TEXAS TECH UNIVERSITY

Master's Report

USGS Flow Gage Station Near Hereford, TX

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Table of Contents

Introduction

The Texas Department of Public Transportation has funded a gage station through USGS to monitor peak flows under a bridge crossing on U.S. 60 near Hereford, TX since 1994. At the bridge crossing the Tierra Blanca Creek has approximately 194 acres of drainage area as stated by Dr. Asquith from the USGS (William H. Asquith, USGS, written column, 2012). The purpose of the gages are to insure that the storm flows seen in Tierra Blanca Creek do not exceed the design flows used for the U.S. 60 bridge crossing and put the structure at risk. To monitor the flow, the USGS installed two crest-stage gages downstream of the bridge crossings in order to perform two-section slope-area flow measurements. Crest-stage gages are used because of their low maintenance, low cost, and simple installation.

Figure 1: Tierra Blanca Creek and U.S. 60 Crossing

In 2007 the two crest-stage gages were moved for easier access to a section of the Tierra Blanca Creek on the John Pittman Municipal Golf Course about 1.5 miles downstream of the U.S. 60 crossing. The golf course allows for easy access along with a maintained channel free of debris. Since the area is maintained and under supervision, there is a reduced risk of vandalism. This 2007-present location is the area of interest for the following report. A Google Earth image of the site area is provided on the following page with the two gages highlighted in the red circles.

Figure 2: Crest-Stage Gages on John Pittman Municipal Golf Course near Hereford, TX

The empty storm flow channel can be seen in Figure 2 which moves from West to East along the centerline of the image. The two gages are station in the thalweg of the channel in order to capture greatest depth of water from the invert or lowest part of the channel bottom to the water surface. The two gages are located approximately 250 feet apart from each other to insure a sufficient change in water surface elevation. The USGS has taken cross-sectional data for each gage along with reference elevations in order to perform their two-section slope-area flow measurements. The USGS has taken four peak water surface elevation measurements from 2007-2011 at the current site. In 2010 the golf course made a change to the site, which Dr. Asquith from the USGS believed created a scenario for study.

During the year of 2010 the John Pittman Municipal Golf Course constructed three golf cart path bridges spanning the storm channel of interest near the USGS gages. One bridge was constructed just upstream of the second gage (upstream gage), a second bridge was constructed just downstream of the first USGS gage (downstream gage), and a third bridge was constructed about 350 feet downstream of the second USGS gage (See Figure 3). Dr. Asquith of the USGS was concerned that these new bridges could manipulate the flow in a way that would directly affect the peak water surface measurements at the gages. If these bridges caused the water surface elevation to increase, compared to prior conditions, it would cause the two-section slope-area method to compute flow higher than actual conditions. This in turn could cause unneeded concern by TXDOT about the safety of their upstream structure. Therefore, Dr. Asquith believed the area needed to be studied in order to determine if there was any effect on the water surface elevations read at the gages by the small bridge crossings.

Figure 3: Crest-Stage Gages with Golf Cart Bridge Crossings

As seen in Figure 3, two of the bridge crossings labeled B and C are extremely close to the USGS gages. Bridge C is approximately 37 feet upstream of the adjacent gage, while bridge B is approximately 26 feet downstream of its adjacent gage. The main concern is the bridge B would dam up the water under high flow conditions which would raise the water surface mark at USGS gage 1 and cause an unrealistic flat water surface slope from gage 2 to gage 1. Images of the USGS gages are provided below which show their close proximity to the bridge crossings.

Figure 5: USGS Gages Looking Upstream

Figure 4: USGS Gage Looking Downstream

Each of the three bridge crossings are solid reinforced concrete structures with a single 48in diameter corrugated metal culvert. The culverts are kept relatively clean and free of debris, but scour holes are present both upstream and downstream of each culvert. Each bridge is around 10 feet in width and extremely flat. Images of the bridge and culvert systems are shown below.

Figure 6: Bridge Crossing

Figure 7: 48in Diameter Corrugated Metal Culvert

Figure 8: Bridge Crossing C and Upstream USGS Gage

Figure 9: Downstream of Bridge Crossing C

From the images above it is obvious the bridges crossings change the geometry of the channel that the water can flow through. Their close location to the gages is an obvious concern, so it was decided that the area would be studied using a 1-D modeling program. The modeling program that was chosen with the approval of Dr. Asquith was EPA SWIMM due to its ability to accurately compute overland and closed conduit flow. In order to decipher if the bridges had a significant impact on the gages, a SWIMM modeled needed to be created with the current geometry of the channel (bridges and culverts in place) and another without the bridge crossings. An elevation survey had to be done on the area to create the geometry data for a model. The only current elevation data available was taken by USGS for the two cross-sections pertaining to the two crest-gages. This was not useful since the bridge crossings along with surrounding area needed to be included to build an accurate and realistic model. The following sections will explain the methods used along with process of constructing the EPA SWIMM model.

Hi-Resolution Survey

A hi-resolution survey was needed to create the geometric data for the EAP SWIMM. A total station, data collector, and a surveying rod & prism were provided by the Texas Tech Water Resources Center to perform the survey. A total station was chosen based on its ability to take a large amount of data points in a, efficient amount of time while using a data collector. Dr. Asquith, Dr. Cleveland, Janice Rainwater, and Travis Kaatz drove to Hereford, TX and performed the survey on October 3, 2012. Dr. Asquith was able to the reference markers used by the USGS on the site. Having a USGS reference marker made it possible to tie the new survey to the existing USGS data, so that computed water surface elevation by SWIMM could match the USGS recorded water surface elevations.

To capture the geometric data needed cross-sections were taken perpendicular to the channels flow direction. The main cross-sections taken were at the areas of interest which included the locations of the USGS gages and the bridge crossings. Intermediate cross-sections were taken in between the areas of interest along with a cross-section upstream of bridge A to insure accuracy. Special attention was paid to the bridge crossings since the bridges caused a drastic change in channel geometry over a short channel length. One cross-section was taken along the downstream edge of each bridge crossing in order to capture the culvert exit invert. Another cross-section was taken only inches upstream of the first which captured the downstream edge of the bridge top just above the culvert exit invert. A third cross-section was along the upstream edge of the bridge crossings. The fourth was taken only inches further upstream which captured the culvert inlet invert. A figure depicting the layout of the each bridge crossings corresponding cross-sections were taken is shown below in Figure 10.

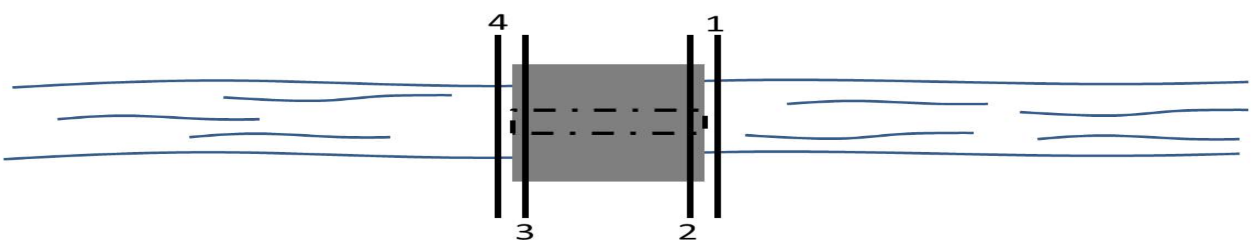

Figure 10: Bridge Cross-Sections

In total 19 cross-sections were taken over a 550-ft channel length which added up to 285 data points. The distance between cross-sections did not exceed 130-ft. Each cross-section was taken wide enough to include a maximum flow depth greater than 5 feet which around the maximum flow depth seen in the channel by the USGS. This caused the cross-section be several hundred feet wide because the flood plains were extremely flat as seen in Figure 11.

Figure 11: Floodplains

Once all the data points were collected they were imported into Surfer which is a 3D surface creation program. Surfer was able to contour the site from the data provided which resulted in Figure 12. Importing the data into Surfer was an easy was to check that our data look realistic compared to the field. If there was a major mistake such as a bad surface elevation point there would be a noticeable discrepancy in the contours. The data points taken are represented by the "X" in Figure 12. Figure 21 in the Appendix shows the locations of the bridge crossings highlighted in the red ellipses on the surfer image. You can see the large cluster of data points at this location from the four close proximity cross-sections taken at each bridge crossing. Figure 22 in the Appendix shows the locations of the two USGS gages on the Surfer image. Figure 23 also in the Appendix shows lines depicting the cross-sections taken in the field. The data points in these cross-sections are in X, Y, & Z which poses a problems because SWIMM handles geometric data in station and elevations. The next step was to convert these three dimensional coordinates into station and elevations.

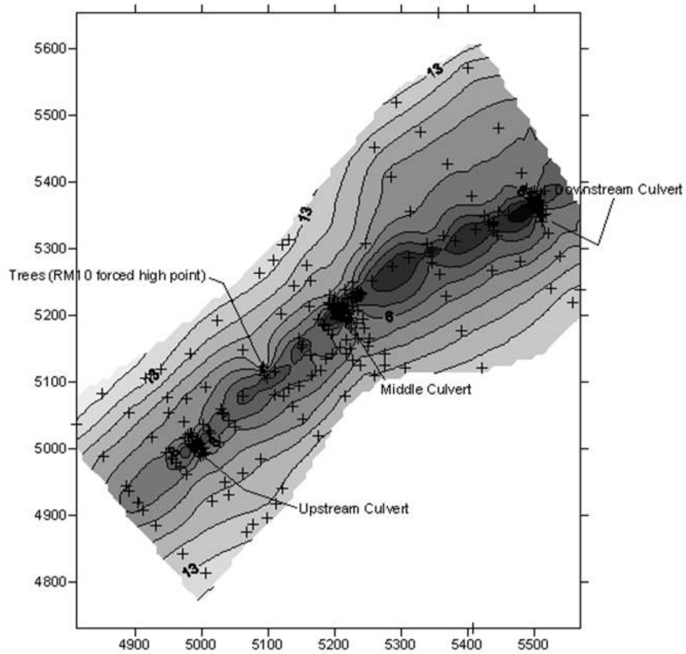

Figure 12: Surfer Contouring (0.5-ft Intervals)

In order to turn the 3D coordinates into station and elevations and reference had to be made for the stations. To accomplish this, a reference line was drawn on the left bank of the channel parallel to the flow of the channel. This figure depicting this line can be seen in Figure 24 located in the Appendix. This reference line represents station 0 where it intersects each cross-section. A data point was digitized for each cross-section where it intersected the reference line. From this point a simple distance equation (Equation 1) was run to each data point on the cross-section to get its corresponding station.

$$Station = \sqrt{(X_2 - X_1)^2 + (Y_2 - Y_1)^2} \tag{1}$$

The Z coordinate for each data point was kept as the elevation. Since the area is so flat the error in station that is possible, because a data points are in a perfectly straight line perpendicular to flow, was neglected. Figure 25 and Table 3 in the Appendix help represent this process. The cross-sections were numbered, as seen in Figure 23, increasing as they moved upstream. The most downstream cross-section was assigned XS-0 and the most upstream cross-section was given XS-18. In EPA SWIMM's link-node system the distance in between cross-sections must be known to compute the length each cross-section spans. This length was computed using Equation 1 between the two data points located on the thalweg of adjacent cross-sections. This provided an accurate distance in between the critical part of the cross-section, its invert.

An issue that arose that called for special attention were cross-sections 2 & 3 located just upstream of the most downstream bridge (Bridge A). The data points for these cross-sections did not follow a straight line perpendicular to flow, but instead meandered around it. The distances from the perpendicular line drawn in Figure 23 called for action. To solve this issue and reduce error, data points were digitized from surfer. Data points were digitized along the cross-section line drawn in Figure 23 at each 0.5-ft contour interval. This provided stations and elevations that were along a line perpendicular to the flow path. Figure 26 and Table 4 in the Appendix visual represent this process. Once these cross-sections were complete the geometric data was complete and ready to be imported into SWIMM for modeling.

Hydraulic Model

Current Conditions – With Bridge Crossings

EPA SWIMM is a relatively simple hydraulic modeling program to use. SWIMM uses a link-node system to define the geometry of the system. The first step in creating the model was to lay out the link node system that would represent the channel. A total of 17 links were laid out to represent the cross-sections created. Only 17 were used because only one cross-section was used to represent the two upstream bridges, since the width of the bridges are only 10 feet and they are very flat. Three more links were added to represent the three culverts through each of the bridge crossings. The USGS gages are located at XS-8 and XS-11 in the system. The bridge crossings were treated as overflows or bypasses, while the culverts were used as the main link of flow. This means that SWIMM would route the flow through the culverts until some boundary condition was met and then route the flow through both the culver and bridge cross-section. This boundary condition was set to be an offset from the culvert inlet to the elevation of the bridge. Therefore, the water surface had to rise above this offset height before SWIMM let it spill over into the bridge cross-section. A figure displaying the link-node system described about in SWIMM is provided below.

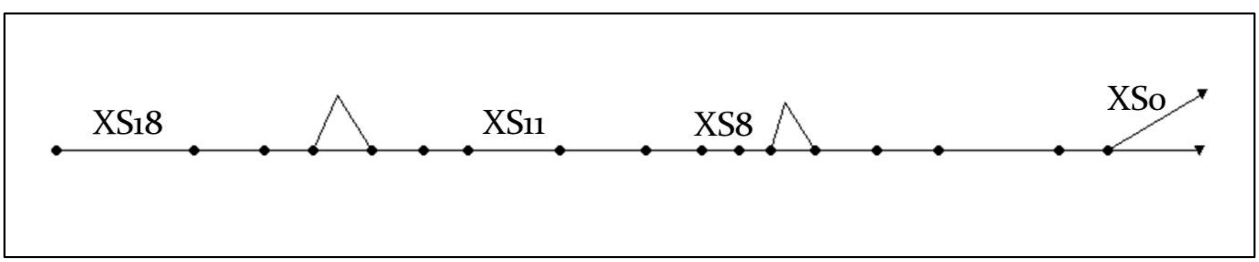

Figure 13: SWIMM Link-Node System

The two triangles on the far right of Figure 13 represent the outfalls of the system. When the flow reaches this point of the system it spills over as a free outfall. Since these outfalls are approximately 350 feet downstream of the USGS gages this boundary condition is appropriate. Two separate outfalls were used instead of a single outfall because SWIMM had a hard time converging on a solution with two outfalls. As stated before the bridge crossings were defined by using an offset from the culvert entrance invert to the top of the bridge. The downstream part of the bridge was also defined as such. The outlet offset of the bridge cross-section were defined from the top of the bridge to the culvert exit invert. This defined the height the water had to spill over the bridge to the downstream cross-section.

The cross-sectional data was entered into SWIMM as an irregular conduit. Since SWIMM is primarily a closed conduit hydraulics model most of the preset geometric data is pipes and culverts. An irregular condition is defined as a set of stations and elevations which are joined by the order they are entered. A figure displaying one of the cross-sections entered into SWIMM as an irregular condition is shown in the figure below.

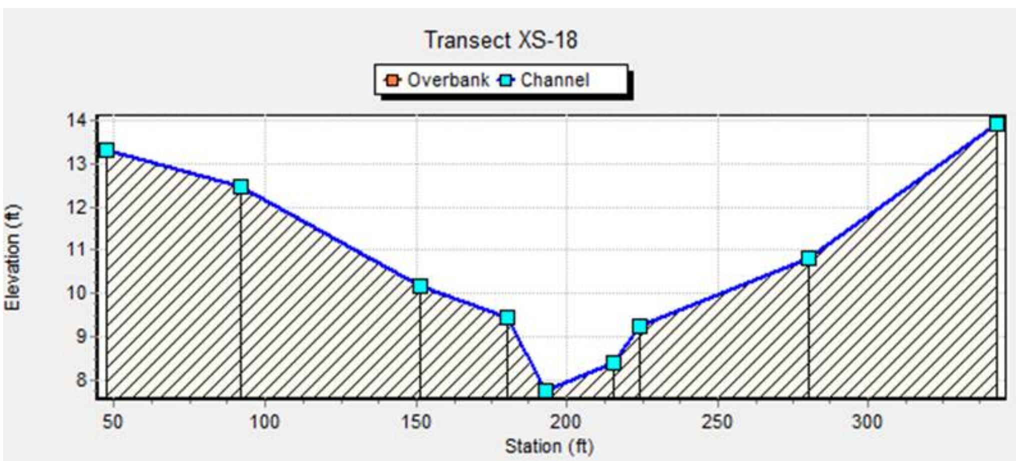

Figure 14: SWIMM Irregular Conduit

After the stations and elevations were inputted for each cross-section the roughness coefficients had to be defined. Manning's roughness was chosen due to its wide approval and easy definition. Table 4-7 in the TXDOT Hydraulic Design Manual states that a clean, straight, no rift channel has a manning's roughness of 0.3. This roughness was chosen because the description best fit the channel characteristics. A table representing all the data entered for a sample cross-section is provided in Table 5 in the Appendix.

The links representing the culverts in the SWIMM model were defined somewhat differently. The geometry of the culverts was set as circular closed conduits. The max depth of the conduit was defined as the 48 in diameter of the corrugated metal pipe. The entrance inverts and outlet inverts were set to be the inverts measured at the cross-sections just upstream of the culvert and just downstream of the culvert. The conduit lengths corresponded to the width of the bridges they spanned since they did not protrude from the headwall. The last parameters defined were the manning's roughness of the conduit and the entrance loss coefficient. Table 4-9 in the TxDOT Hydraulic Design Manual states that the manning's roughness for a corrugated metal pipe is 0.2. Table 8-5 in the manual defined the entrance loss for a corrugated metal pipe with square edge headwalls to be 0.5. The exit losses of the culvert were ignored since they are normally very minimal. This completed the geometric data for the current conditions with the bridge and culverts installed. A profile view of the SWIMM model with the bridge crossing is shown below.

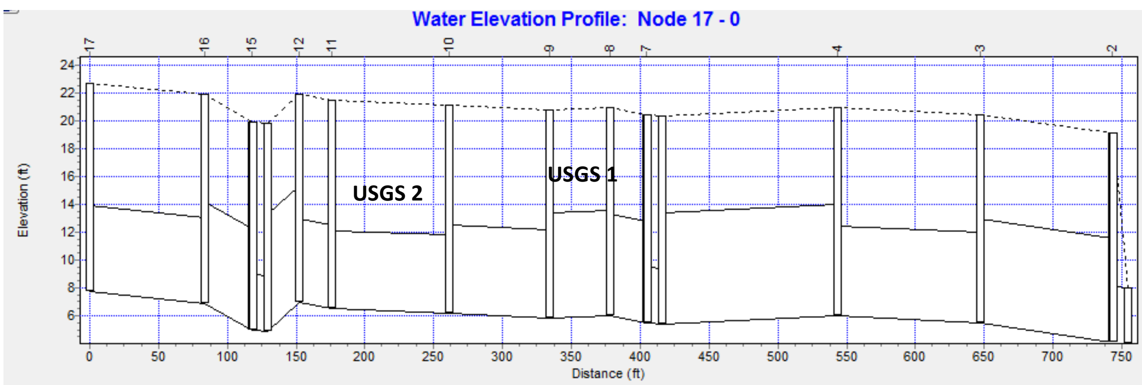

Figure 15: Profile View of SWIMM Model with Bridge Crossings

Pre-Conditions – Without Bridge Crossings

To view the effects the bridge crossings have on the gages a model without the bridge crossings had to be created. The roughness data along with cross-sectional data was left alone. To represent conditions before the construction of the bridges the culverts and bridge cross-sections were removed. Since the cross-sections just upstream and downstream of the bridge sections had large scour holes they were also removed. The cross-sections upstream of the scour holes for each bridge crossing were simply extended to the next cross-section downstream of the bridge. Since the area is relatively flat with a slope less than 1% it was decided this was an appropriate representation of previous conditions. A figure highlighting the removed cross-sections is shown in Figure 16, where all the sections inside the red ellipse were removed. Figure 17 shows the resulting SWIMM model with the removed cross-sections. Figure 18 is a profile view of the SWIMM model without the culverts. As you can observe the bottom of the channel is much smoother due to the lack of scour holes caused by the culverts.

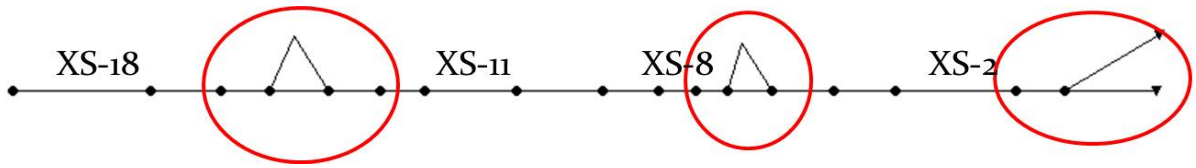

Figure 16: Sections Removed from Model for Pre-Conditions

Figure 17: Pre-Condition SWIMM Model

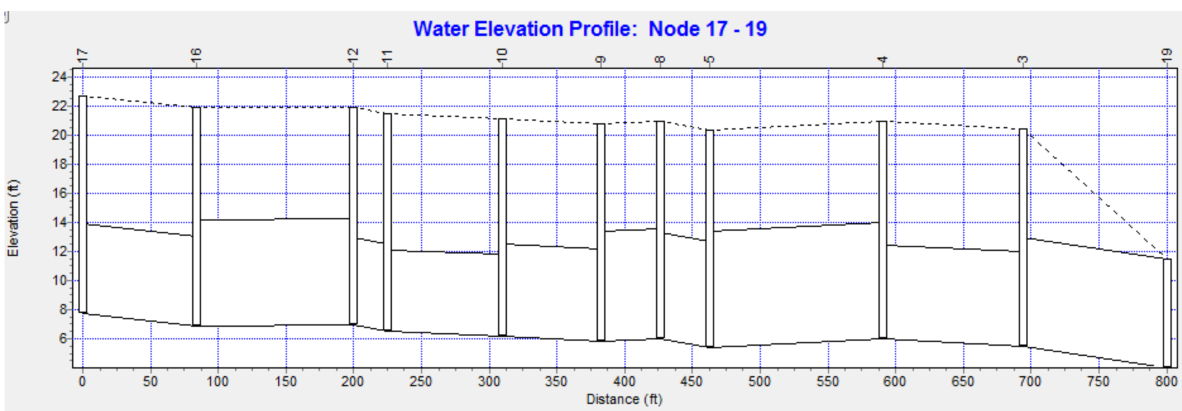

Figure 18: Profile View of SWIMM Model without Culverts

Model Calibration

Ideally the SWIMM model would be calibrated to the recorded data from the USGS. This would involve entering the calculated flowrates from the USGS and calibrating the roughness values so that the SWIMM water surface elevations matched the water surface elevations measured in the field. This issue that arose was that there were only 5 measured data points since the gages had been moved their current location. Only two measurements were made after the construction of the bridges and one was incomplete. Attempts were made to calibrate the pre-bridge model to the recorded data but it resulted in unrealistically high roughness values. The question under study is if the bridges affect the water surface elevations at the gages. This can be done even with an uncalibrated model because the bridges effect of the bridges will not be greatly affected by the roughness values.

Simulation

The model was run for a duration of 18 hours of simulation time to insure that SWIMM converged on a solution and a steady-state was reached. An increase in this time had no effect on the model results. The flow in the model was defined as an input flow into the most upstream node. This node was approximately 100 feet upstream of Bridge A which allowed the boundary condition to have little effect on the gage which was approximately 150 feet downstream. Four trials were run for the two models to see if the variation affected the difference in results between the two models. The highest flowrate run was 500 cfs which was slightly above the highest flowrate calculated by the USGS. The lowest flowrate run was 200 cfs because for flowrate lower than this SWIMM could not converge on a solution. The two intermediate flowrates run were 300 cfs and 400 cfs. More intermediate flows were not run because the channel floodplain is so flat a small difference in flow does not drastically change the water surface elevations.

Results and Interpretation

Results

For each of the simulated flowrates EPA SWIMM was able to provide the computed water surface. SWIMM was able to converge on a solution for each flowrate without any errors or issues. The simulation time of 18 hours proved to be a sufficient amount of time to allow convergence on a steady-state solution. A profile view of the water surface for both the current condition model and the pre-bridge crossing model is provided below for the highest flowrate of 500 cfs.

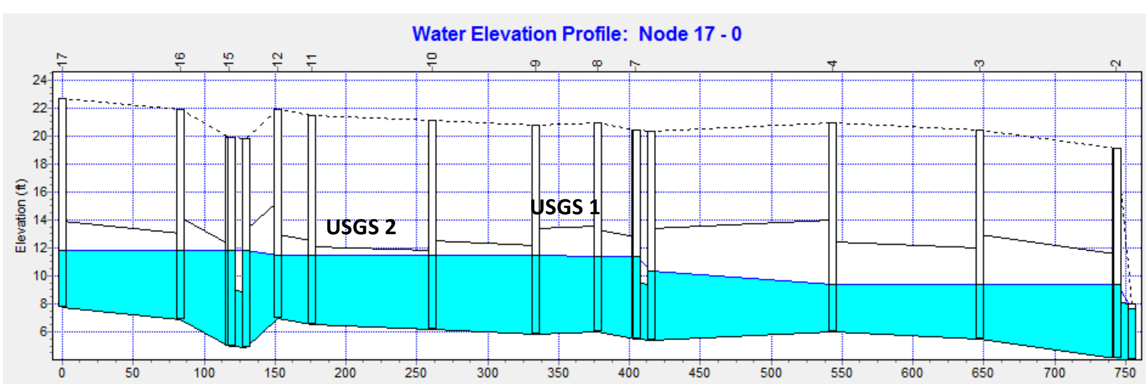

Figure 19: Profile View of Current Conditions Model Q=500 cfs

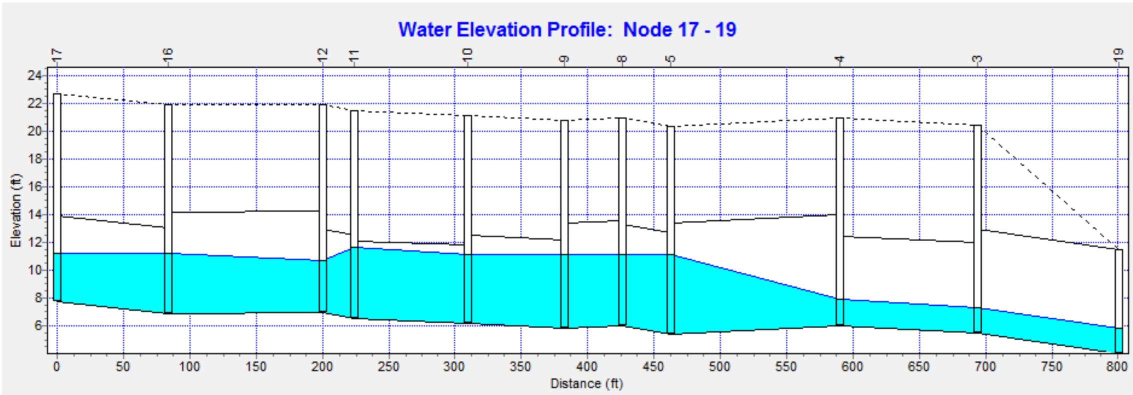

Figure 20: Profile View of Pre-Bridge Crossing Model Q=500 cfs

As you can see in Figure 19 with a flow of 500 cfs the water surface elevation exceeded the bridge and spilled over. This was also true for the 400 cfs flow but not for the 300 cfs or 200 cfs flowrates. However the main question was if the bridge crossings were affecting the water surface elevations. To compare the results from the two models the water surface elevations are provided for each of the flow rates in the following table.

Table 1: Simulation Results from SWIMM Models

Flow (cfs)	Gage	WS Elevation (ft)		% Difference
		Culverts	No Culverts	
500	US	11.63	11.61	0.17
	DS	11.54	11.28	2.30
400	US	11.47	11.36	0.97
	DS	11.37	11.09	2.52
300	US	11.2	11.01	1.73
	DS	11.11	10.77	3.16
200	US	10.85	10.04	8.07
	DS	10.76	9.84	9.35

In the above table you can see for the higher flowrates of 500 cfs and 400 cfs no significant change in the water surface elevation between the two models can be seen at the upstream gage. For the downstream USGS gage (Gage 1) there is a 2.3% and 2.5% difference for the 500 cfs and 400 cfs flowrates respectively. This values does raise some concern because a 2% difference in water surface elevation with such wide floodplains as we see here can make a large difference in flow. Another trend is that the percent difference in water surface elevation between the models increases with decreasing flow. At the lowest flowrate of 200 cfs there is a 9.3% difference in water surface at the downstream gage. This is most likely due to the culver restricting the flow at the downstream gage. This extreme large difference is not seen at the higher flowrates because the water spills over the bridge allowing a large amount of flow to continue downstream neglecting the restricting flow of the culvert. Another result that is interesting is the difference in elevation change from gage 2 to gage 1, which is used for the two-section slope-area method calculations, between the two models.

Table 2: Change in Elevation between Gages

Flow (cfs)	Change in Elevation (ft)		% Difference
	Culverts	No Culverts	
500	0.09	0.33	72.73
400	0.1	0.27	62.96
300	0.09	0.24	62.50
200	0.09	0.2	55.00

As you can observe the percent difference between the two models for the change in head of the water surface is drastic. It is concluded that his is due to Bridge B which backs up the flow downstream gage. This causes the water surface to be flatter under current conditions than it would under previous conditions without the culverts. This would make it appear during slope-area calculations that the water is moving at a slower velocity. In this case the percent difference increases with increases flow, which again is caused by the damming of the flow against Bridge B.

Recommendation

Due to the large difference in the change in water surface elevation of the gages between the two models I recommend that the gages be moved to another location. The bridge crossings are obviously having a significant impact on the water surface elevations especially at the downstream gage. Another reason I have this recommendation is if during a storm the culvert become clogged with debris it was increase the water surface elevations even more since the water will be restricted further in the culvert. This would cause an even greater incorrect reading and may raise some unnecessary concern in TxDOT.

The crest-stage gages are very expensive to remove and install so cost is not an issue. The gages could simply be moved further upstream or downstream on the golf course away from the bridges.

Sources of Error

Some sources of error that raise some concern are the scour holes caused by the culverts and a lack of complete convergence at the 200 cfs flowrate. One-dimensional flow modeling which is used by EPA SWIMM has a hard time realistically modeling scour holes or abrupt changes in elevation. This would more accurately be modeled in the two-dimensional modeling program which can compute swirling and recirculating affects. A figure showing the extremity of the scouring at one of the bridge crossings is provided in Figure 27 in the Appendix.

The second source of error is the incomplete convergence for the 200 cfs and 300 cfs flowrate for the pre-bridge crossing model at the upstream gage (Gage 2). The water surface depth provided by SWIMM varied about 0.1 and 0.25 ft for the 200 cfs and 300 cfs flowrate respectively. A plot of the water surface elevation for the 200 cfs flowrate at this location can be seen in Figure 28 in the Appendix.This issue was not seen on the current condition model. Since the variation was somewhat minor it was ignored, and the average water surface elevation was taken. This incomplete convergence did not improve with increased simulation time. The variation in flowrate had no significant effect on percent difference in water surface elevation between the models.

Appendix

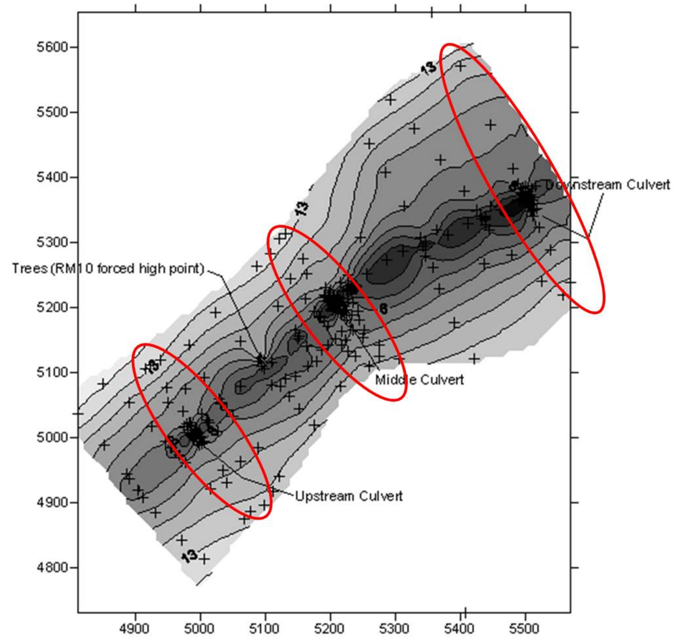

Figure 21: Surfer with Bridge Crossings

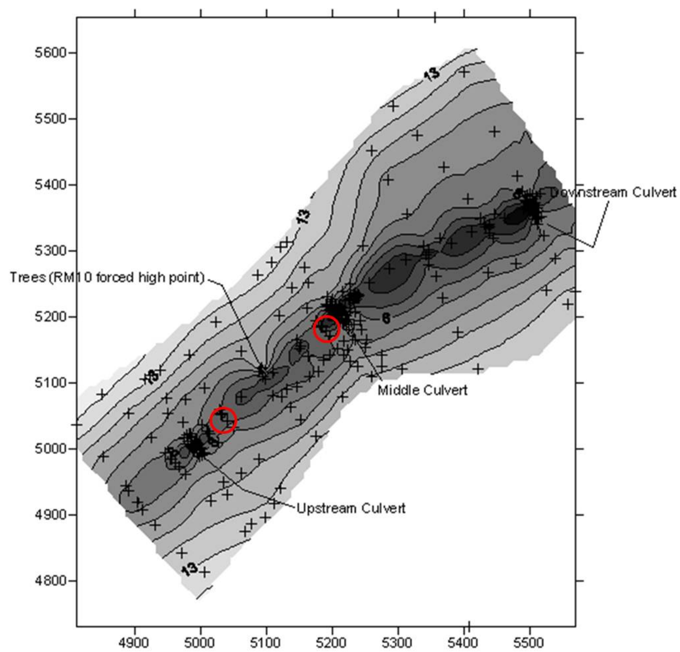

Figure 22: Surfer with USGS Crest-Stage Gages

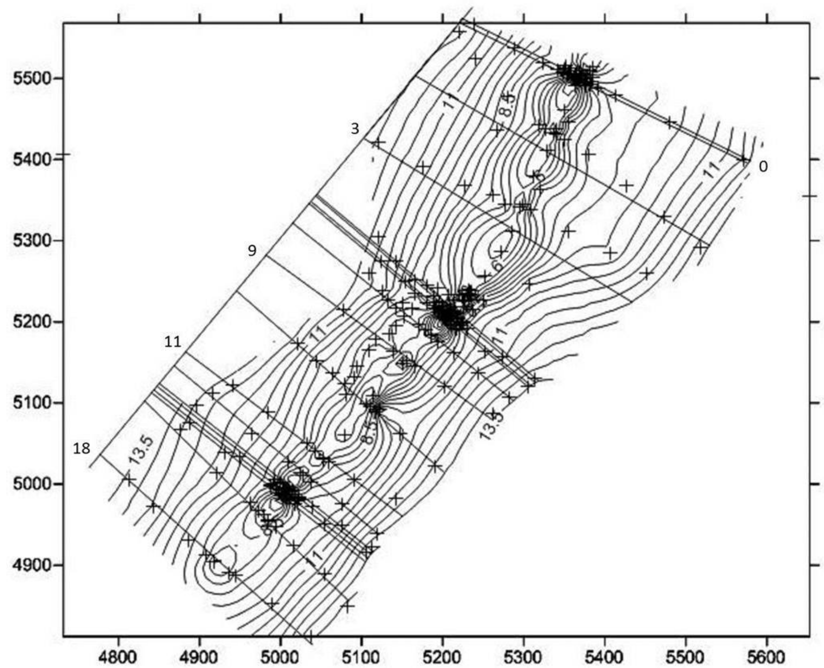

Figure 23: Surfer with Cross-Sections taken in Field

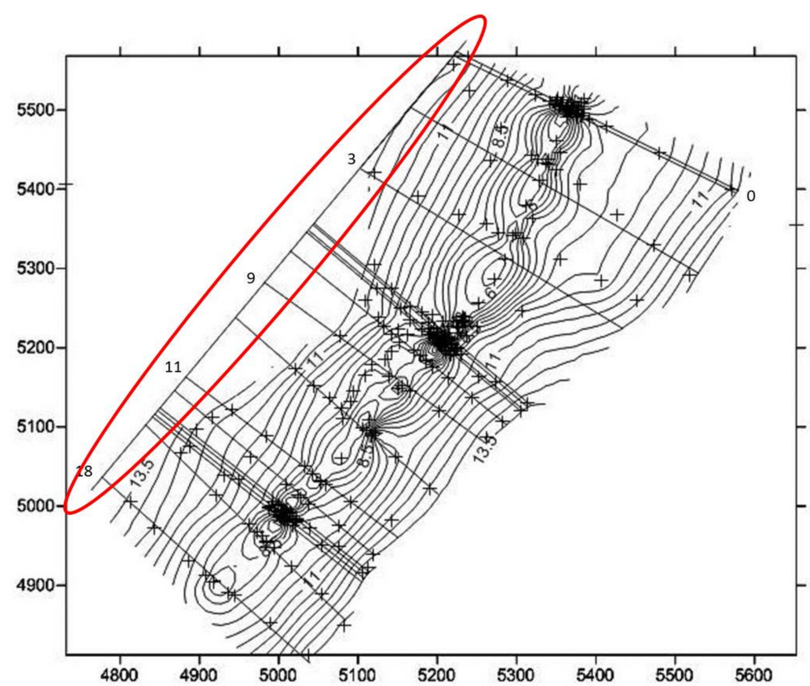

Figure 24: Surfer with Stationing Reference Line

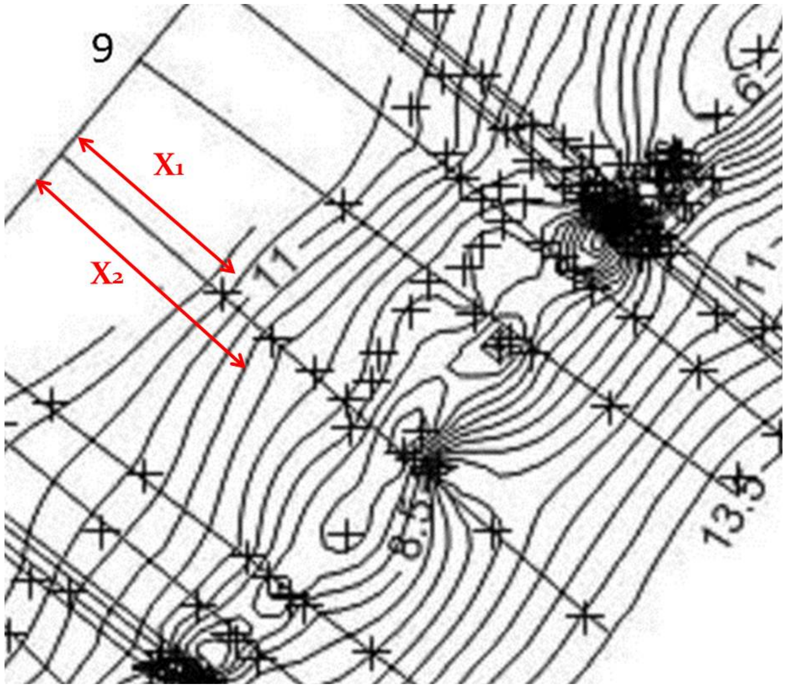

Figure 25: Stationing from Reference Line

Table 3: Stationing

XS-	17			
Pt. No.	X	Y	Station	Elevation
Start	4833.171	5102.79	0	Start
118	4876.261	5067.721	55.55728	12.5174
117	4920.985	5014.641	124.4248	11.2849
116	4962.224	4976.973	180.2342	9.2953
115	4973.398	4967.362	194.9471	7.3663
114	4979.317	4961.88	203.0129	7.9098
113	4985.004	4956.242	211.0205	6.9013
112	4994.054	4947.114	223.8715	9.4086
111	5016.903	4924.337	256.1302	9.7944
110	5054.23	4889.859	306.9311	11.8456
109	5082.983	4849.772	355.5618	14.2031

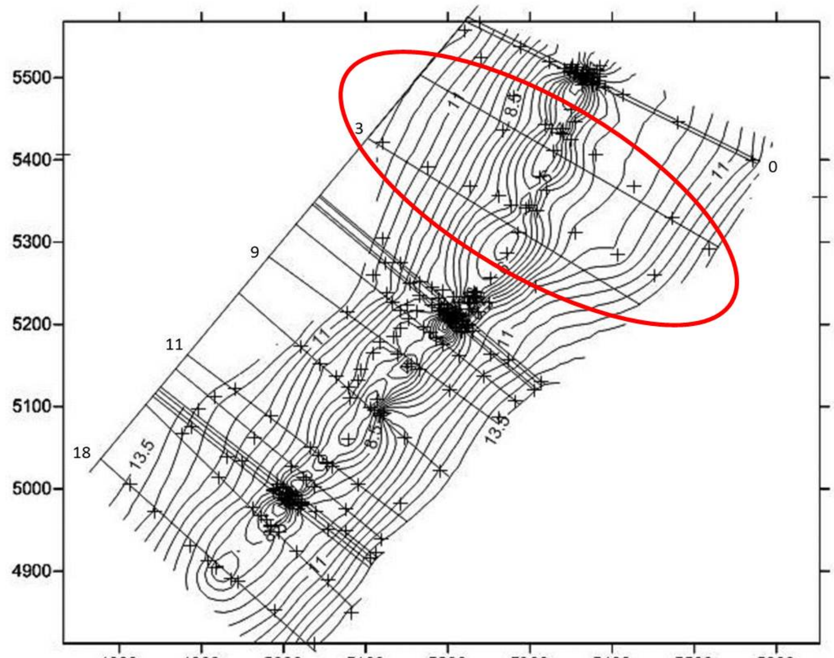

Figure 26: Digitized Cross-Sections

Table 4: Digitized Cross-Section Points

	XS-	3		
Pt. No.	X	Y	Station	Elevation
Start	5101.124	5426.138	0	Start
Digitized	5115.575	5418.416	16.38477	12.5
Digitized	5134.346	5406.918	38.38111	12
Digitized	5149.598	5396.829	56.6458	11.5
Digitized	5165.084	5387.678	74.63279	11
Digitized	5178.459	5379.934	90.08614	10.5
Digitized	5189.956	5372.895	103.5661	10
Digitized	5200.281	5366.325	115.8003	9.5
Digitized	5212.013	5359.521	129.3607	9
Digitized	5223.275	5352.716	142.519	8.5
Digitized	5232.192	5346.85	153.1842	8
Digitized	5241.812	5341.453	164.2092	7.5
Digitized	5252.606	5334.649	176.9662	7
Digitized	5262.93	5328.313	189.0791	6.5
Digitized	5272.316	5322.682	200.0246	6

Table 5: SWIMM Cross-Sectional Data

Transect Name: XS-18

Description:

	Station (ft)	Elevation (ft)
1	47.40644689	13.311
2	91.69887603	12.4597
3	151.4273841	10.1896
4	180.0976121	9.4423
5	193.0708242	7.7621
6	215.329954	8.4217
7	224.0986337	9.2354
8	279.9614411	10.8296
9	342.5130746	13.9085
10		

Property	Value
Roughness:	
Left Bank	0.030
Right Bank	0.030
Channel	0.030
Bank Stations:	
Left	0.0
Right	342.5
Modifiers:	
Stations	0.0
Elevations	0.0
Meander	0.0

Figure 27: Scouring at Bridge Crossing B

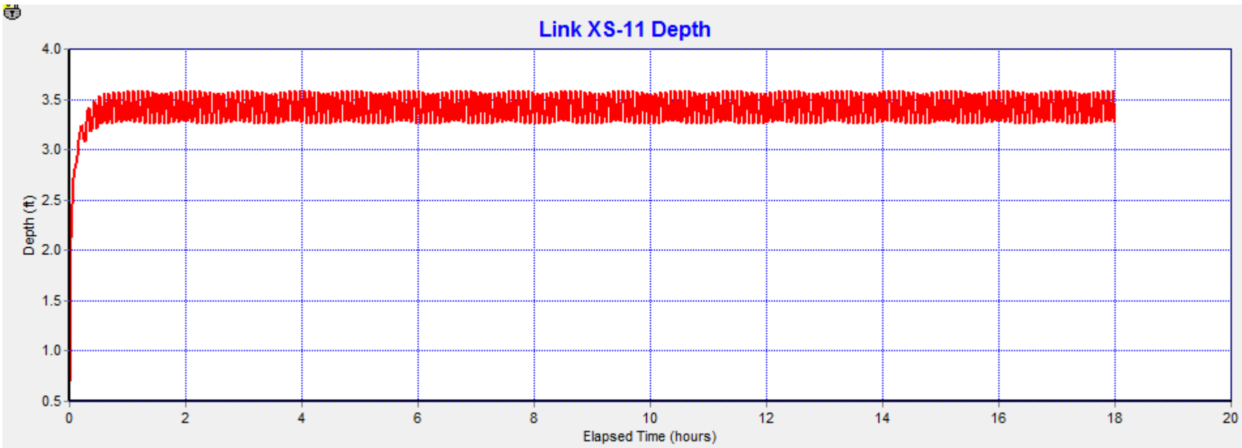

Figure 28: Incomplete Convergence at Upstream Gage for Q=200 cfs