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Evaluation Of Streambank Stabilization Options On The Tongue River In Cavalier, ND Utilizing Hec-Ras Hydraulic And Sediment Transport Simulations

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EVALUATION OF STREAMBANK STABILIZATION OPTIONS ON THE TONGUE RIVER IN CAVALIER, ND UTILIZING HEC-RAS HYDRAULIC AND SEDIMENT TRANSPORT SIMULATIONS

by

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Bachelor of Science, University of North Dakota, 2011

A Thesis

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Master of Science

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This thesis, submitted by Alexa Rene Ducioame in partial fulfillment of the requirements for the Degree of Master of Science from the University of North Dakota, has been read by the Faculty Advisory Committee under whom the work has been done and is hereby approved.

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This thesis is being submitted by the appointed advisory committee as having met all of the requirements of the School of Graduate Studies at the University of North Dakota and is hereby approved.

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May 4, 2018
Date

PERMISSION

Title:	Evaluation of Streambank Stabilization Options on the Tongue River in Cavalier, ND utilizing HEC-RAS Hydraulic and Sediment Transport Simulations.
Department:	Civil Engineering
Degree:	Master of Science

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Alexa Rene Ducioame
April 25, 2018

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ABSTRACT

The Tongue River in Cavalier has experienced severe erosion and bank failures in recent years. Two homes have been evacuated and demolished, and another lost ten ft of their yard overnight when a tree slumped into the river. The city lies downstream of Renwick Dam and nine other dams upstream of it which have greatly reduced the average flows through town. The resulting lowered water surface is a probable source of the streambank instability. A series of rock weirs was proposed to be installed in town to raise the minimum water surface elevation and potentially provide benefit to all problem sites identified in town. The existing conditions and several potential weir locations were analyzed using the Hydrologic Engineering Center River Analysis System (HEC-RAS) sediment transport analysis. The Park Street weir location was chosen as optimal for maximizing benefit at all problem sites, minimizing scour, and addressing the most urgent needs.

The Red River Riparian Project team originally identified twenty problem sites in town that needed attention. Since then, several of the locations have installed projects to protect the streambanks. Three potential weir locations were identified for this study: Woodland Terrace, Park Street, and Division Ave. The Woodland Terrace site is immediately downstream of the last original problem site. However, that site and the next upstream of it have had projects constructed. The Park Street weir is immediately downstream of the last problem site that has had no remediation. The Division Ave

location is closer to the area where more severe erosion was occurring and houses needed to be removed.

The existing conditions unsteady flow model geometry was created using LiDAR, survey, and other available data. Hydrology was developed with a U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) model of the Pembina River watershed and U.S. Geological Survey (USGS) gage data. Then the unsteady flow model was calibrated to 2013 flow and elevation data. The sediment transport model used the gradation data collected at the time of survey, USGS gage data, and several assumed parameters. The results do not precisely model actual erosion and deposition depths due to these assumptions and some software limitations. This study compares potential project impacts from each weir location. The parameters stay the same across all models and a preferred option can be selected by a relative comparison. The results can be verified with more detailed data and modeling in the future.

CHAPTER 1. INTRODUCTION

1.1 Location

The city of Cavalier is in Pembina County in the northeast corner of North Dakota. Cavalier lies along the Tongue River, a tributary of the Pembina River which then drains into the Red River of the North. Figure 1 shows a satellite image of the Tongue River. Renwick Dam is shown in the lower left corner of the image, and the city of Cavalier is shown downstream to the east. The reservoir created by the dam is an integral part of Icelandic State Park. The straight-line distance between the dam and city is about 8 miles. However, the river has a high sinuosity making the length of the river roughly 16 miles.

1.2 Problem

The Tongue River has experienced active stream bank erosion in the city of Cavalier. Several sites were already having serious bank failures at the commencement of the study. One home was evacuated due to an exposed foundation, and another lost ten ft of their yard overnight when a tree slumped down into the river. Renwick Dam was also the subject of national news when rainfall events in May of 2013 threatened its overtopping and the city of Cavalier was evacuated (Nicholson, 2013). That large flood event caused further streambank stability issues and another home was deemed unsafe.

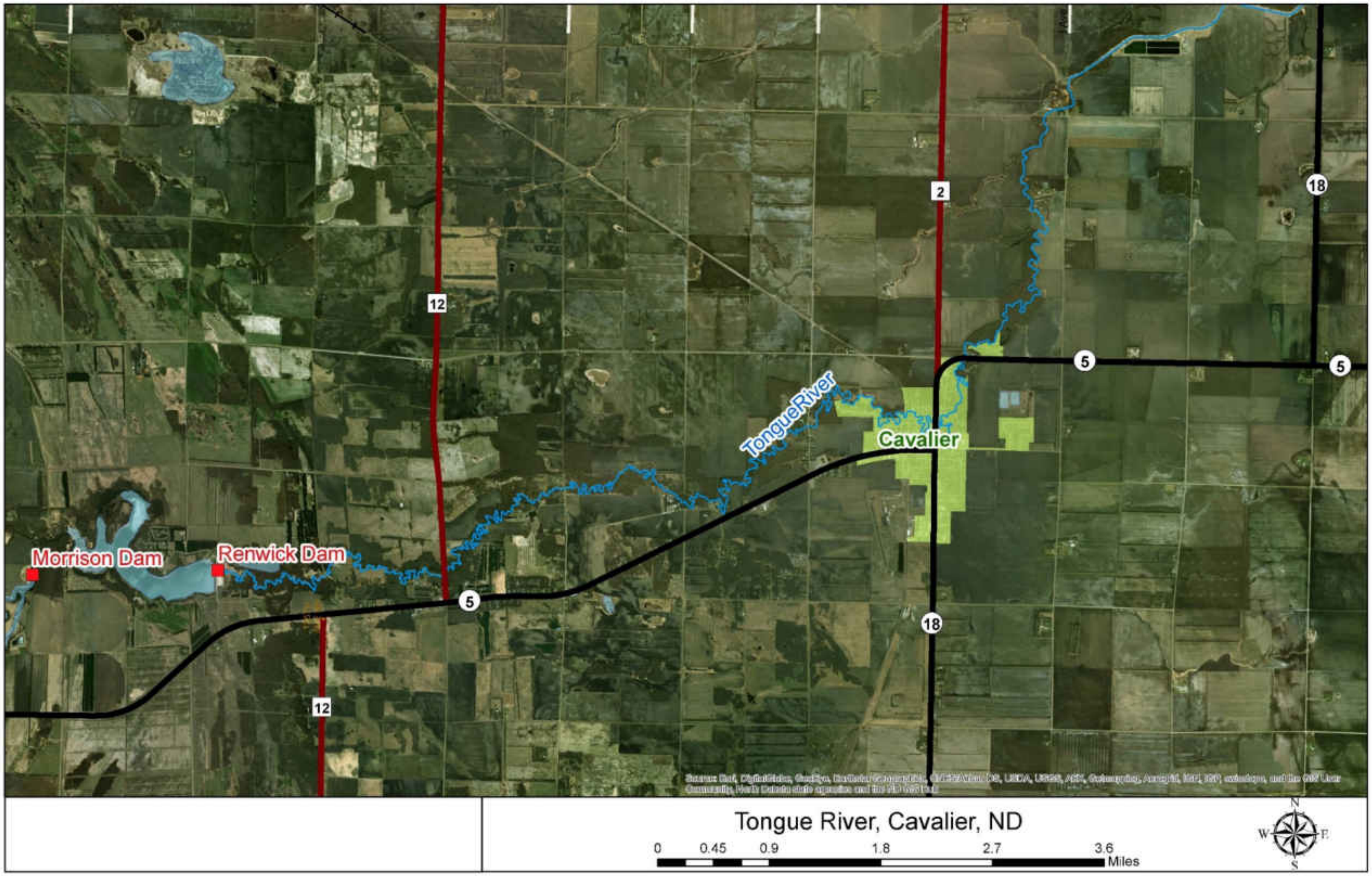

Figure 1. Tongue River at Cavalier, ND Map

1.3 Potential Solutions

The decision of whether to manage causes or treat the symptoms in the design will determine whether a passive nonstructural or active restoration technique is needed (Federal Interagency Stream Restoration Working Group (FISRWG), 1998). One of the factors for this decision is what effect past management activities have had. An option for restoration that the FISRWG offers is Riparian Buffer Strips, or an erodible corridor (Piégay, Darby, Mosselman, & Surain, 2005). These would reduce erosion while providing habitats for the area wildlife. In some locations in the city of Cavalier, however, there is very little space between the river and homes. Another option offered is surface armor, which includes using stone, concrete, grouted rip rap, or gabions to protect the stream bank. Indirect methods such as dikes, barbs, and bendaway weirs are also listed.

The Red River Riparian Project team has inspected many locations along the river, as shown in Figure 2. This team discusses bioengineering solutions in more detail (Red River Basin Riparian Project, 2004). Fiber rolls with wetland plant plugs can be installed for toe protection with erosion control fabric. A similar bioengineering method using willow fascines between barbs can also be done for toe protection. Bundles may be placed vertically along the banks of the river in small dug out trenches, or Juniper tree bundles can be placed along the toe with the butts pointed upstream and tops pointed downstream. These are overlapped by about one third the length of the tree. Locations with limited space, high velocities, and high bed load may consider using a log crib wall.

It is recommended that any locations with trees leaning into the river are remedied by cutting down the tree, but leaving their root systems in the banks for stability. Removing the weight of the tree increases the channel stability.

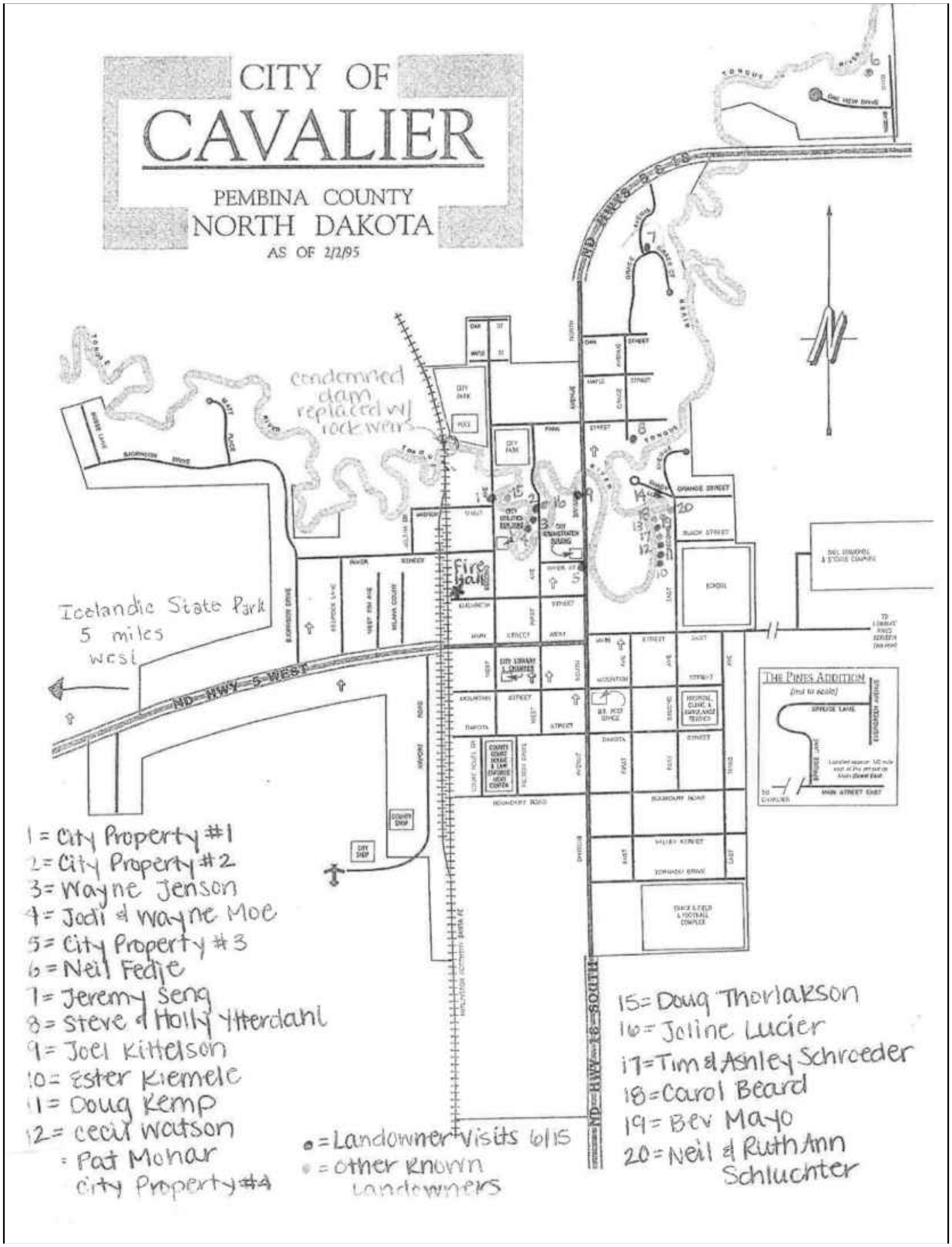

Figure 2. Red River Riparian Project Problem Site Map

In 2012 and 2013, Brian Mager and Howe Lim used the data collected to perform streambank stability assessments at 201 Woodland Terrace (Site 6) and 902 Grace Ave (Site 7) using the Bank Stability and Toe Erosion Model (BSTEM) software developed in Excel (Simon, Pollen-Bankead, & Thomas, 2011). This software has since been incorporated into HEC-RAS and is included in v5.0.3. Using cross section and soil data, BSTEM outputs a safety factor of the bank material with any value under 1.0 being unstable, values between 1.0 and 1.3 being conditionally stable, and values above 1.3 are considered stable. The safety values came back as 0.92 and 0.79, respectively, confirming their instability.

The influence of the height difference between the water table and the stream elevation on bank stability is shown in Figure 3 and Figure 4. The multiple dams installed upstream of the City have reduced water levels dramatically. This effect is multiplied by the fact that the mainly silty clay loam soil tends to be unstable with lower water contents. Within BSTEM the water surface was raised approximately three ft to simulate the effects that a downstream dam would create. The safety of the stream bank rose to 1.02 and 0.92, respectively. The low head concrete dam necessary to raise the water surface would limit fish passage, so a series of rock weirs was proposed instead. This would create additional habitat for fish as well (Brookes & Shields Jr, 1996).

The study noted that this scenario would decrease velocities to essentially create a lake or pond, and BSTEM toe erosion results would be inaccurately modeled. It also recognized that individual property solutions could pass the problem on to neighboring properties. Installing a series of rock weirs could likewise increase erosion further downstream.

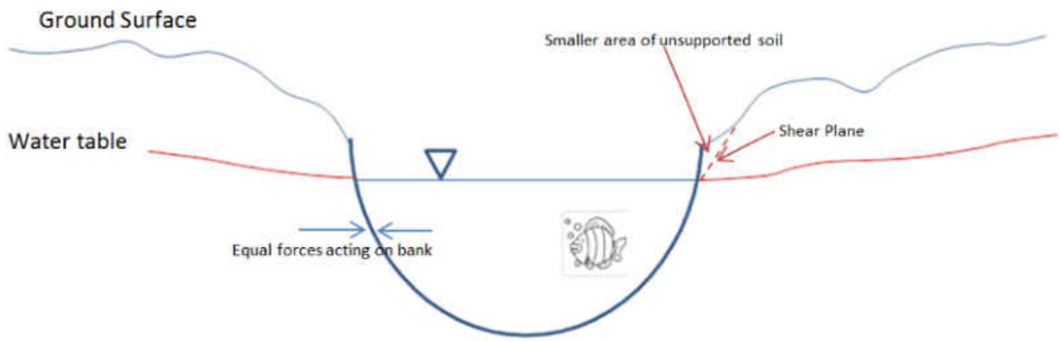

Figure 3. High water effects at a cross section

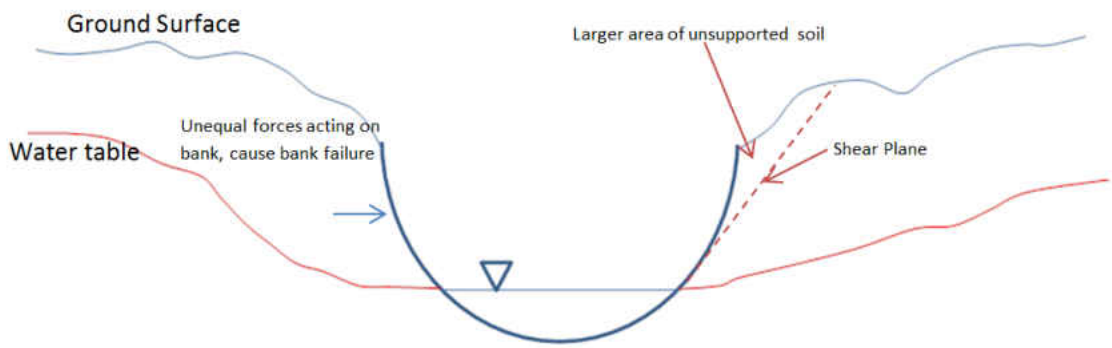

Figure 4. Low water effects at a cross section

This study continued with physical modeling of rock weirs (Lim & Mager, 2015). A series of experiments were conducted in the University of North Dakota Civil Engineering Hydraulic Laboratory using a constructed flume bed, sands with a known grain size distribution, and model rock weirs. Results found that with a ten to twenty foot weir spacing, no correlation was found to reduce scour. However, as the number of weirs increased, the scour also increased. It was recommended to allow more space between weirs such that each acts independently.

1.4 Single Site Projects

The two homes that were deemed unsafe are Sites 3 and 4 on the map in Figure 2. A comparison of the Google Earth imagery shown in Figure 5 and Figure 6 shows that the two unsafe homes have been removed since 2013. There are also large changes at the outer bends near W 2nd Ave N and W 1st Ave N. This was part of a slope stabilization project completed in 2015 by AE2S, the city engineer (AE2S, 2016). The project over-excavated the natural soils and replaced them with a granular backfill in a stepped cross section. The granular backfill acted as a base for fiber-reinforced cellular concrete that could provide structural integrity for the adjacent roads without overburdening the riverbank. Riprap was also added to the river slopes. The polyethylene fibers are advantageous for the climate because they can control effects of shrinkage and frost.

Several landowners applied for assistance in streambank stabilization projects at individual sites. At 201 Woodland Terrace, Site 6 on the map, a cribwall was installed in October of 2012. After removing trees and debris from the channel, a fabric was laid down, then a rock foundation was added along the river toe. Log cribwalls were built on the banks, then a backhoe was used to put them into place on top of the rock. The hollow cells were then filled with soil to keep everything in place. Photos of the process are shown in Figure 7.

Figure 5. Sept 14, 2013 Google Earth aerial imagery between W 1st and 2nd Ave N

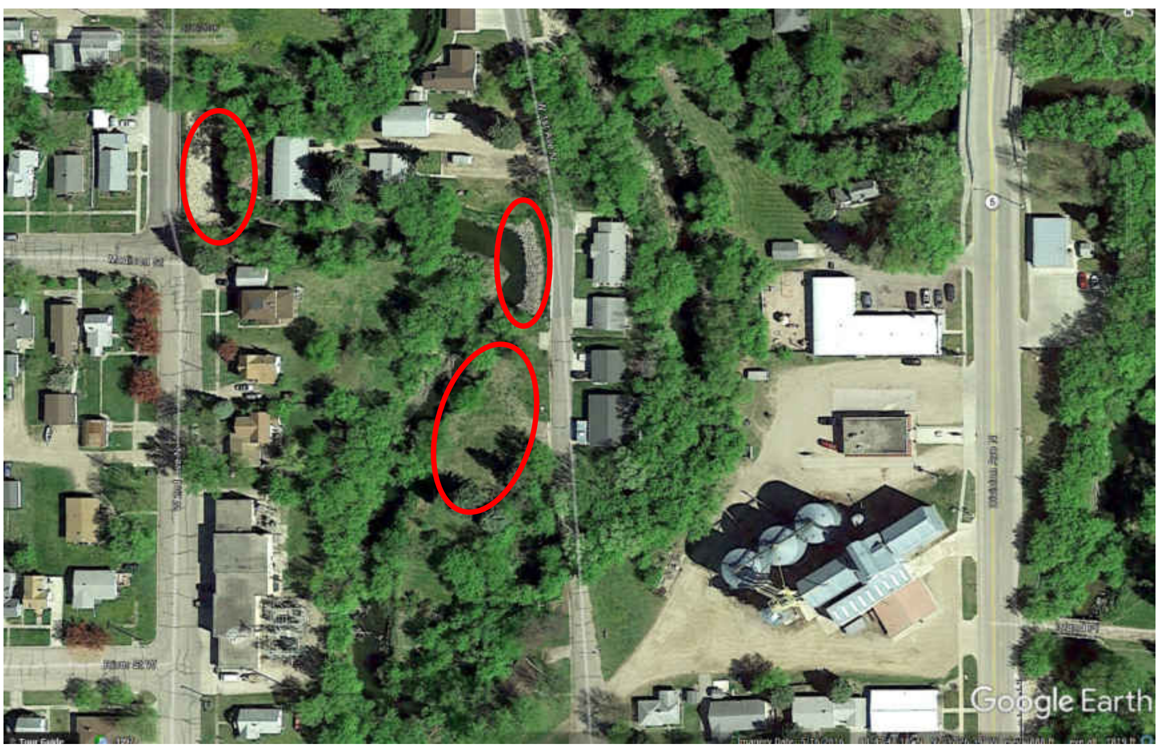

Figure 6. May 16, 2016 Google Earth aerial imagery between W 1st and 2nd Ave N

Removing fallen trees from the river

Building cribwalls

Installing rock along river toe

Installing cribwall on rocks

Figure 7. Photos: Cribwall installation at 201 Woodland Terrace, Cavalier, ND. Credit Neil Fedje

Site 7 on the map in Figure 2 also applied for assistance. The Google Earth imagery in Figure 8 and Figure 9 show that several trees were removed and the bank was stabilized with riprap. This means that the most downstream site identified by the Red River Riparian Project team that has not had a project is Site 8 at Park Street.

Figure 8. Sept 14, 2013 Google Earth aerial imagery at Grace Ave

Figure 9. May 16, 2016 Google Earth aerial imagery at Grace Ave

1.5 Objectives and Hypothesis

A majority of the sites identified by the Red River Riparian Project team have not been remedied. This is mostly due to landowner's skepticism of restoration project effectiveness and the affordability of a project. Even with cost share it can be a large bill for a single homeowner. A single project that would improve streambank stability for multiple landowners throughout the city would be ideal. The installation of a series of rock weirs, as discussed in Section 1.3, should be investigated. It is important to also consider the potential negative effects of a rock weir project on neighboring properties.

It is hypothesized that if a low head dam were installed at a location in town that it would improve erosion for all of the remaining sites, while increasing erosion at downstream properties. The null hypotheses to be tested: (1) there is not a weir location that can improve erosion for all of the remaining problem sites, and (2) the proposed project will not affect the downstream properties negatively.

A full hydraulic investigation needs to be carried out to test these. A calibrated sediment transport model would provide insight on the overall state of sediment movement in the river. The Hydrologic Engineering Center River Analysis System (HEC-RAS) software provides functions for unsteady flow sediment transport and mobile bed computations (HEC, 2016). The proposed hydraulic model would include the length of the river starting at Renwick Dam and extending to roughly 2 miles downstream of Cavalier.

CHAPTER 2. DATA COLLECTION

A wide range of data is necessary to create an accurate HEC-RAS model. The following were collected for this study: surface and elevation data; streamflow and stage in the river; suspended sediment readings; soil properties at various locations along the river; land use; bridge and weir dimensions; high water marks; and any useful information from related studies.

2.1 Datum

All data and models use the following coordinate system, projection, datum, and unit of measure.

Coordinate System and Projection:	NAD83 USA Contiguous Albers Equal-Area Conic
Vertical Datum:	North American Vertical Datum of 1988 (NAVD 88)
Unit of Measure:	U.S. Survey Ft

2.2 Mapping and LiDAR Data

Light Detection and Ranging (LiDAR) 1-meter Digital Elevation Model (DEM) data is available for the Tongue River riverbed and floodplain through the use of the Red River Basin Decision Information Network (RRBDIN, 2008). The elevation data was collected between April 19th and May 2nd of 2008. This DEM was imported into ArcGIS software and processed.

2.3 Models

A HEC-HMS model of the Pembina River Watershed and a HEC-RAS model of the Tongue and Pembina Rivers were received from the U.S. Army Corps of Engineers (USACE). This was not a detailed hydraulic study of the Tongue River and did not include any bathymetry, structures, nor calibration.

2.4 Streamflow Data

United States Geological Survey (USGS) gage height and discharge daily data; daily, monthly, and annual statistics; peak streamflow; field measurements; and a rating curve for the Tongue River at Akra were available online (USGS, 2018). The site for USGS data collection is shown in Figure 10.

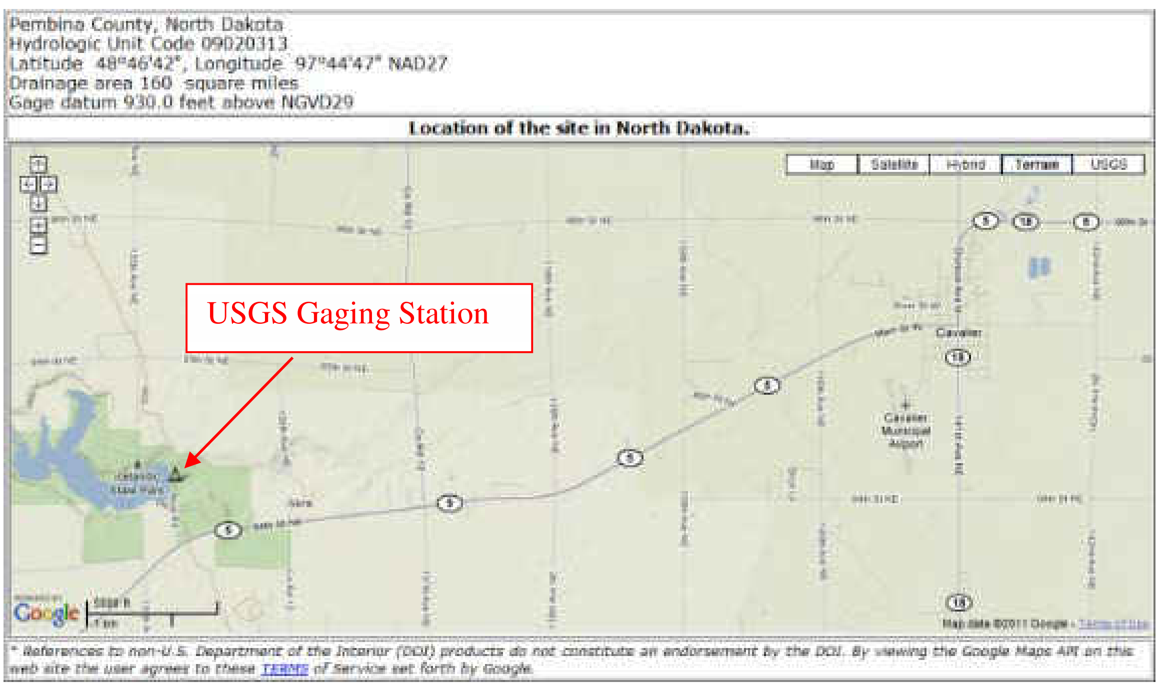

Figure 10. Location of USGS Site for Streamflow Data (USGS, 2018)

Peak streamflow data, recorded at this site, can be seen in Table 1 and is recorded from the years 1939 to present. The largest peak of 11,800 cfs occurred in 1950. The largest flow after dam construction is 1,550 cfs in 2013, followed by 1,150 cfs in 2009.

The 2013 discharge hydrograph is plotted in Figure 11. It shows that the event had three peaks due to heavy rainfalls that summer. The largest peak occurred on May 22nd. Field measurements gathered manually by USGS are added to the plot to verify the automated system's readings.

Table 1.
USGS Peak Streamflow for the Tongue River at Akra, ND (05100460)

Pre-Renwick Dam		**Post-Renwick Dam**					
Year	**Discharge (cfs)**	**Year**	**Discharge (cfs)**	**Year**	**Discharge (cfs)**	**Year**	**Discharge (cfs)**
1939	34	1961	60	1980	180	1999	380
1940	280	1962	473	1981	76	2000	40
1943	490	1963	210	1982	308	2001	413
1944	440	1964	286	1983	354	2002	554
1945	920	1965	580	1984	33	2003	220
1946	690	1966	492	1985	243	2004	630
1949	970	1967	412	1986	275	2005	496
1950	11,800	1968	160	1987	480	2006	616
1951	420	1969	606	1988	38	2007	491
1952	260	1970	567	1989	49	2008	161
1953	178	1971	568	1990	15	2009	1,150
1954	187	1972	325	1991	35	2010	462
1955	700	1973	118	1992	80	2011	507
1956	1,350	1974	595	1993	492	2012	139
1957	340	1975	76	1994	138	2013	1,550
1958	78	1976	313	1995	341	2014	241
1959	485	1977	64	1996	523	2015	209
1960	654	1978	429	1997	737	2016	323
		1979	900	1998	541	2017	552

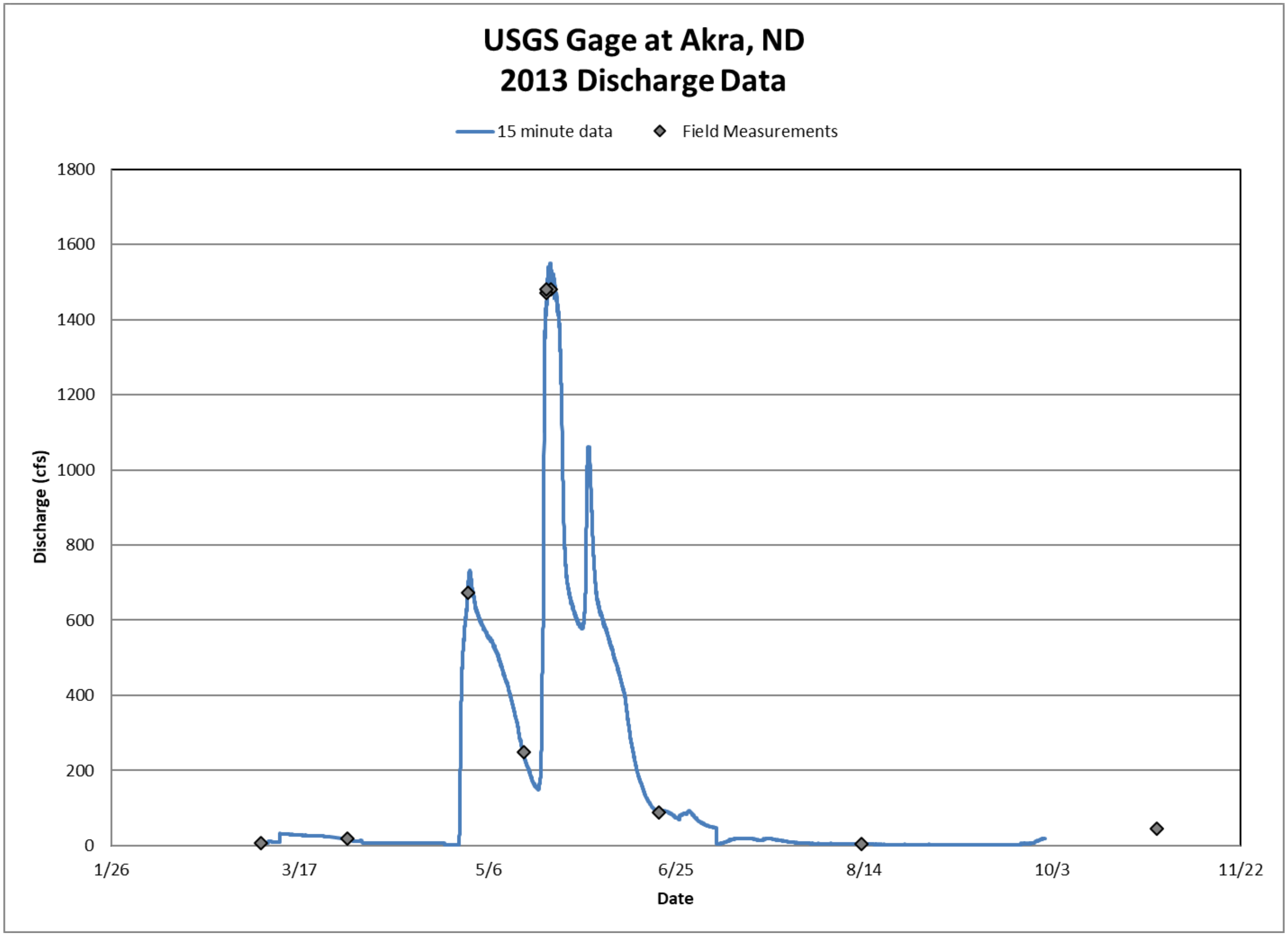

Figure 11. USGS gage at Akra, ND 2013 discharge hydrograph

2.5 Sediment Data

USGS Sediment Transport Data was available for the Tongue River at Akra (USGS, 2018). The USGS gaging site is shown in Figure 10. Although extensive streamflow data is recorded for this site, sediment data was only recorded from March 2003 to September 2004, as shown in Table 2.

Table 2.
USGS Sediment Transport Data

Date	Time	Time Datum	Instantaneous discharge ft3/s	Suspended Sediment, sieve diam, % < 0.0625 mm	Suspended sediment concn, mg/L
3/26/2003	13:45	CST		88	30
4/10/2003	13:50	CDT	201	51	642
4/11/2003	8:20	CDT	218	43	813
5/13/2003	13:30	CDT	25	17	59
5/20/2003	10:15	CDT	159		
6/3/2003	15:10	CDT	30		
6/10/2003	10:10	CDT	40	100	9
7/22/2003	8:30	CDT	2.3		
7/22/2003	8:35	CDT		97	10
8/28/2003	10:05	CDT	4.2	90	14
9/18/2003	12:00	CDT	3.4	74	32
10/8/2003	13:30	CDT	11		
3/3/2004	13:10	CST	2.8		
3/23/2004	12:05	CST	35		
3/30/2004	15:50	CST	567		
3/30/2004	15:55	CST	567	98	271
4/6/2004	16:05	CST	348	74	355
4/20/2004	15:25	CDT	151	73	136
5/26/2004	7:05	CDT	65	97	52
6/3/2004	8:20	CDT	220	40	193
6/28/2004	16:35	CDT	18	97	21
8/19/2004	13:40	CDT	1.1	98	22
9/14/2004	9:10	CDT	0.1	98	11

2.6 Soil Data

The Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS) has substantial information about soil types in the area (NRCS, 2017). The WSS image is shown in Figure 12 below. Though numerous soil types exist in the vicinity of the Tongue River, the WSS showed that the river followed a path with only 3 different types of soils. Abbreviated soil types in Figure 12 are listed in Table 3. As shown in the table, all three of the soil types are varying classifications of silty clay loams. In order to interpret the NRCS map, Table 4 defines average d_{50} values for various soil texture classes (Tomer, et al., 2005). For silty clay loams, approximately 27-40% is clay (< 0.002 mm in diameter) and < 20% is sand (0.0625-2 mm). Some general values for the angle of repose are shown in Table 5.

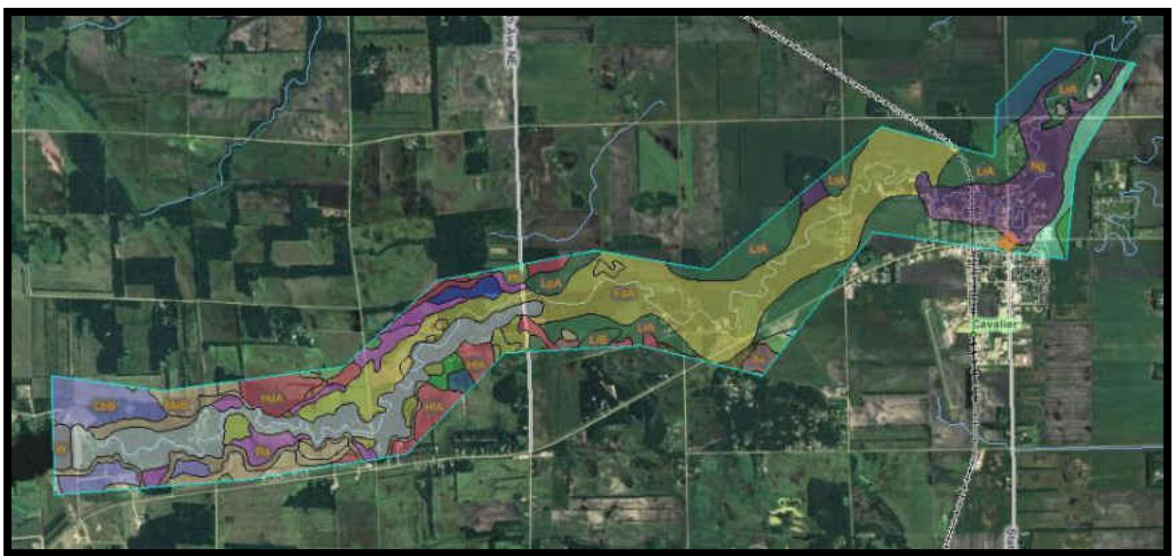

Figure 12. NRCS Soil Data (NRCS, 2017)

Table 3.
NRCS Soil Region Data along Riverbed

Soil Region	Description
LvD (grey)	La Prairie Fairdale Silty Clay Loam
FaA (yellow)	Fairdale Silty Clay Loam
Ng (purple)	Neche Silty Clay Loam

Table 4.
D_{50} values for various soil texture classes

Soil Texture Class	D_{50} (mm)
Clay	0.023
Silty clay	0.024
Sandy clay	0.066
Silty clay loam	0.025
Clay loam	0.018
Sandy clay loam	0.091
Silt	0.019
Silt loam	0.027
Loam	0.035
Very fine sandy loam	0.035
Fine sandy loam	0.080
Sandy loam	0.098
Coarse sandy loam	0.160
Loamy fine sand	0.120
Loamy sand	0.135
Loamy coarse sand	0.180
Very fine sand	0.140
Fine sand	0.160
Sand	0.170
Coarse sand	0.200

Table 5.
Angle of repose for various soil types

Soil Type	Slope Ratio (Width to Height)	Slope Angle
Granular soils: crushed rock, gravel, on-angular, poorly graded sand, loamy sand	1.5:1	34
Weak cohesive soils: angular well graded sand, silt, silty loam, sandy loam	1:1	45
Cohesive soils: clay, silty clay, sandy clay	0.75:1	53

2.7 Land Use Data

The National Agricultural Statistics Service Cropland Data Layer (NASS-CDL) was downloaded for the project area to show land use (USDA NASS, 2017).

2.8 Survey

While the LIDAR data collected shows the dry land accurately, where the river is located it only represents the elevation of the water surface at the time the data was collected and there is no information regarding the riverbed elevation. Field survey was necessary to ensure the bathymetry of the cross sections are defined. A summary of the survey data collected is shown in Table 6.

The Hydrologic Engineering Center published a Master of Science thesis project (Hunt, 1995) that investigated bridge expansion and contraction reach lengths and coefficients. A total of 76 cases were modeled with varying river slopes, bridge opening widths, overbank to channel n-value ratios, and abutment type. From the results, regression analyses were performed to develop predictor equations for contraction and expansion reach lengths, ratios, and coefficients. Figure 13 illustrates the transition lengths and ratios.

Table 6.
Summary of surveyed data

Activities	Period	Tasks Accomplished	Students/Engineers
Topographic field survey and sediment data collection	Nov 2011 - Apr 2012	- Surveyed the full cross sections of selected Tongue River locations between Cavalier and Renwick Dam (24 sections) and at 5 sections at the rock weirs - Collected soil samples at six sites	Alexa Ducioame James Norberg Matthew Erickson Brian Mager Derrick Deering Hasin S. Munna Ethan Kitsch Howe Lim (advisor)
Collect soil and cross section data	Jun - Jul 2012	- Collected topographic features of two representative slope failure sites - Collected soil samples	Brian Viall Ahmed Yusuf Howe Lim (advisor)

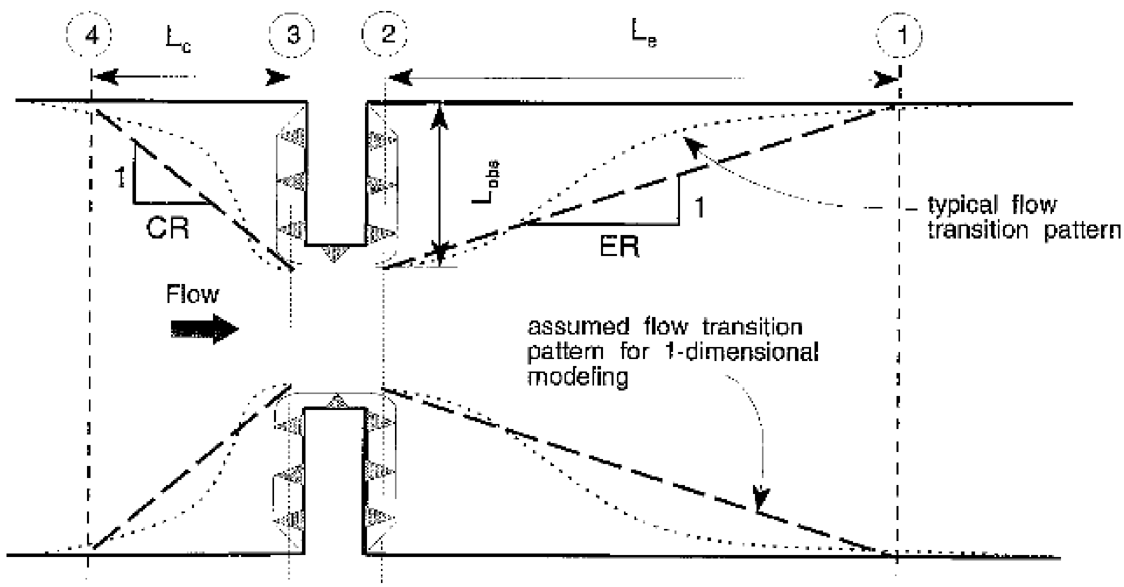

Figure 13. Illustration of bridge transition lengths (Brunner, 2016)

Traditionally the expansion reach lengths (Le) were ruled to have a 4:1 ratio. In this study the ratio was less than 4:1 for all of the idealized cases. The mean and median values were both approximately 1.5:1. This means that the length will be overestimated with the tradtional approach. The contraction reach lengths (Lc) had been recommended at a 1:1 ratio. The results from this study had a range of 0.7 to 2.3 with an average of 1.1:1. This study was used as a guide when collecting cross section data near structures.

In November 2011 several cross sections within town were collected for the preliminary model using a theodolite, or total station. For this field trip only two students went, and collected a limited amount of data. The river was frozen over, so a sledge hammer was used to create holes in the ice for the surveying rod to sit in and measure the elevation of the river bed. The rock weir average elevations were collected and an upstream cross section for the Division Avenue Bridge was obtained.

On March 31st a survey team of seven went for the entire day and collected a cross section at the dam, as well as four cross sections each for the bridges at 136th Ave, County Road 12, 138th Ave, 139th Ave, 140th Ave, the Railroad, 1st Ave, and the downstream side of Division Ave. Locations on each bridge were chosen to be bench marks and recorded. Gradation samples were also collected at the Dam, 136th Ave, 138th Ave, 140th Ave, 1st Ave, and the Island Bridge in town. In early April a crew of four returned with a GPS unit to determine the elevations of the bench marks and wrap up cross section survey. AE2S has a calibrated benchmark within the city of Cavalier that was used for some of the survey.

In the summer of 2012 two students returned to survey cross sections at two problem sites: 201 Woodland Terrace and 902 Grace Ave. These cross sections were collected to use independently in the BSTEM software, so elevations are in relation to a chosen “zero” point. The bank survey was used to estimate a conversion to NAVD88 later. The channel side slopes are in the order of 1:1 and 1:1.7, respectively. In comparison, a 1:3 slope is a sustainable slope for the silty clay loam soil found predominantly around the sites.

2.9 Structures

During the summer of 2011, the condemned railroad dam in the city of Cavalier was replaced with a series of three rock weirs. The original dam was 60 ft wide with a top width of 3 ft and bottom width of 20 ft. The North Dakota State Water Commission (NDSWC) was responsible for the design of the rock weirs. When design drawings and as-built data were requested from the SWC, the information provided stated that the top weir was to be placed slightly above the sediment pool, and the others were to step down at a 5% slope. The boulders used were 18-inches or greater in diameter. As noted in Section 2.8, the weirs were surveyed in November of 2011.

Bridge inspection reports were obtained from the North Dakota Department of Transportation (NDDOT). These contain span lengths, deck thickness, and pier information needed for the HEC-RAS model.

2.10 High Water Marks

The only official river measurements available are at the Akra USGS gage immediately downstream of Renwick Dam. Additional high water marks are helpful for a better calibrated model. Pembina County Emergency Services, the Red River Riparian Project, AE2S, Icelandic State Park, the Cavalier Chronicle, and local residents were contacted to request any photos taken along with the date of the photos. Thirty one photos of the river and adjacent properties taken between April 30th and June 14th, 2013 were received. Exact times are not known, however they were all taken during the day. A time of 12:00 PM will be assumed for modeling purposes. The photos used are shown in Figure 14 through Figure 17. Note that the property in Figure 17 had a cribwall installed in October 2012.

A number of photos near Renwick Dam were also collected, but they are not included in this report because there is gage elevation data there already. Two aerial images of the floodplain were also collected from a pilot.

While these high water marks are not precise elevations, they are useful to know whether the model results should show water overtopping a road or inundating a backyard for an approximate calibration.


Grace Ct area near curve


Hwy 5 curve


2nd Ave bridge looking east


2nd Ave bridge looking west

Figure 14. Photos: May 3, 2013 flooding at various locations in Cavalier, ND. Credit Cavalier Chronicle

Figure 15. Photos: May 21, 2013 flooding at 305 W 1st Ave N, Cavalier, ND. Credit Wayne Moe

Island Bridge

City park and W 2nd Ave

Road leading out of pool parking lot looking Southeast

Sand Hill Lane west of Cavalier

Figure 16. Photos: May 26, 2013 flooding at various locations in Cavalier, ND. Credit Cavalier Chronicle

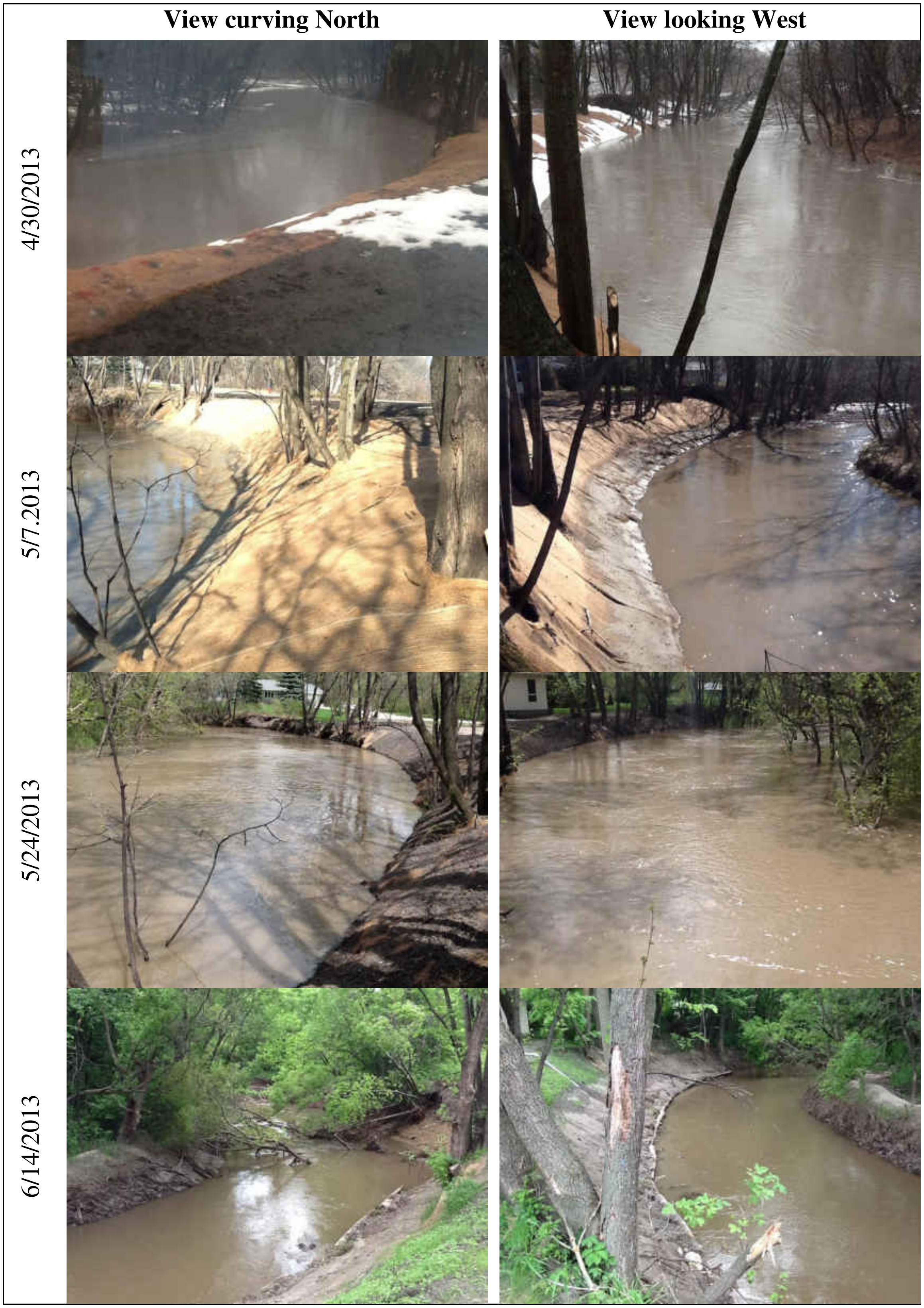

Figure 17. Photos: 2013 flooding at 201 Woodland Terrace, Cavalier, ND. Credit Neil Fedje

CHAPTER 3. METHODOLOGY

3.1 Sediment Sample Analysis

The six samples were collected according to approved standards D75 and T2 (ASTM, 2014) (AASHTO & NDDOT, 2015). The bank sample was collected from multiple locations for a single sample. The collected sample size was four times the size of the desired amount for testing. It was then mixed and split down to a desired amount following C702 and T248 (ASTM, 2018) (AASHTO & NDDOT, 2015). For a fine aggregate sample, the size of the sample after drying needed to exceed 300g. (ASTM, 2014).

The sample was dried at a temperature of 230± 9° F until it reached a constant mass. Once this was achieved, the original weight was taken and the sample was washed. After the sample was agitated, it was decanted over stacked No. 16, 100, and 200 sieves. The No. 16 sieve was used to help remove any remaining organic material that had not been picked out by hand. The No. 100 sieve was added to the washing process to help prevent the sieves from clogging and losing material since these were very fine samples. The process was repeated until the wash water ran clear. The material remaining on the sieve was then rinsed back into the sample and the sample was again dried at a temperature of 230± 9° F (AASHTO & NDDOT, 2015). Once dry, the sample was weighed, then run through the chosen sieves (AASHTO & NDDOT, 2015).

To clean the sieves, a wire brush can be used for anything greater than the No. 40 sieve. For the finer mesh sieves, a soft brush should be used. The cumulative weights, percent retained, and percent passing were then calculated. It is also important to perform a weight check to ensure that no material was lost.

Table 7 shows a comparison of the six gradations, their estimated d_{50}, and standard deviation. The dam sample contained the most distributed sample and larger particles than the other five. The upstream d_{50} values are similar to the average value of 0.025 listed for silty clay loam in Table 4, while the samples closer to town are bit finer. The full gradation results for each of the six locations are included in APPENDIX A – SIEVE ANALYSIS RESULTS.

Table 7.
Six Tongue River gradations compared by percent passing

(mm)	Ret.	Dam %Pass	136th %Pass	138th %Pass	140th %Pass	Division Ave. %Pass	1st Ave. %Pass	Standard Deviation
4.75	No. 4	99.07	100.00	99.82	100.00	99.97	99.78	0.36
2.36	No. 8	98.04	100.00	99.88	99.96	99.82	99.75	0.76
2.00	No. 10	96.95	100.00	99.83	99.95	99.70	99.65	1.18
1.18	No. 16	92.70	99.98	99.59	99.83	99.24	99.06	2.81
600µm	No. 30	82.71	99.03	99.09	99.54	98.26	97.97	6.59
425µm	No. 40	73.54	96.33	98.26	99.26	97.21	96.79	9.87
300µm	No. 50	62.51	89.62	96.18	98.63	95.51	95.24	13.60
150µm	No. 100	32.49	51.25	68.88	81.70	77.04	82.38	19.93
75µm	No. 200	14.05	17.18	22.86	22.99	32.49	41.31	10.12
Estimated d_{50} (µm):		237.49	294.49	238.46	219.01	208.94	181.73	37.90

3.2 Hydrologic Analysis

The Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) v4.1 was used for this study (HEC, 2017).

3.2.1 Historic Events

Daily stage and discharge data from the USGS gage at Akra was collected for the 2013 and 2009 events. The 2013 event was chosen as the calibration event, with the 2009 event used for verification. Not only is the 2013 event the largest since the dam has been in place, but there is more data available.

The subwatershed downstream of Renwick Dam ends approximately 8.5 miles downstream of where the HEC-RAS model was cut off for this study. The spring drainage area ratio equation for the Red River of the North Basin was used to convert the calibrated HEC-HMS hydrographs to a smaller area of approximately 17.5 mi^2 (USGS, 2005). The new hydrograph was added as a uniform lateral inflow in the HEC-RAS model.

3.2.2 Synthetic Events

The HEC-HMS calibrated model from the USACE had 2-, 5-, 10-, 25-, 50-, and 100-year 24-hour events; 25-, 50-, 100-year and 100-year runoff 10-day events; and 2002, 2005, and 2013 historic events.

There are ten dams upstream of the city of Cavalier that were all constructed between 1955 and 1961, as shown in Figure 18 and Table 8 (NDSWC, 2018). Due to this, the HEC-HMS model was modified to reanalyze the flows. Two HEC-HMS model scenarios were created for each event: one mimicked unregulated conditions by removing all dams, while the other mimicked fully regulated conditions.

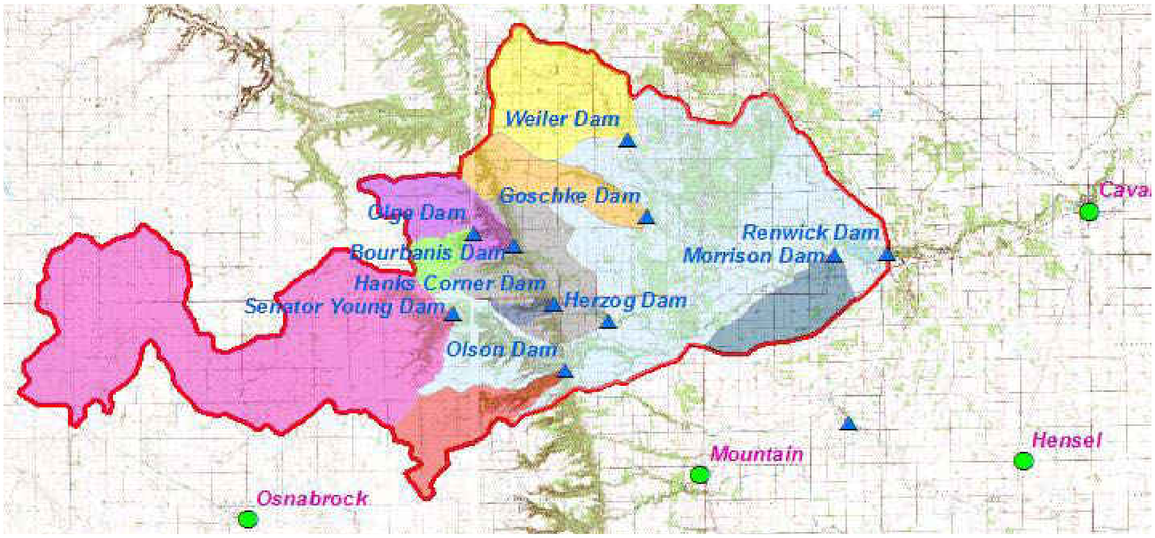

Figure 18. Tongue River Watershed dam map

Table 8.
Tongue River Watershed dam construction dates

Dam	Built
Olga Dam	1955
Hanks Corner Dam	1955
Morrison Dam	1956
Bourbanis Dam	1957
Olson Dam	1957
Herzog Dam	1957
Weiler Dam	1957
Goschke Dam	1958
Senator Young Dam	1961
Renwick Dam	1961

The model was run for each scenario and the comparison of the regulated and unregulated results is shown in Table 9. The average of the regulated vs unregulated ratios is 0.334. The USGS annual peak data (Table 1) was adjusted to represent unregulated data before it was entered into the Hydrologic Engineering Center Statistical Software Package (HEC-SSP) to develop flood frequency curves using the WRC Bulletin 17B method (HEC, 2017) (Interagency Advisory Committee on Water Data, 1982). The 1939-1954 unregulated peaks were kept as is, the 1955-1960 peaks were omitted because there were various numbers of dams being constructed, and the years after 1960 were divided by the 0.334 ratio to simulate unregulated flows as shown in Table 10. The HEC-SSP resulting values were then multiplied by the regulated vs unregulated ratio of 0.334 and the results are shown in Table 11. For comparison, the 2013 USGS peak streamflow at Akra was 1,550 cfs (USGS, 2018).

Table 9.
Regulated and Unregulated HEC-HMS Output

Event		Peak Outflow (cfs) at Akra Gage			Ratio
		Regulated	**Unregulated**	**Diff**	
24-hour	2-year	300.5	3606.6	3306.1	0.08
	5-year	617.9	2047.4	1429.5	0.30
	10-year	1045.1	2898	1852.9	0.36
	25-year	1393.5	4368.5	2975	0.32
	50-year	1750	5086.8	3336.8	0.34
	100-year	2150.3	6223	4072.7	0.35
10-day	25-year	1334.2	2669.8	1335.6	0.50
	50-year	1450.5	3606.6	2156.1	0.40
	100-year	1885.1	4740	2854.9	0.40
	100-year RO	1855.2	4082.9	2227.7	0.45
Historic	2002	554.9	1957.7	1402.8	0.28
	2005	423.5	1809.1	1385.6	0.23
	2013	1551.6	4947.1	3395.5	0.31

The 1979 flood was chosen as a typical regulated event to use as a pattern hydrograph. Multipliers were applied to create the peaks in Table 11. The resulting hydrographs are plotted in Figure 19.

The local inflow downstream of the dam determined for the 2013 event was also used as a pattern hydrograph and adjusted for each synthetic event using multipliers.

Table 10.
USGS Peak annual streamflow with unregulated flow adjustments

Water Year	Flow (cfs)	Water Year	Unreg Adj Flow	Water Year	Unreg Adj Flow	Water Year	Unreg Adj Flow
1939	34	1961	720	1979	2,496	1997	2,442
1940	280	1962	2,021	1980	2,160	1998	1,909
1943	490	1963	2,520	1981	912	1999	1,623
1944	440	1964	3,433	1982	3,697	2000	480
1945	920	1965	2,046	1983	4,249	2001	1,764
1946	690	1966	1,736	1984	396	2002	1,958
1950	11,800	1967	1,760	1985	2,916	2003	2,640
1951	420	1968	1,920	1986	3,301	2004	2,087
1952	260	1969	2,008	1987	2,050	2005	1,809
1953	178	1970	2,000	1988	456	2006	2,041
1954	187	1971	2,004	1989	588	2007	1,732
		1972	3,901	1990	180	2008	1,932
		1973	1,416	1991	420	2009	3,189
		1974	1,972	1992	960	2010	1,974
		1975	912	1993	1,736	2011	1,789
		1976	3,757	1994	1,656	2012	1,668
		1977	768	1995	4,093	2013	4,947
		1978	1,833	1996	1,845	2014	2,892

Table 11.
Akra Gage Adjusted Regulated Flows for Various Return Periods

Percent Chance Exceedance	Return Period (Year)	Adjusted Regulated Flow (cfs)
0.2	500	3,094
0.50	200	2,523
1.00	100	2,112
2.00	50	1,720
5.00	20	1,238
10.00	10	905
20.00	5	602
50.00	2	252
80.00	1.25	93
90.00	1.11	53
95.00	1.05	32
99.00	1.01	12

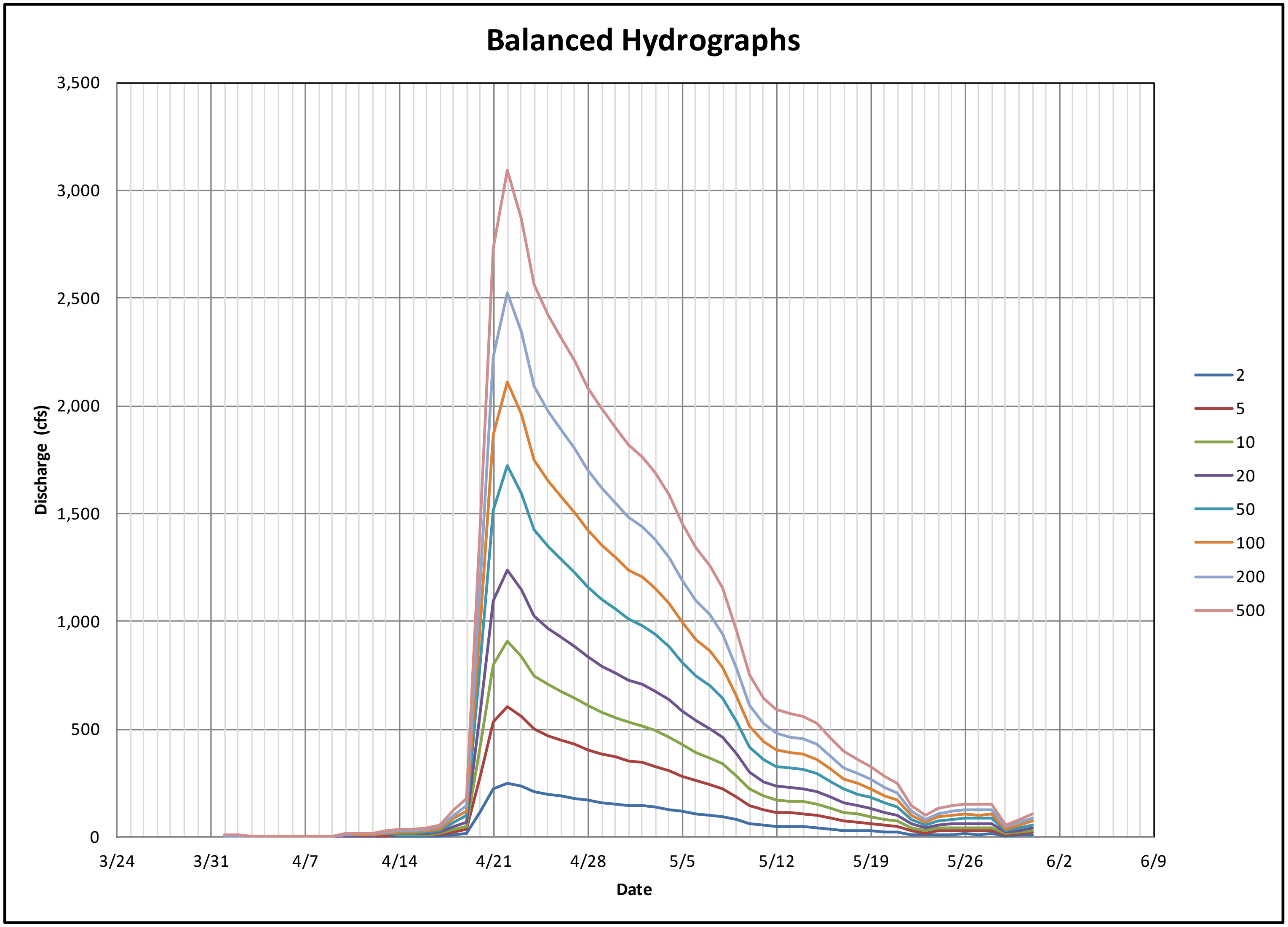

Figure 19. Synthetic balanced hydrographs using 1979 USGS data as pattern hydrograph

3.3 Hydraulic Analysis

The Hydrologic Engineering Center River Analysis Software (HEC-RAS) version 5.0.3 was used for this study (HEC, 2016).

3.3.1 Geometry

An example of the HEC-RAS model components is shown in Figure 20. The model utilizes cross sections (green lines) to convey flow within a river channel (blue line) and storage areas (blue polygons) to represent adjacent ponding areas such as fields. The model utilizes lateral structures to convey water from the channel cross sections to adjacent storage areas or other cross sections and storage area connections to convey water between storage areas (both shown as red and black dashed lines). Bridges and inline structures, such as dams, are shown as grey areas between the cross sections. The HEC-GeoRAS toolset within ArcGIS was used to create the cross sections, bridges, inline structures, storage areas, lateral structures, and storage area connections. The full model layout is shown in Figure 21.

A total of 143 cross sections were spaced less than 2,000 ft apart, being much closer together near structures. The cross section editor window is shown in Figure 22. The LiDAR cross section data then needed to be merged with the survey data collected. The surveyed cross sections were interpolated to the same locations as the LiDAR cross sections, then everything was merged within the channel bottom using the graphical cross section editor tool in HEC-RAS. This better represents the channel bathymetry and will allow the model to match the rating curve at Akra more closely than the original model received from the USACE. Ineffective flow areas and obstructions were added to represent constricted flow near bridges and adjacent swales and oxbows more accurately.

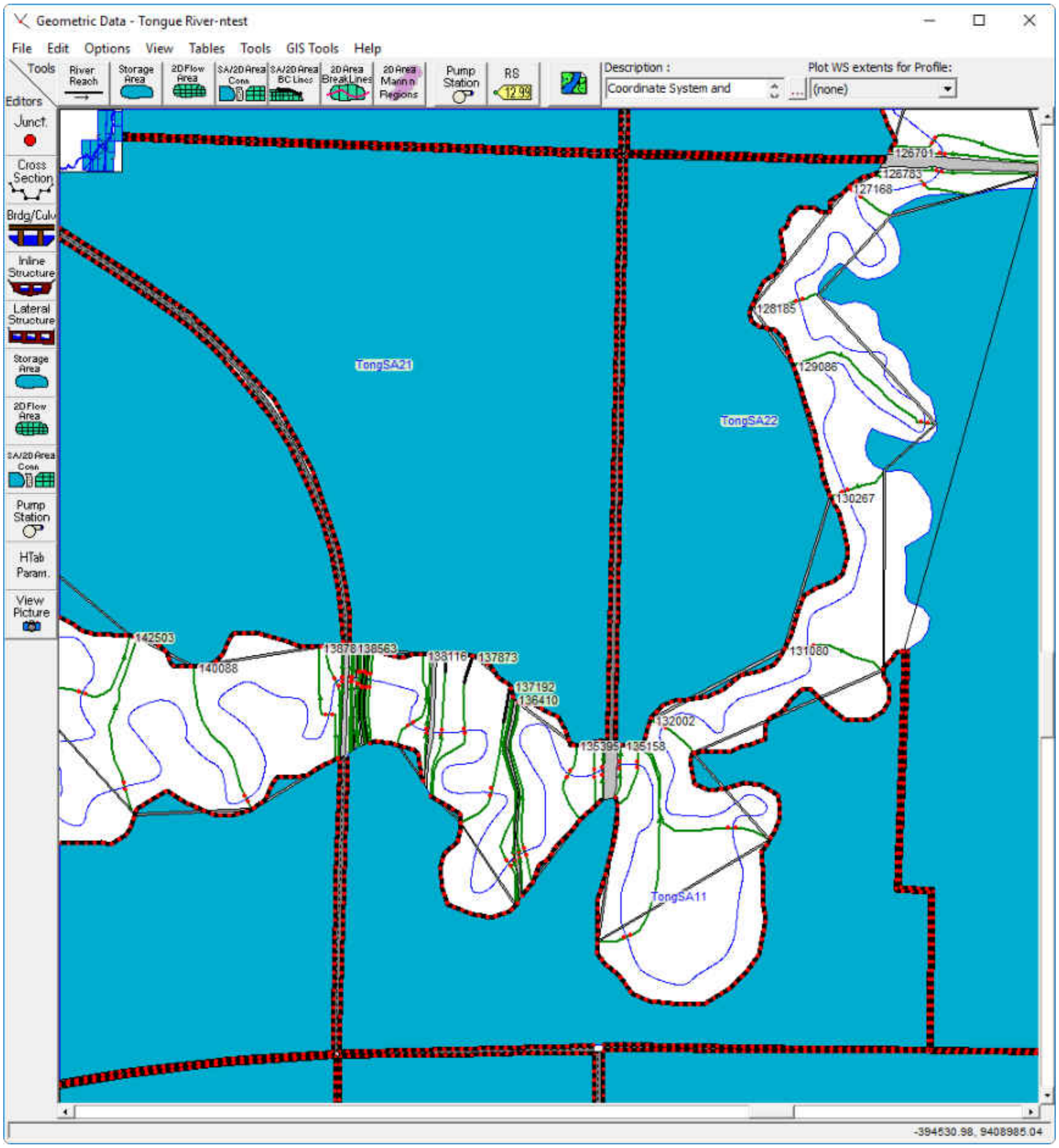

Figure 20. HEC-RAS components

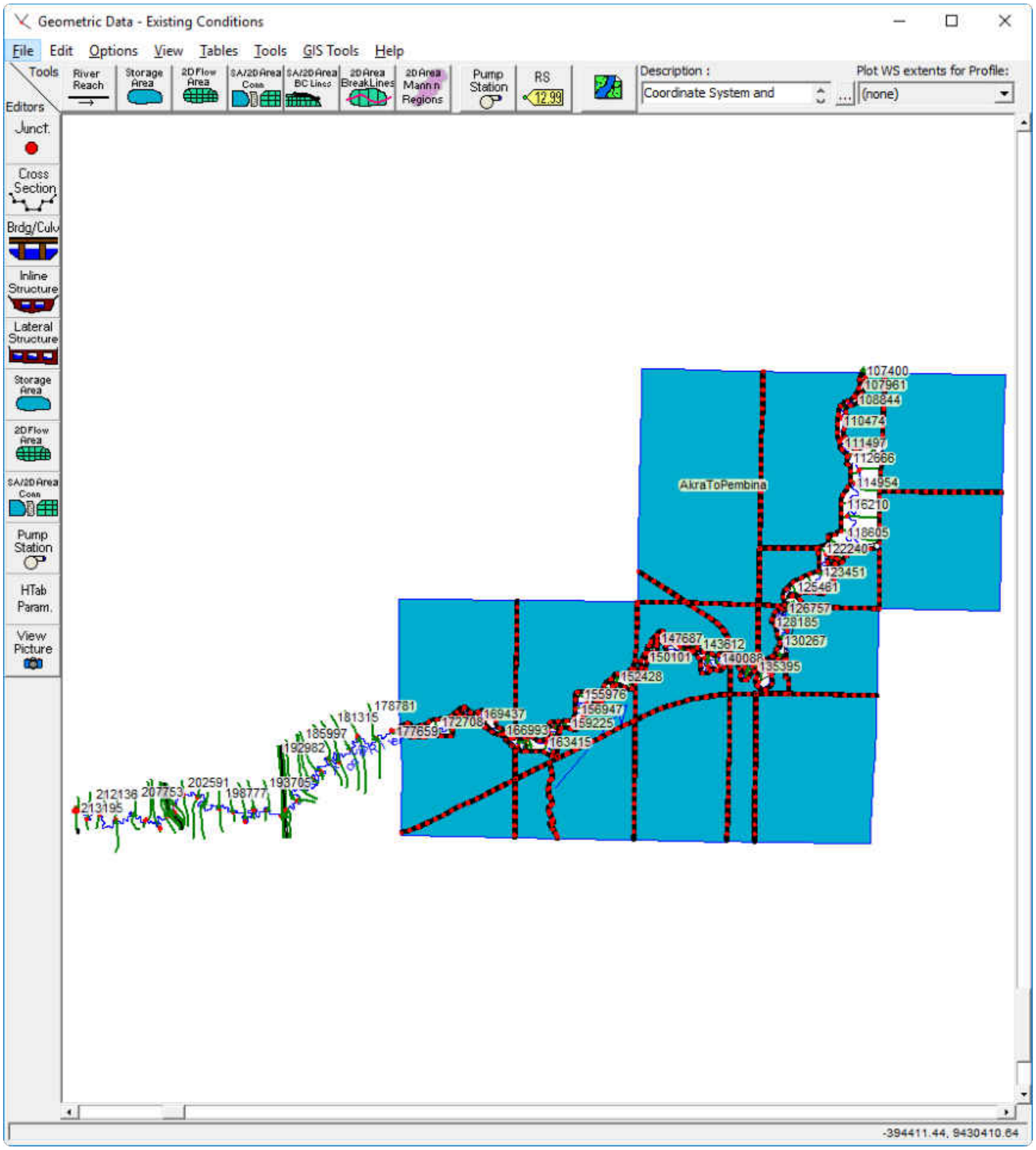

Figure 21. HEC-RAS Tongue River model layout

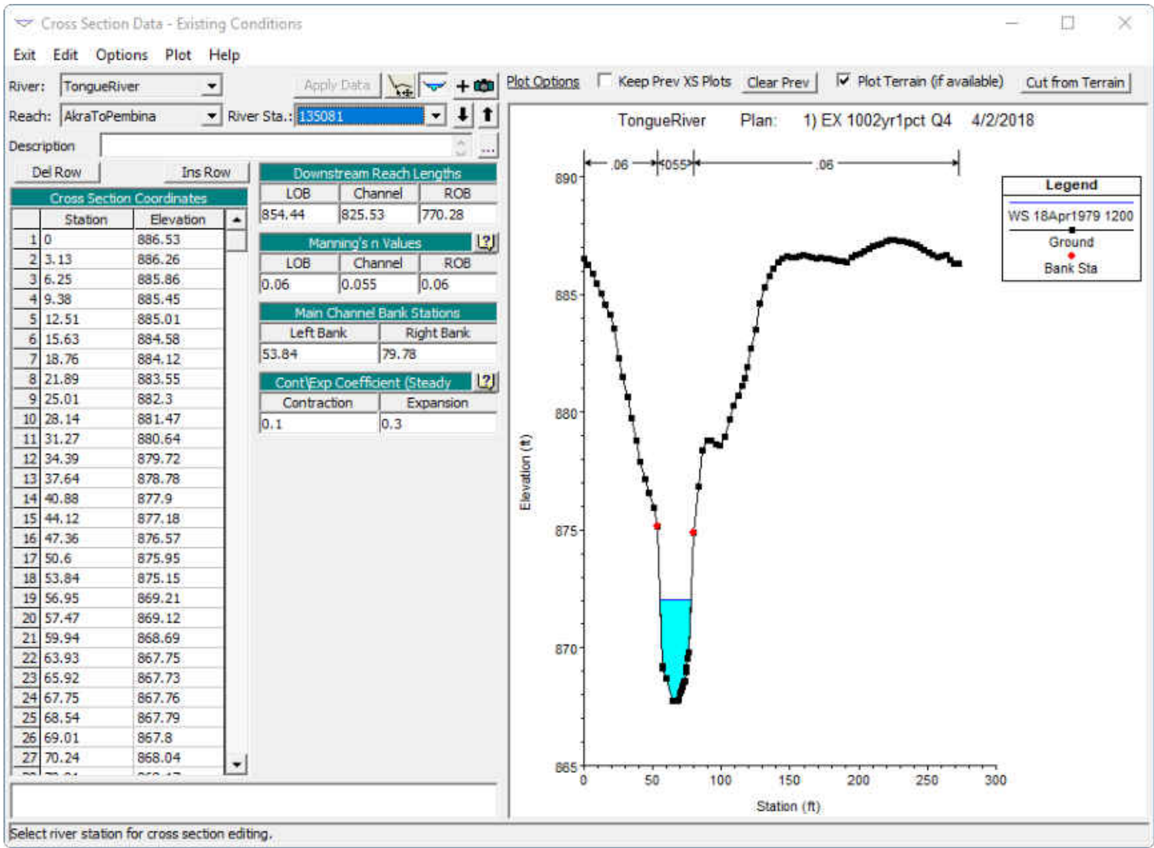

Figure 22. HEC-RAS cross section data editor

Cropscape data was used as a guide to analyze Manning's n values and more accurately represent the cross section overbanks in the model (USDA NASS, 2017). The area had previously been modeled with one overbank value that didn't represent the changes in wooded and crop areas.

Bridges and inline structures were updated with the data collected from survey and the NDDOT inspection reports, as discussed in Section 2.9. The bridge and culvert data editor window is shown in Figure 23.

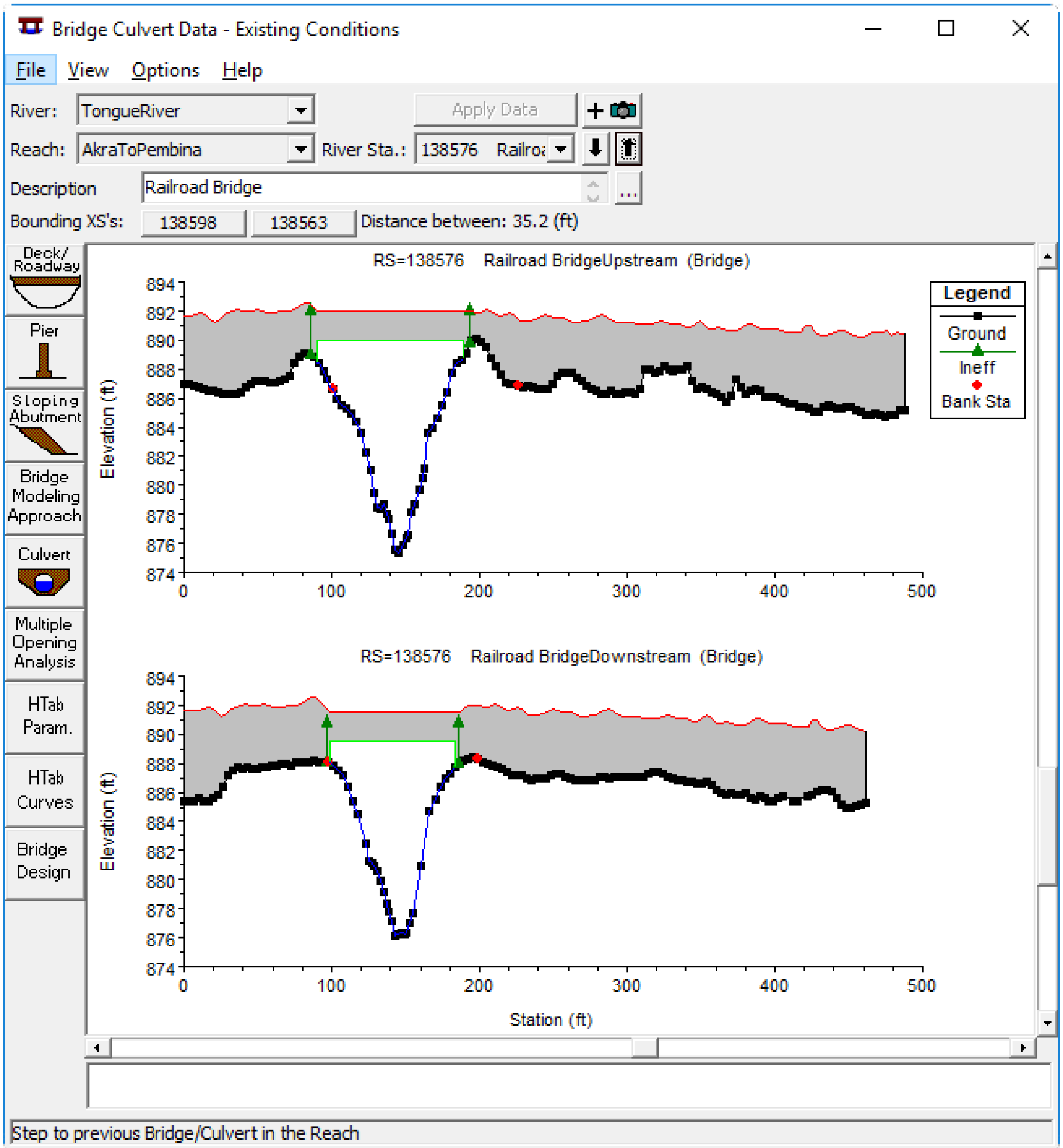

Figure 23. HEC-RAS bridge and culvert data editor

Storage areas were drawn to break at high ground, usually a section line road or natural levee. Storage areas in the model are represented as elevation-volume curves computed using the LiDAR surface in ArcGIS. These curves were cleaned up to reduce instability in the model.

Storage area connections were drawn along the high ground between the storage areas. The HEC-RAS storage area connection editing window is shown in Figure 24. Overland flooding was improved by adding additional culverts to the model. Using aerial imagery and LiDAR, culvert size and locations were estimated to provide reasonable overland conveyance. All culverts were assumed to be 36 inch CMP unless the aerial clearly showed it was a larger structure.

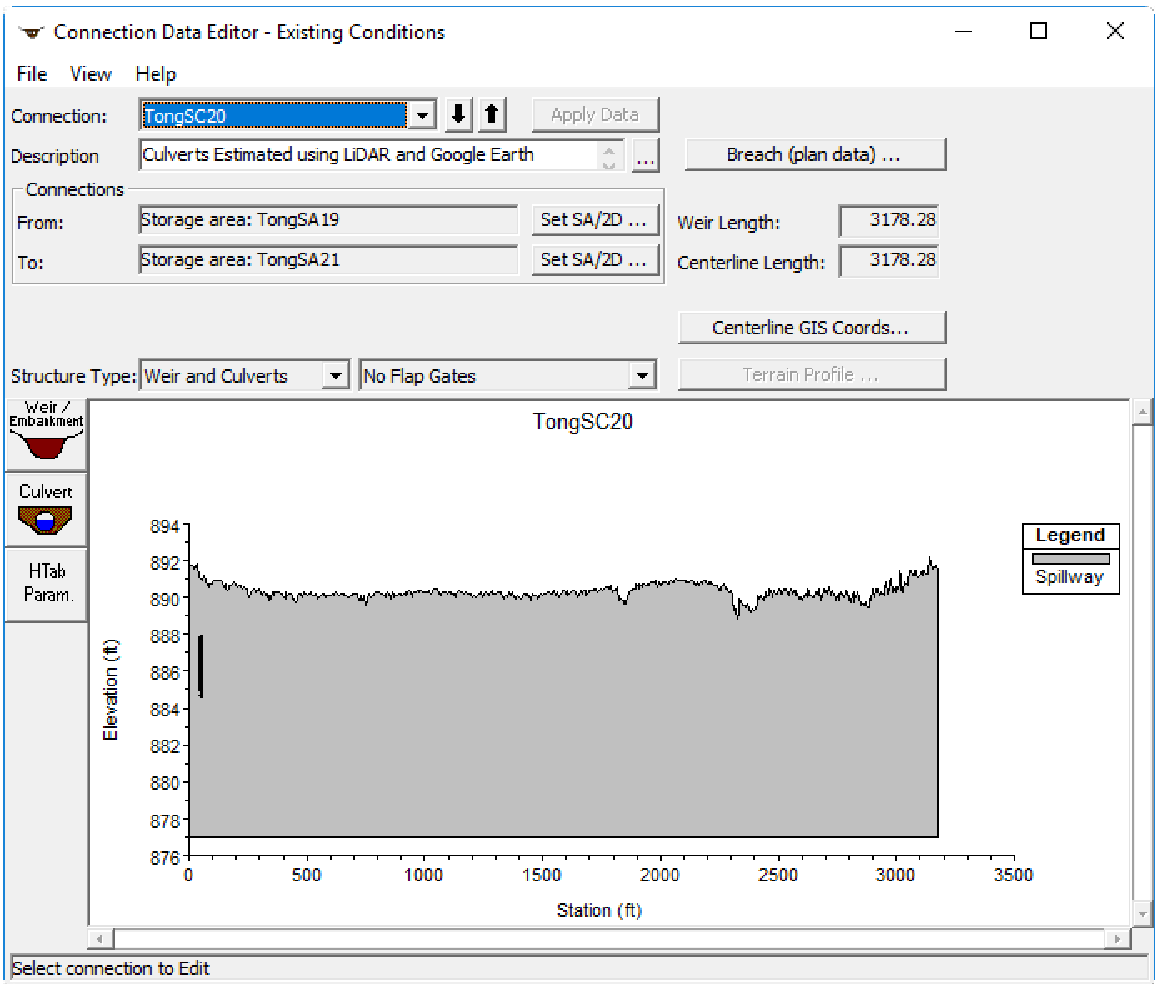

Figure 24. HEC-RAS storage area connection data editor

The lateral structure and storage area connection weir coefficients have been revised based on new guidance from the HEC report entitled "Combined 1D and 2D Modeling with HEC-RAS" (Brunner, Combined 1D and 2D Modeling with HEC-RAS, October 2014). Table 12 shows that the recommended weir coefficients range from 0.1 to 2.2. The inline structure weirs were exempt from this update and remain at higher values of 2.6. The widths of the weirs were also updated using the GIS measuring tool.

Table 12.
HEC-RAS weir coefficient guidance

Description of Weir	Description	Range of Weir Coefficients
Levee/Roadway – 3 ft. or higher above natural ground	Broad crested weir shape, flow over levee/road acts like weir flow.	1.5 to 2.2 (2.0 default)
Levee/Roadway – 1 to 3 ft. elevated above ground	Broad crested weir shape, flow over levee/road acts like weir flow, but becomes submerged easily.	1.0 to 2.0
Natural high ground barrier – 1 to 3 ft. high	Does not really act like a weir, but must flow over high ground to get into 2D area.	0.5 to 1.0
Non-elevated overbank terrain. Lateral structure not elevated above	Overland flow escaping the main river.	0.1 to 0.5

The model uses tabular hydraulic properties (HTabs) to create a family of rating curves for cross sections, bridges, and storage area connection calculations. Cross sections require number of points on the free flow curve and an increment, while storage area connections require a number of points on the free flow curve and a maximum elevation. It is important that these curves span the range of water surface elevations. The cross section points were increased from the default of 20 to 100, then the increment was decreased appropriately. The storage area connection maximum values were chosen to be approximately ten ft above the weir elevation.

3.3.2 Calibration

Preliminary model runs of the 2013 event used a Manning's n value of 0.04 in the channel, which is common for a natural stream in the Red River Valley. While the model results matched the discharge vs elevation rating curve at the USGS gage well for the bottom three ft of the channel, it was 1.5 ft too low at the peak. Google Earth has imagery from September 2013 that shows numerous trees and debris in the channel, as shown in Figure 25. The date means it is possible that they weren't there before the summer flood. Unfortunately, the earlier aerial imagery is not as good of quality. However, the August 2010 imagery shown in Figure 26 also appears to show debris in the channel. A higher Manning's n value of 0.055 was used, which is still within the acceptable range for channels (Chow, 1959).

A comparison of the USGS and model rating curves are shown in Figure 27. The USGS data is shown in black and grey. The dashed line is the calibrated rating curve and the field measurement data was added for verification. The 2009 and 2013 peaks were added as a reference. The 2013 calibrated model curve shown has a peak of 949.93 ft, which is 0.15 ft lower than the USGS recorded peak of 950.08 ft. Similarly, the 2009 event (not plotted) models a maximum elevation of 948.15 ft and the recorded peak was 948.42 ft for a difference of 0.27 ft. While the 2013 rating curve does not match as well at lower elevations, the addition of the bathymetry made a huge improvement over the USACE approximate model shown in purple.

Figure 25. Sept 14, 2013 Google Earth aerial imagery of Tongue River

Figure 26. Aug 10, 2010 Google Earth aerial imagery of Tongue River

The 2013 calibrated model geometry was combined with the maximum synthetic event of 500 years or a 0.2% annual chance event to plot the red line and compare to the USGS data. The 2013 event is the largest regulated event recorded, so the USGS line is extrapolated after that point and has potential for error. Figure 28 shows the 2013 USGS elevation hydrograph compared to the calibration model results.

Table 13 shows several of the high water marks discussed in Section 2.10 where high water mark could be approximated. The assumed date and time of the photo, approximate high water mark elevation, and the HEC-RAS elevation at that date and time are shown. While the USGS gage is calibrated well, the other locations aren't that close. However, the estimated high water mark elevations are all within 0.5 ft from the peak HEC-RAS water surface elevations. That was considered acceptable for this study since the date, time, and elevation of the high water marks are not precise.

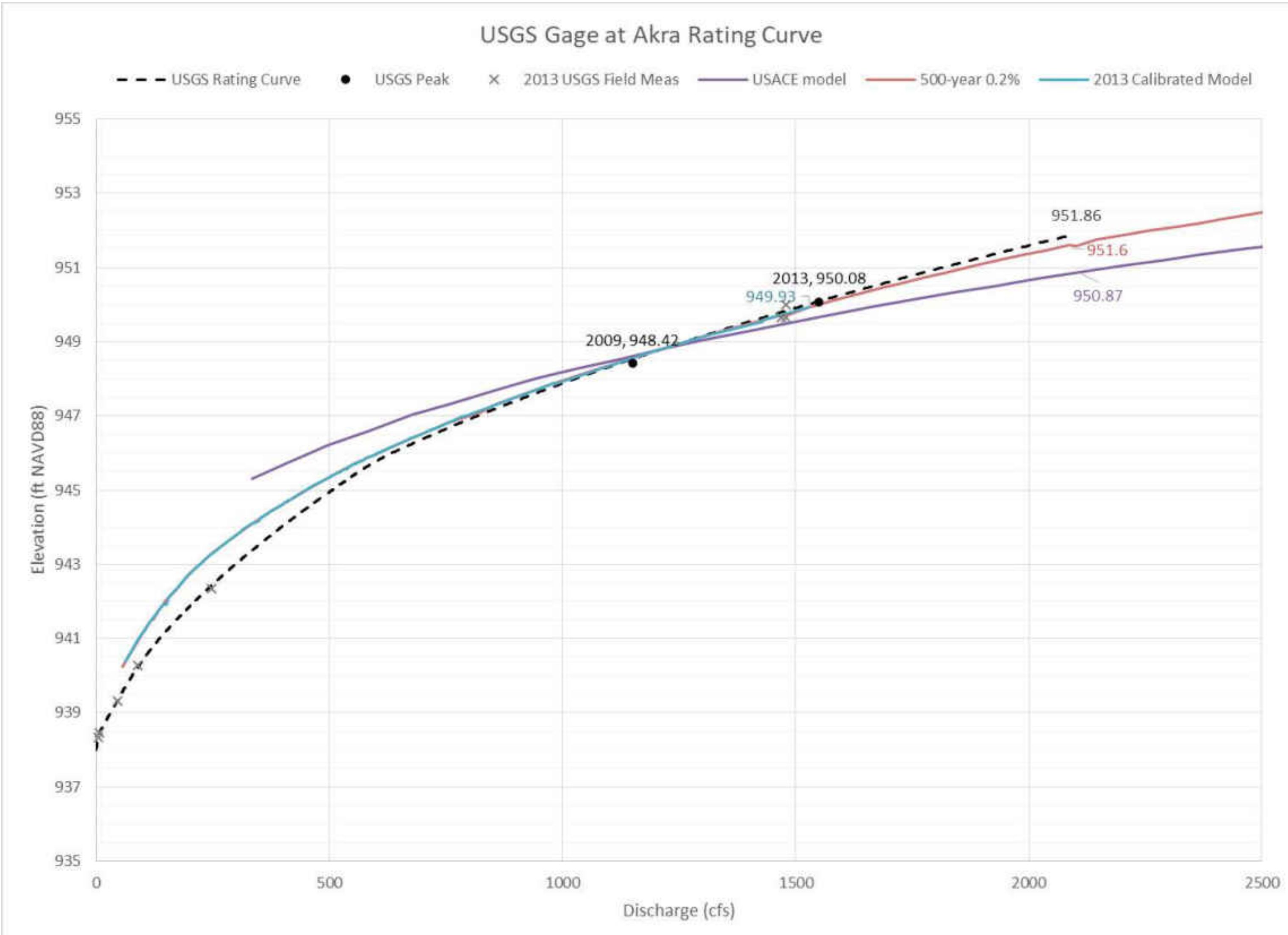

Figure 27. USGS gage at Akra vs model rating curves

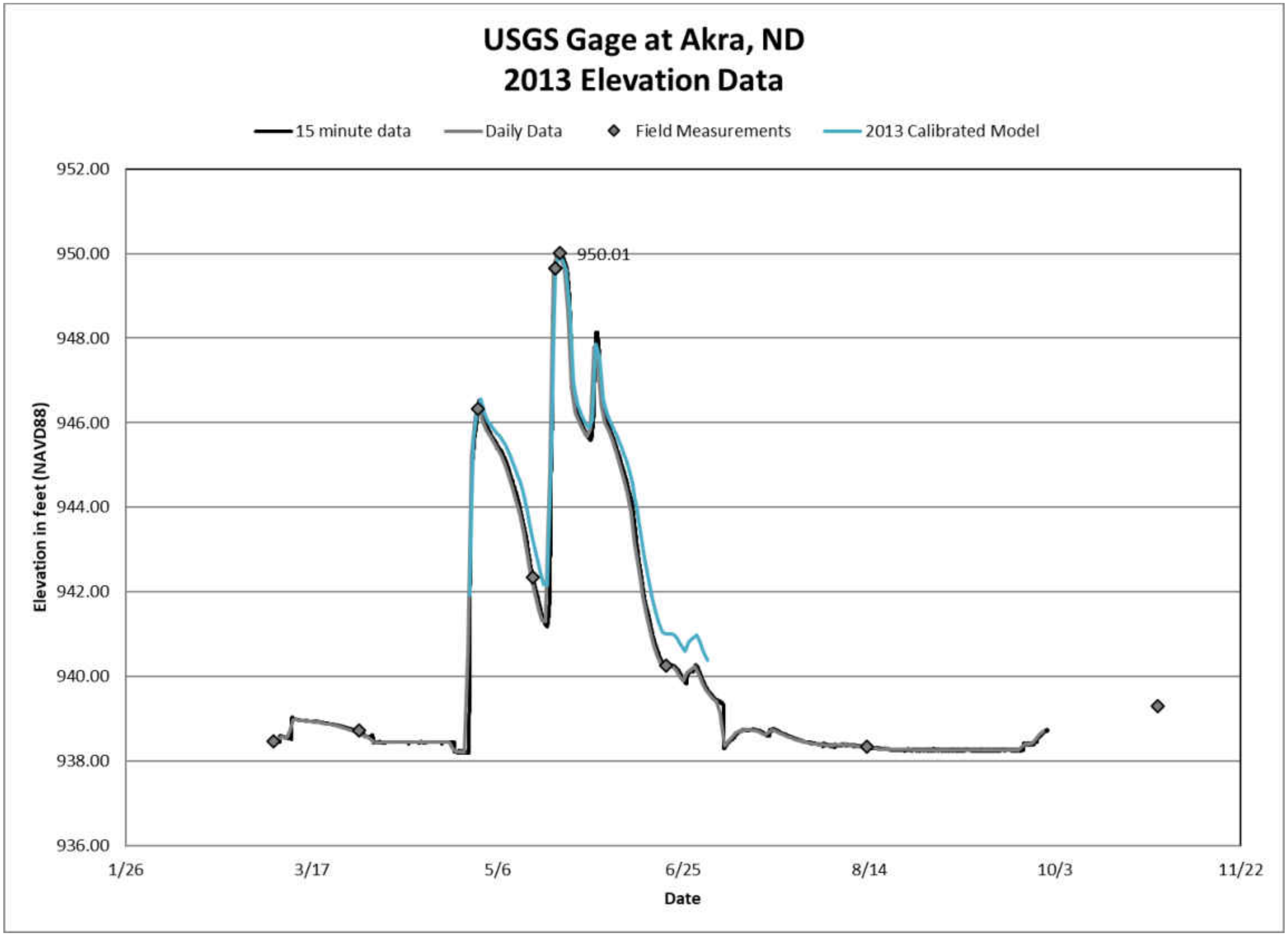

Figure 28. USGS gage at Akra vs model elevation hydrograph

Table 13.
2013 high water mark data with HEC-RAS results

Location	RAS location	Date/Time of HWM	Approx. HWM	HEC-RAS Result	Diff	HEC-RAS Peak time	HEC-RAS Peak	Diff
USGS Gage	213160	5/22/2013 12:00	950.08	949.93	-0.15	5/22/2013 12:00	949.93	-0.15
Pool Parking Lot	138116	5/26/2013 12:00	885.3	883.40	-1.90	5/22/2013 12:00	885.44	0.14
City Park	138039	5/26/2013 12:00	885.3	883.24	-2.06	5/22/2013 12:00	885.26	-0.04
305 W 1st Ave N (Moe)	137192	5/21/2013 12:00	884.75	883.16	-1.59	5/22/2013 12:00	884.12	-0.63
Sand Hill Lane (west of city)	TongSC32	5/26/2013 12:00	904.6	901.72	-2.88	5/23/2013 0:00	904.95	0.35

3.3.3 Sediment

The calibrated model was run with synthetic 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year hydrology. The most downstream site that was evaluated in Figure 2 is the residence at 201 Woodland Terrace, which had a cribwall installed as discussed in Section 0. The cross section at this site is plotted in Figure 29 with water surface elevations for all of the synthetic events. The banks are close to the 2-year event, commonly called the channel forming discharge. The sediment analysis that follows focuses on the 2- and 100-year events.

HEC-RAS sediment transport models used to require using quasi-unsteady flow data. The quasi-unsteady flow creates a stepped hydrograph as opposed to the smooth unsteady hydrograph. A smaller time-step of 1 hour can be used to limit the differences in the two hydrographs. A zoomed in view of the 2-year hydrographs is shown in Figure 30 to compare. The latest versions allow use of unsteady flow, which also allows for the use of storage areas and lateral structures so the flow can break out. However, it is noted that the quasi-unsteady model is less likely to have stability issues.

Water temperature data is also required with the flow data to reflect viscosity changes. Recorded data from the USGS gage was averaged for April and May dates and used to fill in the table with values ranging from 35-60 °F. If no value is added the model uses a default value of 55°F, which would be too high.

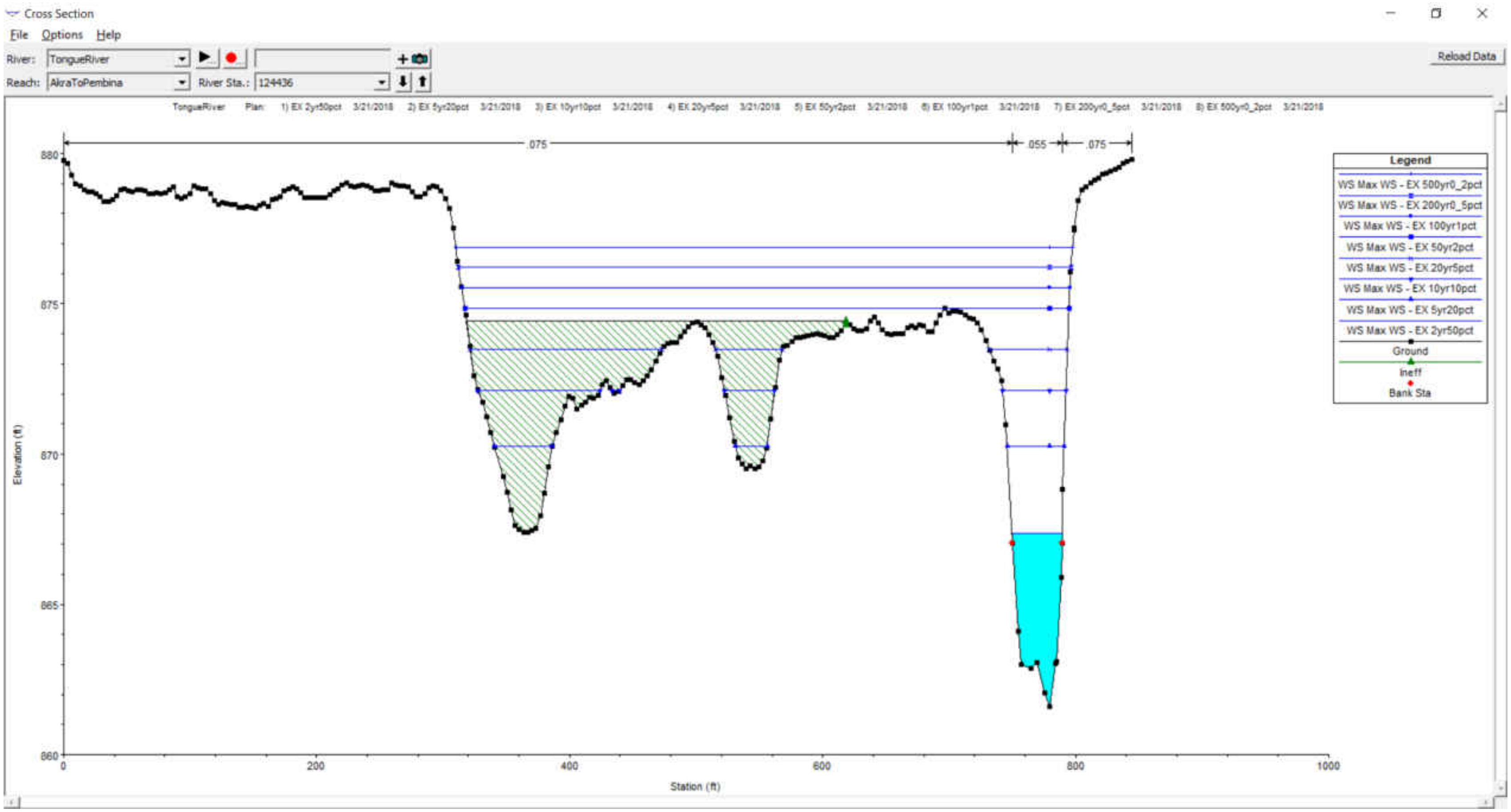

Figure 29. HEC-RAS synthetic event results at RS 124436 or 201 Woodland Terrace

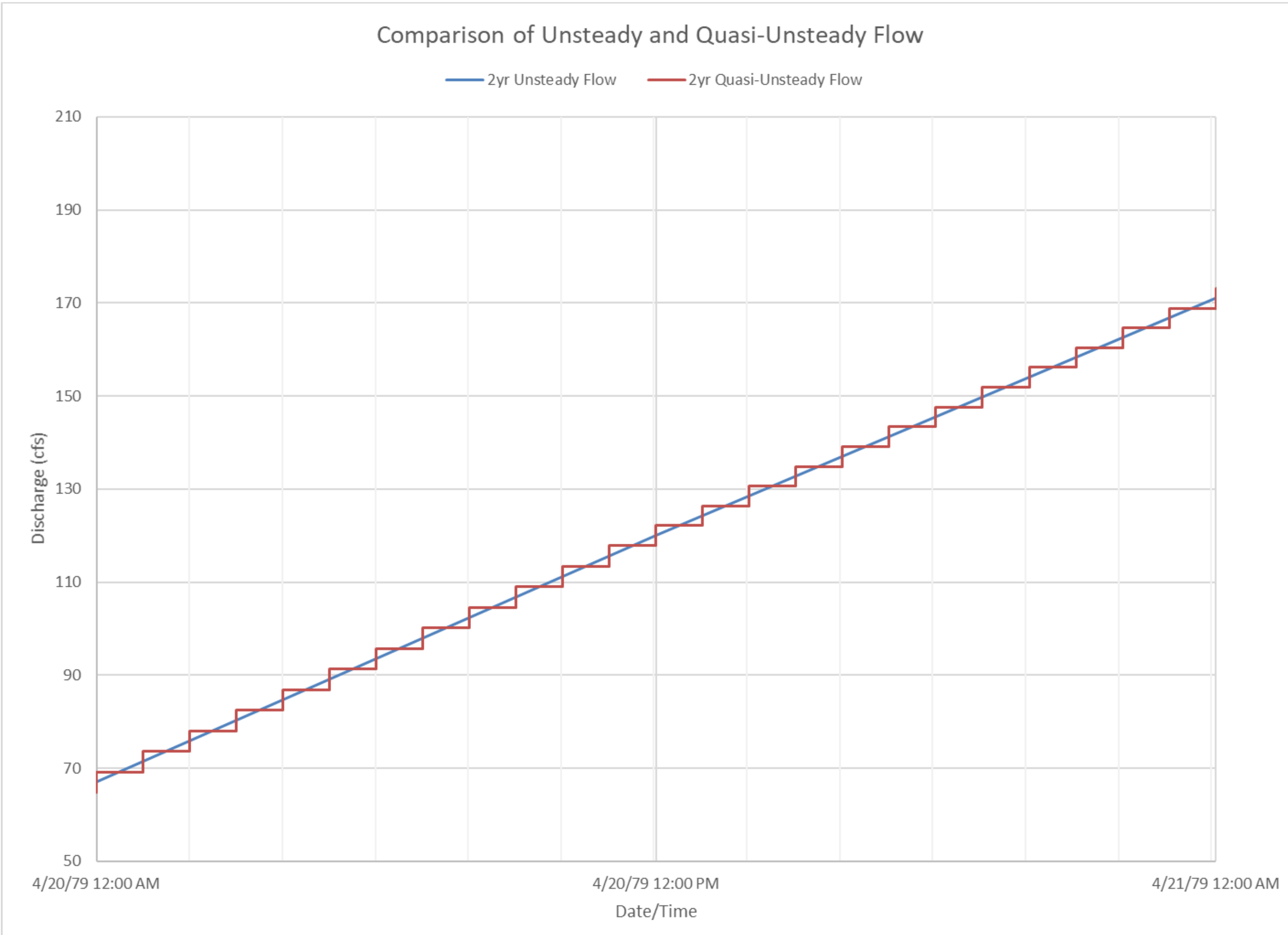

Figure 30. Comparison of unsteady and quasi-unsteady hydrographs

The Sediment Data window shown in Figure 31 contains three tabs: Initial conditions and transport parameters; boundary conditions; and BSTEM. The initial conditions tab requires erodibility limits and gradation data for each cross section, as well as a transport function, sorting method, and fall velocity method to be applied to all. The gradations analyzed in Section 3.1 were entered into HEC-RAS for the cross sections as shown in Table 14. Cross sections between those listed had interpolated gradations. The left and right stations need to be chosen carefully so as not to create unreasonable results.

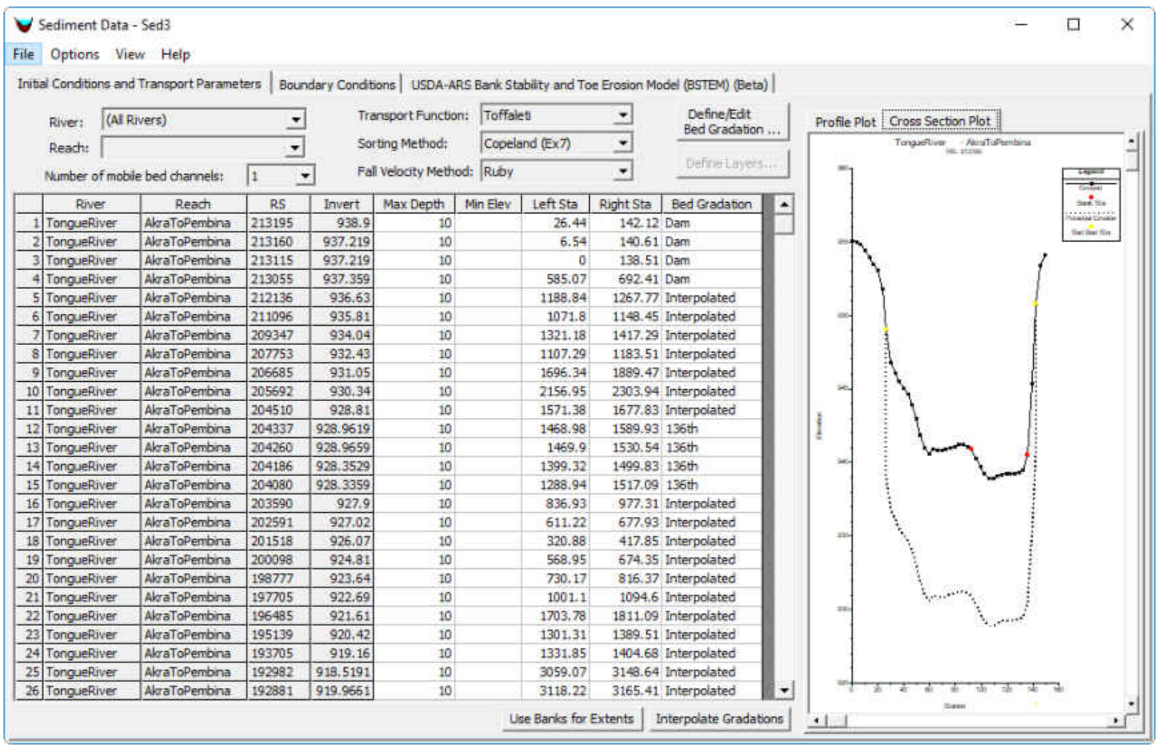

Figure 31. Sediment data window

For the boundary conditions tab, a rating curve comparing flow in cfs to the total load in tons/day is needed. The USGS suspended sediment data was only available for select dates in 2003 and 2004. The rating curve for each year is quite different, as shown in Figure 32. A simple linear trend line was created with all of the data for flows under 400 cfs and was entered into the model.

Table 14.
Gradation data distribution across HEC-RAS model cross sections

Location		Gradation Applied	
Description	**RS**	**Upstream**	**Downstream**
Dam	213195	213195	213055
136th	204223	204337	204080
138th	177305	177431	177179
140th	151236	151316	151137
1st Ave.	136396	136442	136355
Division Ave./Island Bridge	134256	135158	107400

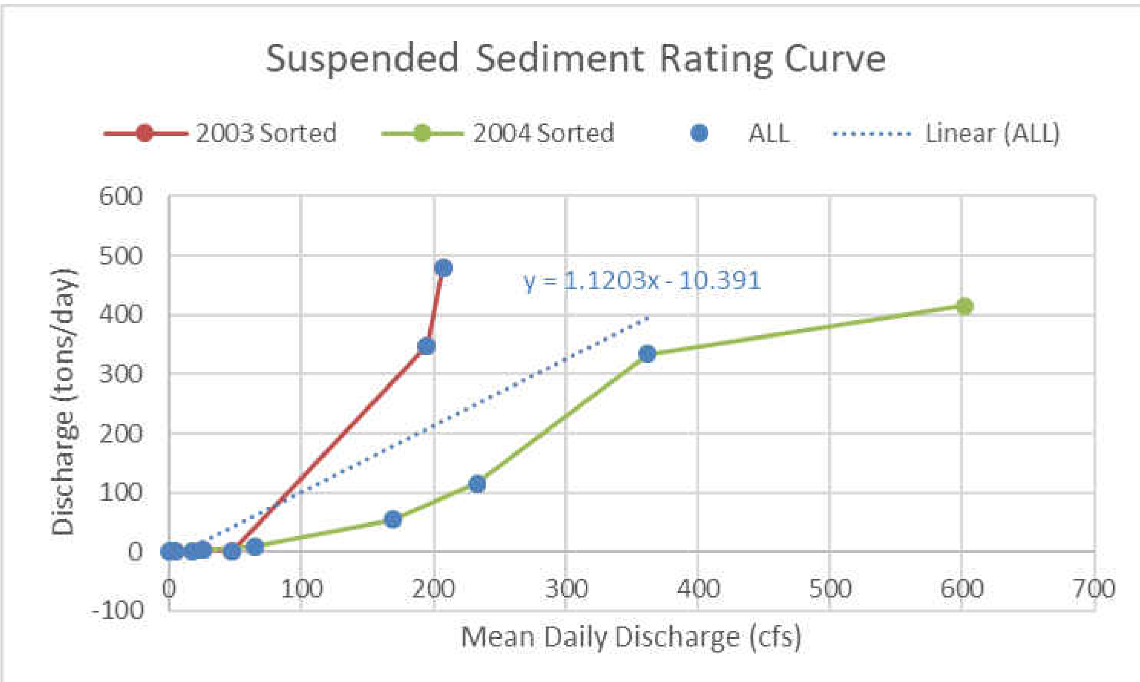

Figure 32. Suspended sediment rating curves

The various sediment transport functions were developed under different conditions, and therefore have a wide range of results. It is very important to choose the correct method. Table 15 from 1998 shows the acceptable range of various input values for each method compared to the Tongue River model. According to these results, the Laursen (field) and Toffaleti (field) functions would both be acceptable for this model.

Table 15.
Range of input values for sediment transport functions (Sam User's Manual, 1998)

Function	d (mm) Overall particle diameter	d_m (mm) Median particle diameter	s Sediment specific gravity	V (fps) Average channel velocity	D (ft) Channel depth	S (ft/ft) Energy gradient	W (ft) Channel width	T (°F) Water tempe rature
Tongue River Model	*<0.0625 - 4.0*	*~0.23*	*Est 2.65*	*2yr 1.9 100yr 3.8*	*100yr water depth ~15ft R 1-12 ft*	*~0.001*	*2yr 40 100yr 300*	*35-60*
Ackers-White (flume)	0.04-7.0	NA	1.0-2.7	0.07-7.1	0.01-1.4	0.00006-0.037	0.23-4.0	46-89
Englund-Hansen (flume)	NA	0.19-0.93	NA	0.65-6.34	0.19-1.33	0.000055-0.019	NA	45-93
Laursen (field)	NA	0.08-0.7	NA	0.068-7.8	0.67-54	0.0000021-0.0018	64-3640	32-93
Laursen (flume)	NA	0.011-29	NA	0.7-9.4	0.03-3.6	0.00025-0.025	0.25-6.6	46-83
Meyer-Peter Muller	0.4-29	NA	1.25-4.0	1.2-9.4	0.03-3.9	0.0004-0.02	0.5-6.6	NA
Toffaleti (field)	0.062-4.0	0.095-0.76	NA	0.7-7.8	0.07-56.7 (R)	0.000002-0.0011	63-3640	32-93
Toffaleti (flume)	0.062-4.0	0.45-0.91	NA	0.7-6.3	0.07-1.1 (R)	0.00014-0.019	0.8-8	40-93
Yang (field-sand)	0.15-1.7	NA	NA	0.8-6.4	0.04-50	0.000043-0.028	0.44-1750	32-94
Yang (field-gravel)	2.5-7.0	NA	NA	1.4-5.1	0.08-0.72	0.0012-0.029	0.44-1750	32-94

*R = Hydraulic Radius

However, the latest version of the model has a slightly different list of functions that are not broken down between field and flume scenarios. The Laursen method is replaced with the Laursen (Copeland) method, which generalizes the equation for gravel transport so it is applicable for graded beds. A brand-new Wilcock-Crowe method that analyzes armoring for graded beds with both sand and gravel was added. A new function combining Meyer-Peter Muller (MPM) and Toffaleti is also included. The Hydraulic reference manual does not describe these in detail like the others. Therefore, Laursen (Copeland), Toffaleti, and MPM Toffaleti are likely the best methods for this stream.

The sorting method is the next option to be chosen. Figure 33 shows schematics of the mixing layers for sorting and armoring methods. The left image applies to the Thomas and Copeland methods that include armoring, while the right image only has an active layer without cover. For the Tongue River, the armored option should be chosen. The Thomas method was originally developed for coarse systems and therefore tended to underpredict erosion on finer systems. Copeland's adjustments to the method will apply better to the silty clay loam in the area.

The last method to be chosen in this tab is the fall velocity calculations. Five options are available in the model: Rubey, Toffaleti, Van Rijn, Report 12, and Dietrich. The shape factor used in these equations is more important for soils classified as medium sands and larger. The Rubey method has been tested as adequate for silts, so it appears to be the best fit.

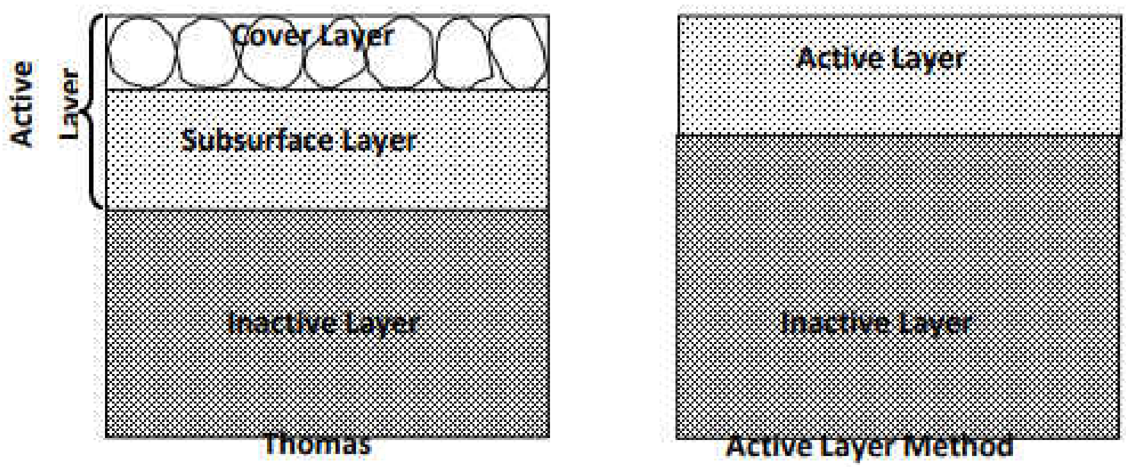

Figure 33. Thomas/Copeland armored layer vs active layer schematics of the mixing layers (Brunner, 2016)

Another factor to consider is the influence of cohesive soils. Most of the transport equations discussed previously were developed with sand and/or gravel data, and are less accurate for a silty clay loam. Fine sediments erode differently than sands or gravels with flocculation and electrostatic forces influencing erosion and deposition. The fines range from 14 to 41 percent in the six gradations, so it is important to adjust the calculations as necessary.

The sediment data window has the ability to set cohesive options. Three options are available: use selected transport functions for all grain sizes (default), use Krone/Partheniades for clay and silt size fractions, and use Krone/Partheniades (HEC 6T capacity method). If the default option is used for fine material the equations will extrapolate and calculate unreasonable results. The other two options produce similar results.

Using the Krone/Partheniades methods requires four additional parameters: critical shear for particle erosion, critical shear for mass erosion, and the erosion rates for each. The critical shear stress is the minimum amount of shear stress required to initiate soil particle motion. Below this critical shear stress no erosion occurs and above which erosion starts. If the bed shear gets greater it will begin to remove clods from the bed: mass erosion. The erosion rates describe the time it takes for these to occur, as shown in Figure 34. Many studies have recommended that these values be measured or calibrated because no correlation has been found that is reliable due to variance in environmental factors.

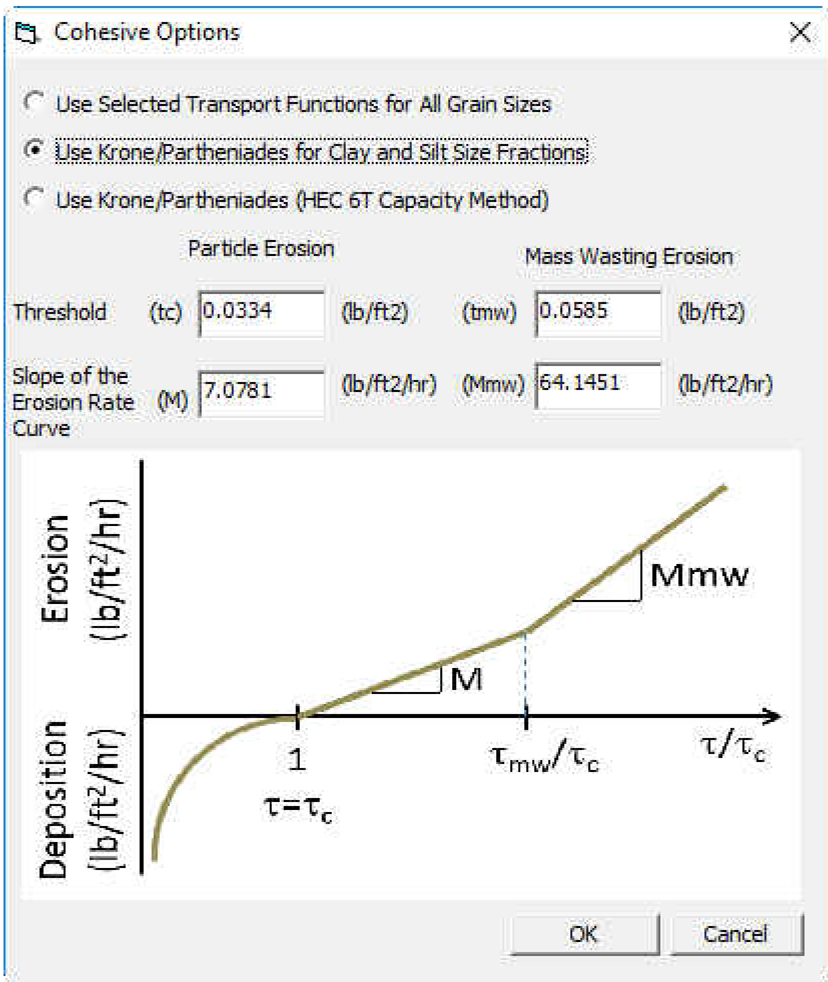

Figure 34. HEC-RAS Cohesive Sediment Options (Brunner, 2016)

As part of the Fargo-Moorhead Area Diversion project, twenty one soil samples were collected at various depths from twelve sites along or near the diversion (HMG, 2013). Undisturbed cohesive sediment samples were tested in the laboratory for erosive response to increasing velocity and shear stress using the Erosion Function Apparatus (EFA) method (Briaud, et al., 2011). The relationship between the erosion rate and shear stress for each sample was derived and used to calculate the critical shear stress (Anderson, 2012). A representative value of 2.75 Pa, or 0.0574 lb/ft^2 was ultimately chosen. While these tests are not on the Tongue River, they are in the Red River Valley.

Unfortunately the Red River Valley does not have any values determined for the erosion rate, mass wasting threshold, nor the mass wasting rate. These values were not able to be measured or calibrated for this study. Instead, similar studies were found in Maryland and the Chesapeake Bay area (Langland & Koerkle, 2014) (West Consultants, Inc., 2017). The four shear parameters from the studies are shown in Table 16.

Table 16.
Cohesive soil shear parameters for model

Study	Critical Shear Threshold (lb/ft2)	Erosion Rate (lb/ft2/hr)	Mass Wasting Threshold (lb/ft2)	Mass Wasting Rate (lb/ft2/hr)
(Langland & Koerkle, 2014)	0.0183	33.1	0.031*	134.3
(West Consultants,	0.0021	76.6792	0.0167	238.1479
Inc., 2017)	0.0334	7.0781	0.0585	64.1451

*Listed as 0.31 in 2014 report. The 2017 report clarified that 0.031 had been used in the model and the report was incorrect.

3.3.4 Sediment Sensitivity

The purpose of the Tongue River study is to compare various potential project impacts, so the parameters will stay the same across all options and a preferred option can be selected. The results do not need to precisely model actual erosion and deposition depths, since it is a relative comparison. However, various parameters were tested to find the most reasonable option.

While the calibration model was run without issue, when the sediment option was checked the unsteady flow computations reported an error in the geometry at the first bridge 204223. The model runs the quasi-unsteady flow regime without issue.

Working with the Hydrologic Engineering Center (HEC), it was discovered that there was a bug in the software preventing bridges to be run in unsteady flow sediment transport models. Due to the timeline of this thesis, the project moved forward using cross section “lids” in the place of bridges in the unsteady flow model. While a bridge in a model can only be defined by two cross sections, a lid can be added to any number of cross sections in a row and is commonly used to model long tunnels.

To test the use of lids vs bridges, the quasi-unsteady model was run with both options to compare. Figure 35 compares the 100-year event invert change results at the final time step from the two quasi-unsteady flow models as well as the unsteady flow model with lids. There is little difference between the two quasi-unsteady models, however the unsteady flow model is largely different from the quasi unsteady flow. This is especially apparent at the first bridge, which was eroding nearly 10 ft in the quasi-unsteady model. The unsteady flow model erodes 3.6 ft at the same location. HEC had also suggested the possibility of removing the bridges altogether. Figure 36 shows a comparison of the invert change at the final time step for the unsteady flow model with lids and without any crossings. There are minor differences in the profiles. It was chosen to move forward with the lidded cross section model.

The gradation testing did not include testing past the No. 200 sieve. The NRCS Web Soil Survey classified the area as silty clay loam (NRCS, 2017). The minimum and maximum clay percentages for each gradation were estimated using the USDA soil texture chart in Figure 37 (USDA, n.d.). Silty clay loam should have be 27-40% clay, 40-73% silt, and 0-20% sand. The gradation at the dam had a higher sand content at 67.5% and a mile downstream at 136th it was still at 48.8%. For the samples that couldn't classify as silty clay loam, the most suitable type of loam was chosen. The model was run with the original gradation, minimum clay specified and maximum clay specified. The clay specifications increased erosion at every cross section; at some locations unrealistically. It was decided to keep the gradations in the model as tested.

The cohesive properties were also tested in several models. Each of the three rows in Table 16 along with a fourth option that used the critical shear stress from the (HMG, 2013) study with the maximum values from the (West Consultants, Inc., 2017) study. The results varied across cross sections. For a stretch within town the invert changes ranged by 1.5 ft, however for most of the river the difference between the options was 0.01 or less. The West values were the most reasonable for the Tongue River study. The HMG critical shear stress value of 0.0574 lb/ft^2 was very close to the 0.0585 lb/ft^2 value used for the mass wasting threshold, so it was determined that the West maximum values would be best to use for the Tongue River model.

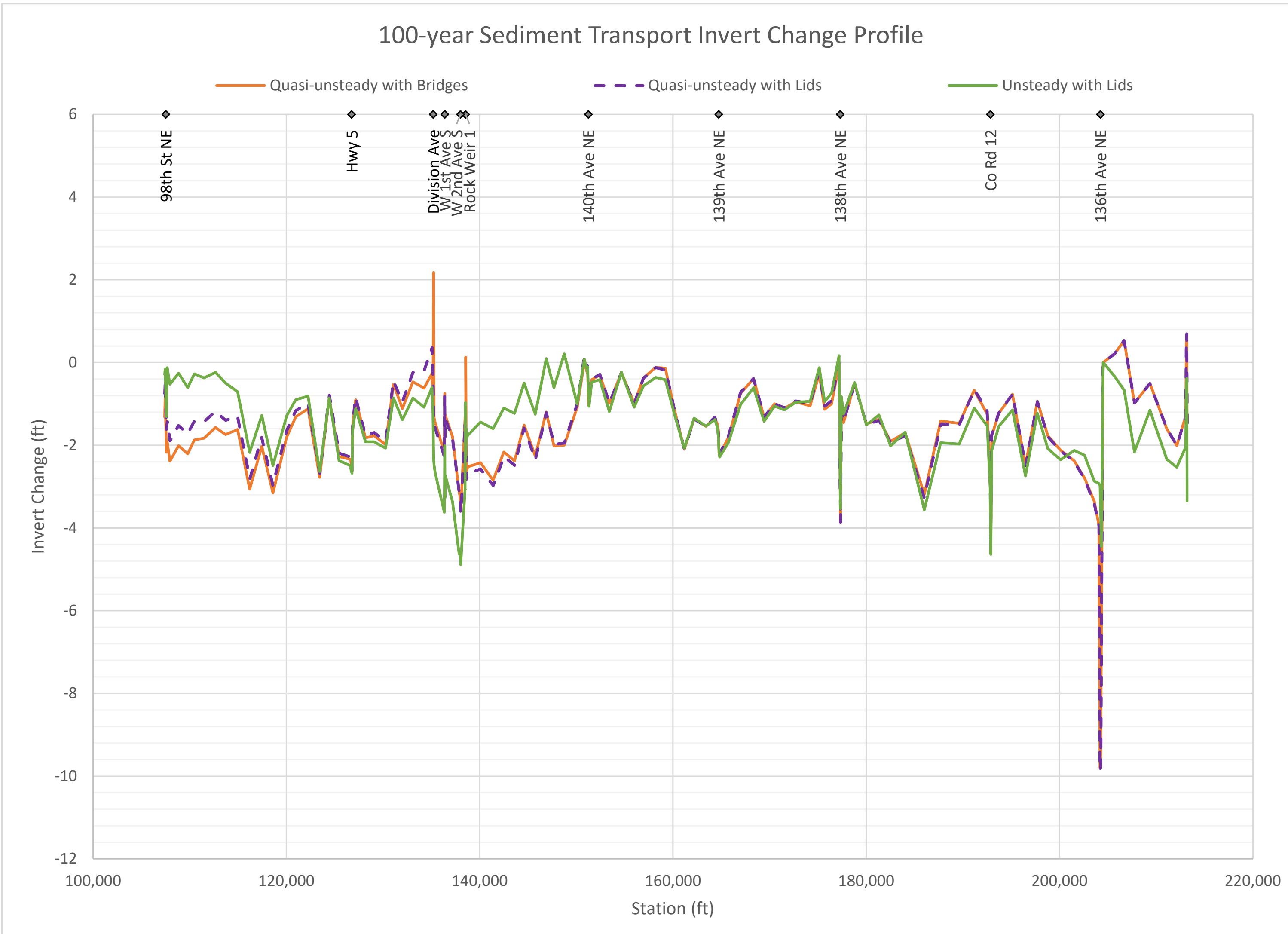

Figure 35. 100-year quasi-unsteady vs unsteady flow invert change results with bridges and lids

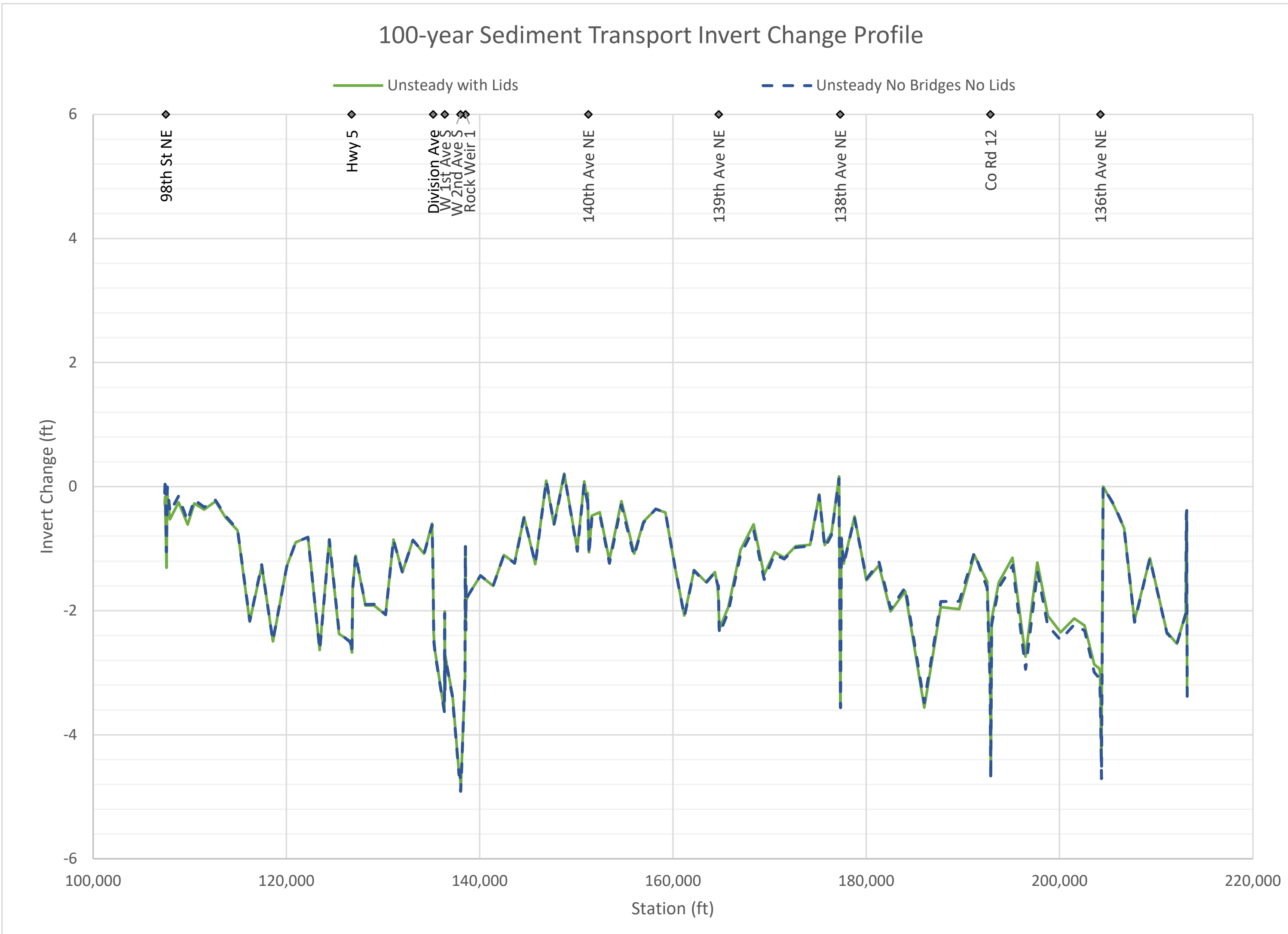

Figure 36. 100-year unsteady flow invert change results with lids and without crossings

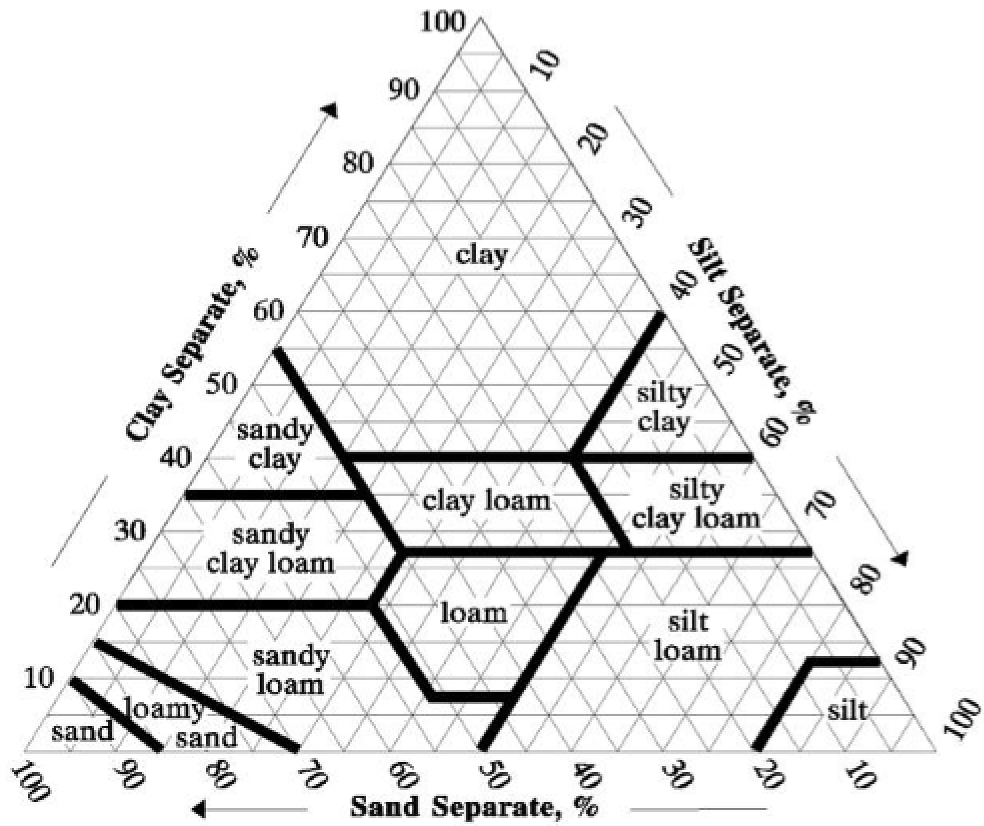

Figure 37. USDA soil texture chart (USDA, n.d.)

As determined in Section 3.3.3, Laursen (Copeland), Toffaleti, and MPM Toffaleti are likely the best methods for this stream. These three transport functions were tested with the quasi-unsteady flow model and the invert change at the final time steps are compared at all cross sections in Figure 38. As discussed previously, the unsteady flow results are more reasonable at the peaks, however the quasi-unsteady results have the opportunity to show differences more clearly with more extreme results. The Toffaleti and MPM-Toffaleti results are very similar, as expected. However, the MPM-Toffaleti results are slightly larger at the peaks. The Laursen (Copeland) method generally increases the erosion across the entire reach, and also increases the deposition at the large peaks. The Toffaleti method was chosen for the Tongue River modeling.

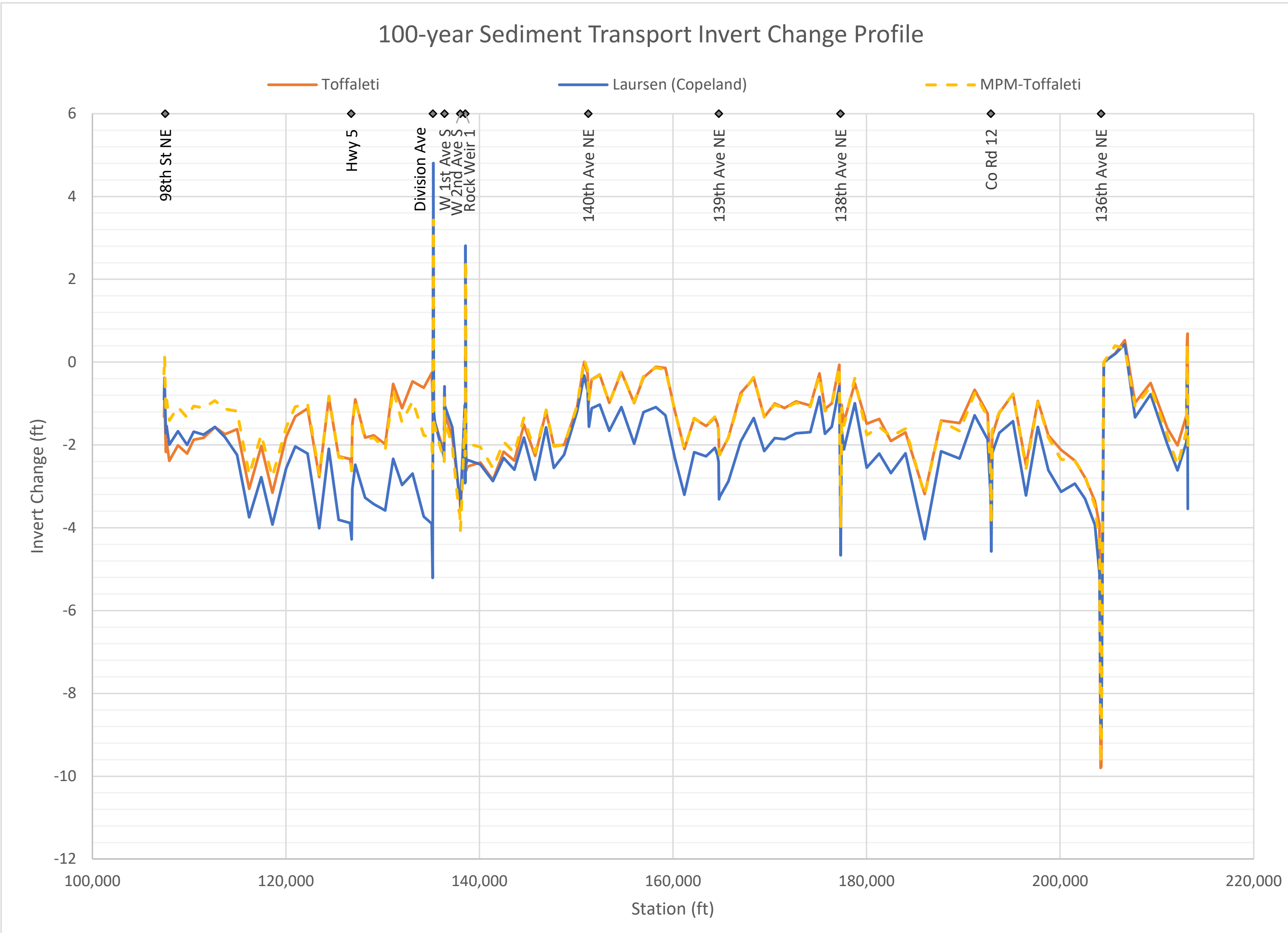

Figure 38. 100-year quasi unsteady flow invert change results for various transport functions

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3.3.5 Proposed Project Alternatives

Sensitivity runs were created to test the hypotheses with three weir locations: immediately downstream of Site 6 in Figure 2 near Woodland Terrace, immediately downstream of Site 8 near Park Street, and immediately downstream of the Division Ave bridge, as shown in Figure 39. Site 6 was chosen because it is the most downstream site identified by the Red River Riparian Project and it had fairly severe erosion. A tree slumped into the river overnight, reducing the distance between the house and riverbank considerably. However, a project has been implemented at this location as well as Site 7. Therefore, site 8 was chosen as the farthest downstream that hasn't been remediated. The most dangerous erosion was at two neighboring homes between 1st and 2nd Ave that had foundations exposed. The Division Ave weir location was chosen to provide greater stability to this area. Those homes have been removed, but there are still sites of concern in this area. For simplicity and model stability, a single weir was modeled at each location rather than a series of stepped weirs. The weirs were also run at various heights, as shown in Table 17.

Table 17.
Weir sensitivity run parameters

Location		**Elevations**			**Weir heights (ft)**		
Name	**RS**	**Invert**	**banks**		**3**	**4**	**5**
Division Ave	135119	868.44	874.62	874.31	871.44	872.44	873.44
Park Street	131067	865.71	873.25	873.59	868.71	869.71	870.71
Woodland Terrace	123986	861.25	866.76	866.84	864.25	865.25	866.25

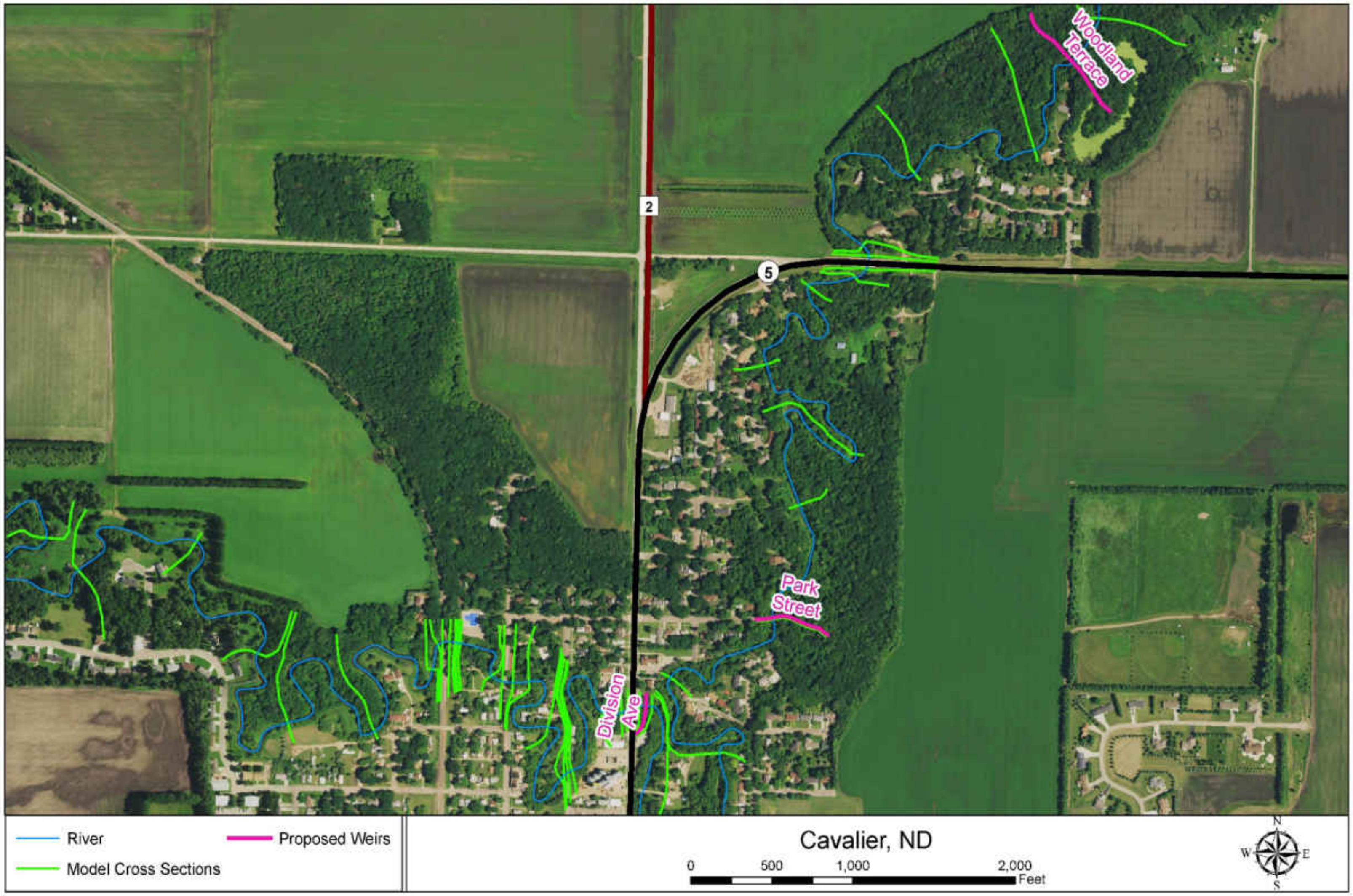

Figure 39. Potential stepped rock weir locations

CHAPTER 4. DISCUSSION

4.1 Results

Three potential weir locations were identified in Section 3.3.5: immediately downstream of Site 6 in Figure 2 near Woodland Terrace, immediately downstream of Site 8 near Park Street, and immediately downstream of the Division Ave bridge. Each weir was modeled at 3, 4, and 5 ft heights.

Figure 40 through Figure 42 show the river profile and invert change results for Division Ave, Park St, and Woodland Terrace, respectively. Each plot uses a black line to represent existing conditions, then increasing shades of a color are used to depict the 3, 4, and 5 ft weir heights.

Figure 43 shows the 100-year channel invert changes within town for a 5 ft weir at Division Ave, Park Street, or Woodland Ave. The Woodland Terrace weir causes the largest scour hole downstream of the weir and it takes several miles for the effects to dissipate. The downstream effects of the Park Street and Division Ave weirs both dissipate near Highway 5. The Woodland Terrace weir decreased the erosion upstream to Division Ave, where the differences become much smaller, then completely disappear at the existing rock weirs. The Park Street weir behaves similarly. The Division Ave weir provides more benefit than the others between Division Ave and the existing rock weirs, however it is a much shorter distance of benefit than the other two.

Figure 44 shows the invert change as a percent difference from the existing conditions model for each of the 5 ft weir locations. While the Division Ave and Woodland Terrace weirs cause erosion increases of up to 360 and 215%, respectively, the Park Street weir stays well within the 100% lines and shows considerable improvement between the weir and Division Avenue, where a majority of the remaining problem sites identified in Figure 2 are located.

Additionally, the problem sites identified between the Park Street and Woodland Terrace weirs have already had projects completed to protect their individual sites. The effects of both the Division Ave and Park Street weirs dissipate near Hwy 5, which means they would not adversely affect the two projects in place.

Figure 45 through Figure 50 show the cumulative mass change over time for the 2- and 100-year events at the cross sections with the most erosion for each five foot weir option. The lighter green lines illustrate the total mass change, while the remaining lines are broken down by sediment sizes. All three weirs use the same "Island" gradation in the model. The majority of the mass change in these models is class CM, coarse silt. However, the gradations weren't tested past the No. 200 sieve, so this volume is all of the silt and clay combined. The dark red line next on the graph is VFS, very fine sand. The graphs also show some FS, fine sand, being moved.

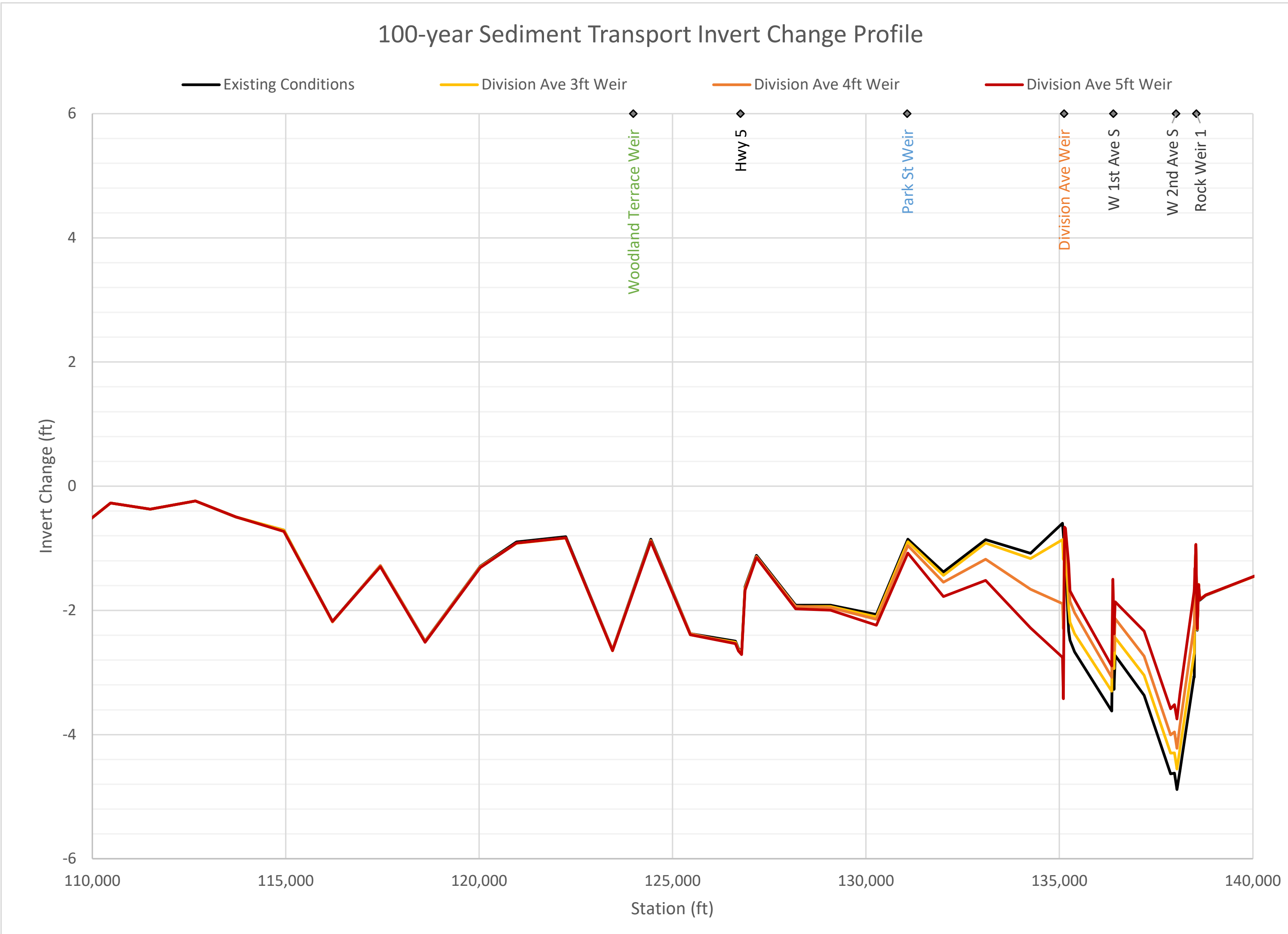

Figure 40. 100-year unsteady flow invert change results with Division Ave weirs at 3, 4, and 5 ft

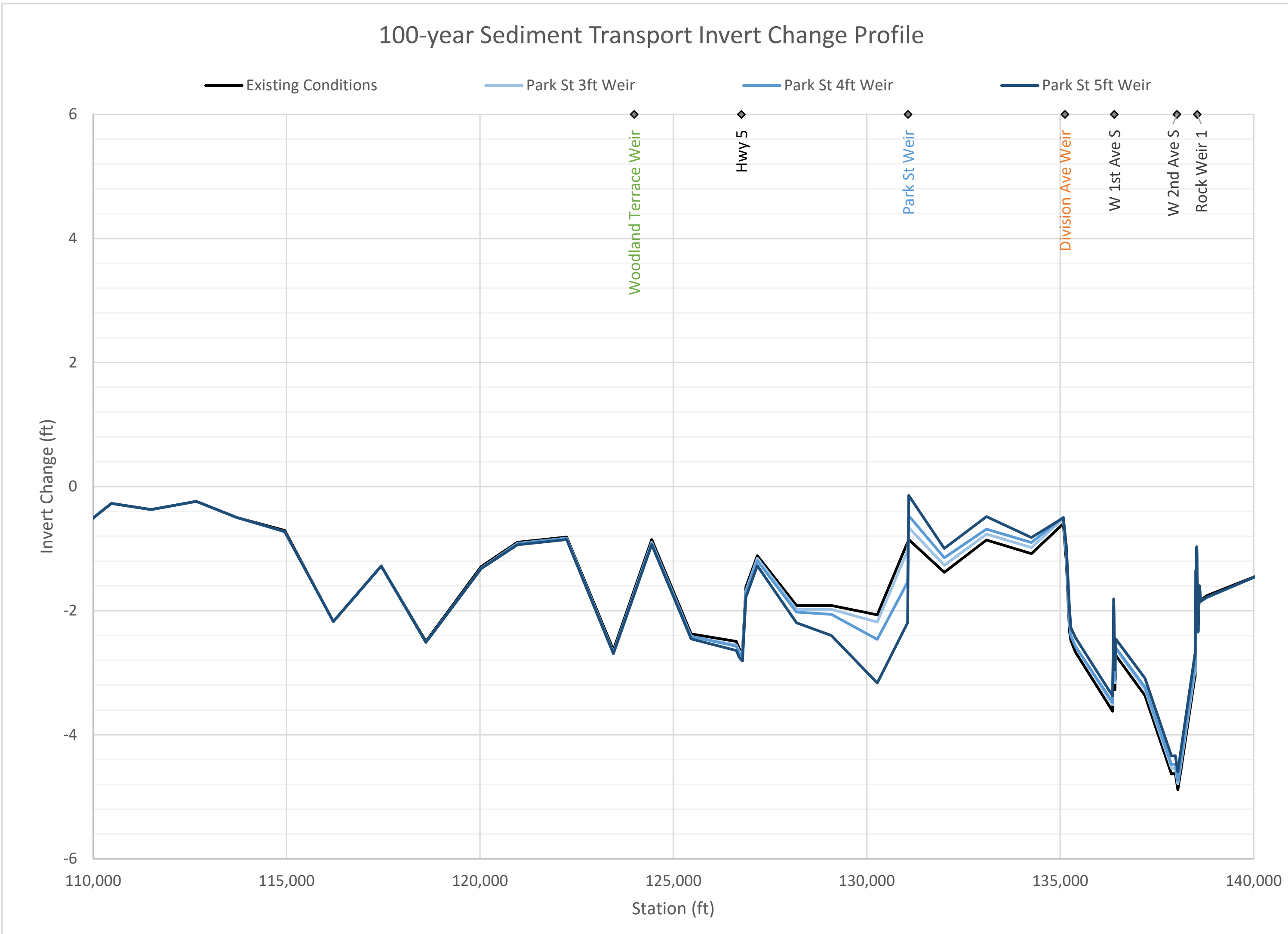

Figure 41. 100-year unsteady flow invert change results with Park Street weirs at 3, 4, and 5 ft

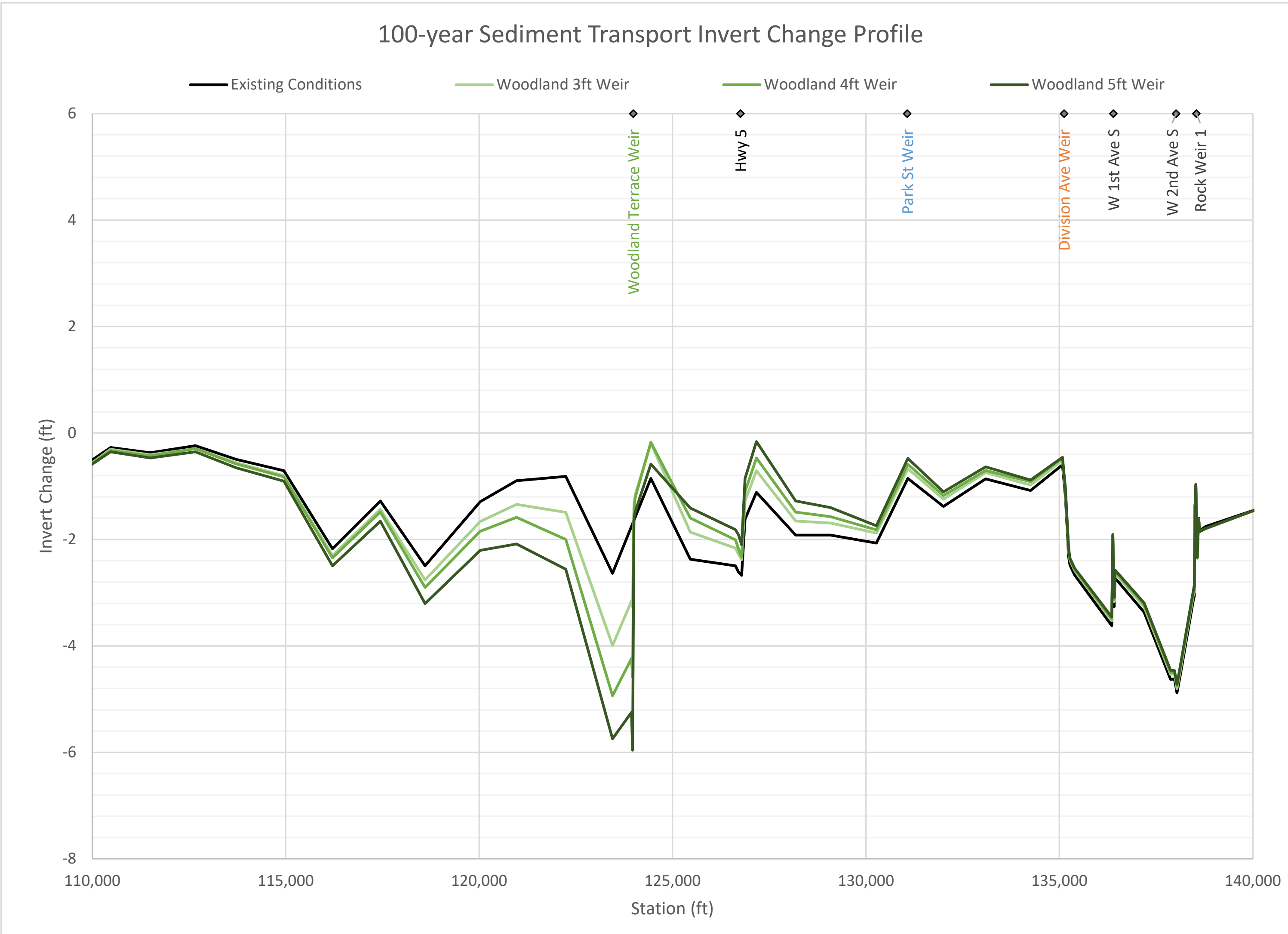

Figure 42. 100-year unsteady flow invert change results with Woodland Terrace weirs at 3, 4, and 5 ft

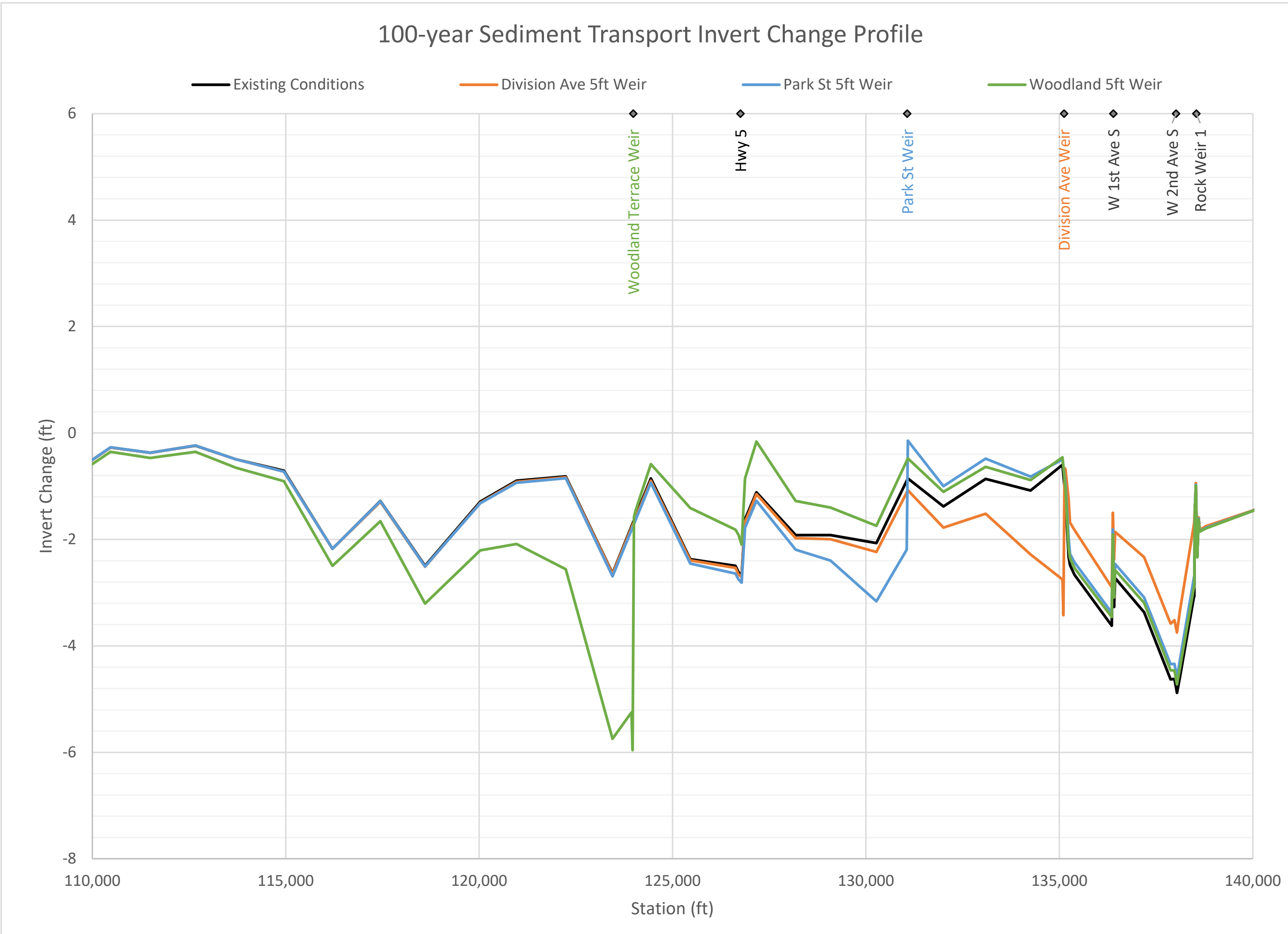

Figure 43. 100-year unsteady flow invert change results with Division Ave, Park St, and Woodland Terrace weirs at 5 ft

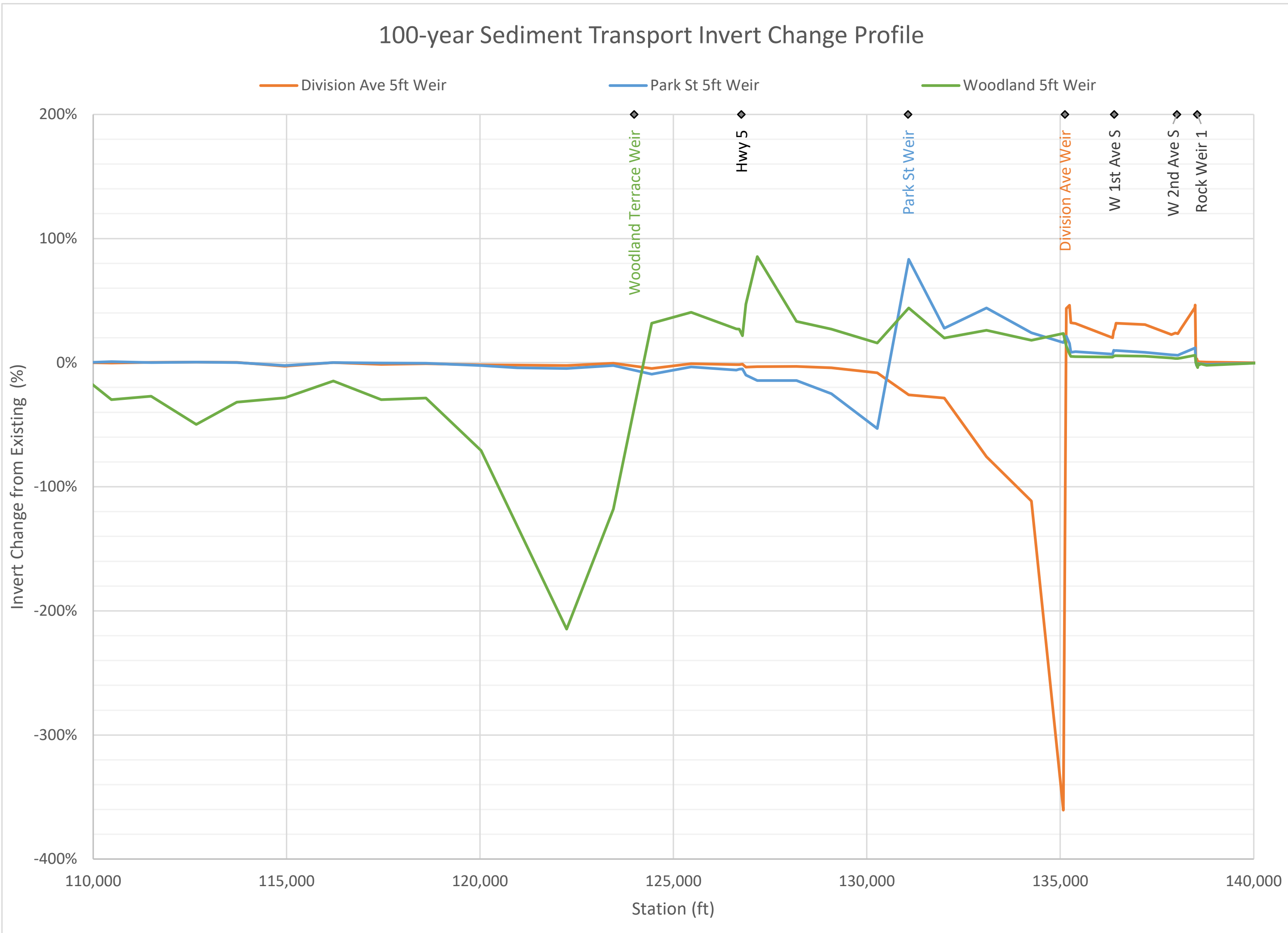

Figure 44. 100-year unsteady flow invert percent change results with Division Ave, Park St, and Woodland Terrace weirs at 5 ft

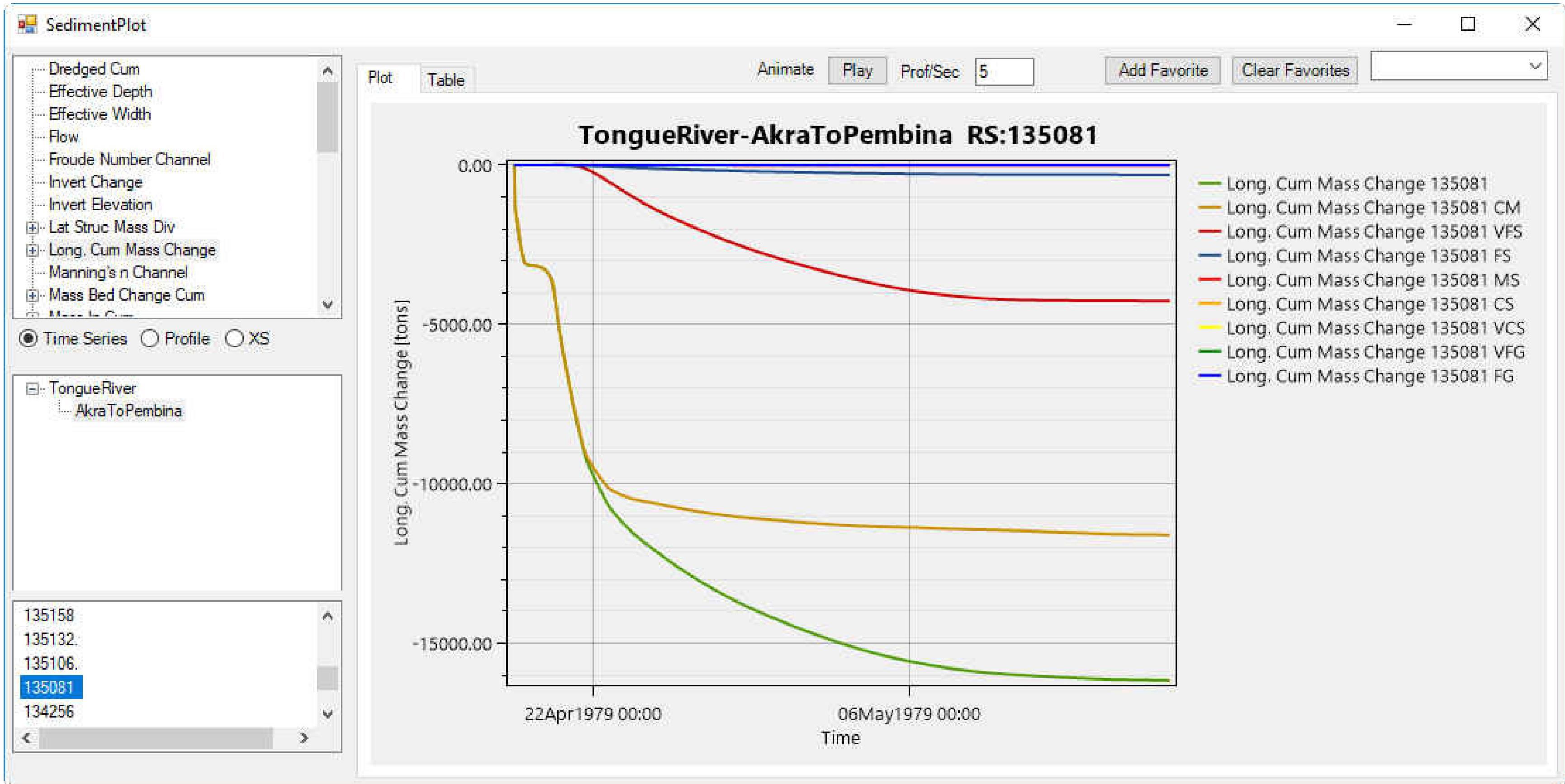

Figure 45. HEC-RAS 2-year sediment cumulative mass change at maximum erosion cross section for Division Ave 5ft weir model

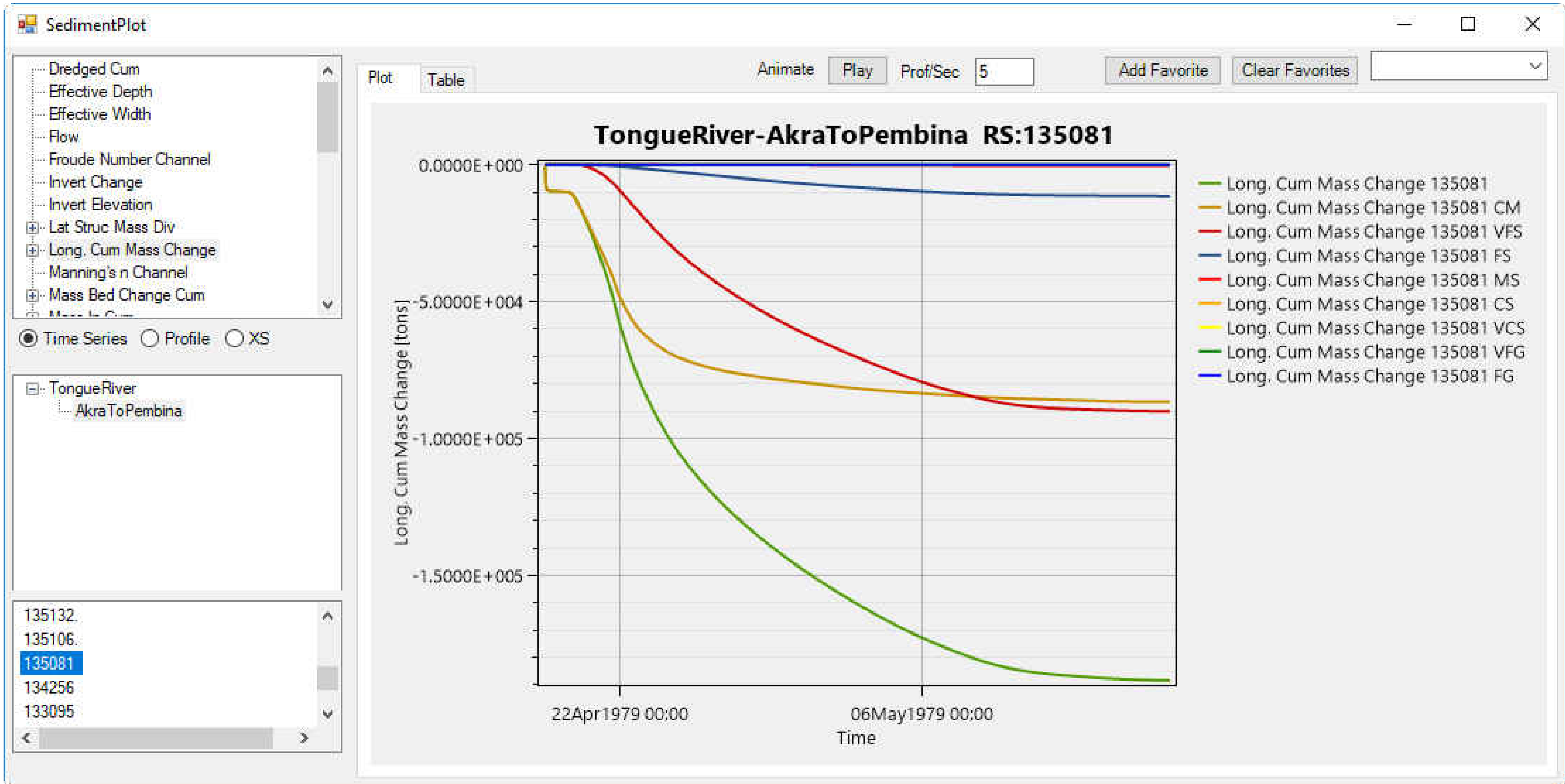

Figure 46. HEC-RAS 100-year sediment cumulative mass change at maximum erosion cross section for Division Ave 5ft weir model

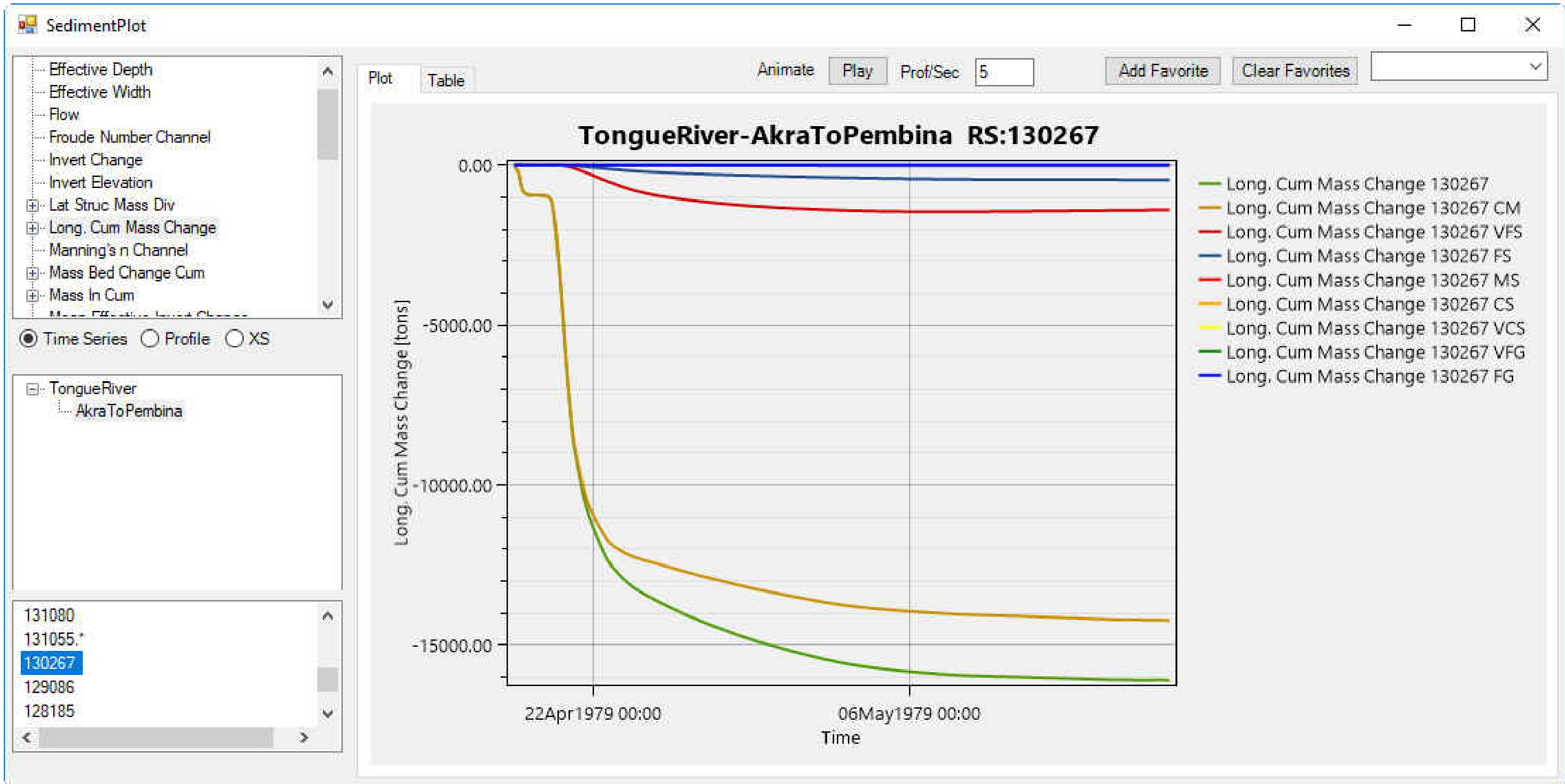

Figure 47. HEC-RAS 2-year sediment cumulative mass change at maximum erosion cross section for Park St 5ft weir model

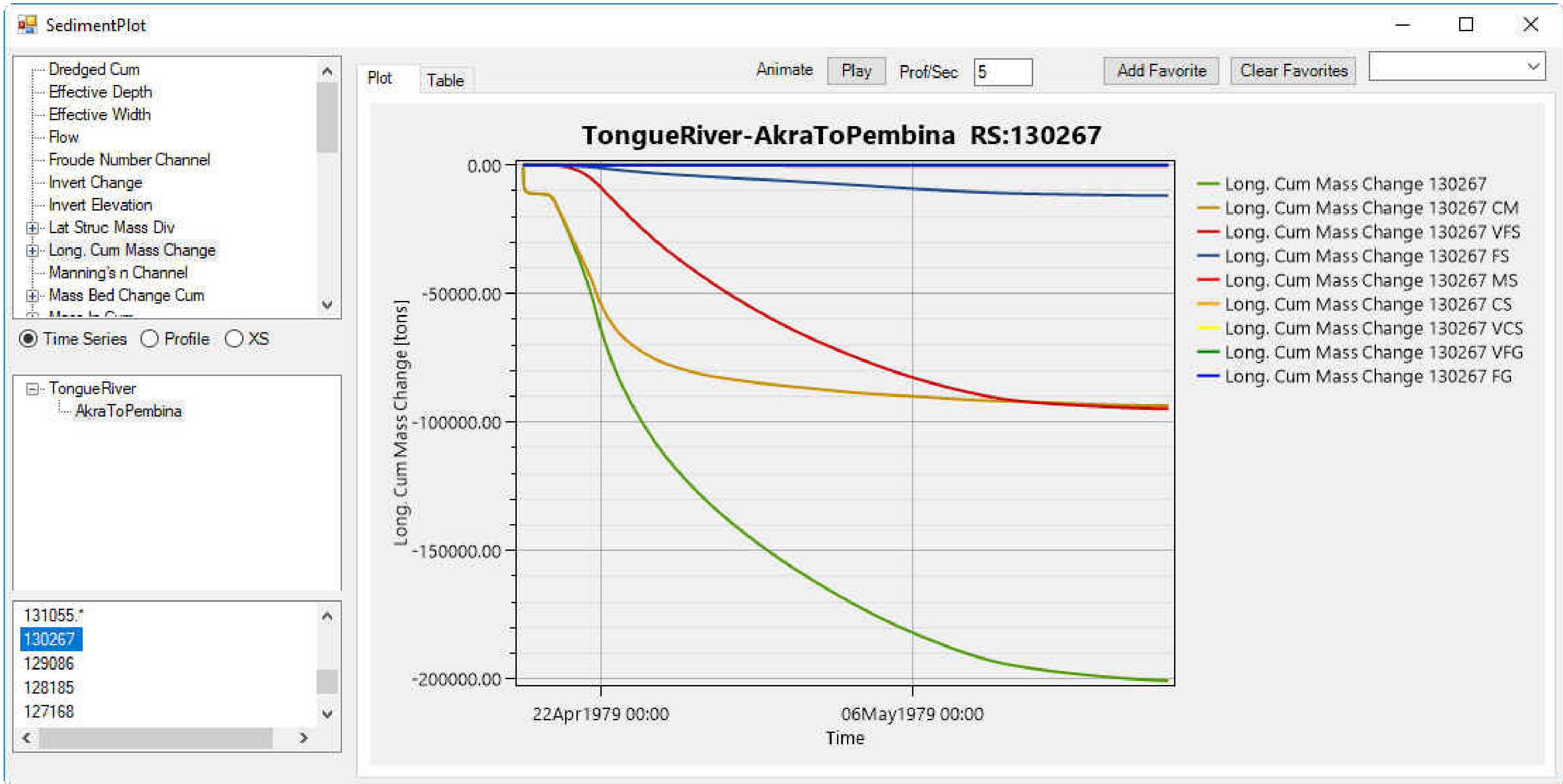

Figure 48. HEC-RAS 100-year sediment cumulative mass change at maximum erosion cross section for Park St 5ft weir model

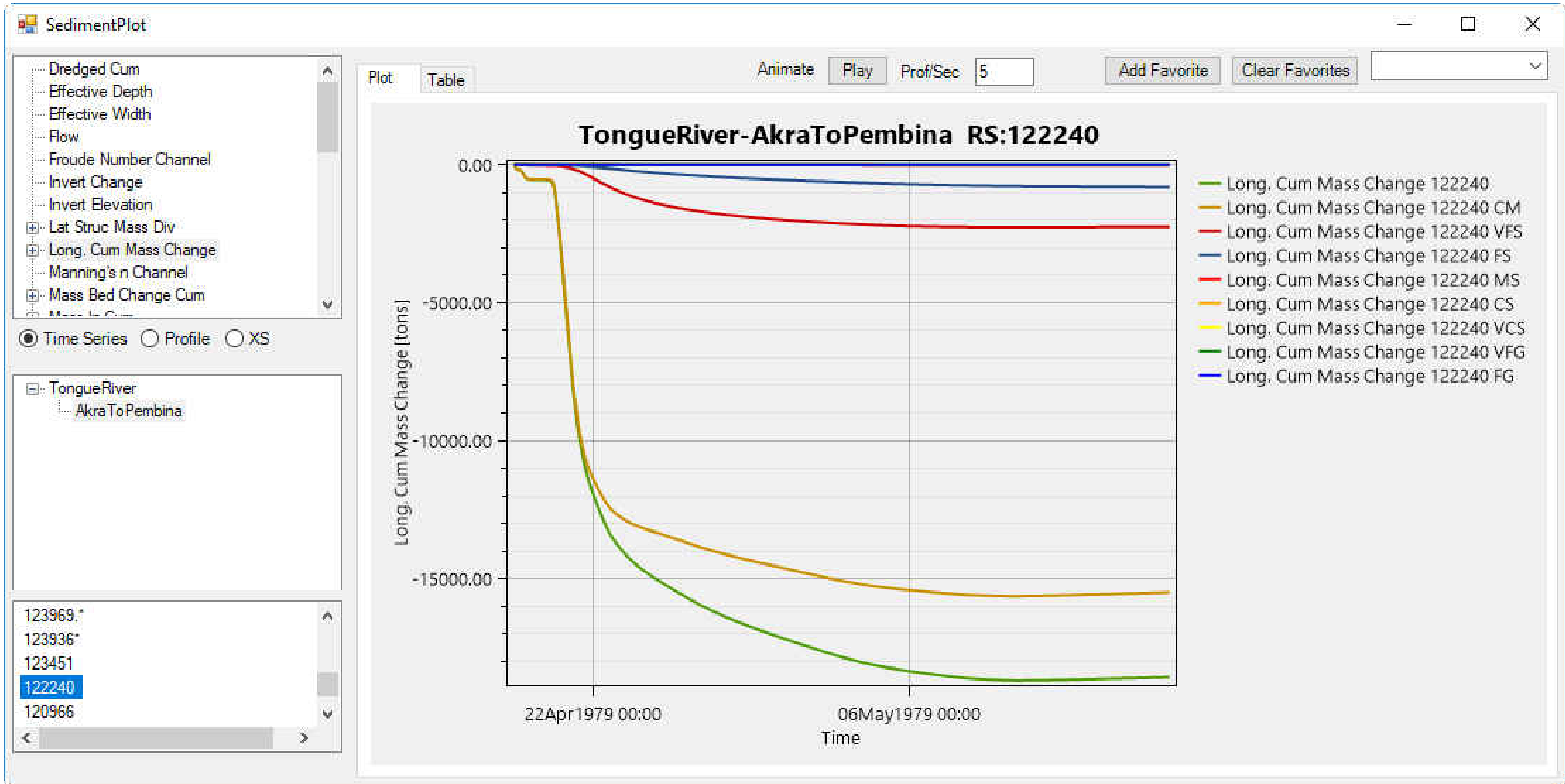

Figure 49. HEC-RAS 2-year sediment cumulative mass change at maximum erosion cross section for Woodland Terrace 5ft weir model

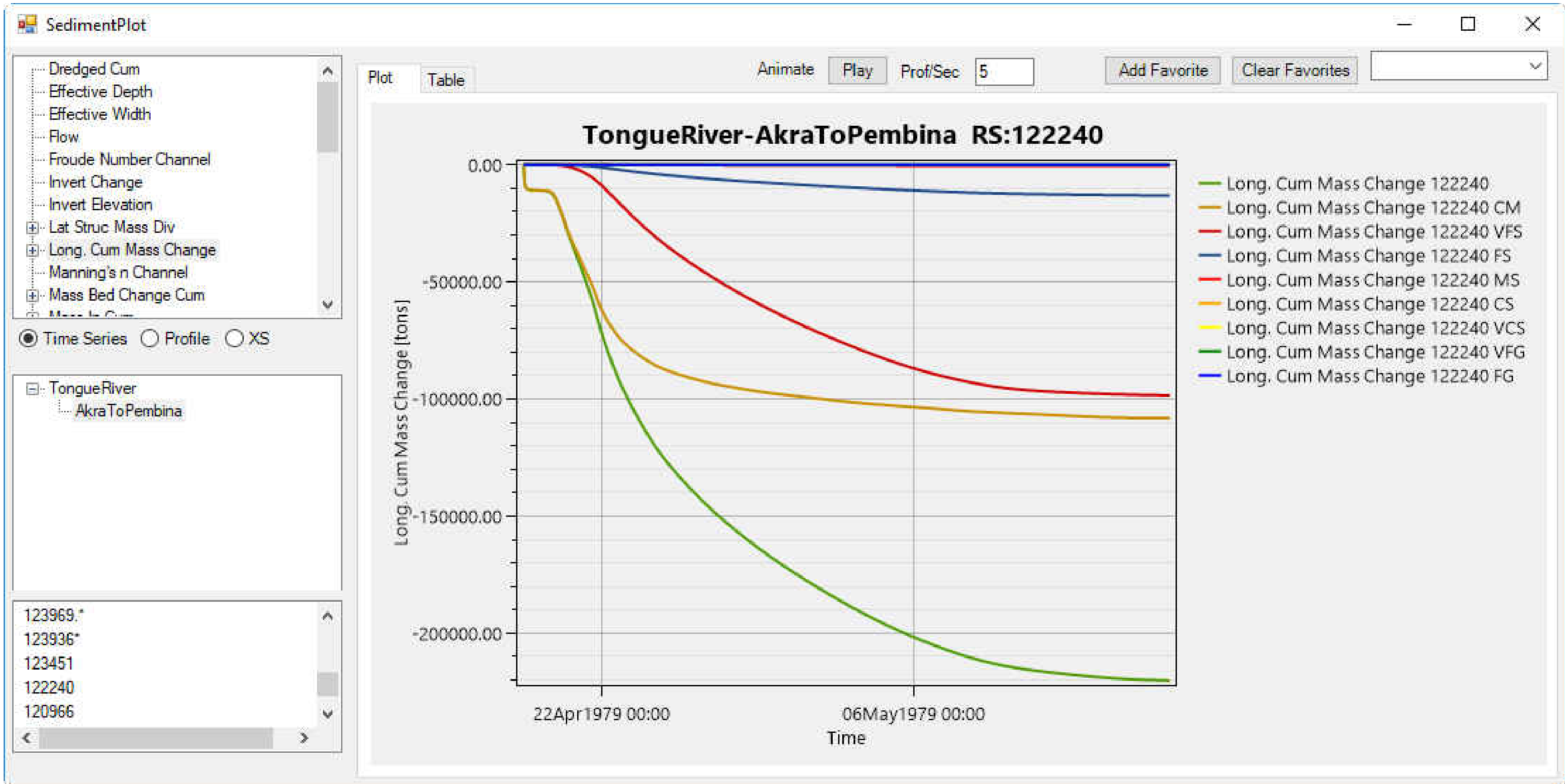

Figure 50. HEC-RAS 100-year sediment cumulative mass change at maximum erosion cross section for Woodland Terrace 5ft weir model

4.2 Summary of Assumptions and Limitations

Several assumptions were made when creating the geometry. The first limitation is that the GPS data collected during the channel bathymetry survey near the dam and bridges, excluding the Island Bridge, Highway 5 (96^{th} St NE), and 98^{th} St NE was not accurate. An adjustment value had to be applied later to fix the channel inverts. LiDAR elevations were assumed to provide a close estimate since the benchmark was typically the top of the bridge deck. Channel inverts between these surveyed locations were also interpolated and assumed to follow a constant slope connecting the known elevations.

None of the culverts through lateral structures and storage area connections have been surveyed. Elevations were estimated from LiDAR and sizes were assumed to be 3 ft unless the aerial imagery clearly indicated that it was larger.

There is no survey before and after a flood event that would allow the sediment model to truly be calibrated to actual erosion and deposition patterns.

There was also a limited amount of sediment data available. The model requires suspended sediment and water temperature data. Fortunately the USGS gage immediately downstream of the dam has recorded some of this data. However, the data was only collected during a short time in 2003 and 2004. The model assumes that these values are representative of normal operations.

As discussed in Section 3.3.3, the cohesive properties need to be measured or calibrated because no correlation has been found that is reliable due to variance in environmental factors (Brunner, 2016). The critical shear stress, mass erosion threshold, and erosion rates have not been measured for the Tongue River. The critical shear stress has been measured in the Fargo-Moorhead, but none of the remaining values have been measured in the Red River Valley to the author's knowledge. These values were reported in two Chesapeake Bay studies with somewhat similar gradations to the Tongue River and were used in the model (Langland & Koerkle, 2014) (West Consultants, Inc., 2017).

More assumptions have been made in the sediment transport modeling regarding the transport function, fall velocity, and sorting method calculations. Sensitivity runs using various transport methods have helped to choose the correct option, as discussed in 3.3.4.

There are several limitations in relation to the capability of the software as well. This is a one-dimensional model, so water does not adjust around the bends to have more erosion on the outside. In addition, HEC-RAS uses the veneer method to change cross section geometry. All of the nodes that are wetted and within the movable bed limits will change the same vertical distance. An example of this is shown in Figure 51.

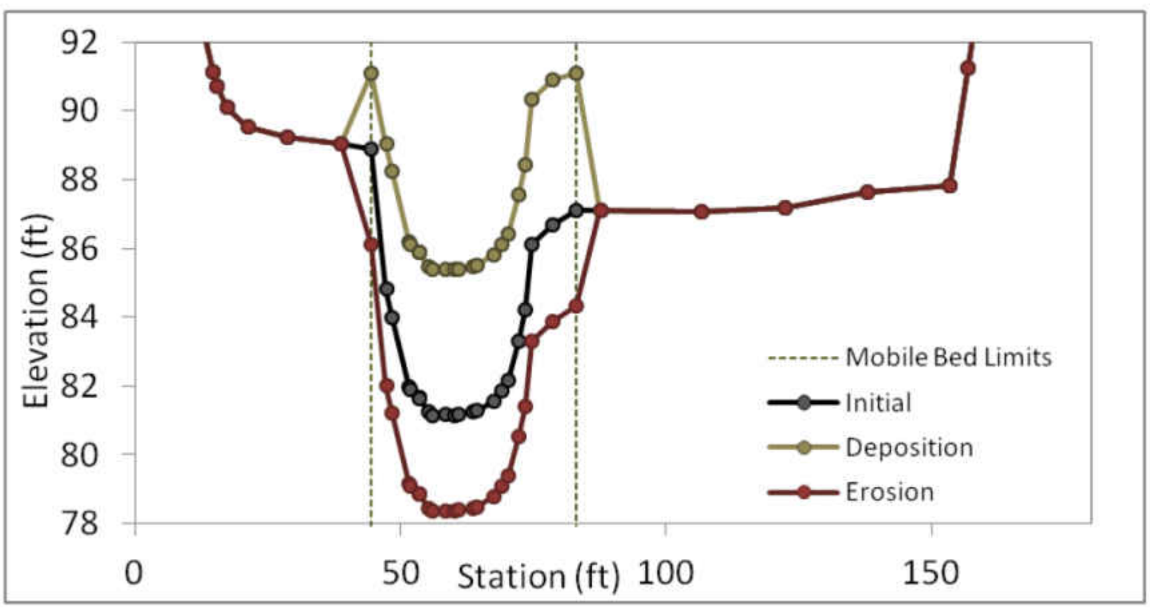

Figure 51. Example of the veneer method of erosion and deposition in HEC-RAS (Brunner, 2016)

4.3 Suggestions for Future Work

The model geometry could be refined with additional channel bathymetry survey, particularly at the proposed weir locations and the bridges downstream of Division Avenue. A great way to calibrate the sediment model would be to survey some channel cross sections in the fall, then again in the spring after the snowmelt peak. Preferably in a year with considerable flow to be sure the difference isn't negligible. The sediment model could then be calibrated with historical USGS data and survey.

Another way to greatly improve the accuracy of the modeling would be to physically test soil properties. The suspended sediment and temperature samples recorded at the USGS gage immediately downstream of the dam were limited to only 2003 and 2004. Gathering additional data there as well as farther downstream where the dam effects have dissipated would be beneficial to the model.

The cohesive soil properties have been discussed as needing to be measured and not estimated. The UND Civil Engineering or Geological Engineering departments may have access to a tri-axial soil testing machine, however it would not be able to test all four properties required. The SEDflume is commonly used to measure the cohesive parameters, usually from Shelby tube samples. Currently the equipment is available at the Corps' sediment lab in the Engineer Research and Development Center (ERDC) and several universities (Brunner, 2016).

CHAPTER 5. CONCLUSION

Past studies in the area suggested the installation of stepped rock weirs to raise the water surface in the channel would increase the channel stability. HEC-RAS v.5.0.3 one-dimensional sediment transport modeling was used to analyze whether a single set of rock weirs could provide benefit to all of the properties with erosion problems on the Tongue River in Cavalier, ND. Some locations have already completed projects to protect their individual sites, such as Site 7 and Site 6 between the Park Street and Woodland Terrace weirs. These are areas of reduced risk and it is important not to negatively impact their properties with a new project. Three potential weir locations were identified: immediately downstream of Site 6 in Figure 2 near Woodland Terrace, immediately downstream of Site 8 near Park Street, and immediately downstream of the Division Ave bridge. Each weir was modeled at 3, 4, and 5 ft heights.

The existing conditions unsteady flow model geometry was created using LiDAR, survey, and other available data. Hydrology was developed with a USACE HEC-HMS model of the Pembina River watershed and USGS gage data. Then the unsteady flow model was calibrated to 2013 flow and elevation data. The sediment transport model used the gradation data collected at the time of survey, USGS gage data, and several assumed parameters. The results do not precisely model actual erosion and deposition depths due to these assumptions and some software limitations. This study compares potential project impacts from each weir location. The parameters stay the same across all models and a preferred option can be selected by a relative comparison.

The Park Street and Woodland Terrace weirs both benefit all of the remaining problem sites, however the Park Street weir has greater benefit and less scour. The effects of both the Division Ave and Park Street weirs dissipate near Hwy 5, which means they would not adversely affect the two projects already in place. Division Ave is farther upstream and provides less benefit to the town, as well as creating a larger scour depth. The taller weirs created more benefit, but also more severe erosion immediately downstream of the weirs, as expected.

The best option to maximize benefit to all of the remaining problem sites, minimize scour, and address the most urgent needs, would be a weir at the Park Street location. A combination of the Park Street and Division Ave weirs could also be investigated. This would need to be verified with more detailed modeling.

Results indicate that the first null hypothesis, in which a single weir location cannot improve erosion for all of the remaining problem sites, can be rejected. The second null hypothesis, that the project will not adversely affect the downstream properties, can also be rejected. This supports the research hypotheses that a low head dam installed at a location in town would improve erosion for all of the remaining sites, while increasing erosion at downstream properties.

APPENDIX A – SIEVE ANALYSIS RESULTS

Table 18.
Dam Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4	7.5	7.5	0.930983	99.06902
2.36	No. 8	15.8	15.8	1.961271	98.03873
2	No. 10	8.8	24.6	3.053625	96.94638
1.18	No. 16	34.2	58.8	7.298908	92.70109
600µm	No. 30	80.5	139.3	17.29146	82.70854
425µm	No. 40	73.9	213.2	26.46475	73.53525
300µm	No. 50	88.8	302	37.48759	62.51241
150µm	No. 100	241.9	543.9	67.5149	32.4851
75µm	No. 200	148.5	692.4	85.94836	14.05164
Minus No. 200		35.7			
Original Wt.		805.6			
Wt. After Wash		729.4			
Wash Loss		76.2			
Wt. Check		804.3			

Table 19.
136th Ave. Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4		0	0	100
2.36	No. 8		0	0	100
2	No. 10		0	0	100
1.18	No. 16	0.1	0.1	0.016239	99.98376
600µm	No. 30	5.9	6	0.974342	99.02566
425µm	No. 40	16.6	22.6	3.670023	96.32998
300µm	No. 50	41.3	63.9	10.37675	89.62325
150µm	No. 100	236.3	300.2	48.74959	51.25041
75µm	No. 200	209.8	510	82.8191	17.1809
Minus No. 200		41.3			
Original Wt.		615.8			
Wt. After Wash		549.5			
Wash Loss		66.3			
Wt. Check		617.6			

Table 20.
138th Ave. Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4	1.1	1.1	0.182059	99.81794
2.36	No. 8	0.7	0.7	0.115856	99.88414
2	No. 10	0.3	1	0.165508	99.83449
1.18	No. 16	1.5	2.5	0.41377	99.58623
600µm	No. 30	3	5.5	0.910295	99.08971
425µm	No. 40	5	10.5	1.737835	98.26216
300µm	No. 50	12.6	23.1	3.823237	96.17676
150µm	No. 100	164.9	188	31.11552	68.88448
75µm	No. 200	278.1	466.1	77.14333	22.85667
Minus No. 200		40.7			
Original Wt.		604.2			
Wt. After Wash		508			
Wash Loss		96.2			
Wt. Check		603			

Table 21.
140th Ave. Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4		0	0	100
2.36	No. 8	0.3	0.3	0.036919	99.96308
2	No. 10	0.1	0.4	0.049225	99.95078
1.18	No. 16	1	1.4	0.172286	99.82771
600µm	No. 30	2.3	3.7	0.455329	99.54467
425µm	No. 40	2.3	6	0.738371	99.26163
300µm	No. 50	5.1	11.1	1.365986	98.63401
150µm	No. 100	137.6	148.7	18.29929	81.70071
75µm	No. 200	477.1	625.8	77.01206	22.98794
Minus No. 200		122.5			
Original Wt.		812.6			
Wt. After Wash		748.2			
Wash Loss		64.4			
Wt. Check		812.7			

Table 22.
1st Ave. Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4	1.7	1.7	0.215435	99.78456
2.36	No. 8	2	2	0.253453	99.74655
2	No. 10	0.8	2.8	0.354835	99.64517
1.18	No. 16	4.6	7.4	0.937777	99.06222
600µm	No. 30	8.6	16	2.027626	97.97237
425µm	No. 40	9.3	25.3	3.206184	96.79382
300µm	No. 50	12.3	37.6	4.764922	95.23508
150µm	No. 100	101.4	139	17.615	82.385
75µm	No. 200	324.1	463.1	58.68711	41.31289
Minus No. 200		149.9			
Original Wt.		789.1			
Wt. After Wash		615.3			
Wash Loss		173.8			
Wt. Check		786.8			

Table 23.
Division Ave/Island Bridge Sieve Analysis

(mm)	Ret.	Wt. Ret. Non-Cum.	Wt. Ret. Cum.	% Ret.	% Pass
	No. 4	0.2	0.2	0.031842	99.96816
2.36	No. 8	1.1	1.1	0.175131	99.82487
2	No. 10	0.8	1.9	0.3025	99.6975
1.18	No. 16	2.9	4.8	0.76421	99.23579
600µm	No. 30	6.1	10.9	1.735392	98.26461
425µm	No. 40	6.6	17.5	2.786181	97.21382
300µm	No. 50	10.7	28.2	4.489731	95.51027
150µm	No. 100	116	144.2	22.95813	77.04187
75µm	No. 200	279.8	424	67.50517	32.49483
Minus No. 200		85			
Original Wt.		628.1			
Wt. After Wash		508.6			
Wash Loss		119.5			
Wt. Check		628.5			

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