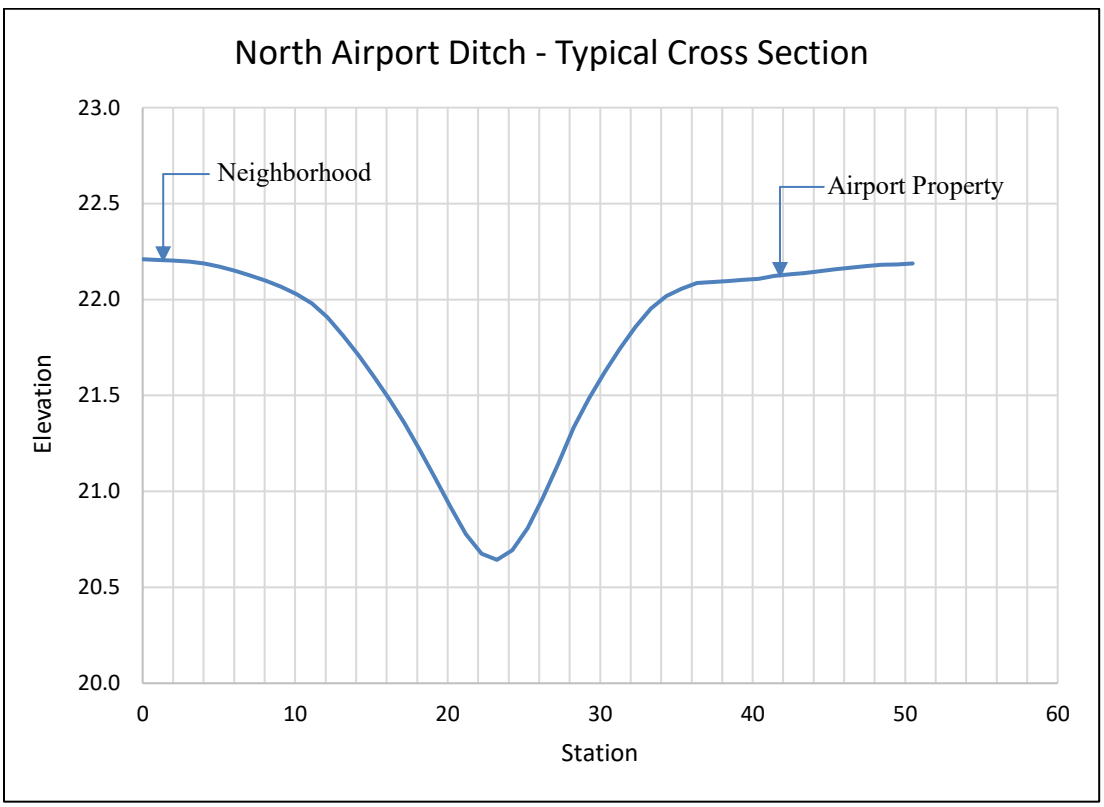


Figure 7 – Northern Channel Typical Cross Section

The maximum conveyance was determined for the converged channel by calculating flow at bankfull elevation using Manning's Equation with a roughness coefficient of 0.035. This method resulted in an approximate maximum conveyance of 64 CFS.

Four study points were chosen to evaluate any overflow that occurs between the airport site and the neighborhood along North Avenue H. Figure 8 presents the location of the study points, located along the northern property boundary of the airport.

Figure 8 – North Avenue H Study Points

The water surface elevation at different storm events was compared to the elevation at the airport property northern boundary, and from that differential the overflow rate was calculated. The existing XPSWMM model was expanded to calculate the overflow rate at each study point. Links were added to simulate runoff leaving the channel and flowing over the northern boundary. The Links were given a width matching the distance to the upstream study point, so that all overflow along the northern airport channels was accounted for in four stream segments. The following tables present the comparison of road elevation, water surface elevation, and overflow rate for existing conditions at each of the four study points.

Table 8 – North Avenue H Water Surface Elevations – Existing Conditions

Study Point	Road Elevation	WSE					
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	23.29	23.65	23.63	23.61	23.58	23.55	23.50
2	23.50	23.00	22.92	22.84	22.71	22.58	22.41
3	23.50	22.98	22.90	22.82	22.69	22.56	22.39
4	22.22	22.87	22.77	22.69	22.56	22.43	22.23
Study Point	**Road Elevation**	**Difference (WSE - Road El.) [ft]**					
		100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	23.29	0.36	0.34	0.32	0.29	0.26	0.21
2	23.50	-0.50	-0.58	-0.66	-0.79	-0.92	-1.09
3	23.50	-0.52	-0.60	-0.68	-0.81	-0.94	-1.11
4	22.22	0.65	0.55	0.47	0.34	0.21	0.01

Table 9 - North Avenue H Overflow Rates – Existing Conditions

Study Point	Overflow Rate (cfs)					
	100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	11.63	9.84	7.87	5.81	4.36	1.08
2	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
4	73.90	46.59	28.25	10.01	2.23	3.01
Total:	85.53	56.44	36.12	15.83	6.59	4.09

The water levels overtop into the neighborhood along North Avenue H at Study Points 1 and 4. The City of La Porte design standards require the neighborhood storm sewer system to maintain an HGL below the street gutter elevation during the 5-year storm event. However, this is beyond the scope of this report and HT&J did not evaluate the neighborhood storm sewer system.

A total of approximately 86 cfs of sheet flow is introduced to the neighborhood during the 100-year storm event, and approximately 7 cfs is introduced during the 5-year storm. This likely results in roadway conveyance during this storm event. Total sheet flow introduced will result in an increase flooding in the neighborhood, but the extent is not uniform throughout. The resulting additional flooding depth varies from 0.01 ft. to 0.65 ft. in the existing conditions.

Proposed Conditions Drainage Analysis

Proposed Drainage Improvements Farrington Drive

The drainage improvements discussed below are currently recommended to the TxDOT Aviation Division for drainage on the airport property. These improvements will consequently improve drainage for the bordering neighborhoods.

A 1,150 LF section of ditch is proposed to expand to function as a linear detention (See Exhibit 9). The expansion will provide approximately 3.6 ac-ft. of storage. The downstream culvert structures are proposed to be replaced with 2-42" pipes to increase conveyance. A typical cross section of the ditch is presented in Figure 9.

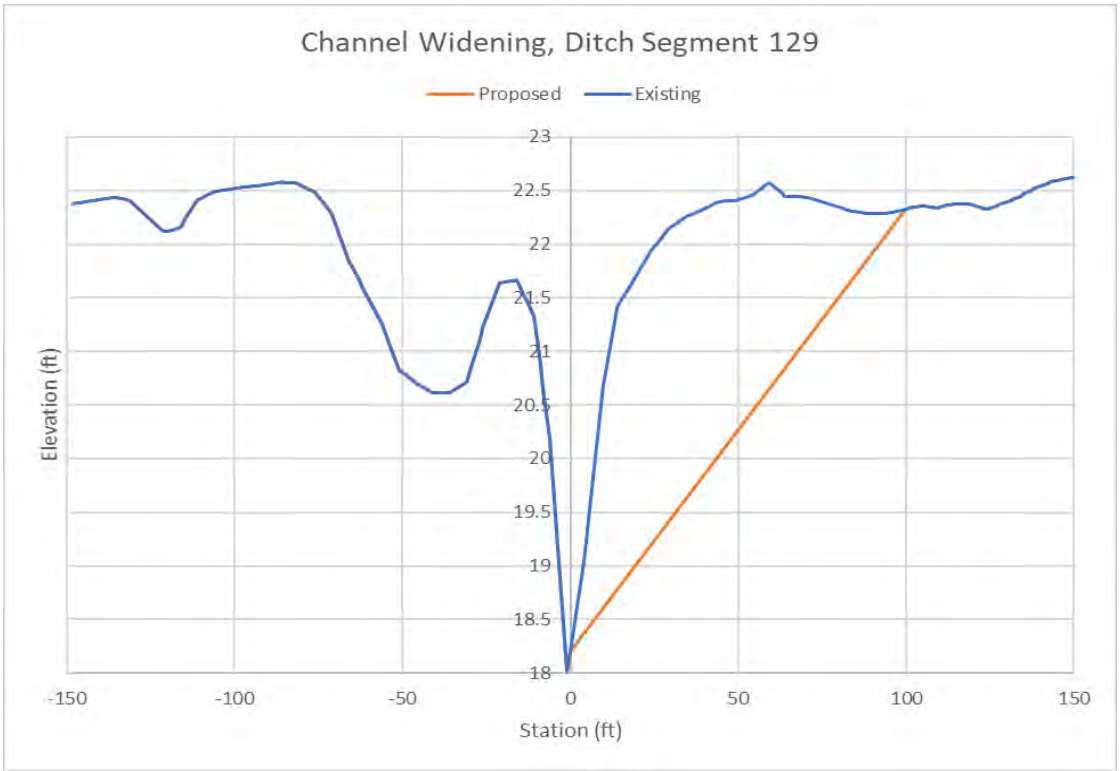


Figure 9 – Widened Ditch Geometry along Farrington Road

Proposed Drainage Improvements North Side

Expansion of the ditch system draining the northeastern quadrant of the airport is proposed to alleviate flooding to the northern neighborhood. The flowline of the main drainage ditch running along the border of the airport is proposed to drop by 2.5 ft. This lowers the hydraulic grade line below the neighboring bank and provides linear detention.

The linear detention is proposed at two locations:

1) Northern Ditch Section (See Exhibit 3) - For approximately 1000 LF, the existing swale will be expanded to 24 ft. bottom width channel, and the average depth is about 2.0 ft. The existing drainage structure 24 will be replaced by a 24" pipe. This pipe functions as a restrictor for this linear detention basin. This detention provides approximately 2.66 ac-ft. of storage.
2) East Ditch Section - The right bank of the ditch (towards the airport side) will be sloped at 1% from the channel bottom to the existing ground elevation. The Ditch segments 7, 16, and 21 are proposed to function as linear detention. The proposed channel section is about 300 ft. wide at the top (See Figure 10). The modifications to Ditch Segments 7 and 21 provide approximately 4.80 ac-ft. of additional storage.

Improvements to the northern section involve about 7.5 ac-ft of soil excavation for both the linear detention and conveyance improvement.

Figure 10 – Linear Detention Geometry

Other modifications to the ditch system include:

1) Several ditch sections will be lowered with minimal changes to bottom width or side slopes.

2) A section will be widened by approximately 10ft to provide additional conveyance. (See Figure 11).
3) An existing 12" culvert will be upsized to 24" at the downstream end of the northern linear detention.

Figure 11 – Widened Ditch Geometry

The proposed ditch and culvert improvements are shown in Exhibit 9.

Hydrologic Analysis

No changes were made to hydrologic parameters from the existing model as there are no significant anticipated land use changes.

Hydraulic Analysis– La Porte Airport Property

The existing XPSWMM model was updated to reflect proposed changes to structures and ditches.

With the proposed improvements the La Porte Municipal Airport satisfies the FAA requirements while improving drainage conditions to the neighboring communities.

During the 10-year storm event, the proposed improvements contain flow within the ditches. Though some overtopping occurs at the west side of the airport (along Farrington Road), flooding conditions are significantly improved. During the 5-year event flow is mostly contained within the roadside ditch along the western drainage ditch with a minimal impact at the south most end.

Exhibit 10 presents the ponding during the 5-year storm event for the proposed conditions.
Exhibit 11 presents the ponding during the 10-year storm event for the proposed conditions.
Exhibit 12 presents the ponding during the 100-year storm event for the proposed conditions.

Farrington Drive (West of Airport) Hydraulic Analysis

After the proposed improvements to the La Porte Municipal Airport system were made to the XPSWMM model, the interaction between the airport property and the Glen Meadows neighborhood, west of Farrington Drive, was examined again. The location of the study points remained the same, however the channel was altered as described in the section "Proposed Improvements" earlier in this report. The following tables present the water surface elevations, overflow rates, and the comparison of overflow rates between the existing and proposed conditions.

Table 10 – Farrington Drive Water Surface Elevations – Proposed Conditions

Study Point	Road Elevation	WSE					
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	21.81	22.02	22.02	21.89	21.68	21.51	21.23
2	20.61	20.98	20.81	20.69	20.49	20.30	20.01
3	20.79	20.80	20.57	20.46	20.24	20.02	19.70
4	19.75	20.42	18.89	18.75	18.33	18.28	17.95
Study Point	**Road Elevation**	**Difference (WSE - Road El.) [ft]**					
		100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	21.81	0.21	0.21	0.08	-0.13	-0.30	-0.58
2	20.61	0.37	0.20	0.08	-0.12	-0.31	-0.60
3	20.79	0.01	-0.22	-0.33	-0.55	-0.77	-1.09
4	19.75	0.67	-0.86	-1.00	-1.42	-1.47	-1.80

Table 11 – Farrington Drive Overflow Rates – Proposed Conditions

Study Point	Overflow Rate (cfs)					
	100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
1	1.45	1.29	0.12	0.00	0.00	0.00
2	11.56	2.12	0.26	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
4	44.22	0.00	0.00	0.00	0.00	0.00
Total:	57.22	3.41	0.37	0.00	0.00	0.00

Table 12 – Farrington Drive Overflow Rates – Difference (PR–EX)

Study Point	Overflow Rate (cfs)					
	100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	-12.96	-9.10	-6.02	-1.93	-0.17	0.00
2	-24.71	-28.25	-24.21	-17.32	-10.76	0.00
3	-0.07	0.00	0.00	0.00	0.00	0.00
4	8.85	-8.29	-5.20	-2.16	-0.82	0.00
Net:	-28.90	-45.63	-35.43	-21.41	-11.75	0.00

With the proposed drainage improvements to the airport property, the overflow rate across Farrington Drive was decreased by approximately 29 cfs for the 100-year storm event and decreased completely (approximately 12 cfs) for the 5-year storm event. Table 12 presents the net change in the overflow rate for all storm events studied. The runoff from 10-year storm event is contained within the roadside ditch. The maximum sheet flow depth is reduced throughout and is limited to 0.21 ft. for 50-year storm event. Though the flow is reduced for 100-year storm event, the maximum flooding depth is unaffected for the 100-year storm event due to the resulting impact of the tailwater conditions.

North Avenue H (North of Airport) Hydraulic Analysis

After the proposed improvements to the La Porte Municipal Airport system were made to the XPSWMM model, the interaction between the airport property and the neighborhoods along North Avenue H were examined again. The location of the study points remained the same, however the northern channels was altered as described in the section "Proposed Improvements" earlier in this report. The same methods were used to obtain values for water surface elevation and overflow rate. The following tables present the water surface elevations, overflow rates, and the comparison of overflow rates between the existing and proposed conditions.

Table 13 – North Avenue H Water Surface Elevations – Proposed Conditions

Study Point	Road Elevation	WSE					
		100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	23.29	23.07	22.95	22.69	22.69	22.37	22.13
2	23.50	22.81	22.43	22.08	22.08	21.59	21.26
3	23.50	22.80	22.40	22.05	22.05	21.52	21.15
4	22.22	22.73	22.30	22.00	22.00	21.47	21.10
Study Point	**Road Elevation**	**Difference (WSE - Road El.) [ft]**					
		100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	23.29	-0.22	-0.34	-0.60	-0.60	-0.93	-1.16
2	23.50	-0.69	-1.07	-1.42	-1.42	-1.91	-2.24
3	23.50	-0.70	-1.10	-1.45	-1.45	-1.98	-2.36
4	22.22	0.51	0.08	-0.22	-0.22	-0.75	-1.12

Table 14 – North Avenue H Overflow Rates – Proposed Conditions

Study Point	Overflow Rate (cfs)					
	100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
4	36.67	0.15	0.00	0.00	0.00	0.00
Total:	36.67	0.15	0.00	0.00	0.00	0.00

Table 15 – North Avenue H Overflow Rates – Difference (PR–EX)

Study Point	Overflow Rate (cfs)					
	100-yr	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
1	-11.63	-9.84	-7.87	-5.81	-4.36	-1.08
2	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
4	-37.23	-46.45	-28.25	-10.01	-2.23	-3.01
Net:	-48.86	-56.29	-36.12	-15.83	-6.59	-4.09

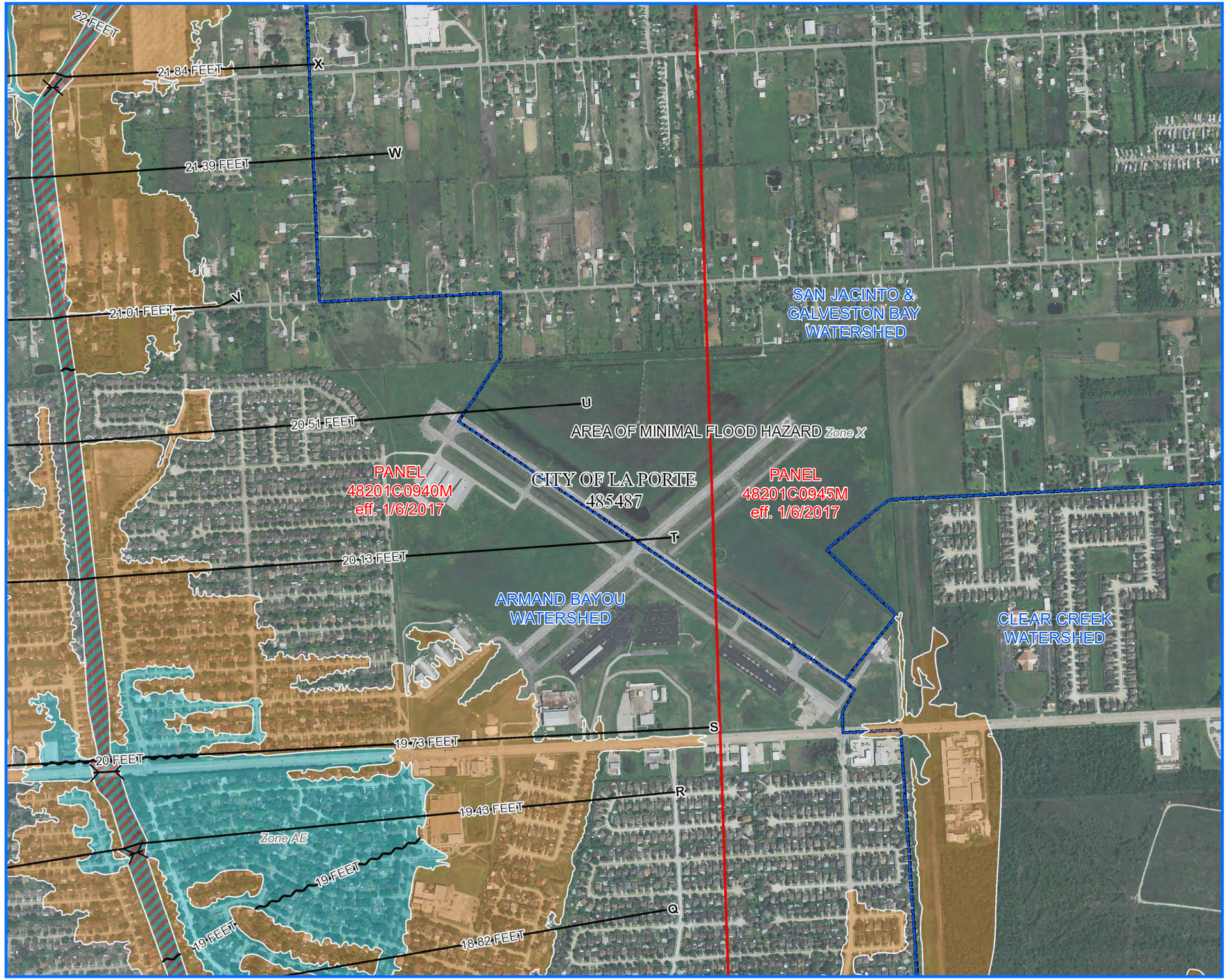
With the proposed drainage improvements to the airport property, the overflow rate into the North Avenue H system was decreased by approximately 49 cfs for the 100-year storm event and decreased completely (approximately 7 cfs) for the 5-year storm event. Table 15 presents the net change in the overflow rate for all storm events studied. The largest decrease in overflow rate occurred at Study Point 4, where the two northern ditches converge into one drainage ditch. The runoff is contained within the channel for 25-year storm event and the flooding depth is limited to 0.08 ft. during the 50-year storm event, and 0.51 ft. during the 100-year storm event.

5. Conclusions and Recommendations

Given the current conditions, runoff overflows from the La Porte Municipal Airport property into the bordering western and northern neighborhoods generally at the 5-year storm event. With the proposed improvements currently recommended to the TxDOT Aviation Division for the airport drainage system, there will be a significant reduction in storm water overflow to the neighborhoods. The improvements to the drainage ditches along the west and north sides of the airport property will help contain the airport runoff and convey it to the outfall system. The runoff will generally contain in a channel for storm event up to 25-year return period.

It is the opinion of HT&J that the proposed improvements to the La Porte Municipal Airport drainage system will be beneficial to the City of La Porte. The improvements will minimize the impacts of excess floodwaters from the airport property and keep the neighborhood drainage systems from becoming overwhelmed during large storm events. The peak runoff in a 100-year storm event will be reduced by approximately 29 cfs along Farrington Drive and by approximately 49 cfs along North Avenue H.

EXHIBITS



0 800 1,600
Feet

Harris County Watershed Boundaries

1% Annual Chance Flood Hazard
Regulatory Floodway
Special Floodway
Area of Undetermined Flood Hazard
0.2% Annual Chance Flood Hazard
Future Conditions 1% Annual Chance Flood Hazard
Area with Reduced Risk Due to Levee

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 1
Effective Floodplain

HT&J, LLC
Professional Engineering Services

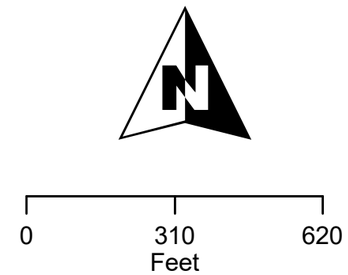
0 410 820
Feet

Study Points
Drainage Area
Ditch
Culvert

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 2
Farrington Drive
Drainage Overview

HT&J, LLC
Professional Engineering Services



Study Point
Ditch
Culvert
Drainage Area

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 3
Northern Ditch
Drainage Overview

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

No Surcharge

Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 4
Existing Drainage System
5-year Inundation

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

No Surcharge

Surcharge

Ponding Depth

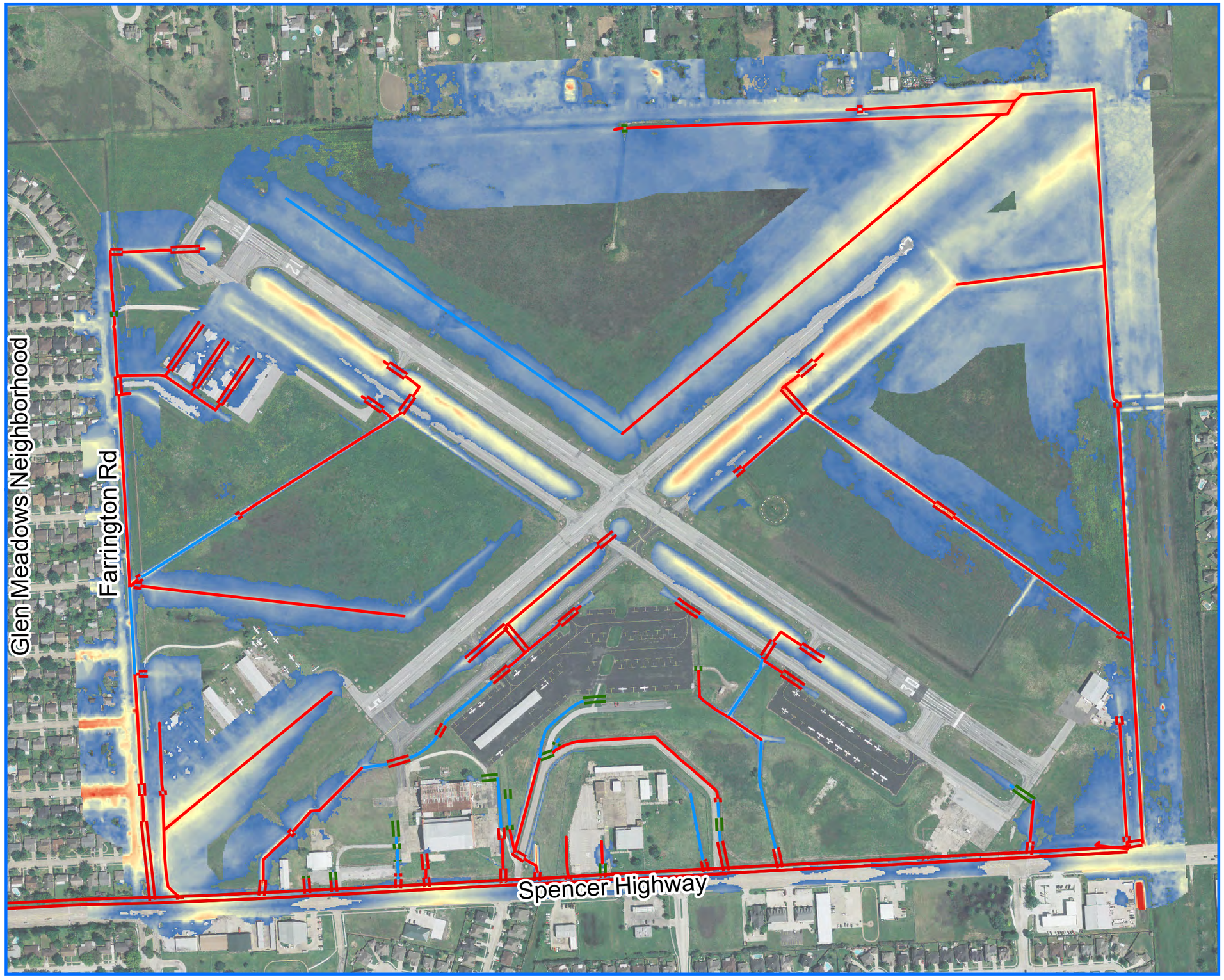
High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 5
Existing Drainage System
10-year Inundation

HT&J, LLC
Professional Engineering Services



0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 6
Existing Drainage System
100-year Inundation

HT&J, LLC
Professional Engineering Services

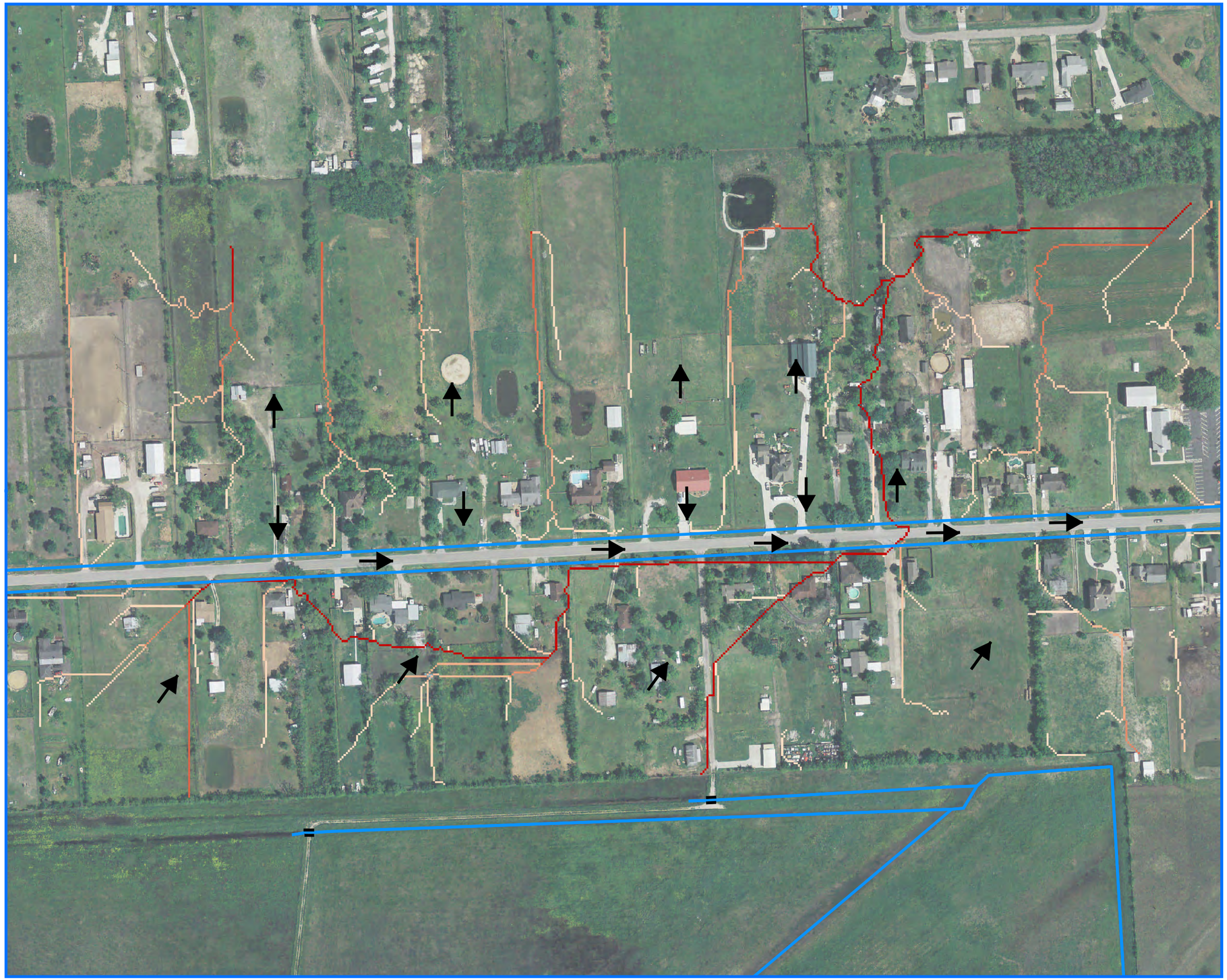
0 400 800
Feet

Contributing Area
140 Acres
0.5 Acres

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 7
Sheet Flow Patterns
West Neighborhood

HT&J, LLC
Professional Engineering Services



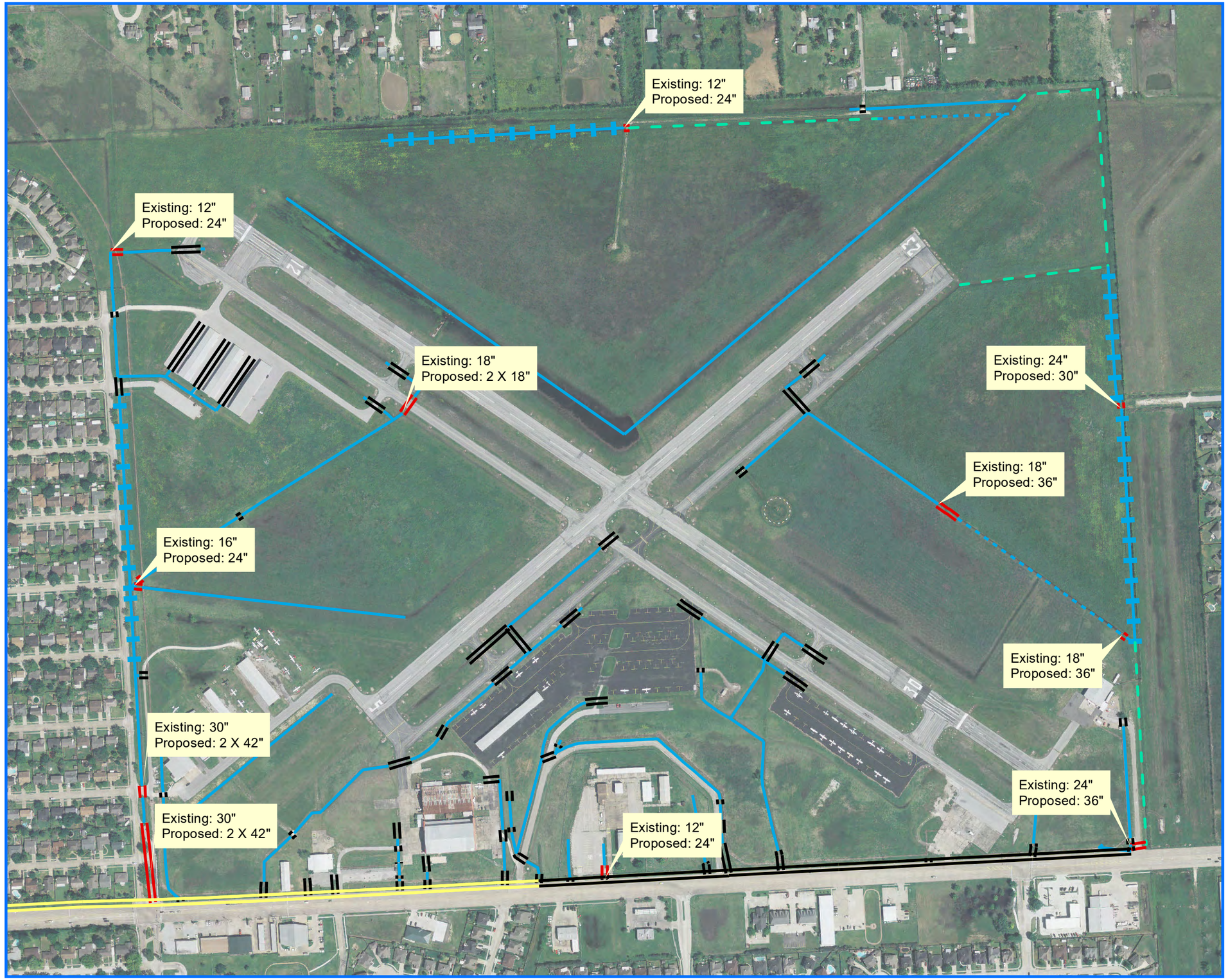
0 230 460
Feet

Contributing Area
80 acres
0.5 acres

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 8
Sheet Flow Patterns
North Neighborhood

HT&J, LLC
Professional Engineering Services



N

0 400 800
Feet

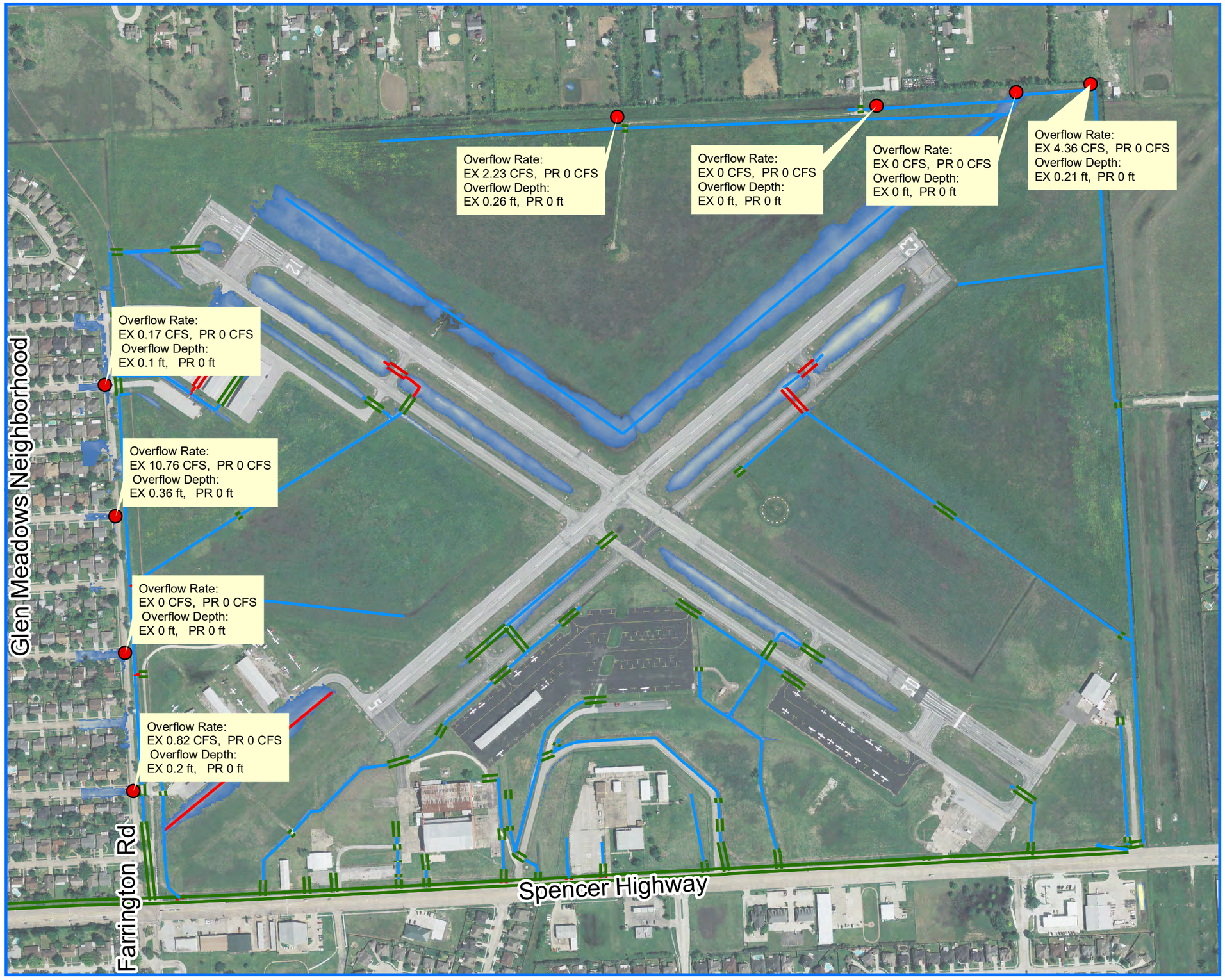
Structure Type

- Circular Pipe
- Box Culvert
- Ditch
- Improved Culvert
- Lowered Ditch
- Linear Detention
- Widened Ditch

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 9
Proposed Drainage Improvements

HT&J, LLC
Professional Engineering Services



0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

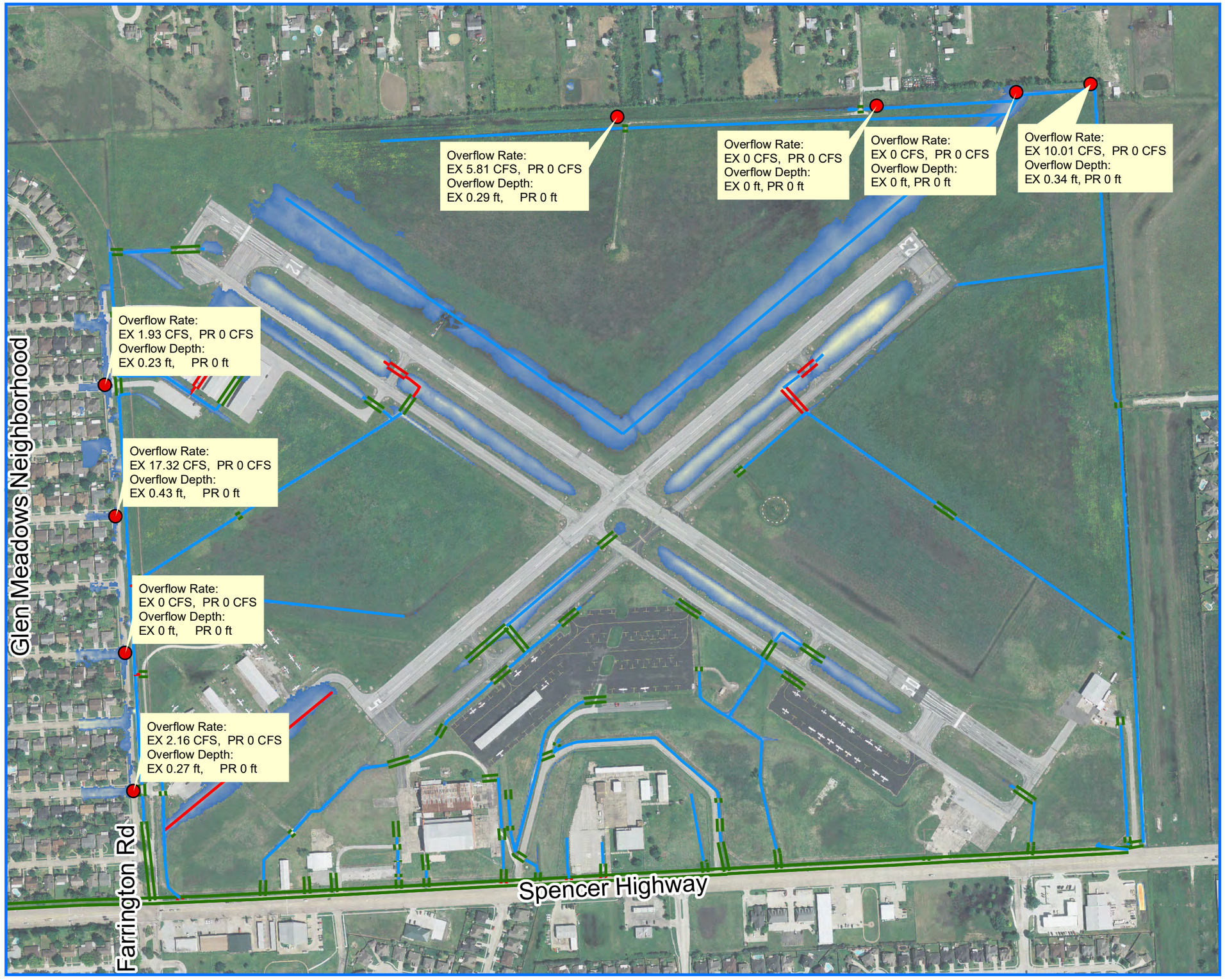
High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 10
Proposed Drainage System
5-year Inundation

HT&J, LLC
Professional Engineering Services



0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

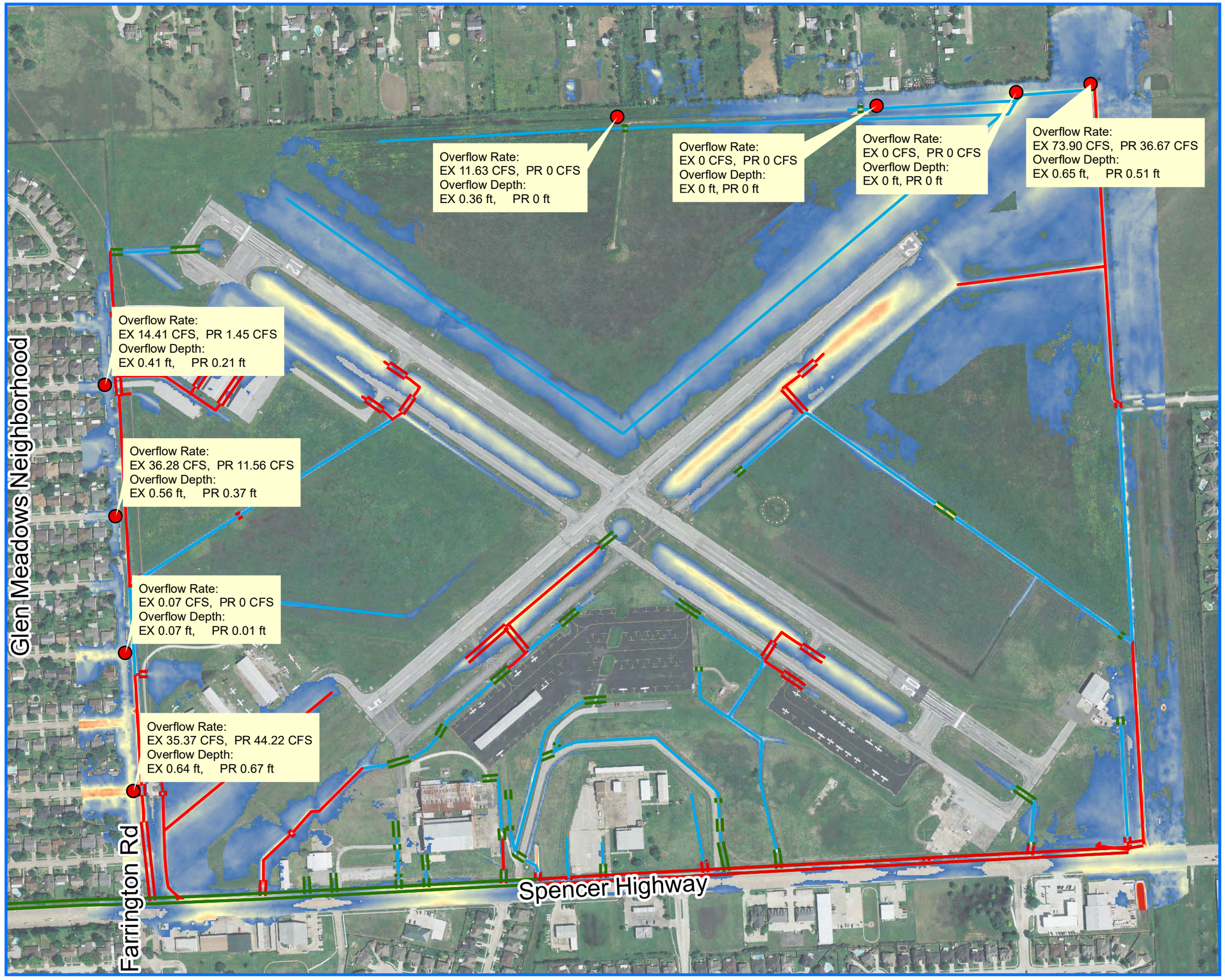
High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 11
Proposed Drainage System
10-year Inundation

HT&J, LLC
Professional Engineering Services



0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 12
Proposed Drainage System
100-year Inundation

HT&J, LLC
Professional Engineering Services

Preliminary Engineering Report

AIRFIELD PAVEMENT REHABILITATION AND DRAINAGE IMPROVEMENTS – 2017

La Porte Municipal Airport

La Porte, Texas

TxDOT Aviation Division Project Number

CSJ 1712LPRTE

THIS DOCUMENT IS ISSUED FOR INTERIM REVIEW ON THE AUTHORITY OF:
THOMAS D. DODSON, PE (TX 83909)
ON DECEMBER 22, 2017
AND NOT TO BE USED FOR BIDDING OR CONSTRUCTION.

LA PORTE MUNICIPAL AIRPORT

PRELIMINARY ENGINEERING REPORT

AIRFIELD PAVEMENT REHABILITATION AND DRAINAGE IMPROVEMENTS – 2017

Texas Firm No. F-18997

December 22, 2017

Executive Summary

This report identifies existing pavement and drainage issues and recommends improvements within the airfield of the La Porte Municipal Airport at La Porte, Texas. TxDOT Aviation (Client) has programmed pavement rehabilitation for all parts of the airfield pavement (including Runways 12/30 and 5/23, as well as drainage improvements on the airport. Specifically, this report identifies pavement rehabilitation of patching, crack sealing, and seal coat surface treatment of the pavement.

The existing airport classification is a “B-II”, but the existing airfield geometry does not support this classification. The pavement rehabilitation does not change the pavement geometry. For this project the designs were carried out for identified improvements meeting FAA criteria for airport design and for drainage. The Taxiway Design Group was determined to be “1A” based on the representative aircraft listed on the ALP and observation of traffic at the airport. For drainage, the design storm period was set to a more stringent 10 year return period at the request of the City of La Porte.

Data collection for drainage study and pavement surface condition assessment was accomplished by the use of aerial drones, supplemented with on-the-ground survey.

With the exception of a portion of Taxiway A between Taxiway B and Taxiway A1, all the pavement within the airfield will benefit from a seal coat surface treatment. Many areas of pavement will need patches and crack seal prior to seal coat. All pavements will be re-marked as well.

For drainage, there were several locations where ponding of stormwater was at levels not permitted by the FAA for pavement inundation. The drainage study identified many locations where existing culverts on the airfield were undersized and were up-sized to provide better drainage on the airfield. To accommodate limitations in the receiving system (most of the airport drains to Spencer Highway), several locations of linear detention were identified. The geometry of the linear detention system achieves volume by creating wide flat-side ditches.

Due to limitation on the programmed funding of just under $2,000,000, some of the identified improvements (consisting of TW A between TW B and TW A1, seal coat topping of the aprons, and much of the linear detention) can be removed from this project and programmed into later project(s).

The recommended project is estimated to total of $2,131,500.00 and includes construction and professional services for the administration of the construction. The programmed professional services appeared underfunded for a project of this size.

The rehabilitation will be constructed following FAA specifications to keep with the grant assurances associated with the FAA funding on the project. There are 3 non-standard aspects of the project that may require a Modification of Standards (MOS) during design.

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B. Watershed Data
C. XPSWMM Model Results
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E. Tabulation of Probable Construction Costs

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Appendix 1 – XPSWMM 5-year Existing Model Output
Appendix 2 – XPSWMM 10-year Existing Model Output
Appendix 3 – XPSWMM 100-year Existing Model Output
Appendix 4 – XPSWMM 5-year Proposed Model Output
Appendix 5 – XPSWMM 10-year Proposed Model Output
Appendix 6 – XPSWMM 100-year Proposed Model Output

1. Purpose and Need

Civil PEs has been selected to perform professional services for the design of pavement and drainage improvements for La Porte Municipal Airport and the City of La Porte via a block grant administered by the Texas Department of Transportation – Aviation Division. The purpose of the project is to make improvements to the airport's assets to meet grant obligations in keeping the airport facilities in good condition for use by the flying public. This project will identify improvements for the pavement consisting of pavement surface treatment(s) to preserve the pavement condition and enhance safety. This project will also analyze and design improvements to the airfield drainage to promote drainage of the airfield and reduce the opportunity for stormwater to pond, creating both a structural issue with saturated pavement foundations and the attraction of wildlife to the airfield. The majority of the site drains to the Spencer Highway drainage system. For this reason, the Spencer Highway system was also evaluated for capacity. The recommendation for on-site improvements are derived to meet the local, state, and federal regulatory requirements, and are discussed in subsequent sections.

1.1. Scope

The programmed scope of the project, as listed by Aviation Division in the most recent Capital Improvement Program as well as scope indicated in the request for proposals, consists of pavement improvements to rehabilitate and mark Apron 1, Apron 2, and Apron 3; rehabilitate and mark Runways 12/30 and 5/23, rehabilitate Taxiways A, B, the northwest hangar access taxiway at Runway 12; and rehabilitate Taxiways C and D. The project also generally includes a program line item for "Drainage Improvements". In that, this report identifies drainage problems on the airfield and develops solutions to correct the problems.

2. Criteria

2.1. General

La Porte Municipal Airport is a reliever class general aviation airport. It forms a part of the Texas Airport System Plan (TASP) and the National Plan of Integrated Airport Systems (NPIAS) as a reliever airport. The airport has 2 runways with full parallel taxiways on each runway. There are 3 Fixed Base Operators (FBOs) on the airport along with self-serve fueling. The airport does not have a public terminal.

The current layout of the airport is presented in **Exhibit 1**.

2.2. ALP Information and Traffic Levels

The current Airport Layout Plan (ALP)[1], approved in January 1989, indicates that the airport is based on Aircraft Approach Category "B" (speeds of about 90 knots to 120 knots)

[1] Coffman and Associates, 1989.

standards. This design standard sets geometric and safety offset requirements for Runway Safety Areas (RSA), Object Free Areas (OFA), and other dimensions for runways and the runway-to-taxiway offset. The ALP also indicates the geometric design group (ADG) is "II" (wingspans up to 79 feet). This aircraft size standard sets requirements for taxiway and apron OFAs, parking configuration, hangar locations, and taxiway-to-taxiway separation.

It is noteworthy that the ALP lists the B-II standard, but further specifies the "critical aircraft" in a table listing for the airport standards at La Porte. The aircraft listed (King Air 100 for Runway 12-30 and Cessna 421 for Runway 5-23) are not B-II, but are each B-I aircraft. For operating maximum takeoff weights (MTOW)[2], the Cessna is listed at 7500 lbs., while the King Air 100 is listed at 12,000 lbs. Conversely, the ALP lists the traffic weight ratings as 23,000 and 25,000 pound aircraft weight limits for Runways 12/30 and 5/23 respectively.

The most recent airport data posted from FAA in Form 5010[3] shows traffic operations have remained steady at a total of 29,050 annual operations. This total is split 1/3 as itinerant with the other 2/3 being local operations. The data does reflect an increase in based aircraft. All aircraft are listed as either single- and multi-engine aircraft, with no jets listed. The airport also claims 1 based helicopter.

2.3. FAA Design Criteria

As noted above from the ALP information, the airport is listed as a B-II airport. This defines such requirements as the runway to taxiway separation of 240 feet.

Runway Design Group

The runways for La Porte are listed as B-II runways, with a standard width of 75 feet, which is applicable on approach limits down to ¾ mile visibility approaches. There is only one non-visual approach developed for La Porte, and that is a GPS approach to Runway 30. Under this "LPV" approach to approach category C, the minimum visibility range is 1 mile. Therefore, the runway width is applicable with room to get a better approach to ¾ mile before the standard width on the runway is too small. Other dimensions applicable to this category include a RSA width of 150 feet and a ROFZ of 500 feet.

Taxiway Design Group

Since the last ALP update, the FAA has added criteria for taxiway design based on aircraft geometry, namely the cockpit-to-main-gear and the main gear width dimensions, for ensuring taxiway edge safety margin (the distance for the outside aircraft wheel to the edge of pavement) is maintained in taxiway intersection fillets.

Drainage Criteria

FAA AC 150/5320-5D - Airport Drainage Design, Unified Facilities Criteria Surface Drainage Design, states the 5-year storm event is to be used with no encroachment

[2] FAA Worksheet: Aircraft Characteristics Database-v1-201609.xlsx
[3] FAA Airport Data Form 5010, for 12 months ending 12/30/2016.

of runoff on runway and taxiway pavements. The center 50-percent of runways and taxiways is to be kept free of stormwater ponding resulting from the 10-year storm event.

FAA AC 150/5200-33B, Hazardous Wildlife Attractants on or Near Airports, requires storm water detention ponds to allow a maximum 48-hour detention period for the design storm and recommends avoidance or removal of retention ponds and detention ponds featuring dead storage to eliminate standing water.

2.4. CRITERIA FOR THIS PROJECT

During the project kick-off meeting for this project, the subject of the airport classification versus the criteria for airfield geometrics was discussed. Civil PEs noted that the standards were not going to be met if the airport design group was maintained and that since the ALP "design aircraft" were both B-1 (small aircraft), that the standard could be changed and would bring the airport geometry to standards. TxDOT Aviation Division staff reviewed the observation and directed Civil PEs to proceed with the project but not change the airport classification. Aviation Division noted that future projects involving the taxiways could then consider the geometry and relocation of the parallel taxiways.

Taxiway Design Group

The Taxiway Design Group (TDG) is also based on the two listed aircraft from the ALP – the Cessna 421 and the King Air 100. The Cessna has a Main Gear Width (MGW) of 14'-8"[4]. The King Air 100 has a MGW of 13'-0"[5]. The Cockpit to Main Gear (CMG) dimension is not listed in the advisory circulars, but a conservative estimate is that the cockpit to main gear reference is half the distance from the nose wheel to main gear. For the Cessna 421, the wheel base is 11'-8"[6], and the King Air 100 has a wheel base of 14'-11"[7]. Therefore, the Cessna has an estimated CMG of 5'-10" and the King Air 100 has an estimated CMG of 7'-6". The results of these determinations are shown in **Figure 1** and indicates the appropriate TDG is 1A. During the several site trips to the airport, the aircraft seen using the airport did not indicate a need to change the TDG based on observed traffic.

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[4] FAA Advisory Circular150/5300-13: *Airport Design*, Appendix 12 – Airplane Data, Figure A12-7, dimension F, September 1989.
[5] Ibid Figure A12-20, dimension F.
[6] Ibid Figure A12-7, dimension D.
[7] Ibid Figure A12-20, dimension D.

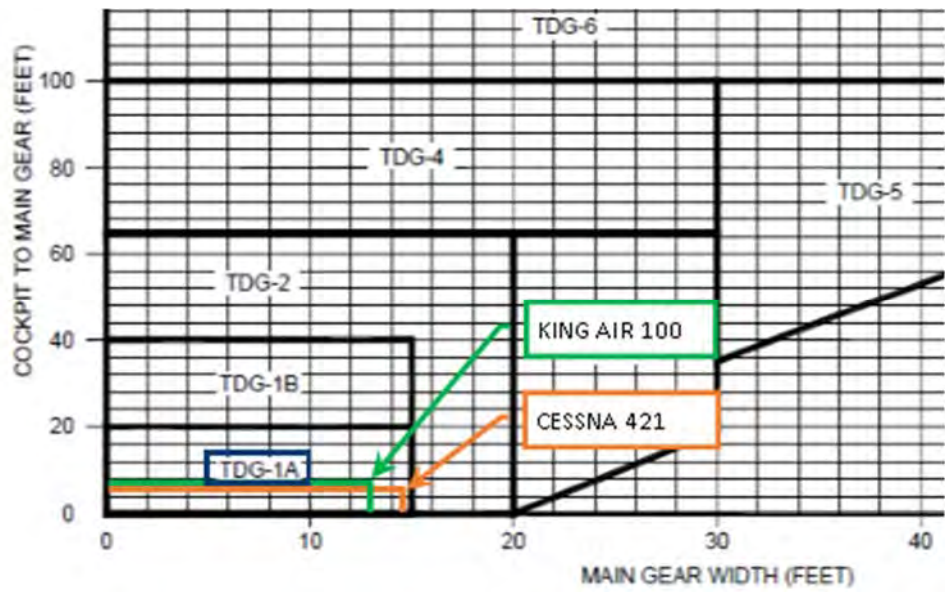

FIGURE 1 - TAXIWAY DESIGN GROUPS

Specific Drainage Design Criteria

A more stringent limit of keeping runoff from the 10-year storm event clear of the runways and taxiways was used for this study for the following two reasons:

1) With the recent rain events in Houston and surrounding areas, National Oceanic and Atmospheric Administration (NOAA) along with Harris County and City of Houston is in the process of updating the rainfall data. These updates are expected to release in the near future and expected to increase by as much as 15%. In order to avoid immediate short coming on the design, the 10-year storm event is suggested.

2) During the kick off meeting, Public Works Director indicated the capacity of the receiving system should be considered for selecting the design event.

All drainage improvements on the site were designed such that all linear detention on site, including natural channels could drain within 48 hours, as required by the FAA. This will also minimize the risk of wildlife attraction to the area.

The FAA AC permits the application of rational methods and hydrograph methods as applicable to determine the peak runoff. This study utilizes the allowable methodologies in conjunction with the state and the local intensity duration frequency (IDF) parameters to develop peak flows and the hydrographs for runoff simulation.

Texas Department of Transportation (TxDOT) Design Criteria

Per the TxDOT Hydraulic Design Manual Chapter 4 Section 12, revised July 2016, the Rational Method was used to calculate peak discharge values for each drainage basin on the site. TxDOT values for Harris County were used for the “b”, “d”, and “e” rainfall intensity coefficients, as shown in Table 1 below.

TABLE 1 - RAINFALL INTENSITY COEFFICIENTS

Rain Event Frequency	b	d	e
2-year	68	7.9	0.800
5-year	70	7.7	0.749
10-year	81	7.7	0.753
25-year	81	7.7	0.724
50-year	91	7.7	0.728
100-year	91	7.9	0.706

The Kerby-Kirpich Method was used in adherence with the TxDOT Hydraulic Design Manual Chapter 4 Section 11 to calculate the Time of Concentration for each basin. The peak flow values were used to determine the storage coefficient for each individual drainage area on site.

City of La Porte Design Criteria

The City of La Porte Public Improvement Criteria Manual (PICM) Chapter 5, Storm Water Design Criteria, was reviewed and adhered to wherever there was no overlap with the TxDOT Hydraulic Design Manual. Where standards or methods were listed in both documents, the TxDOT regulations were used.

2.5. REGULATORY COORDINATION

TxDOT (Highways) Coordination

Internal TXDOT coordination will be necessary for any alteration to infrastructure entering the storm sewer in Spencer Highway, as that is a TxDOT controlled highway.

City of La Porte Coordination

It was discussed at the kick off meeting with the City of La Porte Public Works Director that the design storm event for the drainage evaluation to be considered at the capacity of the receiving system (the storm sewer in Spencer Highway). The storm sewer system along Spencer Highway has a capacity for 10-year storm event. As a result, the 10-year storm was used as the design storm.

Federal Emergency Management Agency (FEMA) Coordination

Any alteration to FEMA effective floodplain will require coordination with FEMA. The proposed improvements will not result in any changes in the existing floodplain, and FEMA coordination is not expected.

United States Department of the Army Section 404 Permit Requirements

We do not anticipate a need to consider the Section 404 Permit, as no dredged or fill material will be discharged into any waters of the United States. The National Wetlands Inventory through the U.S. Fish and Wildlife Service was reviewed as well, and there are no wetlands on the La Porte Municipal Airport site.

Texas Commission on Environmental Quality (TCEQ) Coordination

The coordination with TCEQ may be necessary to determine the need for Municipal Separate Storm Sewer Systems (MS4) permit. This report did not evaluate the MS4 requirements as it is outside the defined scope.

3. Existing Conditions

Drone Data Collection

The use of aerial photogrammetry for data collection of the pavement condition was based on trends in the industry for increasing use of remote data collection, coupled with increased visual capability of photogrammetry due to increasing camera resolutions allowing for tighter (less than ¼ inch pixel resolution) imagery. The use of hyper-spatial resolution (sub-centimeter or sub-inch) natural color aerial photography (HSR-AP) was studied by the University of New Mexico and a document was published in May 2016[8] outlining the results of aerial data imagery and 3D surface modeling captured aerially and comparatively studied on the ground by conventional methods with a comparison of the two methods. A sensitivity analysis was made on the results of the two methods and the data collected for rut depth, alligator cracking, and transverse cracking of flexible pavement in various sites in New Mexico. The results of the sensitivity analysis showed that the differences in measures between the aerially derived measures and the ground measures were not statistically different, even at a 95% confidence level[9]. Because of this accuracy, we proposed and collected data aerially to evaluate the pavement condition and quantified pavement deterioration for rehabilitation on this project.

The aerial data collected for this report was gathered by two unmanned aerial vehicles. The first (for 3D model derivation) is an Ebee fixed wing aircraft. This aircraft gathered topographic information to derive a surface model of the site to use with the drainage investigation. This platform also gathered a low-resolution (comparable with Google Earth) imagery, but of the entire project area. The second aircraft was a DJI Phantom quadcopter by Parrot. This aircraft collected much higher resolution imagery to document pavement distress for condition assessment and quantity calculations of pavement rehabilitation.

Conventional Survey Collection

Some data for the drainage investigation was not able to be obtained by remote means. For this, the conventional on-the-ground survey was used for measuring features like silted-in culvert flowlines and ditch and swale elevations that were below

[8] Characterizing Pavement Surface Distress Conditions with Hyper-Spatial Resolution Natural Color *Aerial Photography* by Zhang, Leppitt, Bogus, Neville, et al (University of New Mexico) republished by *Remote Sens*. 2016, 8, 392.

[9] Ibid, pg 14 of 23.

water surfaces. This survey information is included into the overall surface model data.

Follow up Information Collection

After the provided survey information and pavement imagery assessment was completed, a site investigation of the pavement condition was made on December 15, 2017 to confirm the imagery assessment and clarify issues that were discovered during the aerial data assessment. The mapping of the pavement distresses is shown on **Exhibits 2.1** through **2.9.** Below is a summary by pavement segment of the assessment:

3.1. Pavement Condition

Runway 12/30

The asphalt pavement surfacing on Runway 12/30 is in good shape with heavy vegetation growth into the edges of the pavement and isolated block cracking. A total of approximately 7,700 feet of block cracking and 900 square feet of alligator-type cracking, mostly in the edges, was identified. The runway has 7 connecting taxiways, two of which are oversize and not to standard geometry. Each oversize connector is aligned with the displaced thresholds of the runway. There is no significant previously placed crack sealing on the runway.

Figure 2 - Runway 12 End Looking Southeast

The pipeline corridor crosses the runway between crossing Runway 5/23 and crossing of Taxiway B. There is no evidence of utility patches or deterioration at or near the crossing.

The northwest end of the runway (12) is displaced from the end of pavement by 190 feet, and measures on the ground confirm the published[10] displacement. The threshold markings may have been configured in the pre-1994 arrangement of wider bars and has a non-standard "blacking out" of the old markings.

[10] FAA Form 5010

Similarly, the southeast end of the runway (30) is also displaced from the end of the pavement by 402 feet. Survey also confirms the published displacement. This threshold also has same blacking out of old markings.

The markings on Runway 12/30 are standard runway markings for a non-precision runway, with threshold bars and arrowheads for a standard displaced threshold. However, the previous marking standards allowed for the threshold markings to be dimensioned similarly with or without a threshold bar at either 10 or 20 feet of gap. This is no longer the standard and a 20-foot gap must always be used. The thresholds for Runway 12/30 (both ends) each use a 10-foot gap, and therefore proper separation between the threshold bar and the threshold markings is no longer maintained. The connecting taxiways each have holdlines, generally offset from the runway centerline at 125 feet. The holdlines are co-located with the mandatory runway hold signs. This offset distance does not meet the current standard, but with the non-standard parallel taxiway offset, the holdline would encroach into the parallel taxiway.

Taxiway "A"

Taxiway A has an asphalt surface that is in varying condition. The taxiway has an outboard runup pad beyond the displaced 30 end. The runup pad is separated from the taxiway by a concrete valley. The runup pad is also asphalt and has asphalt (black) staining near the concrete valley at the northwest end.

FIGURE 3 - TAXIWAY A LOOKING SOUTHEAST

The taxiway condition is fair to poor west of Runway 5/23, where the taxiway is losing profile and has areas of sinking and sloughing of the edges. This is creating significant block cracking and displacement of the pavement surface. The pavement is in good condition to the east of Runway 5/23.

West of Runway 5/23, the tabulation of cracking was not made due to the overwhelming amount of cracking observed. To the east of Runway 5/23, a total of approximately 3,500 feet of block cracking and 2,610 square feet of alligator-type cracking was identified.

The pipeline corridor crosses the taxiway east of its crossing with Taxiway B. There is no evidence of utility patches or deterioration at or near the pipeline crossing.

The taxiway has previously had significant crack sealing east of TW B to opposite Apron 1 connector at Taxiway A3.

Taxiway edge lighting is at maximum allowed distance from edge of pavement; so moving the pavement edge in cannot be done without violating design criteria[11].

Taxiway "D"

Taxiway D is a right-angle connector taxiway that was a result of the changed alignment from old angled connector to RW 30 threshold. The taxiway was installed in 2008[12]. The pavement surface is asphalt in good condition and does not appear to have ever been seal coated.

Northwest Hangar Access Taxiway and Apron at Runway 12

On the northwest side of the airfield, a connector taxiway tying to Taxiway A serves three t-hangar buildings. While some of the pavement was installed before 1989, the hangars were constructed between 2002 and 2004.

In the same 2008 project that created Taxiway D, the apron and connector was re-paved and widened to its current configuration.

The pavement is in good condition and does not appear to have ever been seal coated. There is some cracking on the pavement, but it is mostly confined to asphalt joints. The aprons between the hangars are drained by a fin-type slotted drain that is flanked by concrete pavement. One end of the drain in the middle section is showing significant deterioration and is creating FOD. A total of approximately 3,100 feet of block cracking and 25 square feet of deteriorated was identified for this apron.

The aprons between the T-hangars have centerline striping that ends at the fin drains. However, the clearances do not meet the standards for a taxilane.

Runway 5/23

The asphalt pavement surfacing on Runway 5/23 is also in good condition with limited block cracking and heavy vegetation growth into the edges of the pavement. The runway appears to have last been seal coated sometime before 2002. A total of approximately 3,000 feet of block cracking and 1060 square feet of alligator-type cracking was identified within the runway environment.

The runway has five connecting taxiways, with the two end connectors also being oversize and not standard geometry. There is no significant previously placed crack sealing on the runway.

The pipelines cross the runway at an acute angle north of 12/30 intersection and there is no evidence of patches or deterioration at pipeline crossings.

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[11] AC 150 / 5340-30H, Design for Airport Visual Aids, 7/14/2004, pg. 7.
[12] Record Drawings, La Porte Airport Improvements, C.T. Brannon Corp., July, 2008

The existing striping for the runway is standard for a visual runway classification[13] – which is appropriate. As a visual runway, there is no threshold bar, threshold markings, nor aiming point. As with the primary runway, the connecting taxiways each have holdlines, generally offset from the runway centerline at 125 feet. The holdlines are co-located with the mandatory runway hold signs. This offset distance does not meet the current standard either.

FIGURE 4 - RUNWAY 5 MARKINGS

Taxiway "B"

Taxiway B is an asphalt pavement surface taxiway with significant crack seals along the full length of the taxiway and its connectors to the runway. Overall, the pavement is in good condition. Most of the crack seals are in the edge portion of the taxiway. There are numerous additional cracks within the previously sealed crack areas. A total of approximately 1,900 feet of block cracking and 6,100 square feet of alligator-type cracking was also identified, mostly in the edges of the taxiway.

The same project that constructed Taxiway D and the replacement of the northwest hangar pavement also installed a rejuvenator / seal coat (EB-44A) in 2008. The portion of TW B between TW A and Runway 12/30 received seal coats in both the 2002 and 2008 projects and is noticeably darker compared to the other taxiways.

The pipelines cross the taxiway at an acute angle between TW A and Runway 12/30. There is no evidence of patches or deterioration at the crossings.

Taxiway "C"

Taxiway C is an asphalt surface pavement that was constructed in 2008. This pavement is in good condition, with existing crack sealing along the pavement. Unlike TW B, the sealed cracks are all across the pavement rather than focused in the edges.

[13] FAA AC 150/5340 – 1L, Standards for Airport Markings, Table 2-1, 9/27/2013.

Apron 1

Apron 1 is an open tie-down parking apron composed of an asphalt surface. The apron contains 24 tie-downs.

Prior to being an apron, Runway 9/27's alignment traversed the apron, but at an angle. The runway was removed sometime before 1978 and it is unclear if any of the runway's foundation was used for the apron.

The apron was seal coated in 2008 with a thermoplastic coal-tar sealer (then-EB35), which was known as "Grip-Flex". The grip-flex surface is still in place and the aggregate within the seal coat is still providing a friction surface. However, the pavement has significant block cracking that is reflected through the grip-flex. The apron has locations of depressions that are holding water (birdbaths) that is likely accelerating the cracking of the surface as the water penetrates the cracks and weakens the underlying pavement support.

The large amount of cracking on the apron prevented the individual crack counting and instead was sampled. A 50-foot square sample box was determined along the centerline of the apron and was used to represent the average condition of the entire apron. Within this sample box, 205 feet of block cracking was counted. Extrapolating this rate to the entire apron, results in 9,200 feet of block cracking on the apron.

Apron 2

Similarly, Apron 2 is also an open tie-down parking apron that is flanked on the west side by the pipeline corridor, with three connectors crossing the corridor to Apron 3. The apron has 34 tie-downs.

FIGURE 5 - APRON 2 BLOCK CRACKING

The apron is an asphalt pavement surface that also was seal coated in 2008 with grip-flex. The surfacing is also still in place and, like apron 1, the pavement has significant block cracking that is reflected through the grip-flex. The apron has locations of birdbaths and some areas of grass vegetation growing in the cracks.

Like Apron 1, the large amount of cracking on the apron prevented the individual crack counting and instead was sampled. Again a 50-foot square sample box was determined within the apron that was representative of the average condition of the

entire apron. Within this sample box, 762 feet of block cracking was counted. Extrapolating this rate to the entire apron results in about 34,900 feet of block cracking on the apron.

Apron 3

Apron 3 is across the pipeline corridor from Apron 2. It also is an asphalt pavement that was seal coated with grip-flex in 2008. The surface still has aggregate and provides good friction. The apron has 38 tie-downs and 1 T-hangar building within the apron.

Like Aprons 1 and 2, the large amount of cracking on the apron prevented the individual crack counting and instead was sampled. However, this time three different 50-foot square sample boxes was determined across the apron because there is some variability in the condition of the entire apron. The multiple samples provides a way to average the overall quantity of cracking for better estimating. Within the three sample boxes, 1270 feet of block cracking was counted. Extrapolating this rate to the entire apron results in 29,300 feet of block cracking on the apron.

3.2. Drainage Condition

Assumptions and Constraints

The FEMA effective models were used for evaluation of tailwater in the Big Island Slough channel. The HEC-RAS model for the channel (B106-00-00) and the HEC-HMS model for the Armand Bayou watershed were obtained from Harris County Flood Control District (HCFCD) and appropriate information was used.

When modeling natural channels, cross sections were taken at the mid-point of each channel and the channel was assumed to be uniform using that cross section. The maximum length for this purpose is limited to 200 LF.

Texas Department of Transportation (TXDOT) intensity duration parameters were used to calculate the rainfall intensity to use in rational method for runoff calculation from each sub-basins. The XPSWMM model is calibrated at the sub-basin level against the rational method to generate the similar peak flow.

Manning's Roughness Coefficient (n) values were assumed, and are displayed below in Table 2.

Table 2 - Manning's n Values

Material Type	Manning's n
Reinforced Concrete Pipe (RCP) Culvert	0.014
Concrete Pilot Channel	0.025
Natural Channel	0.035
Natural Channel Overbanks	0.040

The design event was selected based on the factors discussed in Section 2.3, and is set to the 10-year storm event. This exceeds the FAA requirements.

Floodplains

The La Porte Municipal Airport is split between FEMA FIRM panels 48201C0940M and 48201C0945M, both effective 1/6/2017. Only the southwest corner of the airport site is located within the 500-year FEMA floodplain, as shown in **Exhibit 3**. The rest of the site is located in the "Area of Minimal Flood Hazard"; outside the 100-year regulatory floodplain.

Exhibit 3 presents the FEMA Floodplain at the airport site. The 100-year Base Flood Elevation (BFE) ranges from elevation 19.70 near Spencer Highway to elevation 21.0 at the north property boundary line.

Topography and Drainage

Topography of the site was developed using the detailed survey provided by Gessner Engineering. Digital Terrain Model (DTM) was developed from the detailed survey and drainage areas were determined for each culvert and drainage structure using GIS tools. Overall, the site generally slopes north to south, with isolated high and low points. The ground elevations on site range from approximately 24 feet to 20 feet.

For the modeling, the total site is divided into 120 sub basins for detailed analysis.

Land Use

The majority of the 300-acre airport site is open area covered in native grasses. The impervious area on the site is made up of runways, taxiways, parking lots, storage lots, and some commercial area. Approximately 20 percent of the site is impervious.

Existing Drainage Structures

The drainage infrastructure is made up of Reinforced Concrete Pipe (RCP) culverts and natural channels in a connected network that outfalls into an underground storm sewer in Spencer Highway. The existing on-site RCP culverts range in diameter from 15-inch to 36-inch.

The entire site primarily drains into a roadside ditch along Farrington Drive to the west, a shallow swale along the north and east property line that are ultimately connected to storm sewer system at Spencer Highway. The Roadside ditch, on average, is about 2.5 feet deep and top width varies to maximum 30 feet. Two culvert structures 118 and 166 (See Exhibit 3.3) force this roadside ditch to act as a linear detention pond.

In a regional watershed divide, the area north east of Runway 12/30 is delineated to the San Jacinto and Galveston Bay watershed and is supposed to flow northwards. However, the previously installed internal drainage of interconnected swales along the north and the east property line allows runoff to flow southwards instead.

See **Exhibit 3** for the Harris County watershed boundaries and **Exhibit 4.0** for the drainage infrastructure overview.

Exhibits 4.1 through **4.4** present a detailed look at the drainage infrastructures throughout the site at the existing conditions.

Exhibit 5 presents the existing on-site drainage ditches.

Existing Outfalls

There are total of 15 drainage structures from the La Porte Municipal Airport site along Spencer Highway. Based on the survey data, we assume these drainage structures are connected the storm sewer system at Spencer Highway. This storm sewer line ranges in size from 48-inch RCP to 10 x 6 feet concrete box culvert, and ultimately outfalls into the Big Island Slough to the west of the airport property.

Existing Pipelines and Utilities

Based on the available public record from Texas Railroad Commission, there are three pipelines that exist on the La Porte Municipal Airport site. Two lines are designated as 'in service', a propane pipeline owned by Exxon Mobile and a crude oil pipeline owned by Enterprise Crude Pipeline LLC. The third line is designated as 'abandoned' crude oil pipeline owned by Exxon Mobile. There are also several petroleum products pipelines located just to the east of the airport site. Figure 6 presents the location of the pipelines on and near the airport site. The pipelines running at the middle of the airport property are identified as located within the pipeline corridor as shown on the most recent Airport Layout Plan (ALP).

FIGURE 6 - PIPELINES AT THE AIRPORT

3.3. EXISTING DRAINAGE SYSTEM ANALYSIS

Hydrologic Analysis

Hydrologic calculations were performed in accordance with the TxDOT Hydraulic Design Manual. The Rational Method was used to calculate peak flows using the TxDOT rainfall intensity coefficients in Table 1. Contributing drainage area boundaries were established based on the topographic survey and drainage culverts detail data provided by Gessner Engineering. Rainfall totals were obtained from the Harris County Flood Control District H&H Manual for the purpose of developing hydrograph and model simulation, and can be found in **Table 3**.

TABLE 3 - HARRIS COUNTY HYDROLOGIC REGION 3 RAINFALL (INCHES)

Duration	Storm Frequency							
	2-yr	5- yr	10- yr	25- yr	50- yr	100- yr	250- yr	500- yr
5 Minutes	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
15	1.1	1.4	1.5	1.7	1.9	2.1	2.3	2.5

Duration	Storm Frequency							
	2-yr	5- yr	10- yr	25- yr	50- yr	100- yr	250- yr	500- yr
30	1.5	1.9	2.1	2.4	2.7	3.0	3.4	3.7
60	2.0	2.5	2.9	3.4	3.8	4.3	5.0	5.5
2 Hours	2.4	3.1	3.7	4.4	5.0	5.7	6.8	7.7
3 Hours	2.7	3.5	4.2	5.1	5.9	6.8	8.2	9.4
6 Hours	3.2	4.4	5.3	6.6	7.7	9.1	11.2	13.1
12 Hours	3.8	5.3	6.4	8.0	9.5	11.1	13.6	15.9
24 Hours	4.5	6.4	7.8	9.8	11.6	13.5	16.6	19.3
2 Days	5.3	7.5	9.0	11.2	13.1	15.1	18.1	20.7
4 Days	6.2	8.7	10.5	12.9	14.8	16.9	19.8	22.3

The Rational Method Runoff Coefficient for each drainage area was calculated with land cover imperviousness data downloaded from the National Land Cover Database and verified with the latest aerial photographs. The average percent imperviousness was converted to a runoff coefficient (C) value decimal with 100 percent impervious equal to a 0.9 C value and zero percent impervious equal to 0.2 C-value.

In order to fully integrate the system and define the overall drainage impacts, an XPSWMM model was set up with links and nodes representing the drainage areas, and the drainage structures.

The Green-Ampt Method was used to calculate infiltration losses, and Harris County Region 3 loss parameters from HCFCD design manual were used. **Table 4** presents the parameters.

TABLE 4 - HARRIS COUNTY HYDROLOGIC REGION 3 LOSS PARAMETERS

Parameter	Value
Initial Loss (in)	0.100
Moisture Deficit	0.385
Suction (in)	12.45
Conductivity (in/hr)	0.024

For each individual drainage area, the Clark Method was used to develop a hydrograph. Time of Concentration (TC) was calculated using the Kerby-Kirpich Method, per the TxDOT Hydraulic Design Manual Chapter 4 Section 11.

$$T_{ov} = overland\ flow\ time\ (min) = K(L\ x\ N)^{0.467} S^{-0.235}$$

$$K = 0.828$$

$$L = overland\ flow\ length\ (feet)$$

$$N = dimensionless\ retardance\ coefficient = 0.15$$

$S = dimensionless\ slope\ of\ terrain$

$T_{ch} = channel\ flow\ time\ (min) = KL^{0.770}S^{-0.385}$

K=0.0078

L=channel flow length (feet)

S=dimensionless main channel slope

The Reservoir Storage Coefficient (R) is necessary to develop the runoff hydrograph from the sub-basins in a Clark Hydrograph method. The R value is adjusted to calibrate the XPSWMM model against the Rational Method, and the peak flow at the hydrograph is matched to the peak flow estimated from rational method.

Supplemental Table A presents the watershed data for each drainage area.

Hydraulic Analysis

An XPSWMM model was built of the existing drainage conditions on the La Porte Municipal Airport site. All culverts and natural channels were modeled, with all flow exiting the system via the storm sewer located in Spencer Highway. Figure 3 displays the layout of the XPSWMM model. As-built drawings dated 03/01/1994 were used to model the Spencer Highway storm sewer line, which outfalls into the Big Island Slough channel to the west of the site.

FIGURE 7 - XPSWMM MODEL LAYOUT

For the 5-year and 10-year storm events, the tailwater was modeled with the water surface at the top of the outfall pipe (10 x 6 feet box outfall) into the Big Island Slough. For storm events larger than the 10-year event, the tailwater conditions were modeled using a stage time series. The time series was constructed from the HEC-HMS hydrograph and the HEC-RAS flow vs. stage relationships in the Big Island Slough channel (B106 00 00). Flow vs. stage data for different storm events was obtained from FEMA effective HEC-HMS and HEC-RAS models.

The neighborhoods to the west of the airport property (both north and south of Spencer Highway) have their own storm sewer systems that outfall to the Big Island Slough separate from the Spencer Highway system.

Ditches that overbanked or culverts that were surcharged were identified and on-site ponding was delineated. **Exhibits 5, 6,** and **7** show the inundation conditions in the existing system for the 5-year, 10-year, and 100-year storm events. Supplemental **Table C** presents the maximum water surface elevation for each culvert and natural channel for the 5-year, 10-year, and 100-year storm events.

Existing System Analysis Results

The existing drainage conditions do not currently comply with the FAA criterion of no encroachment of runways/taxiways during the 5-year storm event and no ponding on the center 50% of runways/taxiways during the 10-year storm event (See **Exhibits 5** and **6**).

The existing conditions 5-year XPSWMM model results indicate taxiway encroachment at the north ends of Taxiways A and B.

The existing 10-year event XPSWMM model results indicate that ponding would occur on the taxiways adjacent to the northern portions of Runways 5/23 and 12/30. The 10-year model also indicates that there would be ponding on the service road that connects the southwest corner of the airport site to Runway 5/23.

The existing 100-year event XPSWMM model results indicate that there would be significant ponding on the northern portions of Runways 5/23 and 12/30, as well as on the adjacent taxiways. There is slight encroachment on the southern portions of the taxiway along Runway 12/30 as well. The 100-year model also indicates that there would be significant ponding on the service road that connects the southwest corner of the airport site to Runway 5/23. The storm sewer system in Spencer Highway is also overwhelmed during the 100-year event, leading to ponding in the road. But the ponding on the street is as expected for the extreme events.

During 10 and 100-year storm events the culverts and ditches along the western edge of the airport site overtop and spill onto Farrington Drive and the nearby neighborhood to the west. There is ponding in the northeast corner of the airport that likely flows into the bordering neighborhood system along North Avenue H.

Exhibit 6 presents the ponding during the 5-year storm event for the existing conditions.

Exhibit 7 presents the ponding during the 10-year storm event for the existing conditions.

Exhibit 8 presents the ponding during the 100-year storm event for the existing conditions.

4. Rehabilitation Design

4.1. Pavement Rehabilitation

The tabulation of block cracking from the pavement surface analysis was used to quantify the amount of crack seal and repairs needed for the rehabilitation of the pavement. Crack seal material is commonly available, and is specified by the FAA under specification item P-605. For those areas that need repairs due to alligator-type cracking, the anticipated size of the repair equipment determined the width of the repair. It is expected that a six-foot paving lane width is the minimum to achieve a well-compacted, quality finish. This means that a six-foot repair width was set even though the area of alligator cracking may have been much less. Additionally, the length of the repair was extended 10 feet beyond the observed limit of the cracking. This was to account for additional soft pavement at the limits of the cracking. Our experience has shown that providing additional length beyond the damage allows for proper tying of the repair to existing pavement. Therefore, the quantities of repairs for alligator cracking are far larger than the area counted and discussed in the existing conditions sections for the pavement.

Seal coat surface treatment is to follow the FAA specification item P-631: Refined Coal Tar Emulsion with Additives, Slurry Seal Surface Treatment. Knowing that Harris County is in the EPA nonattainment district "Moderate" classification for 8-hour ground level ozone concerns, we have investigated any issue that may interfere with the use of a coal-tar emulsion in the rehabilitation. Research into Texas Commission on Environmental Quality rules under 30TAC115 does not indicate there is any limitation on the use of a coal-tar emulsion product in Harris County. There are neither local nor statewide limitations on the use of this material. Therefore, the statewide success of this material as a seal coat warrants the use of the product in this project.

For asphalt patching of the pavement, the anticipated material specification for the asphalt is P-403, using the TxDOT state highways specifications for asphalt, with a "bumped" performance grade to accommodate for aircraft tire pressures. The current standard for FAA asphalts limits the P-403 asphalt to use on pavements serving aircraft weights of 12,500 lbs. or less. The pavement is currently rated for much heavier aircraft.

The layout depicting the limits of the pavement rehabilitation are included in **Exhibit 9**.

Runway 12/30

Given that the pavement within the limits of Runway 12/30 and the connecting taxiways to the holdlines are good, the pavement is determined to need some patch repairs over the alligator-type crack areas and crack sealing of those block cracks identified. After proper curing of the patches, the entire length and width of the runway is to receive a seal coat in accordance with the specifications for item P-631.

For the striping, the existing pavement markings on the runway must be removed – especially in those areas where old markings have been blacked out. The rehabilitation will include requirements to remove the paint markings prior to the pavement seal coating. This will also allow for best adhesion of the seal coat to the pavement. After the installation of the seal coat and curing time, the markings will be replaced. Recent updates to the pavement marking standards[14] will have some minor changes to bring the markings to the current standard.

One of the tenants on the airport asked if changing the striping of the displaced thresholds could allow for a longer takeoff declared distance. The existing markings on the displacements are not configured as a taxiway aligned with the runway (which would affect declared distances), so it is not clear there are ways to report declared distances that would be different than the runway length. Further investigation is beyond the scope of this report. While it does not appear that declared distances exist for either runway, it may be worth investigating the possibility of this use.

The rehabilitation of the pavement should also include re-dressing the pavement edges. This will be accomplished by the stripping of the turf and topsoil at the

[14] FAA AC 150/5340-1L, Standards for Airport Markings, 4/14/2014.

pavement edge and re-grading the innermost 10 feet off the runway edge to the alignment of the runway edge lights. The intent is to re-establish the standard[15] of a 1-1/2 inch drop-off from the pavement edge. The re-graded strip is sodded to a width of 3 feet off the pavement with the remaining graded area re-seeded.

Taxiway A

Taxiway A rehabilitation is split into two components. The west end of the taxiway, but not including the portion from the end connection to connector Taxiway A1, is in need of reconstruction. This can be accomplished (if the intent is not to move the taxiway to the current standard offset) by recycling the existing pavement in-place as a foundation for a new asphalt pavement. After proper curing of the asphalt, the surface would be seal coated. This is done to preserve the pavement surface from the beginning, but also to maintain a color consistency. The surrounding seal-coated pavements will maintain a dark color long after any new asphalt pavement has oxidized and turned light gray, so seal coating the new pavement helps maintain a consistent color overall for the airfield pavement.

To the east of Runway 5/23 (and the far west portion of the taxiway), the pavement is in good condition and set to receive patches, crack seal, and a seal coat as described above in the Runway 12/30 rehabilitation. After curing of the seal coat, the pavement will receive a new taxiway centerline striping. It was recognized that all across the airport, the radii of the taxiway centerlines in the intersections varied. The intent of the rehabilitation is to consistently stripe taxiway radii. Therefore, the intent on taxiway centerline radii is to use a 60 foot radius for runway lead-offs and entrances, and a 25 foot radius for all other intersections and curves. This radius meets the current standard[16] for a TDG 1A classification. For the taxiway portion that is not reconstructed, edge grading to remove the vegetative buildup at the pavement limit is also included into the rehabilitation, along with the sodding and seeding of the graded area as described in the Runway 12/30 rehabilitation.

Taxiway D

The pavement within Taxiway D is generally in good condition. As noted in the other pavements for rehabilitation, Taxiway D is selected to receive a seal coat (there were no significant block cracks to seal) and edge grading. The limits of the rehabilitation are at the tenant apron edge.

All along the edges of the taxiway, edge grading to remove the vegetative buildup at the pavement limit is also included into the rehabilitation, along with the sodding and seeding of the graded area as described in the Runway 12/30 rehabilitation.

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[15] FAA AC 150/5300-13A, Airport Design – Change 1, Figure 4-33, 2/26/2014.
[16] ibid, Table 4-3.

Northwest Hangar Access Taxiway at RW 12

For the pavement and hangar apron in this area, the rehabilitation consists of crack sealing mostly old asphalt paving joints and seal coating the pavement in a P-631 surface treatment. There does not appear to be justification to use a fuel-resistant thermoplastic seal coat. The damaged pavement adjacent to the fin drain is to be repaired before the seal coat and crack seal to provide the best overall finish, and the seal coat is NOT to cover the concrete of the fin drains. The striping will consist of taxiway centerline. Providing taxiway centerlines between the hangars cannot be made due to the lack of clearance between the T-hangars.

The outer edges of the apron and along the taxiways is to have edge grading to remove the vegetation build-up. The rehabilitation also includes sodding and seeding the disturbed area.

Runway 5/23

As with the primary runway, the pavement surface within the limits of Runway 5/23 and the connecting taxiways to the holdlines is good. Even though the runway is shorter, there is slightly more total alligator-type cracking. As with the primary runway, the pavement is determined to need some patch repairs over the alligator-type crack areas and crack sealing of those block cracks identified. After proper curing of the patches, the entire length and width of the runway is to receive a seal coat in accordance with the specifications for item P-631.

For the striping, the existing pavement markings on the runway must be removed – to allow for best adhesion of the seal coat to the pavement. After the installation of the seal coat and curing time, the markings will be replaced.

As in the other pavement rehabilitations, both edges are to be graded to remove the vegetative build-up and promote flow of runoff from the edge of the runway pavement. The first 3 feet off the edge of the pavement will be sodded, with the remaining disturbed area being reseeded.

Taxiway B

Taxiway B is in good shape, but has many areas of alligator-type cracking that will be patched. The block cracks within the taxiway will be sealed and after proper curing of the patches, the entire taxiway is to receive a seal coat in accordance with the specifications for item P-631.

Taxiway C

The pavement within Taxiway C is generally in good condition and is selected to receive a seal coat (the vast majority of block cracks have been previously sealed) and edge grading. The limits of the rehabilitation are along the alignment of the taxiway.

All along the edges of the taxiway, edge grading to remove the vegetative buildup at the pavement limit is also included into the rehabilitation, along with the sodding and seeding of the graded area as described in the Runway 12/30 rehabilitation.

Aprons 1, 2, and 3

Since the method of rehabilitation on each of the aprons is the same, they are grouped here. As part of the investigation of designs for the apron rehabilitation, Civil PEs reached out to Ameriseal of Ohio to discuss other projects that are similar to LaPorte. Out of that discussion, the following was developed to make improvements to the aprons. The block cracks can be sealed using the same specification as the other crack seals on the airport. The depressed areas that hold water will be filled. This will be achieved by spreading a thickened version of the seal coat and screeding off the surface to fill the low spot. In addition, some of the cracks have become very wide. In instances where the crack opening exceeds the maximum gap of the P-605 specification, the crack sealing will use the thermoplastic screed material but with a higher aggregate content to thicken the material so it can be spread into the cracks. Once the low areas are filled and the cracks are sealed, the entire apron is to be seal coated with a sprayed, thin layer of the thermoplastic seal coat – without aggregate – to top the surface. Since the surface already has a good friction component, the addition of a thin surface top should not dramatically lower the friction component of the pavement.

4.2. Drainage Rehabilitation

Drainage Ditches and Swales

The multiple drainage improvements are proposed to keep the runways and taxiways free from flooding in a 10-year storm event. The proposed improvements include enlarging the existing culverts, improving open drainage ditches, and adding linear detention. The linear detention system will drain within 48 hours or less.

The significant proposed improvements include:

1) The expansion of the ditch system draining the northeastern quadrant of the airport. The flowline of the main drainage ditch running along the border of the airport is proposed to drop by 2.5 ft. This allows the runoff to stay off the pavements and lower the hydraulic grade line. This will also function as linear detention and the water level will rise and fall in the same manner as it does in the existing conditions.

2) Replacement and/or enhancement of 12 culvert structures.

3) The expansion of roadside ditch along Farrington Road.

The linear detention is proposed at three locations:

1) Ditch Segment 16 (See **Exhibit 5**) - For approximately 1000 LF, the existing swale will be expanded to 24 ft. bottom width channel, and the average depth is about 2.0 ft. The existing drainage structure 24 will be replaced by a 24" pipe. This pipe functions as a restrictor for this linear detention basin. This detention provides approximately 2.66 ac-ft. of storage.

2) Ditch Segments 7 and 21 - The right bank of the ditch (towards the airport side) will be sloped at 1% from the channel bottom to the existing ground elevation. The Ditch segments 7, 16, and 21 are proposed to function as linear detention. The

proposed channel section is about 300 ft. wide at the top (See Figure 8). The modifications to Ditch Segments 7 and 21 provide approximately 4.80 ac-ft. of additional storage.

3) Ditch Segment 122 and 129 (Roadside ditch along Farrington Road) – These ditch segments were proposed to expand to function as a linear detention (See Figure 10). The expansion will provide approximately 3.6 ac-ft. of storage. The culvert structures 118 and 166 are proposed to replace with 2-42" pipes.

The overall drainage improvement work will result in about 15 ac-ft. of dirt excavation for both the linear detention and conveyance improvement, of which approximately 11 ac-ft. will be utilized for detention purposes. The excess improvement will result in conveyance improvements to the system.

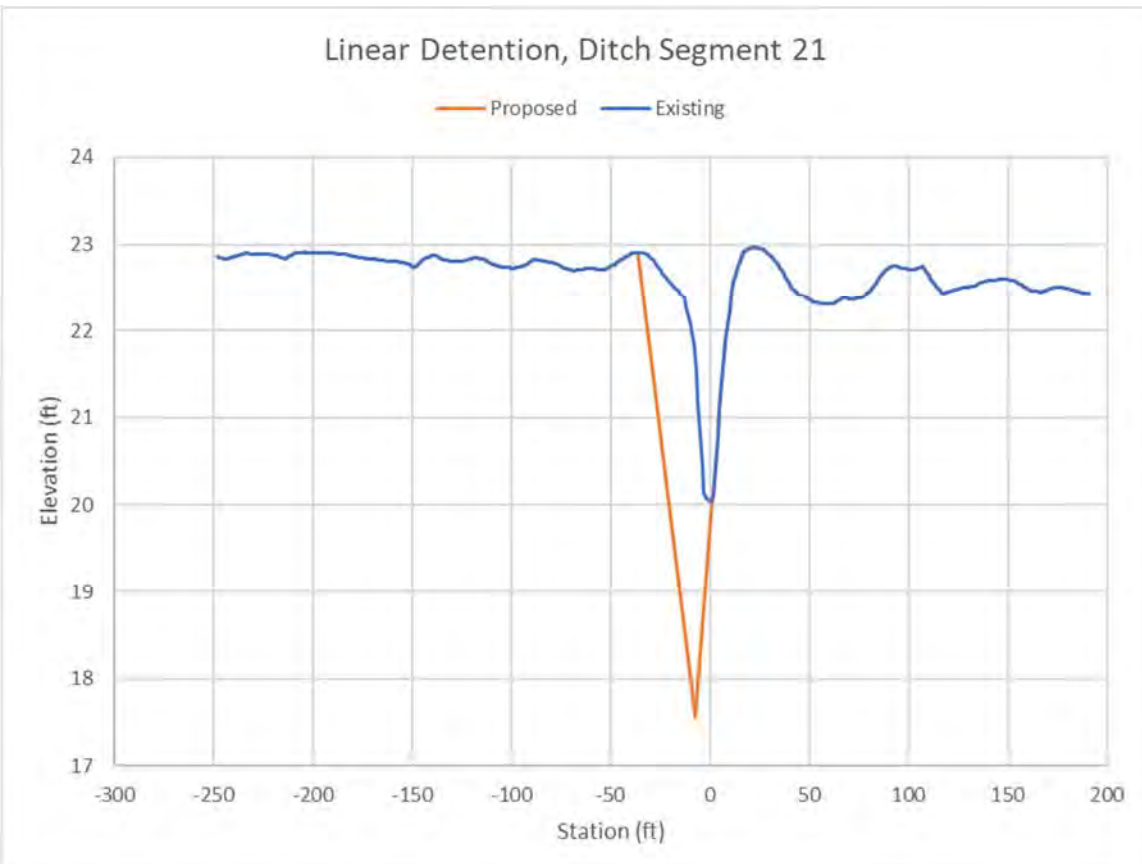


FIGURE 8 - LINEAR DETENTION GEOMETRY

Other modifications to the ditch system include:

1) The ditch segments 17, 19, 20, 26 and 148 were lowered with minimal changes to bottom width or side slopes.

2) The ditch segments 15, 18, 38, and 177, were widened to provide additional conveyance. These segments were widened by approximately 10 ft. (See Figure 9). The flowline of these segments is not altered.

FIGURE 9 – WIDENED DITCH GEOMETRY

FIGURE 10 - WIDENED DITCH GEOMETRY ALONG FARRINGTON ROAD

Culverts

Culverts resulting in major head loss during the 10-year storm event were deemed to have insufficient capacity. These structures are upsized to lower the water level upstream of the structure. To ensure cost effectiveness of drainage improvements, improvements to structures crossing runways, taxiways or other pavements were avoided wherever possible. The exception was structure 110, crossing Taxiway A adjacent to T-Hangars, (See Exhibit 3.1) which is currently an 18" crossing. However, no other utility work interfering with the pavement is slated for this section of the taxiway. It is recommended a parallel 18" diameter pipe be installed at this location. However, this can be deferred to combine with the future taxiway rehabilitation work as the 5-year storm event barely encroaches the Taxiway A in the existing conditions.

The proposed ditch and culvert improvements are shown in **Exhibit 10**.

The following structures are proposed to be replaced with bigger pipes.

TABLE 5 - PROPOSED CHANGES TO DRAINAGE STRUCTURES

Culvert	Existing Size	Full Flow Capacity (cfs)	Proposed Size	Full Flow Capacity (cfs)
166	30"	14.8	2 X 42"	36
118	30"	14.8	2 X 42"	36
106	12"	2.4	24"	9.6
25	24"	9.6	30"	14.8
24	12"	2.4	24"	9.6
125	16"	3.7	24"	9.6
136	12"	2.4	24"	9.6
27	24"	9.6	36"	22.2
37	18"	4.2	36"	22.2
36	18"	4.2	36"	22.2
123	24"	9.6	2 X 36"	44.4
110	18"	4.2	2 X 18"	8.4

Proposed System Analysis

No changes were made to hydrologic parameters from the existing model as there are no significant anticipated land use changes.

Two catchments were altered to reflect proposed changes in grading in the northern section of the airport. 9.8 acres will be diverted from ditch segment 149 to ditch segment 16, which will be expanded to provide linear detention. Figure 11 displays the change in catchment areas.

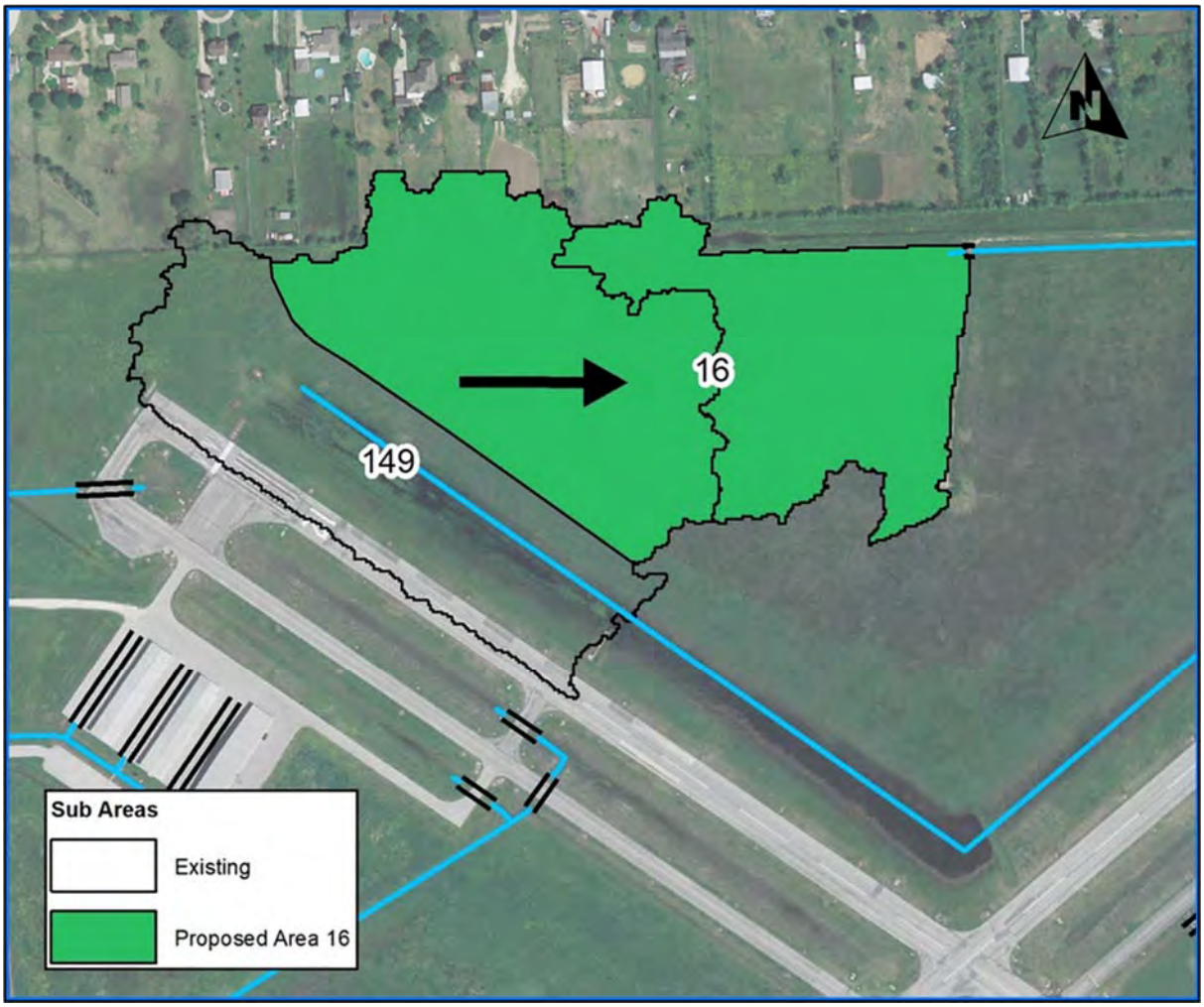


FIGURE 11 - ALTERED CATCHMENT AREAS

Hydraulic Analysis

The existing XPSWMM model was updated to reflect proposed changes to structures and ditches.

No new ditch segments were added to the existing XPSWMM model. The significant change in the model layout was extending ditch segment 16 along the northern boundary from 30 ft. to 1000 ft. Proposed upsized structures and modified ditch segment geometries were reflected in the model. Tailwater conditions and Manning's roughness coefficient were retained from the existing model. Resulting water surface elevations were then compared with the existing model results. In addition the resulting delineated ponding depths were also compared to adjacent edge of pavement elevations.

Proposed System Analysis Results

The proposed improvements will bring the drainage system to compliance with FAA criteria, i.e. no runway/taxiway encroachment during the 5-year event and no

ponding on the center 50% of runways/taxiways during the 10-year event. Generally, the proposed improvements exceed these requirements and are designed to keep the 10-year storm event within ditch banks and limit the number of culverts surcharging.

The 100-year storm event XPSWMM model results indicate that the improvements lower the water levels and prevent water ponding on the two runways and most buildings with the exception of hangar buildings in the southwest corner of the airport. The ponding at the hangar buildings area is limited to a few inches as shown in Figure 12.

FIGURE 12 - PONDING AT THE HANGAR BUILDINGS DURING 100-YR STORM EVENT

During the 10-year storm event, the proposed improvements contain flow within the ditches. Though some overtopping occurs at the west side of the airport (along Farrington Road), flooding conditions are improved (See **Exhibit 12**). During the 5-year event flow is mostly contained within the roadside ditch along the western drainage ditch.

Exhibit 11 presents the ponding during the 5-year storm event for the proposed conditions.

Exhibit 12 presents the ponding during the 10-year storm event for the proposed conditions.

Exhibit 13 presents the ponding during the 100-year storm event for the proposed conditions.

Supplemental Table C presents the maximum water surface elevation comparison with existing conditions.

Supplemental Table D presents a comparison between water surface elevation and the nearest edge of pavement.

Analysis of Ties to Offsite Drainage Systems

Areas along the southwest and northeast borders are prone to flooding during the 10-year storm event. A cursory analysis was undertaken to see the relationship between these areas and the bordering offsite regions. In both cases, the airport does not receive any runoff from the neighborhoods. No negative impacts to the airport would be caused by overflow from these systems.

5. Affected Facilities not in this Project

5.1. PIPELINES

There are no pavement improvements that are expected to affect the pipelines.

There are at least 3 existing pipelines running across the airport. The existing pipelines are shown in Figure 6. Three pipelines are in the pipeline corridor as identified in ALP, and one pipeline (stand-alone) is running parallel and close to east property line and swings north east. During the notification process, the location of this pipeline could not be confirmed, but it is suspected that it is actually within the major corridor to the east of the airport. A total of 4 conflicts are encountered, of which three fall into the stand-alone pipe. We were unable to locate any pipeline markings for this stand-alone pipe based on the desktop review. Further investigation is needed in design to confirm the actual location of the pipeline.. Figure 13 shows the potential conflicts.

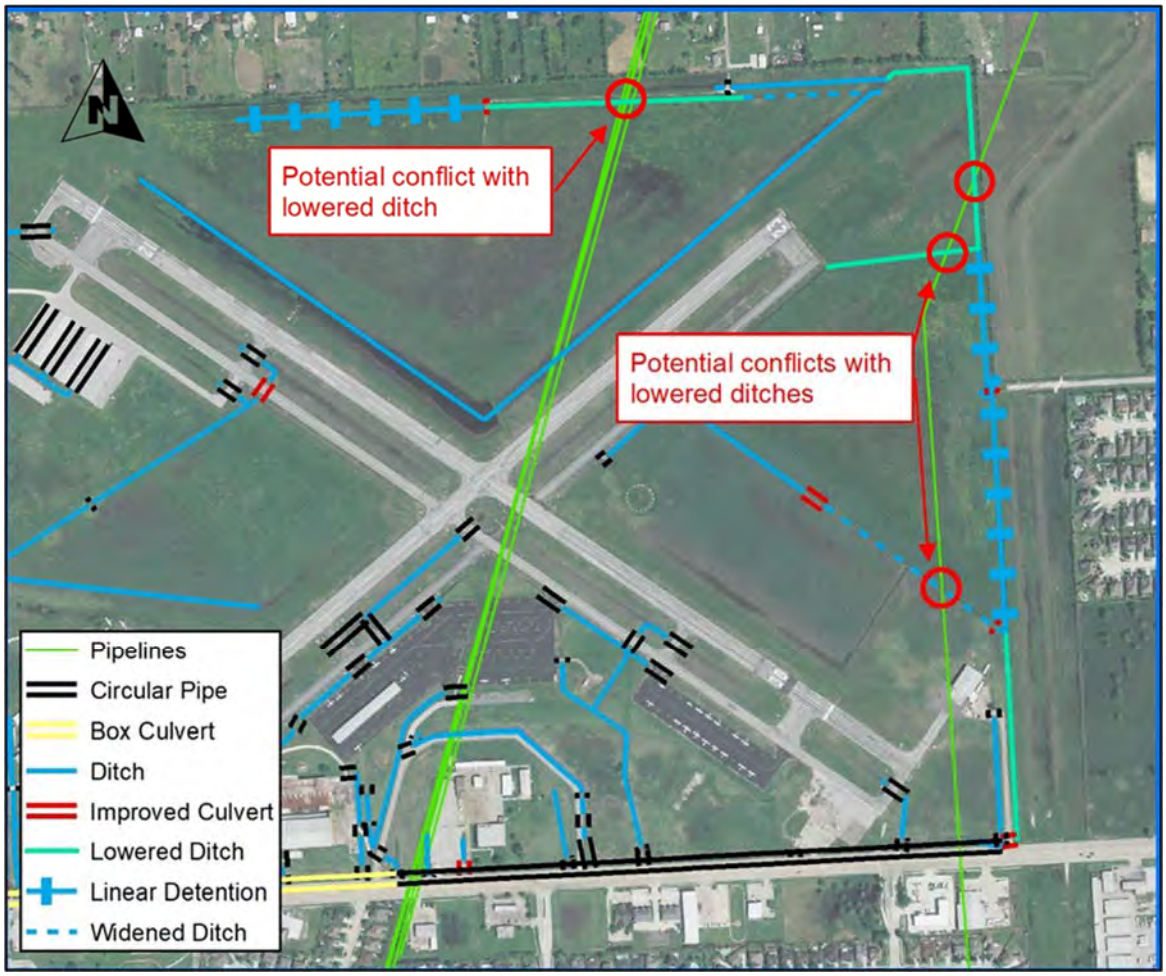


FIGURE 13 - POTENTIAL PIPELINE CONFLICTS

Discussions with Don Pennell will lead to design of drainage across the pipeline corridor being set to avoid the pipelines, rather than relocating the pipelines. This will include inlet structures on each side of the corridor, with a bored conduit sized to carry flow beneath all the pipelines. The expected flowline will be below any drainage ditch flowline, but the inlets will have access to allow for periodic de-silting of the structures.

6. Conclusions and Recommendations

By following the discussion of the design for each of the items listed for evaluation, the total estimated construction cost for the project is in excess of $3.5 Million. This is far beyond the programmed level of funding. However, the removal of components of the project will help reduce the cost to bring it closer to budget. Therefore, the listing below removes a major item, and removes components of two other items to reduce costs.

6.1. PAVEMENT REHABILITATION

The project as described above, but with the removal of the reconstruction of the western portion of Taxiway A is recommended. Also, while desirable but not necessary, the spray

seal coat surfacing of aprons is also removed from the recommended improvements. These reductions reduce the overall project cost and is discussed in detail below in Section 8.

6.2. Drainage Improvements

FAA requires no encroachment of runoff on runway and taxiway pavements for the 5-year storm event. The center 50-percent of runways and taxiways is to be kept free of ponding resulting from the 10-year storm event.

This report takes 10-year storm event as a design storm event and provides recommendations to keep the runways and taxiways free from the runoff encroachment for storm events up to and including the 10-year storm event for the reasons as noted in Section 2.4.

Additionally, we have evaluated the cost impact of selecting 10-year storm event as a design event. We conclude the cost impact is very minimal. All the structural improvements recommended for 10-year event are necessary even for 5-year storm event. The selecting 5-year event as design event would result in approximately 2 ac-ft. of less detention volume, equivalent of about $26,000.

It is recommended that the airport pursue the proposed enhancements to the drainage system in order to eliminate ponding over the runways for storm events less frequent than the 10-year event. Additionally, the proposed improvements will lower the impact to airport operations during the more severe storm events. The recommended improvements consist of (See Exhibit 8):

1) Modification of 7,600 LF of drainage ditches including 1,600 LF of linear detention

2) Upsizing of 12 drainage structures totaling 860 LF

The anticipated construction cost of all improvements for drainage is approximately $612,000 dollars. Of the total, modification of open channels and linear detention would cost about 45% and modification to structures would cost approximately 55% of the total estimate. The cost estimate includes a 20% contingency.

Due to the limitations of funding, it is recommended that approximately ¼ of the total design volume of detention be constructed in the project. This will bring the recommended drainage improvements total cost to approximately 351,800, which is much closer to the programmed $330,000 budget level.

6.3. Alternates

The former apron pavement near the end of Runway 30 is currently not scoped in the base project for pavement rehabilitation. This apron serves the self-serve fueling on the airport and should be preserved, even though it was not identified in the project. This pavement can be placed into the construction project as an additive alternate. Based on available budget and local interest, this pavement could also be seal coated with the other construction on the airport.

Another possible alternate is to include the apron surface spray treatment. It may be possible that if good pricing is provided during bid, that room in the funding may allow for the seal coat.

A final possible alternate could be to provide additional work in the linear detention areas. Such additional work will provide more detention and ultimately get the airport closer to the "goal" of the drainage improvements that are identified as needed in this report.

7. Schedule

Based on the scale of the project, with the expectation that phasing will limit the closure of both runways at the same time, along with other limitations on phasing of the apron rehabilitation due to need for parking of aircraft, it is expected that the schedule of the project will be slightly longer than normal due to phasing and re-mobilization for certain construction activities (like pavement milling for patching). Given this, the expected timeframe for construction schedule is likely to be between 10 months and 1-year overall duration. The expectation is that the project will not bid until funding is provided for FY 2019 construction, which is usually in the spring. Therefore, the project schedule will likely follow a July 2019 to May 2020 timeframe. If the possibility of moving the project into FY18 becomes a reality, the timeframe would move up 1 year to 2018-2019. Accommodations to accelerate the construction schedule can be made, but the pricing provided in the estimate is based on a standard construction schedule so some additional cost for acceleration would need to be accommodated for if a quicker schedule is desired.

8. Cost Estimates

The cost of the project as recommended is listed below:

TABLE 6 – PROJECT ESTIMATE OF PROBABLE CONSTRUCTION COST

Item	Description	Cost
Runway 12/30 and Connectors A1, A2, A3, A4 to holdline	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Runway markings are replaced.	**$ 435,400**
Taxiway A East and 12 Runup Pad	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Taxiway markings are replaced.	**$ 125,400**
NW Hangar Access and Aprons	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Taxiway markings are replaced.	**$ 90,500**
Taxiway D	The pavement is surface treated. Edges are regraded. Taxiway markings are replaced.	**$ 5,400**
Runway 5/23 and Connectors B1, B2 to holdline	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Runway markings are replaced.	**$ 298,700**

Taxiway B	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Taxiway markings are replaced.	**$ 238,200**
Taxiway C	The pavement is patched, crack sealed, and surface treated. Edges are regraded. Taxiway markings are replaced.	**$ 36,200**
Apron 1	Low areas are filled, and the pavement is crack sealed. Edges are regraded. Apron markings are replaced.	**$ 89,100**
Apron 2	Low areas are filled, and the pavement is crack sealed. Edges are regraded. Apron markings are replaced.	**$ 185,600**
Apron 3	Low areas are filled, and the pavement is crack sealed. Edges are regraded. Apron markings are replaced.	**$ 157,700**
Drainage Improvements	Culvert replacement, channel restoration, 1/4 of total linear detention created.	**$ 351,800**
TOTAL		**$2,014,100**

These costs are only the estimate of the construction total. That programmed value with contingency was $1,941,500. This total is 4 percent over the programmed amount.

A detailed tabulation of costs is included in **Supplemental Table E**.

Additionally, there is a concern for the project budgeting in the construction services. The project programming listed $230,000 for construction services and contingency. However, the contingency is traditionally 10% of the construction or $176,500. This leaves $53,500 for testing, Resident Project Representative, and construction phase services. The cost tabulation has estimates of the professional and testing fees for the project, and an estimate of 1,000 manhours of RPR. We have determined that construction services should be anticipated and programmed at $194,000 for the project, for a grand total of the project estimated at **$2,208,100**.

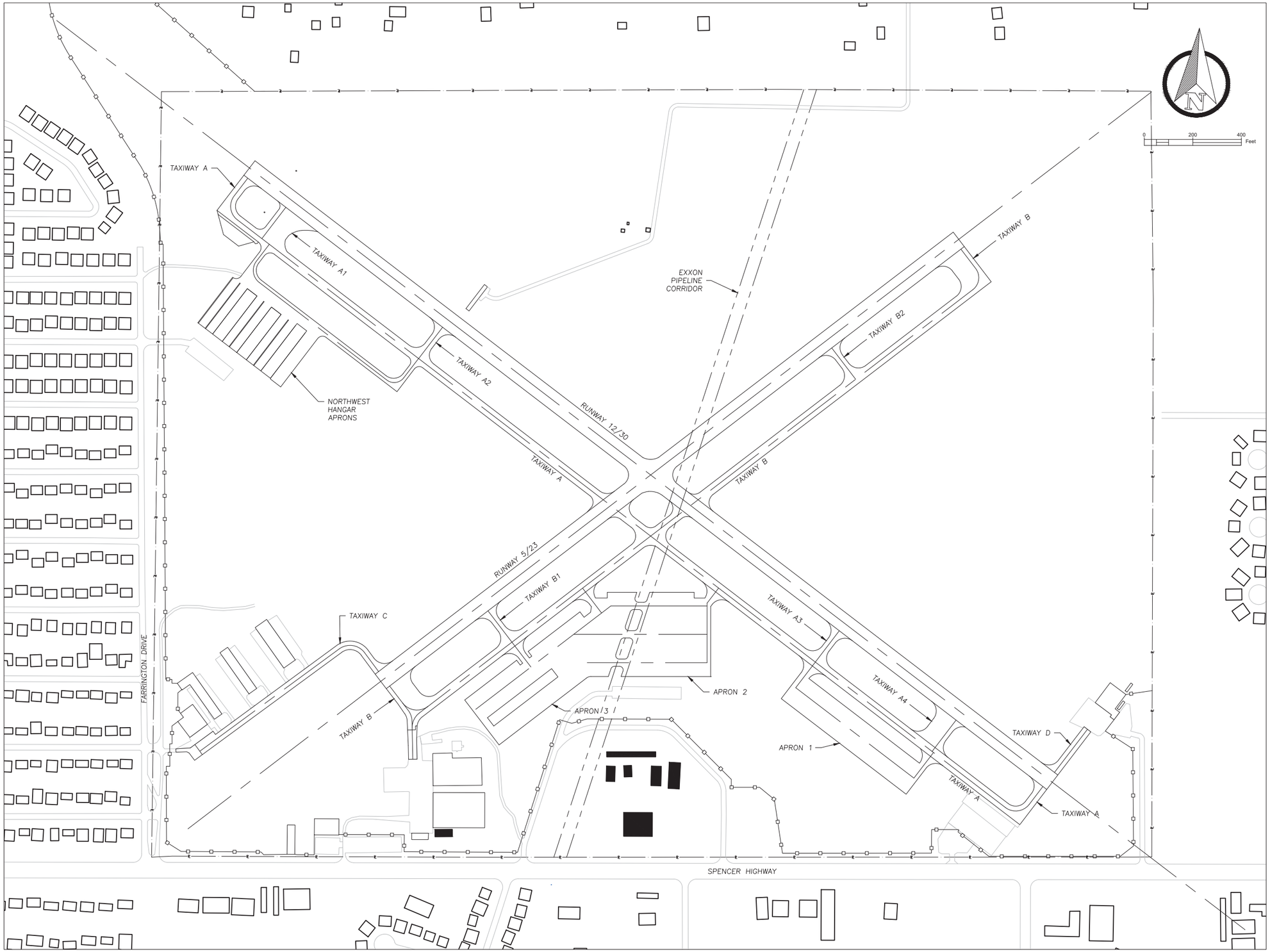
9. Modifications to Standards

There are 3 items that are not within the current standard of the FAA's advisory circular criteria:

1. The taxiway geometry, for both offset from runways, and fillet edges in turns and intersections, does not meet the current standards of 150/5300-13A. However, bringing these components of the airfield are not cost-effective and not within the current programmed costs of the project.
2. The holdlines for the taxiways connecting to both runways are not at the current standard as required in 150/5300-13A. However, the requirement of co-locating the

mandatory signs with standard holdlines (150/5340-18F) is not possible due to the existing taxiway offset from the runway. Correcting this issue is not cost-effective as it also requires the changes listed in 1. above.

3. For limited asphalt pavement patching, use of FAA specification P-403 with state highway specification normally needs a modification for the rated weight limits as listed in the ALP. However, the draft -10H standard specifications sets a tonnage limitation of 3000 tons where the state specification can be used instead (as a repair project – which this is). If this draft specification is finalized and issued prior to bidding, the use of a modification will not be needed.



REVISIONS

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LA PORTE MUNICIPAL AIRPORT

AIRFIELD PAVEMENT REHABILITATION AND DRAINAGE IMPROVEMENTS – 2017

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T. DODSON

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J.T. DODSON

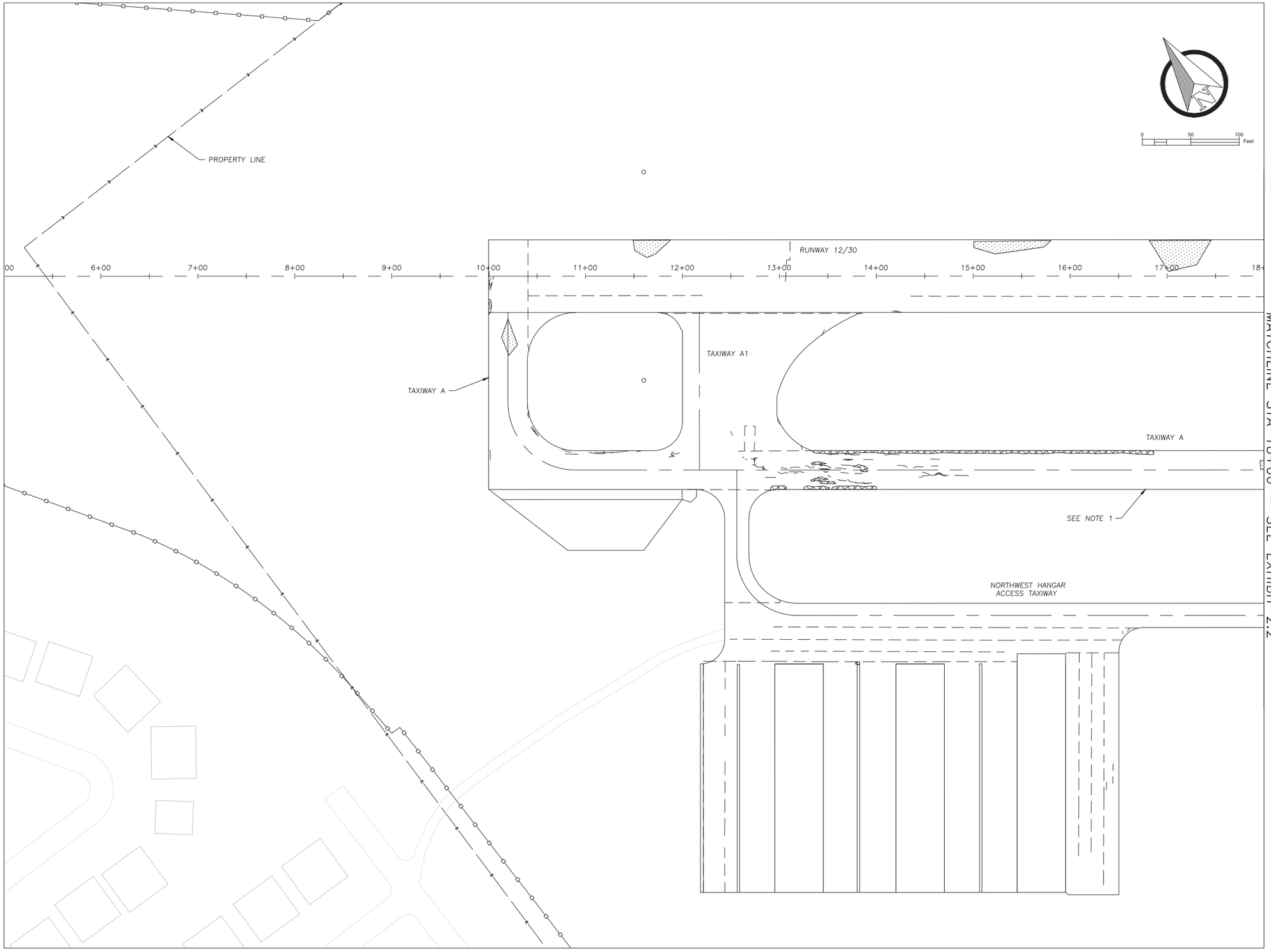
CHECKED BY
J.H. DODSON

PROJ NO
1712LPRTE

CIVIL PEs PROJ NO
1712LPRTE

TITLE
SITE LAYOUT

SHEET
EXHIBIT 1



LEGEND

- ALLIGATOR–TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

NOTES

1. THIS PORTION OF TW A WAS NOT STUDIED FOR BLOCK CRACKS DUE TO EXTREME QUANTITY OF CRACKS AND SURFACE DEFORMITY.

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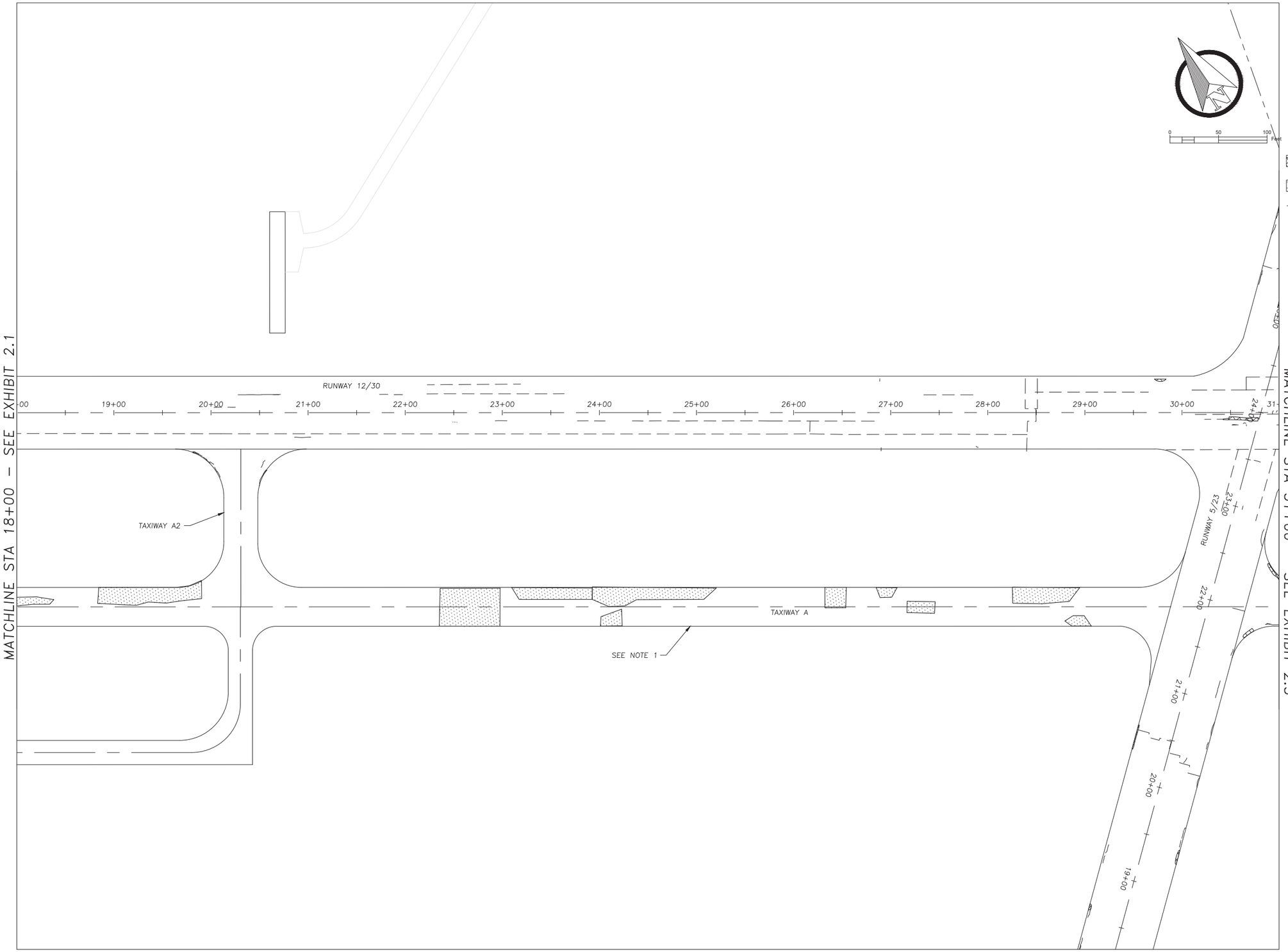
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 1

SHEET
EXHIBIT 2.1



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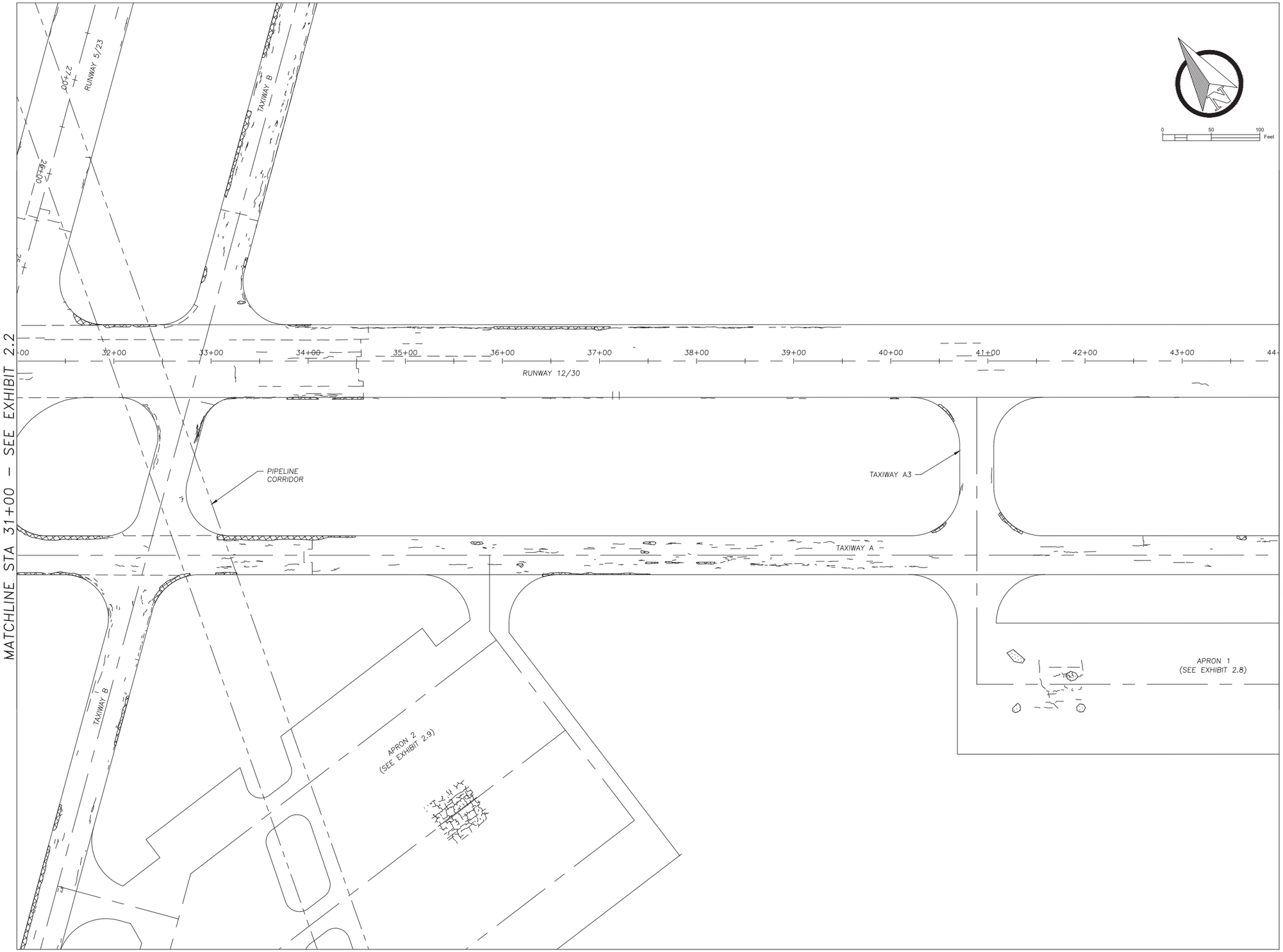
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 2

SHEET
EXHIBIT 2.2



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LA PORTE MUNICIPAL AIRPORT

AIRFIELD PAVEMENT REHABILITATION AND DRAINAGE IMPROVEMENTS – 2017

PROJ MGR
T. DODSON

DRAWN BY
J.T. DODSON

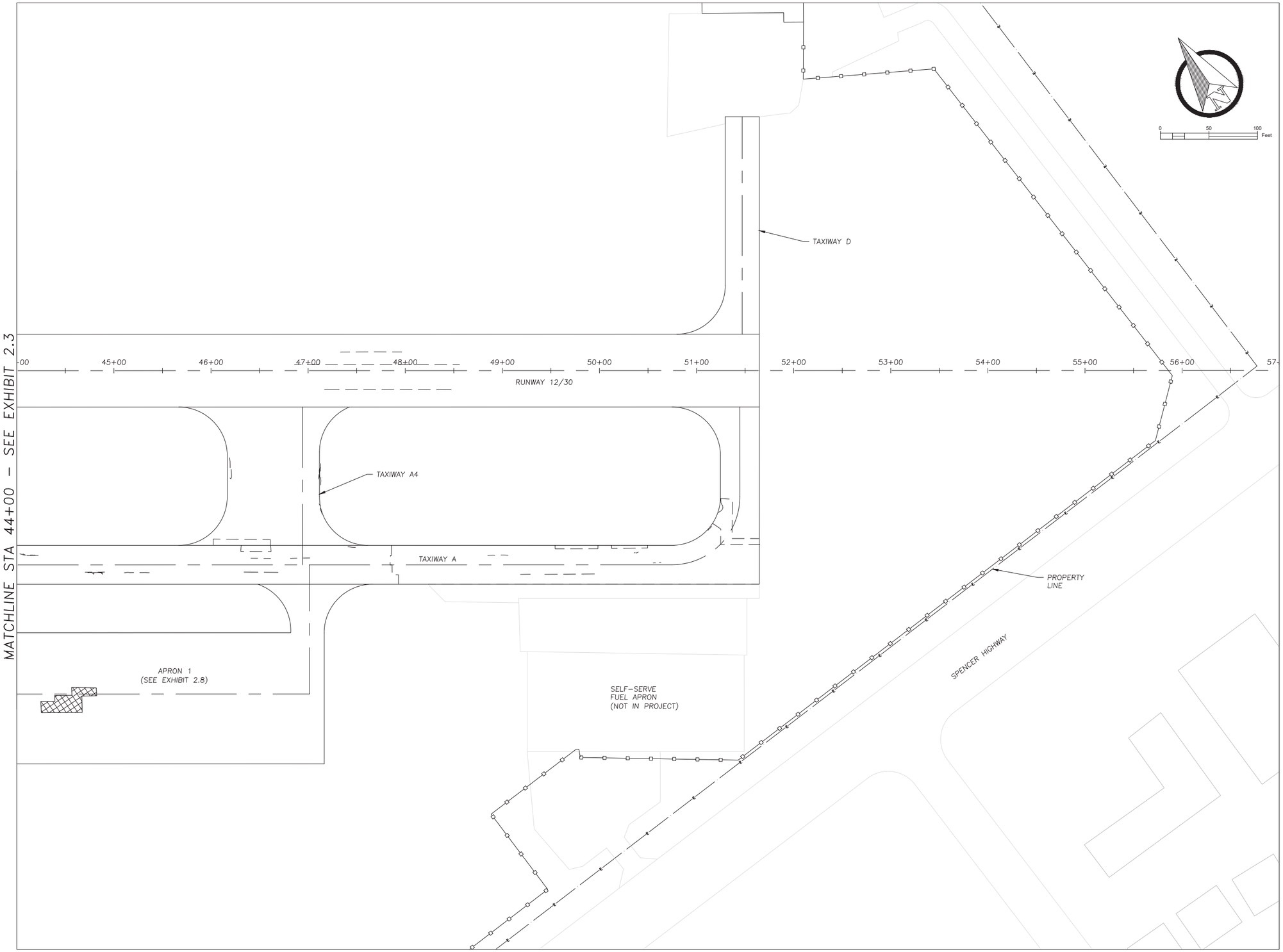
CHECKED BY
J.H. DODSON

PROJ NO
1712LPRTE

CIVIL PEs PROJ NO
1712LPRTE

TITLE
PAVEMENT DISTRESS INVENTORY SHEET 3

SHEET
EXHIBIT 2.3



LEGEND

- ALLIGATOR–TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

REVISIONS

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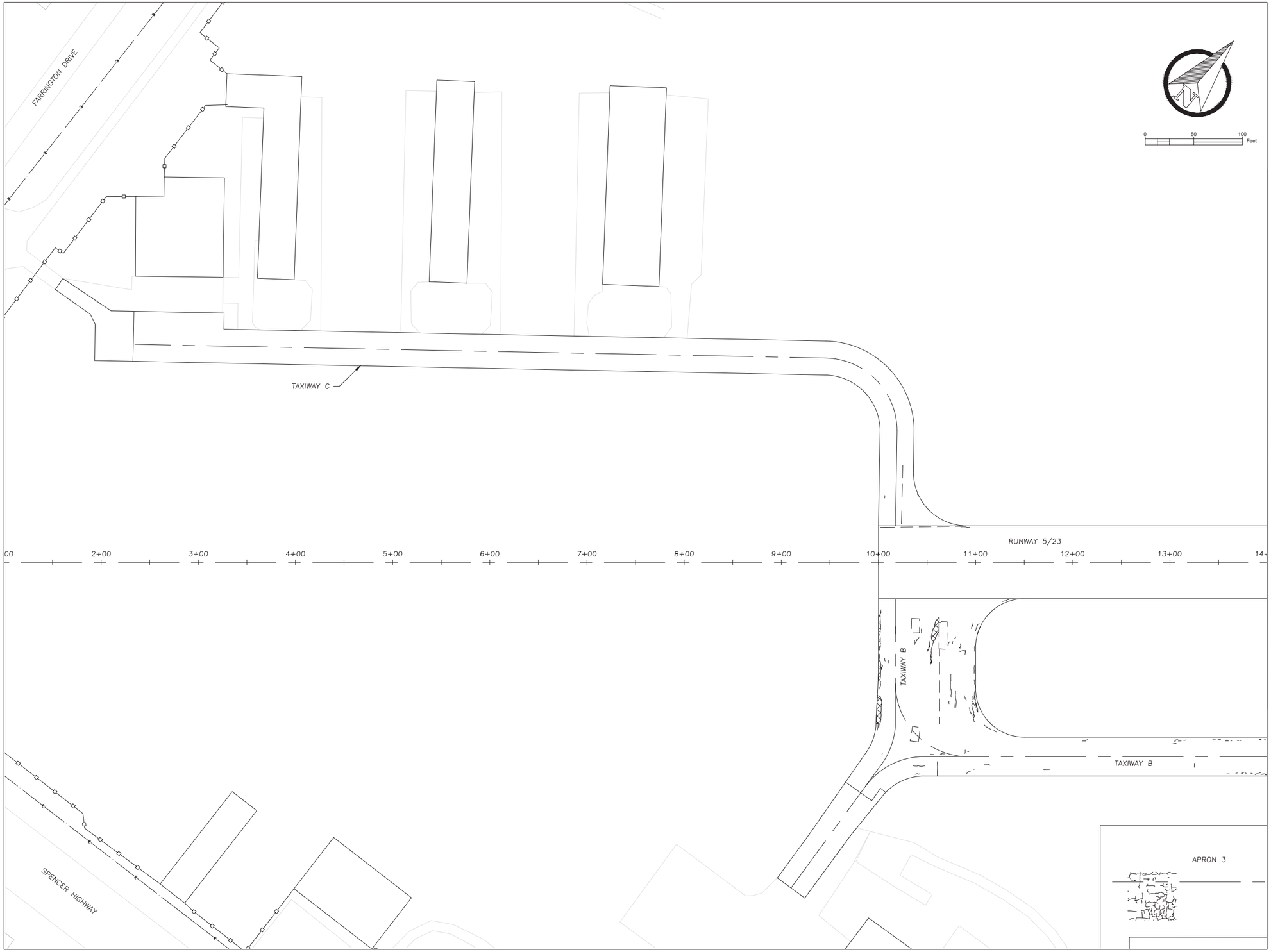
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 4

SHEET
EXHIBIT 2.4



MATCHLINE STA 14+00 – SEE EXHIBIT 2.6

LEGEND

- ALLIGATOR–TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

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CIVIL PEs PROJ NO
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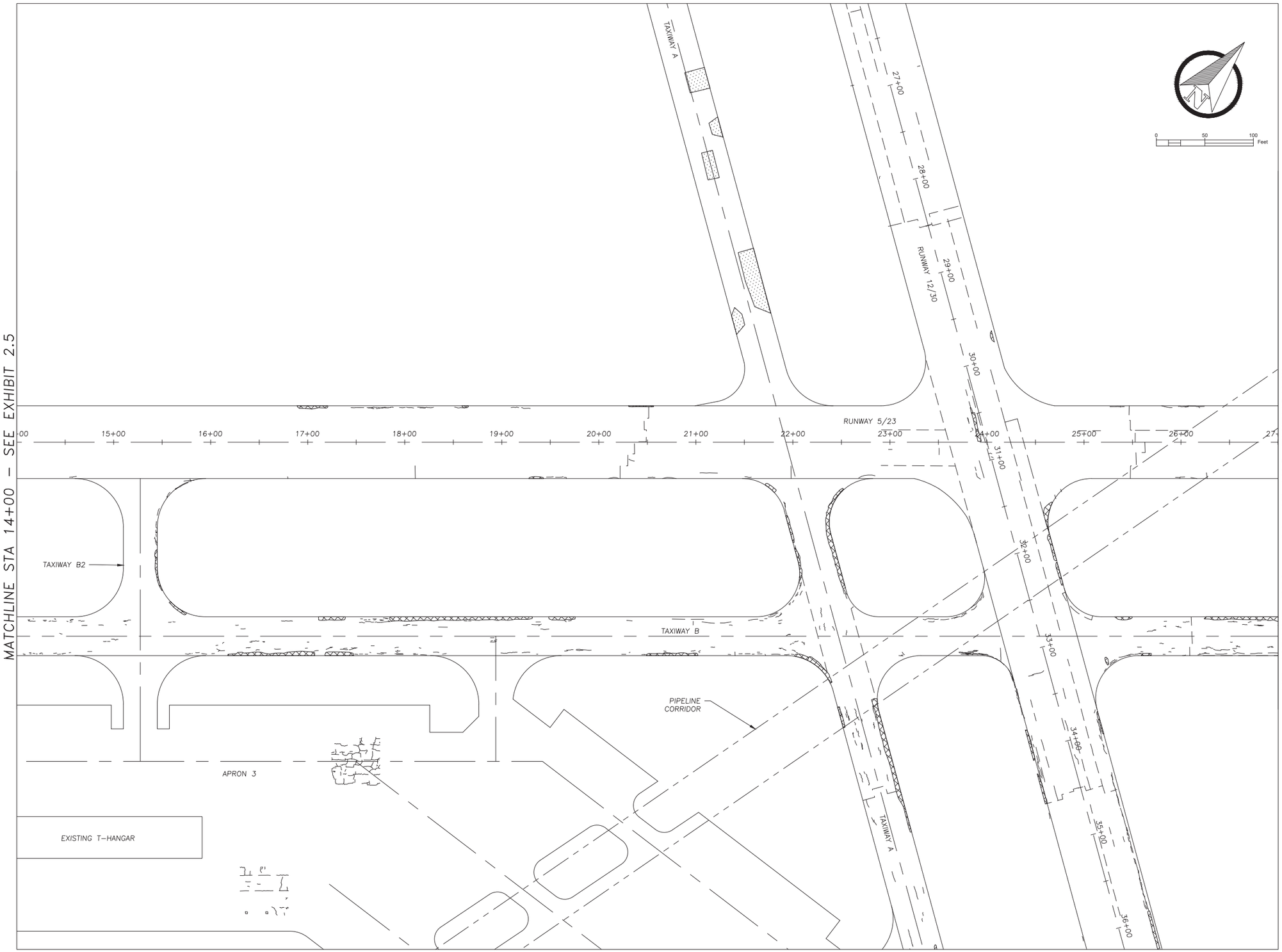
TITLE

PAVEMENT DISTRESS INVENTORY SHEET 5

SHEET
EXHIBIT 2.5



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CIVIL PEs PROJ NO
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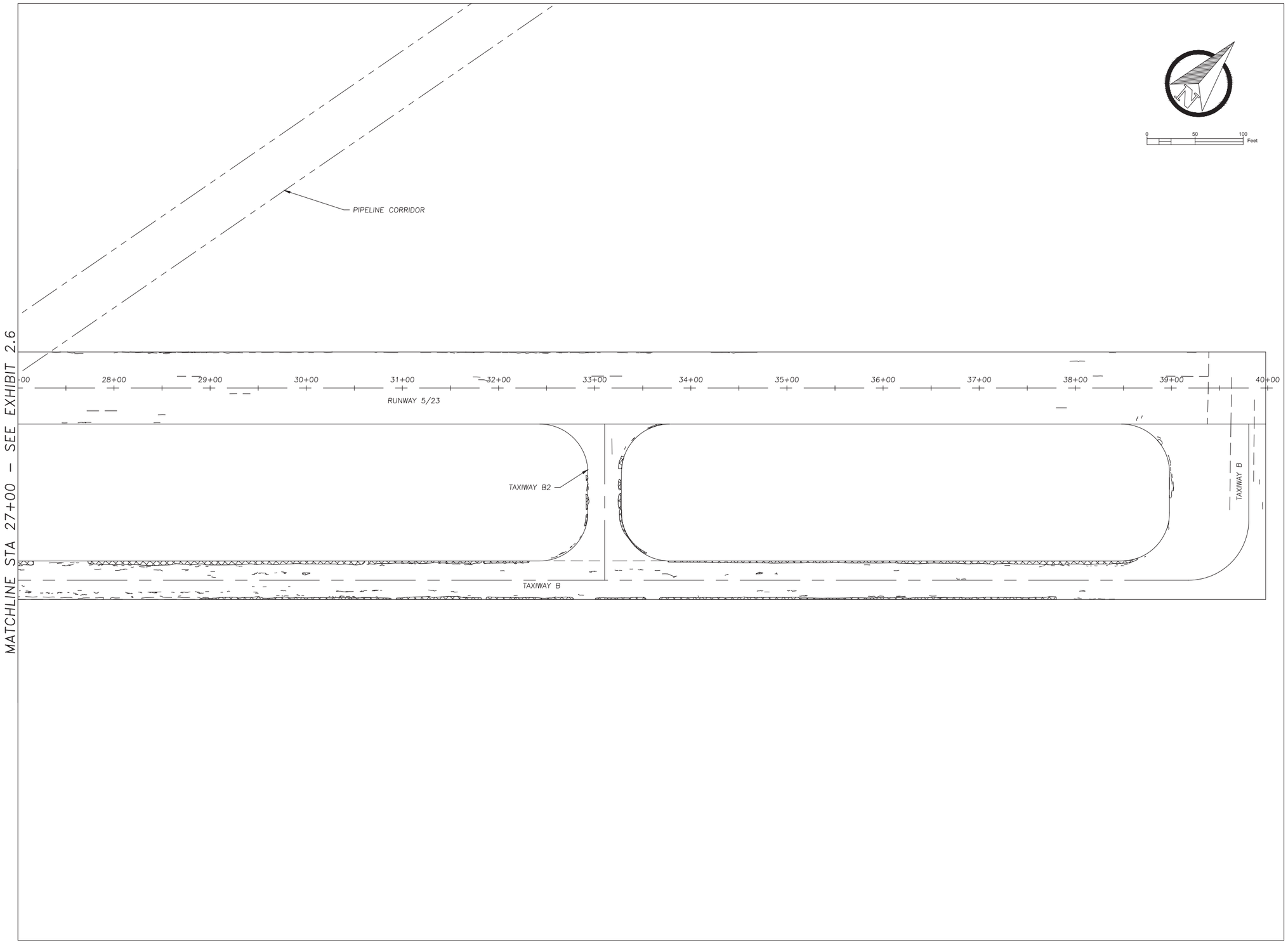
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 6

SHEET
EXHIBIT 2.6



LEGEND

- ALLIGATOR–TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

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CIVIL PEs PROJ NO
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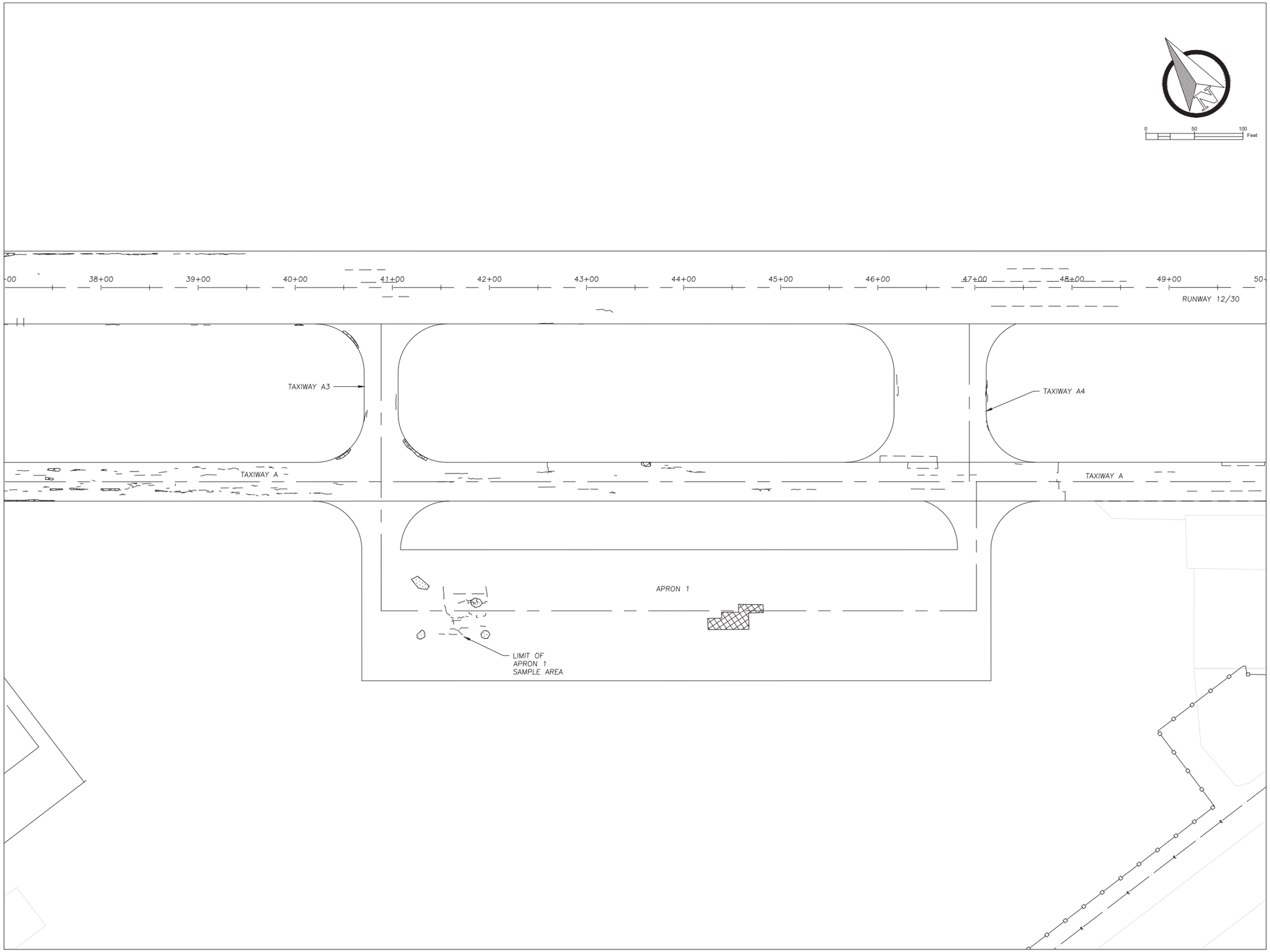
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 7

SHEET
EXHIBIT 2.7



LEGEND

- ALLIGATOR-TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

NOTES

1. APRON 3 WAS SAMPLED FOR PAVEMENT DISTRESS BASED ON A TYPICAL AREA OF DISTRESS.

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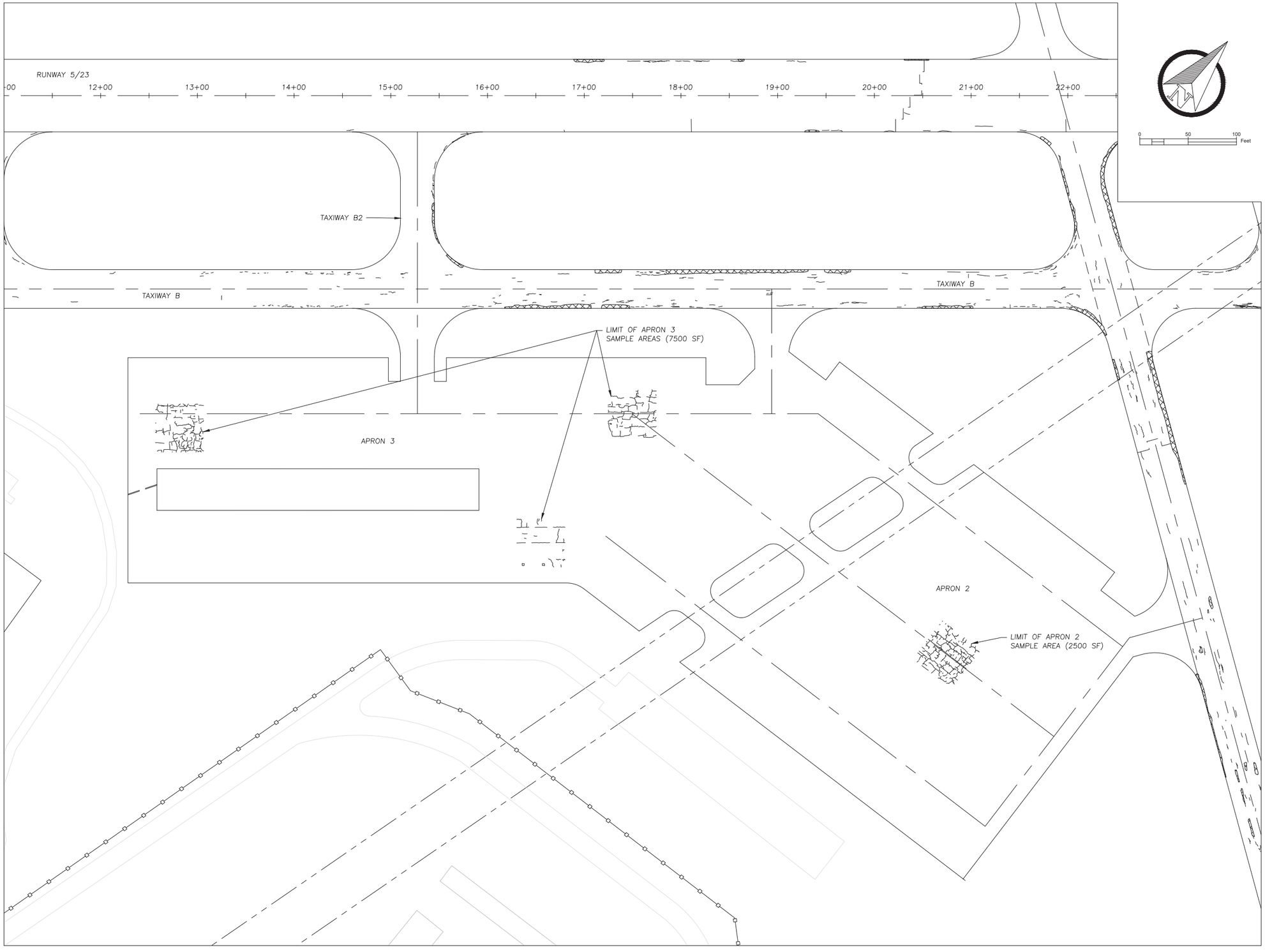
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 8

SHEET
EXHIBIT 2.8



LEGEND

- ALLIGATOR–TYPE CRACK AREA LIMITS
- DEPRESSED PAVEMENT SURFACE LIMITS
- BLOCK CRACKS

NOTES

1. APRONS 2 AND 3 WERE SAMPLED FOR PAVEMENT DISTRESS BASED ON TYPICAL AREAS OF DISTRESS.

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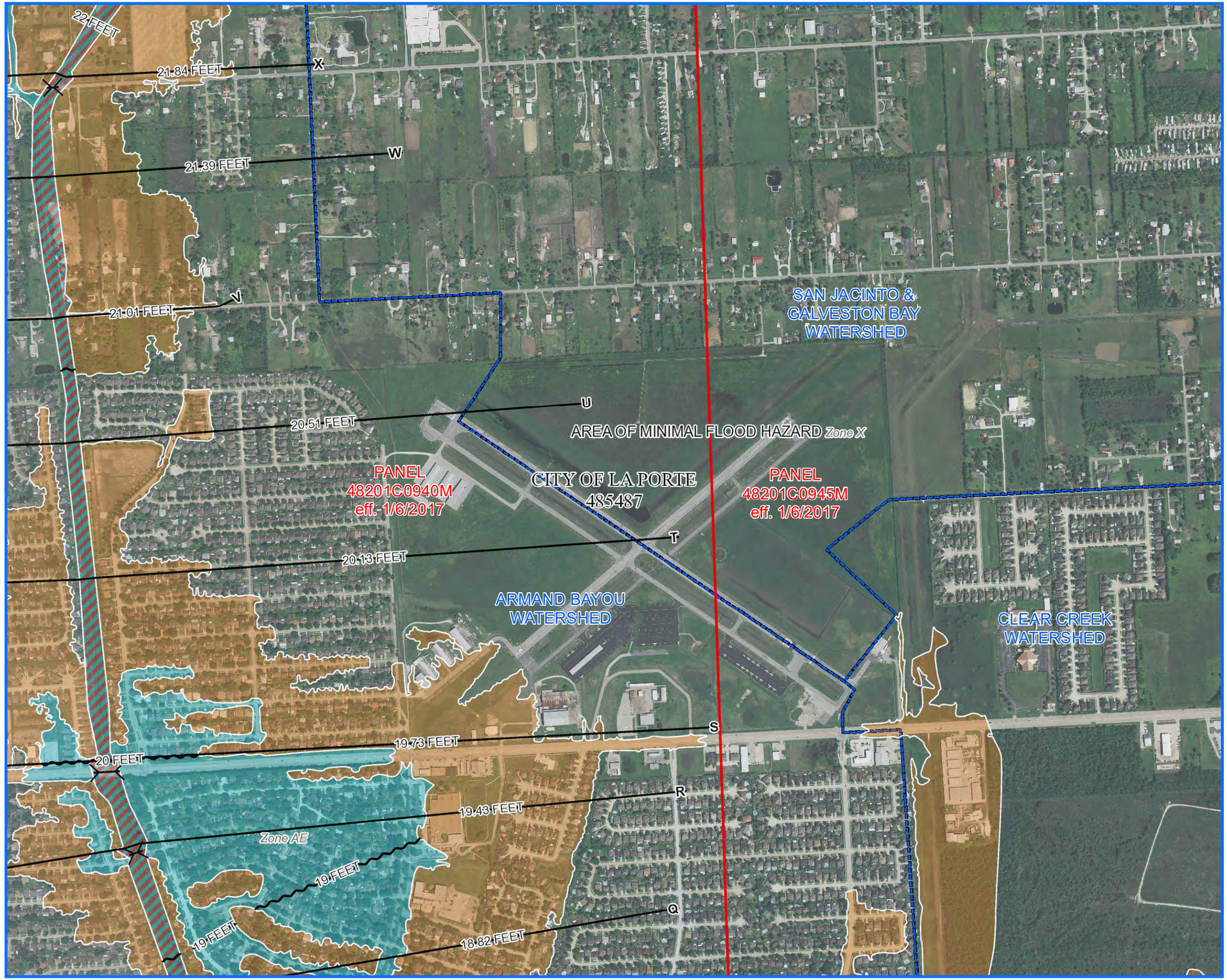
TITLE
PAVEMENT DISTRESS INVENTORY SHEET 9

SHEET
EXHIBIT 2.9



0 800 1,600
Feet

Harris County Watershed Boundaries

1% Annual Chance Flood Hazard
Regulatory Floodway
Special Floodway
Area of Undetermined Flood Hazard
0.2% Annual Chance Flood Hazard
Future Conditions 1% Annual Chance Flood Hazard
Area with Reduced Risk Due to Levee

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 3
Effective Floodplain

HT&J, LLC
Professional Engineering Services

0 575 1,150
Feet

Structure Type

- Circular Pipe
- Drainage Ditch
- Box Culvert
- Flow Direction

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 4.0
Drainage Structures
Overview

HT&J, LLC
Professional Engineering Services

Culvert	Diameter (ft)	Length (ft)	Invert U/S	Invert D/S
24	1.00	24.20	22.85	22.68
43	1.50	86.40	19.99	19.83
101	0.75	234.70	22.80	21.57
102	0.75	234.70	22.85	21.47
103	0.75	234.60	22.70	21.67
105	1.50	122.00	21.25	21.24
106	1.00	52.40	21.23	21.17
108	1.50	87.70	19.85	19.82
110	1.50	85.80	19.80	19.28
117	2.50	78.50	19.57	19.56
120	3.00	24.70	19.60	19.58
123	2.00	22.50	18.60	18.29
125	1.25	34.10	20.17	19.35
130	2.50	20.80	18.74	18.70
132	1.50	85.00	20.70	19.61

N

0 200 400
Feet

Structure Type

- Circular Pipe
- Drainage Ditch
- Box Culvert

Map Index:

4.1	4.2
4.3	4.4

City of La Porte
La Porte
Municipal Airport
Storm Water
Drainage Improvements

Exhibit 4.1
Existing
Drainage Structures

HT&J, LLC
Professional Engineering Services

Culvert	Diameter (ft)	Length (ft)	Invert US	Invert DS
23	1.25	24.60	21.27	20.68
25	2.00	24.80	18.78	18.68
34	1.50	138.20	19.53	19.45
36	1.50	99.50	18.75	18.66
40	1.25	43.20	21.11	20.06
41	1.50	86.60	20.09	20.07

N

0 200 400
Feet

Structure Type

Circular Pipe

Drainage Ditch

Box Culvert

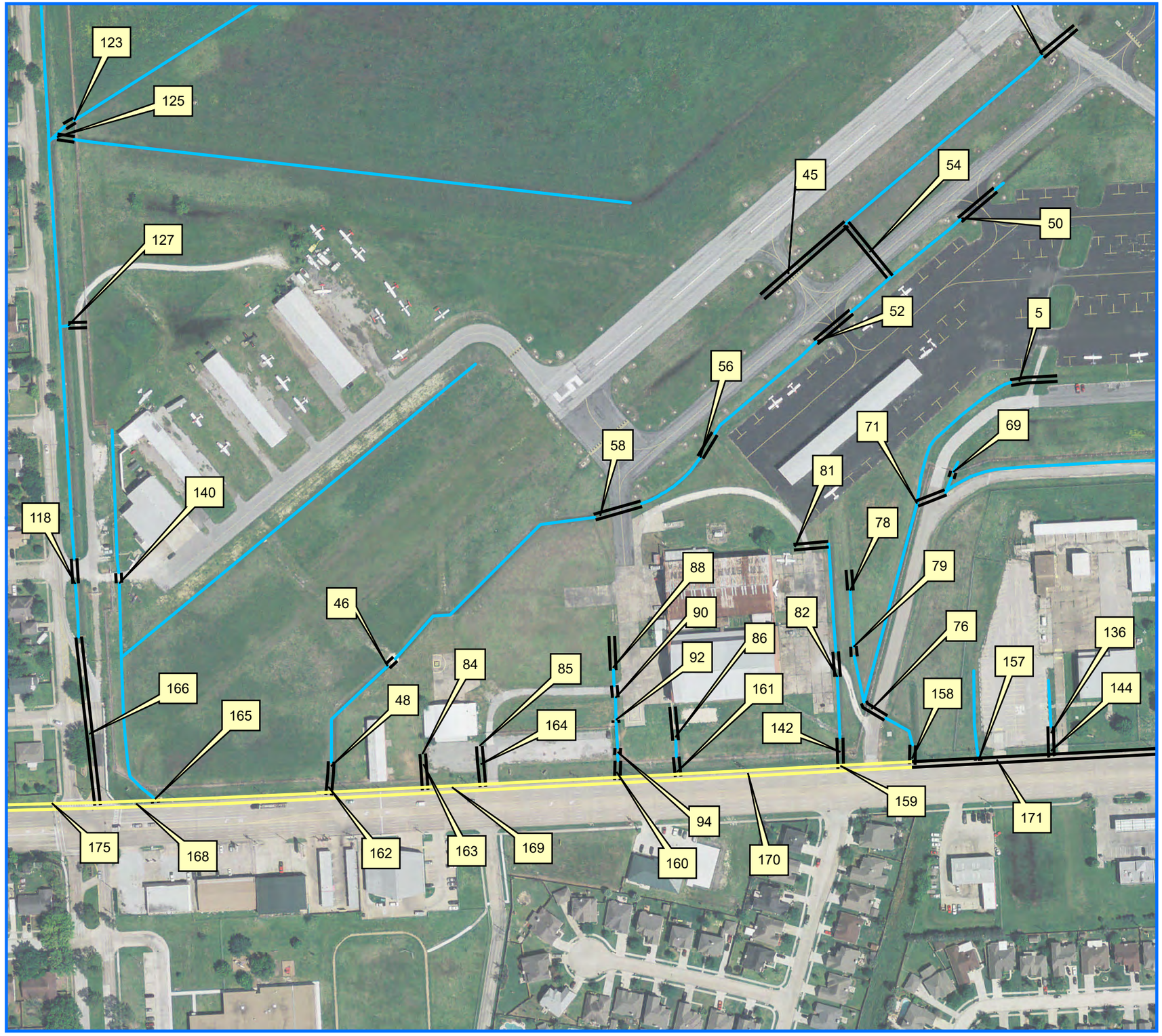
Map Index:

4.1	4.2
4.3	4.4

City of La Porte
La Porte
Municipal Airport
Storm Water
Drainage Improvements

Exhibit 4.2
Existing
Drainage Structures

HT&J, LLC
Professional Engineering Services



Culvert	Diameter (ft)	Length (ft)	Invert US	Invert DS
5	1.25	91.70	21.93	21.66
45	1.25	216.60	19.74	19.58
46	2.00	20.80	18.19	17.91
48	3.50	25.00	17.46	15.17
50	1.25	90.80	20.45	19.93
52	1.50	91.10	19.33	18.91
54	1.50	135.30	19.74	19.60
56	2.00	54.30	18.71	18.65
58	2.00	94.90	18.46	18.36
69	2.00	12.70	20.80	20.67
71	2.00	60.30	20.06	20.02
76	2.00	54.10	17.68	17.23
78	1.00	40.40	20.67	20.54
79	1.25	19.70	20.29	20.00
81	1.00	67.70	20.16	20.06
82	1.25	49.30	19.06	18.96
84	1.00	10.00	20.68	20.42
85	1.00	33.20	20.03	20.02
86	1.25	66.00	18.79	18.69
88	1.25	66.90	20.39	20.17
90	1.25	24.30	20.07	20.06
92	1.25	5.60	19.69	19.59
94	1.25	14.30	19.34	19.00
118	2.50	49.20	16.27	15.79
123	2.00	22.50	18.60	18.29
125	1.25	34.10	20.17	19.35
127	1.25	36.50	19.49	19.29
136	1.00	54.70	21.00	19.50
140	0.75	20.30	19.63	19.43
142	2.00	54.10	17.76	17.00
144	2.00	271.60	19.60	10.64
157	2.00	129.60	18.00	10.64
158	2.00	37.80	17.10	10.64
159	2.00	139.10	17.00	10.64
160	2.00	48.50	18.19	6.42
161	2.00	149.30	18.50	6.42
162	2.00	50.80	15.17	6.18
163	2.00	220.00	20.42	6.18
164	2.00	326.60	20.02	6.18
165	2.00	113.00	0.00	6.05
166	2.00	329.60	0.00	6.05
171	6.00	780.70	11.15	10.64

Culvert	Height (ft)	Width (ft)	Length (ft)	Invert US	Invert DS
168	6.00	8.00	426.40	6.18	6.05
169	6.00	7.00	560.10	6.42	6.18
170	6.00	6.00	601.70	6.85	6.42
175	6.00	9.00	289.30	6.05	5.5

N

0 200 400
Feet

Structure Type

Circular Pipe

Drainage Ditch

Box Culvert

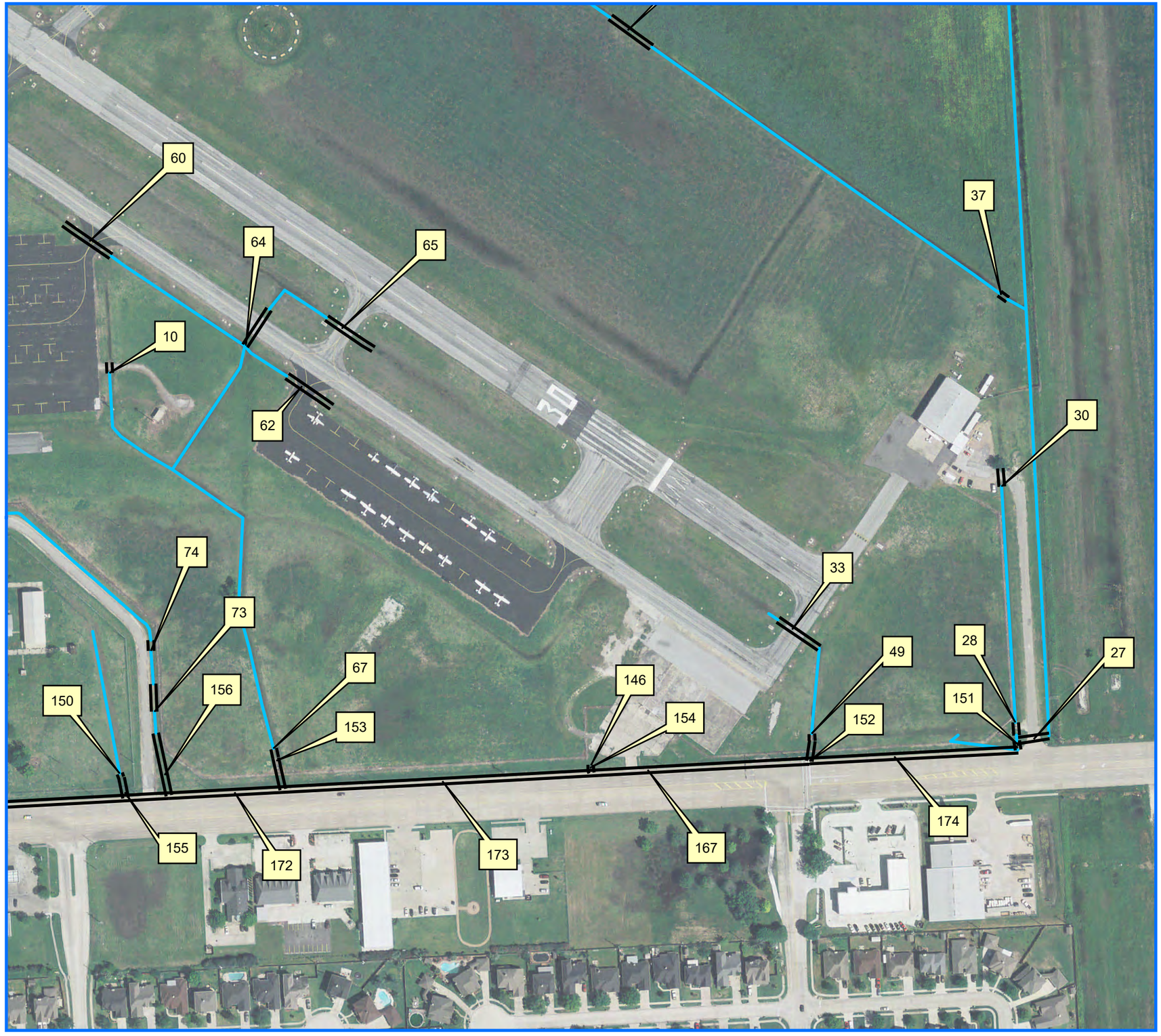
Map Index:

4.1 4.2
4.3 4.4

City of La Porte
La Porte
Municipal Airport
Storm Water
Drainage Improvements

Exhibit 4.3
Existing
Drainage Structures

HT&J, LLC
Professional Engineering Services



Culvert	Diameter (ft)	Length (ft)	Invert US	Invert DS
10	1.25	21.00	21.48	21.41
27	2.00	63.90	17.86	15.88
28	2.00	25.00	18.19	18.13
30	1.50	35.70	19.59	19.11
33	2.00	97.50	20.24	20.01
36	1.50	99.50	18.75	18.66
37	1.50	20.20	18.57	18.56
49	2.00	13.60	19.68	18.27
60	1.25	112.00	19.78	19.74
62	1.25	106.10	19.69	19.48
64	1.25	85.80	19.54	19.26
65	1.25	109.70	20.25	20.05
67	3.00	19.00	18.95	17.20
73	1.25	54.10	20.48	20.47
74	1.25	20.70	20.25	20.21
146	2.00	10.70	21.00	20.00
150	1.25	8.70	18.48	18.05
151	2.00	18.10	15.88	14.07
152	2.00	40.20	18.27	13.92
153	3.00	247.60	17.20	11.87
154	2.00	130.50	20.00	13.02
155	2.00	99.00	18.05	11.15
156	2.00	124.20	20.48	11.15
167	4.50	556.60	13.42	13.02
172	5.50	452.00	11.87	11.65
173	5.00	241.70	12.52	12.37
174	4.00	407.00	14.07	13.92

N

0 200 400
Feet

Structure Type

Circular Pipe

Drainage Ditch

Box Culvert

Map Index:

4.1	4.2
4.3	4.4

City of La Porte
La Porte
Municipal Airport
Storm Water
Drainage Improvements

Exhibit 4.4
Existing
Drainage Structures

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Structure Type

Circular Pipe

Drainage Ditch

Box Culvert

26 = Ditch Segment

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 5
Existing Drainage Ditches

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

No Surcharge

Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 6
Existing Drainage System
5-year Inundation

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

No Surcharge

Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 7
Existing Drainage System
10-year Inundation

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

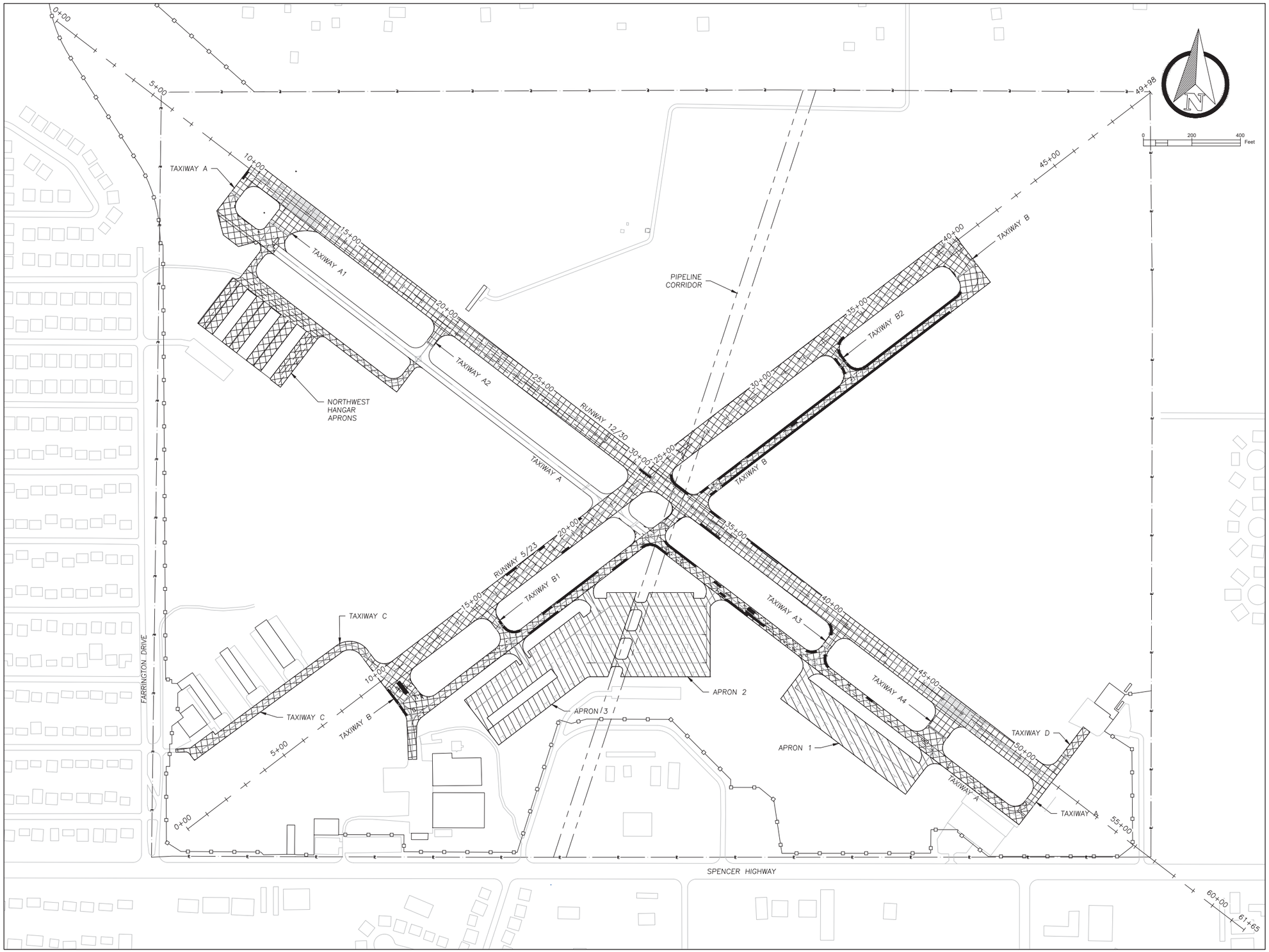
High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 8
Existing Drainage System
100-year Inundation

HT&J, LLC
Professional Engineering Services



LEGEND

- ASPHALT PAVEMENT PATCH REPAIRS
- CRACK SEAL OF ASPHALT PAVEMENT
- THERMO-PLASTIC SEAL COAT TO BE REPLACED
- COAL-TAR PITCH SEAL COAT FOR TAXIWAYS
- COAL-TAR PITCH SEAL COAT FOR RUNWAYS

NOTES

1. APRONS 1-3 AND TW C WERE SAMPLED FOR QUANTITY OF CRACK SEAL. NO INDIVIDUAL CRACK SEAL SHOWN ON APRONS 1-3 AND TW C.

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LA PORTE MUNICIPAL AIRPORT

AIRFIELD PAVEMENT REHABILITATION AND DRAINAGE IMPROVEMENTS – 2017

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T. DODSON

DRAWN BY
J.T. DODSON

CHECKED BY
J.H. DODSON

PROJ NO
1712LPRTE

CIVIL PEs PROJ NO
1712LPRTE

TITLE
PROPOSED PAVEMENT IMPROVEMENTS

SHEET
EXHIBIT 9

0 400 800
Feet

Structure Type

- Circular Pipe
- Box Culvert
- Ditch
- Improved Culvert
- Lowered Ditch
- Linear Detention
- Widened Ditch

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 10
Proposed Drainage Improvements

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 11
Proposed Drainage System
5-year Inundation

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 12
Proposed Drainage System
10-year Inundation

HT&J, LLC
Professional Engineering Services

0 400 800
Feet

Ditches

In Channel

Overbank

Structures

Surcharge

No Surcharge

Ponding Depth

High : 4

Low : 0

City of La Porte
La Porte Municipal Airport
Storm Water Drainage Improvements

Exhibit 13
Proposed Drainage System
100-year Inundation

HT&J, LLC
Professional Engineering Services

SUPPLEMENTAL TABLE A

Pavement Distress Quantities

SUPPLEMENTAL TABLE A - PAVMENT DISTRESS QUANTITIES

MeteorSpec LT areas calculation. Other areas

Type of object	Qty, pcs	A sum, sq.ft
APRON ALLIG CRACKS	2	881.19
RW 12 30 ALLIG CRACKS	15	896.11
RW 5 23 ALLIG CRACKS	17	1055.74
TW A ALLIG CRACKS	15	1196.18
TW B ALLIG CRACKS	27	6073.62
Total in the table:	76	10102.84

Search for objects is carried out on the layer "meteor-spec-lt-area" in Model space
Printed: 22.12.2017 – 16:47 MeteorSpec LT 1.3 FREE SOFTWARE

MeteorSpec LT lengths calculation. Other utilities

Type of object	Qty, pcs	L sum, ft
APRON 1 SAMPLE	27	207.03
APRON 2 SAMPLE	164	762.04
APRON 3 SAMPLE 1 BLOCK CRACK	111	649.07
APRON 3 SAMPLE 2 BLOCK CRACK	24	205.77
APRON 3 SAMPLE 3 BLOCK CRACK	61	478.26
NW HANGAR APRON BLOCK CRACK	33	2549.55
RW 1230 BLOCK CRACK	252	6360.31
RW 523 BLOCK CRACK	176	2429.28
TW A BLOCK CRACK	274	2863.62
TW B BLOCK CRACK	350	1561.22
Total in the table:	1473	18068.78

Search for objects is carried out on the layer "meteor-spec-lt-length" in Model space
The total length of each linear object is calculated as the length of polyline plus the additional length, set by User
Printed: 22.12.2017 – 16:45 MeteorSpec LT 1.3 FREE SOFTWARE

SUPPLEMENTAL TABLE B:
Watershed Data

Supplemental Table B
Watershed Data

Drainage Area	Area (acres)	C-value	Tc (min)	Rainfall Intensity (in/hr)					Runoff (cfs)				
				2-yr	5-yr	10-yr	25-yr	100-yr	2-yr	5-yr	10-yr	25-yr	100-yr
1	3.03	0.48	12.31	4.63	5.75	6.52	8.32	8.88	6.73	8.36	9.47	12.10	12.91
2	0.20	0.42	6.98	5.50	6.70	7.53	9.48	10.09	0.45	0.55	0.62	0.78	0.83
3	0.10	0.34	6.03	5.69	6.91	7.76	9.72	10.35	0.19	0.23	0.26	0.32	0.35
4	0.75	0.32	12.06	4.66	5.79	6.56	8.37	8.93	1.11	1.38	1.56	1.99	2.13
5	0.35	0.67	6.45	5.60	6.82	7.66	9.61	10.23	1.31	1.60	1.80	2.26	2.40
6	1.34	0.33	36.84	2.76	3.59	4.16	5.51	5.92	1.21	1.57	1.82	2.41	2.59
7	3.49	0.24	59.15	2.06	2.74	3.21	4.32	4.65	1.69	2.25	2.63	3.54	3.81
8	1.21	0.38	9.28	5.08	6.25	7.05	8.94	9.53	2.35	2.89	3.26	4.13	4.41
9	0.34	0.31	5.67	5.77	7.00	7.84	9.82	10.45	0.61	0.74	0.83	1.04	1.10
10	2.19	0.71	7.66	5.36	6.56	7.38	9.31	9.92	8.36	10.23	11.51	14.51	15.46
11	0.46	0.32	6.81	5.53	6.74	7.57	9.52	10.14	0.82	1.00	1.12	1.41	1.50
12	0.80	0.38	12.86	4.55	5.67	6.43	8.22	8.78	1.37	1.70	1.93	2.47	2.64
13	0.30	0.34	7.65	5.37	6.56	7.39	9.31	9.92	0.55	0.67	0.76	0.95	1.02
14	3.64	0.27	15.30	4.26	5.34	6.07	7.80	8.34	4.18	5.24	5.96	7.67	8.19
15	18.04	0.26	62.87	1.98	2.64	3.09	4.17	4.49	9.40	12.55	14.67	19.81	21.32
16	7.30	0.22	11.58	4.73	5.86	6.64	8.46	9.03	7.62	9.45	10.71	13.64	14.56
17	5.04	0.23	32.42	2.97	3.84	4.43	5.85	6.28	3.41	4.42	5.10	6.72	7.21
18	1.96	0.26	21.57	3.66	4.65	5.33	6.93	7.42	1.89	2.40	2.75	3.57	3.83
19	0.90	0.26	20.36	3.76	4.77	5.46	7.08	7.58	0.89	1.12	1.29	1.67	1.79
20	4.57	0.21	50.79	2.27	3.00	3.50	4.69	5.04	2.22	2.94	3.42	4.58	4.93
21	1.65	0.26	16.41	4.14	5.20	5.92	7.63	8.16	1.75	2.19	2.50	3.22	3.44
22	3.10	0.28	8.24	5.26	6.45	7.26	9.17	9.77	4.51	5.53	6.23	7.87	8.39
26	0.98	0.34	27.21	3.26	4.19	4.82	6.32	6.77	1.09	1.40	1.61	2.11	2.26
28	0.02	0.35	2.62	6.54	7.81	8.69	10.75	11.42	0.05	0.06	0.07	0.08	0.09
29	0.03	0.35	2.77	6.49	7.76	8.64	10.70	11.36	0.06	0.07	0.08	0.10	0.10
30	1.70	0.36	7.17	5.46	6.66	7.49	9.43	10.04	3.35	4.09	4.59	5.78	6.16
31	3.07	0.34	17.41	4.04	5.08	5.80	7.48	8.00	4.22	5.31	6.06	7.82	8.36
32	2.33	0.52	7.84	5.33	6.53	7.35	9.27	9.87	6.48	7.93	8.93	11.26	12.00
34	0.14	0.31	9.51	5.04	6.21	7.01	8.88	9.47	0.22	0.27	0.31	0.39	0.42
35	8.30	0.23	27.10	3.27	4.20	4.83	6.33	6.78	6.19	7.94	9.14	11.97	12.84
38	0.06	0.24	5.66	5.77	7.00	7.85	9.82	10.45	0.08	0.10	0.11	0.14	0.14
39	1.25	0.41	14.00	4.41	5.51	6.26	8.02	8.57	2.27	2.83	3.22	4.12	4.41
40	1.06	0.42	6.43	5.61	6.82	7.66	9.62	10.24	2.51	3.06	3.43	4.31	4.59
42	2.84	0.41	8.90	5.14	6.32	7.13	9.02	9.61	5.99	7.36	8.30	10.50	11.20
44	6.61	0.45	36.96	2.76	3.59	4.15	5.50	5.91	8.10	10.55	12.21	16.18	17.37
47	1.12	0.26	11.73	4.71	5.84	6.61	8.43	9.00	1.35	1.68	1.90	2.42	2.59
48	0.05	0.24	2.28	6.63	7.91	8.80	10.86	11.53	0.08	0.10	0.11	0.14	0.15
49	0.10	0.49	3.19	6.38	7.64	8.52	10.56	11.22	0.31	0.37	0.42	0.52	0.55
51	1.09	0.36	6.70	5.55	6.76	7.60	9.55	10.17	2.18	2.65	2.98	3.74	3.98
53	0.89	0.58	9.93	4.97	6.13	6.93	8.79	9.38	2.57	3.16	3.57	4.53	4.84
54	0.15	0.44	8.18	5.27	6.46	7.27	9.18	9.79	0.34	0.42	0.47	0.60	0.63
55	1.41	0.51	15.18	4.27	5.35	6.09	7.82	8.36	3.07	3.84	4.37	5.62	6.01
57	0.54	0.50	9.30	5.08	6.25	7.05	8.93	9.52	1.38	1.70	1.92	2.43	2.59
59	2.42	0.75	12.56	4.59	5.71	6.48	8.28	8.84	8.28	10.29	11.67	14.91	15.92
60	1.30	0.35	12.50	4.60	5.72	6.49	8.29	8.85	2.12	2.64	3.00	3.83	4.09
61	0.24	0.40	6.35	5.62	6.84	7.68	9.64	10.26	0.53	0.64	0.72	0.90	0.96
62	2.96	0.63	10.14	4.94	6.10	6.89	8.75	9.33	9.27	11.44	12.92	16.41	17.51
63	1.13	0.23	36.86	2.76	3.59	4.16	5.51	5.92	0.71	0.93	1.07	1.42	1.52
65	2.51	0.41	10.09	4.95	6.11	6.90	8.76	9.34	5.10	6.29	7.11	9.02	9.62
66	1.01	0.53	8.23	5.26	6.45	7.26	9.17	9.78	2.81	3.45	3.88	4.90	5.22
67	0.03	0.23	1.74	6.80	8.08	8.98	11.06	11.73	0.05	0.05	0.06	0.07	0.08
68	1.05	0.49	6.18	5.66	6.88	7.72	9.68	10.31	2.95	3.58	4.02	5.04	5.37
70	0.03	0.54	2.26	6.64	7.92	8.81	10.87	11.54	0.10	0.12	0.13	0.16	0.17
72	0.11	0.35	4.71	5.99	7.23	8.09	10.09	10.73	0.23	0.27	0.31	0.38	0.41
75	1.27	0.28	45.05	2.45	3.22	3.74	4.99	5.36	0.86	1.13	1.31	1.75	1.89
77	2.81	0.33	9.16	5.10	6.27	7.08	8.96	9.55	4.67	5.74	6.48	8.20	8.74
78	0.67	0.70	6.55	5.58	6.80	7.63	9.59	10.21	2.62	3.19	3.58	4.50	4.79
80	0.07	0.47	3.77	6.22	7.48	8.35	10.38	11.03	0.21	0.26	0.29	0.36	0.38
81	0.13	0.58	5.47	5.81	7.04	7.89	9.88	10.51	0.43	0.52	0.59	0.73	0.78
83	1.44	0.72	8.57	5.20	6.38	7.20	9.09	9.69	5.39	6.61	7.45	9.42	10.04
84	0.05	0.54	2.78	6.49	7.76	8.64	10.69	11.36	0.18	0.21	0.23	0.29	0.31
85	0.09	0.56	5.86	5.73	6.95	7.80	9.77	10.40	0.28	0.34	0.38	0.48	0.51
86	2.00	0.61	8.75	5.17	6.35	7.16	9.05	9.65	6.33	7.78	8.77	11.09	11.82
87	0.16	0.75	4.28	6.09	7.34	8.21	10.22	10.87	0.75	0.90	1.00	1.25	1.33
95	2.67	0.39	7.31	5.43	6.63	7.46	9.39	10.01	5.63	6.88	7.74	9.74	10.38
97	8.36	0.23	32.05	2.99	3.86	4.46	5.88	6.31	5.69	7.37	8.50	11.21	12.02
98	6.09	0.26	33.07	2.94	3.80	4.39	5.80	6.22	4.70	6.09	7.04	9.29	9.97
99	1.96	0.52	12.77	4.57	5.68	6.44	8.24	8.80	4.61	5.74	6.51	8.32	8.88
100	0.56	0.47	11.75	4.70	5.84	6.61	8.43	9.00	1.24	1.54	1.74	2.22	2.37
101	0.79	0.88	9.78	5.00	6.16	6.96	8.83	9.41	3.47	4.27	4.83	6.12	6.53
102	0.46	0.81	8.94	5.14	6.31	7.12	9.01	9.61	1.89	2.33	2.62	3.32	3.54
103	0.53	0.44	9.62	5.02	6.19	6.99	8.86	9.45	1.17	1.45	1.63	2.07	2.21
104	0.70	0.47	5.53	5.80	7.03	7.88	9.86	10.49	1.89	2.29	2.56	3.21	3.42
107	3.72	0.45	7.37	5.42	6.62	7.45	9.38	9.99	8.97	10.96	12.33	15.52	16.54
109	0.55	0.40	11.07	4.80	5.94	6.72	8.56	9.13	1.07	1.32	1.50	1.91	2.03
111	3.22	0.31	11.07	4.80	5.94	6.73	8.56	9.13	4.72	5.84	6.61	8.41	8.97
115	2.03	0.32	7.54	5.39	6.59	7.41	9.34	9.95	3.49	4.27	4.80	6.05	6.44

Supplemental Table B
Watershed Data

Drainage Area	Area (acres)	C-value	Tc (min)	Rainfall Intensity (in/hr)					Runoff (cfs)				
				2-yr	5-yr	10-yr	25-yr	100-yr	2-yr	5-yr	10-yr	25-yr	100-yr
116	0.39	0.48	32.93	2.94	3.81	4.40	5.81	6.23	0.55	0.72	0.83	1.09	1.17
119	0.19	0.55	5.05	5.91	7.15	8.00	9.99	10.63	0.61	0.73	0.82	1.03	1.09
121	5.03	0.26	12.52	4.60	5.72	6.48	8.28	8.84	6.08	7.56	8.58	10.96	11.70
122	6.89	0.31	25.34	3.38	4.33	4.97	6.50	6.97	7.27	9.31	10.69	13.98	14.98
127	3.00	0.31	8.39	5.23	6.42	7.23	9.13	9.74	4.78	5.86	6.61	8.35	8.90
129	0.56	0.38	14.50	4.35	5.44	6.18	7.94	8.48	0.91	1.14	1.30	1.66	1.78
133	3.38	0.40	8.24	5.26	6.45	7.26	9.17	9.77	7.05	8.64	9.73	12.29	13.09
134	1.08	0.25	29.63	3.12	4.02	4.63	6.09	6.53	0.84	1.08	1.24	1.63	1.75
135	2.93	0.63	11.13	4.79	5.93	6.72	8.55	9.12	8.82	10.92	12.36	15.73	16.79
136	0.08	0.68	3.19	6.38	7.64	8.52	10.56	11.22	0.34	0.41	0.45	0.56	0.60
137	10.17	0.37	28.60	3.18	4.09	4.71	6.18	6.63	11.89	15.30	17.62	23.14	24.81
138	0.51	0.40	8.36	5.24	6.42	7.24	9.14	9.74	1.06	1.30	1.47	1.85	1.97
139	0.21	0.56	6.60	5.57	6.79	7.62	9.57	10.19	0.67	0.82	0.92	1.15	1.22
141	0.76	0.47	10.79	4.84	5.99	6.77	8.62	9.19	1.74	2.15	2.43	3.09	3.30
142	0.06	0.38	3.50	6.29	7.55	8.43	10.46	11.12	0.15	0.18	0.20	0.25	0.26
143	0.91	0.40	7.71	5.36	6.55	7.37	9.30	9.90	1.94	2.37	2.67	3.36	3.58
145	1.62	0.28	7.24	5.45	6.65	7.48	9.41	10.02	2.50	3.05	3.43	4.32	4.60
146	0.02	0.31	1.52	6.87	8.16	9.05	11.14	11.82	0.04	0.05	0.06	0.07	0.08
147	13.59	0.22	37.20	2.75	3.58	4.14	5.49	5.89	8.25	10.74	12.43	16.48	17.69
148	25.12	0.23	30.71	3.06	3.95	4.55	5.99	6.43	17.52	22.61	26.07	34.33	36.82
149	18.77	0.26	46.61	2.40	3.16	3.67	4.90	5.27	11.53	15.17	17.64	23.56	25.33
150	0.03	0.20	2.10	6.69	7.97	8.86	10.93	11.60	0.03	0.04	0.05	0.06	0.06
167	6.16	0.48	25.73	3.36	4.30	4.94	6.46	6.93	9.92	12.71	14.60	19.10	20.47
168	1.90	0.26	25.31	3.39	4.33	4.98	6.51	6.97	1.65	2.11	2.43	3.17	3.40
169	1.50	0.49	26.21	3.33	4.26	4.90	6.41	6.87	2.43	3.12	3.58	4.69	5.03
170	1.35	0.47	23.11	3.54	4.52	5.18	6.75	7.23	2.27	2.89	3.32	4.32	4.63
171	1.94	0.47	28.82	3.16	4.07	4.69	6.16	6.61	2.86	3.69	4.25	5.58	5.98
172	1.32	0.33	21.99	3.63	4.62	5.29	6.88	7.37	1.59	2.03	2.32	3.02	3.24
173	0.52	0.36	13.35	4.49	5.60	6.35	8.13	8.69	0.84	1.05	1.19	1.52	1.63
174	1.62	0.49	23.38	3.52	4.49	5.15	6.72	7.20	2.82	3.59	4.12	5.37	5.75
175	0.64	0.63	10.27	4.92	6.08	6.87	8.72	9.31	1.99	2.45	2.77	3.52	3.76
176	4.06	0.42	9.07	5.11	6.29	7.09	8.98	9.57	8.66	10.65	12.01	15.20	16.21
177	0.54	0.24	9.04	5.12	6.29	7.10	8.99	9.58	0.65	0.80	0.91	1.15	1.22
178	3.19	0.42	8.18	5.27	6.46	7.28	9.19	9.79	7.02	8.60	9.69	12.23	13.04
179	0.78	0.26	14.02	4.41	5.50	6.25	8.02	8.56	0.88	1.10	1.25	1.61	1.72
180	4.29	0.30	22.81	3.56	4.54	5.21	6.78	7.26	4.56	5.81	6.66	8.68	9.29
181	0.77	0.65	7.84	5.33	6.52	7.34	9.26	9.87	2.65	3.24	3.64	4.60	4.90
182	3.90	0.35	11.35	4.76	5.90	6.68	8.51	9.08	6.58	8.15	9.23	11.75	12.54
183	1.52	0.26	10.25	4.93	6.08	6.87	8.73	9.31	1.97	2.43	2.74	3.48	3.72
184	0.29	0.43	6.91	5.51	6.72	7.55	9.49	10.11	0.68	0.83	0.93	1.17	1.24
185	7.18	0.27	43.90	2.49	3.27	3.79	5.06	5.43	4.83	6.33	7.36	9.81	10.54
186	21.86	0.21	39.64	2.65	3.46	4.00	5.32	5.71	12.38	16.17	18.73	24.89	26.73
187	7.74	0.23	39.02	2.67	3.49	4.04	5.36	5.76	4.71	6.15	7.13	9.47	10.17

SUPPLEMENTAL TABLE C: XPSWMM Model Results

Supplemental Table C
XPSWMM Model Results

Link	10-year Storm Event			100-year Storm Event		
	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)
1	20.32	20.32	0.00	21.83	21.65	-0.18
2	19.66	19.66	0.00	20.78	20.74	-0.04
3	20.85	20.85	0.00	21.18	21.16	-0.02
4	19.45	19.45	0.00	21.19	21.17	-0.02
5	22.36	22.36	0.00	22.54	22.54	0.00
6	22.76	21.52	-1.25	23.42	22.90	-0.52
7	21.55	20.49	-1.06	22.51	22.60	0.10
8	18.86	18.86	0.00	21.11	21.02	-0.08
9	20.95	20.95	0.00	21.13	21.10	-0.03
10	22.05	22.05	0.00	22.25	22.19	-0.05
11	22.44	22.44	0.00	23.30	23.29	-0.01
12	21.62	21.62	0.00	22.28	22.23	-0.05
13	19.68	19.68	0.00	20.55	20.57	0.02
14	19.53	19.53	0.00	20.54	20.57	0.02
15	22.13	20.58	-1.55	23.21	22.64	-0.57
16	24.04	22.37	-1.68	24.44	23.07	-1.38
17	22.78	21.59	-1.19	23.43	22.91	-0.52
18	22.77	21.55	-1.21	23.42	22.91	-0.52
19	22.70	21.47	-1.23	23.42	22.88	-0.54
20	22.69	21.03	-1.66	23.42	22.87	-0.54
21	22.68	20.83	-1.85	23.40	22.87	-0.54
22	22.76	22.12	-0.65	23.42	22.91	-0.51
23	22.76	21.95	-0.81	23.42	22.91	-0.52
24	23.50	22.31	-1.20	23.64	22.91	-0.73
25	21.95	20.59	-1.36	22.68	22.62	-0.06
26	19.37	20.19	0.82	22.38	22.53	0.15
27	17.66	19.12	1.46	22.11	22.03	-0.07
28	19.15	19.15	0.00	22.11	22.03	-0.07
29	18.96	19.12	0.17	22.11	22.03	-0.07
30	20.53	20.53	0.00	22.13	22.05	-0.08
31	19.30	19.30	0.00	22.12	22.04	-0.08
32	21.39	21.39	0.00	21.84	21.76	-0.08
33	21.05	21.05	0.00	21.83	21.66	-0.17
34	22.52	21.32	-1.20	23.22	22.68	-0.54
35	22.51	20.85	-1.66	23.22	22.66	-0.56
36	22.17	20.74	-1.44	23.21	22.65	-0.57
37	21.57	20.54	-1.03	22.51	22.60	0.09
38	21.55	20.49	-1.06	22.51	22.60	0.10
39	22.52	21.32	-1.20	23.22	22.68	-0.54
40	22.52	21.40	-1.12	23.22	22.68	-0.54
41	22.82	22.33	-0.49	23.62	23.33	-0.29
42	22.82	22.44	-0.38	23.62	23.33	-0.29
43	21.75	21.75	0.00	22.66	22.66	0.00
44	21.75	21.75	0.00	22.66	22.66	0.00

Supplemental Table C
XPSWMM Model Results

Link	10-year Storm Event			100-year Storm Event		
	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)
45	21.75	21.75	0.00	22.66	22.66	0.00
46	19.33	19.33	0.00	20.46	20.48	0.02
47	18.05	18.05	0.00	20.43	20.45	0.02
48	15.88	15.88	0.00	20.54	20.56	0.01
49	19.01	19.01	0.00	21.81	21.64	-0.17
50	21.33	21.33	0.00	22.04	22.05	0.00
51	21.39	21.39	0.00	22.22	22.22	0.00
52	20.72	20.72	0.00	21.15	21.15	0.01
53	21.32	21.32	0.00	22.04	22.04	0.00
54	21.32	21.32	0.00	22.04	22.04	0.00
55	20.40	20.40	0.00	21.04	21.05	0.01
56	20.26	20.26	0.00	20.89	20.90	0.02
57	20.20	20.20	0.00	20.87	20.88	0.02
58	19.87	19.87	0.00	20.57	20.60	0.02
59	21.10	21.10	0.00	21.37	21.37	0.00
60	21.63	21.63	0.00	22.28	22.23	-0.05
61	21.62	21.62	0.00	22.28	22.23	-0.05
62	21.62	21.62	0.00	22.28	22.23	-0.05
63	19.69	19.69	0.00	21.87	21.75	-0.12
64	21.62	21.62	0.00	22.28	22.23	-0.05
65	22.44	22.44	0.00	23.30	23.29	-0.01
66	21.53	21.53	0.00	22.25	22.19	-0.05
67	18.38	18.41	0.03	21.84	21.75	-0.08
68	21.29	21.29	0.00	21.47	21.47	0.00
69	21.13	21.13	0.00	21.41	21.41	0.00
70	21.12	21.12	0.00	21.41	21.41	0.00
71	21.10	21.10	0.00	21.37	21.37	0.00
72	20.71	20.71	0.00	21.10	21.07	-0.03
73	20.85	20.85	0.00	21.12	21.09	-0.03
74	21.34	21.34	0.00	21.69	21.69	0.00
75	21.12	21.12	0.00	21.41	21.41	0.00
76	18.59	18.59	0.00	21.02	20.99	-0.03
77	17.71	17.71	0.00	21.02	20.99	-0.03
78	21.17	21.17	0.00	21.26	21.26	0.00
79	20.56	20.56	0.00	21.19	21.17	-0.02
80	19.23	19.23	0.00	21.19	21.17	-0.02
81	20.42	20.42	0.00	20.78	20.75	-0.04
82	19.62	19.62	0.00	20.77	20.73	-0.04
83	18.54	18.54	0.00	20.77	20.73	-0.04
84	20.63	20.63	0.00	20.69	20.69	0.00
85	20.25	20.25	0.00	20.31	20.31	0.00
86	19.51	19.51	0.00	20.50	20.50	0.00
87	19.46	19.46	0.00	20.50	20.50	0.00
88	19.46	19.46	0.00	20.53	20.53	0.00

Supplemental Table C
XPSWMM Model Results

Link	10-year Storm Event			100-year Storm Event		
	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)
89	19.46	19.46	0.00	20.53	20.53	0.00
90	19.46	19.46	0.00	20.52	20.52	0.01
91	19.46	19.46	0.00	20.52	20.52	0.00
92	19.46	19.46	0.00	20.52	20.52	0.00
93	19.46	19.46	0.00	20.52	20.52	0.01
94	19.56	19.56	0.00	20.52	20.52	0.00
95	19.20	19.20	0.00	20.52	20.53	0.00
96	21.67	21.15	-0.52	22.32	21.83	-0.49
97	21.66	21.48	-0.18	22.38	22.15	-0.24
98	22.99	22.41	-0.57	23.52	22.91	-0.61
99	22.22	22.25	0.03	22.63	22.75	0.11
100	21.12	19.74	-1.39	21.65	21.45	-0.20
101	22.27	22.29	0.02	22.64	22.75	0.11
102	22.22	22.25	0.04	22.63	22.75	0.11
103	22.19	22.24	0.05	22.63	22.75	0.12
104	22.99	22.54	-0.45	23.53	22.92	-0.61
105	22.99	22.47	-0.51	23.52	22.92	-0.61
106	22.14	22.24	0.10	22.64	22.79	0.15
107	22.21	22.07	-0.14	23.14	22.90	-0.24
108	22.17	21.98	-0.19	23.06	22.72	-0.34
109	22.17	21.98	-0.19	23.06	22.72	-0.34
110	21.89	21.85	-0.04	22.55	22.45	-0.11
111	21.89	21.85	-0.04	22.55	22.44	-0.11
112	22.27	22.29	0.02	22.64	22.75	0.11
113	22.22	22.25	0.03	22.63	22.75	0.11
114	22.19	22.24	0.05	22.63	22.75	0.12
115	21.91	21.94	0.03	22.37	22.42	0.04
116	22.09	22.17	0.08	22.60	22.73	0.13
117	21.91	21.94	0.03	22.37	22.42	0.04
118	21.07	19.36	-1.71	21.65	21.42	-0.22
119	21.07	19.27	-1.80	21.74	21.46	-0.28
120	22.11	22.19	0.08	22.61	22.74	0.13
121	22.11	22.20	0.09	22.62	22.76	0.14
122	21.36	21.18	-0.19	21.82	21.79	-0.03
123	21.36	21.18	-0.19	21.82	21.79	-0.02
124	21.36	21.18	-0.19	21.82	21.79	-0.03
125	21.36	21.17	-0.19	21.82	21.79	-0.03
126	21.36	21.18	-0.19	21.82	21.79	-0.03
127	21.17	20.43	-0.74	21.67	21.52	-0.15
128	21.17	20.37	-0.80	21.67	21.52	-0.15
129	21.17	20.37	-0.80	21.67	21.52	-0.15
130	21.62	21.41	-0.21	22.28	21.98	-0.30
131	21.89	21.85	-0.04	22.55	22.44	-0.11
132	21.89	21.85	-0.04	22.55	22.45	-0.11

Supplemental Table C
XPSWMM Model Results

Link	10-year Storm Event			100-year Storm Event		
	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)
133	21.99	21.98	-0.02	22.85	22.80	-0.05
134	21.53	21.18	-0.35	22.20	21.81	-0.40
135	22.67	21.79	-0.88	23.30	22.12	-1.18
136	20.58	20.83	0.25	20.81	21.22	0.41
137	19.95	19.95	0.00	21.61	21.39	-0.22
138	18.25	18.25	0.00	21.60	21.38	-0.22
139	19.95	19.95	0.00	21.61	21.39	-0.22
140	19.95	19.95	0.00	21.63	21.39	-0.24
141	20.24	20.24	0.00	21.59	21.37	-0.22
142	18.01	18.01	0.00	20.76	20.72	-0.04
143	18.56	18.56	0.00	20.76	20.72	-0.04
144	20.46	20.70	0.23	20.76	21.05	0.29
145	17.51	19.13	1.62	22.08	21.99	-0.09
146	20.04	20.04	0.00	21.58	21.37	-0.20
147	21.67	21.10	-0.57	22.32	21.83	-0.48
148	22.69	21.03	-1.66	23.42	22.87	-0.54
149	23.42	23.42	0.00	23.60	23.59	-0.01
150	18.41	18.41	0.00	21.09	21.01	-0.08
151	17.51	19.13	1.62	22.08	21.99	-0.09
152	17.02	17.71	0.70	21.78	21.61	-0.16
153	15.71	16.07	0.36	21.33	21.22	-0.11
154	16.02	16.45	0.44	21.42	21.29	-0.12
155	15.51	15.51	0.00	21.07	20.99	-0.08
156	15.34	15.34	0.00	21.07	20.99	-0.08
157	18.46	18.46	0.00	20.76	20.72	-0.04
158	12.67	13.28	0.60	20.76	20.72	-0.04
159	17.89	17.89	0.00	20.76	20.72	-0.04
160	18.97	18.97	0.00	20.49	20.49	0.00
161	19.34	19.34	0.00	20.49	20.49	0.00
162	11.96	12.23	0.27	20.28	20.31	0.03
163	20.16	20.16	0.00	20.28	20.31	0.03
164	20.16	20.16	0.00	20.28	20.31	0.03
165	17.67	17.67	0.00	20.17	20.19	0.02
166	17.01	16.79	-0.22	20.17	20.19	0.02
167	16.02	16.45	0.44	21.42	21.29	-0.12
168	11.76	11.99	0.23	20.17	20.19	0.02
169	11.96	12.23	0.27	20.28	20.31	0.03
170	12.27	12.59	0.32	20.49	20.49	0.00
171	13.34	13.54	0.20	20.76	20.72	-0.04
172	14.92	15.20	0.29	21.07	20.99	-0.08
173	15.71	16.07	0.36	21.33	21.22	-0.11
174	17.02	17.71	0.70	21.78	21.61	-0.16
176	22.82	22.33	-0.49	23.62	23.33	-0.29
177	22.76	21.52	-1.25	23.42	22.90	-0.52

Supplemental Table C
XPSWMM Model Results

Link	10-year Storm Event			100-year Storm Event		
	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)	Maximum WSE Existing	Maximum WSE Proposed	Difference (Pr. - Ex.)
178	22.44	22.44	0.00	23.30	23.29	-0.01
179	21.53	21.53	0.00	22.25	22.19	-0.05
180	21.50	21.50	0.00	22.24	22.19	-0.05
181	21.31	21.31	0.00	22.04	22.04	0.00
182	22.17	21.98	-0.19	23.06	22.72	-0.34
183	22.09	22.17	0.08	22.60	22.73	0.13
184	22.19	22.24	0.05	22.63	22.75	0.12
185	23.17	23.15	-0.01	23.46	23.31	-0.14
186	22.77	21.55	-1.21	23.42	22.91	-0.52
187	22.79	22.47	-0.32	23.43	22.95	-0.47

SUPPLEMENTAL TABLE D:

Water Surface vs. Pavement Elevations

Supplemental Table D
Water Surface vs. Pavement Elevations

Link	Culvert Diameter (ft)	10-year Storm Event Maximum WSE Existing	10-year Storm Event Maximum WSE Proposed	100-year Storm Event Maximum WSE Existing	100-year Storm Event Maximum WSE Proposed	Nearest Edge of Pavement (EOP) Elevation	Nearest Pavement Centerline Elevation	Difference b/w EOP and 10-year Maximum WSE (Existing)	Difference b/w EOP and 10-year Maximum WSE (Proposed)
5	1.25	22.36	22.36	22.54	22.54	23.0	23.5	0.64	0.64
10	1.25	22.05	22.05	22.25	22.19	23.0	23.5	0.95	0.95
23	1.25	22.76	21.95	23.42	22.91	23.5	23.5	0.74	1.55
24	1.00	23.50	22.31	23.64	22.91	23.0	23.0	-0.50	0.70
25	2.00	21.95	20.59	22.68	22.62	23.0	23.0	1.05	2.41
27	2.00	17.66	19.12	22.11	22.03	20.0	20.0	2.34	0.88
28	2.00	19.15	19.15	22.11	22.03	22.0	22.0	2.85	2.85
30	1.50	20.53	20.53	22.13	22.05	22.2	22.2	1.67	1.67
33	2.00	21.05	21.05	21.83	21.66	24.0	24.0	2.95	2.95
34	1.50	22.52	21.32	23.22	22.68	22.0	22.5	-0.52	0.68
36	1.50	22.17	20.74	23.21	22.65	-	-	-	-
37	1.50	21.57	20.54	22.51	22.60	23.0	23.0	1.43	2.46
40	1.25	22.52	21.40	23.22	22.68	23.0	23.0	0.48	1.60
41	1.50	22.82	22.33	23.62	23.33	22.5	22.5	-0.32	0.17
43	1.50	21.75	21.75	22.66	22.66	23.0	23.0	1.25	1.25
45	1.25	21.75	21.75	22.66	22.66	22.0	22.0	0.25	0.25
46	2.00	19.33	19.33	20.46	20.48	20.0	20.0	0.67	0.67
48	3.50	15.88	15.88	20.54	20.56	-	-	-	-
49	2.00	19.01	19.01	21.81	21.64	-	-	-	-
50	1.25	21.33	21.33	22.04	22.05	23.5	23.5	2.17	2.17
52	1.50	20.72	20.72	21.15	21.15	22.5	22.8	1.79	1.79
54	1.50	21.32	21.32	22.04	22.04	23.0	23.0	1.68	1.68
56	2.00	20.26	20.26	20.89	20.90	22.0	22.0	1.74	1.74
58	2.00	19.87	19.87	20.57	20.60	22.0	22.0	2.13	2.13
60	1.25	21.63	21.63	22.28	22.23	23.0	23.2	1.37	1.37
62	1.25	21.62	21.62	22.28	22.23	23.6	23.7	1.98	1.98
64	1.25	21.62	21.62	22.28	22.23	23.0	23.5	1.38	1.38
65	1.25	22.44	22.44	23.30	23.29	22.5	23.0	0.06	0.06
67	3.00	18.38	18.41	21.84	21.75	-	-	-	-
69	2.00	21.13	21.13	21.41	21.41	23.0	23.2	1.87	1.87
71	2.00	21.10	21.10	21.37	21.37	23.0	23.2	1.91	1.91
73	1.25	20.85	20.85	21.12	21.09	23.0	23.2	2.15	2.15
74	1.25	21.34	0.00	21.69	0.00	23.0	23.2	1.66	23.00
76	2.00	18.59	18.59	21.02	20.99	22.0	22.0	3.41	3.41
78	1.00	21.17	21.17	21.26	21.26	-	-	-	-
79	1.25	20.56	20.56	21.19	21.17	21.5	21.5	0.94	0.94
81	1.00	20.42	20.42	20.78	20.75	21.5	21.8	1.08	1.08
82	1.25	19.62	19.62	20.77	20.73	21.5	21.5	1.88	1.88
84	1.00	20.63	20.63	20.69	20.69	21.0	-	0.37	0.37
85	1.00	20.25	20.25	20.31	20.31	21.0	-	0.75	0.75
86	1.25	19.51	19.51	20.50	20.50	20.5	-	1.00	0.99
88	1.25	19.46	19.46	20.53	20.53	20.5	-	1.04	1.04
90	1.25	19.46	19.46	20.52	20.52	20.5	-	1.04	1.04
92	1.25	19.46	19.46	20.52	20.52	20.5	-	1.04	1.04
94	1.25	19.56	19.56	20.52	20.52	20.5	-	0.94	0.94
101	0.75	22.27	22.29	22.64	22.75	22.5	-	0.23	0.21
102	0.75	22.22	22.25	22.63	22.75	22.5	-	0.29	0.25
103	0.75	22.19	22.24	22.63	22.75	22.5	-	0.31	0.26
105	1.50	22.99	22.47	23.52	22.92	23.5	24.0	0.51	1.03
106	1.00	22.14	22.24	22.64	22.79	-	-	-	-
108	1.50	22.17	21.98	23.06	22.72	22.5	22.5	0.33	0.52
110	1.50	21.89	21.85	22.55	22.45	22.9	23.0	1.01	1.05
117	2.50	21.91	21.94	22.37	22.42	21.2	21.2	-0.71	-0.74
118	2.50	21.07	19.36	21.65	21.42	20.0	20.0	-1.07	0.64
120	3.00	22.11	22.19	22.61	22.74	21.5	21.5	-0.61	-0.69
123	2.00	21.36	21.18	21.82	21.79	21.5	21.5	0.14	0.32
125	1.25	21.36	21.17	21.82	21.79	-	-	-	-
127	1.25	21.17	20.43	21.67	21.52	21.5	21.5	0.33	1.07
130	2.50	21.62	21.41	22.28	21.98	-	-	-	-
132	1.50	21.89	21.85	22.55	22.45	22.7	22.9	0.81	0.85
136	1.00	20.58	20.83	20.81	21.22	22.0	22.0	1.42	1.17
140	0.75	19.95	19.95	21.63	21.39	20.5	20.5	0.55	0.55
142	2.00	18.01	18.01	20.76	20.72	19.5	19.7	1.49	1.49
144	2.00	20.46	20.70	20.76	21.05	21.0	21.0	0.54	0.30
146	2.00	20.04	20.04	21.58	21.37	20.5	21.0	0.46	0.46
150	1.25	18.41	18.41	21.09	21.01	23.0	23.3	4.59	4.59
151	2.00	17.51	19.13	22.08	21.99	20.0	20.5	2.49	0.87
152	2.00	17.02	17.71	21.78	21.61	21.2	21.4	4.19	3.49
153	3.00	15.71	16.07	21.33	21.22	21.4	21.6	5.69	5.33
154	2.00	16.02	16.45	21.42	21.29	21.2	21.4	5.18	4.75
155	2.00	15.51	15.51	21.07	20.99	21.4	21.6	5.89	5.89
156	2.00	15.34	15.34	21.07	20.99	21.4	21.6	6.06	6.06
157	2.00	18.46	18.46	20.76	20.72	20.0	20.5	1.54	1.54
158	2.00	12.67	13.28	20.76	20.72	20.0	20.5	7.33	6.73

Supplemental Table D
Water Surface vs. Pavement Elevations

Link	Culvert Diameter (ft)	10-year Storm Event		100-year Storm Event		Nearest Edge of Pavement (EOP) Elevation	Nearest Pavement Centerline Elevation	Difference b/w EOP and 10-year Maximum WSE (Existing)	Difference b/w EOP and 10-year Maximum WSE (Proposed)
		Maximum WSE Existing	Maximum WSE Proposed	Maximum WSE Existing	Maximum WSE Proposed				
159	2.00	17.89	17.89	20.76	20.72	20.0	20.5	2.11	2.11
160	2.00	18.97	18.97	20.49	20.49	20.5	-	1.53	1.53
161	2.00	19.34	19.34	20.49	20.49	20.5	-	1.16	1.16
162	2.00	11.96	12.23	20.28	20.31	19.0	19.5	7.04	6.77
163	2.00	20.16	20.16	20.28	20.31	20.3	19.5	0.14	0.14
164	2.00	20.16	20.16	20.28	20.31	20.3	19.5	0.14	0.14
165	2.00	17.67	17.67	20.17	20.19	19.6	20.0	1.93	1.93
166	2.00	17.01	16.79	20.17	20.19	19.0	19.0	2.00	2.22
167	4.50	16.02	16.45	21.42	21.29	20.0	21.0	3.98	3.55
171	6.00	13.34	13.54	20.76	20.72	20.0	21.0	6.66	6.46
172	5.50	14.92	15.20	21.07	20.99	20.0	21.0	5.09	4.80
173	5.00	15.71	16.07	21.33	21.22	20.0	21.0	4.29	3.93
174	4.00	17.02	17.71	21.78	21.61	20.0	21.0	2.99	2.29

SUPPLEMENTAL TABLE E

Tabulation of Probable Construction Costs

La Porte Municipal Airport

1712LPRTE - Airfield Pavement Rehabilitation and Drainage Improvements - 2017

Supp Table E: Estimate of Probable Construction Costs

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
Runway 12/30 and Connectors A1, A2, A3, A4 to holdline					**(4165' X 75')**	
C105-1	Mobilization	LS	1	$ 17,000.00	$ 17,000	
P101-1	Preparation/Removal of Existing Pavements	SY	500	$ 12.50	$ 6,300	
P101-2	Crack Sealing	LF	7,700	$ 3.50	$ 27,000	
P403-1	2" Bituminous Pavement Surfacing	TON	50	$ 300.00	$ 15,000	
P620-1	Marking Removal and Surface Prep	SF	50,200	$ 1.00	$ 50,200	
P620-2	Retro-reflective Paint Markings	SF	27,950	$ 2.50	$ 69,900	
P620-3	Black Outline Paint Markings	SF	8,280	$ 1.50	$ 12,400	
P63X-1	Surface Treatment	SY	40,200	$ 3.00	$ 120,600	
T901-1	Seeding	Ac	3.3	$ 1,500.00	$ 5,000	
T904-1	Sodding	SY	3,150	$ 8.00	$ 25,200	
SS151-1	Edge Grading	LF	9,440	$ 1.50	$ 14,200	
	20% Contingency	Ea	1	$ 72,600.00	$ 72,600	
	Runway 12/30 and Connectors A1, A2, A3, A4 to holdline Costs Total				**$ 435,400**	**Program = $374000**
Taxiway A East and 12 Runup Pad					**(2100' X 40')**	
C105-1	Mobilization	LS	1	$ 5,000.00	$ 5,000	
P101-1	Preparation/Removal of Existing Pavements	SY	600	$ 12.00	$ 7,200	
P101-2	Crack Sealing	LF	3,500	$ 3.50	$ 12,300	
P403-1	2" Bituminous Pavement Surfacing	TON	70	$ 350.00	$ 24,500	
P620-2	Retro-reflective Paint Markings	SF	2,360	$ 2.00	$ 4,700	
P63X-1	Surface Treatment	SY	10,000	$ 3.00	$ 30,000	
T901-1	Seeding	Ac	1.4	$ 1,500.00	$ 2,100	
T904-1	Sodding	SY	1,340	$ 8.00	$ 10,700	
SS151-1	Edge Grading	LF	4,010	$ 2.00	$ 8,000	
	20% Contingency	Ea	1	$ 20,900.00	$ 20,900	
	Taxiway A East and 12 Runup Pad Costs Total				**$ 125,400**	**Program = $264000**

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
Taxiway A West Reconstruction					**(1850' X 40')**	
~~C105-1~~	~~Mobilization~~	~~LS~~	~~1~~	~~$ 31,000.00~~	~~$ 31,000~~	
~~P101-1~~	~~Preparation/Removal of Existing Pavements~~	~~SY~~	~~8,600~~	~~$ 5.00~~	~~$ 43,000~~	
~~P403-2~~	~~4" Bituminous Pavement Surfacing~~	~~TON~~	~~1,920~~	~~$ 250.00~~	~~$ 480,000~~	
~~P620-2~~	~~Retro-reflective Paint Markings~~	~~SF~~	~~1,170~~	~~$ 2.50~~	~~$ 2,900~~	
~~P63X-1~~	~~Surface Treatment~~	~~SY~~	~~8,600~~	~~$ 3.50~~	~~$ 30,100~~	
~~T901-1~~	~~Seeding~~	~~Ac~~	~~1.4~~	~~$ 1,500.00~~	~~$ 2,100~~	
~~T904-1~~	~~Sodding~~	~~SY~~	~~1,270~~	~~$ 8.00~~	~~$ 10,200~~	
~~SS475-1~~	~~Recycling of Existing Base~~	~~SY~~	~~9,400~~	~~$ 5.50~~	~~$ 51,700~~	
	~~20% Contingency~~	~~Ea~~	~~1~~	~~$ 130,200.00~~	~~$ 130,200~~	
	Taxiway A West Reconstruction Costs Total				**$ 781,200.00**	**Program = $0**
NW Hangar Access and Aprons					**(11,500 SY)**	
C105-1	Mobilization	LS	1	$ 3,600.00	$ 3,600	
P101-1	Preparation/Removal of Existing Pavemer	SY	3	$ 200.00	$ 600	
P101-2	Crack Sealing	LF	3,100	$ 3.50	$ 10,900	
P403-1	2" Bituminous Pavement Surfacing	TON	10	$ 400.00	$ 4,000	
P620-2	Retro-reflective Paint Markings	SF	580	$ 2.50	$ 1,500	
P63X-1	Surface Treatment	SY	11,900	$ 3.50	$ 41,700	
T901-1	Seeding	Ac	0.9	$ 1,500.00	$ 1,400	
T904-1	Sodding	SY	840	$ 8.00	$ 6,700	
SS151-1	Edge Grading	LF	2,510	$ 2.00	$ 5,000	
	20% Contingency	Ea	1	$ 15,100.00	$ 15,100	
	Taxiway A West Reconstruction Costs Total				**$ 90,500**	**Program = $99000**
Taxiway D					**(140' X 40')**	
C105-1	Mobilization	LS	1	$ 500.00	$ 500	
P620-2	Retro-reflective Paint Markings	SF	70	$ 2.50	$ 200	
P63X-1	Surface Treatment	SY	600	$ 4.00	$ 2,400	
T901-1	Seeding	Ac	0.1	$ 1,500.00	$ 200	
T904-1	Sodding	SY	90	$ 8.00	$ 700	
SS151-1	Edge Grading	LF	270	$ 2.00	$ 500	
	20% Contingency	Ea	1	$ 900.00	$ 900	
	Taxiway D Costs Total				**$ 5,400**	**Program = $5500**

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
Runway 5/23 and Connectors B1, B2 to holdline					**(2998' X 75')**	
C105-1	Mobilization	LS	1	$ 12,000.00	$ 12,000	
P101-1	Preparation/Removal of Existing Pavements	SY	550	$ 100.00	$ 55,000	
P101-2	Crack Sealing	LF	3,000	$ 3.50	$ 10,500	
P403-1	2" Bituminous Pavement Surfacing	TON	70	$ 400.00	$ 28,000	
P620-1	Marking Removal and Surface Prep	SF	5,130	$ 1.00	$ 5,100	
P620-2	Retro-reflective Paint Markings	SF	6,870	$ 2.50	$ 17,200	
P620-3	Black Outline Paint Markings	SF	3,610	$ 2.00	$ 7,200	
P63X-1	Surface Treatment	SY	29,100	$ 3.00	$ 87,300	
T901-1	Seeding	Ac	2.0	$ 1,500.00	$ 3,000	
T904-1	Sodding	SY	1,890	$ 8.00	$ 15,100	
SS151-1	Edge Grading	LF	5,670	$ 1.50	$ 8,500	
	20% Contingency	Ea	1	$ 49,800.00	$ 49,800	
	Runway 5/23 and Connectors B1, B2 to holdline Costs				**$ 298,700**	**Program = $220000**
Taxiway B					**(3280' X 40')**	
C105-1	Mobilization	LS	1	$ 10,000.00	$ 10,000	
P101-1	Preparation/Removal of Existing Pavements	SY	1,880	$ 12.00	$ 22,600	
P101-2	Crack Sealing	LF	1,900	$ 3.50	$ 6,700	
P403-1	2" Bituminous Pavement Surfacing	TON	220	$ 350.00	$ 77,000	
P620-2	Retro-reflective Paint Markings	SF	1,800	$ 2.50	$ 4,500	
P63X-1	Surface Treatment	SY	14,900	$ 3.50	$ 52,200	
T901-1	Seeding	Ac	1.7	$ 1,500.00	$ 2,600	
T904-1	Sodding	SY	1,640	$ 8.00	$ 13,100	
SS151-1	Edge Grading	LF	4,900	$ 2.00	$ 9,800	
	20% Contingency	Ea	1	$ 39,700.00	$ 39,700	
	Taxiway B Costs Total				**$ 238,200**	**Program = $198000**
Taxiway C					**(1050' X 35')**	
C105-1	Mobilization	LS	1	$ 1,500.00	$ 1,500	
P101-2	Crack Sealing	LF	1,800	$ 3.50	$ 6,300	
P620-2	Retro-reflective Paint Markings	SF	570	$ 2.50	$ 1,400	
P63X-1	Surface Treatment	SY	3,800	$ 3.50	$ 13,300	

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
T901-1	Seeding	Ac	0.5	$ 1,500.00	$ 800	
T904-1	Sodding	SY	490	$ 8.00	$ 3,900	
SS151-1	Edge Grading	LF	1,450	$ 2.00	$ 2,900	
	20% Contingency	Ea	1	$ 6,100.00	$ 6,100	
	Taxiway C Costs Total				**$ 36,200**	**Program = $44000**

Apron 1						**(3280' X 40')**
C105-1	Mobilization	LS	1	$ 5,500.00	$ 5,500	
P101-1	Preparation/Removal of Existing Pavements	SY	110	$ 80.00	$ 8,800	
P101-2	Crack Sealing	LF	9,200	$ 3.50	$ 32,200	
P403-1	2" Bituminous Pavement Surfacing	TON	13	$ 400.00	$ 5,200	
P620-2	Retro-reflective Paint Markings	SF	1,060	$ 2.50	$ 2,700	
~~P629-1~~	~~Surface Treatment~~	~~SY~~	~~10,370~~	~~$ 4.00~~	$ -	
P629-2	Surface Treatment Pre-Lift	SY	520	$ 20.00	$ 10,400	
T901-1	Seeding	Ac	0.7	$ 1,500.00	$ 1,100	
T904-1	Sodding	SY	600	$ 8.00	$ 4,800	
SS151-1	Edge Grading	LF	1,800	$ 2.00	$ 3,600	
	20% Contingency	Ea	1	$ 14,900.00	$ 14,900	
	Apron 1 Costs Total				**$ 89,200**	**Program = $99000**

Apron 2						**(3280' X 40')**
C105-1	Mobilization	LS	1	$ 10,000.00	$ 10,000	
P101-2	Crack Sealing	LF	34,900	$ 3.50	$ 122,200	
P620-2	Retro-reflective Paint Markings	SF	1,460	$ 2.50	$ 3,700	
~~P629-1~~	~~Surface Treatment~~	~~SY~~	~~12,710~~	~~$ 4.00~~	$ -	
P629-2	Surface Treatment Pre-Lift	SY	640	$ 20.00	$ 12,800	
T901-1	Seeding	Ac	0.4	$ 1,500.00	$ 600	
T904-1	Sodding	SY	380	$ 8.00	$ 3,000	
SS151-1	Edge Grading	LF	1,140	$ 2.00	$ 2,300	
	20% Contingency	Ea	1	$ 31,000.00	$ 31,000	
	Apron 2 Costs Total				**$ 185,600**	**Program = $126500**

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
Apron 3					**(3280' X 40')**	
C105-1	Mobilization	LS	1	$ 10,000.00	$ 10,000	
P101-2	Crack Sealing	LF	29,300	$ 3.00	$ 87,900	
P620-2	Retro-reflective Paint Markings	SF	1,650	$ 2.50	$ 4,100	
~~P629-1~~	~~Surface Treatment~~	~~SY~~	~~18,300~~	~~$ 4.00~~	$ -	
P629-2	Surface Treatment Pre-Lift	SY	920	$ 20.00	$ 18,400	
T901-1	Seeding	Ac	1	$ 1,500.00	$ 1,200	
T904-1	Sodding	SY	700	$ 8.00	$ 5,600	
SS151-1	Edge Grading	LF	2,100	$ 2.00	$ 4,200	
	20% Contingency	Ea	1	$ 26,300.00	$ 26,300	
	Apron 3 Costs Total				**$ 157,700**	**Program = $181500**
Drainage Improvements					**()**	
C105-1	Mobilization	LS	1	$ 25,000.00	$ 25,000	
D701-1	18" Reinforced Concrete Pipe	LF	86	$ 80.00	$ 6,900	
D701-2	24" Reinforced Concrete Pipe	LF	165	$ 95.00	$ 15,700	
D701-3	30" Reinforced Concrete Pipe	LF	104	$ 105.00	$ 10,900	
D701-4	36" Reinforced Concrete Pipe	LF	229	$ 120.00	$ 27,500	
D701-5	42" Reinforced Concrete Pipe	LF	780	$ 150.00	$ 117,000	
P152-1	Linear Detention and Swales Excavation	CY	7,000	$ 8.00	$ 56,000	
P101-1	Demolition of Taxiway	SY	120	$ 23.00	$ 2,800	
P403-2	Replacement of Taxiway	SY	120	$ 125.00	$ 15,000	
P610-1	5" thick -12 ft Wide Concrete Lined Channel Bottom	SY	325	$ 50.00	$ 16,300	
	20% Contingency	Ea	1	$ 58,700.00	$ 58,700	
	Drainage Improvements Costs Total				**$ 351,800**	**Program = $330000**
	ESTIMATED CONSTRUCTION TOTAL				**$2,014,100**	**Program = $1941500**

Item	Description	Units	Estimated Quantity	Estimated Cost per Unit	Price	Notes
Construction Phase Services Costs						
	Construction Management	LS	1	$68,000.00	$ 68,000	
	Materials Testing	LS	1	$51,000.00	$ 51,000	
	Resident Project Representative	LS	1	$75,000.00	$ 75,000	
	Construction Phase Services Costs Costs Total				**$194,000**	**Program = $31000**
	ESTIMATED GRAND TOTAL				**$2,208,100.00**	

City of La Porte

Drainage Improvements Preliminary Engineering Report for 6th Street

Prepared by

RPS

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RPS Project No. 007586
February 2018

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PROFESSIONAL ENGINEER

3/2/2018

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Table 6: Cost Estimate for the Recommended Alternative

List of Exhibits

Attached:

Exhibit 1: Vicinity Map

Exhibit 2: Topography Map (2008 LiDAR)

Exhibit 3: Existing Model System Components

Exhibit 4: 5 Year Existing Inundation Map

Exhibit 5: 10 Year Existing Inundation Map

Exhibit 6: 100 Year Existing Inundation Map

Exhibit 7: Proposed Alt #5 Model System Components

Exhibit 8: 5 Year Proposed Inundation Map

Exhibit 9: 10 Year Alt #5 Inundation Map

Exhibit 10: 100 Year Alt #5 Inundation Map

Exhibit 11: 5 Yr WSEL Reduction Alt #5

Exhibit 12: 10 Yr WSEL Reduction Alt #5

Exhibit 13: 100 Yr WSEL Reduction Alt #5

Exhibit 14: Cost Estimate

1 INTRODUCTION

1.1 Authorization

The development of this Preliminary Engineering Report (PER) was authorized by agreement between the City of La Porte (City) and RPS dated May 15, 2017. The goal of this analysis is to evaluate the drainage system along 6th St from W. Main St to W. Madison St along with the overall effects within the F216 Watershed.

1.2 Understanding of the Project

RPS completed a City-Wide Drainage Study in 2008 that identified key drainage and flooding issues affecting the City. Tropical Storm Allison and Tropical Storm Erin created major damage to structures and homes throughout La Porte. The City-Wide Drainage Study analyzed each watershed, including repetitive loss data, to provide recommendations for short term and long term drainage improvements. The Study also included recommendations for standards or drainage criteria and a prioritization of recommended improvements throughout the City. The segment of 6th St from W. Madison St to W. Main St is considered part of Old La Porte, which was generally noted in the City-Wide Drainage Study as not having sufficient storm sewer capacity due to undersized storm sewer, undersized storm inlets, or not enough storm inlets.

1.3 Study Purpose and Intent

The goal of this analysis is to evaluate the existing storm sewer system, identify problem areas within the project limits, and provide recommendations for improvements on 6th St and surrounding areas. RPS will also determine if there any negative impacts downstream of the 6th St storm sewer system. Overall, this analysis will evaluate this area in much more detail than the City-Wide Drainage Study to help the City of La Porte fully understand the interactions between the storm sewer on 6th St and the effects on the overall watershed.

1.4 Prior Studies and Data Sources

New data were gathered from a variety of sources including earlier studies and reports, construction plans, field reconnaissance, and from discussions and correspondence with City staff, consultants, and local residents. The following data, previous plans and studies were obtained and reviewed for this study:

- The City-Wide Drainage Study, RPS, 2008.
- City of La Porte Utility Map Website and Shapefiles, City of La Porte.
- Effective HEC-HMS and HEC-RAS flood mapping models, FEMA.
- Topographic Survey Data, Landpoint, LLC, (2017).
- 2008 LiDAR (for sheetflow determination) (NAVD 88, 2001 Adjustment).
- Aerial Photographs (2017).

2 EVALUATION OF EXISTING CONDITIONS

2.1 Location and Topography

2.1.1 Location

The project area is located within the City limits of La Porte, Texas. It is located east of TX-146, along 6th St bound by W. Madison St on north and W. Main St on south (see Exhibit 1: Vicinity Map).

2.1.2 Topography

The topography of the region is based on the LiDAR 2008 data. Exhibit 2 shows the topography of the area. The storm sewer data were acquired from the City of La Porte Utility Map Website. Additionally, field survey was conducted in key points to verify the accuracy of the storm sewer data and the elevations of LiDAR data. The storm sewer systems outfall into open channels before discharging into Little Cedar Bayou (Harris County Flood Control District Unit F216-00-00). The cross sections of these open channels have also been surveyed.

2.1.3 Datum

The vertical datum used in this analysis, unless specifically noted otherwise, is the North American Vertical Datum of 1988 (NAVD88) with 2001 adjustment. Elevation contours prepared at a 1-foot contour interval are used from LiDAR data flown in 2008.

2.2 Modeling Background

FEMA HEC-HMS and HEC-RAS models are available for Little Cedar Bayou. These two models have been used as starting points for deriving the drainage area and the boundary conditions of the model.

To better find the sheetflow path of the region, a detailed 2D XP-SWMM model was developed using the LiDAR elevation along with the most updated survey data. The drainage areas were delineated using HEC-Geo HMS and the required parameters added as an input into the XP-SWMM model.

3 Hydrology and Hydraulics

3.1 Analysis

The drainage capacity of a system is defined to be the maximum amount of flow a channel or pipe can convey without adverse upstream or downstream impacts.

Several hydrologic and hydraulic analyses were conducted to determine:

1. Capacity: The needed hydraulic capacity of drainage networks (usually pipes, channels or ditches) for existing and proposed conditions (i.e., the channel carries the design flow without bank overflow).
2. Size: The size (i.e., dimensions) of a drainage network (usually pipes, channels or swales) necessary to pass or convey a design flow without adverse impacts.
3. Inlets: The size and location of the inlets to collect the run off and convey into the pipes or boxes.

3.2 Hydrologic Methodology

The drainage areas were delineated with HEC-Geo HMS using the LiDAR 2008 data. Then, a given set of parameters was identified for each drainage area including:

- acreage
- percent impervious
- slope
- width

Per HCFCD's recommendation in Section II.2.4 of H&H Manual, Green & Ampt infiltration methodology was employed for pervious area. Green & Ampt parameters of "Moisture Deficit, Capillary Suction, and Saturated Hydraulic Conductivity" were set according to the FEMA Effective HEC-HMS model for F216-00-00. Sub-catchment runoff routing was calculated utilizing the Runoff option in XP-SWMM.

The impervious area percentages for drainage areas were derived from the National Land Cover Database 2011 (NLCD2011) website. Then, these land uses were modified according to the 2017 aerial images.

3.3 Hydraulic Methodology

The model input data were received from the City of La Porte Utility shapefiles. They were confirmed by the field survey and were corrected accordingly. The 2008 LiDAR was obtained from the Houston-Galveston Area Council (HGAC) for the area, and was used to create the 2D surface. The cell size for the 2D model was initially set at 20' by 20' for all the preliminary models to reduce the modeling computational time. Final simulations were run using a finer grid at 10' by 10' to increase the model precision. All the results provided in this report are based on the finer grid.

Storm sewers were modeled using 1D components with 2D connections at inlets. The 2D surface and the subsurface storm network were able to interact between inlets and manholes, which are represented in the model as nodes. If the storm sewer capacity is exceeded, the water will enter the 2D surface at the node. Once in the 2D surface, XP-SWMM routes flows over the surface to simulate overland flow on the surface for the model.

The developed model has two outfall locations, as seen in Exhibit 3 – Existing Model System Components. The North outfall discharges into Little Cedar Bayou at the intersection of W. Madison St and N. 14th St through an open channel. The South outfall located approximately 500 feet west of TX-146 at W. D St. A 6'x6' RCB runs west under a parking lot and flows into an open channel before discharging into Little Cedar Bayou. Cross sections of all these open channels have been collected by the surveyors and added to the model.

Sensitivity models were run to determine how much the outfall conditions impact on the ponding in the project area (6th St from W. Madison St to W. Main St). Since the FEMA approved HEC-RAS model shows the water surface elevation (WSE) of Little Cedar Bayou for the 10- and 100-year storm events, the maximum WSE could be determined. However, it is very likely that the Little Cedar Bayou reaches its peak after the majority of the project areas have been drained. Therefore, the WSE of the bayou is likely lower than its maximum level when the rainfall is at its peak. The sensitivity models showed that while the areas adjacent to the bayou are highly impacted by the tailwater condition, the project area is nearly independent of the outfall conditions. In fact, due to the elevation of TX-146, the only interaction between west and east sides of the highway is through the cross-streets of W. Barbours Cut Blvd to the north and W. Main St to the south of the project area. Therefore, for the simplicity of the models, the results are presented with the Normal Depth Condition in the outfalls.

After completion of all the updates, the existing condition model was run for 5-, 10-, and 100-year scenarios. Exhibits 4, 5 and 6 show the maximum ponded depths for each 2D cell in the model limit for 5-, 10-, and 100-year storm events, respectively. The ponded depths have been measured from the inlet

levels. Table 1 shows the existing ponded depths of five strategic points along project area. Locations of these points are shown in Exhibits 4 to 6.

Table 1: Existing Condition Ponded Depths

Selected Points	Maximum Ponded Depth (ft)		
	5 - Year	10 - Year	100 - Year
6th St @ W. Madison St (Point A)	1.36	1.60	2.05
6th St @ W. Adams St (Point B)	1.18	1.33	1.60
6th St @ W. Main St (Point C)	0.34	0.45	0.72
TX-146 @ W. Main St (Point D)	0.00	0.23	0.92
TX-146 @ W. Madison (Point E)	0.90	1.03	1.46

4 Proposed Drainage Plan

4.1 Description

The goal is to minimize the ponding along 6th St. 6th St is the most upstream section of the storm sewer system that serves this area, and during the rainfall peak, the pipes get surcharged and flow backward / upstream and bubble out of the inlets along 6th St. As it can be seen in Exhibit 5, the ponding in the street is significant, even in 10-year event. Additionally, it appears that the area is very flat and there is no room for the water to leave the street. Therefore, it ponds in the street and the adjacent yards until the WSE in the pipes recedes and there is volume available for the ponded water to gravity drain dry. Multiple proposed alternatives were tested to find the most optimum scenario that reduces the ponding in the street while avoiding adverse impacts downstream. Table 2 shows the alternatives tested in this study:

Table 2 - Proposed Alternatives Descriptions

Model	Description
Alternative 0 (Alt. 0)	Existing storm sewer inlet & lead improvement evaluation.
Alternative 1a (Alt. 1a)	Doubling the Existing Pipes in 6th St, W. Madison St, W. Polk St, and W. Main St.
Alternative 1b (Alt. 1b)	Doubling the Existing Pipes in 6th St, W. Madison St, and W. Main St, excluding W. Polk St.
Alternative 2a (Alt. 2a)	Tripling the Existing Pipes in 6th St, W. Madison St, W. Polk St, and W. Main St.
Alternative 2b (Alt. 2b)	Tripling the Existing Pipes in 6th St, W. Madison St, and W. Main St, excluding W. Polk St.
Alternative 3 (Alt. 3)	Additional 24” HDPE in 6th St from W. Madison St to W. Main St [Existing Pipes in W. Main St].
Alternative 4 (Alt. 4)	Additional 36” HDPE in 6th St from W. Madison St to W. Main St [Existing Pipes in W. Main St].
Alternative 5 (Alt. 5)	- Additional 36” RCP in 6th St from W. Madison St to W. Main St, - Additional 24” HDPE in W. Main St from 6th St to TX-146, - Additional 36” HDPE/RCP in 6th St at W. Adams St flowing north to W. Madison St, and turning west to TX-146.
Alternative 6 (Alt. 6)	- Additional 48” RCP in 6th St from W. Madison St to W. Main St, - Additional 24” HDPE in W. Main St from 6th St to TX-146, - Improving the existing 18” to 24” RCP to 48” HDPE/RCP in 6th St at W. Adams St flowing north to W. Madison St, and turning west to TX-146.

4.1.1 Inlet Analysis

Reviewing the existing pattern of ponding along 6th St suggests that the storm sewer or street storage is not sufficient. The ability for the peak flows to get into the system was evaluated at each inlet and also for the inlet leads to get the flows to the trunklines.

As shown on Appendix A, Table A-1 summarizes the peak flows entering each intersection along with the available capacity for the inlets and pipe leads. As shown, the pipe leads are the limiting factor in the systems, but each aspect of the existing inlet configuration is too small for the 10-year flow for the most part, and definitely too small during the 100-year event.

For example, when reviewing the 5-year event, we noticed the following:

1. Madison St – The peak flow into that intersection is approximately 32.9 cfs and the combined inlets can drain off around 28.0 cfs, so there is a limiting factor there. The 30" Outfall from the SW Manhole can drain around 30.2 cfs, so it is closer, but still not able to fully drain the 5-year event even if it could get into the pipe. During the 5-year event, water bubbles out of the intersection and becomes sheet flow down Madison and also down 6th street.
2. Tyler St – In a similar analysis, the 21.9 cfs peak flow could be handled by the 28.0 cfs Inlet capacity, but the 18" outfall pipe that drains north along 6th St to Madison is well undersized (8.2 cfs), so the majority of the flows into this intersection pond significantly during a 5-yr event and sheetflow north to Madison and also south to Adams St, across the drainage divide.
3. Adams St – With around 32.6 cfs coming into the intersection during the 5-yr event, the inlets appear to have enough capacity (38.0 cfs) to get the water off the street, but the 18" outfall pipe running south to Polk St does not, so it is limited to approximately 8.2 cfs and causes excessive ponding and sheetflow.
4. Polk St – This intersection appears to have enough capacity for the 5-yr flow (26.6 cfs) in both inlet opening (28.0 cfs) and pipe / lead capacity (30.2 cfs), but this intersection also gets inundated with the excessive sheetflow from Adams St that ultimately the amount of sheetflow that adds to this intersection is more than it can handle and continues to sheetflow south to Main St (approximately 20 cfs).
5. Main St – Main has around 21.6 cfs that drains directly to the intersection, plus the 20+ cfs from sheetflow from the north, completely inundates this intersection and its limited inlet capacity (8.5 cfs) and the 18" outfall pipe / lead running from the NW Manhole (8.2 cfs), so a significant amount of sheetflow runs west along Main St to SH 146 and creates the significant ponding within this street.

During our iterations of the proposed alternatives, we had noted these types of limitations at each of the intersections, and this is how we came about with Alternative 5 and its configuration. It doesn't appear that Inlets are a significant limitation across this 6th St drainage system, but the pipes and leads are a limiting factor and would benefit the most from upsizing or replacing. Adding volume under the street would hold water until there is capacity in the downstream to drain these waters.

4.2 Hydrologic Analysis

The proposed hydrologic parameters remained the same as those described in Section 3.2 of this report. It was assumed that any proposed improvements would not add to the existing impervious cover of the area. Therefore, the land use remains the same for all models. The hydrologic flows and hydrographs remain the same in all models.

4.3 Hydraulic Analysis

The above alternatives were run for 5-, 10-, and 100-year storm events. The goal was to examine the benefits or impact each alternative has in the overall project area and their influence of each improvement on the drainage system outfalls and ponding depths on the streets.

Alternative 1a & 1b

Alternative 1a doubles the existing pipes in 6th St, W. Madison St, W. Polk St, and W. Main St up to TX-146. It adds approximately a total of 1,300' of 18" HDPE, 570' of 24" HDPE, 1,450' of 30" HDPE, and 300' of 36" HDPE. The overall additional volume is 0.31 ac-ft. As can be seen in Tables 3 to 5, this alternative reduces the ponding in the 6th St area for the 5- and 10-year events. However, it raises the outfall flow and has some impacts on the WSE near TX-146 in the 100-year event.

Alternative 1b is effectively the same series of improvements, but with excluding additional barrels along W. Polk St. This scenario was included to try to reduce the increased peak flows allowed to be conveyed westward to SH 146. This alternative did reduce the increased flows at SH 146, but further iterations proved to provide similar results, while focusing additional improvements to maximize benefits in other areas.

Alternative 2a & 2b

To see the impact of increasing the storm sewer volume under the street, the existing pipes in the Alt 1a & 1b were tripled. Therefore, approximately 0.61 ac-ft of volume added in the Alt 2a scenario. According to Tables 3 to 5, this scenario further reduces the ponding depth in 6th St. However, it makes

adverse impact to the ponding depths east of TX-146. Alt 2b is effectively the same series of improvements, but with excluding additional barrels along W. Polk St for similar reasons as noted above. Results also mirrored the findings from Alt 1a vs 1b.

Alternative 3

This alternative examines the impact of adding a set of storm sewers in the 6th St from W. Madison St to W. Main St. Since the south outfall is larger than the north outfall, this new line was tied into the existing W. Main St storm pipes.

Alternative 3 adds approximately 1,850' of new 24" HDPE pipes under 6th St. It adds about 0.13 ac-ft of volume. This alternative did not show much improvement to the ponding in the project area, reducing the ponding about an inch in the 10-year storm event.

Alternative 4

Similar to previous alternative, this alternative is an additional set of 36" HDPE pipes in 6th St. It adds a total volume of 0.30 ac-ft. This alternative had more benefits to the 6th St area but did not show much improvement to the ponding in the north part of the project area. It reduced the ponding about two inches in the 10-year storm event, however most of the benefits were observed on the south side of 6th St near W. Main St. It had minimal benefits in the larger events.

Alternative 5

To improve the north side of 6th St as well as the south side, an additional 36" HDPE/RCP was added to 6th St from W. Tyler St to W. Madison St, and to W. Madison St from 6th St to TX-146. Furthermore, an additional 24" pipe was also added to W. Main St from 6th St to TX-146 to help reduce the ponding in W. Main St too. This alternative adds a total of 985' of 24" HDPE and 3,100' of 36" HDPE/RCP with a total volume of 0.57 ac-ft.

This alternative almost removed all ponding in 6th St in the 5-year event, and significantly reduced the ponding in 6th St and W. Main St in the 10-year event (up to 4" reduction).

Alternative 6

With the same configuration of the Alternative 5 but instead of 36" pipes, this alternative uses the 48" HDPE in 6th St and W. Madison St. Therefore, about 0.97 ac-ft of volume is added to the existing volume in this alternative. There is not much difference in the 5-year event between Alt 5 and Alt 6 since both alternatives almost remove all the ponding. The 10-year event does not show significant additional reductions achieved over Alt 5.

Table 3 - Maximum Ponded Depth of Selected Points in Proposed Alternatives in the 5-year Event

Selected Points	Maximum Ponded Depth (ft) – 5 yr Storm Event								
	Existing	Alt. 1a	Alt. 1b	Alt. 2a	Alt. 2b	Alt. 3	Alt. 4	Alt. 5	Alt. 6
6th St @ W. Madison (Point A)	1.36	0.88	0.89	0.57	0.62	1.38	1.49	1.08	1.19
6th St @ W. Adams (Point B)	1.18	1.01	1.15	0.89	1.15	1.18	1.15	0.92	0.76
6th St @ W. Main (Point C)	0.34	0.23	0.23	0.09	0.15	0.24	0.13	0.00	0.00
TX-146 @ W. Main (Point D)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TX-146 @ W.Madison (Point E)	0.90	0.91	0.90	0.92	0.91	0.90	0.90	0.90	0.90

Table 4 - Maximum Ponded Depth of Selected Points in Proposed Alternatives in the 10-year Event

Selected Points	Maximum Ponded Depth (ft) – 10 yr Storm Event								
	Existing	Alt. 1a	Alt. 1b	Alt. 2a	Alt. 2b	Alt. 3	Alt. 4	Alt. 5	Alt. 6
6th St @ W. Madison (Point A)	1.60	1.28	1.28	1.12	1.12	1.68	1.74	1.37	1.41
6th St @ W. Adams (Point B)	1.33	1.25	1.31	1.17	1.30	1.34	1.34	1.20	1.10
6th St @ W. Main (Point C)	0.45	0.34	0.38	0.25	0.32	0.35	0.31	0.25	0.08
TX-146 @ W. Main (Point D)	0.23	0.32	0.23	0.45	0.29	0.19	0.00	0.23	0.20
TX-146 @ W.Madison (Point E)	1.03	1.04	1.03	1.04	1.04	1.03	1.03	1.03	1.03

Table 5 - Maximum Ponded Depth of Selected Points in Proposed Alternatives in the 100-year Event

Selected Points	Maximum Ponded Depth (ft) – 100 yr Storm Event								
	Existing	Alt. 1a	Alt. 1b	Alt. 2a	Alt. 2b	Alt. 3	Alt. 4	Alt. 5	Alt. 6
6th St @ W. Madison (Point A)	2.05	1.90	1.91	1.83	1.85	2.07	2.10	1.95	1.96
6th St @ W. Adams (Point B)	1.60	1.53	1.56	1.48	1.54	1.61	1.62	1.55	1.53
6th St @ W. Main (Point C)	0.72	0.68	0.69	0.63	0.66	0.47	0.45	0.44	0.41
TX-146 @ W. Main (Point D)	0.92	0.96	0.93	0.98	0.94	0.92	0.91	0.92	0.92
TX-146 @ W.Madison (Point E)	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46

4.4 Sheetflow Analysis

The existing 100-yr model was used to determine the amount and direction of overland sheetflow that bypasses each respective collection system within the project area across 18 selected flow lines. Exhibit B1 in Appendix B summarizes the peak flow in each direction for the 18 selected flow lines. The hydrographs for each of the flow lines have been plotted separately in Appendix B.

Generally, the sheetflow path is from north to south, and east to west, as the sheetflow appears to work its way towards Main Street and SH 146. For example, at the peak, the cumulative bypass flow in a 100-yr rain event leaving the Main Street and 6th Street intersection is around 53 cfs flowing west down Main Street (see Flowline #7 – Exhibit B1). Screenshots of the model illustrating how the sheetflow progresses over time along with the full video-clip have been presented to the City officials at the time of submission of this report.

4.5 Recommended Alternative

After carefully reviewing the six alternatives and considering the optimum cost of construction, Alternative 5 was selected as the recommended alternative. Exhibit 7 shows the model system components.

The recommended alternative almost makes the 6th St area completely dry during the 5-year event (Exhibit 8), and includes a 1 to 3 inch ponding reduction, compared to the existing condition (Exhibit 4). For the 10-year storm event, the recommended alternative also reduces the ponding depth along 6th St and W. Main St from 1 to 4 inches compared to the existing condition (Exhibit 9). This scenario does not adversely impact the downstream outfalls and the ultimate flows into the Little Cedar Bayou remains at or below the existing condition ranges.

The ponding depths along 6th St and W. Main St reduce 1 to 2 inches for the 100-year storm event compared to the existing condition (see Exhibit 10). Also, other streets in the neighborhood benefit from this project by reductions of 1 to 3 inches. However, due to the outfall restriction, this alternative is limited in providing more benefits to the 100-year storm event. For this magnitude of event, much more detention volume is needed to make a larger impact to the area.

Exhibits 11, 12 and 13 summarizes the reduction in ponding depth to the project area in the 5-, 10-, and 100-year storm events. It must be noted that this is the reduction in the maximum WSE observed in the model. However, in addition to the reduction in max WSE, the duration of ponding decreases in all

points along 6th St during all storm events. The reduced duration of ponding is as important of an advantage from the proposed improvements.

4.6 Cost Estimates

The total estimated construction costs summarized in the Table 6 for the recommended alternative. Using a 20% contingency, the total cost is estimated approximately $2,043,957. It includes the storm sewer items along with the street paving, waterline improvements along N. 6th St and other extra items. This cost estimate does not include any right of way acquisition or private utility relocation costs. Details of the cost items are provided in the attached table.

Table 6 - Cost Estimate for the Recommended Alternative

	Item Description	Recommended Alternative Cost Estimate
1	Site Preparation	$230,570.00
2	Paving	$591,890.00
3	Drainage	$554,712.50
4	Water	$181,125.00
5	Miscellaneous	$145,000.00
Subtotal		$1,703,297.50
20% Contingency		$340,659.50
Total Estimated Construction Cost		$2,043,957.00

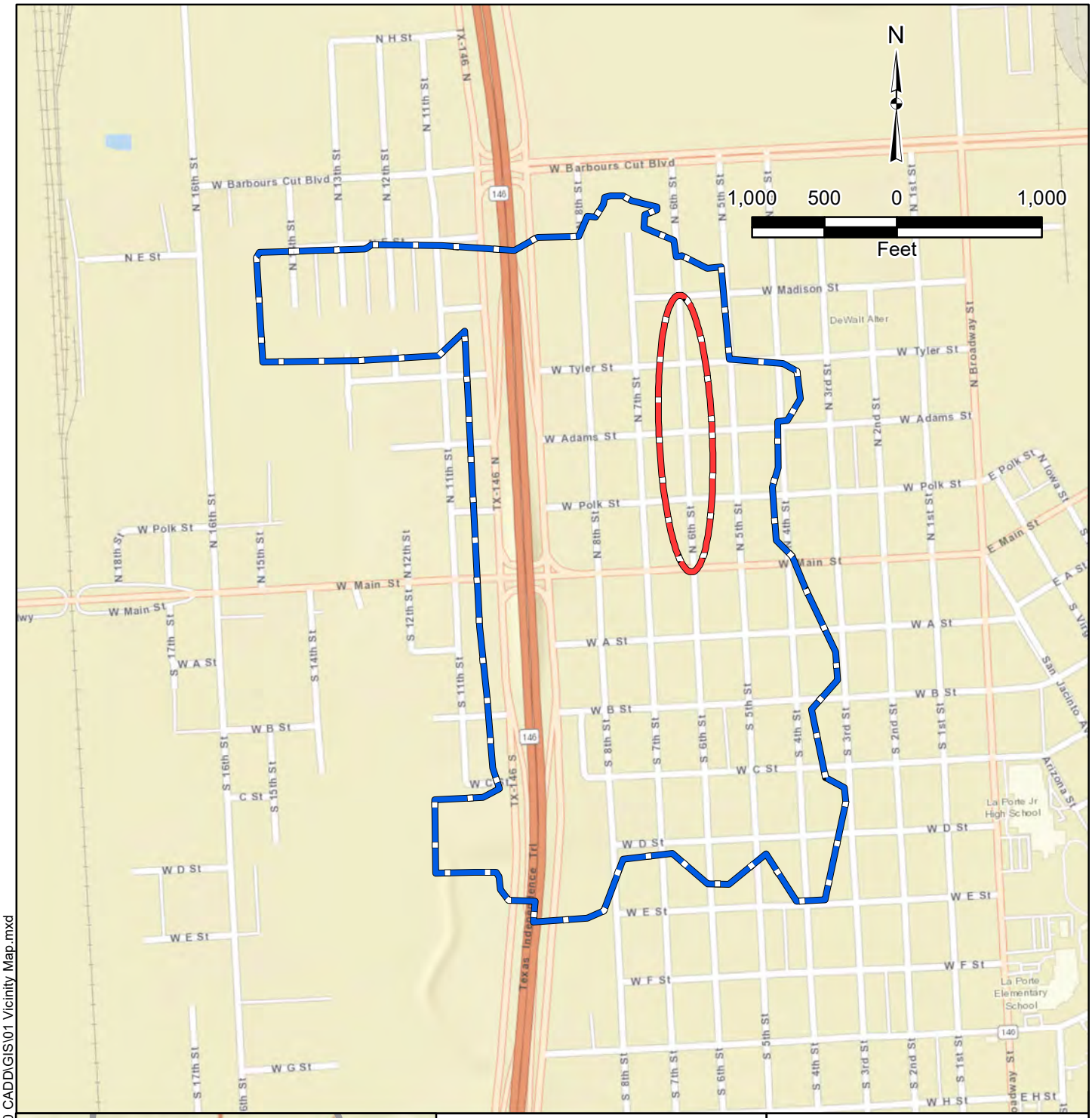
5 Summary and Conclusion

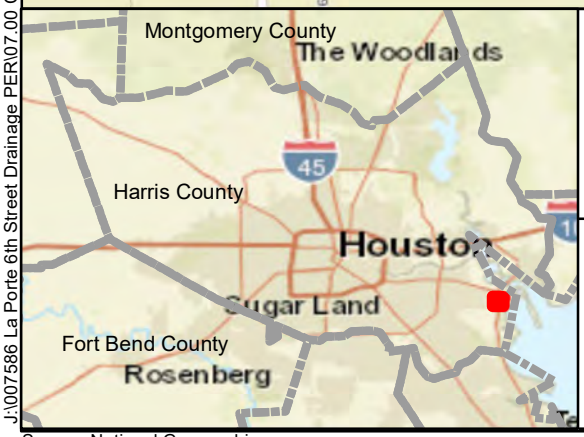
The goal of this analysis is to evaluate the existing storm sewer system, identify problem areas within the project limits, and provide recommendations for improvements along 6th St and surrounding areas. The drainage areas were delineated with HEC-Geo HMS using the 2008 LiDAR data. The storm sewer data were acquired from the City of La Porte Utility Map Website. Field survey was conducted in key points to verify the accuracy of the storm sewer data and the elevations of LiDAR data. A detailed 2D model, using the most updated survey data for the inlets and manholes, was created and run for 5-, 10-, and 100-year storm events.

Sensitivity models were run to determine how much the outfall conditions impact the ponding in the project area (6th St from W. Madison St to W. Main St). The sensitivity models showed that while the areas adjacent to the bayou are highly impacted by the tailwater condition, the project area on the east side of TX-146 is nearly independent of the outfall conditions. The results of all models are presented with the Normal Depth Condition in the outfalls.

Existing and multiple iterations of the proposed models were simulated. The results proved that 6th St from W. Madison St to W. Main St has significant ponding in storm events as low as 5-year event. The proposed models investigated the potential options to reduce the ponding along 6th St while not adversely impacting the downstream system. The recommended proposed improvements include 1,850' of 36" HDPE/RCP in 6th St running south from W. Madison St, and 1,250' of 36" HDPE/RCP in W. Madison St from 6th St to TX-146. Additionally, 985' of 24" HDPE is recommended to be added to W. Main St from 6th St to TX-146. The total added volume is approximately 0.57 ac-ft.

The proposed improvements almost remove all the ponding in 5-year event and reduce the maximum ponding elevation across the project area in 10-year (1 to 3 inches) and 100-year (1 to 3 inches) storm events. Furthermore, the recommended improvements considerably reduce the ponding duration compared to the existing condition by up to two (2) hours.





Legend

- Drainage area
- Project Limits
- HCFCD Channel

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Vicinity Map

La Porte
6th Street Improvements

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Source: National Geographic

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Topography Map
(2008 LiDAR)

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6th Street Improvements

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Legend

- Selected Points
- Existing Nodes
- System A
- System B
- HCFCD Channel
- Parcels

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Existing Model System Components

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6th Street Improvements

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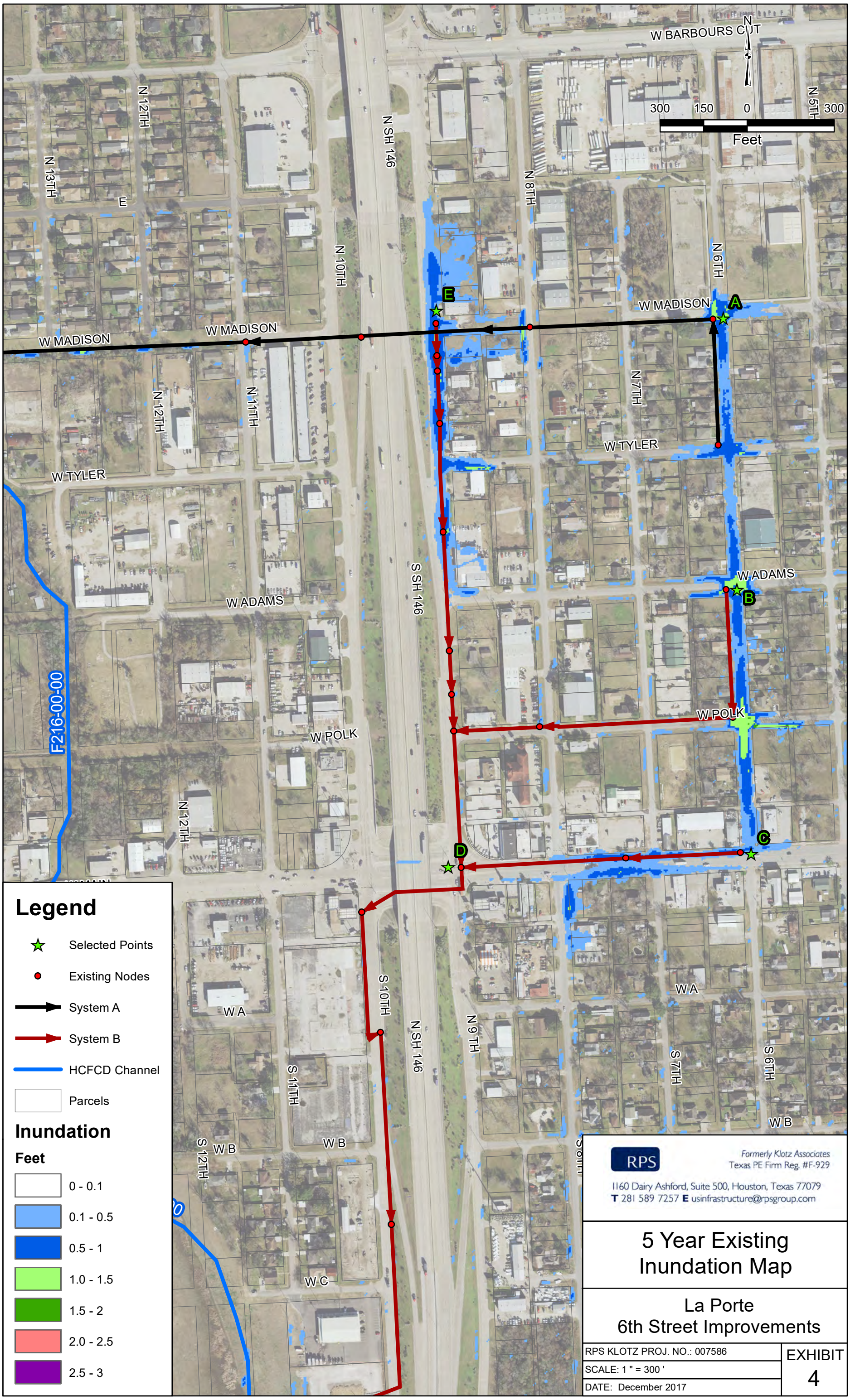
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5 Year Existing
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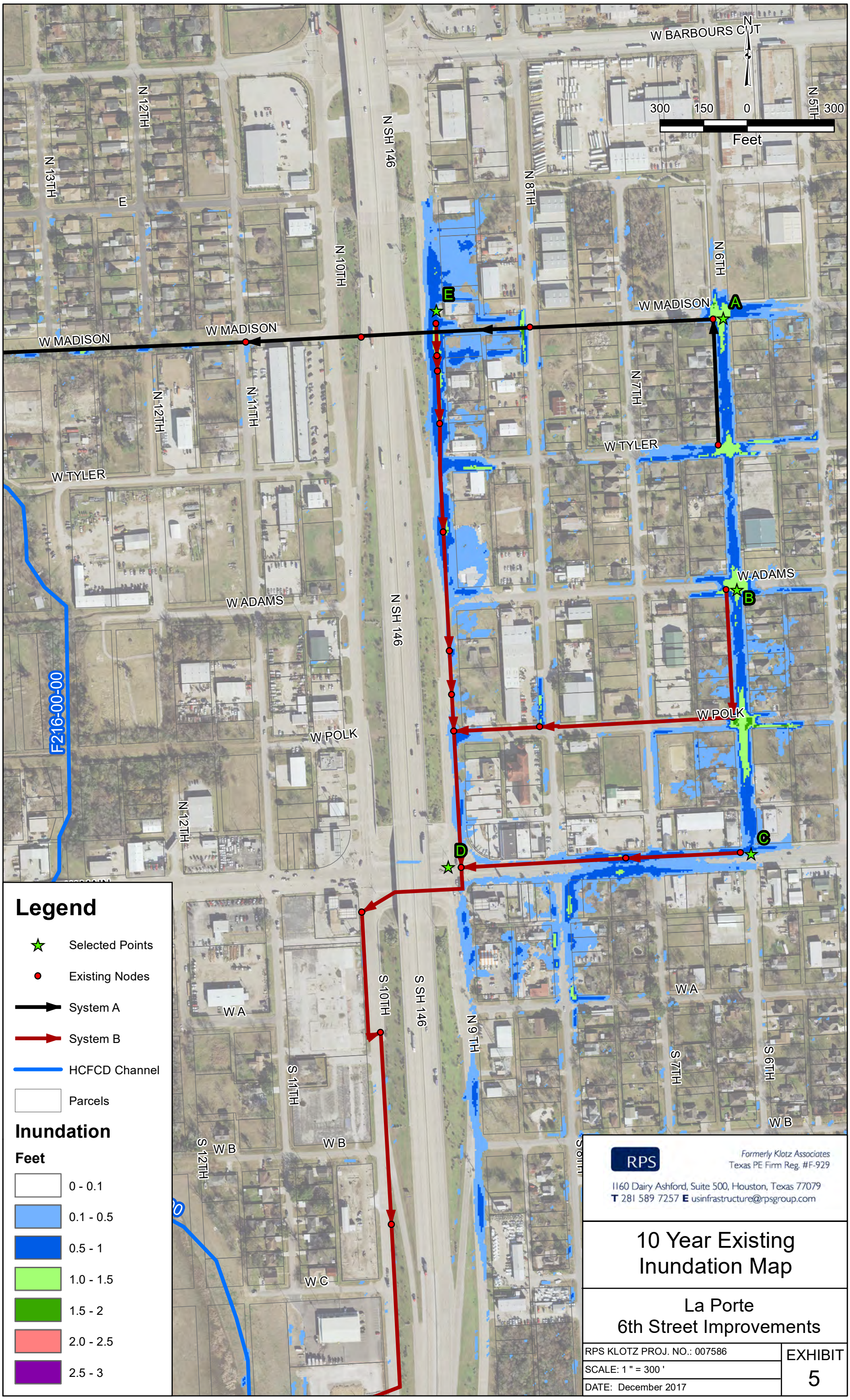
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10 Year Existing
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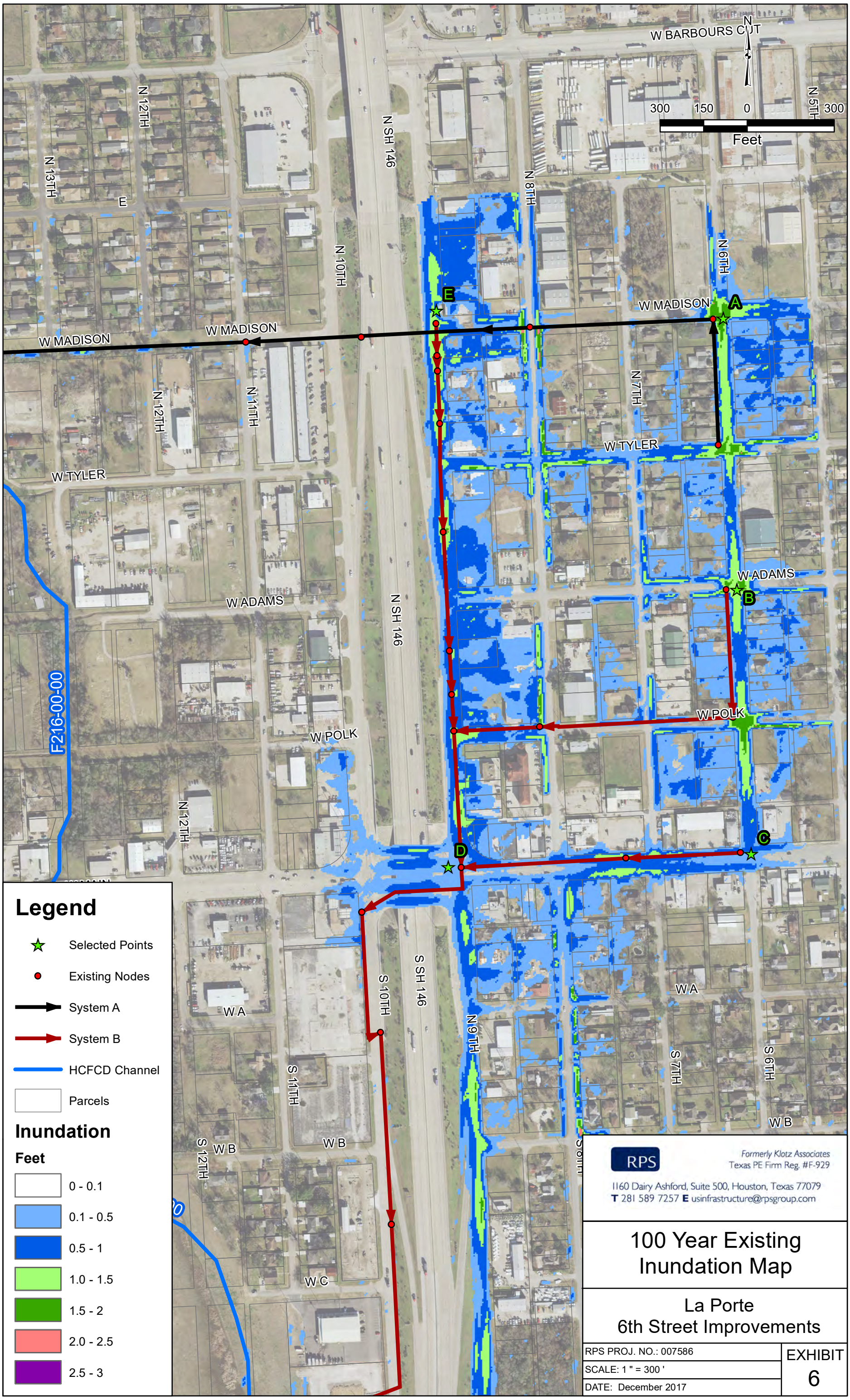
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6th Street Improvements
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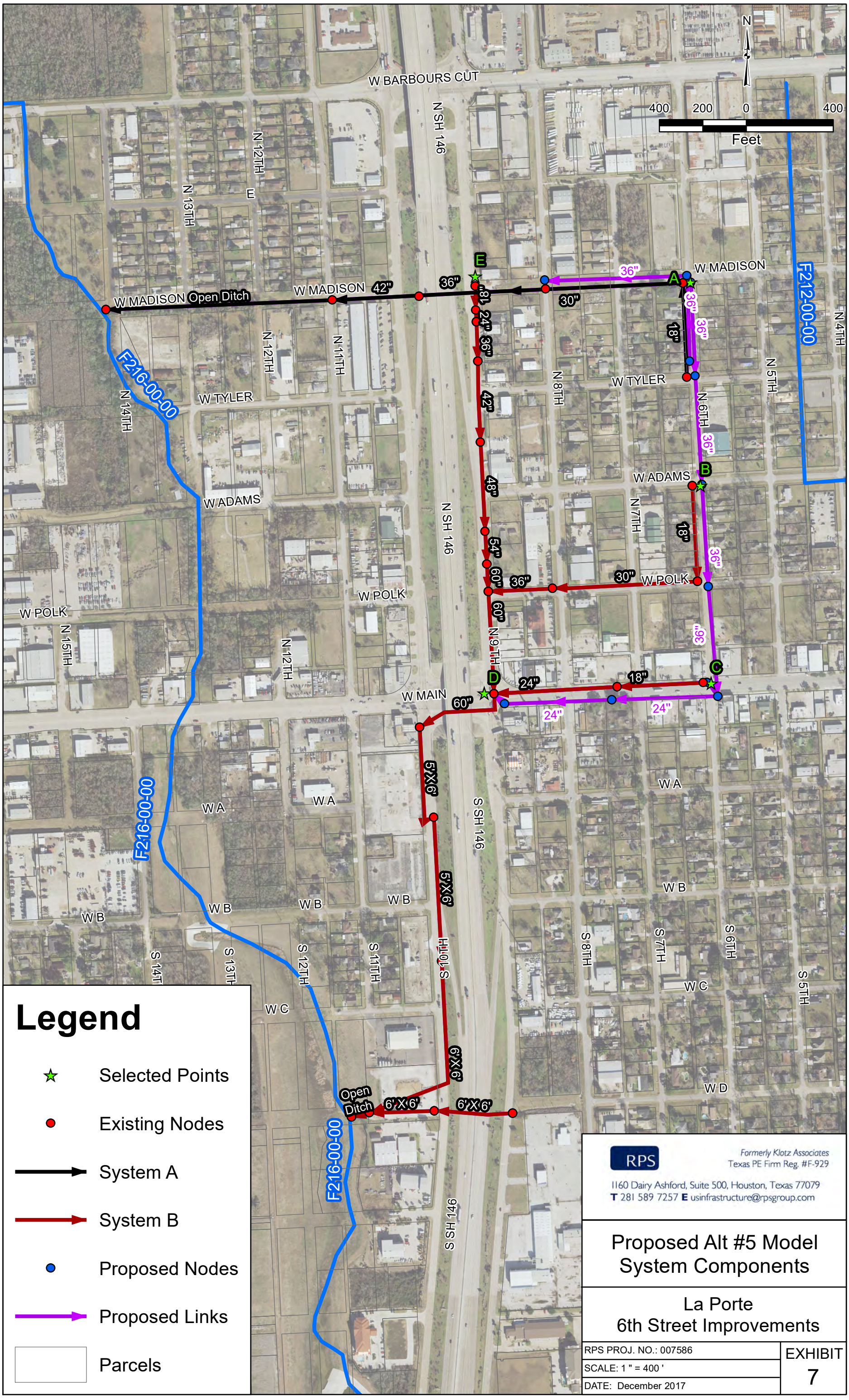

Legend

- Selected Points
- Existing Nodes
- System A
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- Proposed Nodes
- Proposed Links
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Proposed Alt #5 Model System Components

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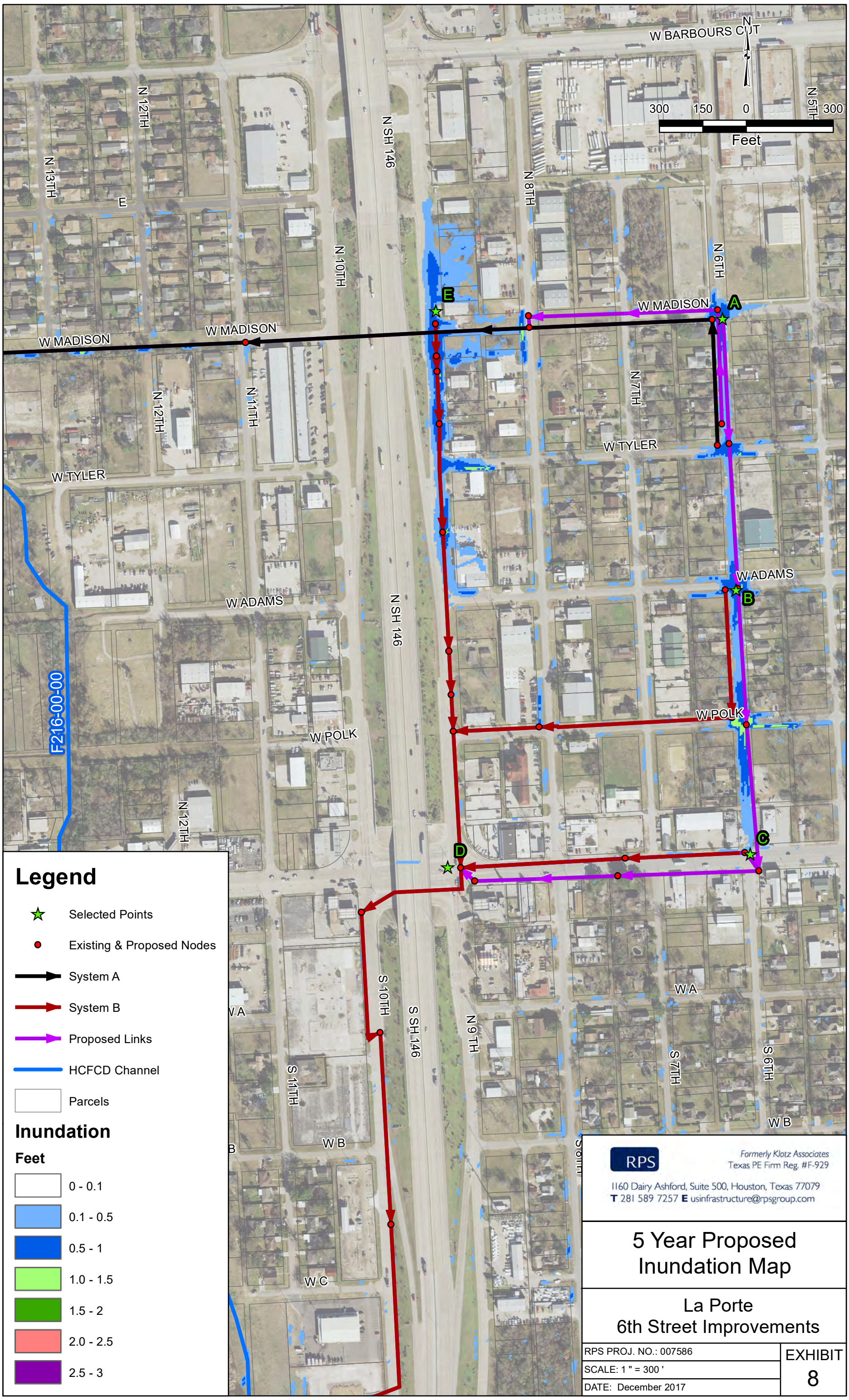
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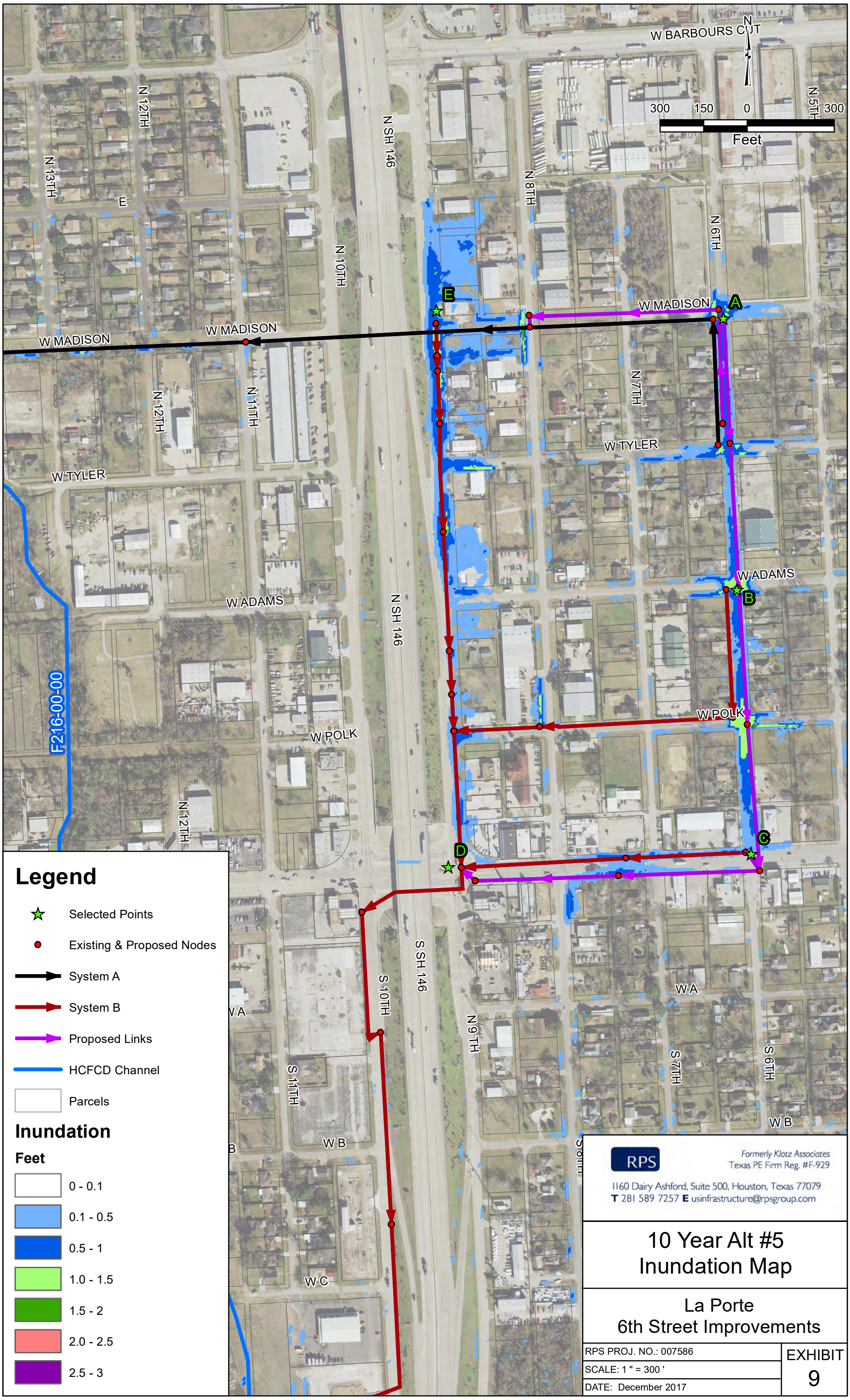
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10 Year Alt #5
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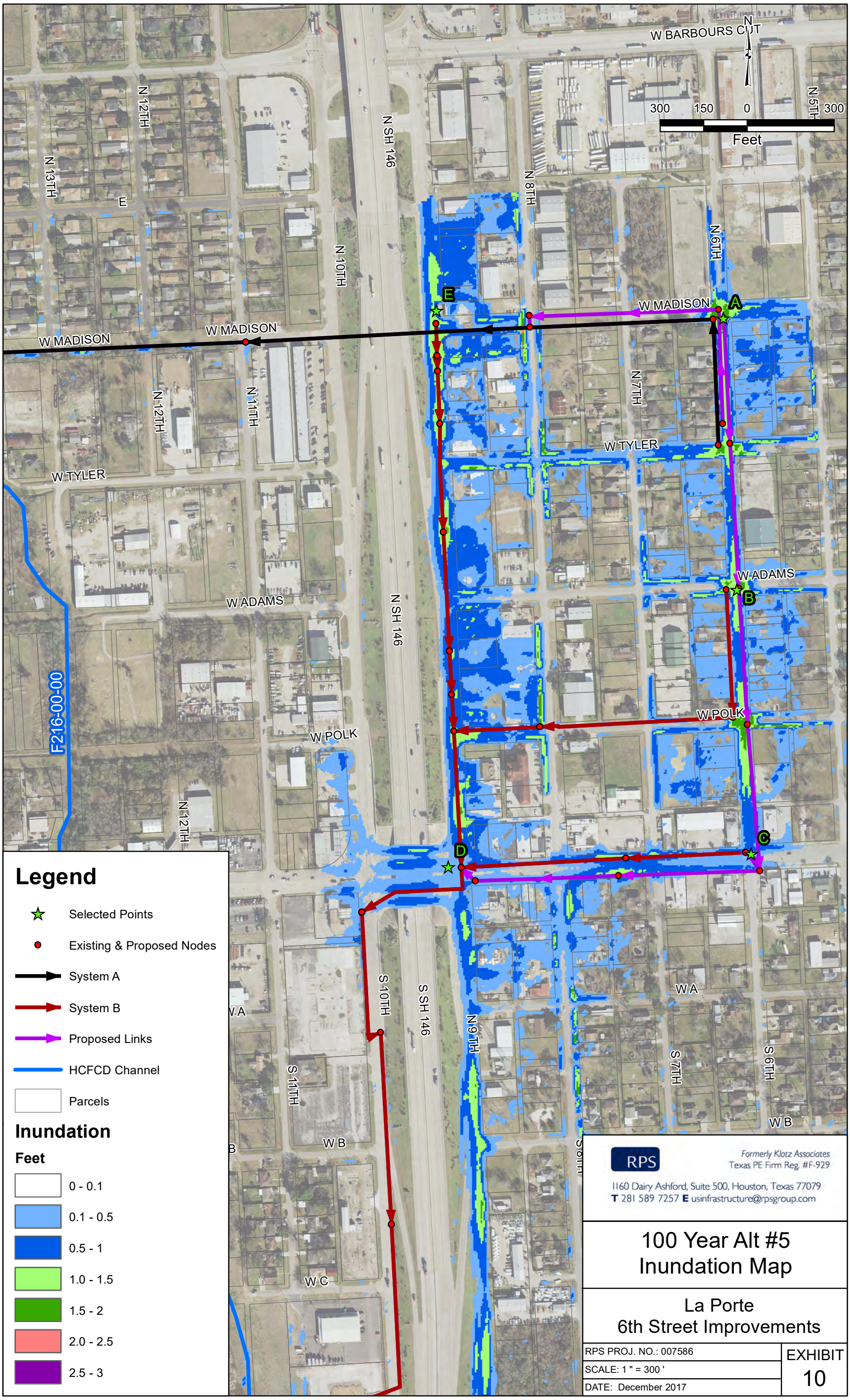
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100 Year Alt #5
Inundation Map
La Porte
6th Street Improvements
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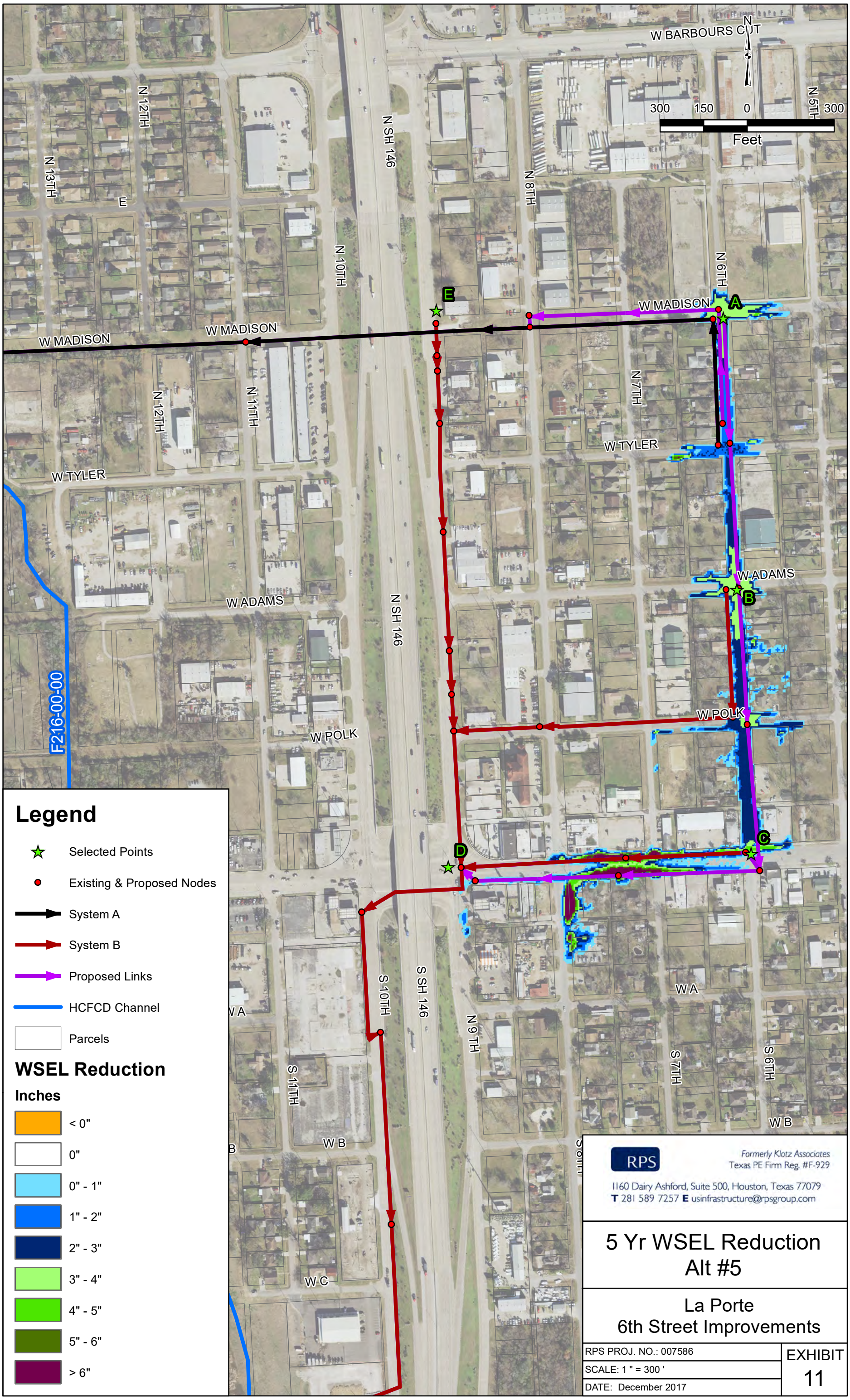

5 Yr WSEL Reduction
Alt #5

La Porte
6th Street Improvements

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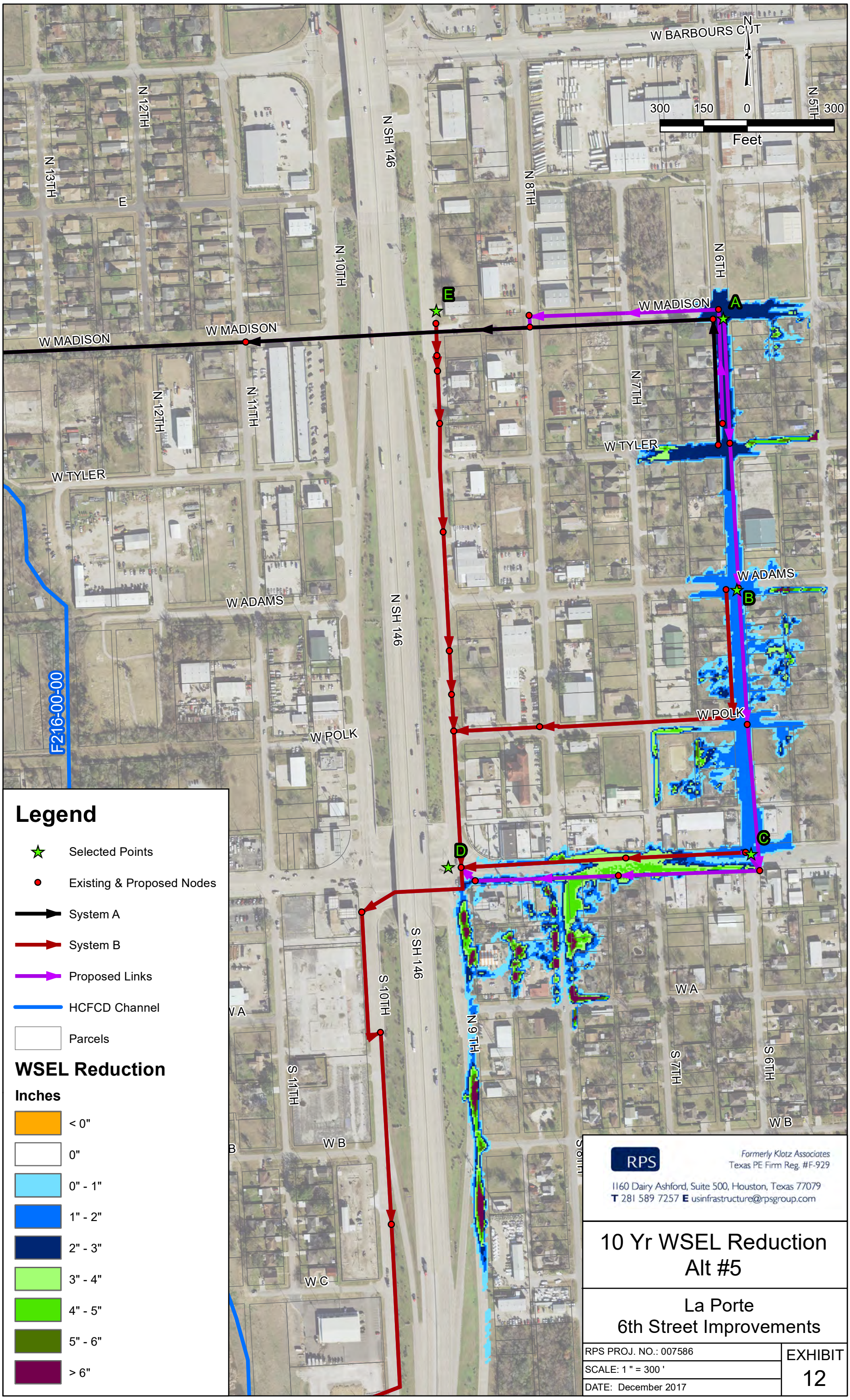
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Texas PE Firm Reg. #F-929
1160 Dairy Ashford, Suite 500, Houston, Texas 77079
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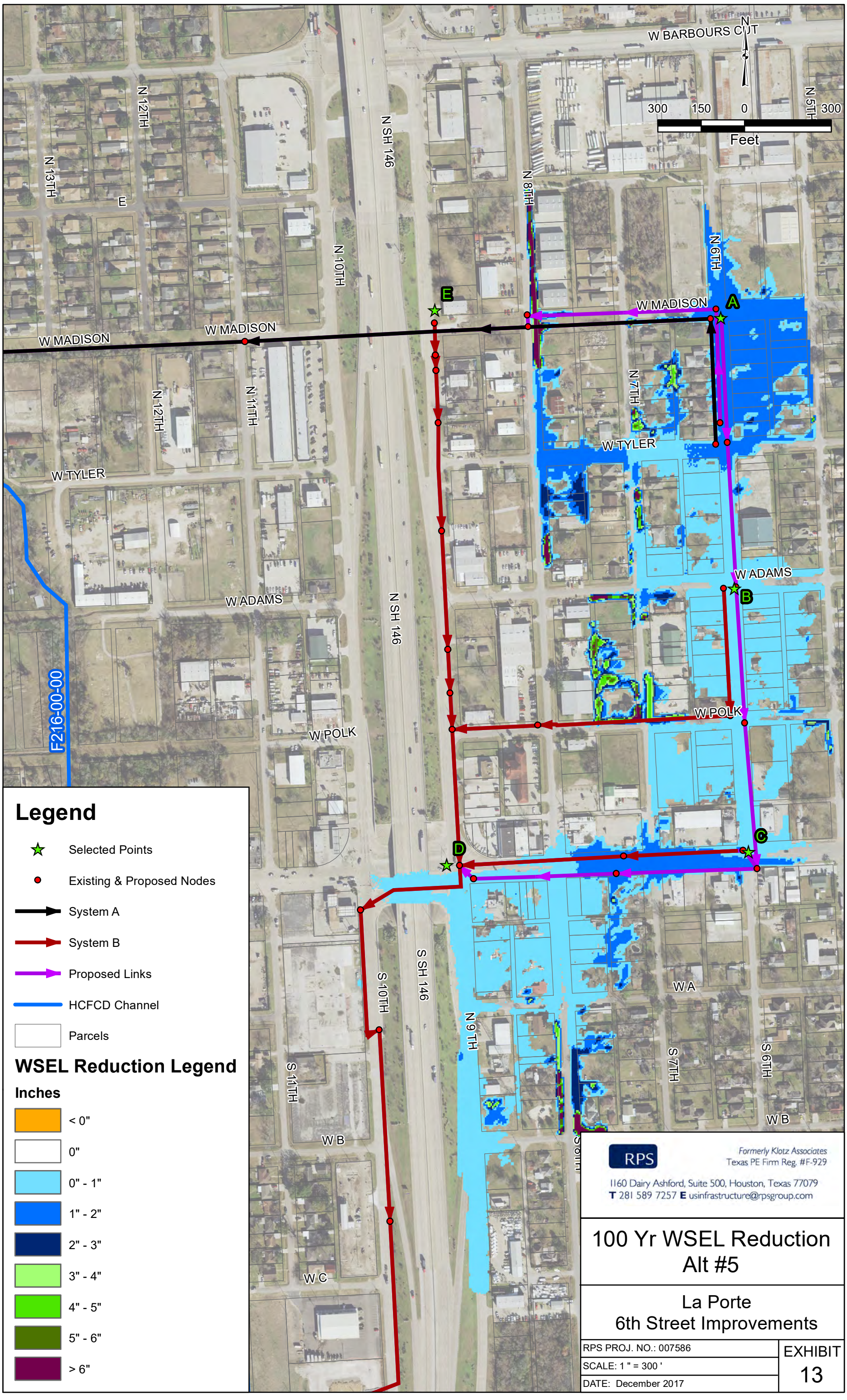
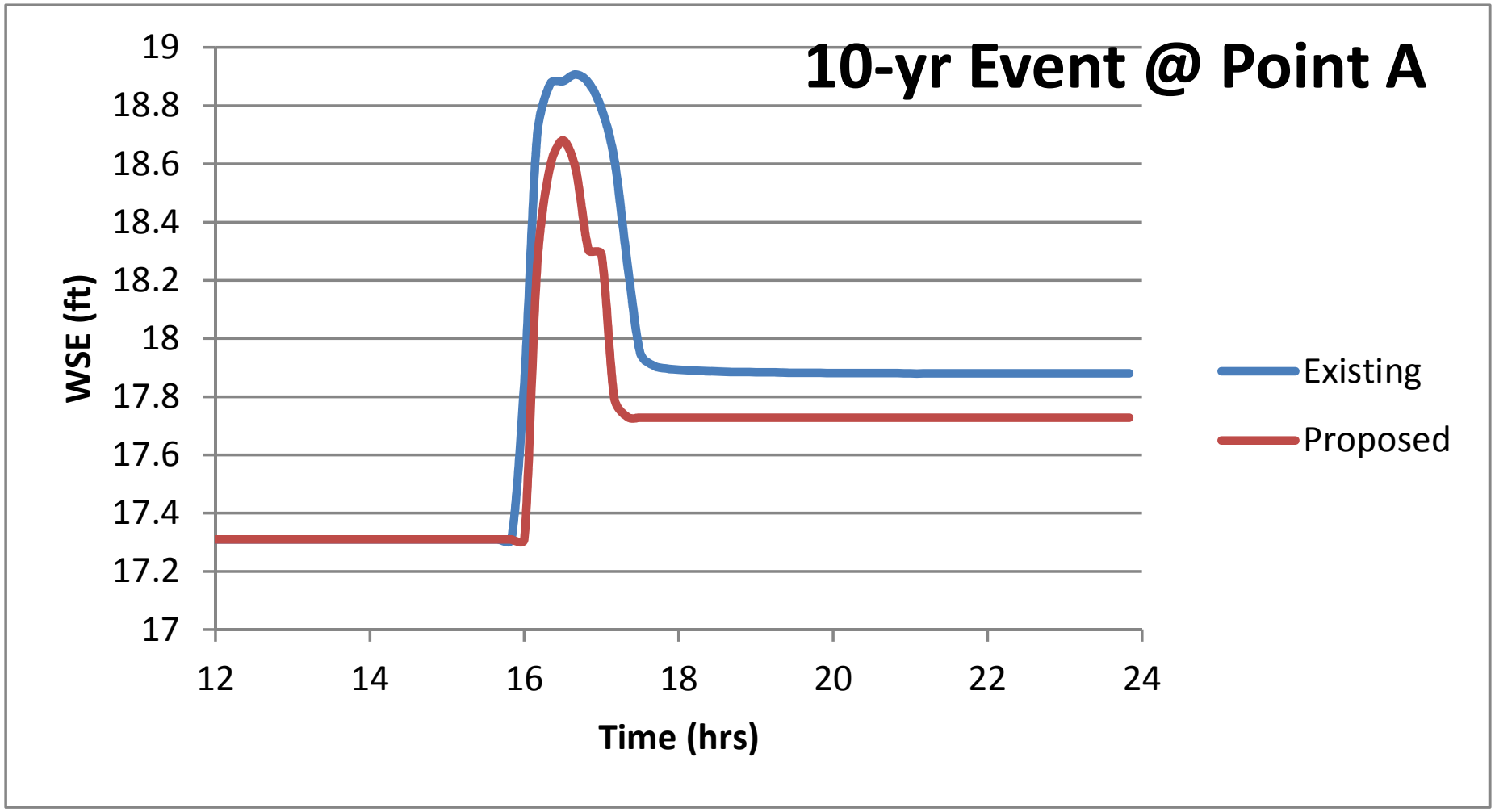
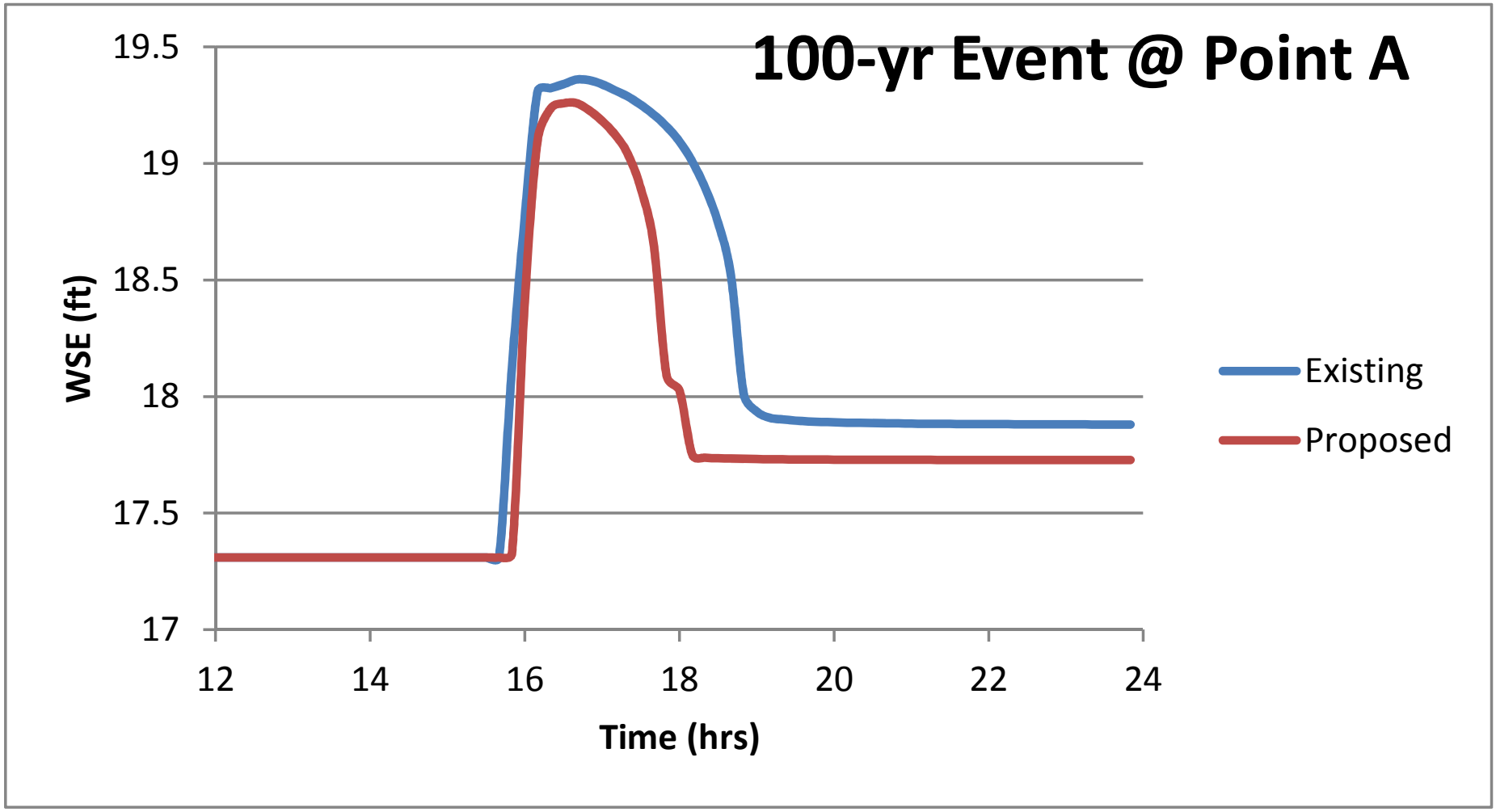
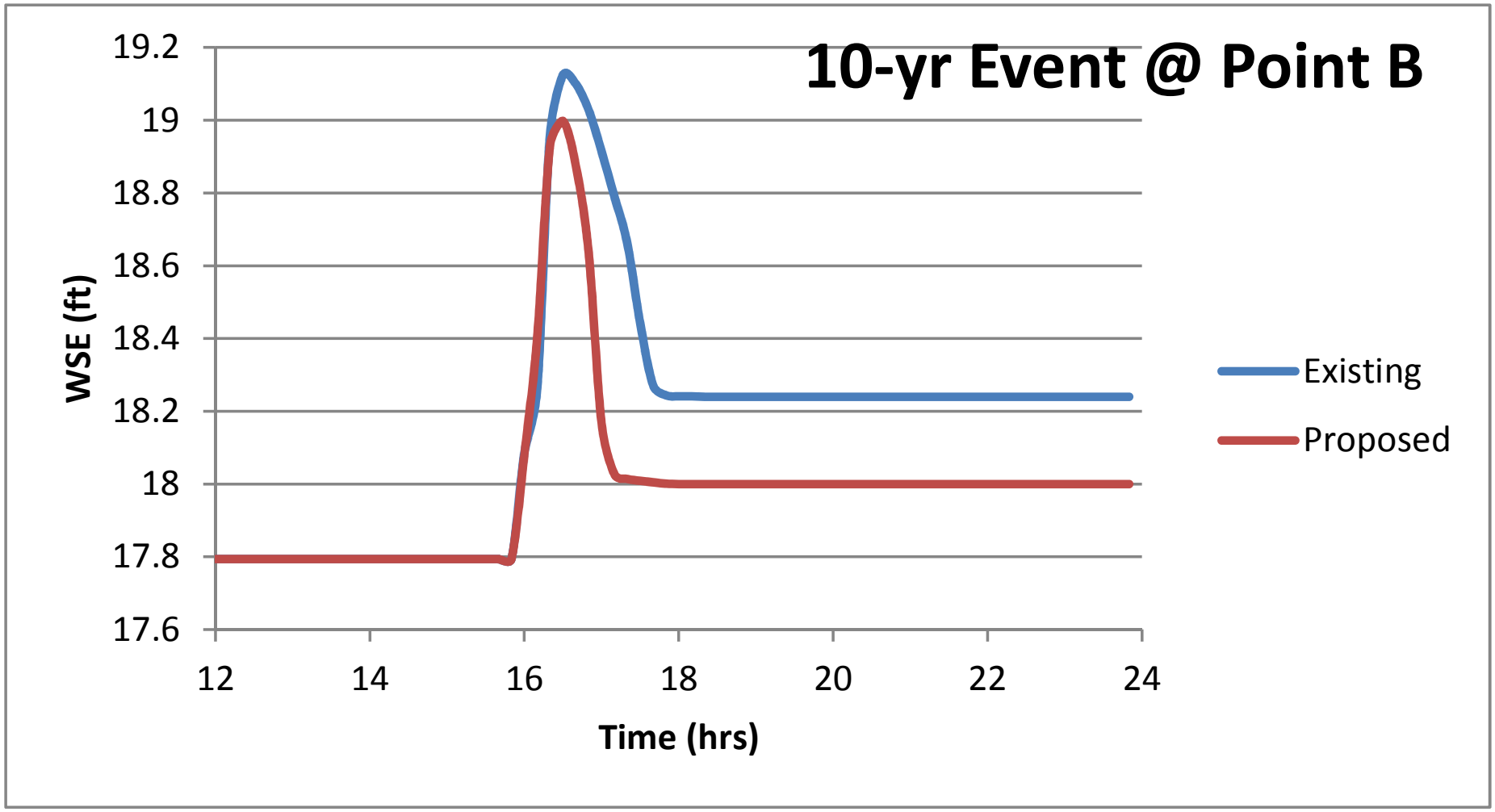
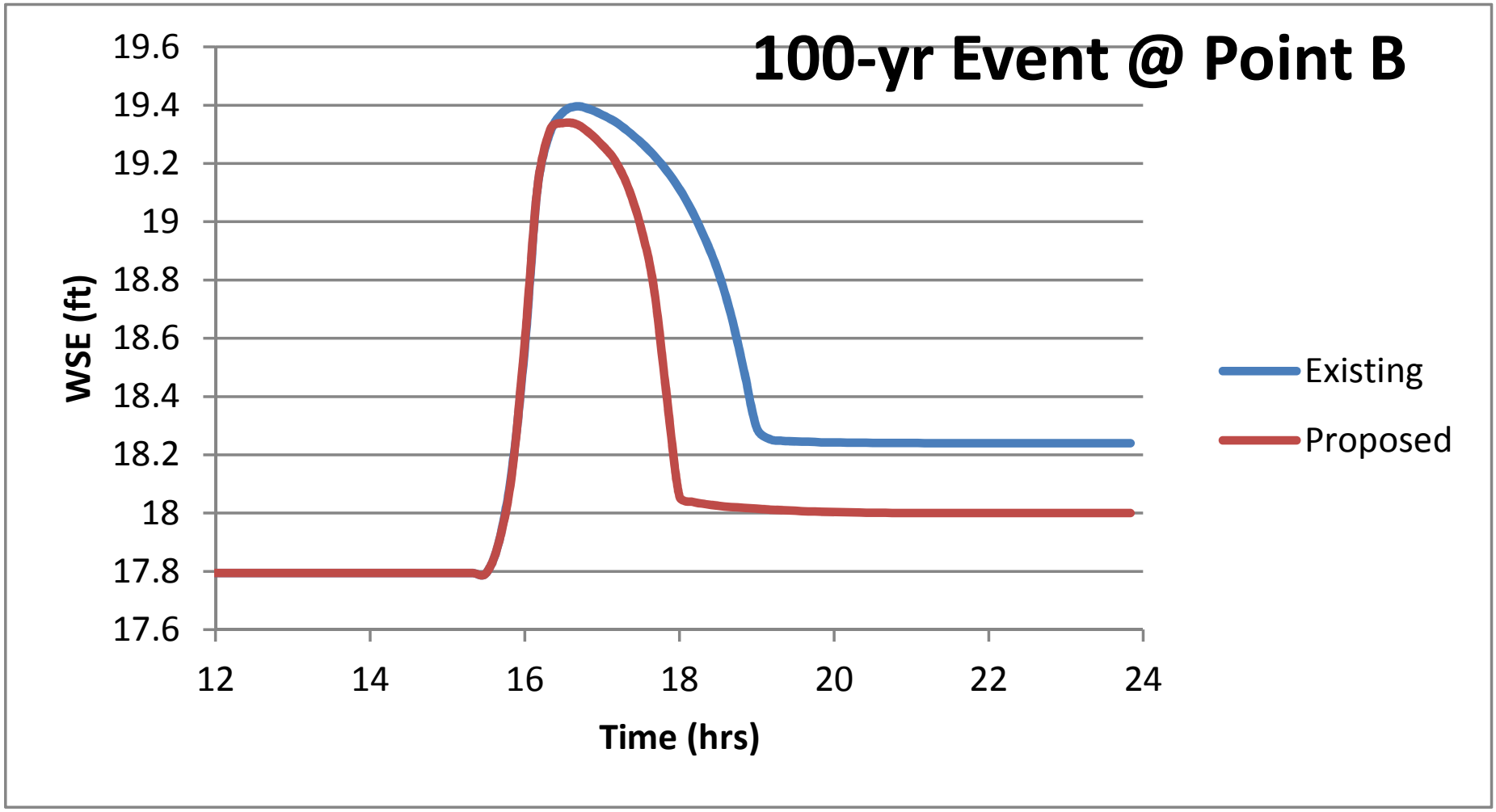
10 Yr WSEL Reduction
Alt #5
La Porte
6th Street Improvements
RPS PROJ. NO.: 007586
SCALE: 1 " = 300 '
DATE: December 2017
EXHIBIT
12

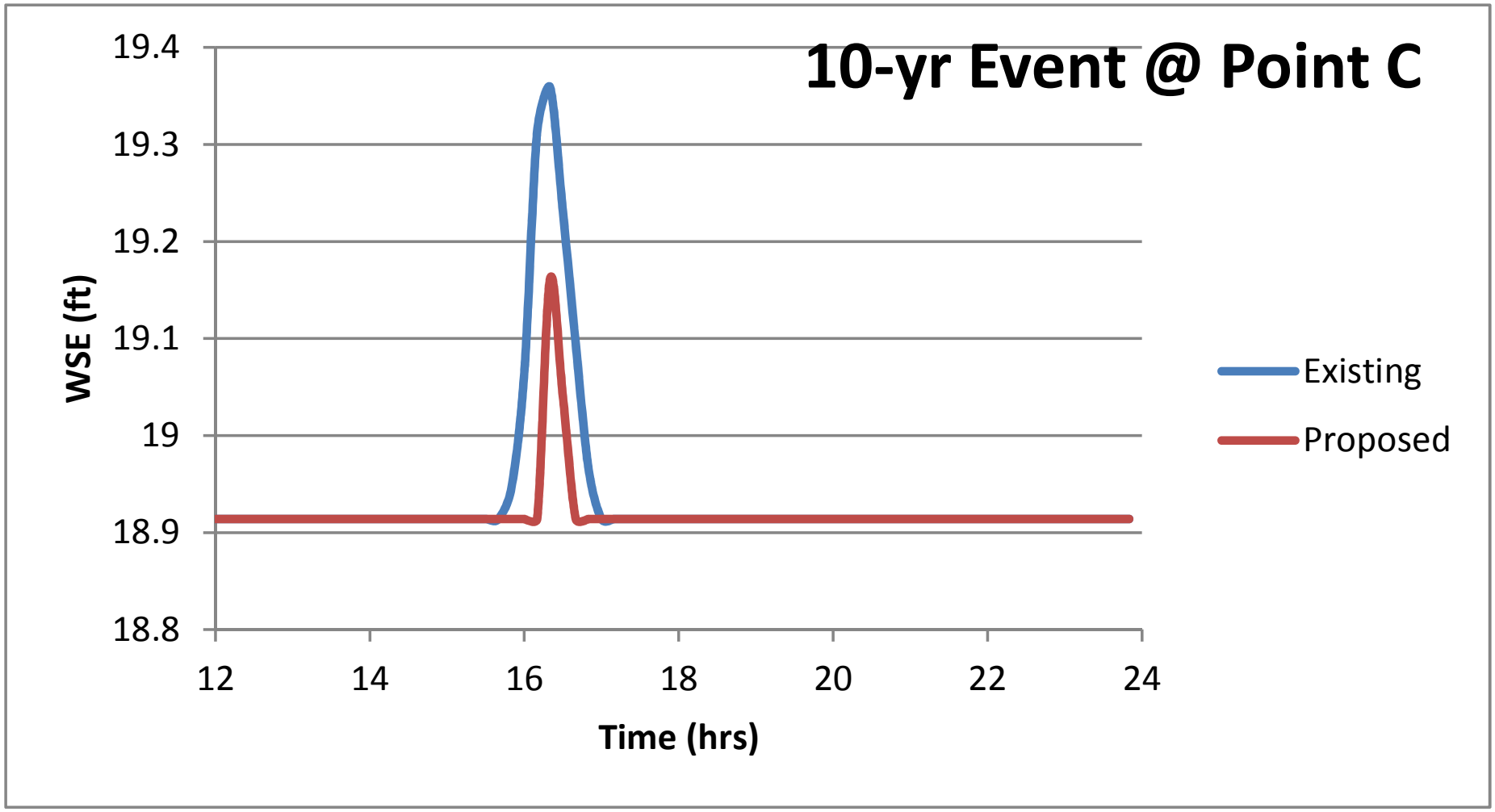

W BARBOURS CUT
N
300 150 0 300
Feet
N 5TH
N 6TH
N 7TH
N 8TH
N 10TH
N 11TH
N 12TH
N 13TH
E
N SH 146
W MADISON
W TYLER
W ADAMS
W POLK
F216-00-00
A
B
C
D
E
S SH 146
S 10TH
S 11TH
N 9TH
S 7TH
S 6TH
W A
W B
W C
Legend
Selected Points
Existing & Proposed Nodes
System A
System B
Proposed Links
HCFCD Channel
Parcels
WSEL Reduction Legend
Inches
< 0"
0"
0" - 1"
1" - 2"
2" - 3"
3" - 4"
4" - 5"
5" - 6"
> 6"
RPS
Formerly Klotz Associates
Texas PE Firm Reg. #F-929
1160 Dairy Ashford, Suite 500, Houston, Texas 77079
T 281 589 7257 E usinfrastructure@rpsgroup.com
100 Yr WSEL Reduction
Alt #5
La Porte
6th Street Improvements
RPS PROJ. NO.: 007586
SCALE: 1 " = 300 '
DATE: December 2017
EXHIBIT
13

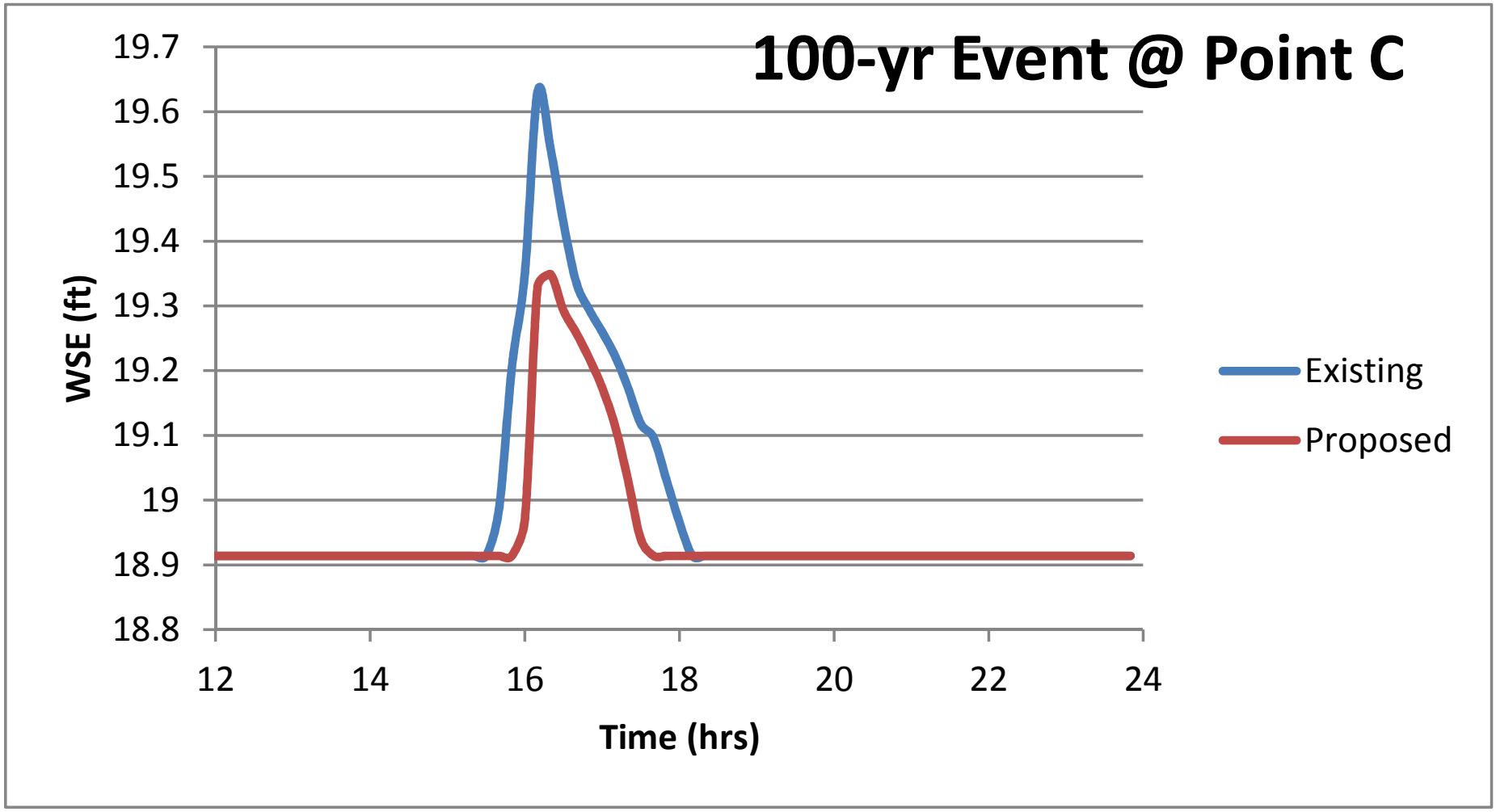

10-yr Event @ Point A
WSE (ft)
19
18.8
18.6
18.4
18.2
18
17.8
17.6
17.4
17.2
17
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed


100-yr Event @ Point A
19.5
19
18.5
18
17.5
17
WSE (ft)
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed


10-yr Event @ Point B
19.2
19
18.8
18.6
18.4
18.2
18
17.8
17.6
WSE (ft)
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed


100-yr Event @ Point B
WSE (ft)
19.6
19.4
19.2
19
18.8
18.6
18.4
18.2
18
17.8
17.6
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed


10-yr Event @ Point C
19.4
19.3
19.2
19.1
19
18.9
18.8
WSE (ft)
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed


100-yr Event @ Point C
19.7
19.6
19.5
19.4
19.3
19.2
19.1
19
18.9
18.8
WSE (ft)
12
14
16
18
20
22
24
Time (hrs)
Existing
Proposed

Appendix A – Inlet Analysis

La Porte - 6th Street Inlet Evaluation

Intersection Summary	Model Flows (cfs)			Inlet Throat Capacity		Pipe Lead Capacity	
	5-yr	**10-yr**	**100-yr**	**Existing Inlets Congifuration**	**Capacity**	**Diameter**	**Capacity**
6th @ W Madison St	32.9	41.2	74.5	NW - Type C w/ 24" (N) & 18" (W) Stub	13.5	18	8.2
74.5 cfs - 100-yr Flow				NE - Type C	2.5	15	6.0
28.0 cfs - Inlet Capacity				SE - Type C	2.5	15	6.0
30.2 cfs - Outfall Lead Capacity				SW - Type C w/ 24" (W) Stub	9.5	***30***	***30.2***
6th @ W Tyler St	21.9	27.4	49.7	NW - Type C (Mod)	3.5	***18***	***8.2***
49.7 cfs - 100-yr Flow				NE - Type C (Mod)	3.5	18	8.2
28.0 cfs - Inlet Capacity				SE - Type C (Mod) w/ 1.5'x1.5' (E) Stub	10.5	15	6.0
8.2 cfs - Outfall Lead Capacity				SW - Type C (Mod) w/ 24" (W) Stub	10.5	15	6.0
6th @ W Adams St	32.6	41.2	76.3	NW - Type C w/ 1.5'x1.5' (W) Stub	9.5	15	6.0
76.3 cfs - 100-yr Flow				NE - Type C w/ 1.5'x1.5' (E) Stub	9.5	15	6.0
38.0 cfs - Inlet Capacity				SE - Type C w/ 1.5'x1.5' (E) Stub	9.5	18	8.2
8.2 cfs - Outfall Lead Capacity				SW - Type C w/ 1.5'x1.5' (W) Stub	9.5	***18***	***8.2***
6th @ W Polk St	26.6	33.7	62.4	NW - Type C w/ 1.0'x1.0' (W) Stub	7.5	***30***	***30.2***
62.4 cfs - 100-yr Flow				NE - Type C w/ 1.0'x1.5' (E) Stub	8.5	18	8.2
28.0 cfs - Inlet Capacity				SE - Type C w/ 1.5'x1.0' (E) Stub	8.5	15	6.0
30.2 cfs - Outfall Lead Capacity				SW - Type C (Mod)	3.5	15	6.0
6th @ W Main St	21.6	26.4	47.5	NW - Type C (Mod)	3.5	***18***	***8.2***
47.5 cfs - 100-yr Flow				*NE - N/A*	-	-	-
8.5 cfs - Inlet Capacity				*SE - N/A*	-	-	-
8.2 cfs - Outfall Lead Capacity				SW - Type BB	5.0	12	4.4

01/19/18

La Porte - 6th Street Alternative Evaluation

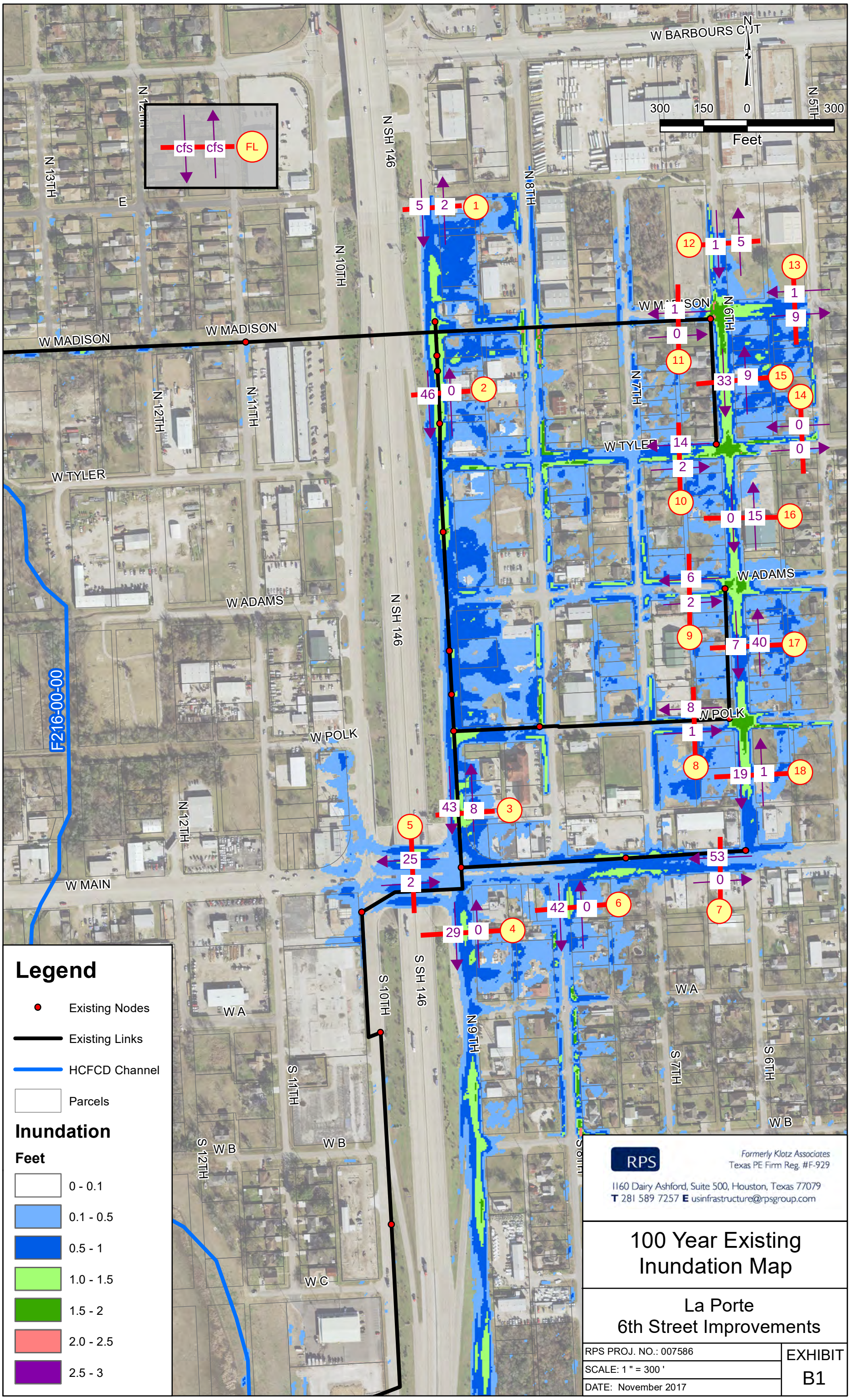
Selected Points	Maximum Ponded Depth (ft) – 5 yr Storm Event					
	Existing	Alt. 1a	*Alt. 1b*	Alt. 2a	*Alt. 2b*	Alt. 5
Point A - 6th St @ W. Madison St	1.36	0.88	*0.885*	0.57	*0.620*	1.08
Point B - 6th St @ W. Adams St	1.18	1.01	*1.153*	0.89	*1.148*	0.92
Point C - 6th St @ W. Main St	0.34	0.23	*0.234*	0.09	*0.151*	0
Point D - TX-146 @ W. Main St	0	0	*0.000*	0	*0.000*	0
Point E - TX-146 @ W. Madison	0.9	0.91	*0.904*	0.92	*0.908*	0.9

Selected Points	Maximum Ponded Depth (ft) – 10 yr Storm Event					
	Existing	Alt. 1a	*Alt. 1b*	Alt. 2a	*Alt. 2b*	Alt. 5
Point A - 6th St @ W. Madison St	1.6	1.28	*1.282*	1.12	*1.116*	1.37
Point B - 6th St @ W. Adams St	1.33	1.25	*1.312*	1.17	*1.304*	1.2
Point C - 6th St @ W. Main St	0.45	0.34	*0.382*	0.25	*0.321*	0.25
Point D - TX-146 @ W. Main St	0.23	0.32	*0.234*	0.45	*0.293*	0.23
Point E - TX-146 @ W. Madison	1.03	1.04	*1.034*	1.04	*1.035*	1.03

Selected Points	Maximum Ponded Depth (ft) – 100 yr Storm Event					
	Existing	Alt. 1a	*Alt. 1b*	Alt. 2a	*Alt. 2b*	Alt. 5
Point A - 6th St @ W. Madison St	2.05	1.9	*1.913*	1.83	*1.853*	1.95
Point B - 6th St @ W. Adams St	1.6	1.53	*1.560*	1.48	*1.544*	1.55
Point C - 6th St @ W. Main St	0.72	0.68	*0.689*	0.63	*0.661*	0.44
Point D - TX-146 @ W. Main St	0.92	0.96	*0.930*	0.98	*0.941*	0.92
Point E - TX-146 @ W. Madison	1.46	1.46	*1.460*	1.46	*1.461*	1.46

01/19/18

Appendix B – Sheetflow Analysis

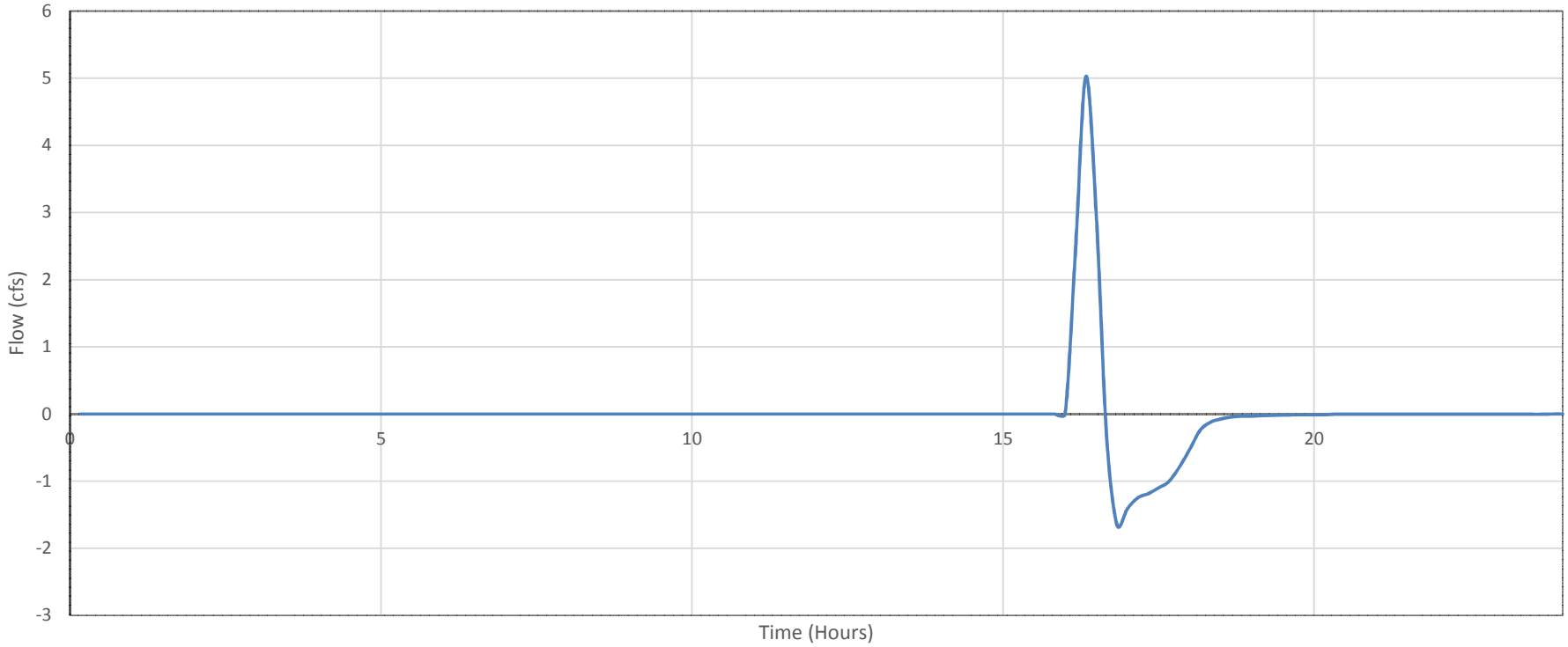


100 Year Existing Inundation Map

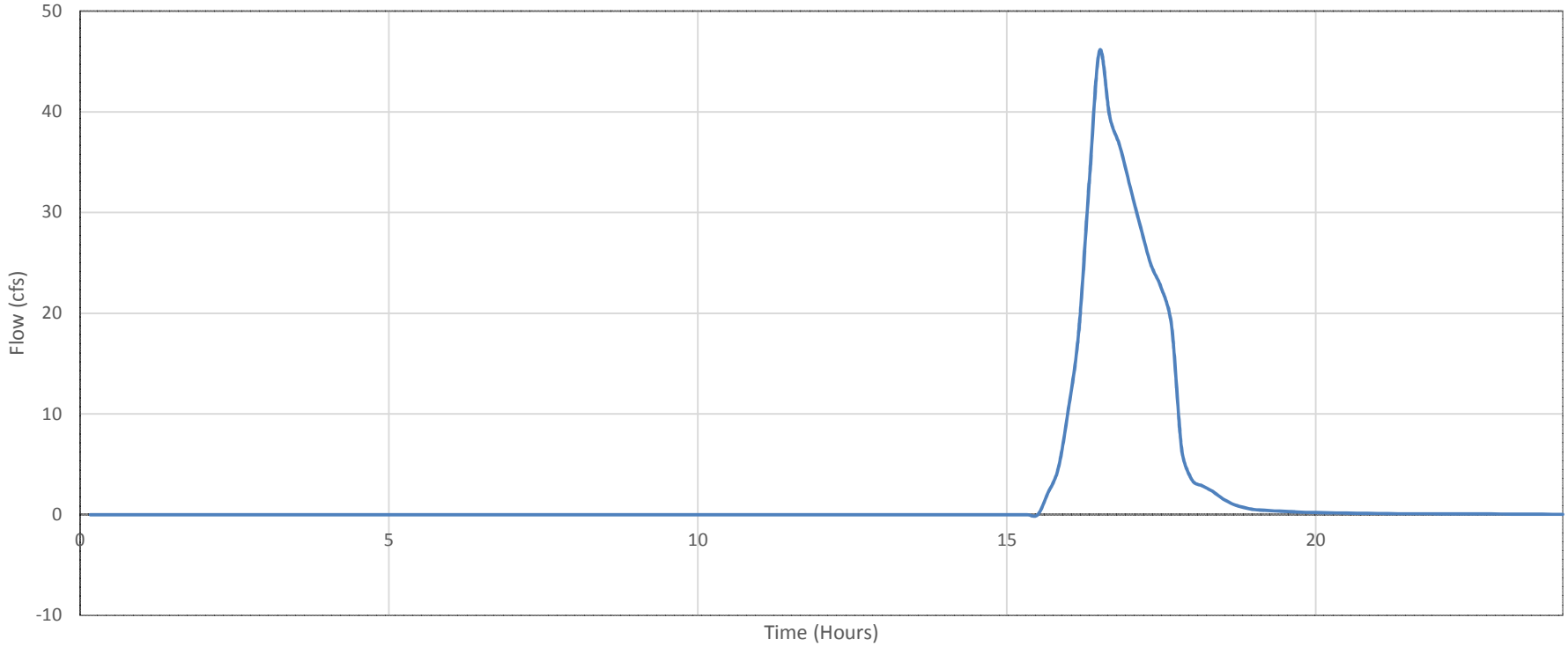
La Porte
6th Street Improvements

RPS PROJ. NO.: 007586	EXHIBIT B1
SCALE: 1 " = 300 '	
DATE: November 2017	

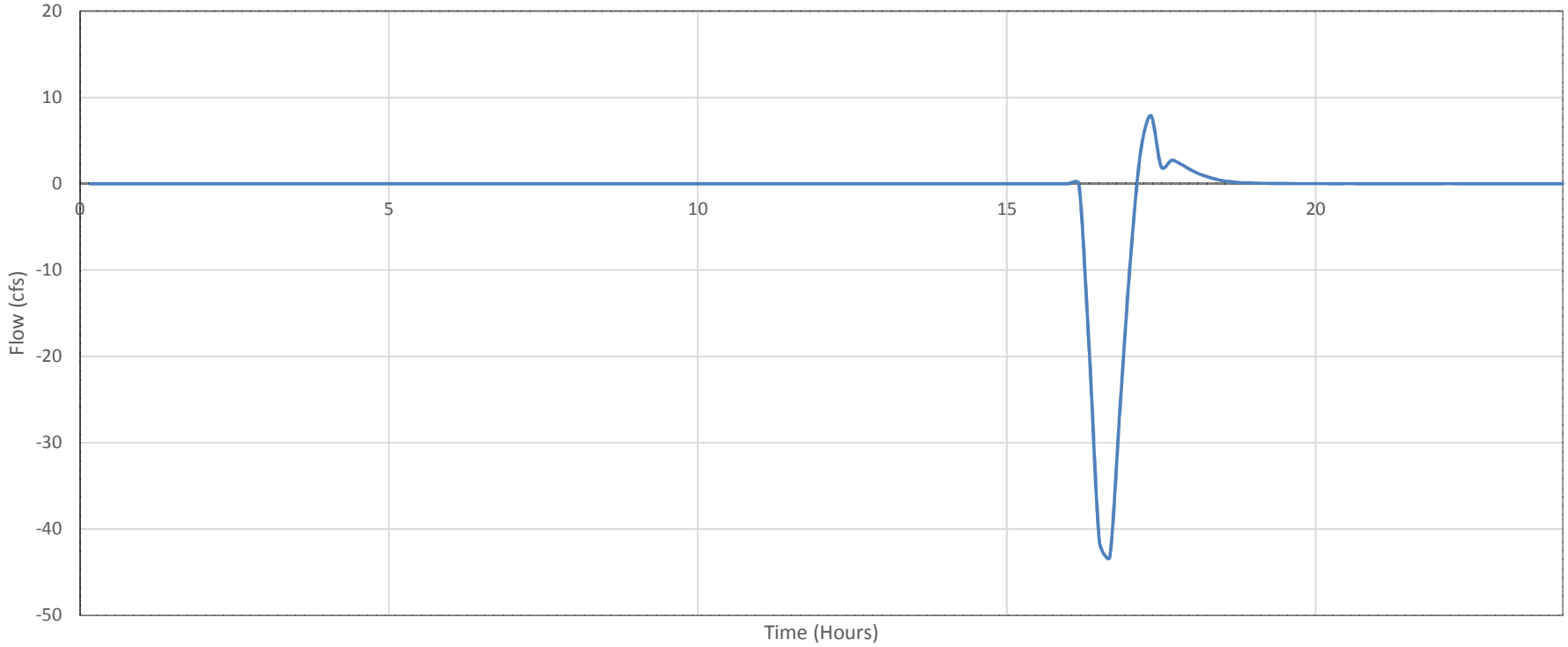
Flow Line # 1
Flow Line Location: TX-146, between Madison St and Barbours Cut Blvd
Max Flow: 5.0 cfs (Flowing North)
Max Flow: 1.7 cfs (Flowing South)
100-yr Storm Event



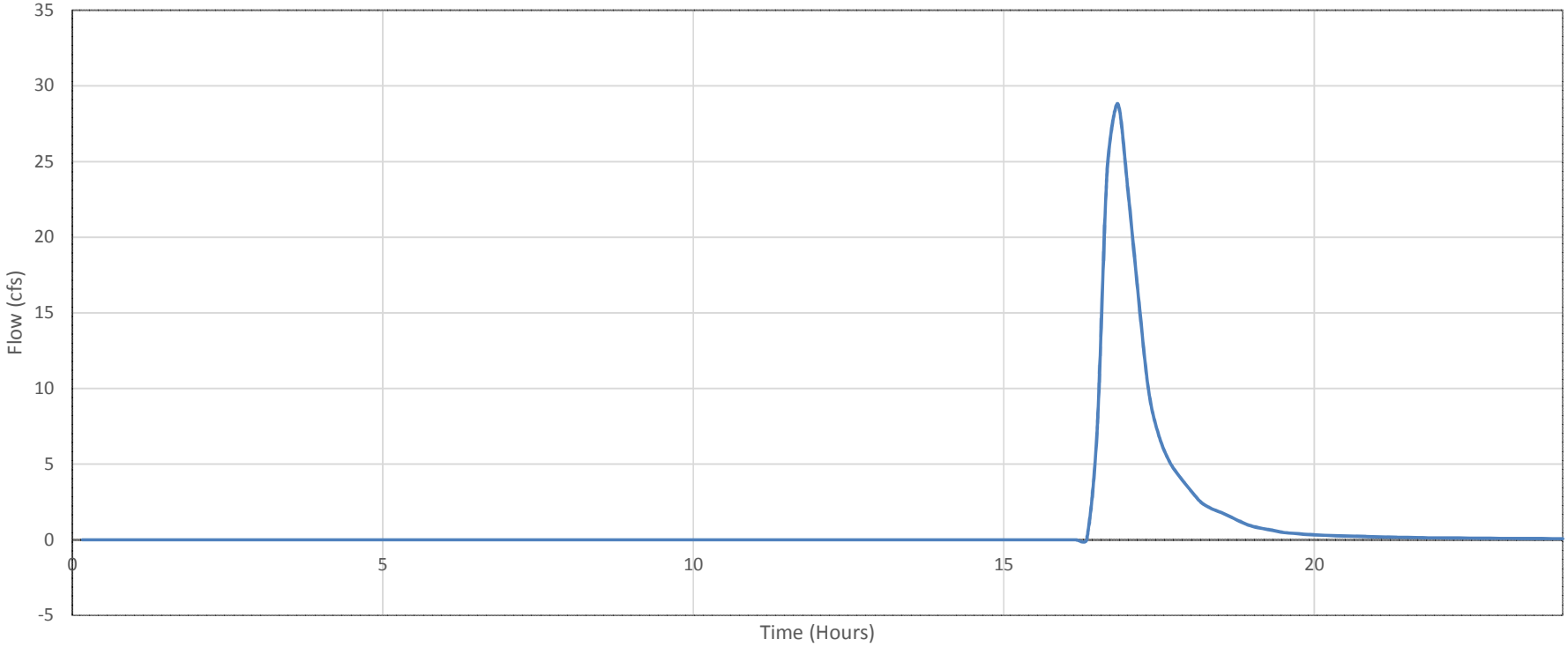
Flow Line # 2
Flow Line Location: TX-146, between Madison St and Tyler St
Max Flow: 0 cfs (Flowing North)
Max Flow: 46.0 cfs (Flowing South)
100-yr Storm Event



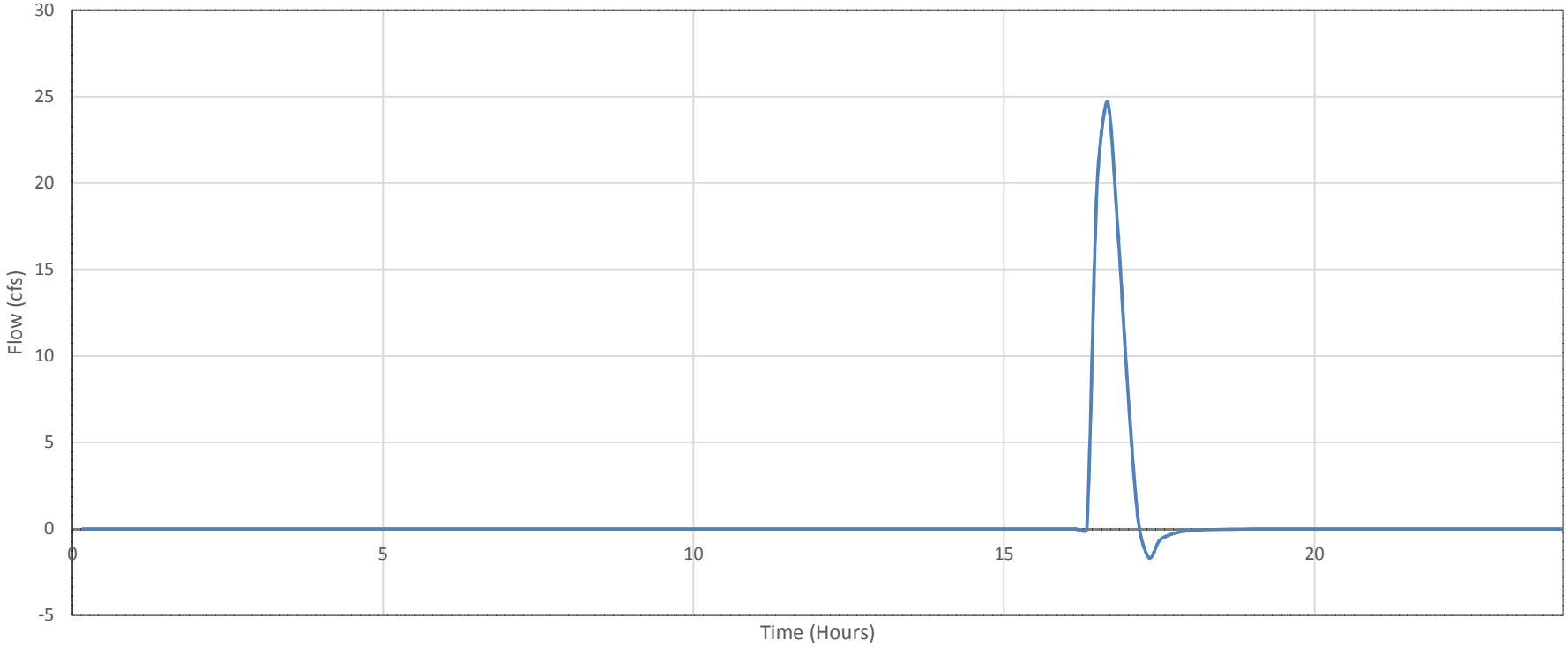
Flow Line # 3
Flow Line Location: TX-146, between Main St and Polk St
Max Flow: 7.9 cfs (Flowing North)
Max Flow: 43.3 cfs (Flowing South)
100-yr Storm Event



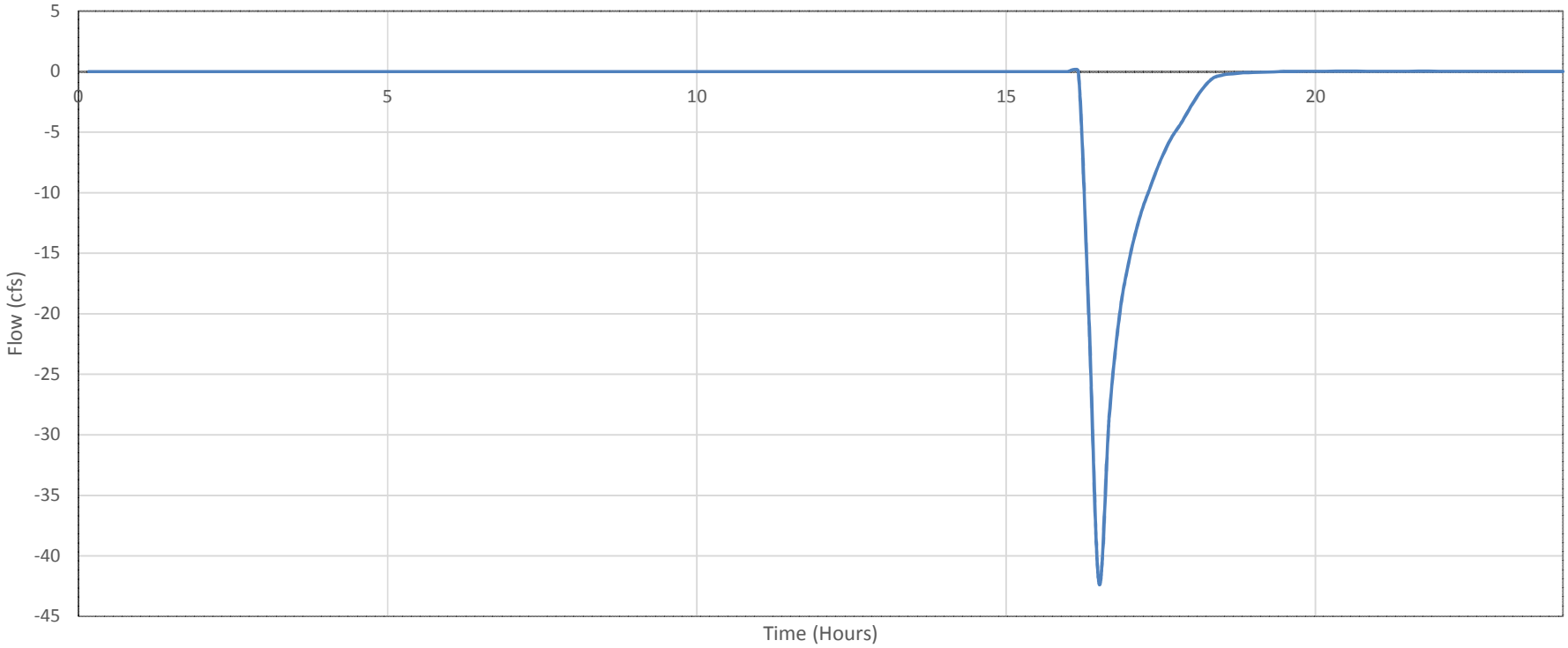
Flow Line # 4
Flow Line Location: TX-146, between Main St and A St
Max Flow: 0 cfs (Flowing North)
Max Flow: 28.8 cfs (Flowing South)
100-yr Storm Event



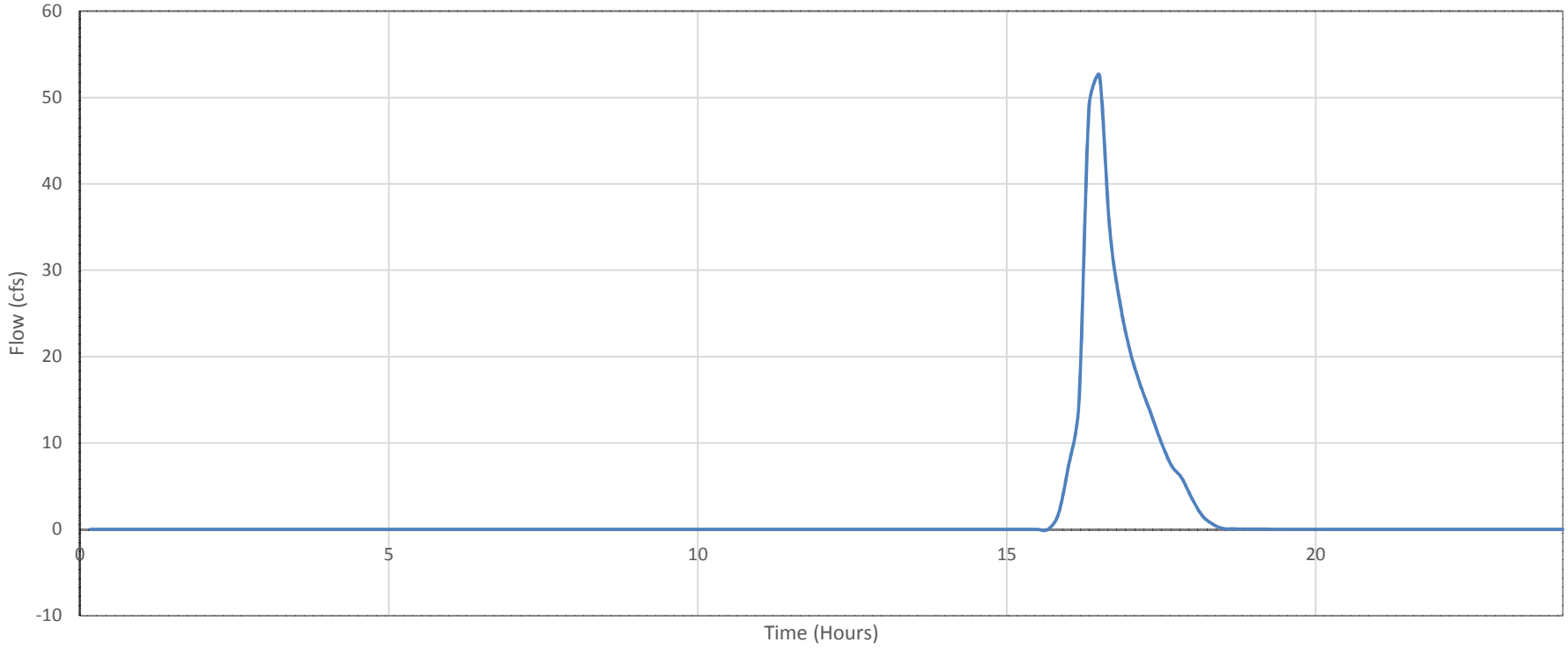
Flow Line # 5
Flow Line Location: Main St, Frontage Road West and East of TX-146
Max Flow: 24.7 cfs (Flowing West)
Max Flow: 1.7 cfs (Flowing East)
100-yr Storm Event



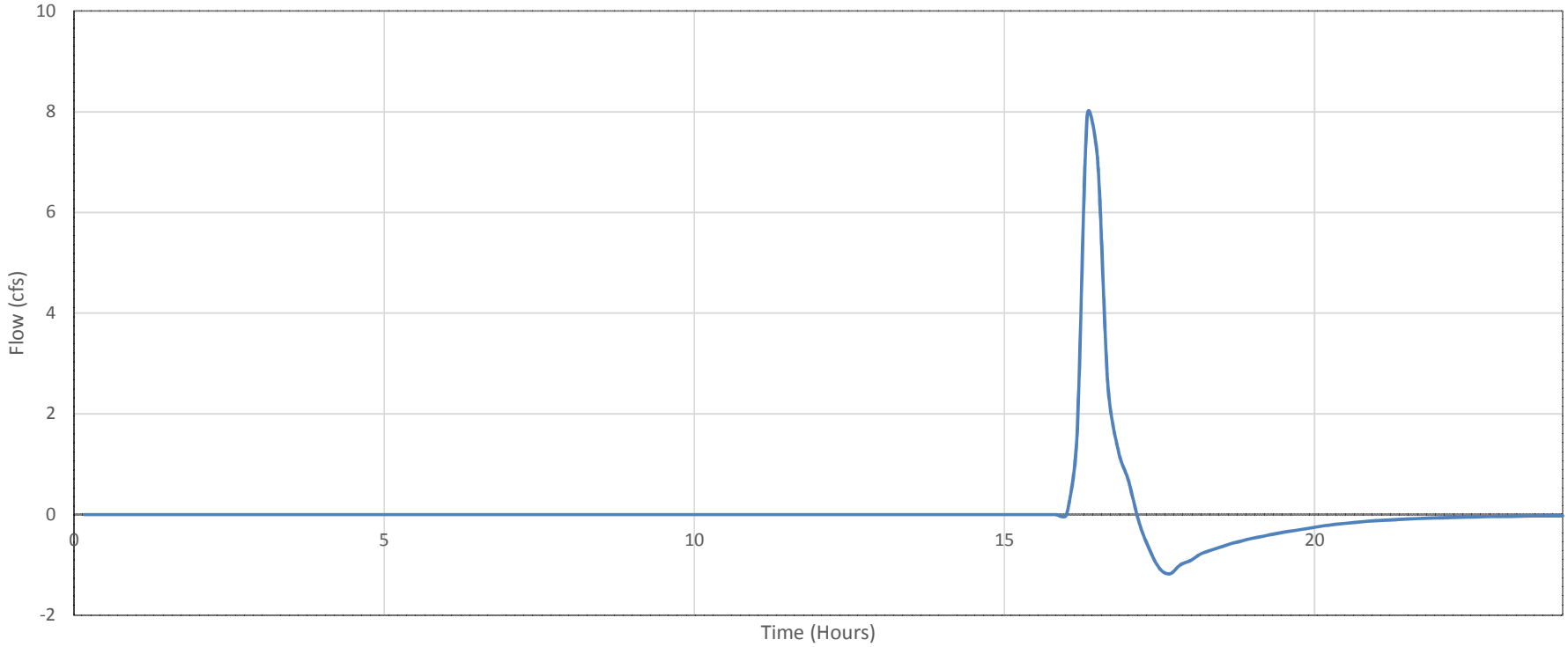
Flow Line # 6
Flow Line Location: 8th St, between Main St and A St
Max Flow: 0 cfs (Flowing North)
Max Flow: 42.3 cfs (Flowing South)
100-yr Storm Event



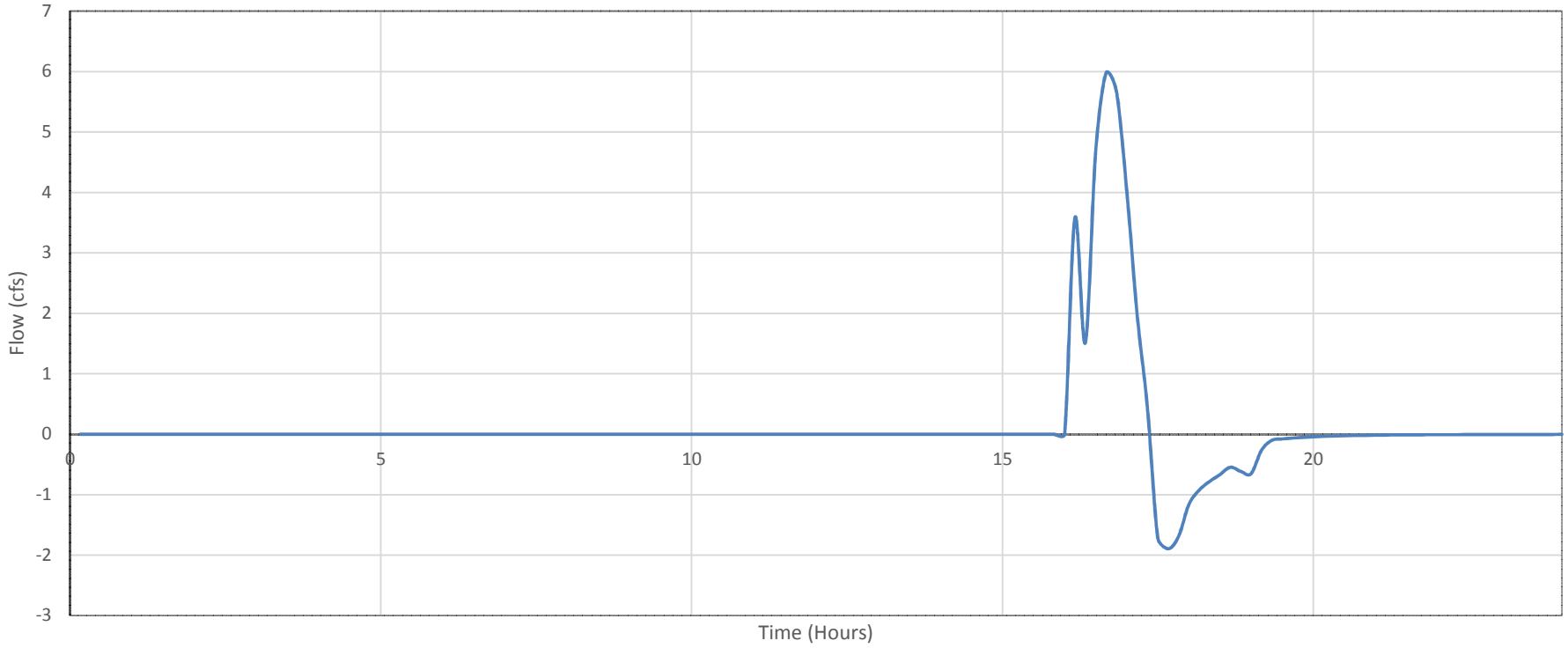
Flow Line # 7
Flow Line Location: Main St, between 6th St and 7th St
Max Flow: 52.6 cfs (Flowing West)
Max Flow: 0 cfs (Flowing East)
100-yr Storm Event



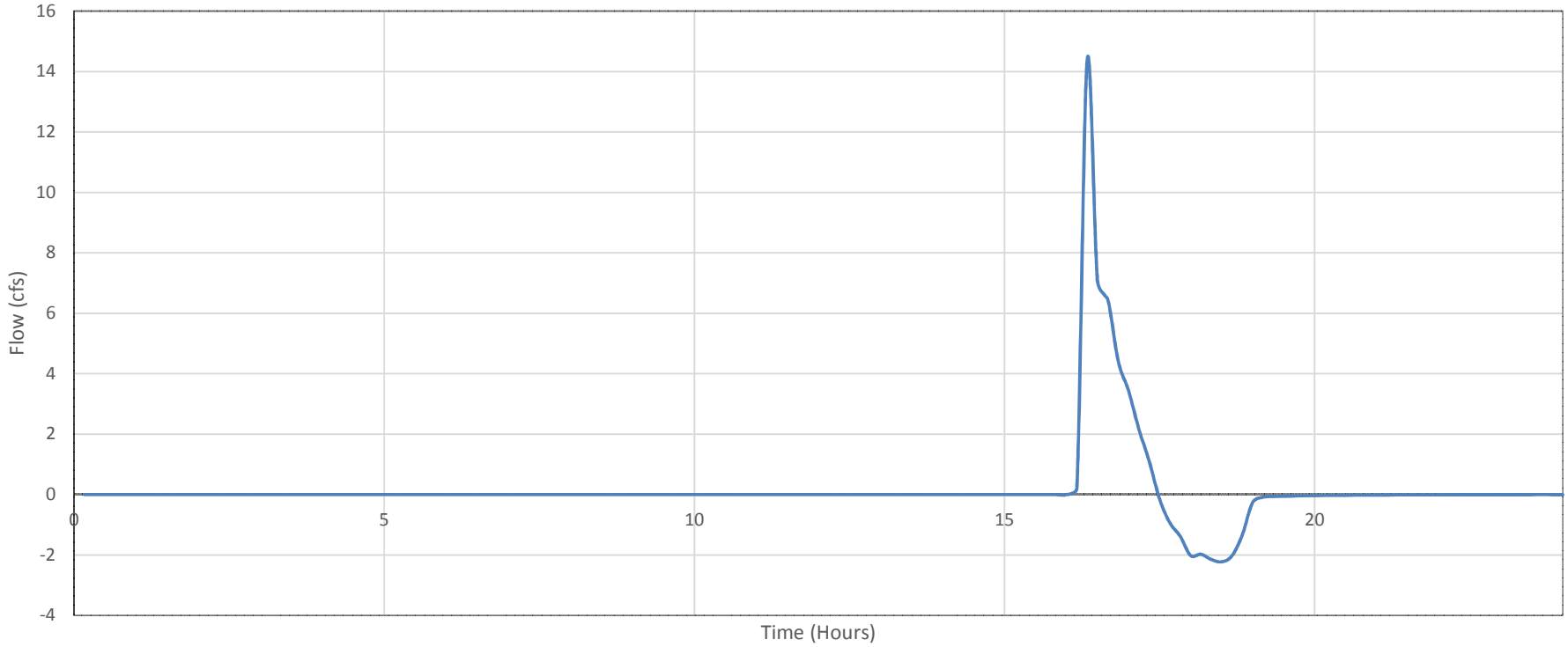
Flow Line # 8
Flow Line Location: Polk St, between 6th St and 7th St
Max Flow: 7.9 cfs (Flowing West)
Max Flow: 1.2 cfs (Flowing East)
100-yr Storm Event



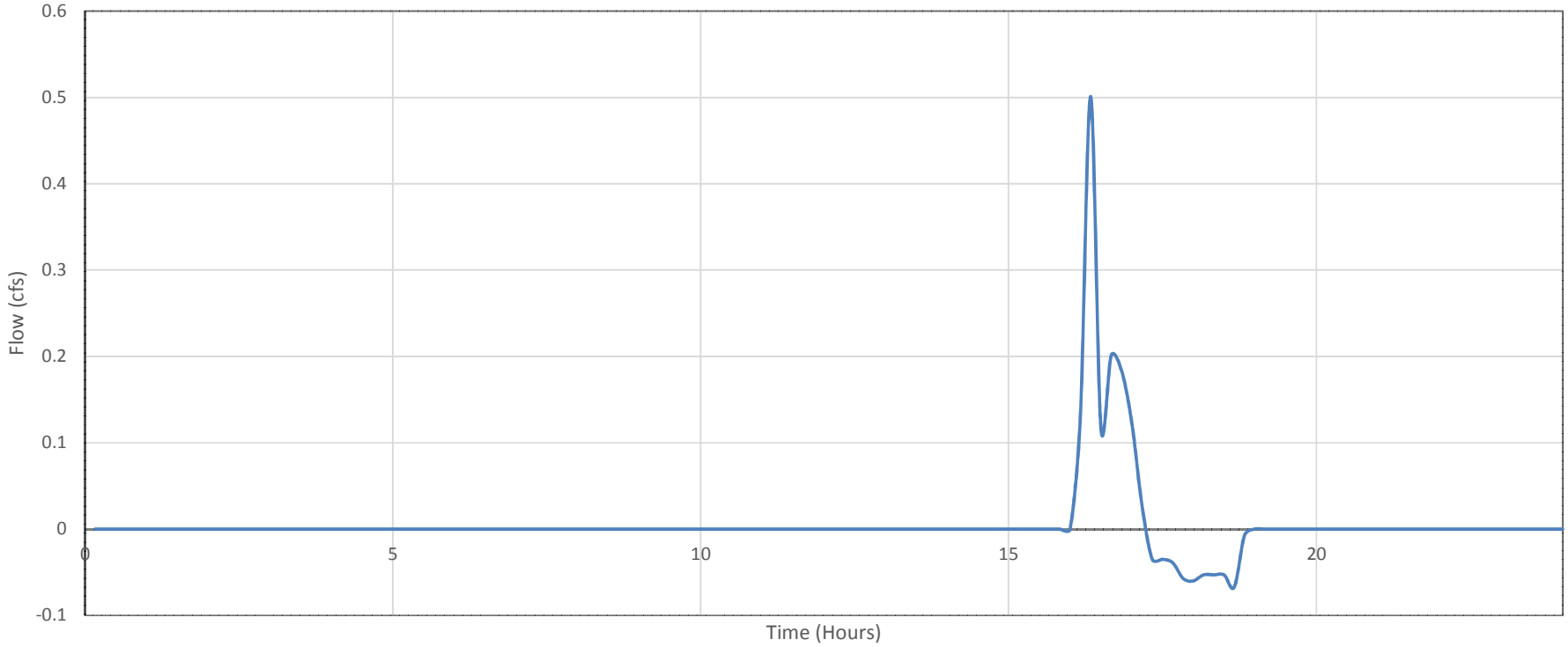
Flow Line # 9
Flow Line Location: Adams St, between 6th St and 7th St
Max Flow: 6.0 cfs (Flowing West)
Max Flow: 1.9 cfs (Flowing East)
100-yr Storm Event



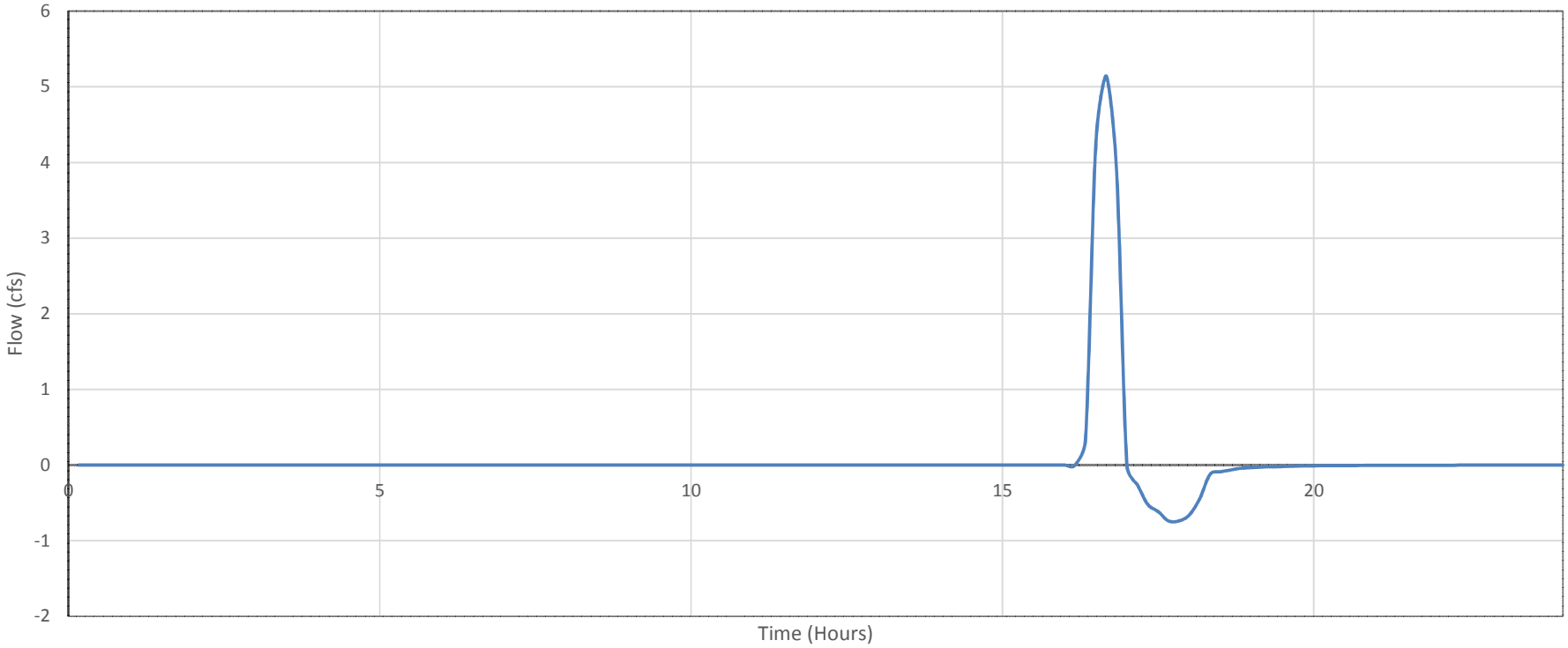
Flow Line # 10
Flow Line Location: Tyler St, between 6th St and 7th St
Max Flow: 14.4 cfs (Flowing West)
Max Flow: 2.2 cfs (Flowing East)
100-yr Storm Event



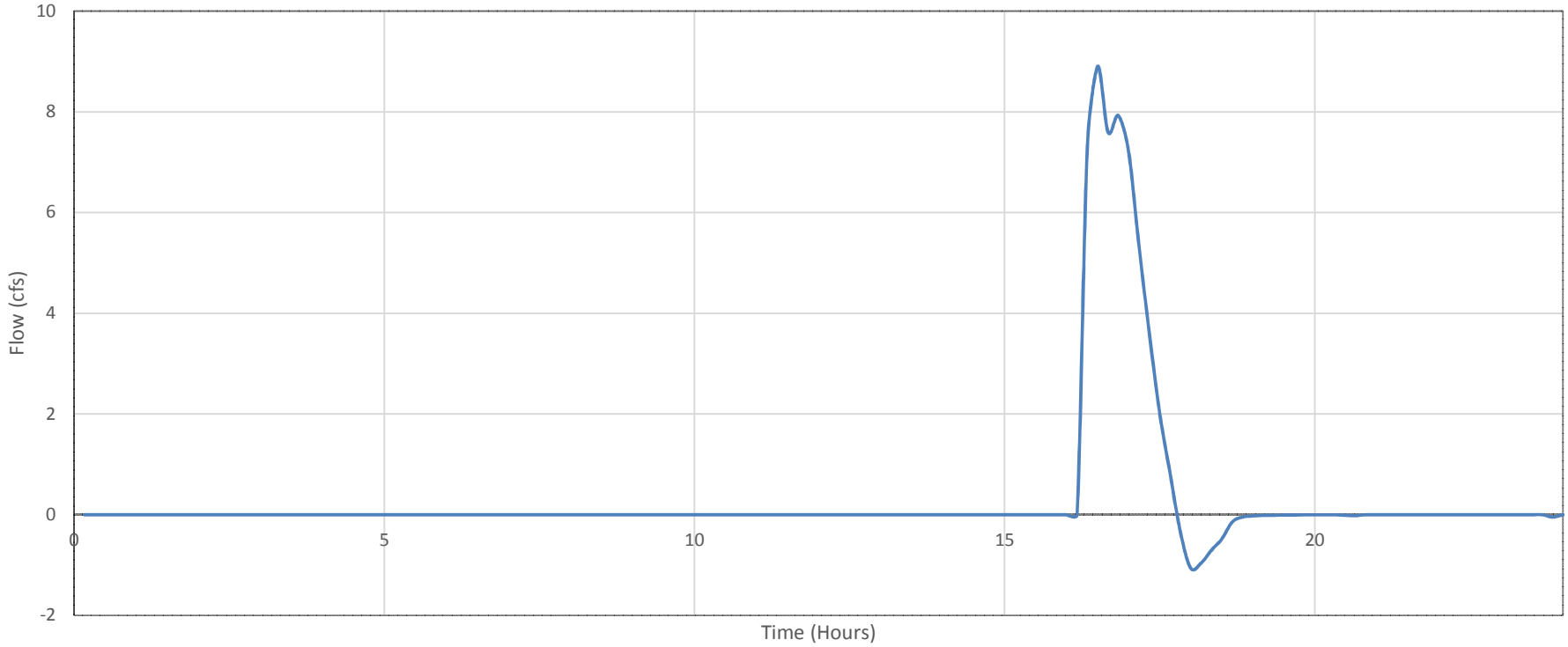
Flow Line # 11
Flow Line Location: Madison St, between 6th St and 7th St
Max Flow: 0.5 cfs (Flowing West)
Max Flow: 0.1 cfs (Flowing East)
100-yr Storm Event



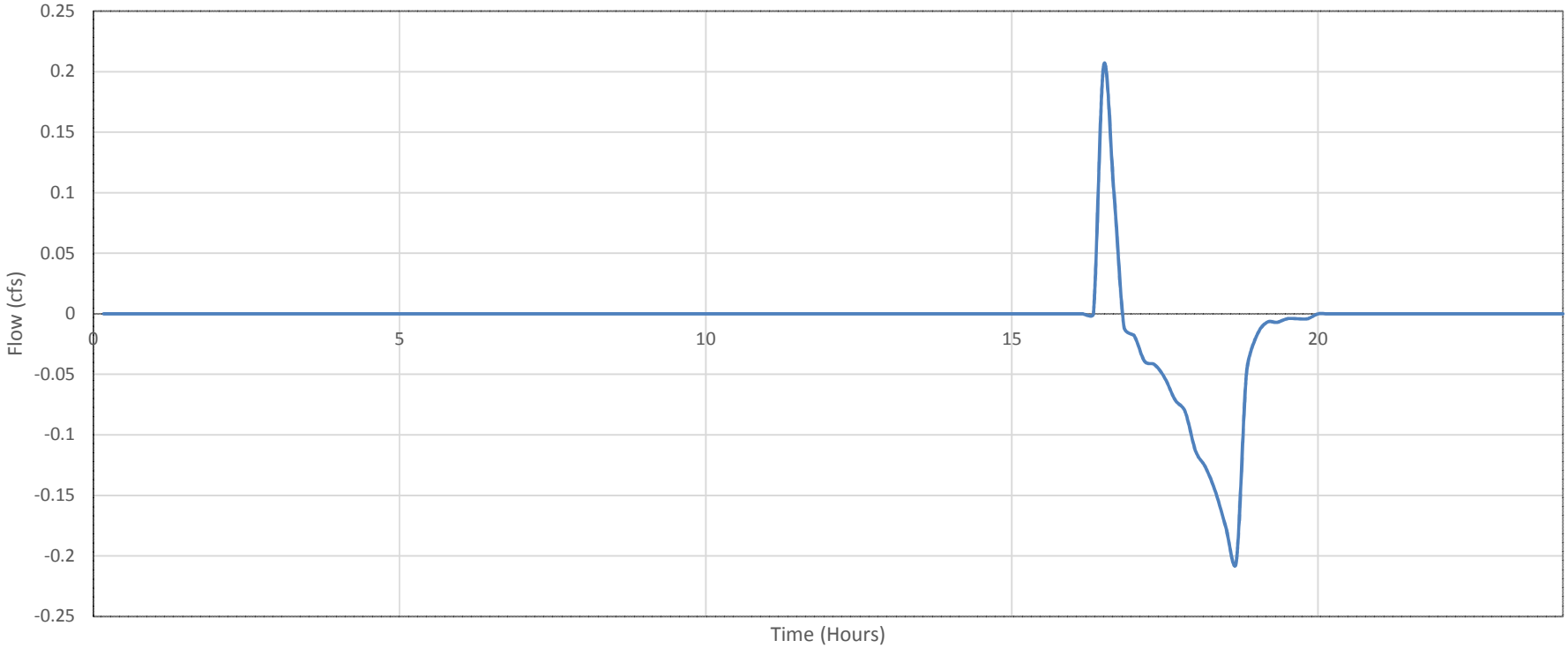
Flow Line # 12
Flow Line Location: 6th St, between Madison St and Barbours Cut Blvd
Max Flow: 5.1 cfs (Flowing North)
Max Flow: 0.7 cfs (Flowing South)
100-yr Storm Event



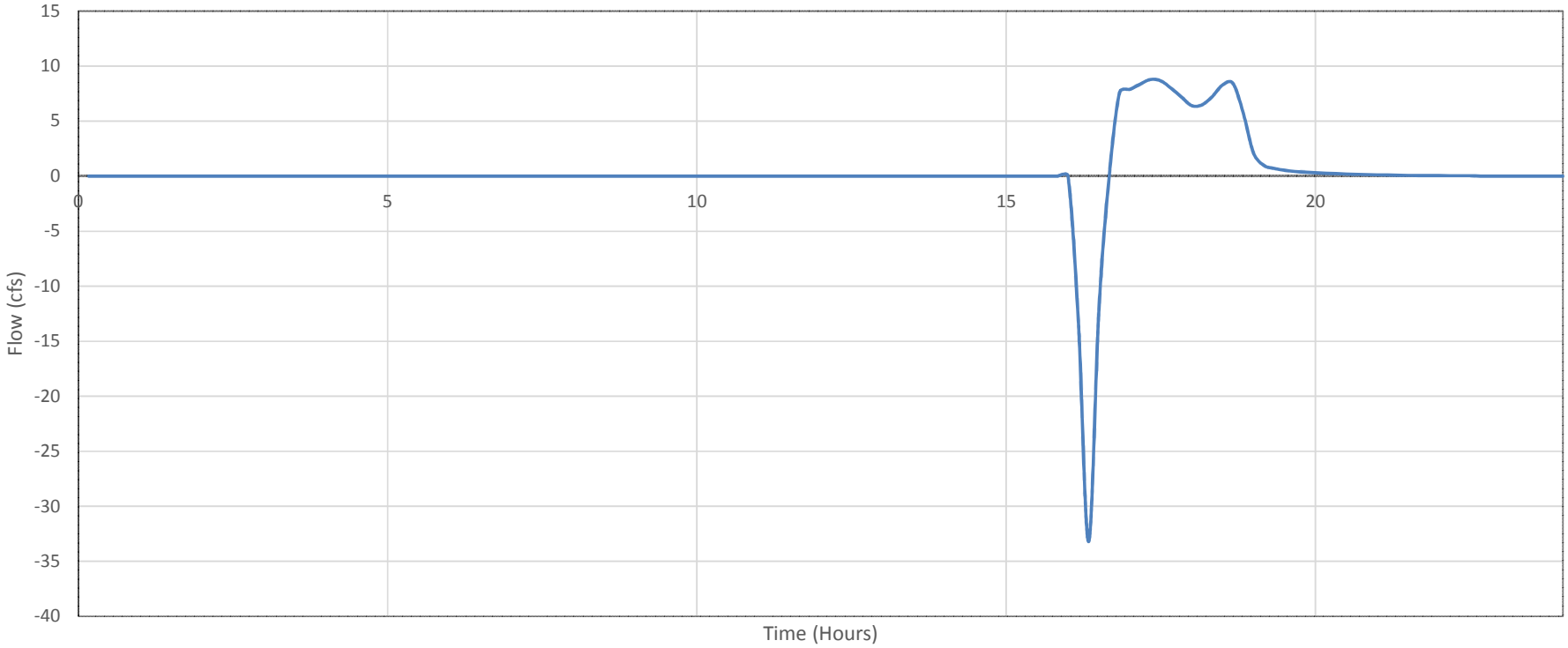
Flow Line # 13
Flow Line Location: Madison St, between 6th St and 5th St
Max Flow: 1.1 cfs (Flowing West)
Max Flow: 8.9 cfs (Flowing East)
100-yr Storm Event



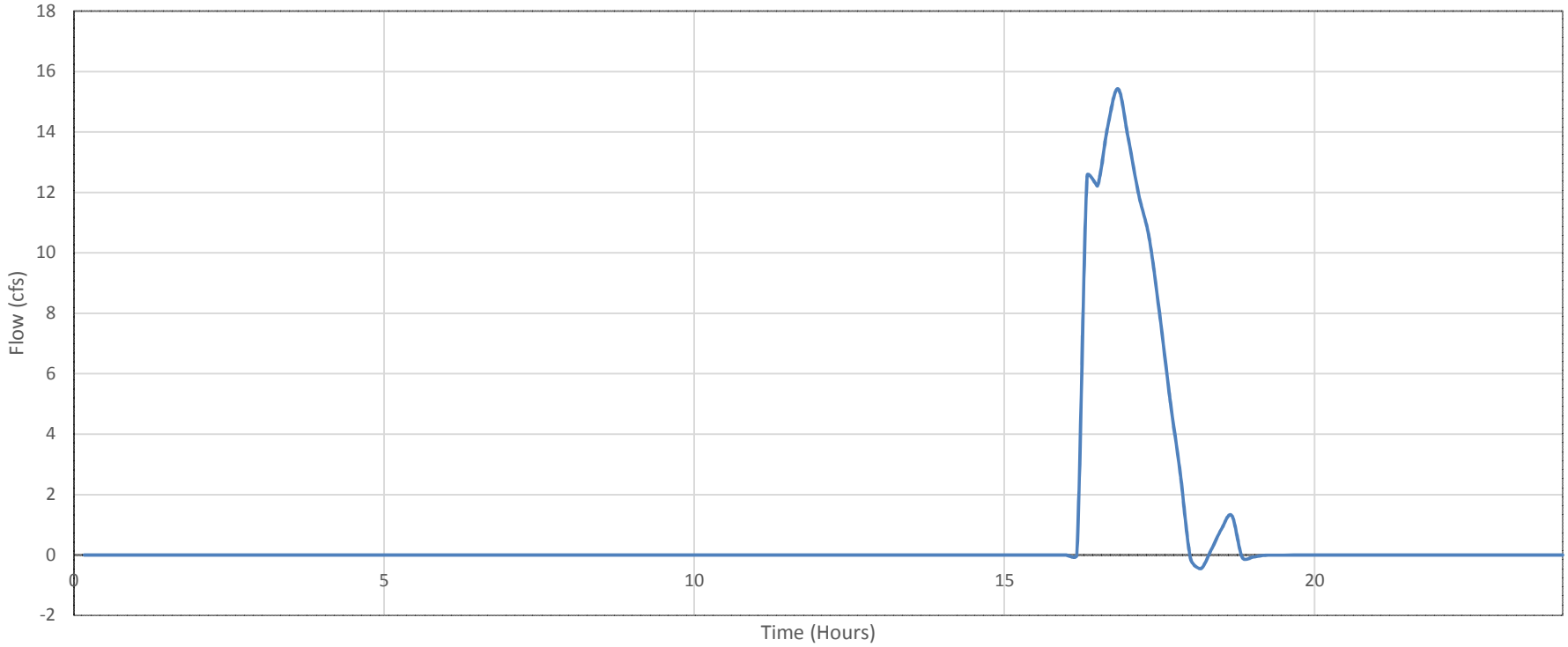
Flow Line # 14
Flow Line Location: Tyler St, between 6th St and 5th St
Max Flow: 0.2 cfs (Flowing West)
Max Flow: 0.2 cfs (Flowing East)
100-yr Storm Event



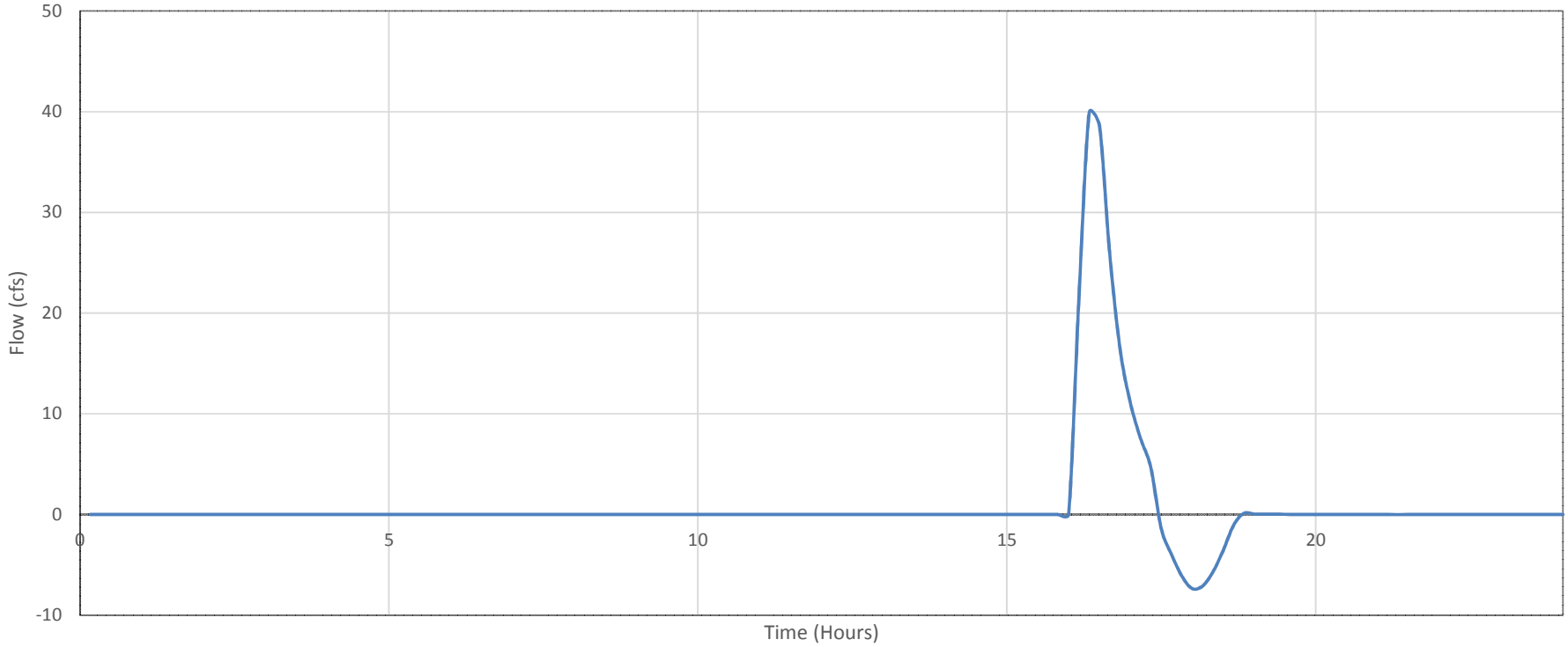
Flow Line # 15
Flow Line Location: 6th St, between Tyler St and Madison St
Max Flow: 8.7 cfs (Flowing North)
Max Flow: 33.2 cfs (Flowing South)
100-yr Storm Event



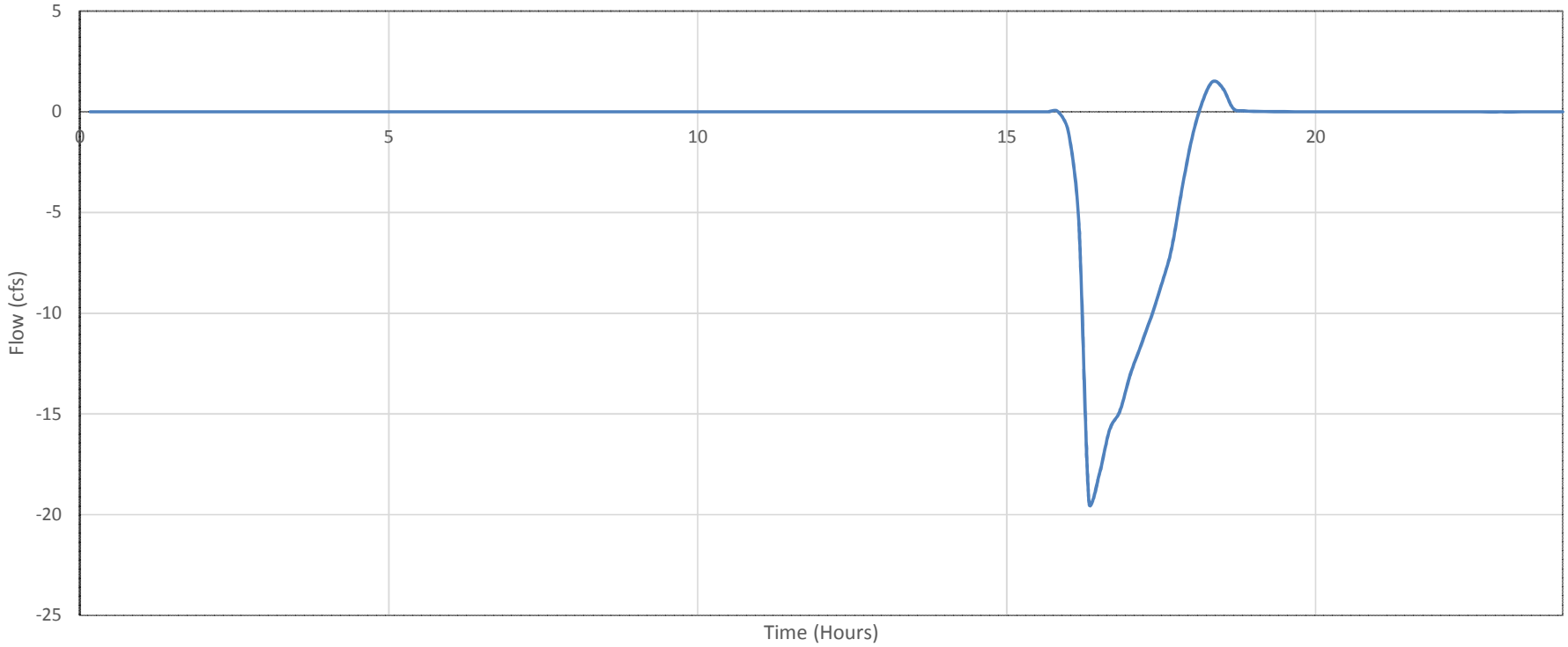
Flow Line # 16
Flow Line Location: 6th St, between Tyler St and Adams St
Max Flow: 15.4 cfs (Flowing North)
Max Flow: 0.4 cfs (Flowing South)
100-yr Storm Event



Flow Line # 17
Flow Line Location: 6th St, between Polk St and Adams St
Max Flow: 39.9 cfs (Flowing North)
Max Flow: 7.1 cfs (Flowing South)
100-yr Storm Event



Flow Line # 18
Flow Line Location: 6th St, between Polk St and Main St
Max Flow: 1.2 cfs (Flowing North)
Max Flow: 19.4 cfs (Flowing South)
100-yr Storm Event



KSA A·DYNAMIC·PERSPECTIVE

816 Park Two Drive
Sugar Land, TX 77478
281.494.3252

6-15-18

TO: Mr. Lorenzo Wingate, P.E., C.F.M.

FROM: Chris Wilde, P.E., C.F.M.

SUBJECT: Harris County Flood Control Channel F101-06-00
Lomax Area Drainage Improvements Feasibility Report

DATE: June 15, 2017

STATE OF TEXAS
CHRISTOPHER J. WILDE
111787
LICENSED
PROFESSIONAL ENGINEER

The purpose of this letter report is to propose alternatives to improve drainage across the Exxon Pipeline Corridor, provide a cost for those improvements and make recommendations.

Background

Landev Engineers performed two studies of Harris County Flood Control District's (HCFCD) Unit F101-06-00 in 1987 and 2013. Klotz Associates completed a city wide drainage study in 2009.

Alternative 1 below of the 1987 masterplan was concluded to be the most feasible masterplan for Unit F101-00-00. The 1987 study by Landev was a masterplan that identified four alternatives:

1. Construction of Channel Improvements to contain the 100-yr
2. Detention Storage to limit the 100-yr flood discharges to the existing capacity of the Southern Pacific Railway Yard Culverts
3. Diversion of the 550 acres west of the Exxon Pipeline Corridor to B106-00-00
4. Diversion of 1100 acres to B106-00-00 to limit the 100-yr flood discharges to the existing capacity of the Southern Pacific Railway Yard Culverts

The *Conveyance Improvements Across the ExxonMobil Pipeline Corridor* study performed in 2013 limited analysis to 100 feet up and downstream of the pipeline corridor. The pipe depths in the ExxonMobil Corridor were surveyed at three different locations. The flows used in the analysis were taken from Discharge Computation Point 6 that is located at the Pipeline Corridor on the original Drainage Area Map provided as Exhibit 1. The 2013 study used flows of 1,326 cfs for the 100-year and 1,169 cfs for the 25-year from the 1987 Drainage Masterplan provided in Exhibit 2. The flows from the 1987 Drainage Masterplan were based on a 583 acre drainage area that was 100% developed. The 1987 masterplan flows were derived from area-discharge curves developed by HCFCD's Armand Bayou Flood Hazard Study performed in 1983 and provided in Exhibit 3. The 2013 study recommended (3) 48 inch culverts and a 40-foot bottom width overflow channel. The 2013 study analyzed the following alternatives:

1. High flow concrete channel and conduit under the pipelines to handle low flows.
2. Detention with gravity or pumped flow across the corridor.
3. Existing channel depth and widen upstream and downstream to convey the design flow.
4. Addition of siphon to high flow concrete channel and low flow conduit.

816 Park Two Drive
Sugar Land, TX 77478
281.494.3252

The 2009 City Wide Drainage Study by Klotz recommended detention at the ExxonMobil Energy Corridor and channel improvements to F101-03-00 that is adjacent to L Street.

Improvements have been constructed up to the drop structure for Unit F101-00-00 and lateral channels have been added to F101-06-00 as seen in Exhibit 4. KSA determined 3 alternatives to convey flows across the corridor.

Limitations

Coordination between the City of La Porte, HCFCD and the pipeline companies will be critical to determining what improvements will be allowed to be constructed.

- HCFCD requires a maintenance berm and back slope interceptor within the channel ROW. The ROW for Unit F101-06-00 currently is 60-foot. A channel sized to pass the 100-yr flow of 1,326 cfs as per HCFCD requirements will not fit within the existing 6-foot ROW. Houses east of Valley View are constructed up to the ROW.
- The channel depth is restricted by the depth of the pipes in the ExxonMobil Corridor. The pipeline companies require two feet of clearance between channel improvements and their pipes in the corridor. The survey at Section B-B shows the flow line of the existing channel to be at elevation 16.1 feet, 1.3 feet from Pipe 165A at 14.8 feet. The proposed alternatives maintain the existing flowline over the corridor as it is already not in compliance with the pipeline companies' requirements.
- HCFCD requires the minimum channel slope to be 0.05% and the maximum velocity to be from 6 to 12 feet per second (fps) depending on the channel type.

Alternatives

The following alternatives were analyzed in previous studies: routing a diversion channel around the corridor, detaining flows beyond the capacity of the channel and installing a siphon. Installing pumps, whether it be a pump station or Archimedes screw pumps to increase flow across the corridor would be an additional cost of approximately $4 million to the required channel improvements required to convey the 100-year flow within the right-of-way. Pumps can be used to bypass high flows or if the channel was deepened upstream of the corridor they could pump what remained after a rain event.

KSA determined three alternative designs for the reach of F101-06-00 spanning from Valley View Drive beyond the ExxonMobil pipeline corridor to the drop structure. These alternatives convey the 100-year flow of 1,326 cfs over the ExxonMobil pipeline corridor to Unit F101-00-00. The alternatives are as follows:

1. The entire reach from Valley View to the drop structure will be lined with concrete. The channel will be widened to maximize the capacity of the 60-foot ROW. This channel will not convey the 100-yr. The channel will not be deepened and the existing flow line will be maintained. The channels are trapezoidal with 2:1 side slopes.
2. The entire reach from Valley View to the drop structure will be lined with concrete. The channel will be widened beyond the ROW to convey the 100-yr and allow for maintenance berms. The existing flow line will be maintained.

816 Park Two Drive
Sugar Land, TX 77478
281.494.3252

3. The reach from Valley View to the ExxonMobil corridor will be concrete lined and grass lined to the drop structure. The concrete section is essential to maximizing the conveyance of the 100-year beyond houses at Valley View. The grass lined channel will require widening the ROW to allow for the maintenance berm and back slope interceptor.

Recommendations

Adequately sizing Unit F101-06-00 to convey the 100-year storm is dependent on the design flow. A hydrologic analysis should be performed to determine a more accurate 100-year flow in Unit F101-06-00 at the ExxonMobil pipeline corridor to avoid oversizing the channel. The calculated 100-year flow should be used to size the Unit F101-06-00. Hydrogeomorphologic characteristics of the channel are evidence that the existing channel is not overwhelmingly undersized. The channel is stable and is not down cutting or widening. The existing channel does not have a lot of erosion taking place.

The 100-year discharge of 1,362 cfs was determined in the 1983 Armand Bayou Flood Hazard Study by HCFCD. The 1983 hydrology would be affected by development and more recent rainfall data. The land use that was used to calculate the hydrology for the ultimate development in 1983 most likely has changed. HCFCD's hydrologic methodologies have been updated from reading the discharge from a discharge-area curve to modeling it in HEC-HMS. Rainfall is modeled using a hyetograph to distribute the rainfall over time and the Green and Ampt method is used to account for losses. New delineations of subbasins to determine exactly what runoff flows to the corridor would be beneficial as development has installed new infrastructure and channel improvements contained in the 1987 masterplan have not yet been constructed. Unit F101-06-02 has not been constructed and should not be constructed. The area should continue to be conveyed by P Street which will take it beyond the corridor. The northern boundary appears to be P Street for the drainage area that flows to the corridor. A high level delineation by KSA found the watershed to be about 310 acres versus 583 acres used in the 1987 masterplan as can be seen in Exhibit 5. Exhibit 5 contains KSA's delineations in purple and an approximate delineation from the 1987 masterplan in red. Drainage channel F-101-06-01 conveys flows downstream of the corridor to the drop structure.

The survey of the pipes located in the ExxonMobil Pipeline Corridor show the depth of the pipes to increase on the south side of the ROW as can be seen in Exhibit 6 and 7. Additional survey to the south of the ROW should be performed to determine if the pipe are buried deeper in this location. If so, the channel could be moved south to allow for a deeper channel depth to increase the capacity of the channel. Deepening the channel at the corridor will depend on the tailwater on F101-06-00 or at the drop structure.

Summary

Unit F101-06-00 has a capacity of less than 200 cfs currently before it rises out of its banks. Alternative 1 will excavate the full 60-foot ROW and construct a concrete channel that will maximize the capacity of the current ROW. That capacity is 700 cfs without deepening the channel. Alternative 1 is depicted on Exhibit 8 and the cross sections as Exhibit 11.

816 Park Two Drive
Sugar Land, TX 77478
281.494.3252

Alternative 2 maintains a concrete channel from Valley View to the drop structure but widens the channel beyond the ROW to convey the 100-yr (1,326 cfs). This channel maintains the existing flow line of the channel. Alternative 2 is depicted on Exhibit 9 and the cross sections as Exhibit 12.

Alternative 3 maintains concrete from Valley View to the ExxonMobil Pipeline Corridor to convey the maximum amount beyond the houses at Valley View. The reach from the corridor to the drop structure is grass lined and the channel was deepened one foot over the corridor. Alternative 3 is depicted on Exhibit 10 and the cross sections as Exhibit 13.

Coordination between the City of La Porte, HCFCD and the pipeline companies will be critical to develop what improvements will be allowed to be constructed. All channel design alternatives analyzed in this report will require variances and/or additional ROW to be acquired.

The hydrology should be recalculated for the corridor watershed to ensure the channel being designed is not oversized. Additional pipeline survey should be taken to the south of the ROW as it appears the pipes are buried deeper in that location which would allow the channel to be shifted south and constructed deeper. One cannot assume that the deeper the channel is the greater the conveyance capacity will be. One must model the hydrology and hydraulics of the whole system and account for the tailwater of Unit F101-00-00 (below the drop structure) on Unit F101-06-00 which will reduce the capacity of the channel.

The costs are provided as Exhibit 14-16.

Alternative	Project Cost
1	$2.4 M
2	$3.2 M
3	$1.5M

Exhibit 1

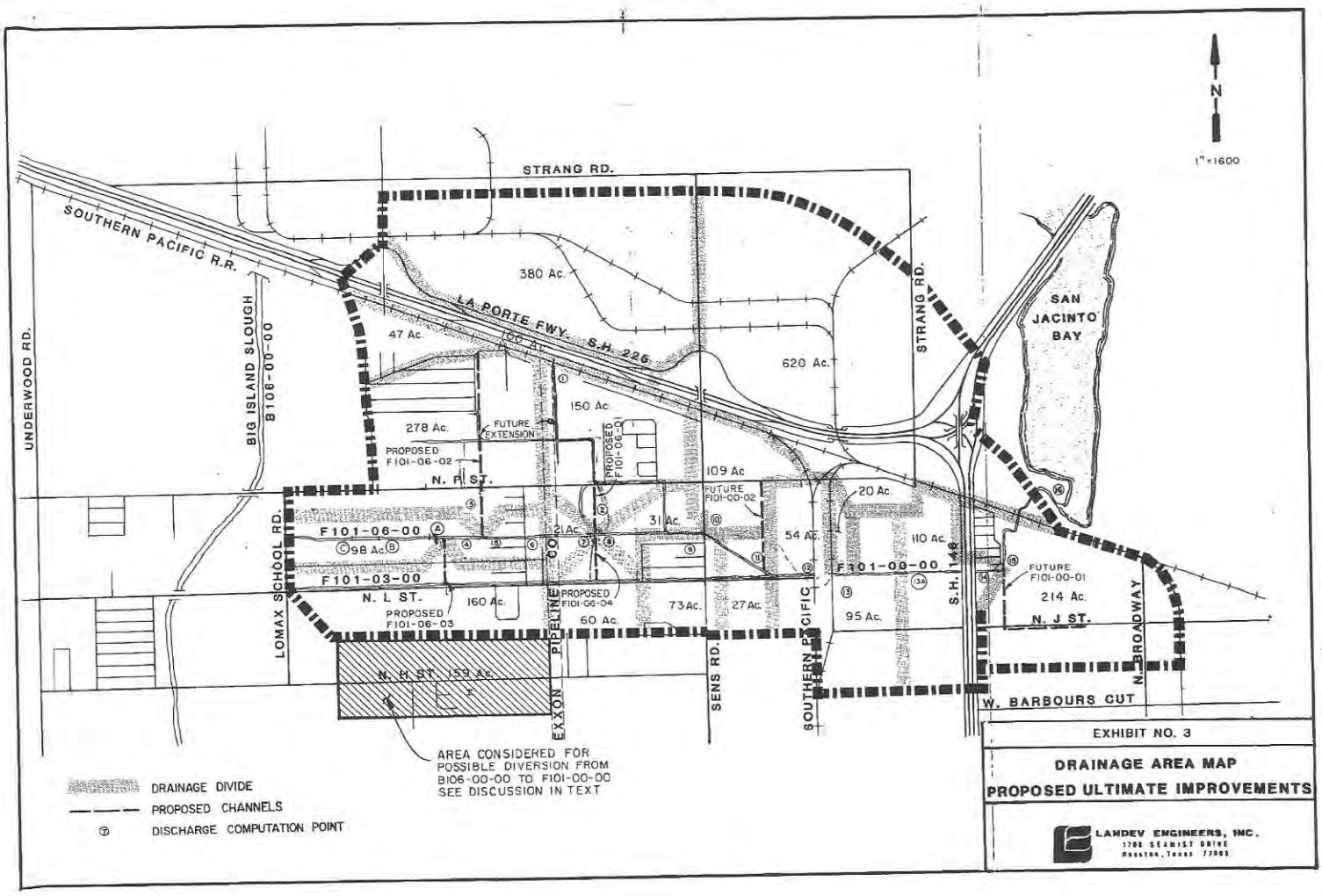

N
1" = 1600
STRANG RD.
SOUTHERN PACIFIC R.R.
UNDERWOOD RD.
BIG ISLAND SLOUGH
B106-00-00
LA PORTE FWY. S.H. 225
CO. RT.
380 Ac.
47 Ac.
620 Ac.
150 Ac.
278 Ac.
FUTURE EXTENSION
PROPOSED F101-06-02
PROPOSED F101-06-01
N. R ST.
109 Ac
FUTURE F101-00-02
20 Ac.
31 Ac.
21 Ac.
54 Ac.
110 Ac.
F101-06-00
98 Ac.
F101-03-00
N. L ST.
160 Ac.
PROPOSED F101-06-03
PROPOSED F101-06-04
60 Ac.
73 Ac.
27 Ac.
95 Ac.
F101-00-00
FUTURE F101-00-01
214 Ac.
N. J ST.
LOMAX SCHOOL RD.
EXXON PIPELINE CO.
SENS RD.
SOUTHERN PACIFIC
S.H. 146
STRANG RD.
SAN JACINTO BAY
N. BROADWAY
W. BARBOURS CUT
N. H ST. 159 Ac.
AREA CONSIDERED FOR POSSIBLE DIVERSION FROM B106-00-00 TO F101-00-00 SEE DISCUSSION IN TEXT
DRAINAGE DIVIDE
PROPOSED CHANNELS
DISCHARGE COMPUTATION POINT
EXHIBIT NO. 3
DRAINAGE AREA MAP
PROPOSED ULTIMATE IMPROVEMENTS
LANDEV ENGINEERS, INC.

Exhibit 2

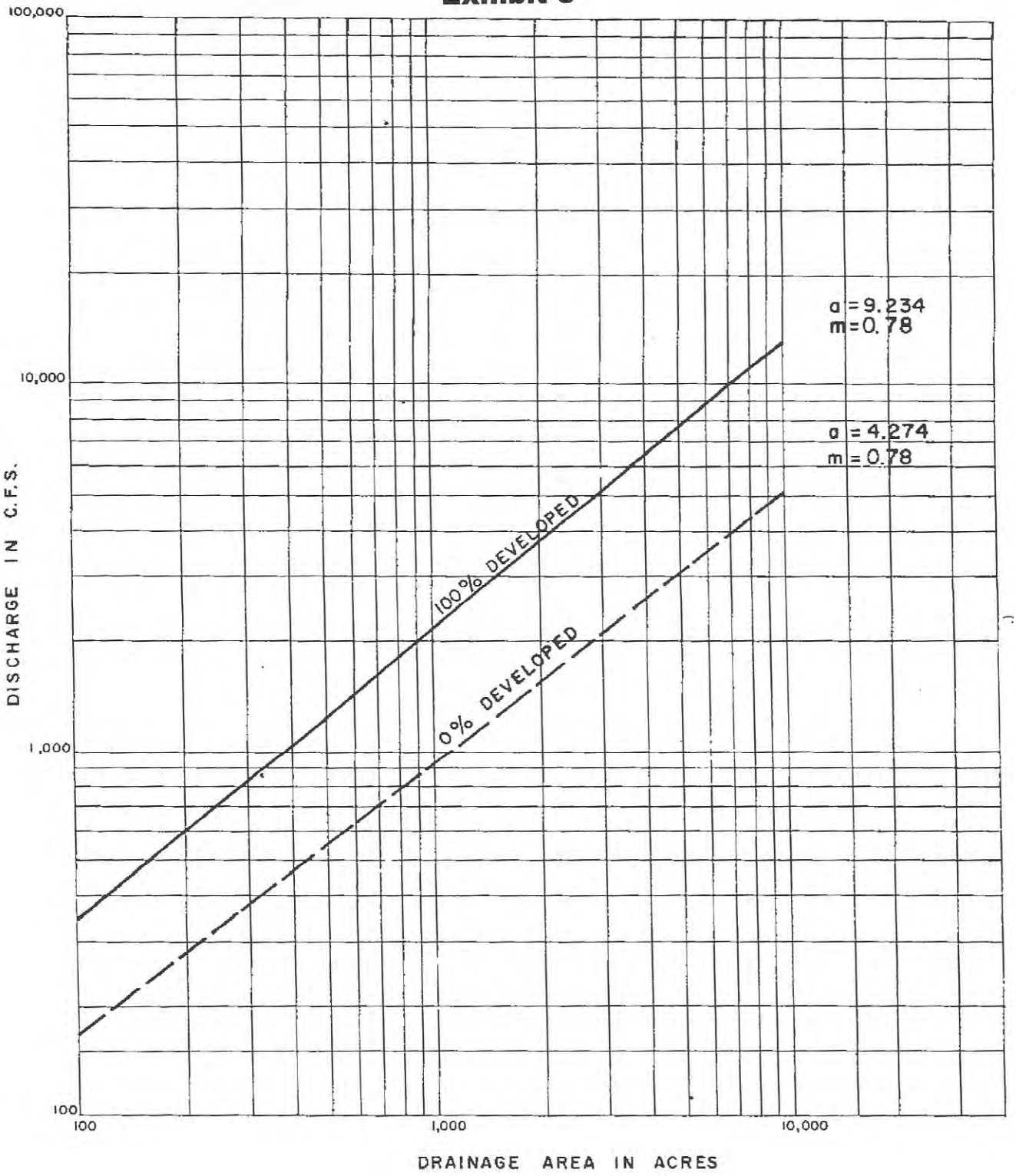
TABLE NO. 1: Drainage Areas and Discharge Computations for the Unit F101-00-00
Alternatives Analyzed

Location	Description	Channelization				Interim Improvements				Detention Storage			Diversion 1100 Acres			Diversion 550 Acres		
		DA	% Devd	100 Year	25 Year	DA	% Devd	25 Year	100 Year	DA	% Devd	100 Year	DA	% Devd	100 Year	DA	% Devd	100 Year
1	@ 225	480	7	570	446	480	7	446	570	527	28	752						
2	Outfall F101-06-01	630	40	955	796	630	14	603	758	677	44	1042						
3	Outfall F101-06-02	325	100	840	741	325	32	440	534	278	100	744						
4	Below F101-06-03	233	100	649	572	233	55	418	492	233	100	649						
5	Below F101-06-02	558	100	1282	1130	558	41	731	875	511	100	1197						
6	@ Pipeline Corridor	583	100	1326	1169	583	44	777	927	536	100	1242						
7	Above F101-06-01	604	100	1363	1202	604	45	806	961	557	100	1280						
8	Below F101-06-01	1294	71	2085	1800	1294	30	1266	1541	1234	69	1984						
9	Above Sens Road	1325	71	2124	1833	1325	30	1289	1570	1265	70	2036	202	100	580	729	48	1138
10	Below Sens Road	1398	73	2243	1939	1398	29	1330	1623	1350	72	2169	287	100	763	814	53	1286
11	Below F101-03	1534	75	2442	2115	1534	29	1430	1745	1500	75	2399	437	100	1059	964	61	1552
12	Above S.P. Yard	1588	76	2524	2188	1588	29	1469	1793	1570	76	2502	507	100	1189	1034	63	1661
13	Below S.P. Yard	1608	76	2549	2209	1608	30	1499	1826	1590	-	1189*	527	100	1226	1054	64	1698
13A	At Proposed S.P. Yard	1703	78	2698	2343	1703	43	1778	2123									
14	Above Trib. from Broadway	1813	79	2851	2477	1813	46	1917	2281	1683	-	1529*	620	100	1391	1147	67	1850
15	Below Trib. from Broadway	2027	81	3148	2739	2027	42	2018	2413	1948	-	2189*	885	100	1837	1412	73	2261
16	Mouth	2647	86	3992	3486	2647	44	2530	3018	2568	-	3289*	1505	100	2779	2032	81	3154
A	Sta 45+00 F101-06	98	100	333	290	98	35	187	215	98	100	333						
B	Sta 55+00 F101-06	54	100	207	180	54	83	164	188	54	100	207						
C	Sta 60+00 F101-06	33	100	141	123	33	100	123	141	33	100	141						

* Flows downstream of detention pond calculated by combining detention pond peak outflow with downstream area contribution (no time lag).

19000.T1

Exhibit 3



$Q = a\,(D.A.)^m$

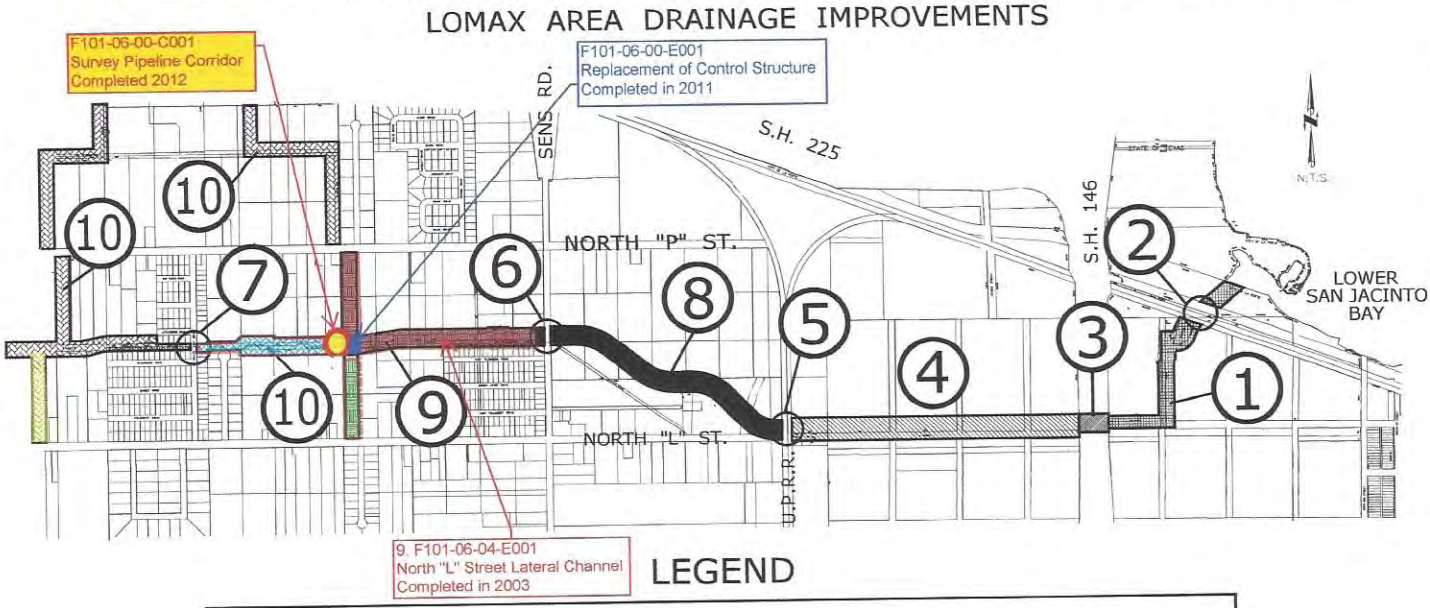
EXHIBIT NO. 16
ARMAND BAYOU 100 YEAR AREA DISCHARGE CURVES

Exhibit 4

CITY OF LA PORTE

F101-00-00 WATERSHED IMPROVEMENTS

LOMAX AREA DRAINAGE IMPROVEMENTS



LEGEND

1. CONCRETE LINING (COMPLETED '93)
2. 4th CULVERT - TXDOT & HPA (AWARD CONTRACT 4/98)
3. REPLACE CULVERTS w/ BRIDGE - TXDOT (PART OF 146/225 PROJECT)
4. CHANNEL EXCAVATION - TXDOT (PT. OF S.H. 146 PROJECT)
5. STRANG YARD CROSSING (COMPLETED '96)
6. SENS RD. BRIDGE (COMPLETED '89)
7. VALLEY VIEW BRIDGE (COMPLETED '88)
8. CHANNEL EXCAVATION (COMPLETED '00)
9. CHANNEL EXCAVATION (COMPLETED '03)
10. FUTURE CHANNEL IMPROVEMENTS

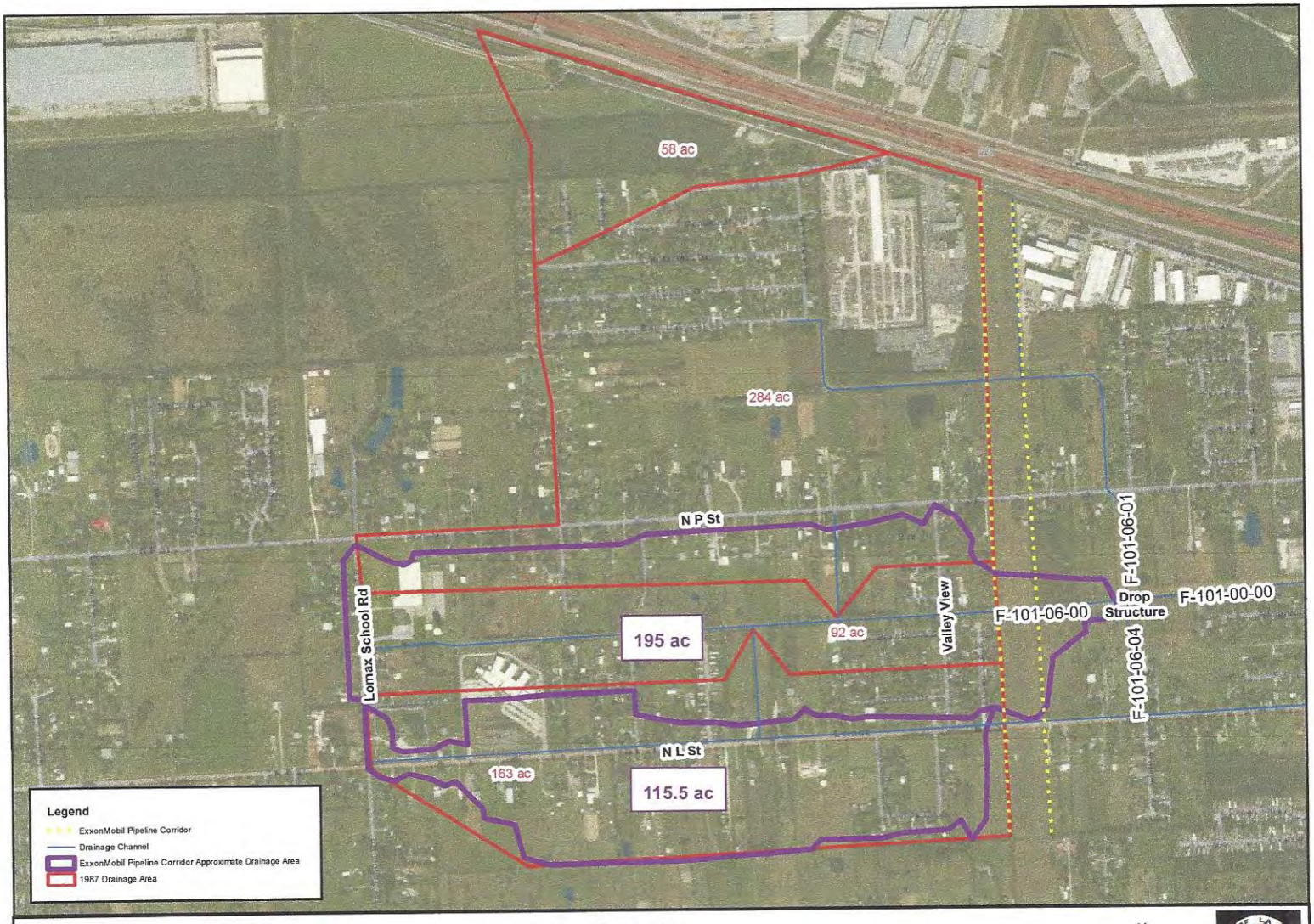


Exhibit 5

KSA

Approximate Drainage Area

0 290 580 1,160 Feet

N W E S

Exhibit 6

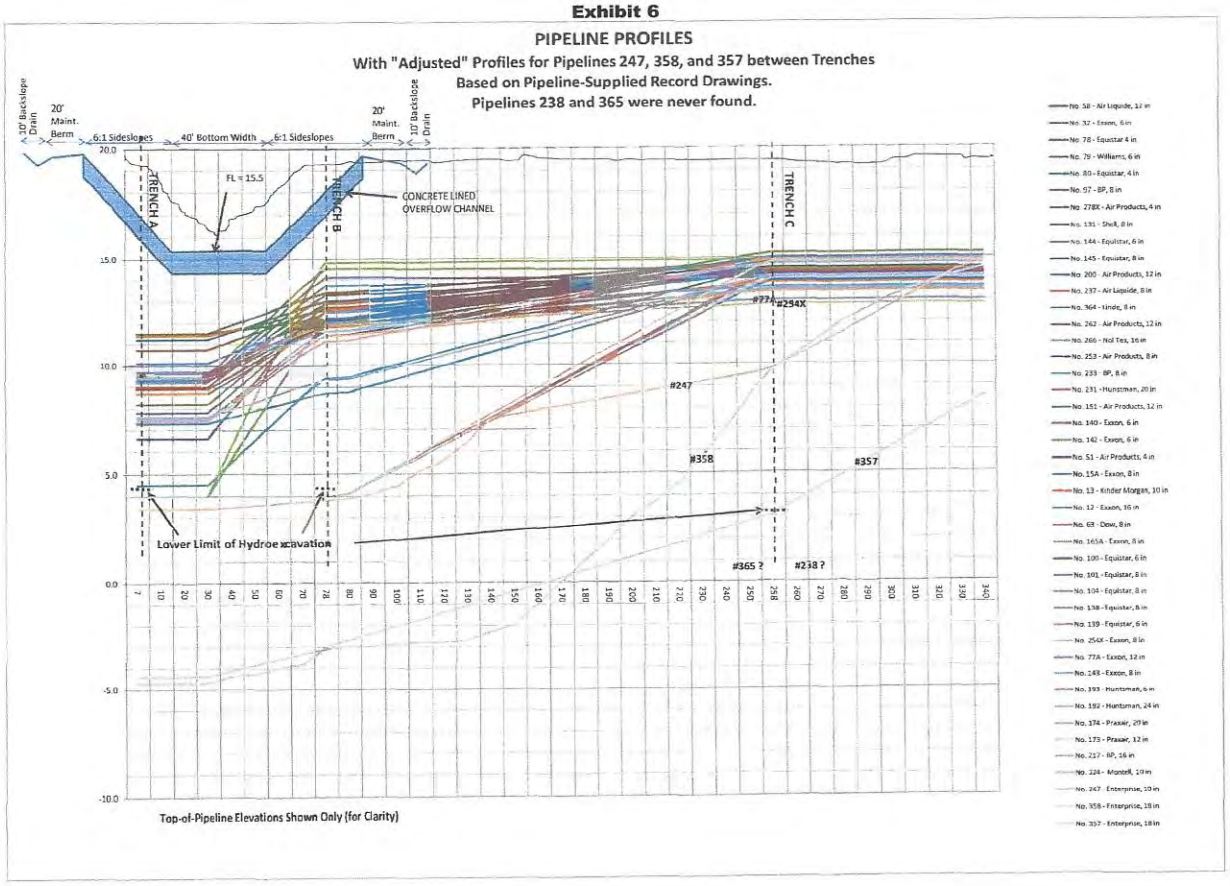

PIPELINE PROFILES
With "Adjusted" Profiles for Pipelines 247, 358, and 357 between Trenches
Based on Pipeline-Supplied Record Drawings.
Pipelines 238 and 365 were never found.
10' Backslope Drain
20' Maint. Berm
6:1 Sideslopes
40' Bottom Width
6:1 Sideslopes
20' Maint. Berm
10' Backslope Drain
TRENCH A
TRENCH B
TRENCH C
FL = 15.5
CONCRETE LINED OVERFLOW CHANNEL
#77A
#254X
#247
#358
#357
#365 ?
#238 ?
Lower Limit of Hydroexcavation
Top-of-Pipeline Elevations Shown Only (for Clarity)
No. 58 - Air Liquide, 12 in
No. 32 - Exxon, 6 in
No. 78 - Equistar 4 in
No. 79 - Williams, 6 in
No. 80 - Equistar, 4 in
No. 97 - BP, 8 in
No. 278X - Air Products, 4 in
No. 131 - Shell, 8 in
No. 144 - Equistar, 6 in
No. 145 - Equistar, 8 in
No. 200 - Air Products, 12 in
No. 237 - Air Liquide, 8 in
No. 364 - Linde, 8 in
No. 262 - Air Products, 12 in
No. 266 - Nol Tex, 16 in
No. 253 - Air Products, 8 in
No. 233 - BP, 8 in
No. 231 - Huntsman, 20 in
No. 151 - Air Products, 12 in
No. 140 - Exxon, 6 in
No. 142 - Exxon, 6 in
No. 51 - Air Products, 4 in
No. 15A - Exxon, 8 in
No. 13 - Kinder Morgan, 10 in
No. 12 - Exxon, 16 in
No. 68 - Dow, 8 in
No. 165A - Exxon, 8 in
No. 100 - Equistar, 6 in
No. 101 - Equistar, 8 in
No. 104 - Equistar, 8 in
No. 138 - Equistar, 8 in
No. 139 - Equistar, 6 in
No. 254X - Exxon, 8 in
No. 77A - Exxon, 12 in
No. 148 - Exxon, 8 in
No. 393 - Huntsman, 6 in
No. 192 - Huntsman, 24 in
No. 174 - Praxair, 20 in
No. 175 - Praxair, 12 in
No. 217 - BP, 16 in
No. 224 - Montell, 10 in
No. 247 - Enterprise, 10 in
No. 358 - Enterprise, 18 in
No. 357 - Enterprise, 18 in

Exhibit 7

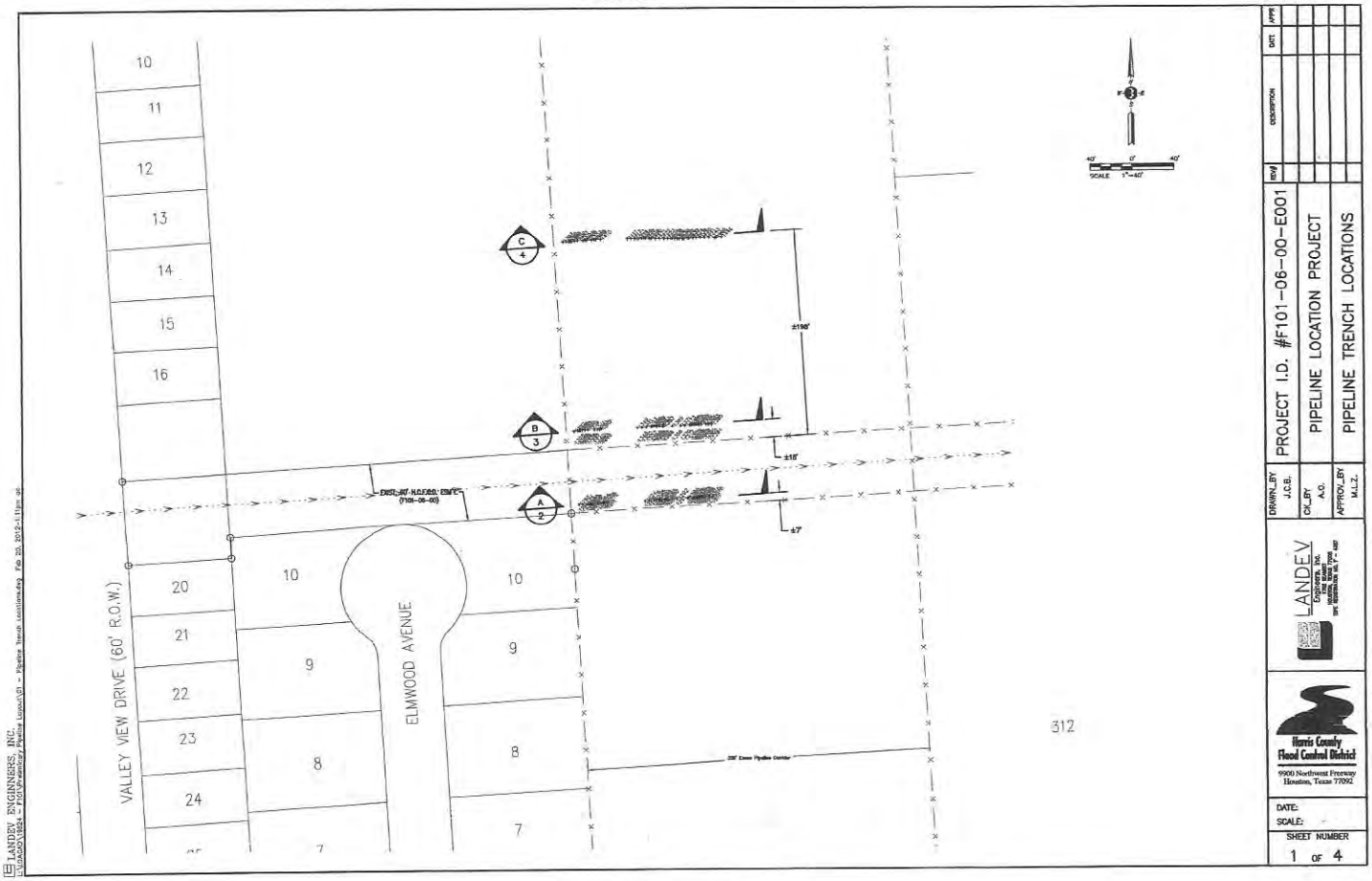
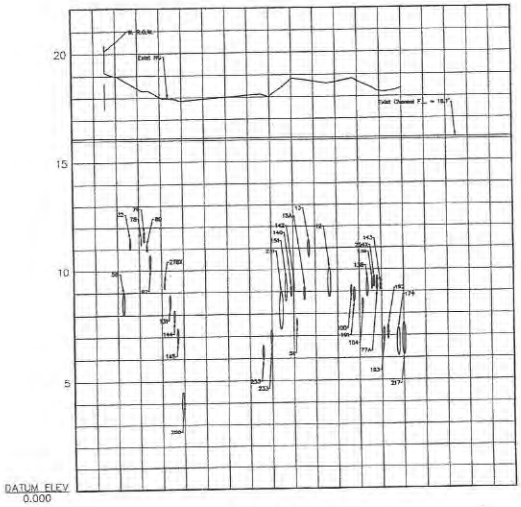

VALLEY VIEW DRIVE (60' R.O.W.)
ELMWOOD AVENUE
10
11
12
13
14
15
16
20
21
22
23
24
10
9
8
7
10
9
8
7
C
4
B
3
A
2
±190'
±18'
±7'
312
SCALE 1"=40'
PROJECT I.D. #F101-06-00-E001
PIPELINE LOCATION PROJECT
PIPELINE TRENCH LOCATIONS
DRAWN_BY J.C.B.
CK_BY A.O.
APPROV_BY M.L.Z.
LANDEV Engineers, Inc.
Harris County Flood Control District
9900 Northwest Freeway
Houston, Texas 77092
DATE:
SCALE:
SHEET NUMBER
1 OF 4
LANDEV ENGINEERS, INC.

Exhibit 7 cont'd



SECTION 'A–A'

SCALE: H: 1" = 20'
V: 1" = 2'

NOTE:

CONTRACTOR HYDROEXCAVATED TO A MAXIMUM DEPTH OF 15 FEET.

TRENCH 1 (Section A-A)

Exxon Mobile Pipeline Company Pipeline Number	Pipeline Owner	Pipeline Size (Dia, in)	Dist. From W. ROW to Centerline P/L	Top of Pipe Elevation (feet)
58	AL Lip Corp.	12"	7' - 1/2"	9.0
32	EMP co.	6"	11' - 1/2"	11.5
78	Equistar	4"	15' - 1-1/2"	11.5
79	Williams	6"	16' - 11-1/2"	11.8
80	Equistar	4"	18' - 7"	11.2
97	BP	8"	19' - 9"	10.7
278X	Air Products	4"	25' - 7"	9.5
131	Shell	8"	28' - 6"	8.9
144	Equistar	6"	30' - 9"	8.2
145	Equistar	8"	31' - 10"	7.4
200	Air Products	12"	34' - 3"	4.5
227	AL Lip Corp.	Not found in this trench		Below 4'
238	AL Lip Corp.	Not found in this trench		Below 4'
262	Air Products	Not found in this trench		Below 4'
364	Linde	Not found in this trench		Below 4'
266	Nat Tex	Not found in this trench		Below 4'
253	Air Products	8"	70' - 11"	6.6
233	BP	8"	74' - 6"	7.3
231	Huntsman	20"	79' - 1"	9.0
151	Air Products	8"	81' - 9"	9.6
140	EMP Co.	6"	84' - 6"	9.3
142	EMP Co.	6"	85' - 6"	9.5
51	Air Products	4"	87' - 1"	7.8
15A	EMP Co.	8"	90' - 5"	9.3
13	Kinder Morgan	10"	92' - 3"	11.4
12	EMP Co.	16"	101' - 5"	10.1
62	Dow	Not found in this trench		Below 4'
165A	EMP Co.	Not found in this trench		Below 4'
100	Equistar	6"	111' - 6"	9.3
101	Equistar	8"	113' - 1"	9.2
104	Equistar	6"	116' - 10"	8.7
138	Equistar	10"	118' - 11"	9.6
139	Equistar	6"	120' - 10"	9.6
254X	EMP Co.	6"	122' - 8"	9.7
77A	EMP Co.	8"	123' - 1"	9.7
143	EMP Co.	4"	124' - 6"	9.4
193	Huntsman	8"	126' - 1"	7.4
192	Huntsman	6"	127' - 8"	7.5
174	Praxair	16"	132' - 1"	7.4
173	Praxair	Not found in this trench		Below 4'
217	BP	8"	134' - 5"	7.6
224	Mobil	Not found in this trench		Below 4'
247	Occidental Chemical	Not found in this trench		Below 4'
358	Enterprise	Not found in this trench		Below 4'
337	Enterprise	Not found in this trench		Below 4'
363	AL Lip Corp.	Not found in this trench		Below 4'

Note: Contractor hydroexcavated to a depth of 15' below N.G.

REV#	DESCRIPTION	DATE	APPR

PROJECT I.D. #F101-06-00-E001

PIPELINE LOCATION PROJECT

CROSS SECTION 'A–A'

DRAWN_BY J.C.B. | CK_BY A.O. | APPROV_BY M.L.Z.

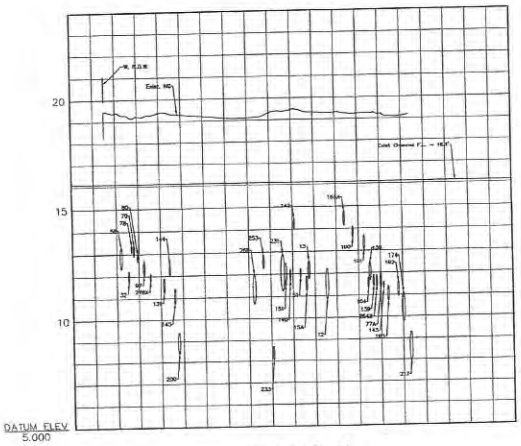
LANDEV CONSULTANTS, INC. ENGINEERING FIRM F-2699 TBPELS REGISTRATION NO. F-1402

Harris County Flood Control District
9900 Northwest Freeway
Houston, Texas 77092

DATE:
SCALE:
SHEET NUMBER
2 OF 4

LANDEV ENGINEERS, INC.
LANDCONWEB - J:\101Pipelines\Pipeline Layout\V2 - Cross Section AA.dwg Feb 10, 2012-11:10am ao

Exhibit 7 cont'd



SECTION 'B–B'

SCALE: H: 1" = 20'
V: 1" = 2'

NOTE:

CONTRACTOR HYDROEXCAVATED TO A MAXIMUM DEPTH OF 15 FEET.

TRENCH 2 (Section B-B)

Exxon Mobile Pipeline Company Pipeline Number	Pipeline Owner	Pipeline Size (Dia, in)	Dist. From W. ROW w Centerline P/L	Top of Pipe Elevation (feet)
58	AL Lip Corp.	12"	4' - 11"	13.3
32	EMP co.	6"	7 - 5"	12.2
78	Equistar	4"	9' - 11"	13.4
79	Williams	6"	10' - 11"	12.4
80	Equistar	4"	12' - 6"	13.0
97	BP	8"	16' - 2"	12.7
278X	Air Products	4"	18' - 8"	12.1
131	Shell	8"	24' - 6"	11.9
144	Equistar	6"	27' - 4"	12.5
145	Equistar	8"	29' - 2"	11.4
200	Air Products	12"	30' - 9"	9.4
237	AL Lip Corp.	Not found in this trench		Below 4'
238	AL Lip Corp.	Not found in this trench		Below 4'
262	Air Products	Not found in this trench		Below 4'
364	Linde	Not found in this trench		Below 4'
266	Nol Tex	10"	65' - 3"	12.0
253	Air Products	8"	69' - 8"	12.9
233	BP	8"	77' - 10"	8.7
231	Huntsman	20"	78' - 3"	12.9
151	Air Products	12"	79' - 8"	12.5
140	EMP Co.	6"	81' - 7"	12.2
142	EMP Co.	6"	82' - 5"	14.5
51	Air Products	4"	86' - 4"	12.2
15A	EMP Co.	8"	87' - 9"	11.9
13	Kinder Morgan	10"	90' - 3"	12.6
12	EMP Co.	16"	95' - 6"	12.2
63	Dow	Not found in this trench		Below 4'
165A	EMP Co.	8"	106' - 8"	14.8
100	Equistar	6"	110' - 0"	14.1
101	Equistar	8"	115' - 6"	13.7
104	Equistar	6"	117 - 7"	11.8
138	Equistar	10"	118' - 7"	12.5
139	Equistar	6"	119' - 2"	11.9
254X	EMP Co.	6"	120' - 6"	11.9
77A	EMP Co.	8"	122 - 6"	12.0
143	EMP Co.	4"	123' - 9"	11.6
193	Huntsman	8"	125 - 8"	11.6
192	Huntsman	8"	130' - 4"	11.6
174	Praxair	16"	132' - 10"	11.3
173	Praxair	Not found in this trench		Below 4'
217	BP	8"	136' - 7"	9.3
224	Morrell	Not found in this trench		Below 4'
247	Occidental Chemical	Not found in this trench		Below 4'
358	Enterprise	Not found in this trench		Below 4'
357	Enterprise	Not found in this trench		Below 4'
365	AL Lip Corp.	Not found in this trench		Below 4'

Note: Contractor hydroexcavated to a depth of 15' below N.G.

REV#	DESCRIPTION	DATE	APPR

PROJECT I.D. #F101-06-00-E001

PIPELINE LOCATION PROJECT

CROSS SECTION 'B–B'

DRAWN_BY J.C.B. | CK_BY A.O. | APPROV_BY M.L.Z.

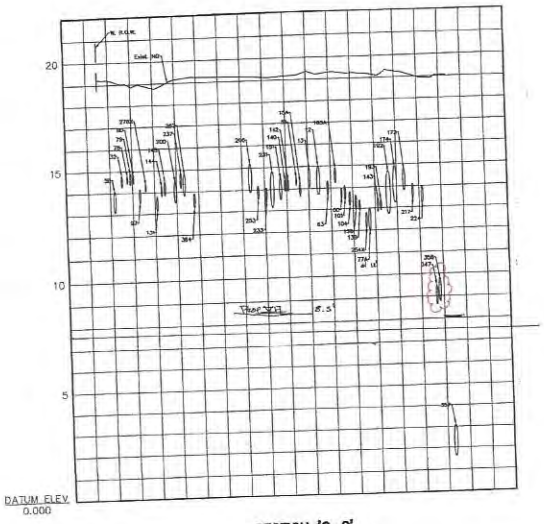
LANDEV Engineers, Inc. TBPE REGISTRATION NO. F-1497 TBPLS REGISTRATION NO. 10194

Harris County Flood Control District
9900 Northwest Freeway
Houston, Texas 77092

DATE:
SCALE:
SHEET NUMBER
3 OF 4

LANDEV ENGINEERS, INC.
L:\LANDEV\Harris Co. FCD\Projects\F101-06-00-E001 Lyon\V03 - Cross Section B\B.dwg Feb 20, 2012-11:09am aa

Exhibit 7 cont'd



SECTION 'C–C'

SCALE: H. 1" = 20'
V. 1" = 1'

NOTE:

CONTRACTOR HYDROEXCAVATED TO A MAXIMUM DEPTH OF 15 FEET.

TRENCH 3 (Section C-C)

Exxon Mobile Pipeline Company Pipeline Number	Pipeline Owner	Pipeline Size (Dia, in)	Dist. From W. ROW to Centerline P/L	Top of Pipe Elevation (feet)
58	AL Lip Corp.	12"	7' - 9"	14.1
32	EMP co.	6"	7' - 4"	14.8
78	Equistar	4"	9' - 7"	14.8
79	Williams	6"	10' - 9"	14.9
80	Equistar	4"	12' - 3"	14.8
97	BP	8"	15' - 1"	14.2
278X	Air Products	4"	17' - 11"	14.4
131	Shell	8"	23' - 1"	13.8
144	Equistar	6"	25' - 2"	14.3
145	Equistar	8"	26' - 7"	14.5
200	Air Products	12"	31' - 4"	14.5
237	AL Lip Corp.	8"	34'	14.2
238	AL Lip Corp.	Not found in this trench		Below 4'
262	Air Products	12"	35' - 8"	14.8
364	Linde	8"	39' - 11"	13.9
266	Nol Tex	16"	65' - 6"	15.2
253	Air Products	8"	68' - 10"	14.2
233	BP	8"	72' - 7"	14.1
231	Huntsman	20"	76'	14.8
151	Air Products	12"	79' - 3"	14.5
140	EMP Co.	6"	81' - 4"	14.4
142	EMP Co.	6"	82' - 7"	14.4
51	Air Products	4"	86' - 4"	14.5
15A	EMP Co.	8"	89' - 5"	14.3
13	Kinder Morgan	10"	90' - 3"	14.9
12	EMP Co.	16"	98' - 6"	15.0
63	Dow	8"	103' - 2"	14.3
165A	EMP Co.	8"	106' - 8"	14.9
100	Equistar	6"	107' - 6"	14.0
101	Equistar	8"	108' - 3"	14.1
104	Equistar	8"	110' - 9"	13.8
138	Equistar	8"	113' - 8"	13.6
139	Equistar	6"	115' - 6"	13.4
254X	EMP Co.	8"	118' - 6"	12.8
77A	EMP Co.	12"	119' - 10"	13.0
143	EMP Co.	8"	123' - 7"	13.5
193	Huntsman	6"	125' - 0"	13.4
192	Huntsman	24"	128' - 8"	14.7
174	Praxair	20"	131' - 7"	14.9
173	Praxair	12"	135' - 7"	14.8
217	BP	8"	139' - 6"	14.1
224	Monsell	10"	144' - 2"	14.0
247	Occidental Chemical	10"	149' - 7"	9.5
358	Enterprise	18"	150' - 11"	9.8
357	Enterprise	18"	155' - 1"	3.1
365	AL Lip Corp.	Not found in this trench		Below 4'

Note: Contractor hydroexcavated to a depth of 15' below N.G.

REV#	DESCRIPTION	DATE	APPR

PROJECT I.D. #F101-06-00-E001

PIPELINE LOCATION PROJECT

CROSS SECTION 'C–C'

DRAWN_BY J.C.B. | CK_BY A.D. | APPROV_BY M.L.Z.

LANDEV Engineers, Inc. TBPE Registration No. F-2487

Harris County Flood Control District
9900 Northwest Freeway
Houston, Texas 77092

DATE:
SCALE:
SHEET NUMBER
4 OF 4

LANDEV ENGINEERS, INC.
L:\HOUCAD\0864 - 7381\Preliminary Pipeline Layout\04 - Cross Section CC.dwg Feb 20, 2013-11:06pm sg

Exhibit 8

KSA

Alternative 1
Drainage Improvements

0 45 90 180 Feet

N W E S

Exhibit 9

KSA

Alternative 2 Drainage Improvements

0 45 90 180 Feet

N W E S

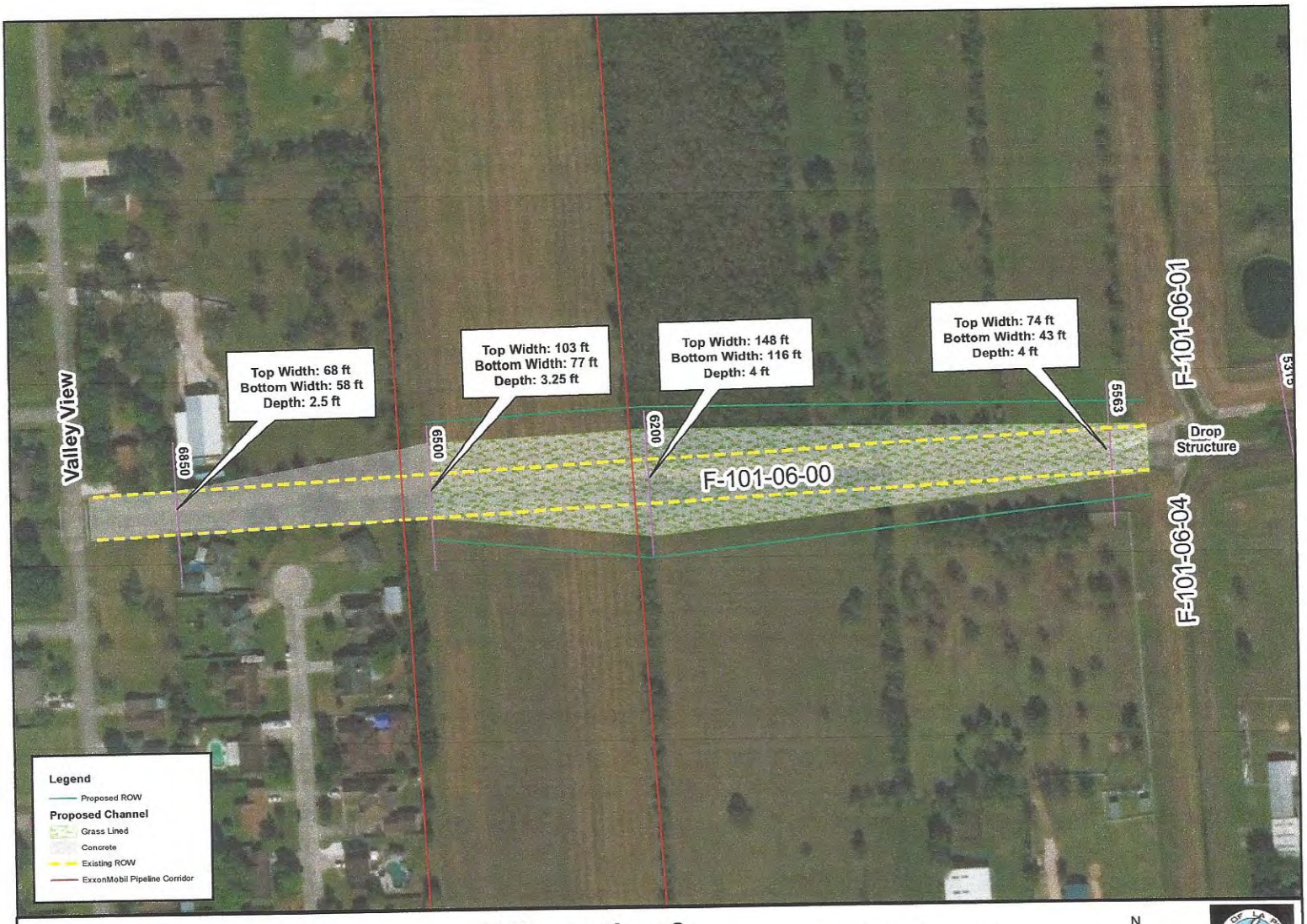


Exhibit 10

KSA

Alternative 3
Drainage Improvements

0 45 90 180 Feet

N W E S

Exhibit 11

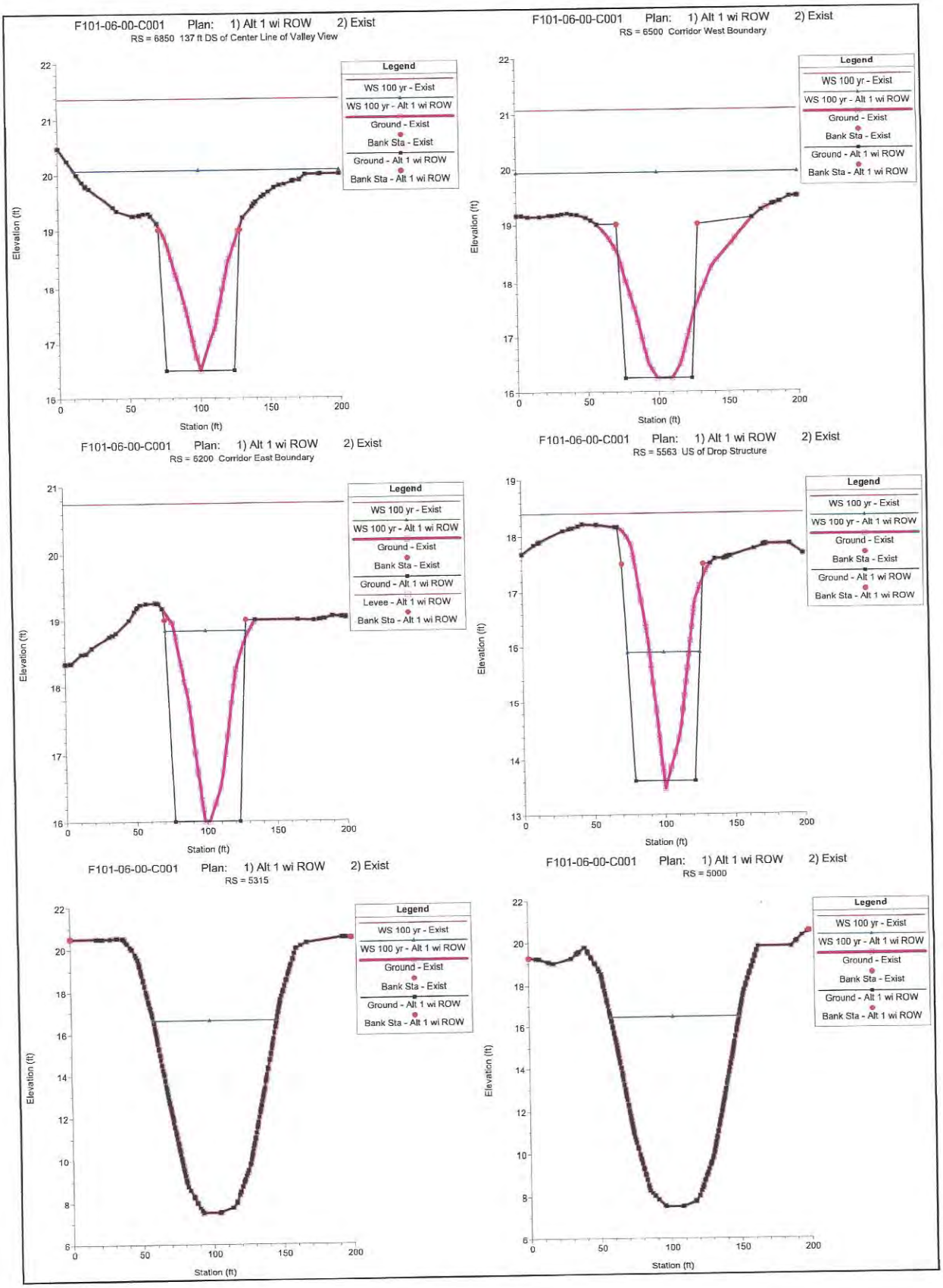

F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 6850 137 ft DS of Center Line of Valley View
F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 6500 Corridor West Boundary
F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 6200 Corridor East Boundary
F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 5563 US of Drop Structure
F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 5315
F101-06-00-C001 Plan: 1) Alt 1 wi ROW 2) Exist
RS = 5000
Legend
WS 100 yr - Exist
WS 100 yr - Alt 1 wi ROW
Ground - Exist
Bank Sta - Exist
Ground - Alt 1 wi ROW
Levee - Alt 1 wi ROW
Bank Sta - Alt 1 wi ROW
Elevation (ft)
Station (ft)

Exhibit 12

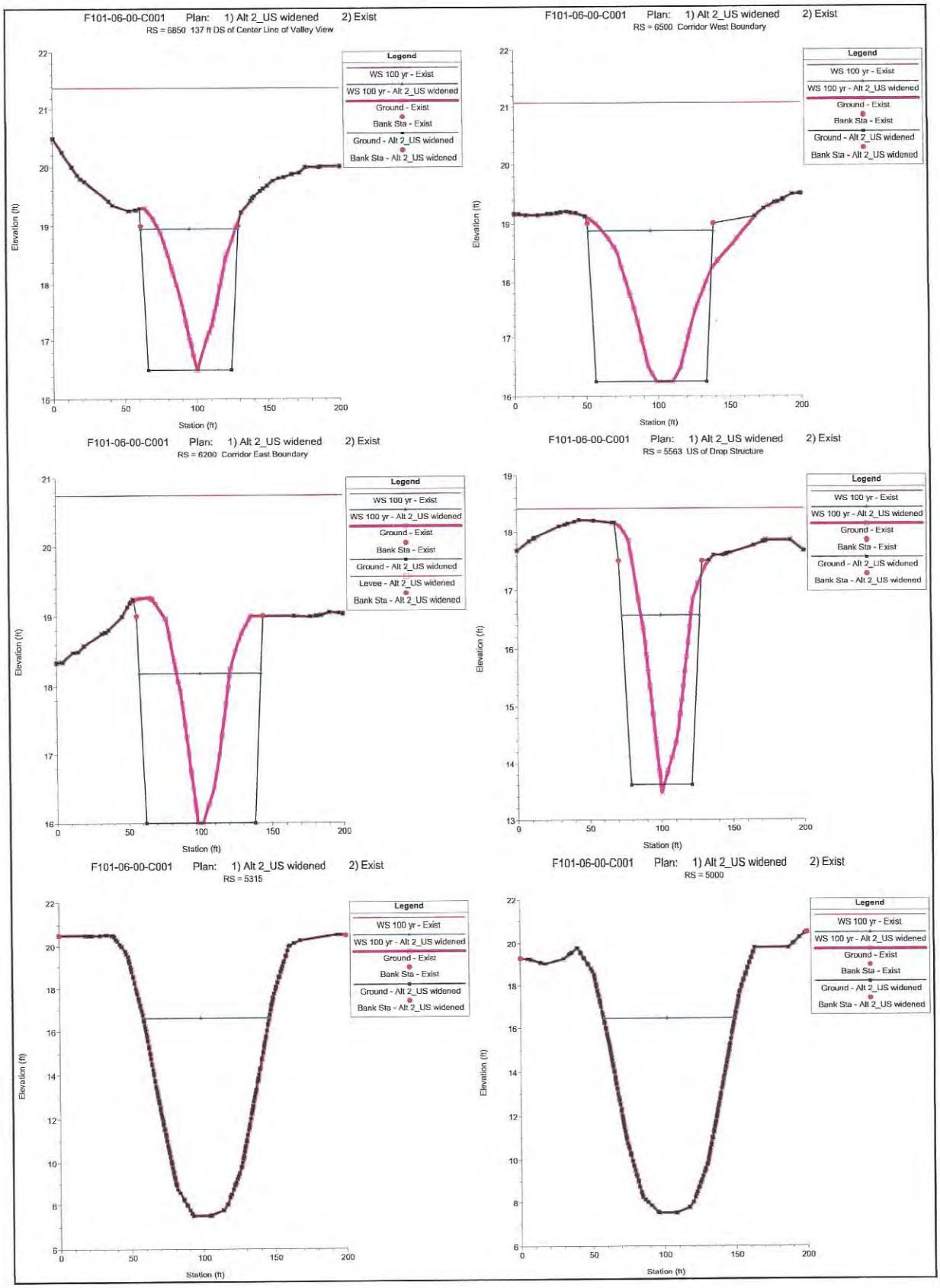
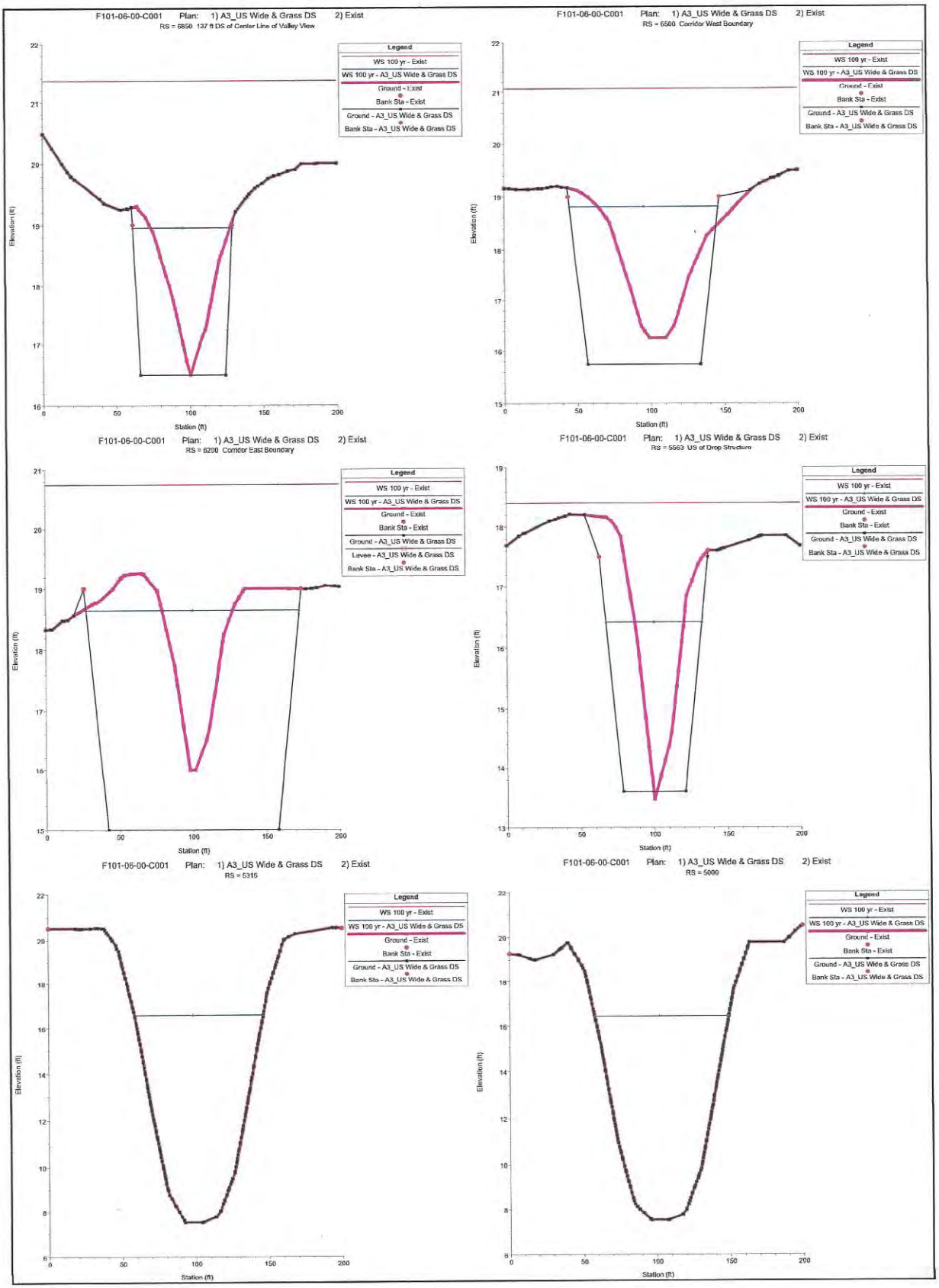

F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 6850 137 ft DS of Center Line of Valley View
F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 6500 Corridor West Boundary
F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 6200 Corridor East Boundary
F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 5563 US of Drop Structure
F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 5315
F101-06-00-C001 Plan: 1) Alt 2_US widened 2) Exist
RS = 5000
Legend
WS 100 yr - Exist
WS 100 yr - Alt 2_US widened
Ground - Exist
Bank Sta - Exist
Ground - Alt 2_US widened
Levee - Alt 2_US widened
Bank Sta - Alt 2_US widened
Elevation (ft)
Station (ft)

Exhibit 13


F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 6850 137 ft DS of Center Line of Valley View
F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 6500 Corridor West Boundary
F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 6200 Corridor East Boundary
F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 5563 US of Drop Structure
F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 5315
F101-06-00-C001 Plan: 1) A3_US Wide & Grass DS 2) Exist
RS = 5000
Legend
WS 100 yr - Exist
WS 100 yr - A3_US Wide & Grass DS
Ground - Exist
Bank Sta - Exist
Ground - A3_US Wide & Grass DS
Levee - A3_US Wide & Grass DS
Bank Sta - A3_US Wide & Grass DS
Elevation (ft)
Station (ft)

CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 1
Exhibit 14 - Opinion of Probable Construction Costs
April 16, 2018

Summary of Construction Administration Costs

Engineering Construction Administration (Estimated):	$	201,501.19
Resident Project Representative (Estimated):	$	142,560.00
Construction Materials Testing (Estimated):	$	20,150.12
Close-Out-Phase (Estimated):	$	10,000.00
Subtotal:	**$**	**374,211.31**

Construction Phase - Base Bid Cost Summary

Item 1: Mobilization and Erosion/Sedimentation Control Measures	$	185,252.36
Item 2: Drainage Channel	$	1,493,923.55
Subtotal:	**$**	**1,679,175.91**
20% Contingencies:	**$**	**335,836.00**
Base Bid Subtotal w/ Contingencies:	**$**	**2,015,011.91**

Base Bid Subtotal w/ Contingencies:	**$**	**2,015,011.91**
Construction Administration (Estimated):	**$**	**374,211.31**
Total Project Construction Cost (Estimated):	**$**	**2,389,223.21**

KSA

TBPE Firm Registration No. F-1356

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** The Engineer has not control over the cost of labor, materials, equipment, or over the Contractor's methods of determining prices or over competitive bidding or market conditions. Opinions of probable costs provided herein are based on the information known to the Engineer at this time and represent only the Engineer's judgement as a design professional familiar with the construction industry. The Engineer cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from the opinions of probable cost.

Basis of Cost Estimate: No Design Completed

CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 1
Exhibit 14 - Opinion of Probable Construction Costs
April 16, 2018

Item No.	Spec. No.	Description	Units	Estimated Quantities	Estimated Unit Price	Subtotal
Item 1: Mobilization and Erosion/Sedimentation Control Measures						
1.01	500	Mobilization	LS	1	$ 152,652.36	$ 152,652.36
1.02	P-156	Stormwater Pollution Prevention Plan (SWP3) Document	LS	1	$ 4,000.00	$ 4,000.00
1.03	506	Temporary Sediment Control Fence	LF	2,000	$ 3.30	$ 6,600.00
1.04	506	Rock Construction Exit	EA	2	$ 3,000.00	$ 6,000.00
1.05	506	Temporary Rock Filter Dam	LF	300	$ 44.00	$ 13,200.00
1.06	164	Seeding	AC	0.7	$ 4,000.00	$ 2,800.00
					Subtotal:	**$ 185,252.36**
					20% Contingencies:	**$ 22,231.00**
					Total w/ Contingencies:	**$ 207,483.36**

Item 2: Drainage Channel						
2.01	100	Preparing ROW (Tree) (Less than 24" Dia)	EA	50	$ 440.00	$ 22,000.00
2.02	104	Removing Concrete (Drop Structure)	SY	310	$ 88.00	$ 27,280.00
2.03	110	Excavation (Widen Channel)	CY	2,974	$ 16.50	$ 49,069.35
2.04	132	Embankment	SY	2,132	$ 33.00	$ 70,369.20
2.05	260	8" Lime Treated Subgrade	SY	10,662	$ 5.50	$ 58,641.00
2.06	260	Hydrated Lime	TON	259	$ 200.00	$ 51,744.00
2.07	420	Concrete (Channel)	SY	11,728	$ 100.00	$ 1,172,820.00
2.08		Purchase ROW	AC	0.7	$ 60,000.00	$ 42,000.00
					Subtotal:	**$ 1,493,923.55**
					20% Contingencies:	**$ 298,784.71**
					Total w/ Contingencies:	**$ 1,792,708.26**

CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 2
Exhibit 15 - Opinion of Probable Construction Costs
April 16, 2018

Summary of Construction Administration Costs

Engineering Construction Administration (Estimated):	$	269,084.26
Resident Project Representative (Estimated):	$	142,560.00
Construction Materials Testing (Estimated):	$	26,908.43
Close-Out-Phase (Estimated):	$	10,000.00
Subtotal:	**$**	**448,552.68**

Construction Phase - Base Bid Cost Summary

Item 1: Mobilization and Erosion/Sedimentation Control Measures	$	238,451.69
Item 2: Drainage Channel	$	2,003,916.88
Subtotal:	**$**	**2,242,368.56**
20% Contingencies:	**$**	**448,474.00**
Base Bid Subtotal w/ Contingencies:	**$**	**2,690,842.56**

Base Bid Subtotal w/ Contingencies:	**$**	**2,690,842.56**
Construction Administration (Estimated):	**$**	**448,552.68**
Total Project Construction Cost (Estimated):	**$**	**3,139,395.24**

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Basis of Cost Estimate: No Design Completed

CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 2
Exhibit 15 - Opinion of Probable Construction Costs
April 16, 2018

Item No.	Spec. No.	Description	Units	Estimated Quantities	Estimated Unit Price	Subtotal
Item 1: Mobilization and Erosion/Sedimentation Control Measures						
1.01	500	Mobilization	LS	1	$ 203,851.69	$ 203,851.69
1.02	P-156	Stormwater Pollution Prevention Plan (SWP3) Document	LS	1	$ 4,000.00	$ 4,000.00
1.03	506	Temporary Sediment Control Fence	LF	2,000	$ 3.30	$ 6,600.00
1.04	506	Rock Construction Exit	EA	2	$ 3,000.00	$ 6,000.00
1.05	506	Temporary Rock Filter Dam	LF	300	$ 44.00	$ 13,200.00
1.06	164	Seeding	AC	1.2	$ 4,000.00	$ 4,800.00
					Subtotal:	**$ 238,451.69**
					20% Contingencies:	**$ 28,615.00**
					Total w/ Contingencies:	**$ 267,066.69**

Item 2: Drainage Channel						
2.01	100	Preparing ROW (Tree) (Less than 24" Dia)	EA	60	$ 440.00	$ 26,400.00
2.02	104	Removing Concrete (Drop Structure)	SY	310	$ 88.00	$ 27,280.00
2.03	110	Excavation (Widen Channel)	CY	6,239	$ 16.50	$ 102,939.38
2.04	132	Embankment	SY	2,803	$ 33.00	$ 92,512.20
2.05	260	8" Lime Treated Subgrade	SY	14,017	$ 5.50	$ 77,093.50
2.06	260	Hydrated Lime	TON	340	$ 200.00	$ 68,021.80
2.07	420	Concrete (Channel)	SY	15,419	$ 100.00	$ 1,541,870.00
2.08		Purchase ROW	AC	1.1	$ 60,000.00	$ 67,800.00
					Subtotal:	**$ 2,003,916.88**
					20% Contingencies:	**$ 400,783.38**
					Total w/ Contingencies:	**$ 2,404,700.25**

CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 3
Exhibit 16 - Opinion of Probable Construction Costs
April 16, 2018

Summary of Construction Administration Costs

Engineering Construction Administration (Estimated):	$	123,837.50
Resident Project Representative (Estimated):	$	113,760.00
Construction Materials Testing (Estimated):	$	12,383.75
Close-Out-Phase (Estimated):	$	10,000.00
Subtotal:	**$**	**259,981.25**

Construction Phase - Base Bid Cost Summary

Item 1: Mobilization and Erosion/Sedimentation Control Measures	$	134,416.28
Item 2: Drainage Channel	$	897,562.75
Subtotal:	**$**	**1,031,979.03**
20% Contingencies:	**$**	**206,396.00**
Base Bid Subtotal w/ Contingencies:	**$**	**1,238,375.03**

Base Bid Subtotal w/ Contingencies:	**$**	**1,238,375.03**
Construction Administration (Estimated):	**$**	**259,981.25**
Total Project Construction Cost (Estimated):	**$**	**1,498,356.28**

KSA

TBPE Firm Registration No. F-1356

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CITY of LA PORTE
LPT.008
F101-06-00 DRAINAGE CHANNEL IMPROVEMENTS - ALTERNATIVE 3
Exhibit 16 - Opinion of Probable Construction Costs
April 16, 2018

Item No.	Spec. No.	Description	Units	Estimated Quantities	Estimated Unit Price	Subtotal
Item 1: Mobilization and Erosion/Sedimentation Control Measures						
1.01	500	Mobilization	LS	1	$ 93,816.28	$ 93,816.28
1.02	P-156	Stormwater Pollution Prevention Plan (SWP3) Document	LS	1	$ 4,000.00	$ 4,000.00
1.03	506	Temporary Sediment Control Fence	LF	2,000	$ 3.30	$ 6,600.00
1.04	506	Rock Construction Exit	EA	2	$ 3,000.00	$ 6,000.00
1.05	506	Temporary Rock Filter Dam	LF	300	$ 44.00	$ 13,200.00
1.06	164	Seeding	AC	2.7	$ 4,000.00	$ 10,800.00
					Subtotal:	**$ 134,416.28**
					20% Contingencies:	**$ 16,130.00**
					Total w/ Contingencies:	**$ 150,546.28**

Item 2: Drainage Channel						
2.01	100	Preparing ROW (Tree) (Less than 24" Dia)	EA	100	$ 440.00	$ 44,000.00
2.02	104	Removing Concrete (Drop Structure)	SY	310	$ 88.00	$ 27,280.00
2.03	110	Excavation (Widen Channel)	CY	13,503	$ 16.50	$ 222,804.45
2.04	132	Embankment	SY	704	$ 33.00	$ 23,232.00
2.05	260	8" Lime Treated Subgrade	SY	3,520	$ 5.50	$ 19,360.00
2.06	260	Hydrated Lime	TON	85	$ 200.00	$ 17,086.30
2.07	420	Concrete (Channel)	SY	3,872	$ 100.00	$ 387,200.00
2.08		Purchase ROW	AC	2.6	$ 60,000.00	$ 156,600.00
					Subtotal:	**$ 897,562.75**
					20% Contingencies:	**$ 179,512.55**
					Total w/ Contingencies:	**$ 1,077,075.30**