Work Task 41 –
Hydrologic and Hydraulic Evaluation of Stormwater Drainage at Marsha Sharp Freeway and University Avenue

Texas Tech Center of Multidisciplinary Research in Transportation (TechMRT)
Texas Tech University, Lubbock, Texas

Research Project: 05-03XX1A001
Research Report: 05-03XX1A001 FINAL REPORT

Final Report
Texas Department of Transportation
Research Project Number

May, 2016

Introduction

Problem Statement

Marsha Sharp Freeway has become inundated at the University Avenue intersection recent intense rainfall events. The flooding was coupled with observed backflow through the Marsha Sharp Freeway storm drainage inlets. Ken Rainwater, professor at Texas Tech University, was contacted by the Lubbock District staff to conduct hydrologic and hydraulic engineering analysis on the local situation. Local District associates, Frank Phillips, Ron Baker, Doug Eichorst, and Edward Sewell, met with Ken Rainwater on May 28, 2015 to outline the scope of work. The main approach was to "start from scratch" to describe the storm drainage conditions for the approximately 300 acres that drains into the Freeway stormwater system at this location.

Project Objectives

The following objectives will be pursued in the proposed work.

1. Review of previous hydrologic and hydraulic engineering work for the area of interest.
2. Update the current description of the local watershed and storm sewer conditions.
3. Compare the 50-yr design storm event to recent intense rainfall events that have led to local flooding.
4. Confirm capacity and operational specifications of the pumped drainage system for the Texas Tech football stadium.
5. Assemble a useful hydrologic and hydraulic simulation model to simulate the impact of various storm events on the watershed and drainage system.
6. Propose potential conceptual alternative solutions to reduce future flooding in the area of interest, with accompanying simulations when appropriate.

Scope of Work

The following specific tasks must be accomplished to achieve the objectives.

1. Review all previous engineering analyses, design, and construction documentation that dealt with the stormwater flow and collection near this intersection.
2. Verify the current contributing watershed area and related hydrologic characteristics. Compare the current contributing watershed to the original drainage areas shown in the Marsha Sharp Freeway construction plans.
3. Meet with TxDOT, Texas Tech University, and City of Lubbock representatives to advise all parties of the project scope and review of common concerns.

4. Compare the 50-yr design rainfall event to other recent high-intensity rainfall events that led to flooding of the Freeway.
5. Evaluate performance and timing of the pumping system that drains the Texas Tech football stadium for various rainfall events.
6. Assemble a stormwater drainage model for the local watershed and storm sewer system using the Environmental Protection Agency's Storm Water Management Model (SWMM).
7. Simulate the behavior of the current watershed and drainage system for the 50-yr design storm both with and without the stadium pumping system.
8. Simulate the behavior of the current watershed and drainage system for selected recent intense rainfall events both with and without the stadium pumping system.
9. Simulate behavior of the future Freeway conditions after center median is filled and capacity increased to three lanes in each direction for the 50-yr design storm with and without the stadium pumping system.
10. Simulate behavior of the future Freeway conditions after center median is filled and capacity increased to three lanes in each direction for selected recent intense rainfall events with and without the stadium pumping system.[1]
11. Based on the results of the simulations, identify potential alternative solutions for discussion with TxDOT, Texas Tech, and the City of Lubbock. These alternatives may include delaying the start times on the Jones /AT&T pumps, installing underground water detention systems, or reconnecting the stadium to the old 48-in outfall line.

Final Report Overview and organization

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[1] Alternations to the geometry of the storm sewer system will not have a substantial impact on the water elevations seen at Inlet-F79, so the future Freeway modifications were not modeled in SWMM.

Review of Prior Work

During the completion of this research project, the Texas Department of Transportation, the City of Lubbock and Texas Tech University served as sources of information regarding the design of Marsha Sharp Freeway and the facilities located on University Avenue and in the Jones Stadium. The documents and resources provided were reviewed in order to gain understanding on the history and design of the Marsha Sharp Freeway drainage system. The model was developed based on the information gained through these resources.

Documents Provided by TxDOT

The Texas Department of Transportation provided several previous studies and plan sets to assist in the knowledge of Marsha Sharp Freeway. These documents included the Federal Aid Project MANH 94(57), which contained the 1994 plans and simulation results for the construction of proposed improvements for the storm sewer outfall line from University Avenue to the Yellow House Canyon Lakes. These plans were used in the compilation of data to model the storm sewer system. Specific data used and their respective sheet numbers are listed in Table 1.

Additional documentation provided included the US 82 East-West Freeway Drainage, containing the 2004 plan and simulation results for the Marsha Sharp Freeway drainage system. This plan set contained plan and profile sheets for the storm sewer network along Marsha Sharp Freeway. The design contained in this document is the most current data regarding the construction of the network that is in place since the opening of the freeway in 2008. This document was used to model the majority of the storm sewer network in SWMM as well as to compare data collected during this study, such as contributing drainage areas and simulation results. Table 1 provides a summary of the sheet numbers related to the referenced data used in this study. The plan and profile MicroStation files for this design were also supplied as a reference by TxDOT.

Table 1: Summary of Data Used from Previous Plans for Modeling

Federal Aid Project MANH 94(57)	
Referenced Data	**Sheet No.**
8’x8’ Box North on Avenue U ending at 2 7’x4’ Outfall	Sheet 72-79
US 82 East-West Freeway Drainage Plans	
Referenced Data	**Sheet No.**
Contributing Drainage Areas	Sheet 450-460
Hydraulic/Hydrologic Data Line C	Sheet 475-478
Hydraulic/Hydrologic Data Line E	Sheet 483-484
Hydraulic/Hydrologic Data Line F	Sheet 485-498

Hydraulic/Hydrologic Data Line G	Sheet 499-500
Hydraulic/Hydrologic Data Line H	Sheet 501-502
Plan and Profile for Line C	Sheet 546-549
Plan and Profile for Line E	Sheet 557-560
Plan and Profile for Line F	Sheet 561-614
Plan and Profile for Line G	Sheet 615-619
Plan and Profile for Line H	Sheet 620-622
Playa Lake Contributing Areas	Sheet 448, 463
Playa Lake 44 Grading Plan	Sheet 643
Culvert 1 Layout US 82	Sheet 648
Culvert 2 Layout Quaker	Sheet 651

The last document provided was Federal Project NH2009(831), including the 2009 plan and simulation results for the U.S. 82 and I.H. 27 interchange. These plans were reviewed for additional information; however, because the scope of this document involved infrastructure further east then the work of Task 41 none of the plan and simulations directly reflected results for this study.

An additional opportunity supplied by TxDOT was to observe a storm sewer video survey completed by MetroPipeInspection company in December 2015. Video was conducted in the vicinity of the storm drains at the intersection of Marsha Sharp and University that exhibited surcharge during the recent storm events. The main 8’x8’ box culvert was recorded to be in good condition with no concerning disjointing or residue accumulation in the boxes.

Documents Provided by the City of Lubbock

The City of Lubbock served as a source for data regarding the structure and alignment of Lubbock’s infrastructure and topography. Lubbock’s transportation centerlines and processed contour lines from 2011 were obtained through the City of Lubbock website. The City also provided the MS4 GIS database of the storm sewer network throughout Lubbock. This database includes documented locations of storm inlets, manholes, outfalls and pipes. Each component of the network included a variation of information regarding individual characteristics including parameters such as, location, elevation, type, size, and length. The information documented in the database was indicated as data collected either by field survey or through engineering documents.

In addition to the contours from the City of Lubbock website, the City also provided the raw data and survey report of the LiDAR taken in 2011. The LiDAR mapping services was provided by Sanborn and encompassed approximately 419 square miles of Lubbock County. The LiDAR was used to develop a triangular irregular network surface in GIS to obtain point elevations where needed for the completion of this study.

Previous video inspection had been completed for the City of Lubbock by InspectIT in March 2011. This video inspection was made available as additional information regarding the Texas Tech storm sewer along University Avenue.

Documents Provided by Texas Tech University

Texas Tech University Physical Plant staff provided documentation to assist in the research regarding the Texas Tech University storm drain system. This included CAD drawings for the storm drain system near the Jones Stadium and detail drawings and records regarding the stadium pump system. The documentation was used to properly develop the SWMM model for accurate simulation.

Jones-AT&T Stadium Pumps

At this beginning of this study, questions were raised about the potential impact of the pumps that remove the stormwater that drains directly from the football field on the timing and amount of flow in the 8-ft by 8-ft Line F conduit. The primary sources of information were TTU Operations Division staff members James Thornton, unit supervisor in Engineering Services, and Jamie Doggett, electrical foreman in Building Maintenance and Construction. We also visited the pump vault with TTU plumber Victor Marquez. Victor visits the pump vault regularly to check the operational status of the pumps.

The pump system was originally installed in 1960 when the football stadium expansion was completed. Figure 1(a-d) shows the location of the pump vault from an aerial view, one of the inlets to the drainage channel, a view looking down the southeast ramp toward the with the vault door on the left, and a direct view of the door, respectively. The storm runoff from the 2.3-ac crowned field drains into the inlets provided at the perimeter of the field and into an open channel that leads toward the sump in the pump vault. Figure 2 displays a side view of the four pumps and motors, the 12 ft by 24 ft by 12 ft deep sump below, and the positions of the floats that are used to actuate the pumps one at a time as the water level in the sump rises. Pump 1 is a 20-hp pump with a nominal design flow rate of 1000 gpm at 50 ft of total dynamic head. Pump 1 was replaced in 1999, and the manufacturer's pump performance curve is shown in Figure 3. Pumps 2, 3, and 4 are still in service with 100-hp motors and target flow rate of 6500 gpm each at 51 ft of total dynamic head. The manufacturer's pump performance curve is shown in Figure 4. Figure 5 (a-d) shows the weir leading to the sump, the float actuators, pump 1, and pumps 2-4. During his visits to the pump vault, Victor Marquez tests each float actuator to make sure the related pump turns on and off appropriately.

The pump system is wired into the Emergency Management Office's SCADA system as shown in Figure 6. Under current programming, the SCADA system can identify that one or more pumps is turned on during a given storm event, but the code does not collect and save data for the time of operation of each pump during that event. This mode of operations means that there are no records for pump operations during any historical or recent storm events. The pumps must operate for water to be removed from the sump after draining from the field, so the pumps have been working in the past. We recommended to Jamie Doggett that the programming should be modified to allow collection of operational timing in the future, and he is ready and able to make that change.

The pump system configuration description was sufficient to allow simulation in our SWMM model for the storm water drainage system. Those efforts are explained in the Hydraulic Analysis section.

Figure 1. Photographs of Lift Station Location and Field Inlets

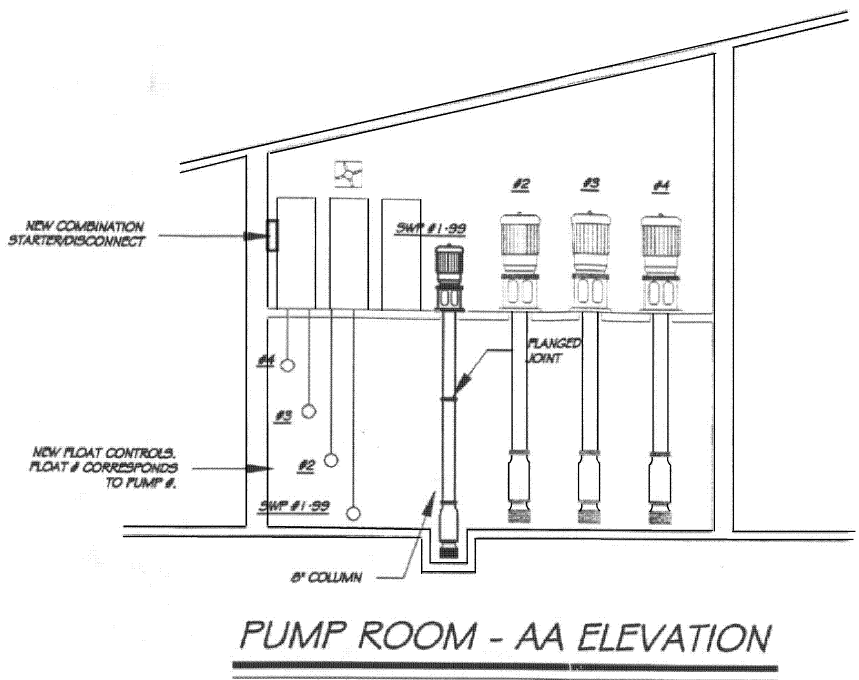


Figure 2: Sketch of Pump Vault from TTU Engineering Services files

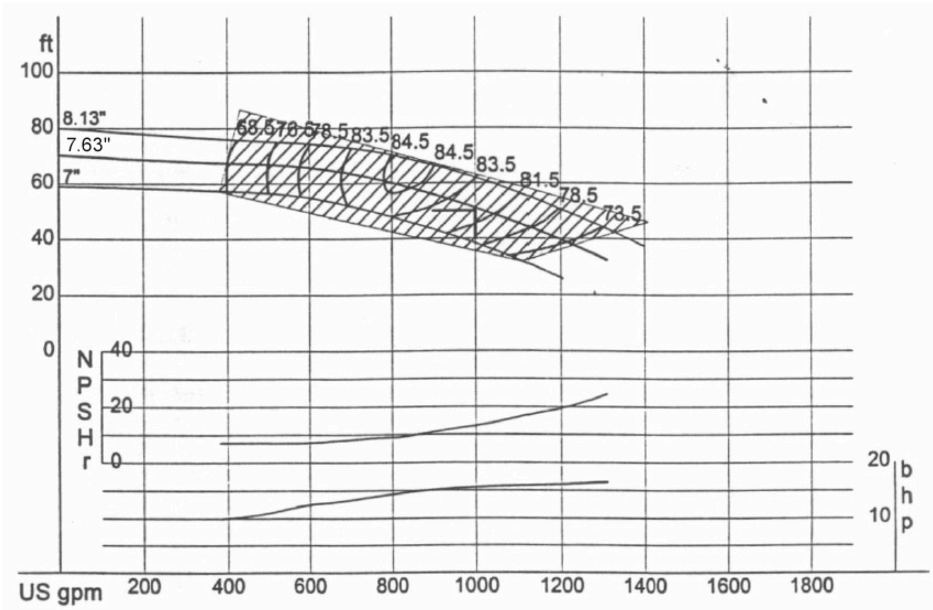


Figure 3: Manufacturer's Performance Curve for Pump

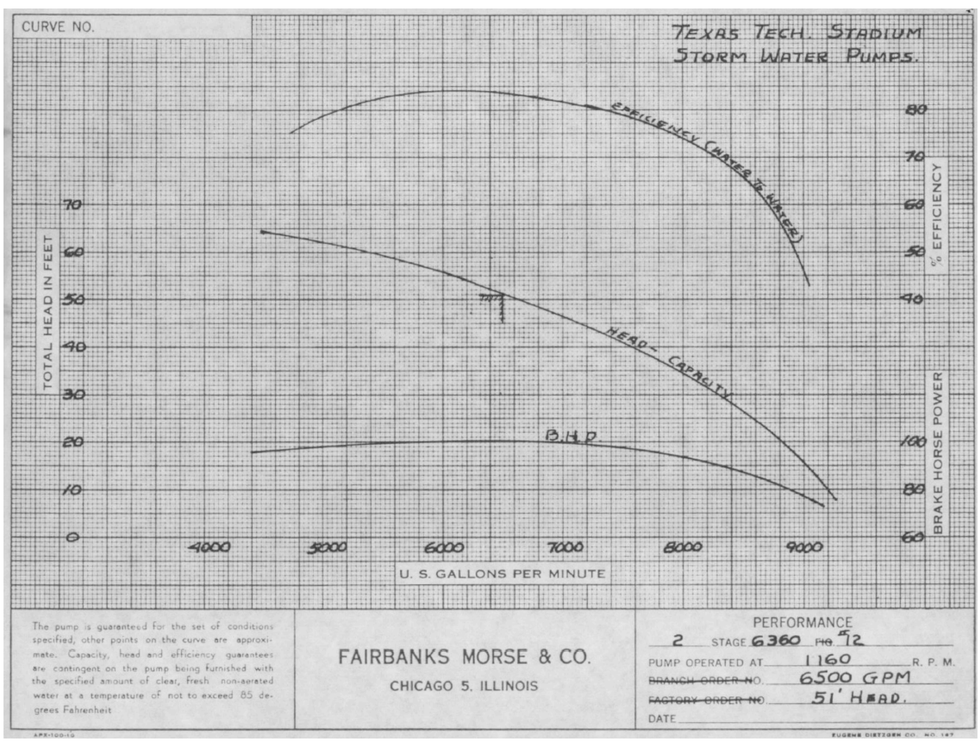
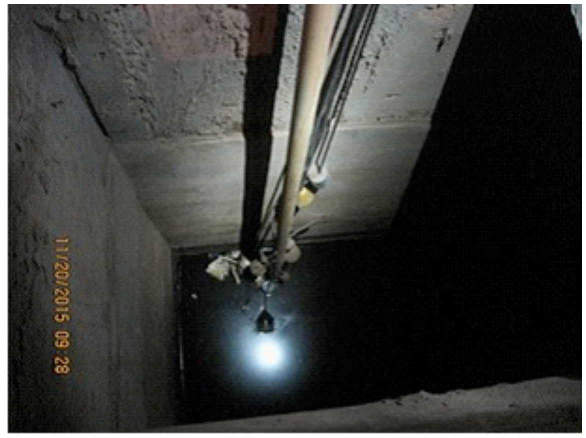


Figure 4: Manufacturer's Performance Curve for Pumps 2, 3, and 4

(a) Overflow weir into sump



(b) Looking down at float actuators

(c) 20-hp 1000-gpm pump

(d) Three 100-hp 6500-gpm pumps

Figure 5: Photo Views within the Pump Vault

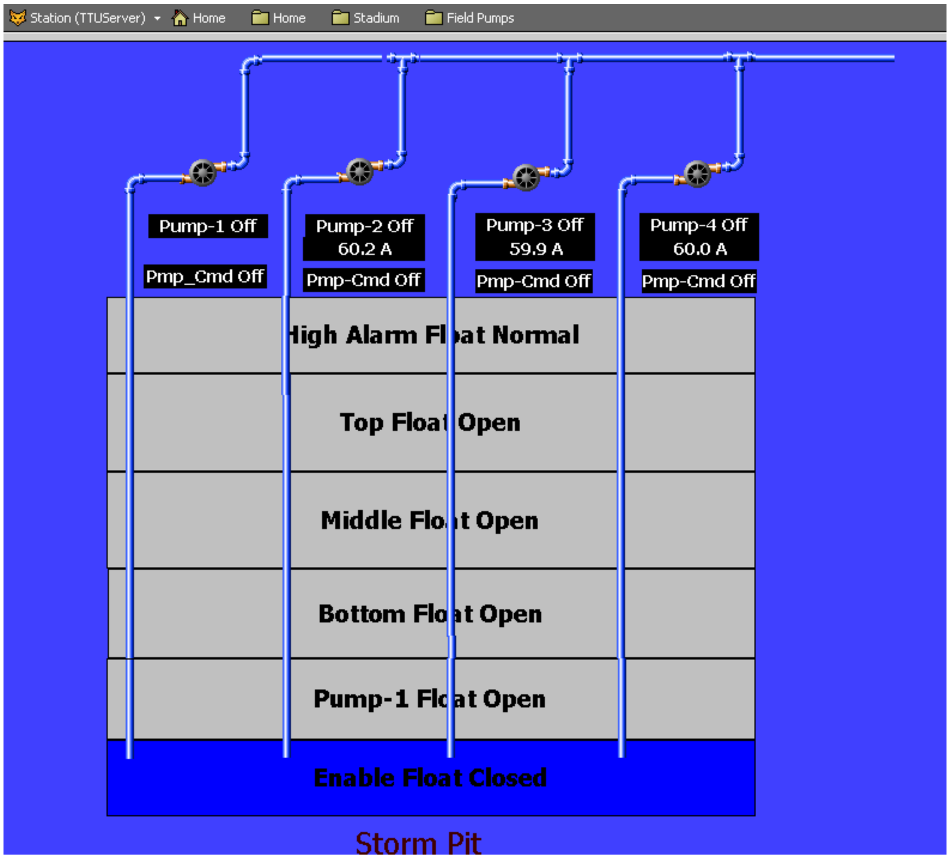


Figure 6: Screen Capture from TTU Emergency Management Office SCADA System

Hydrologic Analysis

Concepts involved

Rainfall Background and Analysis

Rainfall/runoff relationships (e.g. Rational Method) emphasize the application of Depth Duration Frequency (DDF) or Intensity Duration Frequency (IDF) rainfall predictions. The methods for developing such relationships involve the application of statistics and data analytics with time series data. Two types of time series procedures typically used in hydrology are Partial Duration Series (PDS) and Annual Maxima Series (AMS). AMS involves the statistical analysis of yearly maximum values and provides predictions for Annual Exceedance Probabilities (AEP), or the likelihood of the annual maximum event equaling or exceeding a desired threshold. The latter of the two methods, PDS, evaluates the Average Recurrence interval (ARI) of a desired threshold. Historically, both methods have been used to formulate rainfall predictions; however, AMS is typically implemented more frequent.

A review of the hydrologic design for Martha Sharp indicates rainfall values were developed from TP-40 (Hershfield, 1957)/Hydro35 (Frederick et al., 1977). Both reports (TP40/Hydro35) evaluate AMS and converted to PDS using linear model conversions outlined in Table 2 of TP40. The statistical distributions used in both reports were Fisher-Tippett Type I and followed the Gumbel fitting procedure (1958).

In the late 80's statistical procedures such as Probability Weighted Moments (PWM) and Linear Moments (LM) were developed at IBM Research Division by Hosking, J. R. M. These methods are now the most utilized statistical procedures for rainfall and flow statistics; replacing conventional statistics implemented in TP40. In 1998 the USGS performed an AMS rainfall study of Texas using L-moments with 15-muinute, hourly, and daily data up 1994 (Asquith, 1998). In 2004 that study was expanded to what is now known as the Texas Rainfall Atlas 2004 (Asquith and Roussel, 2004). Most Texas Regulatory Agencies have adopted the results of this this study as the new DDF values; replacing NOAA's TP40 and Hydro35.

As part of this research project, we were tasked with evaluating recent pivotal storms in Lubbock from 2014-2015. This objective required updating DDF annual maxima rainfall totals using NOAA 15-minute /hourly and 5-minute Mesonet data. This project implemented statistical methods similar to the ones outlined by USGS's 2004 Atlas, deviating by implementing the Kappa distribution in place of the Generalized Logistic function. The durations evaluated were 5, 10, 15, and 30 minutes and 1, 2, 3, 6, 12, 24 hours for AEP of 0.5, 0.2, 0.1, 0.4, 0.02, and 0.01 (return periods of 2, 5, 10, 25, 50, 100years).

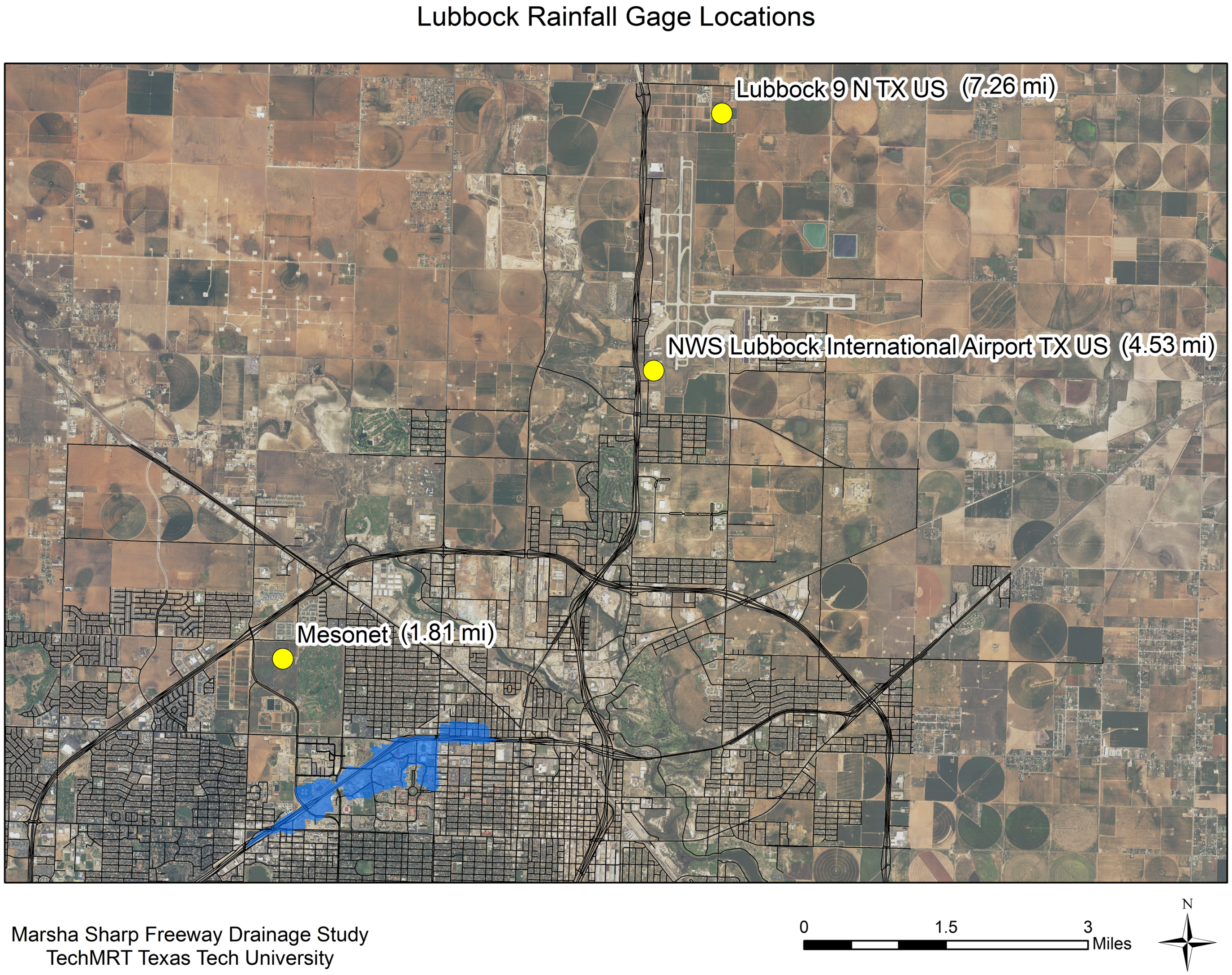


Figure 7: Map of Rainfall Collection Locations

Data

The data used in this study included the following sets.

Mesonet (5-min)

The gauge summary can be found at http://www.mesonet.ttu.edu/site_info.html. This location was established in October of 2002 and provides ~14 years of 5-minute data. The data was retrieved via R scripts at html links such as:
http://meso-file1.tosm.ttu.edu/tech/1-output/site.php?id=33&date=12072015;
The last charter string of the html was modified to retrieve other dates.

NOAA(15-min)

Fifteen-minute rainfall data was downloaded from the National Climatic Data Center (NCDC, http://www.ncdc.noaa.gov/cdo-web/datatools/findstation) at one NOAA site (Lubbock 9N). The years covered from in this data set were from 1971 to 2013. Currently, the data from 2013 to present is in the review process at the NCDC and cannot be released to the public. However, the missing data were provided from Mr. Steve Cobb, Lubbock local NOAA employee, on 12/3/15 via email.

NOAA(1-hour)

Hourly data was downloaded from the NCDC at http://www.ncdc.noaa.gov/cdo-web/search?datasetid=PRECIP_HLY# for two locations (Lubbock N9 and International Airport) covering years 1952-2015.

Annual Maxima Series

The four data sets retrieved from NOAA/Mesonet were processed into annual maxima series for the 10 durations> Mesonet data were used to develop the 5- and 10-minute intervals, 15-minute Lubbock 9 were used to create the 15 and 30 minute intervals, and the hourly Lubbock 9 and International Airport stations were used to develop the 1-24 hour intervals. Tables of the AMS data are provided in the appendices of this report.

Statistical Analysis

The statistical procedures used in this report followed the methods outlined in Asquith (2004). This analysis deviated slightly from the following method by using the Kappa distribution in place of the GLO function. Current research has shown that this distribution provides a slightly better fit to tail ended data set; hence the application in place of the GLO. The statistics were accomplished by using the R Package library LMOMCO at https://cran.r-project.org/web/packages/lmomco/index.html , developed by Dr. Asquith. This package and guidance from Asquith (2011) provided the workflow for calculating the DDF values in this study. The DDF results of this study are provided below.

Table 2: Annual Maxima Depths (in) from DDF Results (2015)

FREQ	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-h	24-hr
2	0.61	0.85	1.05	1.37	1.54	1.83	2.09	2.14
5	0.89	1.27	1.58	2.01	2.26	2.68	3.11	3.09
10	1.05	1.58	1.96	2.46	2.76	3.24	3.76	3.81
25	1.23	2.00	2.48	3.03	3.42	3.91	4.52	4.87
50	1.33	2.34	2.90	3.47	3.92	4.38	5.04	5.78
100	1.42	2.69	3.34	3.91	4.43	4.83	5.51	6.81

Comparison to Previous Studies (TP40/Hydro35)

Table 3: Annual Maxima Depths (in)

FREQ	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
2	0.81	1.10	1.38	1.65	1.81	2.08	2.37	2.68
5	1.06	1.45	1.83	2.20	2.42	2.80	3.21	3.65
10	1.26	1.72	2.18	2.64	2.91	3.38	3.89	4.45
25	1.48	2.02	2.56	3.09	3.40	3.95	4.53	5.18
50	1.63	2.23	2.84	3.44	3.80	4.44	5.13	5.89
100	1.86	2.54	3.23	3.92	4.32	5.04	5.81	6.67

Table 4: Percent Change from Previous Studies to DDF Results

FREQ	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-h	24-hr
2	-25%	-23%	-24%	-17%	-15%	-12%	-12%	-20%
5	-16%	-12%	-14%	-9%	-7%	-4%	-3%	-15%
10	-16%	-8%	-10%	-7%	-5%	-4%	-3%	-14%
25	-17%	-1%	-3%	-2%	1%	-1%	0%	-6%
50	-18%	5%	2%	1%	3%	-1%	-2%	-2%
100	-24%	6%	3%	0%	3%	-4%	-5%	2%

The results of this study appear to be fairly consistent; excluding the 15-minute durations and 2-yr frequencies. The variation between these values is likely from the increase in amount of data between the two studies. Hydro-35 evaluated data up to 1977 (~6 years), this study (~43 years) and the conversion between AMS and PDS performed in Hydro-35 (not performed in this study). Additionally, the reader should draw attention to the depths from 1-3-hour 0.02 AEP (50-year) frequencies. These values are significant to this study because the design of the storm sewer system (sag roadway) required a 50-year frequency. The comparison of this study with TP40 (50-yr freq., 1-3 hours), indicates the magnitude of rainfall has not changed (i.e. the design storm has not changed).

Rainfall Multiplier

In adjustment for SWMM modeling of precipitation was incorporated based on the storm depths based on the arithmetic mean value of the difference between the largest and smallest depth for each storm. The "range adjustment" is the value that the smallest observed depth must be multiplied by to produce the largest value. The multiplier was applied in the SWMM model to reflect that the storms could have been larger on the drainage area than the nearby gage values – the model is assuming that the observed values represent a "smallest value" for three gages a few miles apart, and the "largest value" (result of the multiplier) is what is applied to the drainage model.

The numerical value of the multiplier used is 1.44 and was only applied to the historical storm simulations.

Table 5: Storm Depth Summary Used for Range Adjustment

	MESONET		L9N		LIA	
STORM DATE	DEPTH (INCHES)	DURATION (HOURS)	DEPTH (INCHES)	DURATION (HOURS)	DEPTH (INCHES)	DURATION (HOURS)
09/24/14	2.45	9.33	2.38	9.00	2.94	10.00
05/04/15	4.31	15.83	4.94	7.50	3.56	8.00
05/28/15	2.22	6.75	2.14	4.00	3.21	5.00
07/06/15	1.54	11.00	2.99	7.75	2.49	15.00
10/21/15	2.24	5.16	2.10	9.00	1.96	5.00

Watershed Delineation

When developing the hydrologic conditions for the storm sewer system, a drainage area was manually delineated for each individual storm sewer inlet. The software used for drainage area delineation and measuring additional hydrologic parameters was ArcGIS. The topography used for delineation was supplied by the City of Lubbock and developed based on the 2011 LiDAR of Lubbock.

The areas acquired from delineation based on the 2011 LiDAR were compared to the drainage areas in the 2004 plans. Table XX summarizes the total areas for each storm sewer line in the project as well as the area of Texas Tech campus that drains to the freeway. Line F is the main storm sewer line running parallel to the centerline of Marsha Sharp Freeway. A detailed table comparing each individual inlet drainage area to the 2004 plans is provided in Appendix XX.

Table 6: Summary of Drainage Areas

LINE	AREA (ac)
Line C	48.94
Line E	6.67
Line F	224.28
Line G	19.74
Line H	7.74
Campus	194.59
TOAL	**501.97**

The drainage area designated as campus was initially represented by constant additional flow set approximately at 145 cfs in the 2004 WinStorm model. The additional flow is distributed as 28.96 cfs to five individual nodes, Inlet-G10, F80, and Junction-JG2, JBG2, JBG1. This was used to represent flows that could be anticipated from the 36” pipe coming into the Marsha Sharp network along University. For design, the pipe along University Avenue was assumed to be flowing full to account for the James Stadium pumps. According to this study, campus includes a substantial area of 195 acres that contributes to the drainage along University Avenue towards Marsha Sharp Freeway. This includes approximately 50 acres of drainage area along University Avenue as well as an additional 138 contributing acres from campus.

Drainage Area Conceptualized

Each delineated area is represented in SWMM as a subcatchment and is directly connected to a node. SWMM treats a subcatchment as a rectangular space with uniform properties throughout the drainage area. Parameters specific to each subcatchment included an area, slope, width and curve number. The slope was estimated based on the highest and lowest point in each specific subcatchment and a length was measured based on the furthest point in the watershed to the outlet of the watershed at the inlet. The lengths were used to calculate a representative width based on the total area of a watershed.

Each drainage area that contributes to the storm sewer system was modeled using an NRCS curve number approach. Rainfall is applied to the drainage area and the CN method is used to generate a time history of runoff from that drainage area. These runoff values are then routed (using the SWMM defaults) to the outlet of the drainage area which coincides with various inlets identified from the MS4 GIS database.

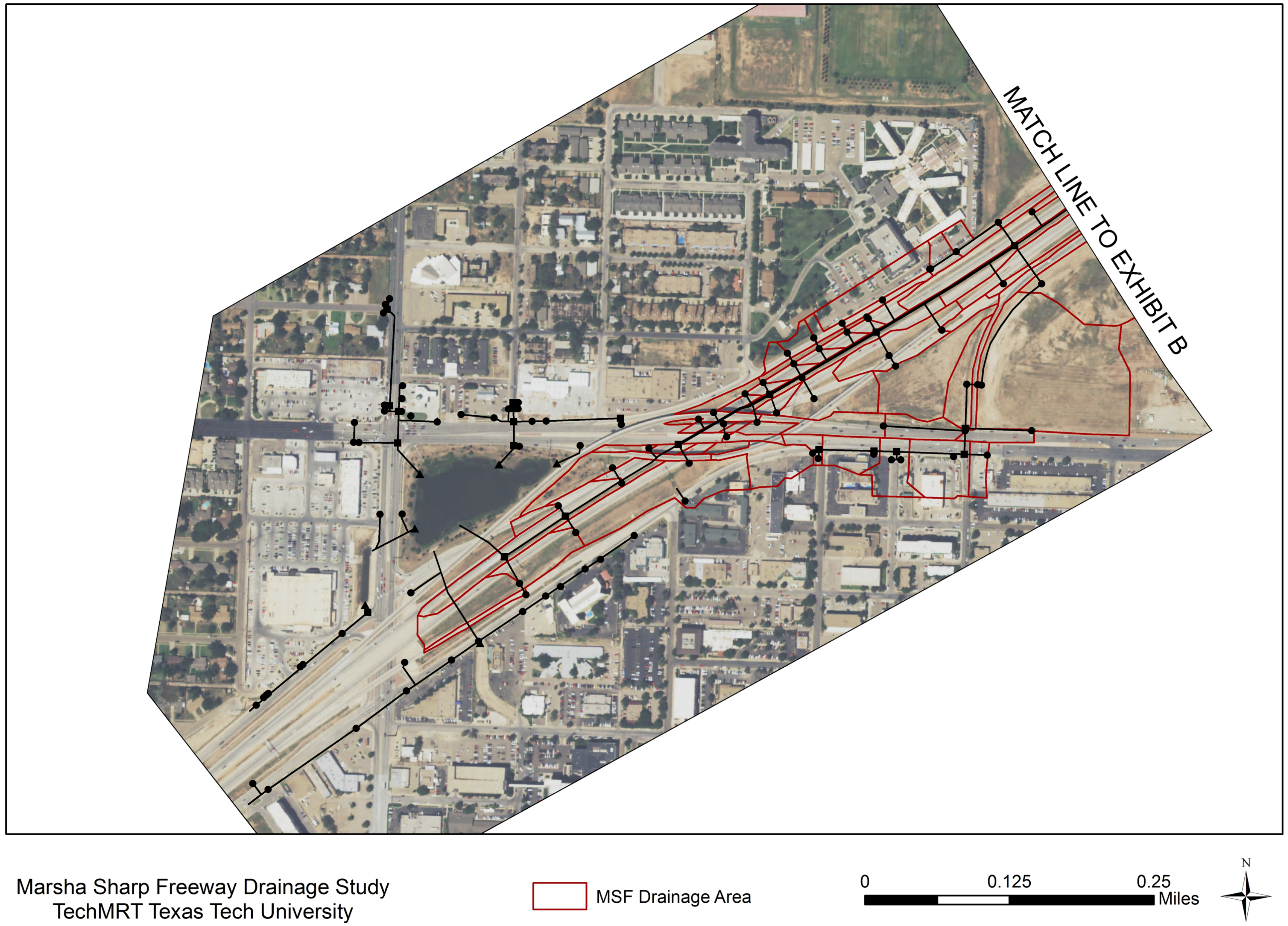
The curve numbers employed are conceptually "composite" CN.[2] The numerical values were determined using the WinStorm discharges from the 1994 design. A 4-hour duration, constant rate storm, which produced the same depth as a design storm with a duration equal to the longest time of concentration storm in the 1994 plans was used.

The use of a storm of duration longer than the time of concentration is to allow the computer model to be close to equilibrium discharge for each drainage area when the comparison is made (and CN adjusted to closely match the discharges).

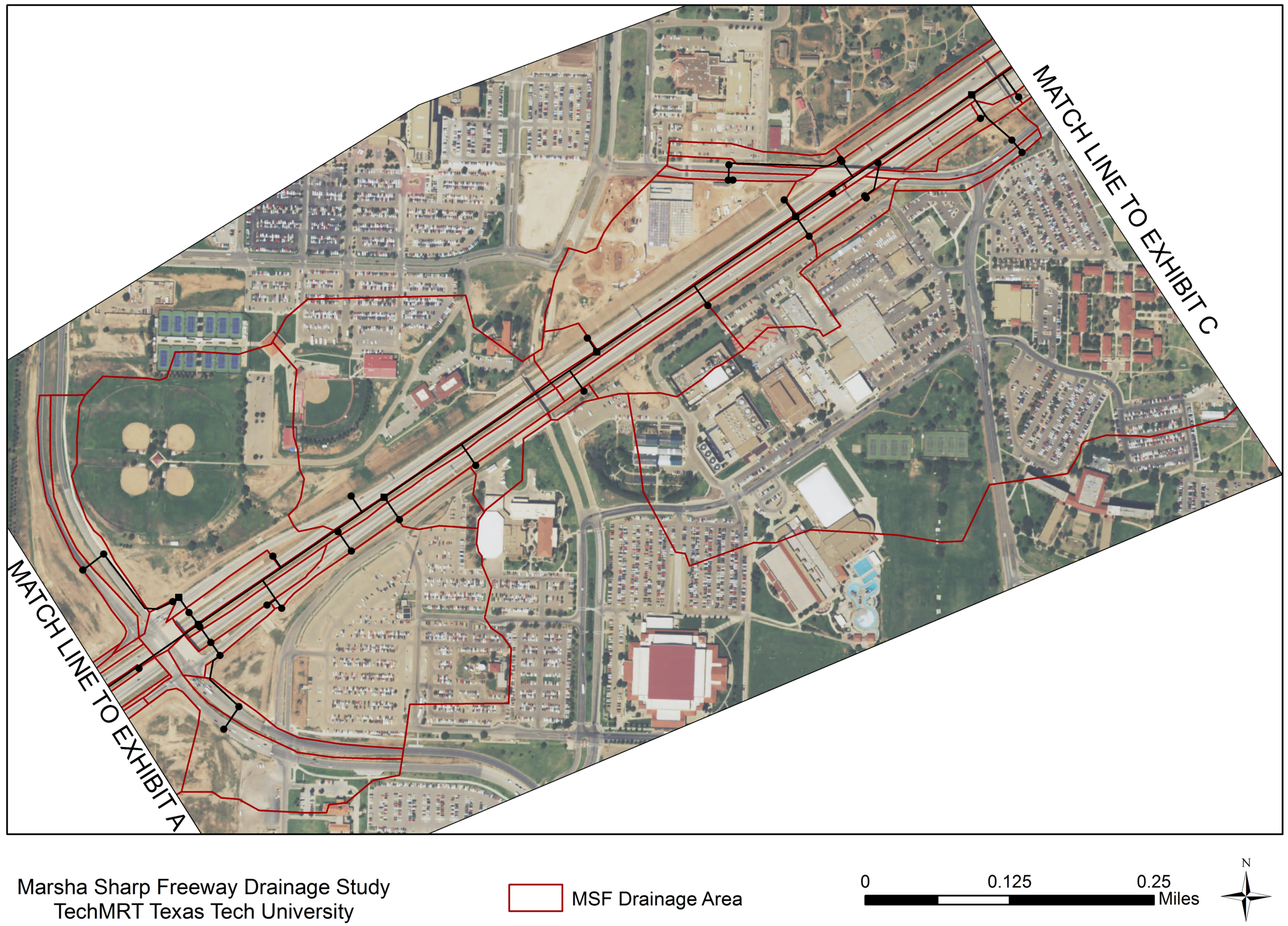
[2] Composite in the same context described in the Hydraulic Design Manual (cite) and NEH-630 (cite).

Marsha Sharp Freeway Drainage Areas Exhibit Overview

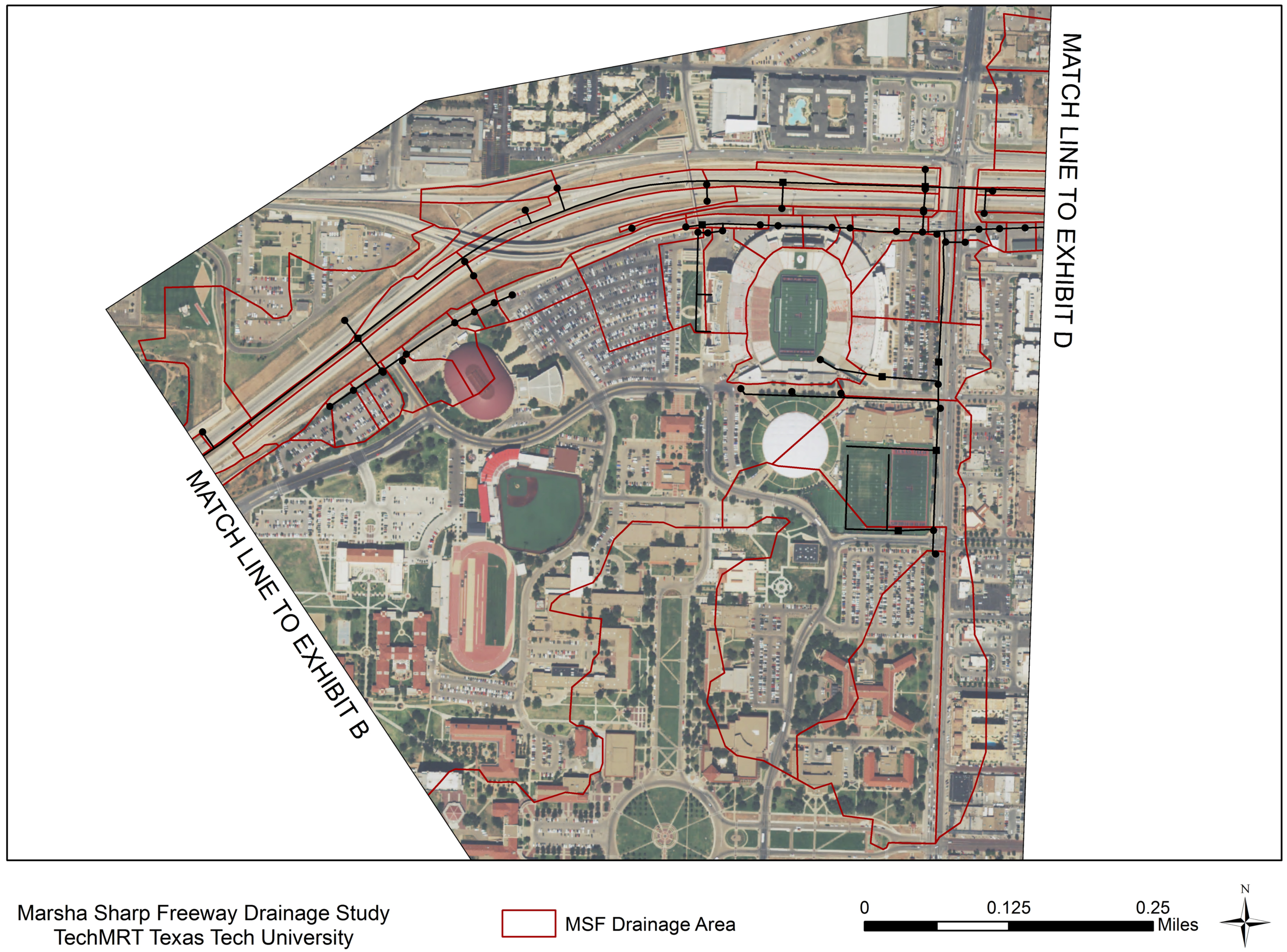
Marsha Sharp Freeway Drainage Areas Exhibit A



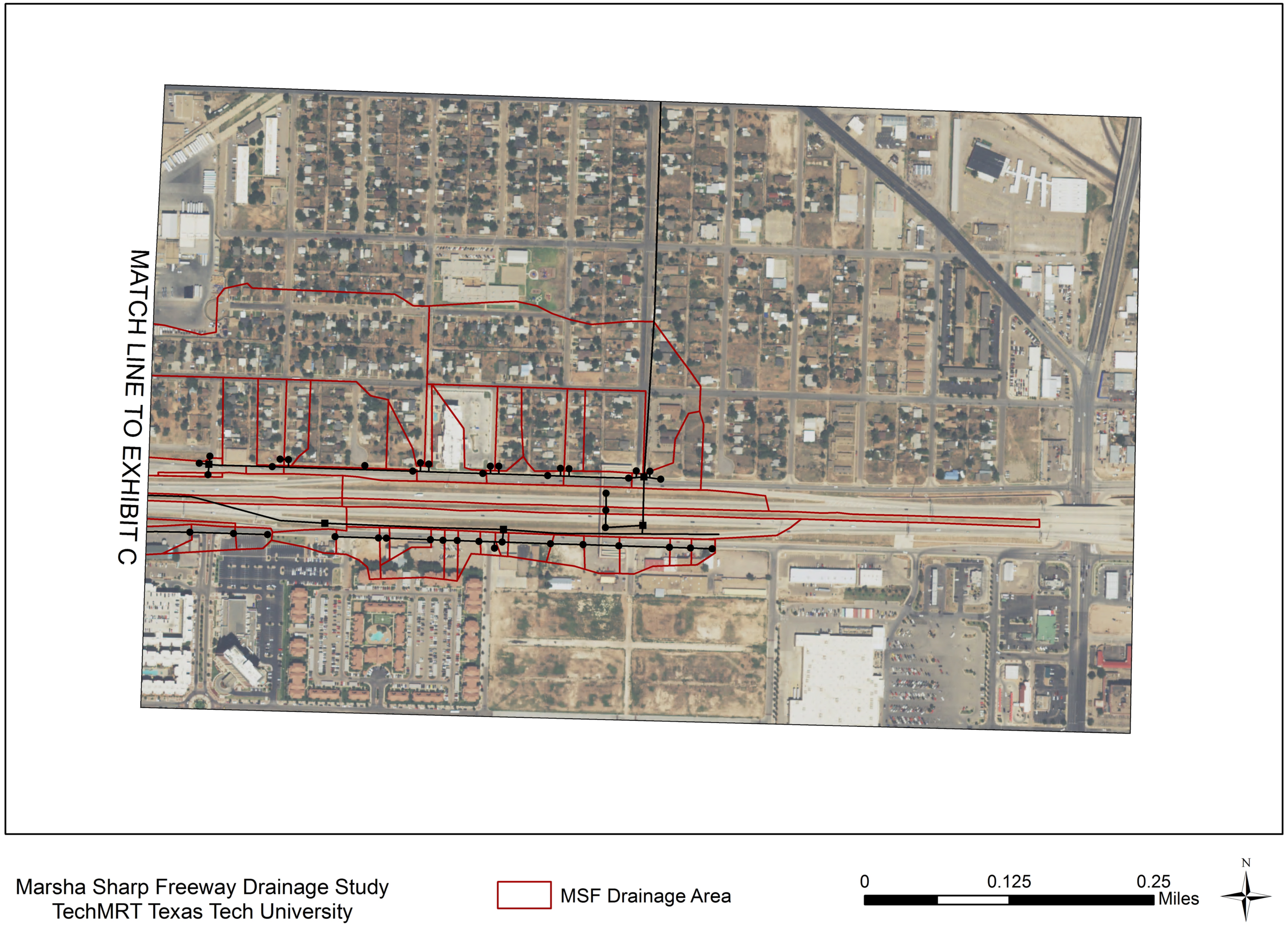
Marsha Sharp Freeway Drainage Areas Exhibit B



Marsha Sharp Freeway Drainage Areas Exhibit C



Marsha Sharp Freeway Drainage Areas Exhibit D



Drainage Areas Inconsistencies

Several inlets were found to have inconsistent documentation during the review and development of the model. The inconsistencies refer to instances where the inlets were included in the hydrologic/hydraulic summary sheets but could not be found in the plan and profile sheets or vice versa. All inlets in question were confirmed by aerial images or by field verification

1. F36: Was an inlet documented in the WinStorm model as an area, but could not be found in the drainage area or in the plan and profile sheets. This inlet was not included in the SWMM model.
2. G2: Had a designated drainage area in the summary drainage area sheet, but was not included in the plan and profile sheets. After driving by and verifying the inlets on the northwest corner of the Jones stadium, Inlet G2 is an existing inlet and was therefore included in the model. A grade was estimated for the inlet by the surface generated from the 2011 LiDAR and the max depth was estimated by measurements taken and documented in the field.
3. F48S: This inlet was included in the summary runs for the WinStorm model as an area, but was not found in plan and profile sheets. This inlet was not included in the SWMM model.
4. C19 and C20: Both of these inlets were included in the detailed drainage area maps and plan and profile sheets, but was not seen in the summaries of the WinStorm model results. These inlets were included in the SWMM model.

Playa Lake 44 Drainage Area

The main storm sewer line along Marsha Sharp, Line F, begins with an 18-inch lateral feeding into the main 48-inch centerline conduit that eventually develops into an 8'X8' box culvert. The 18-inch pipe intakes storm water runoff from the Playa Lake 44. Playa Lake 44 is made up of a West and East Lake located on their respective sides of Quaker Avenue. The areas contributing as runoff into the Playa Lakes were not delineated for this project. The areas used were taken directly from the 2004 study completed by TxDOT. For the East Playa Lake, the areas for Lines S, R, P and Lateral Q2 were taken respectively from their individual hydrologic/hydraulic sheets. Each was considered as an individual subcatchment in the SWMM model. The same was done for the West Playa Lake regarding Line W, Lateral Q1 and Culvert-2 drainage areas. To fulfill the parameters for each subcatchment contributing to the West and East Playa Lake 44 individually, ArcGIS was used to evaluate the topography to estimate a slope and width.

Hydraulic Analysis

A set of US EPA Storm Water Management Model(s) (CITE) were built to examine the hydraulics of the drainage system for selected historical and design storms. Multiple models were built that reflect different physical configurations and storms.

SWMM Model Components

The components used in the SWMM model are sub-catchments, nodes and storage elements, conduit elements, and a lift station element.

Sub-catchments in SWMM are areas that catch precipitation and convert it into runoff according to the modeler specified loss model. In the study, the NRCS CN model as used within SWMM. The numerical values of CN were adjusted by trial-and-error calibration to produce discharge for each catchment that was consistent with the WinStorm model for the same drainage areas in the 2004 (CITE) plans.

Nodes in SWMM are used to connect conduits (links); a node in SWMM has essentially zero storage and in the production model for the study, most connections are made using storage elements (nodes that have a depth-volume relationship). The important data for a node is its invert elevation, which represents the elevation of the bottom of the node above some datum. The invert elevation is synonymous with the flow line elevation. The top of a pipe is the crown elevation, which is synonymous with the soffit elevation.

Conduit elements in SWMM are either regular geometry sections (circular pipes, box culverts) or irregular geometry sections. In the study both kinds of elements were used – the subsurface drainage system was comprised of pipes and box culverts that could either flow partially full (as an open channel) or surcharged (pressurized flow). University Avenue, and portions of the surface drainage system adjacent to Marsha Sharp were modeled as either a trapezoidal channel or a generic street cross section.

Hydraulic connection between the surface and subsurface system occurs at nodes/storage elements where both kinds of conduit elements connect. The different invert elevations of a buried pipe and surface flow conduit are conveyed to the program using inlet and outlet offsets – these offsets are the distance from the connecting node invert elevation to the bottom of the particular conduit element. The direction "inlet" and "outlet" are defined by how the conduits connect two nodes – the inlet is the starting node or the connection and the outlet is the ending node of the connection. Actual computed the algebraic sign of the computed flow maintains actual flow directions during hydraulic calculations.

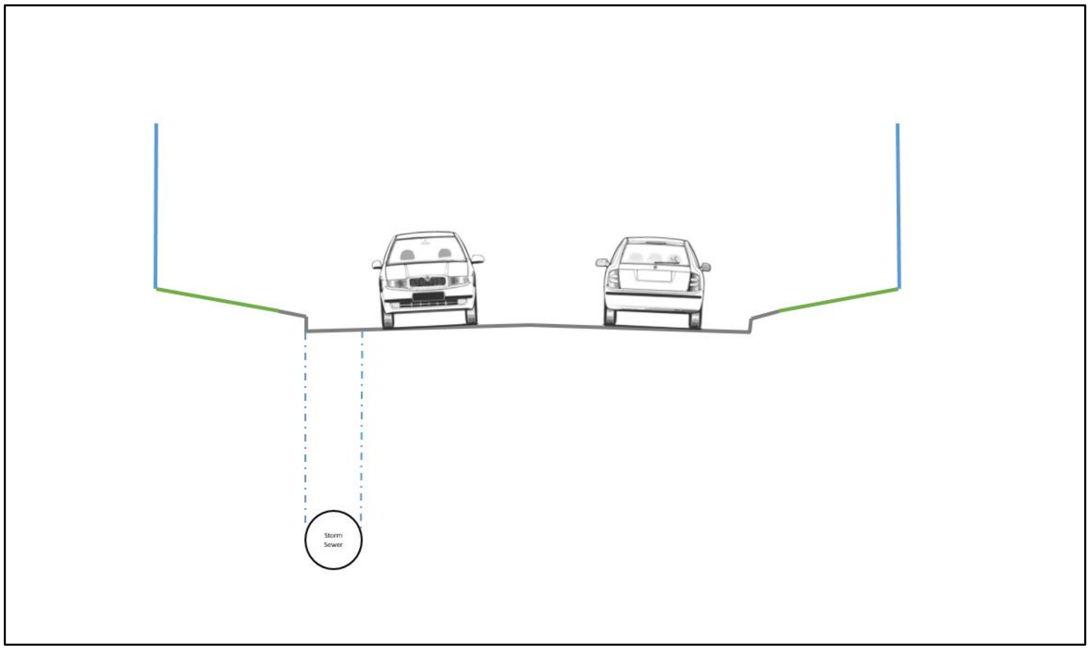


Figure 8: Roadway and Storm Sewer Cross Section

The invert elevations for the nodes were determined directly from the various drawings used to construct the SWMM model topology. Where elevations conflicted between multiple sources (GIS database, and drawings), the 2004(CITE) drawings were used as the authoritative source. In locations where no elevations were on the drawings, nearby elevations were used to guide modeler estimates for these elevations.

The Jones Stadium sub-catchment was modeled as a substantial storage element (the stadium stores all water it catches) or with an active pumping system. In the cases where the stadium was assumed to store its entire precipitation catch, the pump system was simply programmed to delay pumping until 100 feet depth in its inlet node (the stadium would store water at about 13.5 feet above the sump (1 to 1.5 feet at the field level).

When the pump system was enabled in the model, it was simulated using the Type-II pump operating rules in SWMM. A SWMM Type-II pump system produces constant flow rates at different inlet node depths – it would reflect the relatively large capabilities of the Jones Stadium system in conjunction with the relatively small volume wet-well.

Several lift station “performance curves” were developed during the study; these are listed in Table XX below. For most of the comparisons the LOW-SET curve was used because the other two curves performed in a manner that the modelers determine would damage a pump system (cycle on and off too fast).

Table 7: Jones Stadium Pump Station SWMM Curves

PUMP CURVE NAME	Q_{max} (GPM)	REMARKS
JONES-PUMP-LOW-SETS	21,184	No discharge until 3 feet depth in sump; reduces the on/off cycling of multiple pumps in the computer program
JONES-PUMP-ORACLE-SETPOINTS	21,184	Set points match pump switch elevations reported on Jones Pump drawings

Model Calibration Notes

The model was calibrated to anecdotal evidence based on the 09-24-2015 storm, which motivated the study – in particular the photographic evidence of inundation depth near node F79.

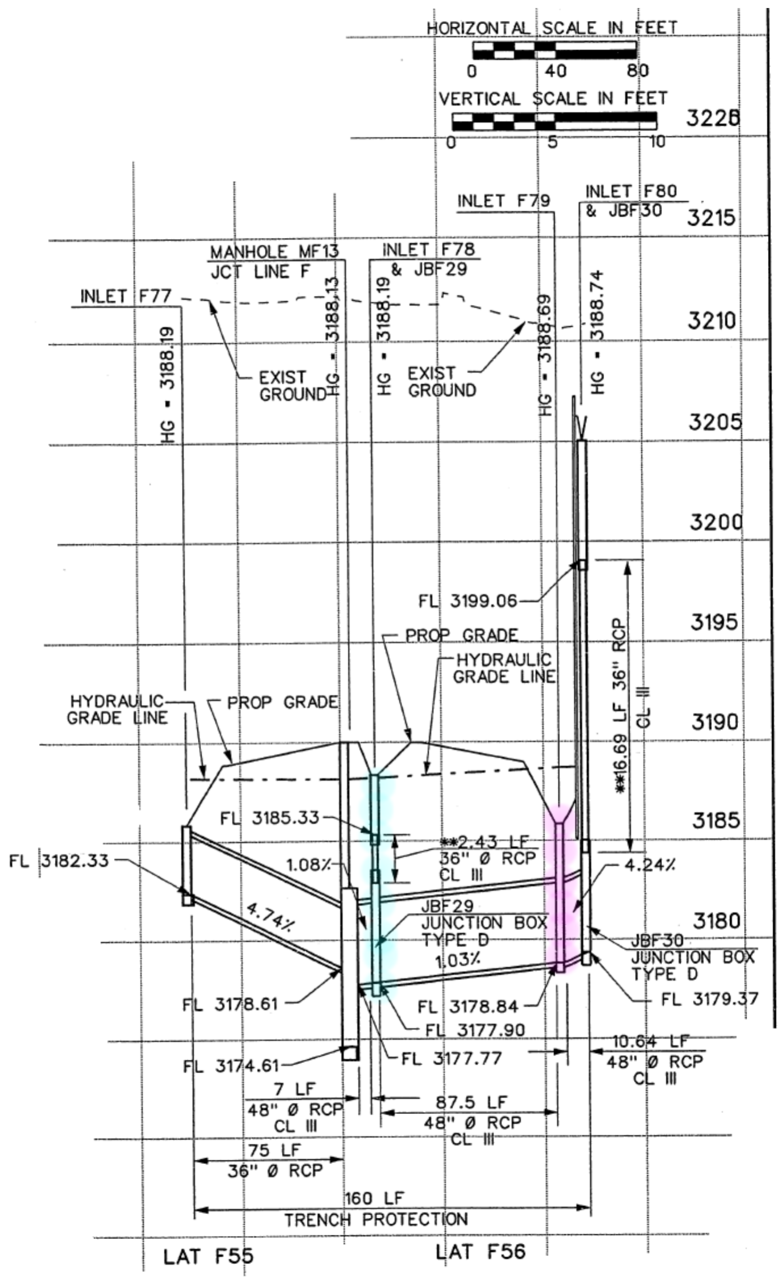


Figure 9: Node F79 (magenta) and Node F78 (cyan) Locations. Flow in the Drawing is from right to left. The 8X8 box is located to the left of Node F78 in the drawing.

Figure XX is a copy from the design drawings that displays the laterals and the nodes in the median, and shoulder edge of the Marsha Sharp Freeway. The two locations of interest are annotated with cyan and magenta. Storm water flow in the diagram is from right to left to the 8X8 box conduit, which is roughly in the middle of the figure. Flow from Node F77 to the box is left to right.

The entry and exit loss coefficients for all conduit elements were eventually set at K=1.5, which is a relatively large value, but consistent with prior studies (Wang et. al, 1998) of complex, surcharging sewer systems. The pipe and surface roughness was selected to be n=0.016 for most conduits in the system.

During model development 7-foot portion of conduit between Node F78 and the box in Figure XX was modeled with a higher roughness coefficient (n=0.035), higher inlet and exit losses (K=3.0), and an intentional soffit elevation mismatch of 6-inches. When these "adjustments" were inserted into the computer model the results were consistent with the observations that motivated the modeling study.

These "calibration adjustments" represent modeler intuition to cause the hydraulics of the model to agree with the anecdotal observations and should not be interpreted as being real. The values of the roughness and loss coefficients are consistent with Wang et. al. 1998, although they are high values for a storm sewer system.

The storm values supplied to the SWMM model were adjusted upward by a factor of 1.4, again to produce depths consistent with the photographic evidence. The increased multiplier was justified by an analysis of the storm depths at the three nearby raingages that …

Once the model produced results consistent with the observed 09-24-15 storm, a series of production models were prepared to reflect different drainage conditions (pumps on/off) and different storm inputs. Table XX lists these various simulations. Table XX lists the filename, storm date used, and pump configuration used. The last "REMARKS" column is a notation of important features of the particular model run.

The actual SWMM input files and output files are available for download at http://cleveland1.ddns.net/research-projects/MSF-DrainageStudy/. Additional data relevant to the study are located at the same URL; these data include the report (this document), scanned images of the design drawings, and other supporting documents used in the study.

Table 8: FILENAME, Storm Date, Pump Configuration used for the modeling study.

DIRECTORY	STORM	PUMP	REMARKS
MSF-PM-DS1	SCS-50	N/A	Store water in stadium
MSF-PM-DS1P	SCS-50	LOW-SETS	Above with pump system enable
MSF-PM-DS1B	SCS-5/50	N/A	50yr line F; 5yr rest of system
MSF-PM-DS1BP	SCS-5/50	LOW-SETS	Above with pump system enable
MSF-PM-DS2N	TX-50	N/A	TxHYETO 50-yr,8 hr. duration storm
MSF-PM-DS2	TX-50	LOW-SETS	Above with pump system enable
MSF-PM-HS1	09-24-14	N/A	Store water in stadium
MSF-PM-HS1P	09-24-14	LOW-SETS	Above with pump system enable
MSF-PM-HS1PO	09-24-14	ORACLE-SETS	Above with pump system enable
MSF-PM-HS2	05-04-15	N/A	Store water in stadium
MSF-PM-HS2P	05-04-15	LOW-SETS	Above with pump system enable
MSF-PM-HS2PO	05-04-15	ORACLE-SETS	Above with pump system enable
MSF-PM-HS3	10-21-15	N/A	Store water in stadium
MSF-PM-HS3P	10-21-15	LOW-SETS	Above with pump system enable
MSF-PM-HS3PO	10-21-15	ORACLE-SETS	Above with pump system enable
MSF-PM-HS4	07-06-15	N/A	Store water in stadium
MSF-PM-HS4P	07-06-15	LOW-SETS	Above with pump system enable
MSF-PM-HS4PO	07-06-15	ORACLE-SETS	Above with pump system enable
MSF-PM-HS5	05-28-15	N/A	Store water in stadium
MSF-PM-HS5P	05-28-15	LOW-SETS	Above with pump system enable
MSF-PM-HS5PO	05-28-15	ORACLE-SETS	Above with pump system enable
MSF-PM-AM2	09-24-14	LOW-SETS	Added inlets
MSF-PM-AM3	09-24-14	LOW-SETS	Permeable pavement storage
MSF-PM-AM4	09-24-14	LOW-SETS	Pump delay approx. by surge tank
MSF-PM-AM5	09-24-14	LOW-SETS	Pump to Golf Course
MSF-PM-AM6	09-24-14	LOW-SETS	Add flow to 48-inch sewer

Results

The historical storm and the design storm models include computation runs with and without the Jones Stadium pump system active. The outputs at nodes F79 (within the MSF median) and S201 (the alley just south of Jones Stadium) for these simulation runs were used to construct plots of depth versus time for the various storms, and for different drainage configurations. These comparisons are used to assess the effect of the pumps on the depth and duration of inundation on Marsha Sharp Freeway and South of the Stadium, as well as to evaluate the effect of several conceptual drainage alternatives.

The alternative models are all compared to the storm of 09-24-2015 (HS1) with the pumps operational.

Design Storms

Three different design storms were applied: (1) An SCS Type II, 50-yr, 24 hour storm on the Line F portion of the system, and an SCS Type II, 5-yr, 24 hour storm on the remainder of the system[3]; (2) An SCS Type II, 50-yr, 24 hour storm over the entire drainage system; and (3) a Texas Hyetograph 50-yr, 8.5 hour storm (CITE) over the entire system.

For each of these storms the SWMM model was used to estimate the depth at Node F79 with and without Jones Stadium pumps contributing to the drainage system, as well as the depth at Node S201.

[3] This configuration replicates the ARI value distribution in Table 4-2 of the Texas Hydraulics Manual (CITE).

SCS Type II, 50-yr/5-yr, 24 hour Design Storm

Figure XX is a plot of the depth of storm water at Node F79 for the SCS Type II, 50-yr, 24-hour design storm applied over the Line-F drainage system and a 5-yr, 24-hour design storm applied over the remainder of the drainage system, as per design guidance in the Texas Hydraulic Manual (CITE). The contribution of the pumps to depth is a bit over 7-inches at the peak of the storm; however for this design storm the freeway would **not** have inundation in either case.

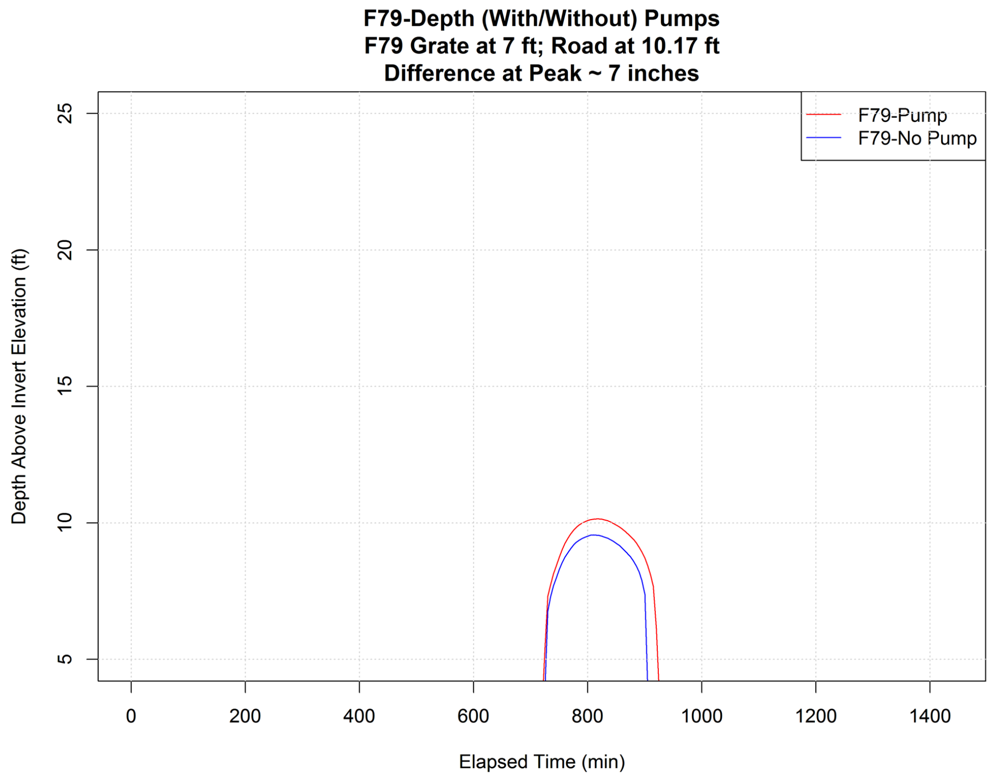


Figure 10: Depth at Node F79 for SCS Type II, 50-yr, 24-hr design storm on Line F, 5-yr, 24-hr on remainder of drainage system. Plotted on same scale as prior plot.

Interpretation

This design storm represents the current guidance reflected in the drainage design manual – that is based on the best design practices at the time of the project, the system is operating as designed.

SCS Type II, 50-yr, 24 hour Design Storm

Figure XX is a plot of the depth of storm water at Node F79 for the SCS Type II, 50-yr, 24-hour design storm applied over the entire drainage system. The contribution of the pumps to depth is a bit over 5-inches at the peak of the storm; however, for this design storm the freeway would be inundated (depth in excess of 10.2 feet) in either case for about 10 hours.

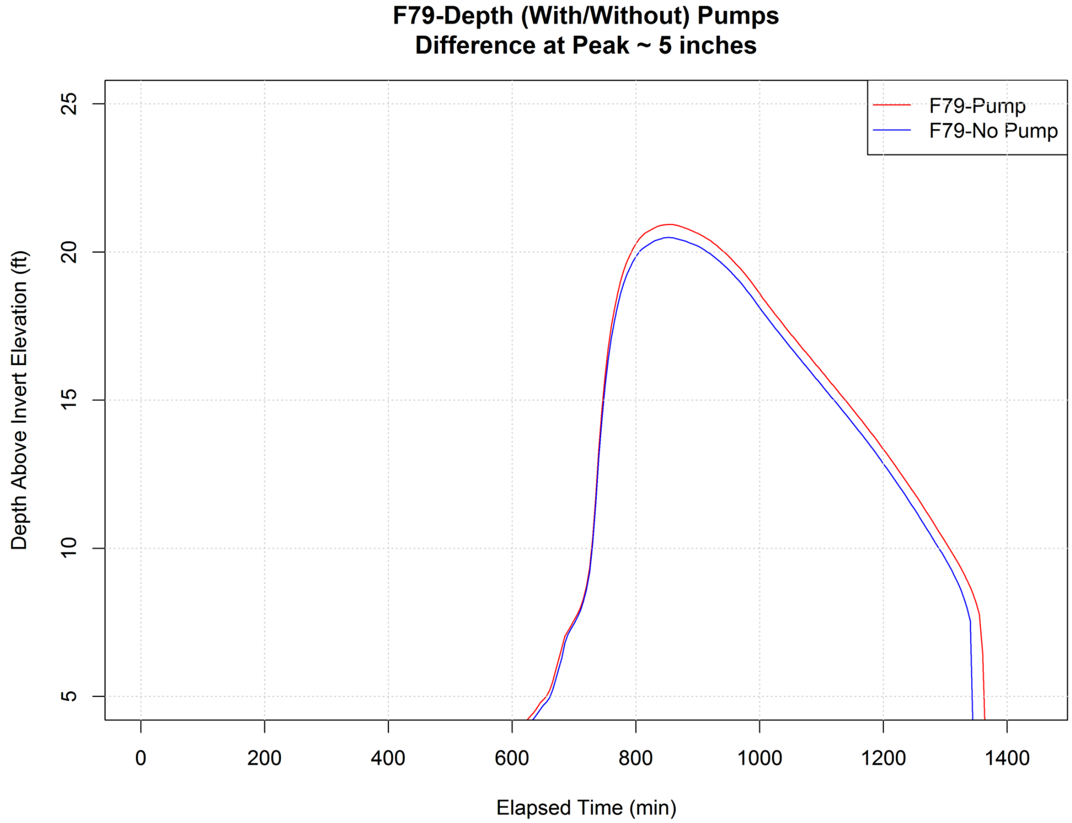


Figure 11: Depth at Node F79 for SCS Type II, 50-yr, 24-hr design storm.

Interpretation

This design storm represents application of the storm over the entire drainage network at the same magnitude – the portions of the system that are designed to accommodate the 5-yr ARI are being supplied with 50-yr magnitudes. Because the model is built such that all drainage west of F79, and south of Marsha Sharp Freeway must pass through Node F79 the inundation depth is rather substantial and of comparatively long duration.

This design storm exceeds the design guidelines at the time of the project and would not have been considered.

Texas Hyetograph, 50yr, 8.5 hour Design Storm

Figure XX is is a plot of the depth of storm water at Node F79 for the Texas Hyetograph, 50-yr, 8-hour design storm applied over the entire drainage system. The contribution of the pumps to depth is a bit over 6-inches at the peak of the storm; however for this design storm the freeway would be inundated (depth in excess of 10.2 feet) in either case for about 9 hours.

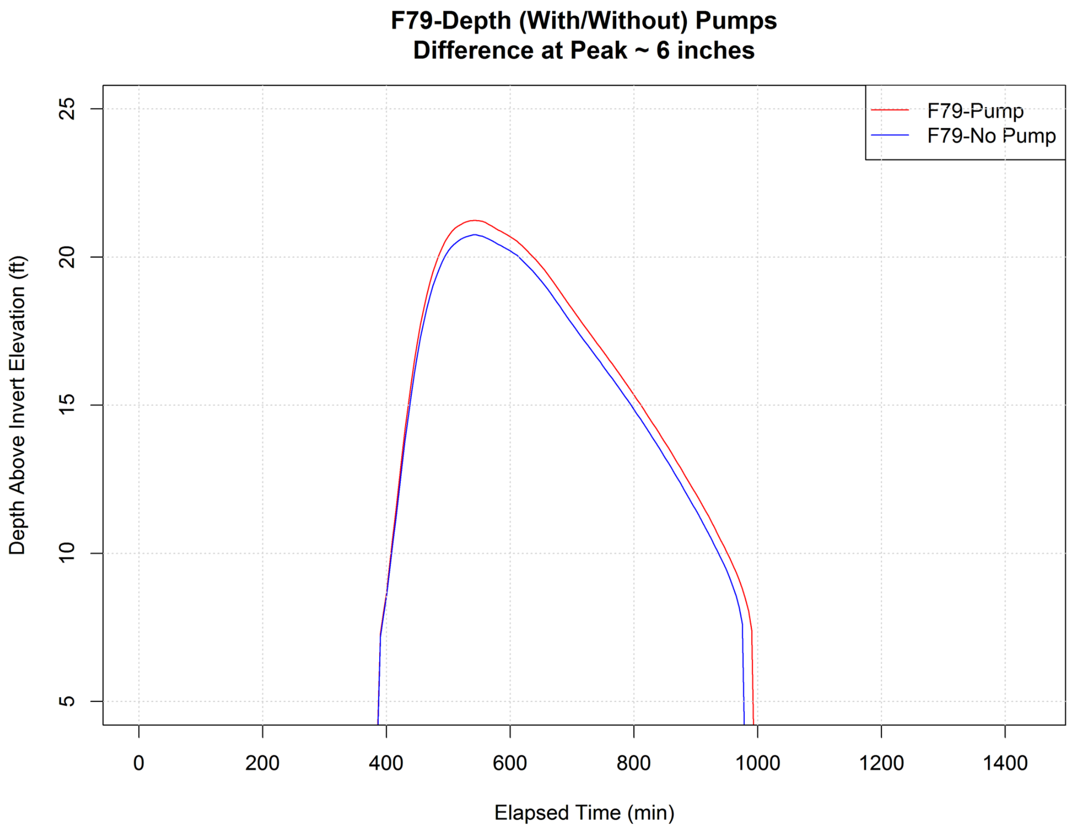


Figure 12: Depth at Node F79 for Texas Hyetograph, 50-yr, 8-hr design storm.

Interpretation

This design storm represents application of the storm over the entire drainage network at the same magnitude – the portions of the system that are designed to accommodate the 5-yr ARI are being supplied with 50-yr magnitudes. The Texas Hyetograph is a relatively recent product of TxDOT sponsored research and reflects the non-centered (in time) nature of Texas precipitation. As with the prior design storm, because the model is built such that all drainage west of F79, and south of Marsha Sharp Freeway must pass through Node F79 the inundation depth is rather substantial and of comparatively long duration.

This design storm (1) exceeds the design guidelines at the time of the project and (2) the research product was not available at the time of the project, thus would not have been considered.

The natural extension of the Texas Hyetograph to a 50/5-yr configuration was not considered because the results of the SCS and Texas Hyetograph at the 50-yr were roughly the same (peaks occurred at different simulation times, but magnitudes were about the same).

Historical Storms

Five (5) recent historical storms were extracted from the NWS database as described in earlier in the report. The storm dates are listed in Table XX.

The storm that motivated the study is designated HS-1 and occurred on 9-24-2014. In all cases the precipitation provided to the SWMM model starts several days before and runs several days after the stated storm date. This ramp-up time was imposed because the downstream boundary condition in the computer model was unstable and the long preceding time provided sufficient model time for the initial flow instabilities to dissipate.

The plots presented show the entire time history for the simulation; the storm portion is reasonably obvious in the traces.

HS-1: 09-24-2015

Figure XX is a plot of Node F79 depths for the 09-24-14 storm. The effect of the Jones Stadium pump system, is small; about 3-inches difference at the peak depth. The depth indicated in the figure would produce roughly 1 foot of water on the travel lanes and about 4 feet in the shoulder ditches.

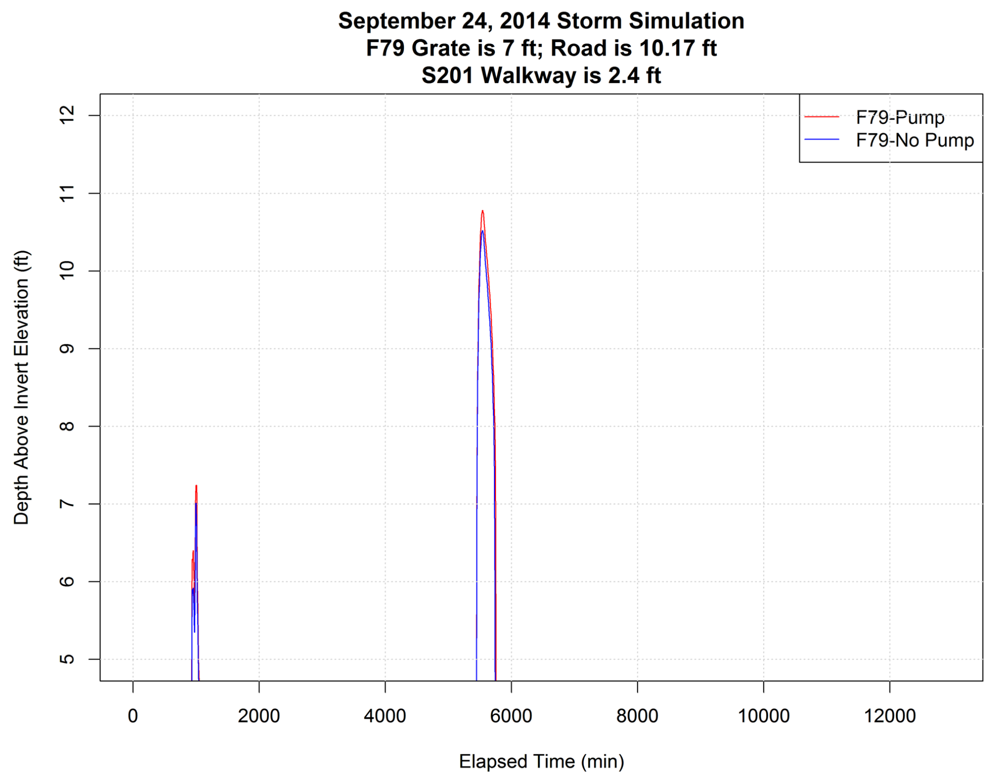


Figure 13: HS-1 Depths at F79 for 09-24-14 storm.

Figure XX is a plot of Node S201depths for the 09-24-14 storm. The effect of the Jones Stadium pumps, is negligible because this location, while physically close to the pump system is hydraulically upstream of the pumps and they exert no “control” over the depths. Plots of the depth at Node 201 for the remaining historical storms are omitted; in every historical storm examined, this node experienced inundation – usually not more than 1 to 1.5 feet of depth above the walkway. Drainage from this area is remarkably slow and the walkway remains under water for several hours – well longer than Node F79 remains under water.

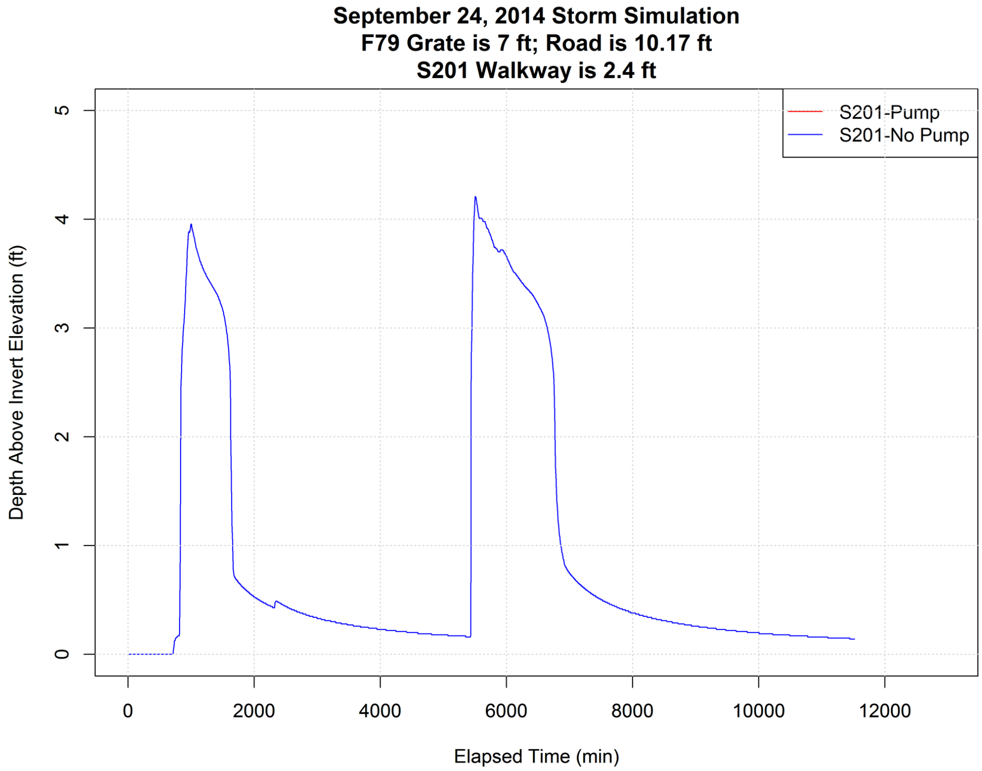


Figure 14: HS-1 Depths at S201 for 09-24-14 storm.

HS-2: 05-04-2015

Figure XX is a plot of Node F79 depths for the 05-04-2015 storm. The effect of the Jones Stadium pump system, is small; about 2-inches difference at the peak depth. The depth indicated in the figure would produce less than 2-inches of water on the travel lanes for less than an hour.

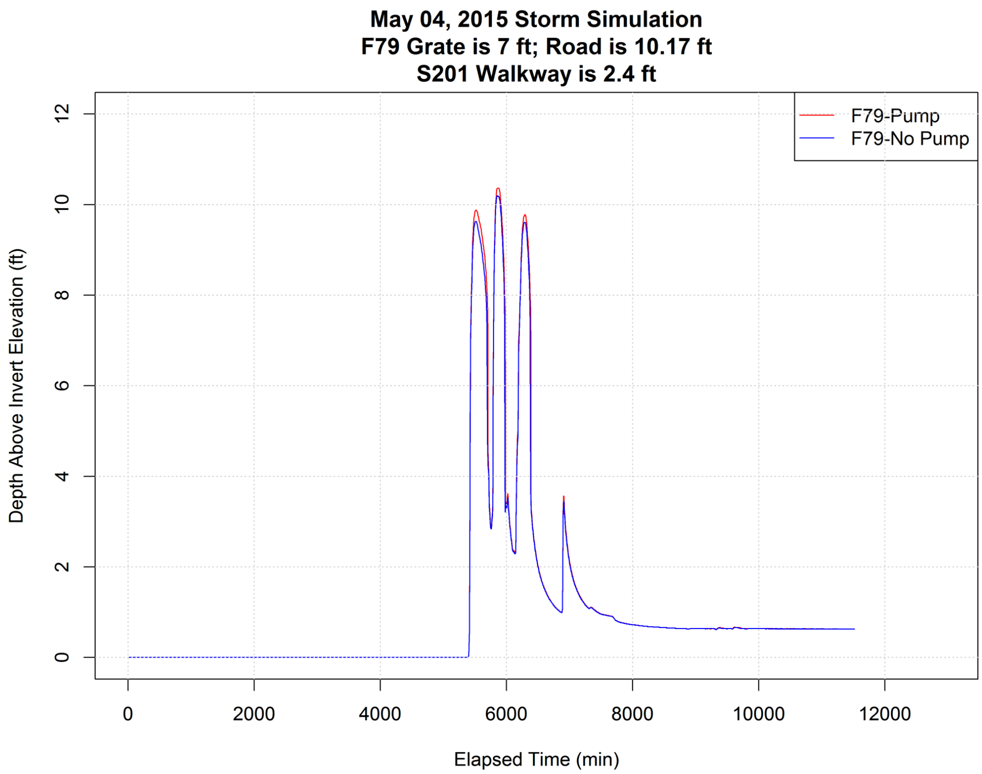


Figure 15: HS-2 Depths at F79 for 05-04-15 storm

HS-3: 10-21-2015

Figure XX is a plot of Node F79 depths for the 10-21-15 storm. The effect of the Jones Stadium pump system, is small; about 2-inches difference at the peak depth. The depth indicated in the figure would produce a maximum of 11-inches of water on the travel lanes, the duration that the lanes would have inundation is about 2 hours, but the 11 inches occurs for a small fraction of the two hour interval.

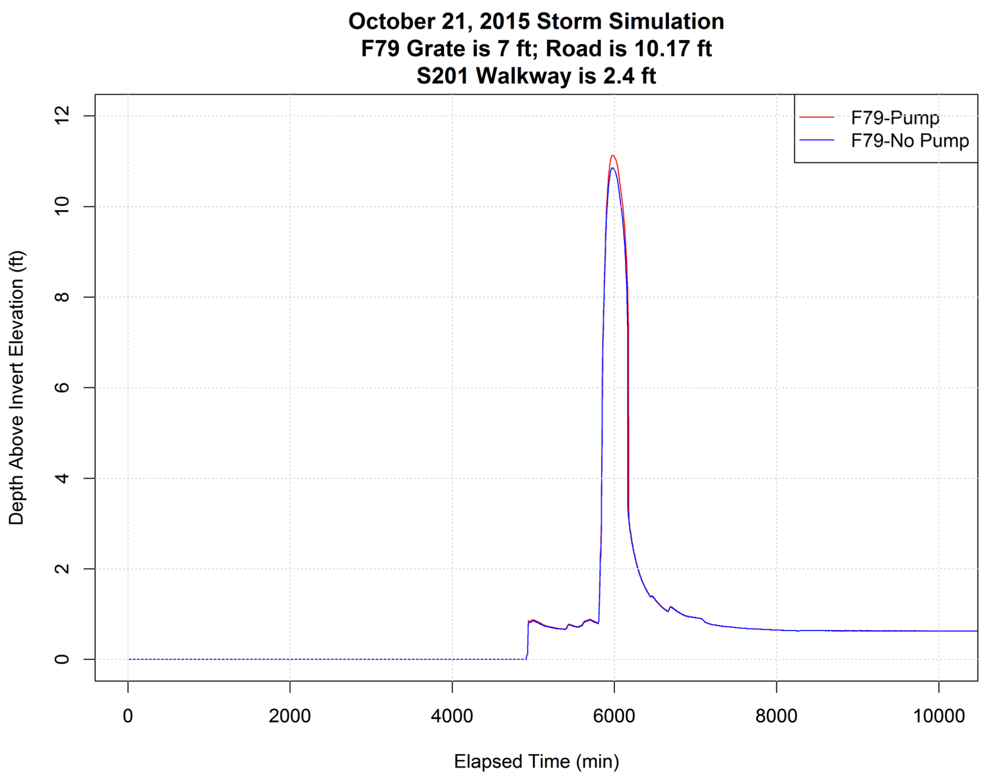


Figure 16. HS-3 Depths at F79 for 10-21-15 storm

HS-4: 07-06-2015

Figure XX is a plot of Node F79 depths for the 07-06-15 storm. The effect of the Jones Stadium pump system, is small; about 2-inches difference at the peak depth. The model estimates that there was no inundation during this storm on the freeway – in fact the model results indicate that the drainage system maintained the hydraulic grade line below the land surface for the entire storm.

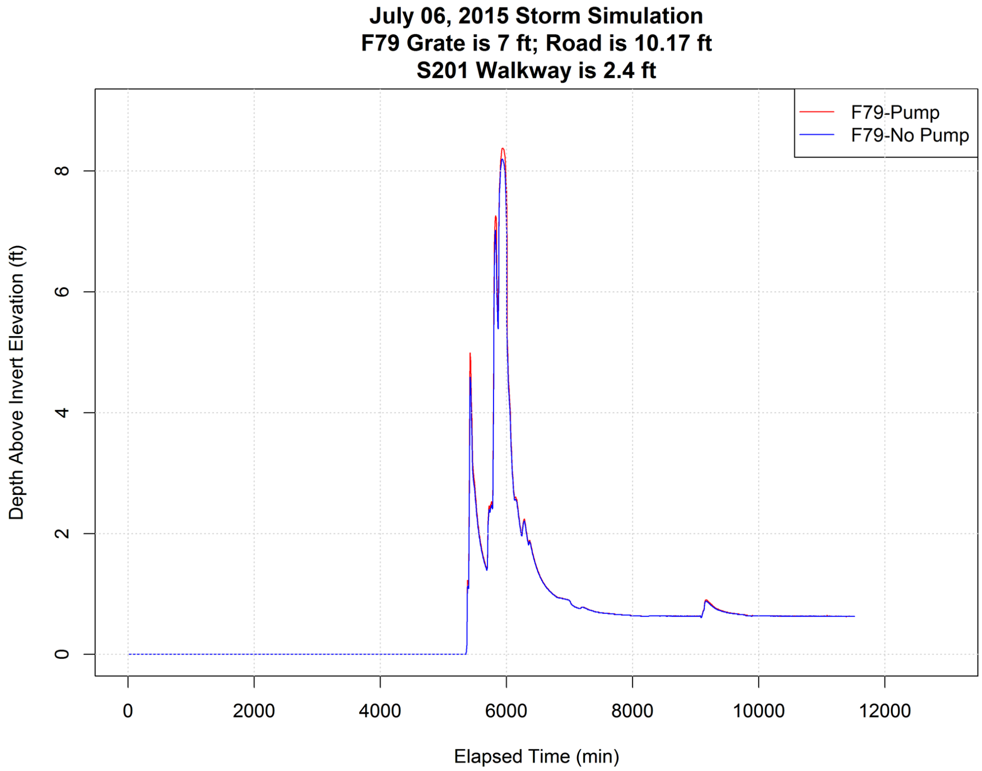


Figure 17 HS-4 Depths at F79 for 07-06-15 storm

HS-5: 05-28-2015

Figure XX is a plot of Node F79 depths for the storm on 05-28-15. The effect of the Jones Stadium pump system is about 6-inches difference at the peak depth. The model estimates that there was inundation during this storm on the freeway about 20 inches at the peak depth. The duration that the model estimates there would have been water in the travel lanes is about 3 hours.

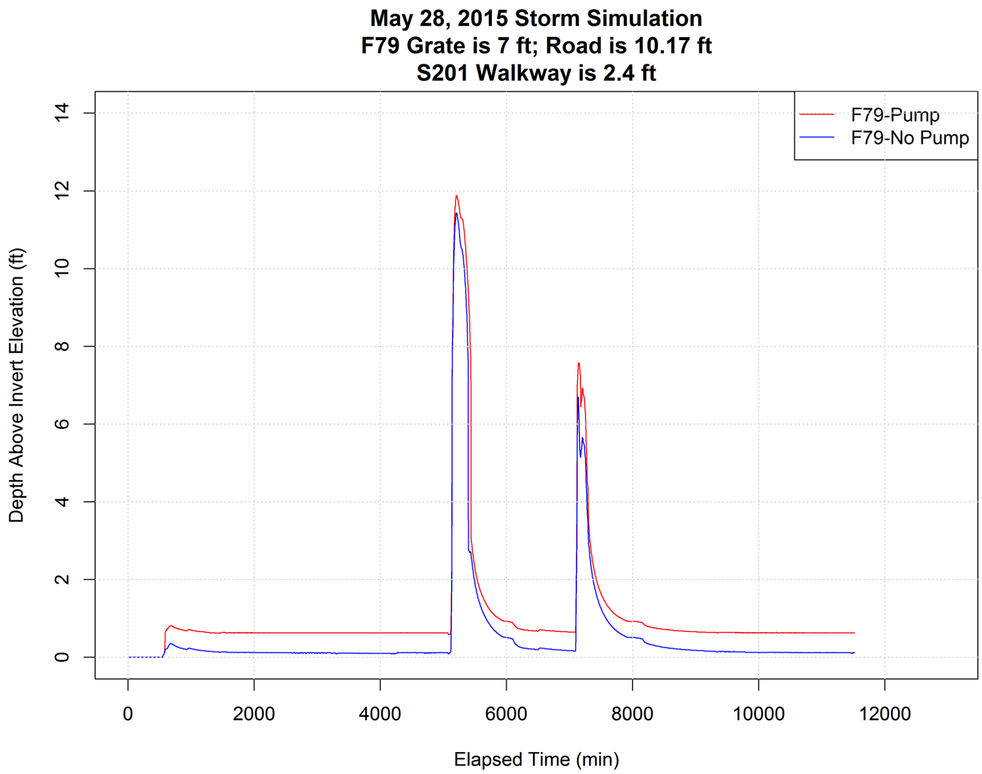


Figure 18. HS-5 Depths at F79 for 05-28-15 storm

Historical Storms Interpretation

The SWMM model estimates water in the travel lanes for some period of time for the historical storms studies, except the 07-06-15 storm. Table XX lists these storms, the estimated maximum water depth on the travel lanes near Node F79, and the estimated duration that the lanes would have any depth of water on them. The table also lists the same information for node S201.

Table 9. Table of Maximum Estimated Innudation Depth (inches), and Duration of Innudation (minutes) for Nodes F79 and S201.

STORM	F79-Depth (in)	F79-Duration (min)	S201-Depth (in)	S201-Duration (min)
09-24-14	~ 12	110	~ 21	1315
05-04-15	~ 2	55	~ 22	965
10-22-15	~ 11	120	~ 22	1260
07-06-15	N/A	N/A	~ 19	1610
05-28-15	~ 20	200	~ 22	960

In terms of depth and duration, the historical storms fall somewhere between 10-yr and 20-yr ARI. As such the storms are greater in magnitude than the supporting drainage system was designed for (5-yr), but less in magnitude than Line F was designed for (50-yr).

The storms that produce inundation in the SWMM model have relatively high intensity early in the storm, whereas the two storms that didn't produce estimated inundation (or very little) were intense, but short multiple-burst rainfalls and the drainage system was able to maintain capacity after the first burst and accommodate the remaining bursts without Node F79 inundation.

Node S201, which represents the walkway just South of Jones Stadium, estimates that in all the storms examined the walkway experienced some inundation – and the duration was lengthy, around one day.

Inlet F79 and S201 Locations

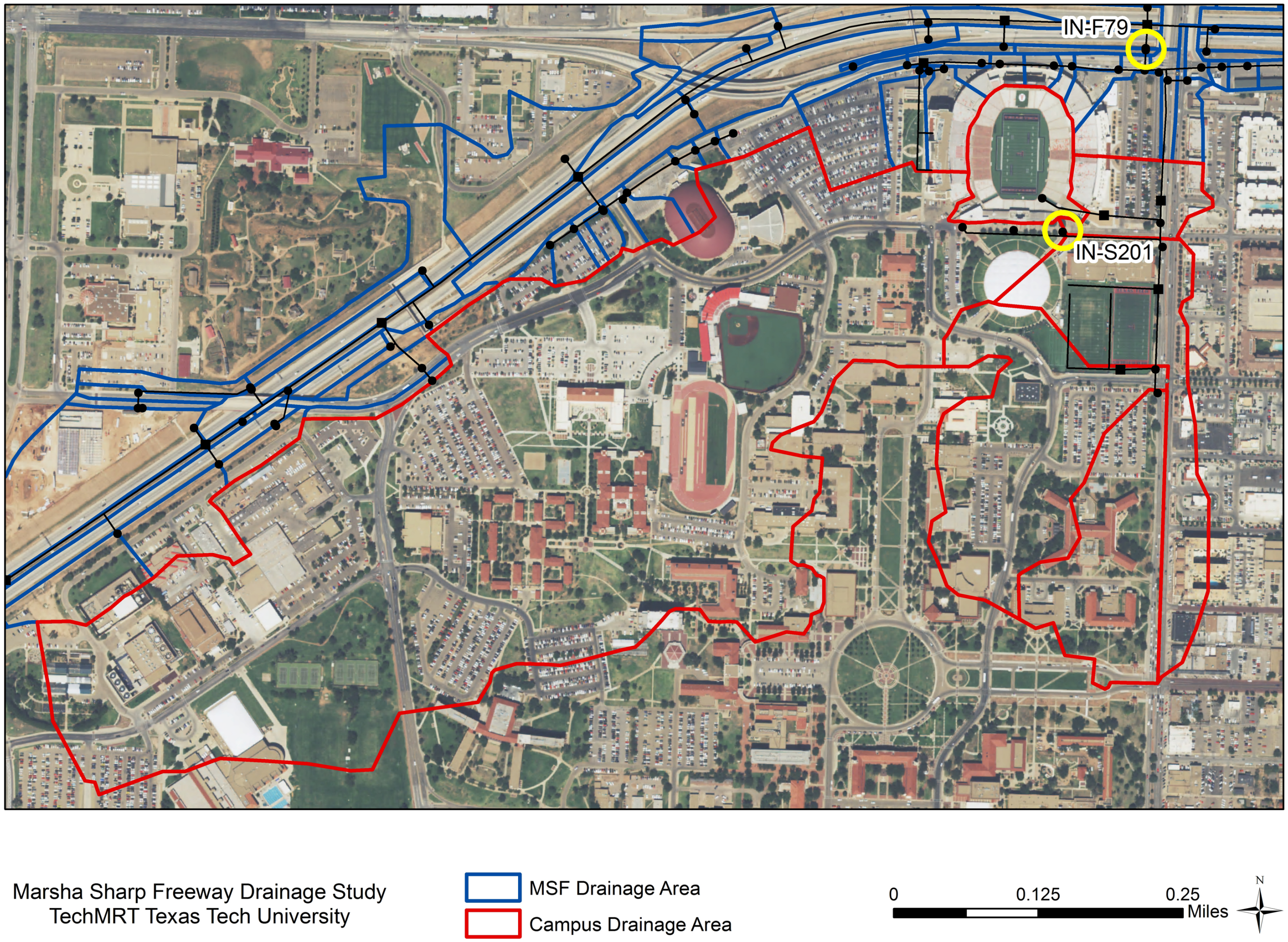


Figure 19: Frequently Referenced Inlets

Potential Alternatives

For the completion of this study, potential alternatives were explored to search for a solution that will prevent future inundation at the intersection of Marsha Sharp Freeway and University Avenue. In order to assess the alternatives, the SWMM model was adjusted to implement the potential alternative and evaluate the positive impacts each solution yielded. The alternatives were all evaluated using the September 24, 2014 historical storm event.

Alternative One: Signage

The first alternative was to install roadway warning signals on Marsha Sharp Freeway to indicate potential inundation during rainfall events. Currently there are static signage along the freeway advising potential flooding approaching the intersection of Marsha Sharp and University Avenue. Additional flashers will be installed as warning signals to ensure that drivers heed caution and exit the freeway to bypass the intersection. This solution is essentially an education approach, where there will be no alterations to the drainage system of Marsha Sharp. An enhanced alteration of this method would be to design a detection system to signal the signs when to activate during a storm.

Alternative Two: Additional Campus Inlets

Alternative two was developed by analyzing the capacity of the 8'x8' box culvert upstream of inlet-F79 located at the intersection of Marsha Sharp and University Avenue. It was found that during the historical storm simulations, the 8'x8' box culvert had approximately two feet of additional capacity during peak flows. With this available capacity, additional inlets on campus were explored to capture runoff and deliver it into the storm sewer system at an earlier position. Instead of letting all campus runoff eventually flow to University Avenue, this alternative will essentially reroute the water into the system upstream of inlet-F79 to use the additional capacity in the main box culvert.

The additional inlets were placed along streets and intersections on campus that were considered to be the natural flow paths for runoff based on the topography. In order to simulate the addition of inlets on campus, a total of three additional inlets were incorporated into the SWMM model. The naming convention used for these alternative inlets was IN-A01, IN-A02, and IN-A03. Drainage areas were approximated based on the 2011 contours to delineate runoff to the suggested locations. The estimated contributing areas for IN-A01, IN-A02 and IN-A03 were 7.06, 6.96, and 62.04 acres respectfully. This takes away approximately 76 acres of runoff draining to University Avenue from the original 138 acres from campus. Some additional grading and adjustments to the topography would need to be considered for implementation in order to maximize the campus runoff to the alternative inlets.

Inlets A01 and A02 are positioned near the baseball field parking lot on the southwest side of the Lubbock City Auditorium and at the intersection of Flint Avenue and Drive of Champions respectfully. The two inlets are connected by a 24” pipe and would ultimately connect into the existing Marsha Sharp network at the junction node of IN-H4. Inlet A03 is placed on Flint Avenue near the northeast corner of the business parking lot. A 36-inch lateral was place to connect A03 to the main storm sewer line at junction node JF29. Point elevations for each alternative inlet were taken from the 2011 LiDAR and the lengths of the pipes were estimated based on horizontal measurements in ArcGIS. The slope of the pipes was estimated based on the slope of the land. These estimations were deemed suitable approximations for modeling the alternatives in SWMM.

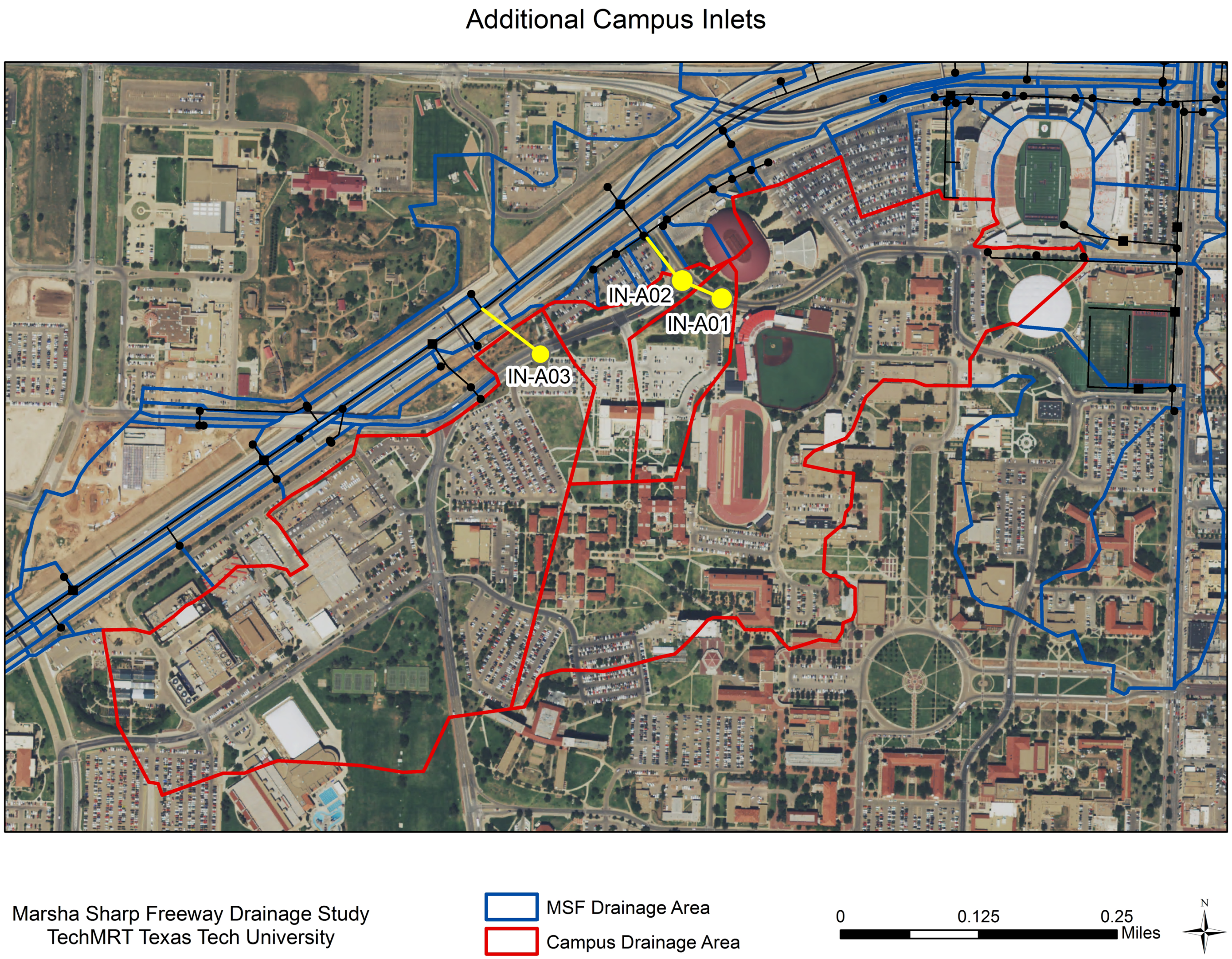


Figure 20: Map of Added Inlets for Alternative 2

Alternative Three: Permeable Pavement Storage

The third alternative's goal was to use existing parking lots on campus to generate storage and delay runoff. This would include implementing permeable pavement in the parking lot areas. The permeable pavement would store approximately 3 to 4 inches of water in a 6-inch pavement lift. The stored water would slowly be release by gravity through a designated outlet. By storing the water in the parking lots and controlling where the runoff discharges would shift the contributing area location on campus. Ultimately this would reduce the total runoff that reaches University Avenue from campus.

The parking lots considered included the business building parking lots, exercise and sport sciences parking lots, the parking lots surrounding the Jones stadium and several residential hall parking lots including, Bledsoe/Gordon, Murray, and Stangel/Murdough. Approximately 35 acres of parking lot area was modeled as additional storage in the SWMM model. Table # summarizes the distribution of areas for each parking lot.

Table 10: Campus Storage Areas

Parking Lot	Area (Ac)
Business 1	3.55
Business 2	1.87
Bledsoe - Gordon	3.44
Commuter North	9.14
East of Stadium	2.42
ESS Building 1	1.85
ESS Building 2	2.20
ESS Building 3	0.60
Murray - Carpenter/Wells	6.10
Stangel - Murdough	3.86
Total	**35.03**

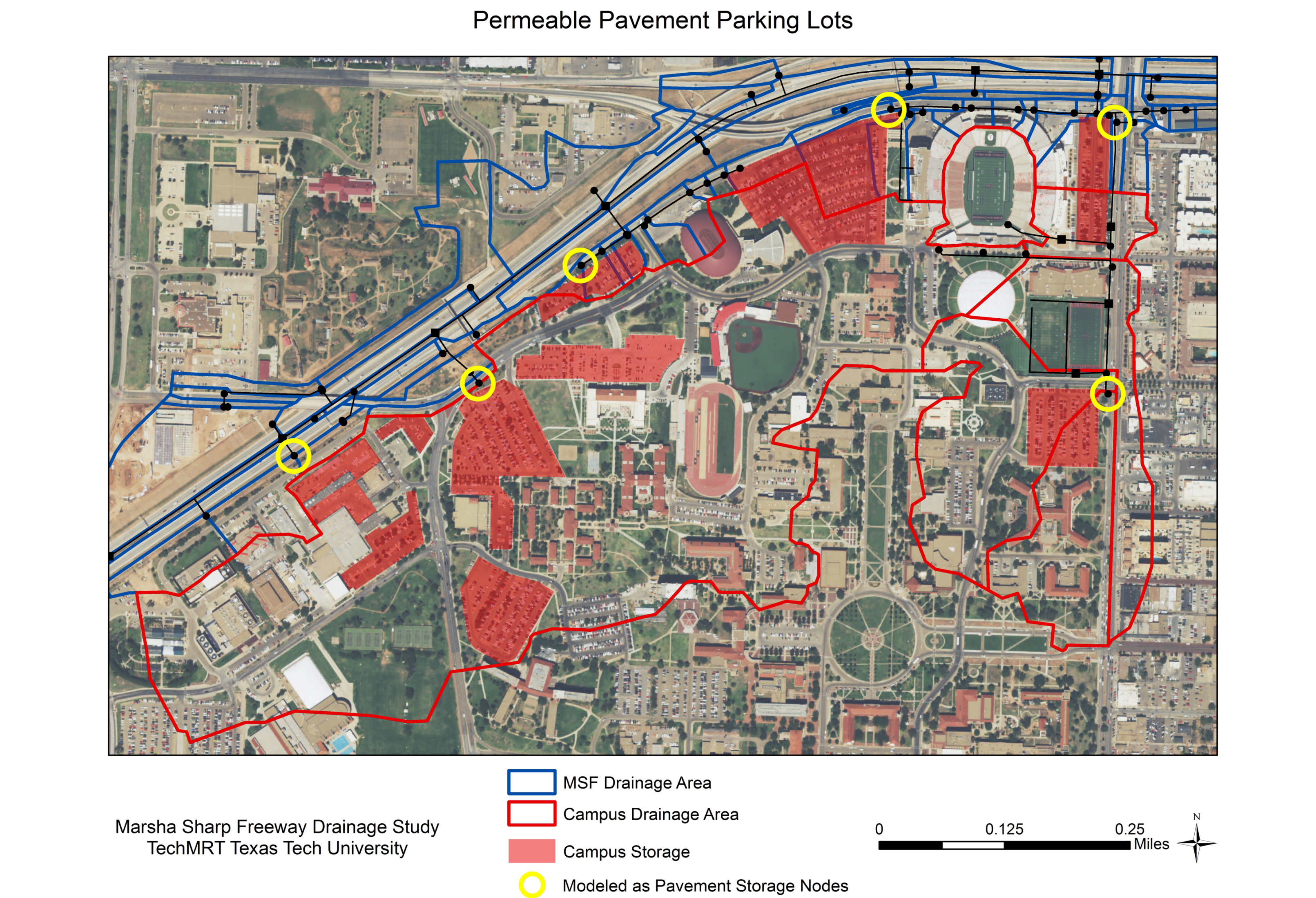


Figure 21: Map of Campus Parking Lots Modeled for Storage

Alternative Four: Delay Pump Schedule

Alternative four was developed based on the project tasks to simulate the current watershed and drainage system without the stadium pumps. Since the stadium is unable to drain without the application of the pumps, this alternative delays the pump schedule so the runoff from the stadium will be introduced to the storm sewer system at a later time than the peak flows experienced in the system during a storm event.

In order to delay the pump schedule in SWMM, a storage unit was inserted in between the pumps and the University 36-inch storm sewer line. The storage unit was modeled with a storage depth of 1000 feet and a functional coefficient of 256 feet. The functional coefficient in SWMM represents the relationship between surface area and storage depth. This storage unit was modeled to drain through a 400 foot, 36-inch pipe connected to the storm sewer line on University Avenue. This adjustment in the model sufficiently delays the arrival of the runoff coming from the stadium pumps into the Marsha Sharp storm sewer system.

Alternative Five: Send all Jones Stadium Pumpage to Golf Course

Fifth alternative was based on the possibility of rerouting the water from the stadium and inlet S203 to the Rawls Golf Course. This alternative involves approximately a 2-mile long pipeline traveling from the stadium to the golf course. For the sake of developing a model, the route of the pipeline was measured along existing streets, but the alignment did not take inconsideration elevations, slopes or obstructions such as existing retaining walls that would cause additional engineering assessment regarding the feasibility of the pipeline construction.

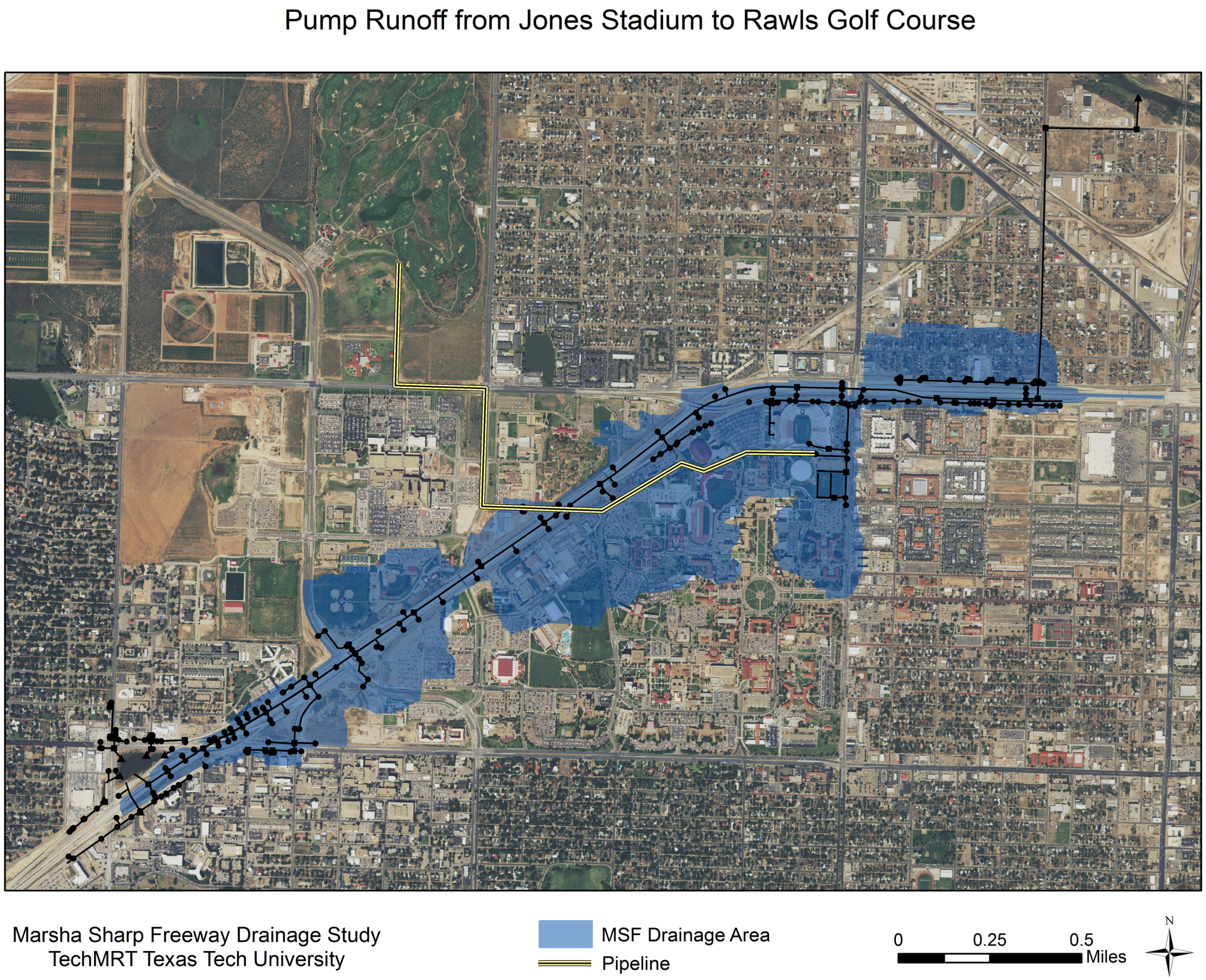


Figure 22: Map of Pipeline to Rawls Golf Course

Alternative Six: Attach to 48-inch Storm Sewer Drainage North

The last alternative explored was to reconnect the storm sewer on University Avenue to the original 48-inch storm sewer pipe that ran north along University Avenue before the construction of Marsha Sharp Freeway. This line was removed in the design of Marsha Sharp Freeway where University Avenue overpasses the freeway. A portion of Marsha Sharp drainage design on the north side of the freeway connects to this existing 48-inch line discharging north. Specifically Line D that runs along the north side of the freeway is connect to the 48-inch pipe. For this alternative a 48-inch line will reconnect junction box G1 (JBG1) to the existing 48-inch storm sewer on the north side of the intersection. The junction box is located at the southwest corner of the Marsha Sharp and University Avenue intersection.

Since the 48-inch line is existing and already in service, it was modeled in SWMM as a 24-inch pipe to account for reduced capacity. This assumption used about a quarter of the geometric area of the existing 48-inch pipe traveling north. The pipe was approximately two miles north before it outfalls in the Yellow House Lake system.

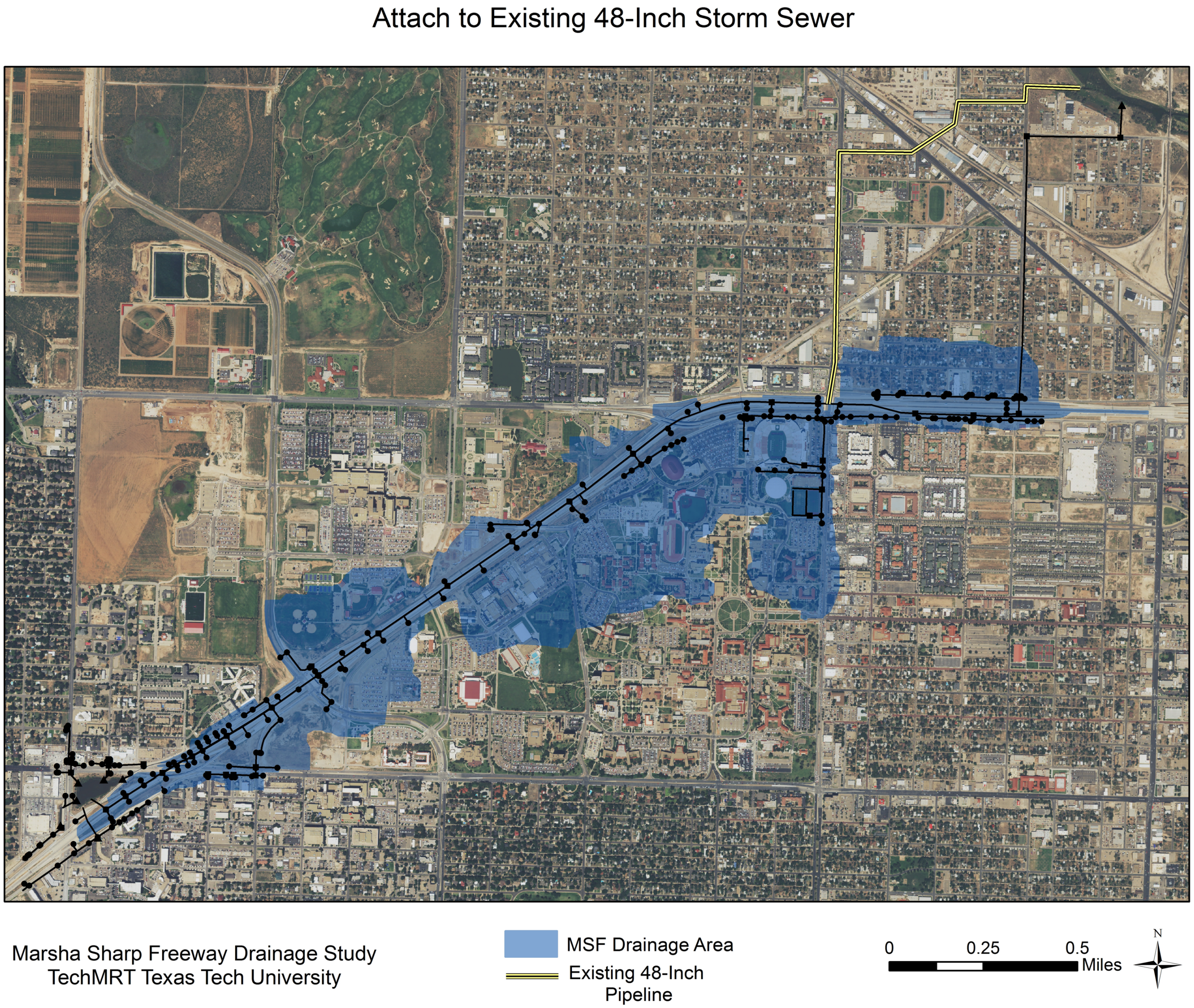


Figure 23: Map of 48-Inch Outfall

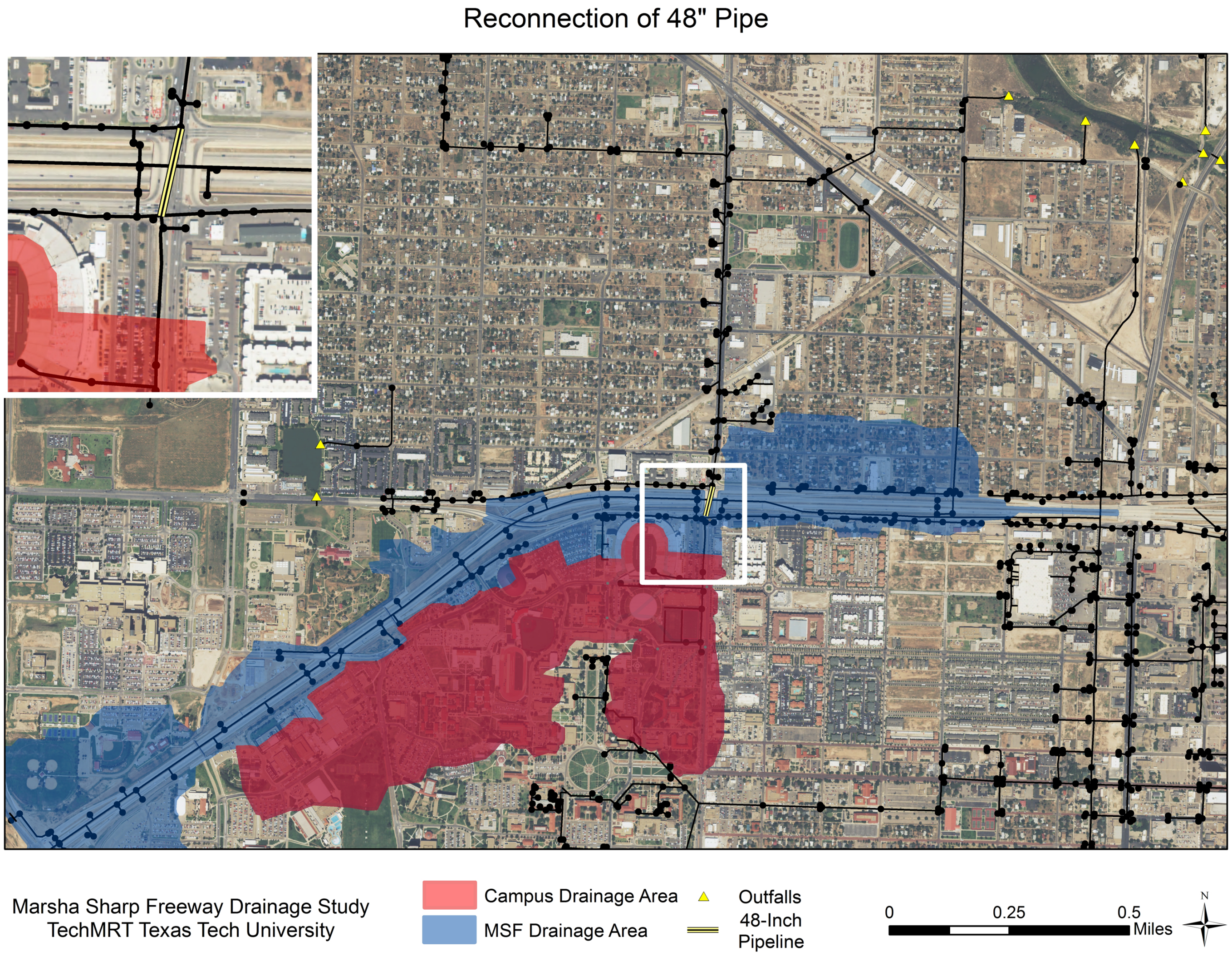


Figure 24: Map of 48-Inch at the Marsha Sharp Freeway and University Avenue Intersection

Hydraulic Performance of Potential Alternatives

Alternative 2: Connect Additional Inlets from TTU to Line H

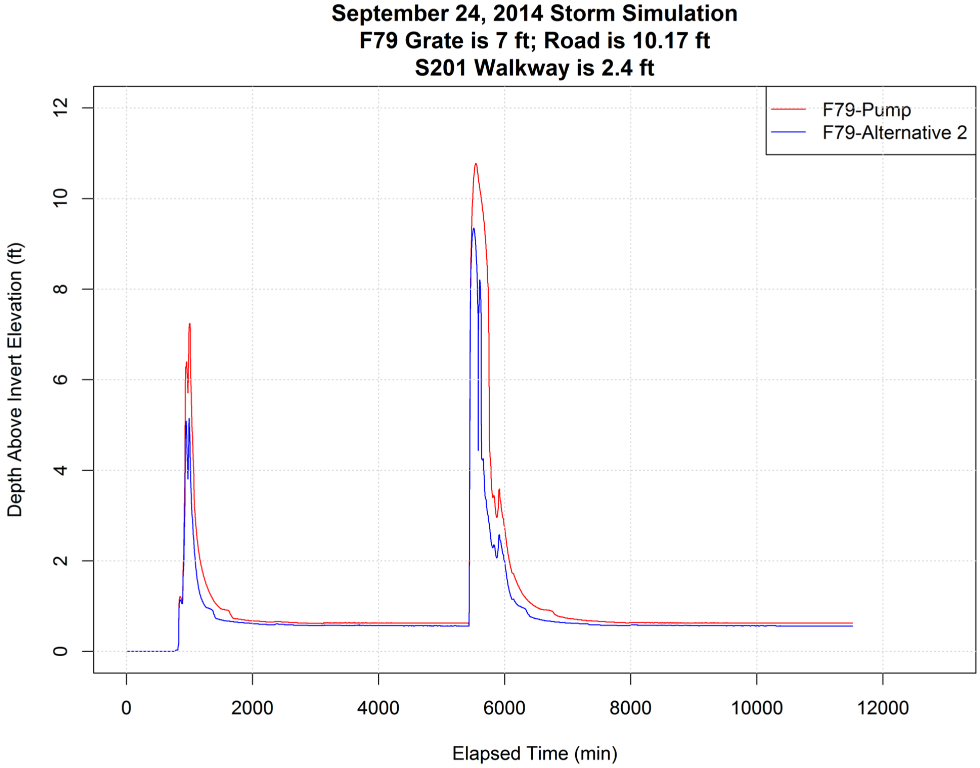


Figure 25: Node F79 Depths for Alternative 2 compared to base conditions.

Alternative 3: Incorporate Permeable Pavement Storage on TTU Parking Lots

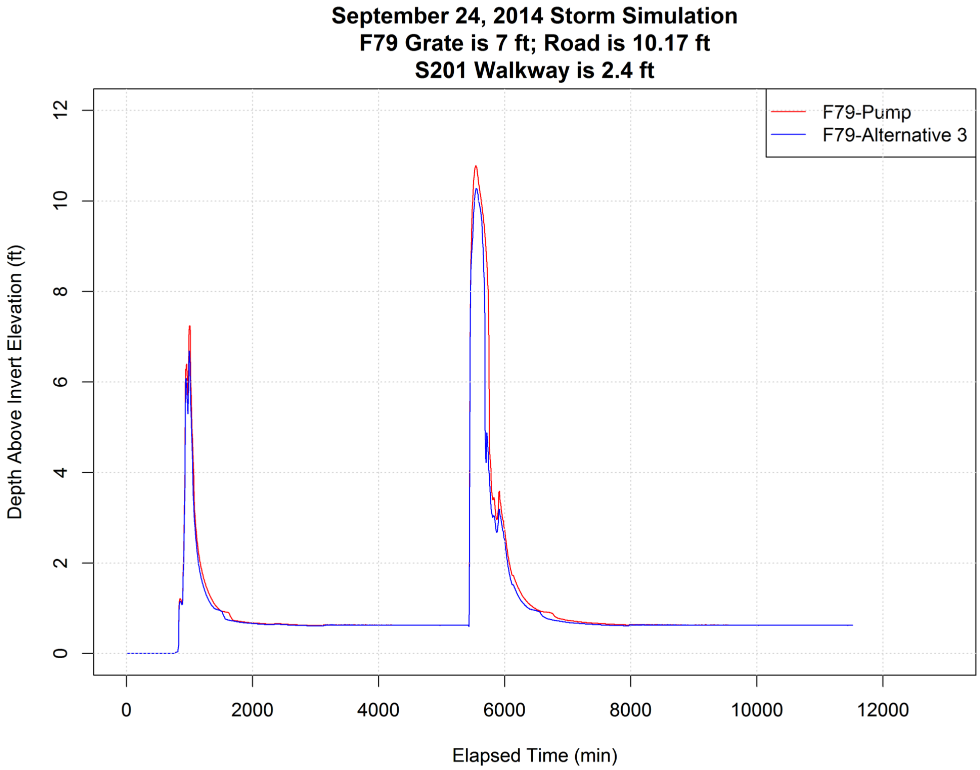


Figure 26. Node F79 Depths for Alternative 3 compared to Base Case

Alternative 4: Delay Pumping (use Stadium as a Storage Tank)

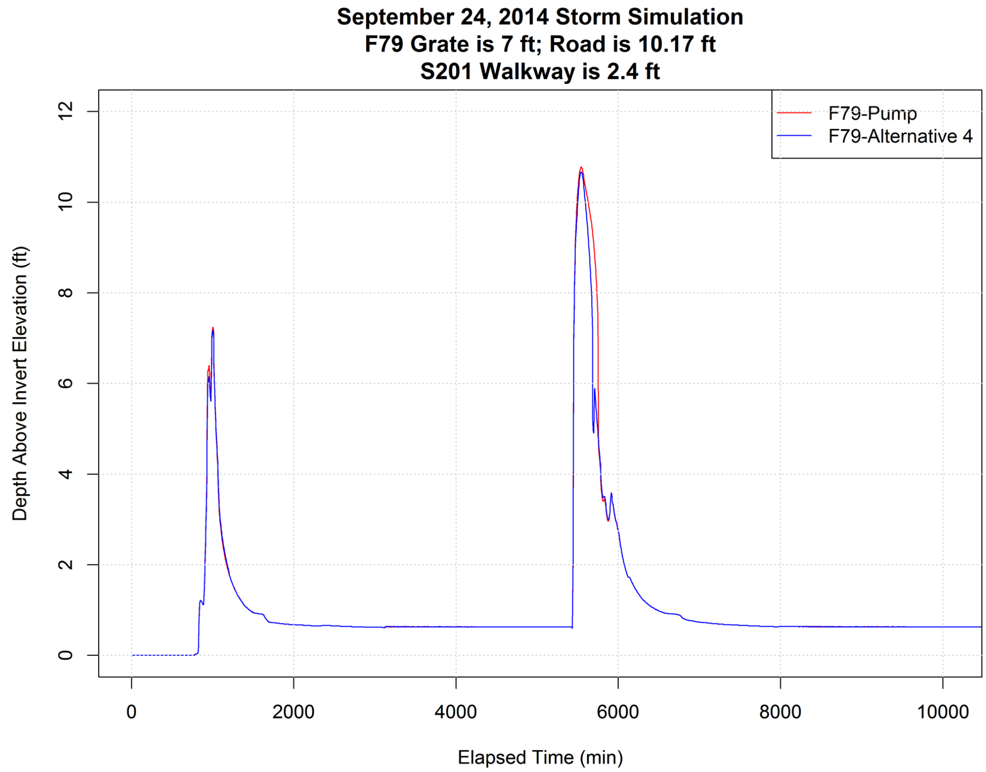


Figure 27. Node F79 Depths for Alternative 4 compared to Base Case

Alternative 5: Divert ~1/3 TTU Campus Drainage to Golf Course

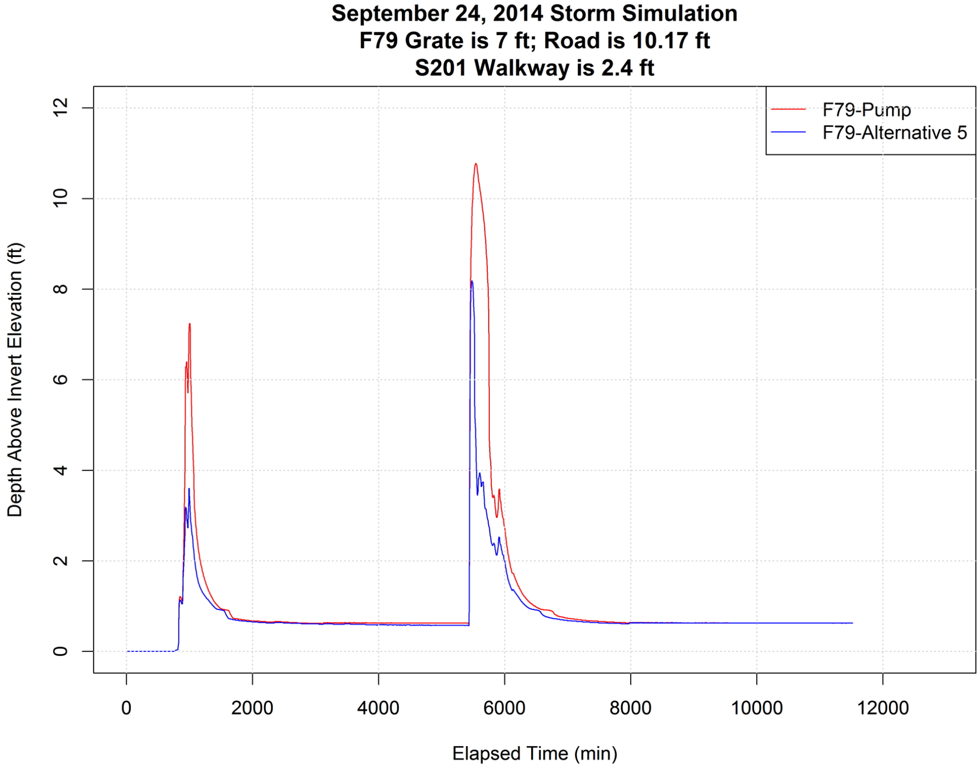


Figure 28. Node F79 Depths for Alternative 5 compared to Base Case.

Alternative 6: Reconnect 48-inch Sewer North of Freeway

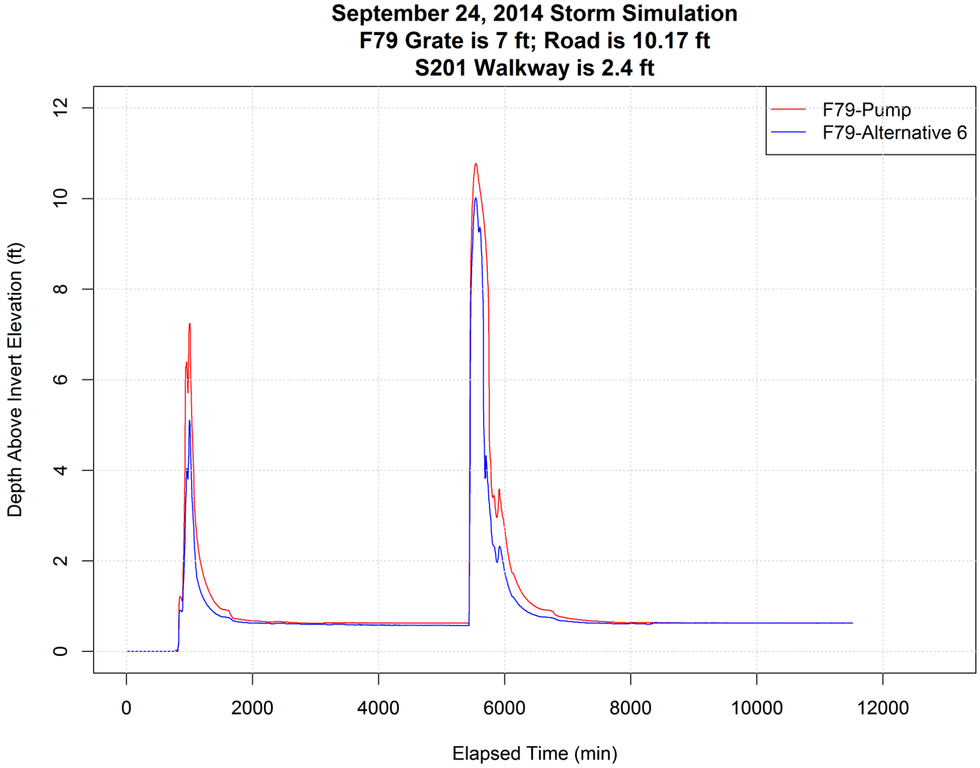


Figure 29. Node F79 Depths for Alternative 6 compared to Base Case

Table XX is a summary of the results comparing the alternatives. The table lists the computed maximum depths above the travel path for Node F79 and above the walkway for Node S201. Alternative A1 depths represent the Base Case conditions so the other reported conditions can be compared to these values.

Alternatives A2, A5, and A6 all reduce inundation at Node F79 to zero (no inundation), Alternatives A2 and A5 are the only alternatives that have meaningful impact on Node S201 maximum inundation depth, although alternatives A2,A3, and A5 all reduce the time that the walkway is inundated.

Table 11. Summary of Hydraulic Performance of Examined Alternatives

ALTERNATIVE	F79-Depth (in)	S201-Depth (in)	Remarks
A1 - Signage	~ 12	~ 21	Automatic depth and flow measurements would be beneficial (No Hydraulic Changes)

A2 - Inlets from TTU to Line H	~ 0	~ 19	Depth and duration of inundation at S201 reduced (7 hours less)
A3 - Storage on TTU Parking Lots	~ 1.3	~ 20	Depth and duration of inundation at S201 reduced (3 hours less)
A4 - Delay Pumping -- use Stadium as Storage	~ 6	~ 20	Negligible benefit at S201
A5 - Divert portion of Campus Drainage to Golf Course	~ 0	~ 15	Depth and duration of inundation at S201 reduced (3½ hours less)
A6 - Reconnect to 48-inch Sewer North of Freeway	~ 0	~ 21	Negligible benefit at S201

Summary and Conclusions

words

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Appendix

May be multiple appendices, for larger exhibits.