

Streambank erosion downstream of a flood control dam: Processes, rates,
and management

by

Kari A. Bigham

B.S., Kansas State University, 2011

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AN ABSTRACT OF A DISSERTATION

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DOCTOR OF PHILOSOPHY

Carl and Melinda Helwig Department of Biological and Agricultural
Engineering

Carl R. Ice College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

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Abstract

For nearly a century, large dams have played a major role in protecting and supporting human life by storing runoff for water supply, hydropower, and flood control. But as with exploiting any natural resource for anthropogenic purposes, dam installation comes with an environmental cost. This research answers the question: Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how? A case study was conducted along a 111 km reach downstream of Kanopolis Dam, a 1948-installed flood control dam located on the Smoky Hill River in central Kansas, USA. This study had three objectives, evaluate the effects Kanopolis Dam closure has had on downstream: (1) flow and channel bed stability (2) channel planform, and (3) streambank erosion processes and rates.

Statistical analysis of historical flow data from four USGS stream gages on the Smoky Hill River showed that peak flows had dampened, resulting in a more step-like versus flashy hydrograph. Additionally, an increase in moderate daily flows between a 5 to 40% daily exceedance probability following dam installation up to 111 river km downstream was observed. Flow rating curves from USGS stream gages were used to assess bed stability at a low flow stage overtime. The bed stability analysis showed aggradation at the USGS gage upstream of the dam following dam closure, with degradation observed immediately downstream of the dam. Based upon a statistical forecasting method, the bed elevation has stabilized throughout the 111 km reach over the last decade. Channel planform was assessed using two metrics: (1) channel migration rates and (2) sinuosity. Based on GIS analysis of historical aerial imagery and mapping, channel migration rates, or the area of floodplain reworked per stream length per time, have decreased by 69% while channel sinuosity has increased by 5% since Kanopolis Dam closure. An increase in sinuosity is likely caused by an increase in lateral migration rates caused by bank erosion. Finally, using a HEC-RAS 1D

quasi-unsteady hydraulic, sediment transport, and bank erosion model, it was shown that toe erosion is the dominant streambank erosion process for a site 86 river km downstream of Kanopolis Dam. Furthermore, if the dam was removed, predicted bank erosion would be higher under a no dam flow condition versus a with dam flow condition. Coupling model results with measured cross sections and observation, it was also demonstrated that toe erosion is occurring in other locations of the stream where bank erosion is typically less common, such as the inside of meander bends and along straight reaches, likely attributed to the step-like hydrograph brought on by flow regulation. System wide toe erosion occurring over longer periods of time along 111 river km downstream of Kanopolis Dam may also explain the observed increase in sinuosity since dam closure.

Understanding the geomorphic context of a stream is invaluable to proper and effective management by stream practitioners. Two stream succession scenarios were observed on the Lower Smoky Hill River: (1) [Rosgen \(2014\)](#) stream succession scenario 1 within the degradational zone downstream of Kanopolis Dam and (2) a proposed stream succession scenario downstream of the degradational zone still influenced by flow regulation. Stream succession scenarios can be applied to assist in erosion management strategies and techniques following dam closure. Streambank erosion management techniques were also discussed and evaluated using the same HEC-RAS 1D model. Modeling results showed that a woody bankfull bench, in combination with 3H:1V bank shaping, would provide stability for at least a 40-year post-dam return interval flow event on the Smoky Hill River. This research demonstrates application of a popular and free numerical model to assist with assessing geomorphic change, as well as potential restoration techniques.

Since closure in 1948, Kanopolis Dam has prevented nearly 2.5 billion dollars in downstream flood damage, proving its value and need. While large dams are and will continue to be an important part of our landscape and livelihood, it is imperative that the environmental cost of dam operation is understood and evaluated. Future research and monitoring efforts should continue to assess and value damages induced by dams, especially under a changing climate, to inform and adapt river management and dam operation through time.

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Approved by:

Co-Major Professor
Trisha Moore

Approved by:

Co-Major Professor
Tim Keane

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Chapter 1

Downstream Geomorphic Effects of Large Dams: Literature Review

1.1 Streambank Erosion and Channel Instability

Streambank erosion is a natural geomorphic process that is necessary to dissipate flow energy and create and maintain habitats ([Florsheim et al., 2008](#)). However, accelerated streambank erosion, brought on by channel instability, is a leading source of sediment in many impaired streams around the globe ([Carter et al., 2003](#); [Gellis & Sanisaca, 2018](#); [Hupp et al., 2009](#); [Koiter et al., 2013](#); [Poletto et al., 2009](#)). Excess fine sediment in streams reduces water quality by increasing suspended material and turbidity, as well as concentrations of sediment-borne pollutants, such as phosphorus, pesticides, and pathogens ([Vorosmarty et al., 2010](#)). As a result, aquatic habitats are threatened and degraded and surface water treatment costs are increased. The process of streambank erosion can result in loss of privately-owned land and can threaten transportation and communication infrastructure ([Morris et al., 1996](#)), as well as downstream reservoir or stream storage through accelerated sedimentation processes ([Fox et al., 2016](#)).

Stream channel size, profile, and planform are determined largely by incoming flow and sediment regimes. As conceptualized in [Lane \(1955\)](#), a change to either of these factors

results in channel instability as the stream adjusts to this change. Channel evolution models attempt to describe the process and implications of channel instability. Channel evolution, or stream succession, is one of the most influential factors on bank erosion processes and rates through time (Rosgen, 2009; Schumm et al., 1984; Simon & Hupp, 1986). Stream succession, defined by Keane & Sass (2017), is the “various three-dimensional forms or stages a channel sequences through in seeking a stable, dynamic equilibrium form following disturbance (pg. 1513).” Further, a stable stream in dynamic equilibrium is defined as one of constant dimension, plan, and profile that is neither aggrading nor degrading that reaches its floodplain, on average, every 1.5 years (Rosgen, 1996).

Over the last 30 years, scientists have been developing models of the channel evolution process, resulting in both linear (e.g., Schumm et al., 1984; Simon & Hupp, 1986) and more cyclic forms (e.g., Cluer & Thorne, 2014; Rosgen, 2009). Although numerous channel evolution models with notable differences exist (Keane & Sass, 2017), they all tend to follow the basic temporal model outlined by Schumm et al. (1984) and illustrated in Figure 1.1: (1) Pre-existing condition, (2) Degradation following disturbance (e.g. change in flow and/or sediment regime), (3) Initial aggradation and channel widening, (4) Continued aggradation and further channel widening, and (5) Equilibrium. Accelerated streambank erosion is most prominent during stages (2) through (4), as degradation can increase bank height and slope, decrease vegetation, and potentially expose new sediment layers while aggradation can redistribute hydraulic forces (Simon & Hupp, 1986). Further, the longitudinal form of the Schumm et al. (1984) model (see Fig. 1.1) shows that channel evolution stage can also vary spatially, as degradation upstream can increase sediment supply, resulting in downstream aggradation. It is noted that this linear form of channel evolution may be more cyclic in nature and may skip or revisit various stages overtime due to the dynamic nature of watershed processes, resembling more of continuum, rather than a discrete, linear process (Cluer & Thorne, 2014; Rosgen, 2009).

The process of channel evolution begins when a disturbance occurs. While natural disturbances can occur over a geological time scale, in the present time, most disturbances that result in stream instability and evolution are influenced by mankind. Examples in-

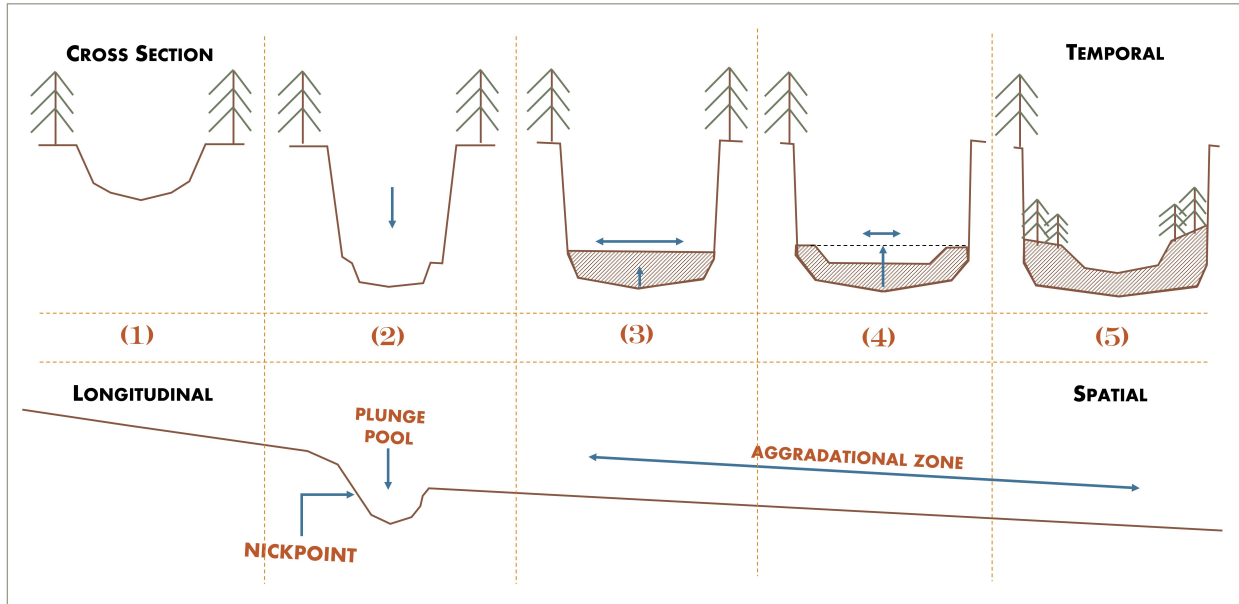


Figure 1.1: The [Schumm et al. \(1984\)](#) Channel Evolution Model in both Cross Section (Temporal) and Longitudinal Forms (Spatial).

clude land use and/or cover changes due to urbanization or agriculture, channelization, and construction of dams ([Simon & Rinaldi, 2000](#); [Trimble, 1997](#); [Williams & Wolman, 1984](#)). These anthropogenic changes to the environment often occurred to acquire or control certain natural resources with little to no consideration of environmental effects. Among the most extreme anthropogenic disturbance to the flow and sediment regimes of streams are the installation of dams.

The goal of this research is to determine if stream practitioners can manage downstream channel instability caused by the installation of a dam, and if so, how. In other words, can dams and downstream stable stream channels co-exist? If so, how can we expedite the evolutionary process? The following literature review summarizes recent research and case studies that evaluate the effects dams have had on downstream channel instability and thus, streambank erosion.

1.2 Downstream Geomorphic Effects of Large Dams

For over 5,000 years, man-made dams have played a major role in protecting and supporting human life (Collier et al., 1996). Following the introduction of heavy machinery in the early to mid-1900s, dams, especially large dams, began to crop up at an exceptional rate throughout the United States and globally (Graf, 1999). According to the [International Commission on Large Dams \(ICOLD\)](#) (2011, p.3), a large dam is defined as one with a “height of 15 meters or greater from lowest foundation to crest or a dam between 5 meters and 15 meters impounding more than 3 million cubic meters.” As of 2020, ICOLD reported over 58,000 large dams worldwide, with the majority of them installed to capture runoff for water supply, hydroelectricity production, and flood control (ICOLD, 2020). Large dams are of special interest when it comes to understanding the geomorphic implications of their installation. While fewer in number, large dams have the ability to hold a significant amount of water. For example, after evaluating over 75,000 dams in the United States, Graf (1999) found that just 3% of these structures (i.e., the proportion of “large” dams) accounted for 63% of the total reservoir storage for these evaluated dams. Given the scale of the amount of water impounded, large dams have more notable and large scale effects to both the hydrologic and sediment regimes.

Beyond ongoing reservoir sedimentation that occurs in these man-made lakes (Fox et al., 2016; Schleiss et al., 2016), dams have a major effect on stream stability both upstream of the resulting reservoir and downstream of the dam (Graf, 2005; Petts & Gurnell, 2005; Wang et al., 2007; Williams & Wolman, 1984). Reported effects upstream of the reservoir are decreased energy slope and sediment transport capacity, resulting in aggradation up to many kilometers upstream on the main channel and its tributaries (Wang et al., 2007). Alternatively, geomorphic change downstream of dams tend to be more drastic, noticeable, and wide-scale, as both the flow and sediment regimes are usually affected (Petts & Gurnell, 2005; Williams & Wolman, 1984). The following sections focus on large dam alterations to the flow and sediment regimes, as well as the reported effects to downstream channel morphology.

1.2.1 Flow Regime Effects

The most obvious downstream effect of dams is the change in flow regime. Often times, peak flows are dampened, resulting in low flow changes, “step-like” versus “flashy” hydrographs, and altered flow patterns on daily to annual time scales (Brandt, 2000; Graf, 2006; Petts & Gurnell, 2005; Phillips, 2003; Schmidt & Wilcock, 2008; Williams & Wolman, 1984). Flow regulation can also impact the bankfull or channel forming discharge below dams. As explained by Leopold et al. (1964)[pg. 71] “...extremely high flows, though effective in geomorphic work of erosion and transportation, are so infrequent that their contribution to the total work is less than that of more modest discharges which occur more often.” These more modest and frequent events are referred to as the bankfull or effective discharge, and typically occur within a 1-year to 2-year return interval (Leopold et al., 1964).

There is disagreement in the literature in regard to how dam installation affects bankfull flow events. Two assumptions are often made when evaluating bankfull or effective discharge downstream of dams:

1. Bankfull discharge (volume/time) stays the same but its annual frequency will increase, decrease, or stay the same (Brandt, 2000; Church, 1995; Page et al., 2005; Petts, 1979; Petts & Gurnell, 2005; Williams & Wolman, 1984)
2. Bankfull discharge changes and its associated daily and/or annual frequency might also change (e.g., Lyu et al., 2020; Ma et al., 2012; Wu et al., 2008)

Ma et al. (2012) argued that bankfull flow may not exist below dams following a 60-year study downstream of four large dams on the Lower Yellow River in China, with varying flow and sediment (i.e., flushing) release schedules across both space and time. Brandt (2000) obtained a similar observation, noting that intermediate flows just below bankfull may do a considerable amount of work downstream of dams. It is unclear which assumption is the correct one; however, it is noted that assumption (1) was the most common among the reviewed literature.

Regardless of what bankfull assumption is made, the duration of a bankfull event can

also change. Intuitively, one might expect that duration may be less or stay the same for areas where snowmelt runoff occurs, as snow itself acts temporarily as surface storage. For example, bankfull flow duration was reduced by half following dam installation on the Murrumbidgee River in Australia, where snowmelt runoff dominates in the headwaters of the river (Page et al., 2005). By contrast, bankfull duration could increase in areas where storm flow or flashy hydrographs would naturally occur, especially downstream of dams meant to protect against flooding caused by unpredictable storm events. Effects of flow regulation on the flow regime decrease and become more naturalized, as unregulated flows from tributaries emerge with distance from the dam (Petts & Gurnell, 2005). How far downstream flow effects of dams persist depends on the site conditions as well as the specific precipitation and runoff event.

1.2.2 Sediment Regime Effects

Coinciding with the flow regime and of equal importance in determining channel size and channel slope is the sediment regime (Lane, 1955; Wohl et al., 2015), often described as sediment supply, transport, and storage. The resultant reservoirs of dams act as large sediment traps, with capture efficiencies greater than 95% for many reservoirs (Juracek, 2011; Williams & Wolman, 1984). Such reduction is an extremely drastic change to the downstream sediment regime, as noted by Leopold et al. (1964)[pg. 457]:

“It is difficult to conceive of a climate change producing a change in vegetation on the watershed such that 95% of the sediment would no longer be delivered to the stream system.”

Because of great reduction in sediment supply, degradation in the stream reach closest to the dam has been the most studied downstream effect of dam installation (Petts, 1979; Williams & Wolman, 1984). Armoring of the streambed can occur if the bed sediment is poorly-sorted (Petts & Gurnell, 2005; Richard et al., 2005; Williams & Wolman, 1984). Armoring is the process in which fines are washed away and not replenished by upstream sources. Williams & Wolman (1984) noted that bed degradation downstream of dams

would subside with time, but how long was unknown and likely dependent on a number of factors. These factors include: (1) the eventual reduction in stream gradient brought on by the process of degradation, (2) the presence of local controls (e.g., armoring, bedrock, grade control), (3) increased sediment load from channel erosion and scour processes, as well as tributaries, with distance downstream, and (4) vegetation encroachment ([Schmidt & Wilcock, 2008](#); [Williams & Wolman, 1984](#)). The process of degradation was found to follow a hyperbolic function, with degradation rates being highest in the early years following dam closure and decreasing steadily with time ([Juracek, 2011](#); [Williams & Wolman, 1984](#)). In addition, [Williams & Wolman \(1984\)](#) assumed that the distance downstream in which the degradational zone may propagate was dependent on factors such as time, types of flow releases, streambed sediment composition, and topography. Degradation induced by dam installation can also cause stream bar surface areas to decrease ([Grams et al., 2007](#); [Li et al., 2019](#); [Lyu et al., 2020](#)), resulting in major habitat and, potentially, tourism issues by reducing possible camping and boating rest areas, as observed on the Colorado River in the Grand Canyon below Glen Canyon Dam ([Collier et al., 1996](#)). Aggradation further downstream of the degradational zone has also been observed (e.g., [Collier et al., 1996](#); [Kong et al., 2020](#); [Petts & Gurnell, 2005](#); [Schmidt & Wilcock, 2008](#)), which follows with the spatial scale of the [Schumm et al. \(1984\)](#) channel evolution model presented in Figure 1.1.

While degradation below dams is most common due to sediment deficiencies, in some cases, incision may not occur or aggradation occurs if flow releases are not competent to transport streambed material. Aggradation has been observed in streams where bed material is very coarse, such as below Flaming Gorge Dam on the Green River of Colorado, Glen Canyon Dam on the Colorado River of Arizona ([Schmidt & Wilcock, 2008](#)), and the Snake River below Jackson Hole Lake ([Marston et al., 2005](#)). Aggradation has also been observed where reservoir flushing of fine sediment occurs, such as below the Sidi Salem Dam in Tunisia of northern Africa ([Zahar et al., 2008](#)). Flushing of fines resulted in downstream aggradation and thus a decrease in channel capacity. Reduction in cross sectional area due to aggradation of the channel resulted in downstream flooding during flow releases that were once contained within the channel boundaries ([Zahar et al., 2008](#)). Dams built for maintaining water supply

can also result in aggradation downstream. An example is downstream of Elephant Butte Dam on the Rio Grande River in New Mexico, where several feet of sediment has deposited in the channel following dam construction. In places where levees exist, the Rio Grande River is now perched above its old floodplain. The cause of aggradation here is a combination of both degradation upstream just below the dam, as well as the lack of flood peaks to (1) transport the uncontrolled sediment supply from tributaries and other sources and (2) drown out encroaching vegetation thus resulting in sediment deposition caused by flow velocity reduction induced by the vegetation (Collier et al., 1996; Petts, 1979).

1.2.3 Channel Morphology Effects

How channel planform (i.e., width, sinuosity, migration) responds to changes in the flow and sediment regime varies across studies, both spatially (e.g., distance from dam, across watersheds) and temporally (Brandt, 2000; Friedman et al., 1998; Petts & Gurnell, 2005; Williams & Wolman, 1984). Table 1.1 summarizes channel width response from 19 published case studies. The distance associated with “further downstream” can be quite variable but is defined in this paper as the area downstream of the degradational zone caused by the reduced sediment load (i.e., downstream of Stage 2 in in Figure 1.1). While there are many factors that may influence channel width response (Table 1.1), common explanations of channel widening include sediment starvation, increased base flows, and changes in water level draw-down brought on by dam operations. Common explanations for channel narrowing include peak flow dampening caused by dam operation, vegetation encroachment due to a change in flow conditions, and aggradation, especially further downstream of the degradational zone.

Additionally, several studies have attempted to summarize geomorphic effects of dams from several locations, time periods, and case studies. (e.g., Brandt, 2000; Friedman et al., 1998; Graf, 2006; Petts, 1979; Petts & Gurnell, 2005; Schmidt & Wilcock, 2008; Williams & Wolman, 1984) Notable observations from these studies include:

Table 1.1: Comparison of Channel Width Response from Case Studies Downstream of Large Dams. *Note:* “Near dam” represents the segment of river within the degradational zone caused by the reduced sediment load from the reservoir; “Further downstream” refers to downstream of the degradational zone; “Downstream” refers to any segment(s) of streams downstream of a dam, with its location not specified by the authors.

Study Location	Width Response	Dam Age	Possible Explanations	Dam	Primary Purpose	Dam Location	Sources
Near Dam	<i>Narrow</i>	59	Peak flow dampening Reduced base flow Vegetation encroachment	Beniarrés Dam	Flood control	Spain	(Sanchis-Ibor et al., 2019)
Near Dam	<i>Narrow</i>	19	Peak flow dampening Vegetation encroachment	Kiri Dam	Water supply	Nigeria	(Tukur & Mubi, 2002)
Near Dam	Widen	62	Sediment starvation	Kanopolis Dam	Flood control	Kansas, USA	(Juracek, 2011)
Near Dam	Widen	48	Sediment starvation	Tuttle Creek Dam	Flood control	Kansas, USA	(Juracek, 2011)
Near Dam	Widen	44	Increased base flows More runoff due to diversion Change in drawdown conditions	Hog Park Dam	Water supply	Wyoming, USA	(Gilliam, 2011)
Near Dam	Widen	35	Sediment starvation	Livingston Dam	Water supply	Texas, USA	(Phillips et al., 2005)
Near Dam	Widen	33	Sediment starvation	Toledo Bend Dam	Hydropower	Texas, USA	(Phillips, 2003)
Near Dam	Widen	6	Sediment starvation	High Savery Dam	Water supply	Wyoming, USA	(Legleiter, 2015)
Further Downstream	<i>Narrow;</i> <i>No Change</i>	27	High sediment load downstream Downstream control by estuary	Livingston Dam	Water supply	Texas, USA	(Phillips et al., 2005) (Wellmeyer et al., 2005)
Further Downstream	No Change	33	Minimal change to flow regime High sediment load downstream	Toledo Bend Dam	Hydropower	Texas, USA	(Phillips, 2003)
Further Downstream	Widen	32	Aggradation Vegetation encroachment	Flaming Gorge Dam	Hydropower	Colorado, USA	(Merritt & Cooper, 2000)
Downstream	<i>Narrow</i>	42	Vegetation encroachment Peak flow dampening	Flaming Gorge Dam	Hydropower	Utah, USA	(Grams & Schmidt, 2005)
Downstream	<i>Narrow</i>	37	Peak flow dampening Aggradation Vegetation encroachment	Glen Canyon Dam	Hydropower	Arizona, USA	(Grams et al., 2007)
Downstream	<i>Narrow</i>	28	Peak flow dampening	Cochiti Dam	Flood control	New Mexico, USA	(Richard et al., 2005)
Downstream	<i>Narrow</i>	21	Aggradation Vegetation encroachment	WAC Bennett Dam	Hydropower	Canada	(Church, 1995)
Downstream	Widen	66	Change in drawdown conditions	Bagnell Dam	Hydropower	Missouri, USA	(Mohammed-Ali et al., 2020)
Downstream	Widen	52	Sediment starvation Change in drawdown conditions	E.B. Campbell Dam	Hydropower	Canada	(Smith et al., 2016)
Downstream	Widen	44	Increased base flows	Roanoke Rapids	Flood control	North Carolina, USA	(Hupp et al., 2009)
Downstream	Widen	9	Increased base flows Reservoir sediment flushing	Xiaolangdi Dam	Flood control	China	(Ma et al., 2012)

- Braided channels, defined as a streambed with more than one channel during baseflow conditions, tended to narrow and become single-threaded following dam installation (Collier et al., 1996; Friedman et al., 1998; Petts & Gurnell, 2005)
- Channel migration rates (i.e., area of reworked floodplain reworked per channel length per year) decreased (Friedman et al., 1998)
- Low flow channels that contain the median daily flow became larger, while high flow channels containing flows greater than the median daily flow narrowed (Graf, 2006)
- Channel width adjustments are highly dependent upon localized boundary conditions (Petts, 1979; Schmidt & Wilcock, 2008; Williams & Wolman, 1984)

These aforementioned observations have been further supported in individual case studies as well (e.g., Kong et al., 2020; Richard et al., 2005; Sanchis-Ibor et al., 2019; Shields Jr. et al., 2000; Surian, 1999; Wang et al., 2007, See Table 1.1).

Streambank erosion can act as a key supplier of sediment in sediment-starved streams, especially in low-gradient, fine-grained streams (Wohl et al., 2015). Furthermore, channel widening is often the most common response downstream of dams, as shown in Table 1.1. However, few studies have evaluated thoroughly the very process that causes channel widening, streambank erosion. Hupp et al. (2009) observed bank erosion and widening downstream of one flood control and three hydropower dams on the Lower Roanoke River in North Carolina, USA 40+ years after dam closure. The researchers attributed channel widening to increased mean daily flows that likely resulted in increased toe erosion and slab failure. In addition, they hypothesized that bank instability attenuates downstream following dam closure, being highest near the dam immediately following closure and then migrating downstream with time (Hupp et al., 2009). A case study completed downstream of a water supply dam (Hog Park Dam) in Wyoming, USA echoed these findings, observing increased toe erosion due to an increase in median daily flow volume (Gilliam, 2011). Mohammed-Ali et al. (2020) also observed increased bank erosion downstream of Bagnell Dam used for hydropower on the Lower Osage River in Missouri, USA. However, the researchers observed

an increase in geotechnical failure due to extreme fluctuations in flow releases depending on the electrical power demand, especially in regions closer to the dam. [Schmidt & Wilcock \(2008\)](#) note the difficulty in predicting channel width response downstream of dams based on changes in the flow and sediment regimes alone, as geotechnical properties of the bank material also exerts an important control.

Identifying and isolating the true impact of dams on downstream channels is difficult given the variability and range of conditions inherent to various watersheds and their streams. This issue is perhaps best summarized by [Graf \(2006\)](#) as a “fluvial Gordian knot (p. 343).” In order to advance the scientific understanding of the complex nature of geomorphic adjustment downstream of dams and their effects on stream ecology, both [Petts & Gurnell \(2005\)](#) and [Graf \(2005\)](#) recommend more case studies to provide a thorough evaluation of the measured adjustments along specific reaches that occur over long time periods (>10 years). Studies such as these are vital to improve the current spatiotemporal generalizations and predictions of the downstream effects of dams ([Graf, 2005](#)).

1.3 Research Questions and Objectives

While the literature clearly demonstrates that installation of dams on alluvial rivers affects the geomorphic stability downstream, to what extent depends on a number of factors including, but not limited to, dam operation, time since installation, distance from dam, hydrophysiographic region, and/or local variability of site conditions. These factors of variability make it difficult to consistently predict physical geomorphic change and to manage its secondary impacts on important abiotic and biotic factors of aquatic ecosystems such as water quality, aquatic and riparian habitats, and species diversity and richness.

The overarching question that drove the following report of research is:

Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how?

The following questions were answered in this research:

1. Is there a shift in downstream streambank erosion process following large dam installation?
2. Is there a bankfull condition downstream of a large dam?
3. Can stream practitioners use basic, readily-available numerical modeling tools to assess downstream geomorphic change following dam installation and thus assist in channel erosion reduction strategies?

To answer these questions, a reach-scale case study was conducted downstream of Kanopolis Dam, a flood control dam located in central Kansas, USA. The following research objectives were completed:

- Evaluate the effects Kanopolis Dam closure has had on flow and channel bed stability up to 111 river km downstream.
- Assess the impact of Kanopolis Dam closure on downstream channel planform.
- Evaluate the effects flow regulation has had on downstream streambank erosion processes and rates.

Chapter [2](#) provides a detailed site description, a summary of past geomorphic change analyses conducted downstream of Kanopolis Dam, and a comprehensive description of the research objectives.

Chapter 2

Kanopolis Dam and the Lower Smoky Hill River, Kansas: Study Area Description and Research Objectives

2.1 Study Area Description

Kanopolis Dam forms an United States Army Corps of Engineers (USACE) impoundment on the Smoky Hill River in Ellsworth County, Kansas. The dam was officially closed on February 17, 1948 to begin storage and flow regulation to help prevent flooding downstream. The dam was originally designed to store 0.55 km^3 of water at flood pool and 0.07 km^3 at conservation pool ([KWO, 2017](#)). As of 2007, the conservation pool had lost at least 33% of its storage to sedimentation ([KBS, 2009](#); [KWO, 2017](#)). The Smoky Hill River watershed drains $51,300 \text{ km}^2$ of northwestern Kansas and a portion of eastern Colorado and combines with the Republican River at Junction City to form the Kansas River. Kanopolis Dam is located at the upper bounds of the Lower Smoky Hill watershed (HUC 10260008), as shown in Figure [2.1](#). Since the downstream effects of Kanopolis Dam are of interest, the following watershed description focuses only on Lower Smoky Hill River watershed.

The Lower Smoky Hill River watershed is located in the Central Great Plains ecoregion

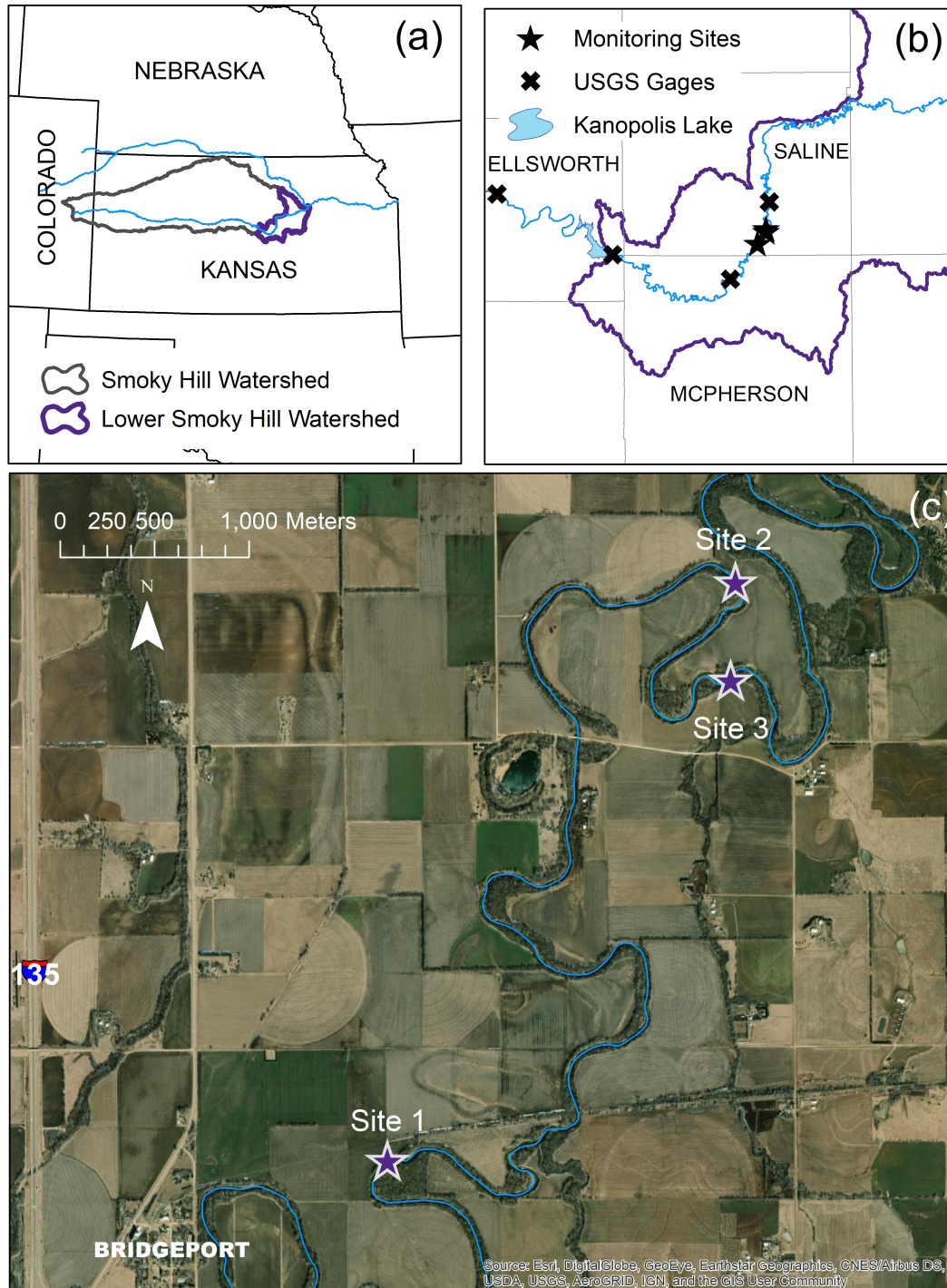


Figure 2.1: Location Map of the Smoky Hill River Monitoring Sites, Downstream of Kanopolis Dam, Central Kansas, USA. (a) Smoky Hill River Watershed, (b) Study area downstream of Kanopolis Dam; (c) Aerial images of monitoring sites.

with the majority of the area within the Smoky Hills ([Chapman et al., 2010](#)). Geology of this region consists of sandstones, limestones, and chinks ([Brosius, 2005](#)). Silt loam is the dominant soil type in the A and B soil horizons, however deposits of sandy loam material exist, especially in channel fill areas located near the Smoky Hill River (A. Layzell, KGS, personal communication, January 20, 2021). Land cover includes a mixture of grassland (49%), cropland (39%), developed land (6%), and forest (5%; [USGS, 2014](#)). The climate in this region is near a transitional zone of semi-arid, hot continental and humid subtropical and consists mainly of hot, humid summers and cold winters ([Peel et al., 2007](#)). Mean annual precipitation for this region ranges from 640 to 890 mm ([PRISM Climate Group, 2011](#)).

From 2016 to 2020, annual geomorphic surveys were conducted along three stream monitoring reaches (Sites 1, 2 and 3 in Figure 2.1; [Bigham et al., 2020](#)). Based on the analysis of these surveys, the Lower Smoky Hill River between Kanopolis Dam and the town of Assaria is a sand-bed stream with measured particle sizes ranging from coarse sand in the upper reaches to medium sand in the lower. Bank heights range from three to eight meters with the majority nearing the latter. Similar to the soil makeup of the watershed, the banks are composed mainly of silt loam soils with occasional deposits of sandy loam material, representative of channel fill. Upper bank materials of roughly 3 m in depth represent post-settlement alluvium (A. Layzell, personal communication, January 20, 2021). The Lower Smoky is a meandering river, with a high measured sinuosity (ratio of channel length to valley length) that ranges from 1.7 to 2.6. Width-to-depth ratios, measured at the 1.5-year return interval flow (see Fig. A.1 in Appendix A), ranged from 11 to 14. The channel gradient was very low, having a measured slope of 0.02% to 0.04%. Given the gradient and the bed sediment composition, the bed consists of a low energy, ripple-dune topography rather than riffle-pool.

Based on [Bigham et al. \(2020\)](#) geomorphic surveys, measured [Rosgen \(1996\)](#) stream classification types range from a B5c to a C5c-, with measurements nearing the G and F stream type classifications at some locations, likely suggesting that the Lower Smoky Hill River is transitioning from an unstable to stable channel in this region and that it may be in a widening and/or aggradational stage of channel evolution (see [Cluer & Thorne, 2014](#); [Rosgen, 2009](#); [Schumm et al., 1984](#); [Simon & Hupp, 1986](#), for channel evolution model

examples). Notable causes of instability include conversion of areas of the Lower Smoky Hill watershed from prairie to cropland in the late 1800s to early 1900s, installation of Kanopolis Lake for flood control in the 1940s, and channelization measures conducted on the Smoky Hill River through the city of Salina in the 1960s.

The region of the Lower Smoky Hill River containing the [Bigham et al. \(2020\)](#) monitoring sites has four long-term USGS stream gages (see Fig. 2.1): USGS 06864500 near Ellsworth (upstream of dam), USGS 06865500 near Langley (downstream and closest to dam), USGS 06866000 near Lindsborg, and USGS 06866500 near Mentor (furthest from dam). It is also noted that the next downstream Smoky Hill River USGS gage is located below the intersection of the Smoky Hill and the Saline River, where flow is controlled jointly by both Kanopolis Dam and the USACE-operated Wilson Dam on the Saline River, making this gage unsuitable for analysis in this study. Locations of the stream monitoring sites (sites 1-3), as well as the assessed USGS gages are provided in Table 2.1. Finally, according to the Kansas Department of Health and Environment (KDHE) 2020 303(d) List of Impaired Waters, the Lower Smoky Hill River is impaired by total suspended solids, the majority of which has been attributed to non-point sources, such as cropland and streambanks ([KDHE, 2011](#)).

Table 2.1: Summary of Stream Monitoring Sites and USGS Gage Locations on the Smoky Hill River Upstream (-) and Downstream (+) of Kanopolis Dam.

Site	Coordinates	Monitoring Period	River km from Kanopolis Dam
Ellsworth USGS	38.726648, -98.233807	1929-2020	-47.9
Langley USGS	38.611373, -97.952629	1941-2020	0.8
Lindsborg USGS	38.564289, -97.666636	1931-2020	63.9
Site 1	38.631438, -97.601311	2016-2018	86.1
Site 2	38.661292, -97.579671	2016-2020	93.4
Site 3	38.656099, -97.580865	2016-2020	94.8
Mentor USGS	38.711317, -97.571100	1925-2020	111.4

To assess the geomorphic response downstream of the degradational zone of Kanopolis Dam, streambank erosion monitoring sites were installed on the Smoky Hill River in Saline County, 86.1 to 94.8 river km downstream of Kanopolis Dam, as summarized in Table 2.1. The total drainage area at these sites ranges from 21,523 km^2 to 21,547 km^2 (Fig. 2.1).

2.2 Reported Downstream Geomorphic Effects of Kanopolis Dam

Previous research conducted by [Williams & Wolman \(1984\)](#) and [Juracek \(2001, 2011\)](#) have evaluated downstream effects of Kanopolis Dam. [Williams & Wolman \(1984\)](#) noted that since the dam was installed in 1948 to 1977, the mean daily discharge at the Langley USGS gage had not significantly changed. However the mean annual peak discharge had decreased by 58%. In addition, the 50% daily exceedance probability discharge had also decreased downstream of the dam since installation. In terms of changes to the sediment regime, [Juracek \(2011\)](#) estimated the Kanopolis Lake sediment trapping efficiency to be 95%. These changes to the flow and, especially, the sediment regime, have resulted in changes to the downstream channel size and slope, as described in [Juracek \(2011\)](#) and [Williams & Wolman \(1984\)](#).

Using long term rating curves at the Langley USGS gage, [Juracek \(2001, 2011\)](#) showed that degradation has occurred since dam completion, with the streambed dropping 1.9 m at Langley USGS gage location since dam closure in 1948 to 2010. As of 2010, degradation appeared to be continuing at the same rate as it had been since the early 1960s at about 2 cm/yr ([Juracek, 2011](#)). [Juracek \(2001\)](#) attributed this stable degradation rate to the erodible sand and gravel materials that make up the streambed, as well as a lack of natural (e.g., armoring, bedrock) or installed grade control.

Both [Williams & Wolman \(1984\)](#) and [Juracek \(2001\)](#) estimated the extent of the degradational zone downstream of Kanopolis Dam. From 1951 to 1961, [Williams & Wolman \(1984\)](#) calculated the average rate of advancement of the degradational zone downstream of Kanopolis Dam to be 0.9 km/yr, with the maximum measured degradation occurring at a distance 0.8 km downstream of the dam. As of 1961, [Williams & Wolman \(1984\)](#) believed that the end of the degradational zone was roughly 15 river km downstream of the dam. [Juracek \(2001\)](#) confirmed that the degradational zone ended somewhere between the Langley and the Lindsborg USGS gage, as a significant decrease in bed elevation did not occur between

1948 to 1965 at the Lindsborg USGS gage, located 64 river km downstream of Kanopolis Dam.

In terms of channel width, [Williams & Wolman \(1984\)](#) note that surveyed cross sections measured 0.8 to 108 river km downstream of Kanopolis Dam increased in width by as much as 15% from 1948 (dam closure) to 1961, with no distinct pattern in which this occurred (i.e., width increases were sporadic with distance). [Williams & Wolman \(1984\)](#) observed that the silty banks of the Smoky Hill River had minimal bank erosion on straight reaches, but that bank erosion appeared to be “... significant at bends or where the thalweg of the channel meandered [p.37].” [Juracek \(2011\)](#) agreed with this observation. While channel widening was not observed by [Juracek \(2011\)](#) at the Langley USGS gage as of 2010, [Juracek \(2011\)](#) did note channel widening between the Langley USGS gage and the Kanopolis Dam outflow. To illustrate this observation, Figure 2.2 was constructed to compare top of bank lines for 1991 and 2002 and were overlaid on 2019 aerial imagery. Similar to the observation made by [Williams & Wolman \(1984\)](#), increased bank erosion tended to occur along the outside of the meander bend, while the Langley USGS gage is located along a straight reach. These observed geomorphic changes, degradation followed by increase bank erosion, agree with the [Schumm et al. \(1984\)](#) channel evolution model presented in Figure 1.1.

Finally, [Friedman et al. \(1998\)](#) used the information presented by [Williams & Wolman \(1984\)](#) to summarize downstream geomorphic responses in streams of the Great Plains. As mentioned previously, [Friedman et al. \(1998\)](#) found that single-thread meandering streams tended to decrease in migration rates while braided streams tended to narrow. While [Williams & Wolman \(1984\)](#) showed that channel width had increased downstream of Kanopolis Dam, [Friedman et al. \(1998\)](#) note that no study has been conducted to determine if channel migration rates had decreased along the Lower Smoky Hill River.

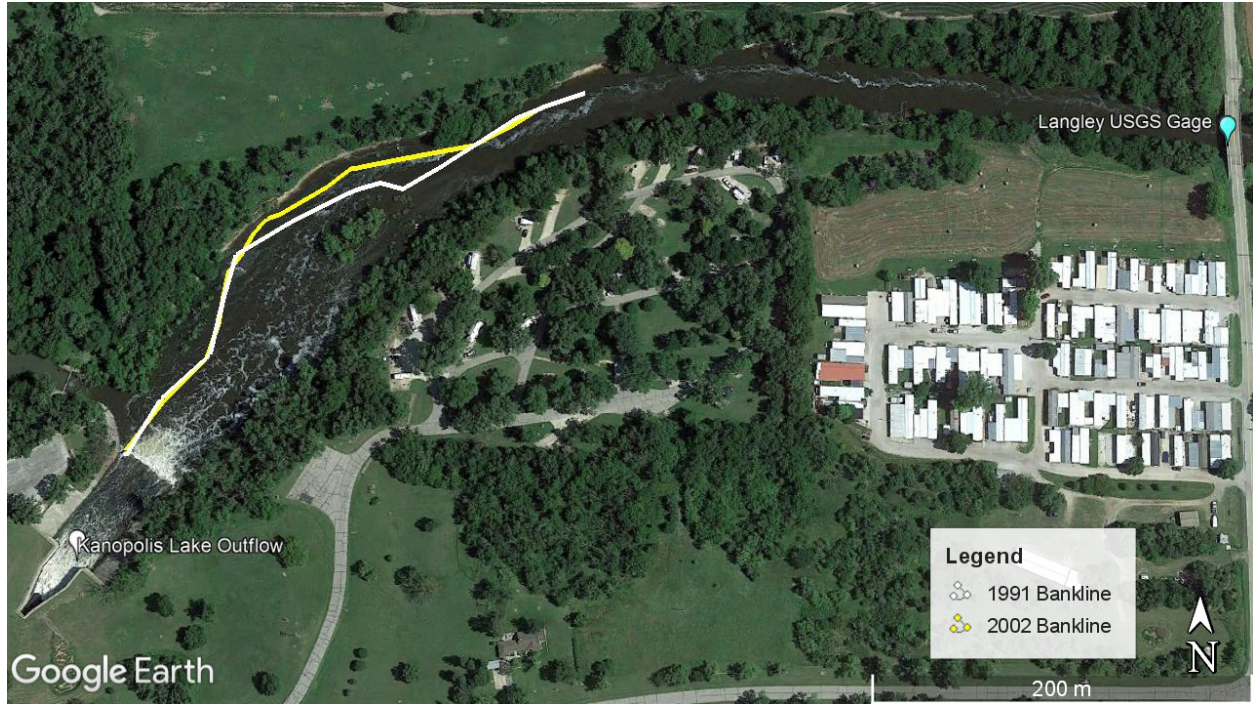


Figure 2.2: Illustration of [Juracek \(2011\)](#) Observation of Channel Widening occurring between Kanopolis Dam and the Langley USGS Gage. Note: White line represents 1991 top of bank line, yellow line represents 2002 top of bank line, while the aerial image is from 2019.

2.3 Summary of Research Objectives

The breadth and depth of geomorphic research (e.g., [Juracek, 2001, 2011](#); [Williams & Wolman, 1984](#)) and monitoring that has been conducted on the Lower Smoky Hill River (e.g., [Bigham et al., 2020](#)) makes it an ideal location for a case study analysis to address the overarching research question presented in Section 1.3, which is: Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how? Additional questions to be answered from this research include:

1. Is there a shift in downstream streambank erosion process following large dam installation?
2. Is there a bankfull condition downstream of a large dam?
3. Can stream practitioners use basic, readily-available numerical modeling tools to assess downstream geomorphic change following dam installation and thus assist in channel

erosion reduction strategies?

As summarized in Section 1.3, three main research objectives are posed to answer these questions:

- Evaluate the effects Kanopolis Dam closure has had on flow and channel bed stability up to 111 river km downstream.
- Assess the impact of Kanopolis Dam closure on downstream channel planform.
- Evaluate the effects flow regulation has had on streambank erosion processes and rates.

The first research objective is to evaluate the effects Kanopolis Dam closure has had on flow and channel bed stability. The study reach length was up to 111 river km downstream of the dam on the Lower Smoky Hill River. Reach length was based on location of (1) Bigham et al. (2020) monitoring sites and (2) nearby USGS stream gages. The first objective was completed by first assessing the effects on flow (Chapter 3) and then the effects on bed stability (Chapter 4). With respect to the flow analysis presented in Chapter 3, the work completed by Williams & Wolman (1984) was extended in:

- Space by assessing two additional downstream USGS gages located up to 111 river km downstream,
- Time by (1) extending the analysis period through water year 2020 and (2) estimating expected natural (without dam) flow conditions at the Mentor USGS gage located 111 river km downstream, and
- Content by assessing and comparing additional discharge metrics.

The hypotheses that median daily and annual peak discharge, annual high pulse count and duration, hydrograph rise and fall rates, and median annual peak date have not significantly changed since and/or due to dam installation were tested. In addition, the changes to the flow duration curves and the flood frequencies were also assessed and compared.

The second component of the first research objective was to assess channel bed stability up to 111 river km downstream of Kanopolis Dam. This was accomplished by extending the work completed by [Juracek \(2001, 2011\)](#) in both space, by adding one more downstream USGS gage, and time, by extending the bed analysis to water year 2020. These extensions to the work completed by both [Williams & Wolman \(1984\)](#) and [Juracek \(2001, 2011\)](#) provide the necessary insight required to address the research questions regarding (1) bankfull and (2) if changes in bank erosion processes and rates have occurred following dam installation.

The second objective was to assess changes in channel planform along a length of the Lower Smoky Hill River extending 111 river km downstream of Kanopolis Dam since its closure in 1948 and is presented in Chapter 5. Channel planform was assessed with two metrics: (1) channel migration rates and (2) channel sinuosity. Methods employed are similar to those of [Shields Jr. et al. \(2000\)](#), [Legleiter \(2015\)](#), and [Gilliam \(2011\)](#). The channel planform assessment was accomplished through GIS analyses of aerial photographs taken in 1985, 2004, and 2019, as well as the geological historical analyses of the Lower Smoky Hill River valley completed by [Dort \(2009\)](#) for points in time between 1857 and 1951. This assessment provides pertinent information to answer the research question if and how streambank erosion has changed since dam installation.

The last objective of this study was to evaluate the effects of flow regulation on streambank erosion processes and rates and is presented in Chapter 6. To assess the effects of flow regulation on streambank erosion, a one-dimensional (1D) hydraulic bank erosion, and sediment transport model was combined with field geomorphic surveys conducted by [Bigam et al. \(2020\)](#) between the Lindsborg and Mentor USGS gages. Flow hydrographs of the expected natural and observed flow conditions at the Mentor USGS gage obtained through the first objective were simulated to compare both bank erosion processes and rates, as well as changes in sediment transport and bed stability with and without the effects of flow regulation.

The bank erosion analysis focuses on comparing two commonly dominant mechanisms of streambank erosion: fluvial erosion and mass wasting due to geotechnical failure. Fluvial erosion, or fluvial entrainment, is the removal of sediment due to an excessive applied hydraulic

force that is able to move the *in situ* sediment (Simon et al., 2000). Geotechnical failure is a result of a streambank’s shear strength and the gravitational and hydraulic forces that are acting on it (Simon et al., 2000). Fluvial erosion and geotechnical failure are influenced by the streambank’s sediment composition, pore-water pressure, bank geometry (Simon et al., 2000), vegetation type and density (Pollen, 2007), and location along the stream, as well as the stream’s water stage, slope (Simon et al., 2000), and channel evolution phase (Schumm et al., 1984). Both fluvial erosion and geotechnical failure are modeled by the bank erosion model adopted herein: Bank Stability and Toe Erosion Model (BSTEM). Results from this evaluation provide additional insight to answer all the research questions presented here.

Finally, it is noted that when comparing these study objectives and methods to those of case studies conducted by Gilliam (2011) downstream of Hog Park Creek Dam in Wyoming, USA and Mohammed-Ali (2020) downstream of Bagnell Dam in Missouri, USA, similarities exist. Table 2.2 provides a summary of how this study differs from these previous case studies. When comparing study objectives across these three studies, this research takes a broader systems analysis approach, considering more factors that might result in a change in bank erosion, such as the flow regime, bed stability, bank composition, and bank erosion process. In addition, Table 2.3 provides a summary of case study characteristics. This study is most similar to the case study characteristics of Bagnell Dam (Mohammed-Ali, 2020); however, differences in bank erosion are likely to be observed due to differences in each dam’s operation (i.e., Bagnell Dam is used for hydropower), as well as their location in different ecoregions. By contrast, this study is less comparable to that of Hog Park Dam (Gilliam, 2011), given the differences in the dam’s purpose (water supply), drainage area, ecoregion, streambed sediment, bank height, and study reach distance from dam. Finally, both studies provide recommendations to change dam operations with the intent to minimize the downstream impact to channel stability and/or bank erosion. However, neither of these studies consider the alternative, and likely more reasonable option, which is to maintain or minimally change the existing dam operation. While it may seem simple to change how a dam releases its stored runoff, this idea ignores the fact that doing so may yet again result in another anthropogenic-induced change to channel stability. This study is unique in that

it provides recommendations on if and how stream practitioners can manage the unintended impact of continued flow regulation.

Chapters 3 through 6 provide results and discussion of each of the aforementioned study objectives. Chapter 7 summarizes and discusses these results collectively and answers the questions posed in this research.

Table 2.2: Comparison of Study Objectives Across Three Bank Erosion Studies Downstream of Dams.

Kanopolis Dam Study Objectives	Hog Park Dam (Gilliam, 2011) Study Objective? (Y/N)	Bagnell Dam (Mohammed-Ali, 2020) Study Objective? (Y/N)	Notable Differences and/or Comments
Assess the hydrological impact of dam installation.	Yes	No	Changes to the hydrogocial regime can result in changes in bank erosion (Hupp et al., 2009). Pre-dam measured flow data does not exist for Hog Park Dam.
Assess the dowsntream bed stability effect of dam installation.	No	Yes	Dam installation often causes bed instablity (Williams & Wolman, 1984) which can affect bank erosion rates (Simon & Hupp, 1986). Bed stability was assessed only through sediment transport modeling below Bagnell Dam.
Assess the impact to channel planform following dam installation.	Yes	No	Migration rates often decrease downstream of dams (Shields Jr. et al., 2000). This study employs same methods as Hog Park Dam. Kanopolis Dam results will add another case study to the downstream dam effect literature.
Assess the impact to streambank erosion follwing dam installation.	Yes	Yes	Hog Park Dam study did not obtain any geotechnical information of eroding streambanks. Bagnell Dam study only assessed bank failure due to geotechnical failure, not fluvial erosion.

Table 2.3: Comparison of Case Study Characteristics Across Three Bank Erosion Studies Downstream of Dams.

Case Study Characteristic	Kanopolis Dam	Hog Park Dam (Gilliam, 2011)	Bagnell Dam (Mohammed-Ali, 2020)
Primary Purpose	Flood Control	Water Supply	Hydropower
Drainage Area (km^2)	20,300	32	36,110
Level 1 Ecoregion	Great Plains	Northwestern Forested Mountains	Eastern Temperate Forests
Average Streambed Sediment (D_{50} , mm)	Sand (0.5)	Gravel (38)	Gravel (22)
Average Bank Height (m)	8	1.5	12
Distance from Dam of Study Reach (km)	111	5	130
Modeling Approach	BSTEM + 1D Flow & Sediment Transport	BSTEM	BSTEM + 1D Flow & Sediment Transport

Chapter 3

Objective 1a: Kanopolis Dam Flow Regulation Assessment

The first study objective was to assess the hydrologic impact of Kanopolis Dam on the Lower Smoky Hill River from the Langley USGS gage to the Mentor USGS gage, located 0.8 to 111 river km downstream of the dam, respectively (Table 2.1). This reach length was based on location of (1) [Bigham et al. \(2020\)](#) monitoring sites and (2) nearby USGS stream gages. Evaluation of the hydrologic impact of Kanopolis Dam was accomplished by (1) summarizing and statistically assessing available measured flow data along USGS gages on the Smoky Hill River near Kanopolis Dam and (2) comparing an expected natural hydrograph (no dam scenario) to the measured hydrograph (with dam scenario) at the USGS gage near Mentor. The following sections describe the methods and results of these analyses.

3.1 Methods

3.1.1 Pre to Post-Dam Flow Data Summary and Statistics

To assess change in flow conditions following the installation of Kanopolis Dam, daily and annual peak flow data were obtained from the Langley (06865500), Lindsborg (06866000), and Mentor (06866500) USGS stream gages, located downstream of Kanopolis Dam, and the

Ellsworth (06864500) USGS stream gage, located upstream of Kanopolis Dam (see Table 2.1 and Fig. 2.1). Flow data for these gages were limited to the time periods of measured flow rates, which ranged from 7 to 92 years, as well as changes in gage location. Locations of gages have changed for both the Lindsborg and Mentor USGS gages. It is unknown where the exact location of Lindsborg gage was prior to 2014 (C. Painter, USGS, personal communication, September 27, 2021), and the Mentor gage was moved to four different locations within 20 km of the current location since 1925 (C. Painter, USGS, personal communication, August 17, 2021). Variable measurement time periods, as well as the changes in locations of two of the assessed gages, have an effect on both flow data summaries and statistical analyses.

Given the flow data available, as well as consideration of the aforementioned limitations, pre to post-dam flow analyses and comparison completed for all four gages are summarized in Table 3.1. These discharge metrics were selected based on review of Richter et al. (1996) who developed a hydrologic assessment software called Indicators of Hydrologic Alteration (IHA) to easily and efficiently assess key hydrological components of flow magnitude, duration, frequency, and timing using available daily flow measurements. Richter et al. (1996) also provides a case study application of the IHA software using daily flow data from the Roanoke River, a regulated river in North Carolina, to compare pre to post-dam hydrologic impacts.

Table 3.1: Flow Analyses Conducted with Mean Daily Flow and Annual Peak Flow Data from Smoky Hill River USGS Gages.

Statistic	Magnitude	Duration	Frequency	Timing
Daily Flow	-Median daily discharge (cms)	-Median high flow duration (days) -Flow duration curves	-Median high flow counts (#/yr) -Flow duration curves	-Median rising and falling limb rates (cms/day) -Median date of peak discharge
Annual Peak Flow	-Median annual peak discharge (cms) -Flood frequency analyses	N/A	-Flood frequency analyses	N/A

The start of water year 1949 (beginning October 1, 1948) was used as the cut-off year between pre and post-dam time periods. In terms of annual flood frequency analysis, a Log-Pearson Type III Flood Frequency Analysis was conducted on annual peak flows to obtain the 1, 2, and 25-year return interval discharges, pre to post-dam installation at all

assessed USGS gages. The Log-Pearson Type III analysis is a preferred method to obtain annual peak flow return intervals as it utilizes statistical parameters to extrapolate return intervals beyond the extent of available USGS gage records (USGS, 1982). The software RIVERMorph (Stantec, 2013) was used to conduct flood frequency analyses at the assessed USGS gages using the localized recommended skew coefficient of -0.2, as described in USGS (1982).

The IHA software was employed to obtain all other discharge metrics, such as the pre and post-dam flow duration curves, median high flow counts per year, median high flow duration in days, median date of the annual peak flow, and the median hydrograph rise and fall rates in cms/day. High flows were defined as greater than the bankfull discharge at the Ellsworth USGS gage, which was determined directly from the flood frequency analysis to be 100 cms (1.3-yr return interval, see Fig. A.2 in Appendix A).

Except when evaluating daily and annual flow frequencies, the non-parametric Mann Whitney U Test was used to compare changes in all other discharge metrics pre to post-dam installation at a 5% significance level at all assessed USGS gages. The Mann Whitney U Test was selected instead of a Student's t test as flow data were not normally distributed (MacFarland & Yates, 2016). An example of the SAS code used to assess pre to post-dam discharge metrics using the Mann Whitney U Test is provided in Appendix C in Section C.1.

In addition, a Before-After-Control-Impact (BACI) analysis was conducted using the Ellsworth (control) and Langley (impact) USGS gages measured flow data. A BACI analysis is a common tool used to assess how anthropogenic environmental modifications impact some measurable environmental factor(s), often times using t-tests or analysis of variance to compare the factor at a modified site (before vs. after) to a control site (control vs. impact) (Conner et al., 2016; Eberhardt, 1976; Gillespie et al., 2015). In this case study, the Ellsworth (control) and Langley (impact) USGS gages flow data were used to determine if downstream observed changes in the discharge metrics listed in Table 3.1 pre to post-dam were due to the impact of the dam and not other outside factors (e.g., climate variability). The BACI analysis was statistically assessed using the Mann Whitney U Test at a 5% significance level for all discharge metrics except for daily and annual flow frequency analysis results.

Finally, to further assess the hydrologic impact of Kanopolis Dam and its possible geomorphic implications, two parameters were calculated. First, a standardized percentage change (P_c) in the median annual peak flow pre₍₁₎ and post-dam₍₂₎ was calculated between the upstream Ellsworth (Y) and downstream Langley (X) USGS gage using Equation 3.1:

$$P_c = \left(\frac{X_2/X_1}{Y_2/Y_1} - 1 \right) * 100 \quad (3.1)$$

Lastly, the ratio of the maximum reservoir storage at flood-pool to the mean annual runoff measured at the Ellsworth USGS gage was calculated to provide an indication of potential geomorphic impact downstream of Kanopolis Dam as described by Graf (1999).

3.1.2 Post-Dam Actual versus Expected Natural Flow Summary and Statistics

As previously mentioned, pre-dam measured flow data (prior to 1948) is limited along the Lower Smoky Hill River, as shown in Table 2.1. Furthermore, Rahmani et al. (2015) showed that annual precipitation in this region is increasing at a rate of 19 mm/yr since 1970, attributing this increase to human-induced climate change. Therefore, the expected natural flow hydrograph since dam construction was modeled to represent what flow conditions could have been if Kanopolis Dam had not been installed. In other words, the modeled expected natural flow condition represents an unregulated river. This expected natural hydrograph (without dam) was compared to the actual observed (with dam) hydrograph to provide an additional line of evidence in the flow impact analysis downstream of Kanopolis Dam. Expected natural flow data were calculated for the period of February 18, 1948 to December 31, 2019 with Equation 3.2:

$$Q_{nat} = Q_{obs} + \sum_{i=1}^3 R_{i-1} (Q_{in} - Q_{out})_{1-i} \quad (3.2)$$

Where Q_{nat} represents the expected mean natural flow (cms) for a specific day, Q_{obs} represents the observed mean flow (cms) at the Mentor USGS gage for the same day, Q_{in} and Q_{out} represents the mean inflow and outflow (cms) of Kanopolis Dam for day $i-1$, respectively, and R represents the routing coefficient for day $i-1$. Q_{in} was obtained from the USACE reservoir inflow database (B. Twombly, USACE, personal communication, December 16, 2021), while Q_{out} was obtained from the Langley USGS gage. Furthermore, the assumed routing coefficients used were based on USACE annual assessment of downstream expected natural flow below Kanopolis Dam and were assumed to be 0.0, 0.4, 0.6 for Day 0 ($i=1$), 1 ($i=2$), and 2 ($i=3$), respectively. These routing coefficients were developed and calibrated using HEC-ResSim, a hydrologic model used to assess various reservoir operation schemes (USACE, 2021b). Finally, the difference between Q_{in} and Q_{out} is dependent upon the three days of inflow and outflow data relative to a specific day, with 0 ($i=1$) representing the current day's difference, -1 ($i=2$) representing the previous day's difference, and -2 ($i=3$) representing the difference in flow two days prior.

As described in Section 3.1.1, the IHA software was used to characterize the metrics presented in Table 3.1 for actual and expected natural flows. Except when evaluating flow frequencies, the Mann Whitney U Test was used to compare differences in all other flow metrics presented in Table 3.1. A Log-Pearson Type III Flood Frequency Analysis was conducted on the expected natural flow conditions without dam regulation to compare to actual. The Mentor USGS gage will be the focus location for this section, given that (1) it has a long post-dam flow observation dataset and (2) it is the furthest location of interest downstream of Kanopolis Dam in this study.

3.2 Results and Discussion

3.2.1 Before-After-Control-Impact (BACI) Analysis of Flow at the Ellsworth and Langley USGS Gages

Metrics characterizing the flow regime of the Smoky Hill River prior to and following installation of Kanopolis Dam in February of 1948 are summarized in Tables 3.2 for the upstream Ellsworth USGS gage and the downstream Langley USGS gage. The BACI analysis indicates a statistically significant decrease in high flow counts, rise and fall rates, and median annual peak discharge since dam construction at the Langley USGS gage located 0.8 km downstream of the dam. A statistically significant increase in high flow duration was also noted at the Langley USGS gage. These same results were obtained when comparing the post-dam flow measurements at the Langley USGS gage to the control Ellsworth USGS gage (located upstream of the dam), confirming that these observed changes in the flow regime are due largely because of the dam and not some other outside factor.

Table 3.2: Before-After-Control-Impact Analysis (BACI) comparing the Pre to Post-Dam Flows at the Langley USGS Gage (Langley column comparison) and Langley Post-Dam Flows to the Ellsworth USGS Gage Flows (grey-filled columns).

Flow Parameter	Langley USGS Gage (Impact)		Ellsworth USGS Gage (Control, 1929-2020)
	Pre-Dam (1941-1948)	Post-Dam (1949-2020)	
Median Daily Discharge (cms)	2.2	2.1 ^N	1.7*
1% Exceed. Probability (cms)	94	83	100
95% Exceed. Probability (cms)	0.4	0.4	0.3
Median High Pulse Count (#/yr)	2.5	0.0*	1.0*
Median High Pulse Duration (days/yr)	1.5	4.0*	2.0*
Median Hydrograph Rise Rate (cms/day)	0.18	0.06*	0.14*
Median Hydrograph Fall Rate (cms/day)	-0.21	-0.06*	-0.14*
Median Date of Annual Peak Flow	7-Sep	2-Jul ^N	2-Jul ^N
Median Annual Peak Discharge (cms)	221	51*	210*
1-yr Return Interval (cms)	19	2.7	10
2-yr Return Interval (cms)	187	44	215
25-yr Return Interval (cms)	860 ¹	170	811

¹Note: 25-yr return interval based on 7 years of flow data; *Significant difference between pre to post-dam flows or post-dam to control flows ($\alpha = 0.05$); ^NComparison conducted but a significant difference was not detected ($\alpha = 0.05$).

Figures 3.1 and 3.2 provide visualizations of changes in the flow regime following dam

construction. Figure 3.1 compares measured discharge from the Ellsworth USGS gage upstream of Kanopolis Dam to the measured discharge downstream of Kanopolis Dam at the Langley USGS gage from 1929 to 2020, while Figure 3.2 compares the flow duration curve for pre to post-dam at the Langley USGS gage and at the control Ellsworth USGS gage. Both Figure 3.1 and the calculated return intervals shown in Table 3.2 illustrate the dampening of peak flows following dam construction. The standardized percentage change (P_c , see Eq. 3.1) in flow between the Ellsworth and Langley USGS gage indicated a 72% decrease in median annual peak flow since dam installation, which is a similar result to that found by Williams & Wolman (1984) when comparing pre-dam flows to post (1948-1977). Results shown in Table 3.2 agree with this finding showing a significant decrease in the the number of high flow events since dam installation.

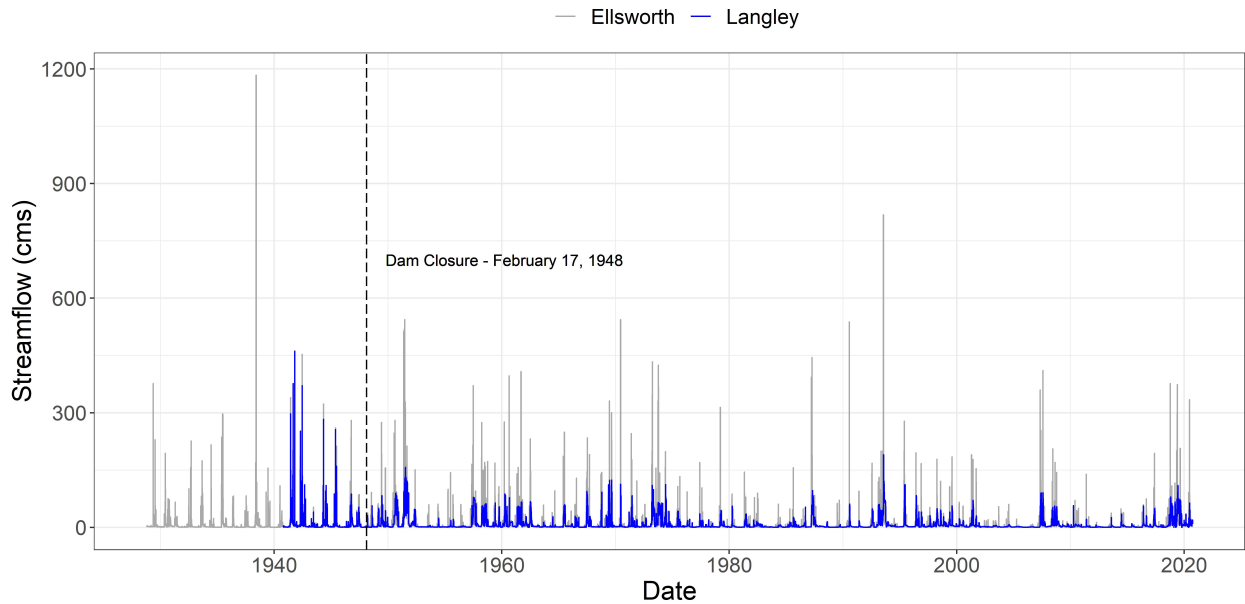


Figure 3.1: Recorded Flow at the Ellsworth (grey) and Langley (blue) USGS Gages, Upstream and Downstream of Kanopolis Dam, Respectively, 1929-2020.

Figure 3.2 also shows that peak flows ($>2\%$ daily exceedance probability) have dampened while more moderate flows ($5-90\%$ daily exceedance probability) have slightly increased. It is noted that this change in moderate flows may not be significant, as shown by the comparisons of median daily discharge in Table 3.2; however, this observation suggests the magnitude of base flows have increased in order to offset the reduction in peak flows. Similarly, results

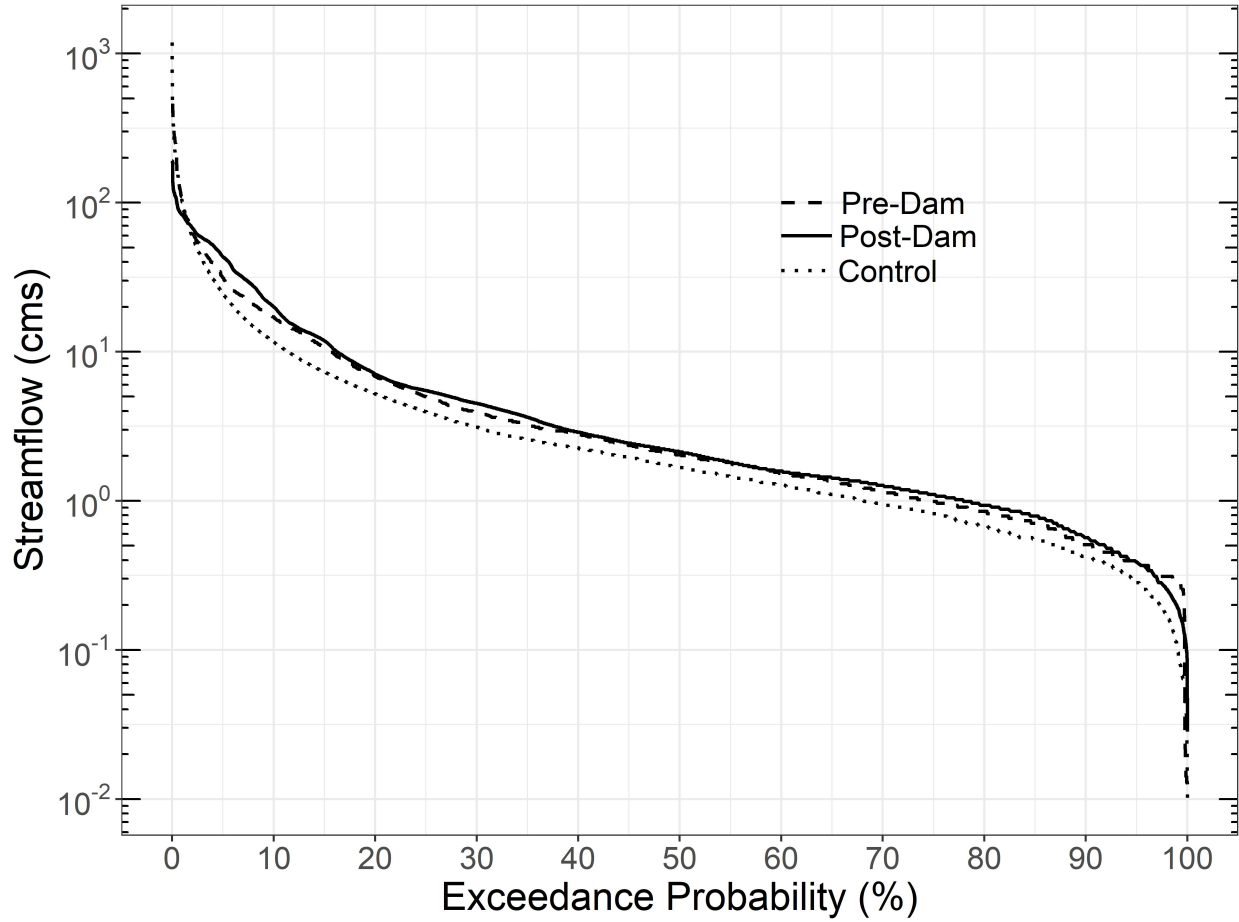


Figure 3.2: Flow Duration Curves at the Langley USGS Gage, Pre-Dam (1941-1948) to Post-Dam (1949-2020), and the Ellsworth USGS Gage, Control (1929-2020)

summarized in Table 3.2 also show that the duration of high flows has increased while the rates of the hydrograph rising and falling limbs have decreased, suggesting that hydrographs have become more “step-like” versus “flashy” in shape. Figure 3.3 provides an example of the shift in hydrograph shape. These flow effect observations are similar to other hydrologic studies downstream of flood control dams (e.g., [Hupp et al., 2009](#); [Ma et al., 2012](#); [Richard et al., 2005](#)). Finally, the median date of the annual peak flow did not significantly change following dam closure.

Lastly, the ratio of maximum flood storage to mean annual runoff at the Ellsworth USGS gage, as described by [Graf \(1999\)](#), was calculated to be 2.20 years, meaning that Kanopolis Dam had (at the start of storage, pre-sedimentation) the ability to store up to 2.20 years

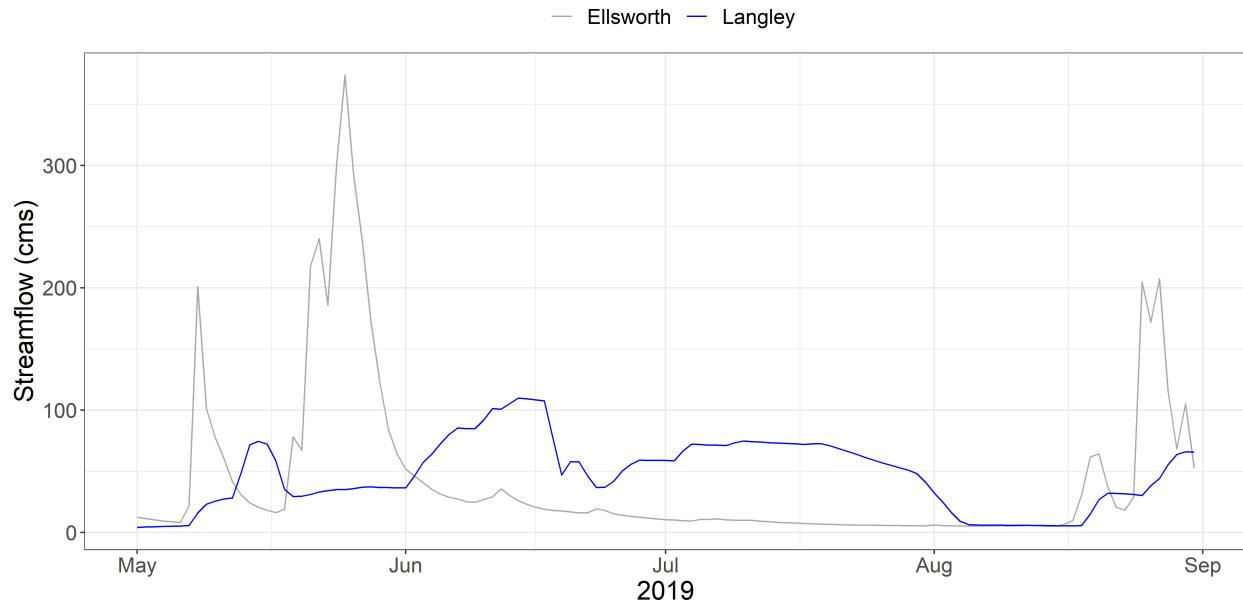


Figure 3.3: Example of Step-Like (flow-regulated Langley USGS gage) versus Flashy (un-regulated Ellsworth USGS Gage) Hydrograph Shape, May-September 2019.

worth of the average runoff volume, if needed, for flood control. Based on [Graf \(1999\)](#) analysis, ratios greater than one characterize streams that will likely experience the greatest change in flow regime, and thus geomorphic change, following dam installation. Again, this conclusion can be supported by the observed flow dampening downstream of Kanopolis Dam compared to upstream, as summarized in [Table 3.2](#) and shown in [Figure 3.1](#). This calculated value is similar to that of the entire Missouri River watershed, of which the Smoky Hill River is a part, which was found to be 2.51 years ([Graf, 1999](#)).

3.2.2 Flow Analysis at the Lindsborg and Mentor USGS Gages

[Table 3.3](#) provides the changes in pre to post-dam flows following dam construction at the Lindsborg and Mentor USGS gages, located further downstream. There was a statistically significant increase in median daily discharge and decrease in the hydrograph falling limb pre to post-dam at both gages. The rise rate at the Mentor USGS gage was also found to be significantly lower than pre-dam rates. In addition, the median annual peak flow decreased at the Lindsborg USGS gage. Similar to the BACI analysis conducted using Langley and

Ellsworth USGS gages, all calculated flood return intervals at these gages also decreased. The flow duration curve was developed for pre to post-dam flows at the Mentor USGS gage, the furthest gage from the dam that was analyzed, as shown in Figure 3.4. This flow duration curve looks similar to that of Figure 3.2, where mid-range flows, especially those between a 5 and 40% exceedance probability, have increased following dam construction. The hydrologic assessment results from the Lindsborg and Mentor USGS gages show that hydrologic effects of Kanopolis Dam installation continues as far as 111 river km downstream, with observed dampened peak flows, higher mid-range flows, and step-like versus flashy hydrographs. However, it is also noted that statistically significant changes in flow becomes less common with distance from the dam. Lack of statistical differences could be due to both data limitations at the Lindsborg and Mentor USGS gages, as well as natural, unregulated flow coming in from tributaries directly upstream of these gages. The observation that the effects of a dam on the flow regime decrease with increasing distance from the dam is similar to that described by [Petts & Gurnell \(2005\)](#). As was observed in the BACI analysis, the Lindsborg and Mentor USGS gage median date of the annual peak flow did not change pre to post-dam at either gages.

Table 3.3: Comparing Pre to Post-Dam Flows at the Lindsborg and Mentor USGS Gages. *Note:* Statistical analysis completed for each site's column.

Flow Parameter	Lindsborg USGS		Mentor USGS	
	Pre (1931-48)	Post (1949-65, 2015-20)	Pre (1925-30, 1948)	Post (1949-2020)
Median Daily Discharge (cms)	2.2	3.3*	3.3	3.5*
1% Exceed. Probability (cms)	114	99	120	93
95% Exceed. Probability (cms)	0.5	1.0	0.7	0.7
Median High Pulse Count (#/yr)	2.0	0.0*	1.0	0.0 ^N
Median High Pulse Duration (days/yr)	2.0	1.75 ^N	2.25	1.75 ^N
Median Hydrograph Rise Rate (cms/day)	0.28	0.28 ^N	0.44	0.17*
Median Hydrograph Fall Rate (cms/day)	-0.27	-0.14*	-0.42	-0.17*
Median Date of Annual Peak Flow	3-Jul	22-Jun ^N	1-Jul	22-Jun ^N
Median Annual Peak Discharge (cms)	147	91*	119	110 ^N
1-yr Return Interval (cms)	37	10.0	21	9
2-yr Return Interval (cms)	157	85	185	111
25-yr Return Interval (cms)	580 ¹	300 ¹	904 ¹	364

¹Note: 25-yr return interval based on <25 years of flow data; *Significant difference between pre to post-dam flows ($\alpha = 0.05$); ^NComparison conducted but a significant difference was not detected ($\alpha = 0.05$).

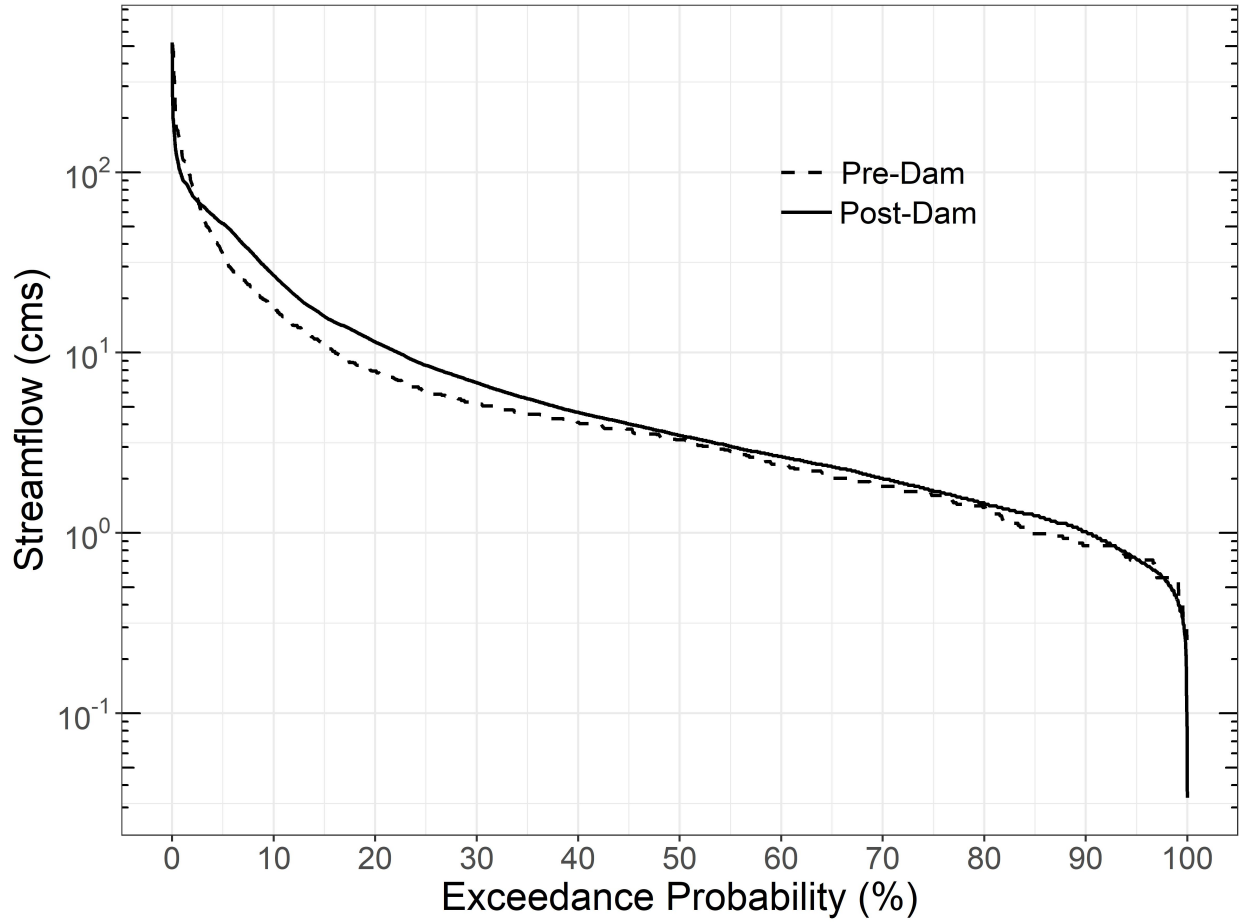


Figure 3.4: Flow Duration Curves at the Mentor USGS Gage (06866500), Pre-Dam (1925-30; 1948) to Post-Dam (1949-2020)

As previously mentioned, there are limitations to this analysis. Both the periods of record and the variable locations of the Mentor and Lindsborg USGS gages affect statistical analyses summarized here. In addition to gage effects, the USACE changed the minimum flow release requirements in 2012 to guarantee at least 0.57 cms at the Mentor gage year-round in an effort to maintain downstream river flows for water supply. Prior to 2012, a minimum flow of 0.25 cms in the winter and 1.42 cms in the summer at the Langley USGS gage was required (D. Hays, USACE, personal communication, July 9, 2021). This recent change in flow releases from Kanopolis Dam may have a long-term effect on median daily flow that cannot yet be detected through the current statistical analysis.

3.2.3 Flow Analysis at the Mentor USGS Gage: Observed versus Expected Natural Flow

Figure 3.5 compares the long-term hydrograph from February 18, 1948 (day after Kanopolis dam closure) to December 31, 2019 for the expected natural (without dam) flow from the USACE analysis and the actual flow measured (with dam) at the Mentor USGS gage. As shown in Figure 3.5, peak flows are considerably higher and more numerous for the expected natural flow conditions compared to the actual flow, which would be expected downstream of a flood control dam.

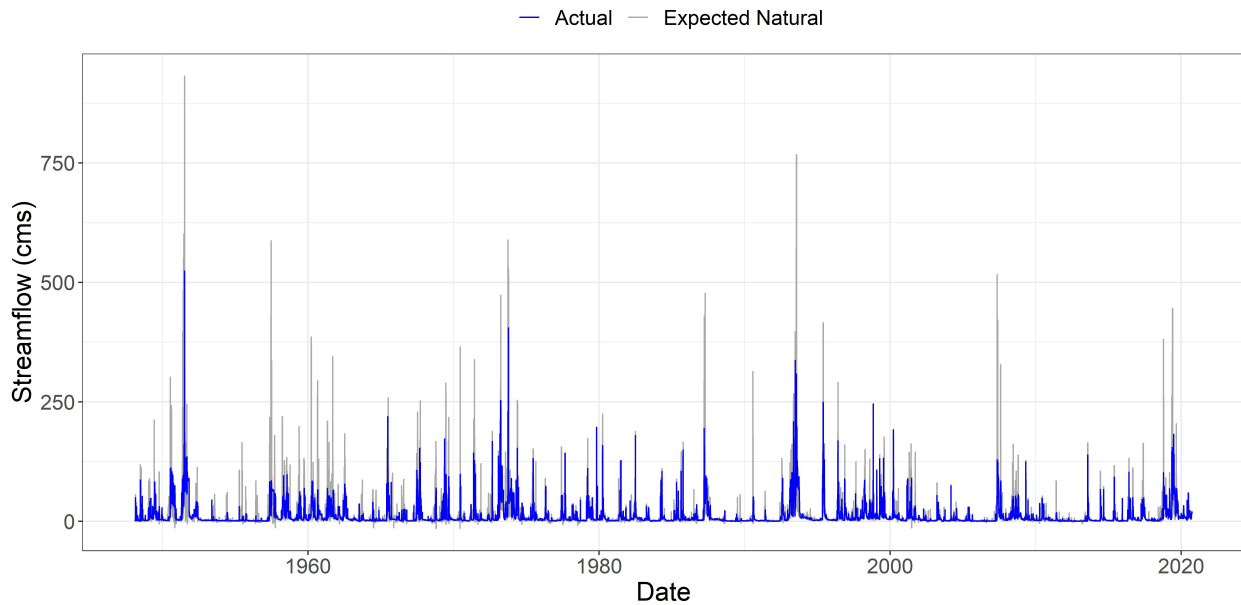


Figure 3.5: Actual Recorded Flow at the Mentor USGS Gage (06866500, blue) Compared to the Expected Natural Flow (grey) without Kanopolis Dam, 1948-2019.

Figure 3.5 also shows negative discharge values for the expected natural flow conditions. Negative discharge values are an artifact of the routing coefficient assumptions as well as uncertainties associated with flow calculations of inflow at Kanopolis Dam, flow measurements at USGS gage sites, floodplain storage, and losses due to water use and seepage (B. Twombly, USACE, personal communication, December 16, 2021). To conduct flow analyses in IHA, all negative values were converted to zero cms. Doing so effectively added a total of 0.13 km^3 of runoff (sum of negative values) to the system over the 70-year period. This

is equivalent to about 6 months of the average annual runoff volume of 0.25 km^3 . Adding runoff to the system would result in higher flow summary statistics than what likely would have occurred under true conditions. However, given that the lowest negative value is -15 cms (or 0.001 km^3 , on October 24, 1968), the expected peak flow values reported here should not be significantly higher than the true.

Table 3.4: Comparison of Actual to Expected Natural Flow Metrics at the Mentor USGS Gage (06866500), 1949-2019.

Flow Parameter	Mentor USGS Gage	
	Actual (1949-2020)	Expected (1949-2019)
Median Daily Discharge (cms)	3.45	3.52*
1% Exceed. Probability (cms)	93	151
95% Exceed. Probability (cms)	0.7	0.1
Median High Pulse Count (#/yr)	0.0	1.0*
Median High Pulse Duration (days/yr)	1.75	2.0*
Median Hydrograph Rise Rate (cms/day)	0.17	0.42*
Median Hydrograph Fall Rate (cms/day)	-0.17	-0.42*
Median Date of Annual Peak Flow	22-Jun	18-Jun ^N
Median Annual Peak Discharge (cms)	110	<i>160*</i>
1-yr Return Interval (cms)	9	<i>12</i>
2-yr Return Interval (cms)	111	<i>151</i>
25-yr Return Interval (cms)	364	<i>617</i>

*Significant difference between actual to expected natural flows ($\alpha = 0.05$); ^NComparison conducted but a significant difference was not detected ($\alpha = 0.05$); Italicized text in the expected natural column was based on mean daily flows and not instantaneous peak flows.

Table 3.4 provides the summary statistics comparing actual flow (with dam) conditions to expected natural (without dam) flow conditions. It is noted that the mean daily flow data were used to calculate annual flow statistics (median annual peak discharge and flood frequency analysis) instead of the instantaneous annual peak flow for the expected natural flow condition. Therefore, values summarized for these parameters are likely *lower* than true conditions. Furthermore, as mentioned in Section 3.1.2, runoff was added to the expected natural system so that negative discharge values were not included in the statistical analysis, resulting in potentially *higher* than true flow summary statistics. With these two limitations in mind, significant differences observed included a higher number of 100 cms or greater discharges (i.e., high pulse), a higher duration of these high pulses, a higher median daily

and annual peak discharge, and more rapid hydrograph rise and fall rates without the dam versus with the dam. In addition, the median date of the annual peak flow was not found to be significantly different between the expected natural and actual flow conditions. These results are comparable to those obtained pre to post-dam conditions at the Mentor USGS gage as described in Section 3.2.2 and summarized in Table 3.3.

When comparing flow duration curves for these two conditions (Figure 3.6), expected natural daily flow conditions were found to be higher on average, instead of lower as shown in Figure 3.4 for pre and post-dam periods at the Mentor USGS gage, except at greater than 75% exceedance probability. The difference in flow duration curves in Figure 3.6 shows that high flows are not controlled/reduced in the expected natural (without dam) flow regime while observed high flows (with dam) have been reduced due to operation of Kanopolis Dam. Furthermore, because of the reduction in low probability daily flows under controlled conditions by Kanopolis Dam, runoff must be released at a higher flow rate as shown by the high exceedance probability mean daily flow rates shown in Figure 3.6. These results differ from Figure 3.4 slightly, where on average, low probability daily flows were higher post-dam than pre-dam except at less than 5% exceedance probability. Observed differences could be due to the relatively short pre-dam record at the Mentor USGS gage, as well as the assumptions that were made here to analyze the expected natural flow conditions (e.g., expected natural curve ends at 95% exceedance probability due to negative flow assumption and adjustments).

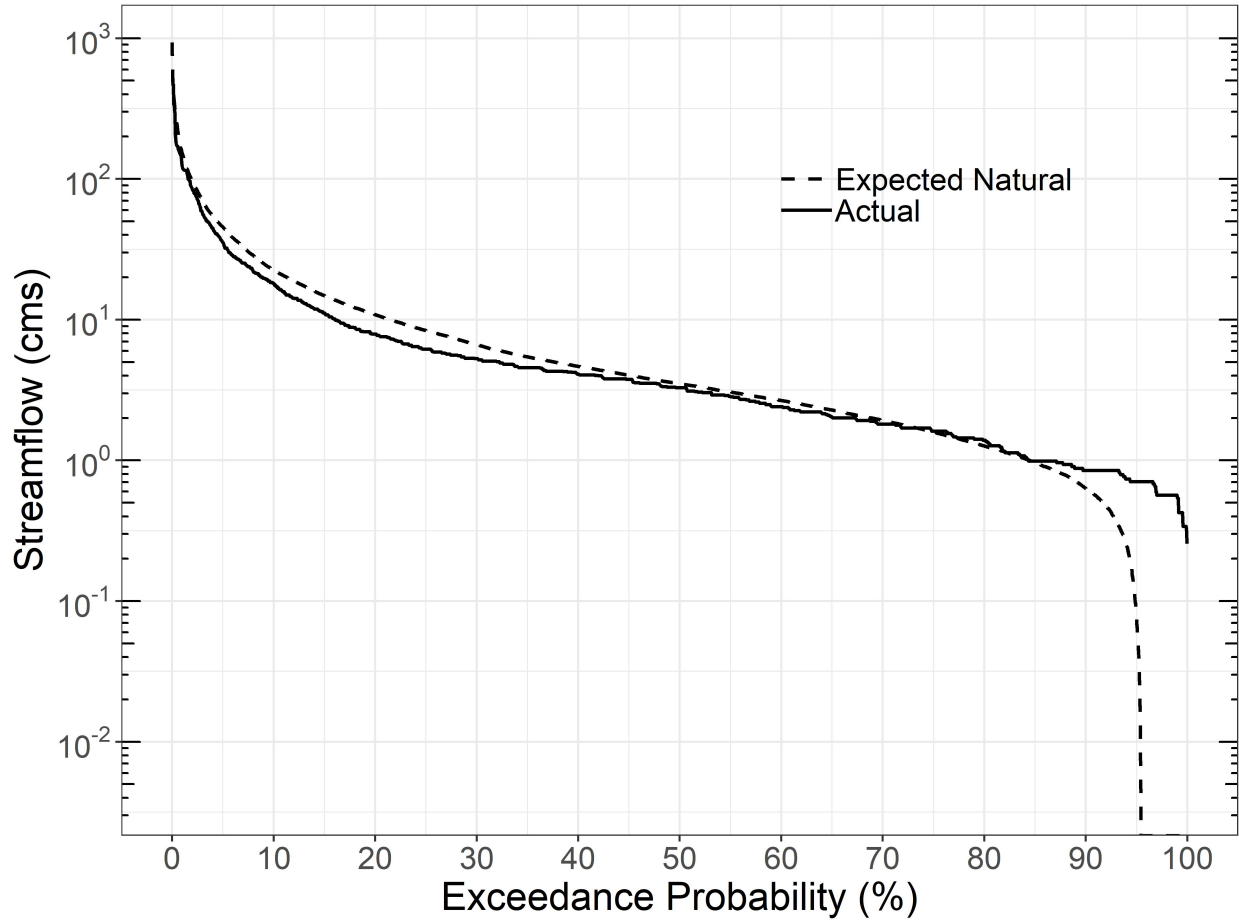


Figure 3.6: Flow Duration Curves at the Mentor USGS Gage (06866500) for Expected Natural (1949-2019) to Actual (1949-2020)

3.3 Summary

In this chapter, the downstream effects on the flow regime following Kanopolis Dam installation were summarized and statistically analyzed. The following conclusions can be drawn from this analysis for the entirety of the study reach, from the dam to 111 river km downstream at the Mentor USGS gage, unless otherwise noted below:

- Peak flows have been dampened by 72% with high flows (>100 cms or the 1.3-year return interval at the Ellsworth USGS Gage) occurring less often.
- Flow duration curves have not changed substantially from pre to post-dam flow conditions, however, moderate flows (5-40% daily exceedance probability) tended to be

higher and high flows tended to be lower.

- Hydrographs have become more step-like versus flashy in shape with general increase in duration of high flows at locations closest to the dam and slower rising and falling limbs at all locations.
- Timing of the annual peak flow event has not changed and continues to occur in the summer months.

The results from this analysis will be used in Chapter 7 to answer the questions related to bankfull and if and how bank erosion process and rates have changed following dam installation. The next chapter provides methods and results of the streambed stability analysis up to 111 river km downstream of Kanopolis Dam.

Chapter 4

Objective 1b: Channel Bed Stability Analysis Downstream of Kanopolis Dam

This chapter evaluates the channel bed stability downstream of Kanopolis Dam from the Langley USGS gage to the Mentor USGS gage (Table. [2.1](#)) This was accomplished by evaluating long-term rating curves at each gage to assess how stage changes overtime at a pre-selected discharge. The following sections provide additional details about the methods employed, as well as the results obtained from each USGS gage.

4.1 Methods

To assess bed stability, long-term rating curve data were obtained for the Langley (06865500), Lindsborg (06866000), and Mentor (06866500) USGS stream gages, located downstream of Kanopolis Dam, and the Ellsworth (06864500) USGS stream gage, located upstream of Kanopolis Dam (see Table [2.1](#) and Fig. [2.1](#)). The analysis was completed by selecting a low flow, reference discharge and plotting the respective stage for the selected flow on a time-series plot to observe bed stability overtime at each gage as described by [Juracek &](#)

[Fitzpatrick \(2009\)](#). Since this was an extension to the work completed by [Juracek \(2001, 2011\)](#), the same reference discharges, which were mean annual discharges of 5.7, 8.5, and 11.3 cms for the Ellsworth, Langley, and Lindsborg USGS gages, respectively, were used again here. The mean annual discharge was also determined for the Mentor USGS gage since the analysis done by [Juracek \(2001, 2011\)](#) did not extend this far downstream. The post-dam mean annual discharge was found to also be 11.3 cms, due to rounding, and was set as the reference discharge for the Mentor USGS gage.

Based on the available stage and discharge data, historical information about each USGS gage location and its datum were evaluated for the periods between water year 2010 to 2020 at the Ellsworth and Langley USGS gages, water year 2015 to 2020 at the Lindsborg USGS gage, and water year 1991 to 2020 at the Mentor USGS gage. No major changes had occurred to location or datum at the Ellsworth and Langley USGS gages since water year 2010 (C. Painter, USGS, personal communication, September 27, 2021). The current location and datum of the Lindsborg USGS gage reflect conditions since the gage was reinstalled in 2014; therefore, the rating curve and bed stability analysis was conducted only for water years 2015 through 2020. [Juracek \(2001\)](#) provides a time-series plot of stage at the reference discharge from 1935 to 1965 at a previous location of the Lindsborg USGS gage; however, the new location and datum cannot be tied into the old gage location and datum due to lack of its previous site description (C. Painter, USGS, personal communication, September 27, 2021). Therefore, only relative changes in elevation during these two time periods were compared.

Finally, the Mentor USGS gage was located 19 km downstream from its current location from water year 1991 until March 7, 2002 (C. Painter, USGS, personal communication, August 17, 2021). While datum shifts are available and are tied into one another, different locations overtime make it difficult to compare bed channel changes directly. Furthermore, only mean daily discharge and mean daily stage were available from water year 1991 to 2007, instead of an instantaneous measured stage at 11.3 cms. This would result in inaccurate stage measurements for a given flow, as it would be influenced by flow fluctuations that may have occurred throughout each individual day. Considering these limitations, reference stage over the entire data period (1991-2020) was plotted for the Mentor USGS gage; however, only

bed stability results from instantaneous stage readings (i.e., 2008-2020) were statistically analyzed.

The Box-Jenkins autoregressive integrated moving average (ARIMA) model was used to analyze the univariate time-series model to (1) test for a time trend and (2) if a time trend was observed, predict future stage changes up to year 2030 based on past and present stage data points. The ARIMA model is the recommended statistical analysis if data appears to have a time trend ([Hamilton, 1994](#)). An ARIMA model can contain three parts: (1) the autoregressive component (p), (2) the integrated component (d), and (3) the moving average (q, [SAS, 2018](#)).

One assumption that the ARIMA model approach requires are equally-spaced time intervals. The reference stage summarized in these time-series plots occurred randomly on unequally spaced dates. To achieve equally spaced time intervals, the reference stage data was first assumed to be true for the entire year in which it was observed. If there were more than one observation of reference stage in a given year, all but the last observation for that year was removed from the dataset. Lastly, for years in which the reference stage was not observed, reference stage information was estimated using linear interpolation between the closest years prior and after.

Statistical tests of stationarity, model fitting, and forecasting intervals were assessed using a 5% significance level. Following proper procedures as described by [SAS \(2018\)](#), the integrated component (d) was set to 1 to transform all reference stage datasets to a stationarity series. Additionally, the autoregressive component (p) was set to 1 autoregressive lag to improve ARIMA model fit. The moving average component (q) was left at 0 as it did not improve model fit. [SAS \(2018\)](#) recommended forecasting only if 30 or more observations were available. Therefore, forecasting analyses were done only at the Ellsworth and Langley USGS gages due to lack of reference stage data at the remaining two USGS gages. An example of the SAS code to conduct an ARIMA analysis and forecast is provided in Section [C.2](#) in Appendix [C](#).

Finally, if a time trend of the reference stage was observed through the ARIMA analysis and it appeared that the trend was non-linear, a change point analysis was conducted. The

statistical software SAS uses the Markov Chain Monte Carlo (MCMC) change-point model (SAS, 2019). The purpose of the MCMC change-point analysis is to determine the year in which the line slope of reference stage over time significantly changed. Change-point years are discussed and compared across assessed USGS gages. An example of the SAS code to conduct a MCMC change-point analysis is provided in Section C.3 in Appendix C.

4.2 Results and Discussion

4.2.1 Ellsworth USGS Gage Bed Stability

The relative consistency in reference stage at the Ellsworth USGS gage in Figure 4.1 since the analysis by Juracek (2011) suggests the bed has remained relatively stable since about 2010, with the exception of the reference stage data from late 2018 to 2020. The ARIMA analysis (Fig. 4.1b) indicated that aggradation has occurred at this gage since dam closure, with the MCMC change-point occurring between 1985 and 1986. The median peak flow was found to be significantly higher for the period of 1949 to 1985 compared to 1986 to 2018, based on the Mann Whitney U Test at a 5% significance level (Fig. 3.1). This observed difference in flow is useful in explaining the changes to the reference stage at the Ellsworth USGS gage.

Following dam closure, the multipurpose pool of Kanopolis Lake was filled in just five months (KWO, 2017), resulting in a sudden change in base level and thus overall water surface slope upstream of the dam (Juracek, 2011; Wang et al., 2007). With continued decades of high peak flows, the bed aggraded at the Ellsworth USGS gage at a rapid rate (Fig. 4.1). Since 1985, when peak flows decreased, the bed elevation has remained relatively stable. Furthermore, following the 2019 high flows (≥ 1.3 -return interval flow event, see Fig. A.2 in Appendix A), scour was observed, rather than aggradation, a significant observation as aggradation was dominant during all flow types in the three decades following dam closure. Therefore, the Ellsworth USGS streambed elevation has likely been in a stable state since the mid-1980s. ARIMA forecasting provides additional evidence of recent bed stability. While

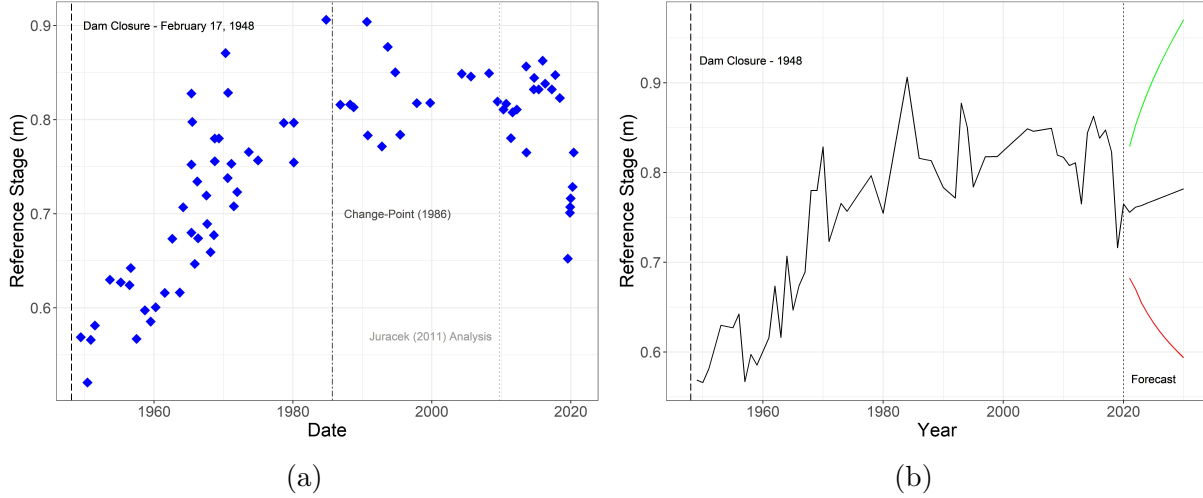


Figure 4.1: (a) Reference Stage (m) at 5.7 cms at the Ellsworth (06864500) USGS Gage, 47.9 km upstream of Kanopolis Dam from 1949 to 2020. (b) Linearly Interpolated Reference Stage from 1949 to 2020 and Predicted Reference Stage from 2021 to 2030. *Note:* Red and green lines represent the 95% confidence interval limits. Data prior to 2011 adapted from [Juracek \(2011\)](#).

the analysis predicts that the bed will continue to rise slowly over the next 10 years at the Ellsworth USGS gage, based on the 95% confidence interval limits (the red and green lines of Fig. 4.1b), the vertical direction of the streambed is unclear, suggesting overall bed stability may have been achieved at this site for the current climatic conditions.

4.2.2 Langley USGS Gage Bed Stability

The Langley USGS reference stage time-series plot (Fig. 4.2a), also points to recent bed stabilization directly below the dam since the [Juracek \(2011\)](#) study. The ARIMA analysis results are shown in Figure 4.2b. At this site, statistically significant degradation has occurred over the time period. It is noted that degradation began prior to dam closure at the Langley USGS gage. Early degradation is likely due to the construction process of the dam, which began in December of 1940 ([KWO, 2017](#)). The change-point from the MCMC analysis occurred in 1974. Flows of similar magnitude and duration to that of 2019 were observed in the year prior, 1973 (Fig. 3.1). This observation suggests that a significant amount of geomorphic work occurred during the 1973 high flows (≥ 1.3 -year return interval, see Fig.

A.3 in Appendix A). The next notable decrease in reference stage occurred in the mid-1990s (see Fig. 4.2), which again coincides with high flows in 1993 (see Fig. 3.1). It is noted that, while flow events of similar magnitude and duration have occurred since 1993 (e.g., in 2007 and 2019), reference stage has continued to center around 1.5 m, suggesting that the bed is stabilizing at the Langley USGS gage. ARIMA forecasting supports this observation. While the reference stage elevation is predicted to continue to decrease slowly, the 95% confidence interval limits suggest that the bed could increase or decrease in elevation over the next 10 years (see Fig. 4.2b), suggesting bed stability has also been achieved at the Langley USGS gage.

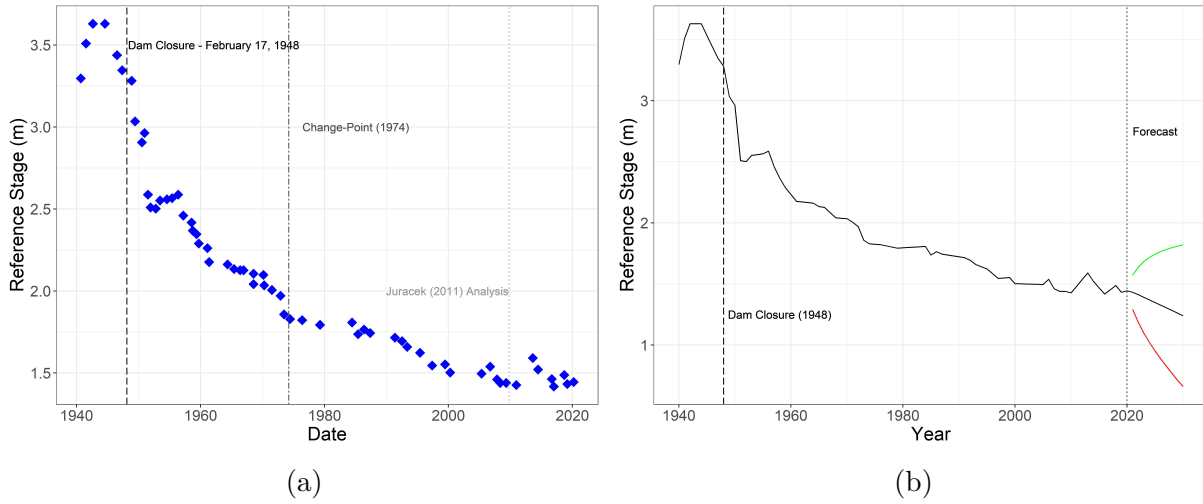


Figure 4.2: (a) Reference Stage (m) at 8.5 cms at the Langley (06865500) USGS Gage, 0.8 km downstream of Kanopolis Dam from 1940 to 2020. (b) Linearly Interpolated Reference Stage from 1940 to 2020 and Predicted Reference Stage from 2021 to 2030. Note: Red and green lines represent the 95% confidence interval limits. Data prior to 2011 adapted from [Juracek \(2011\)](#).

4.2.3 Lindsborg and Mentor USGS Gage Bed Stability

The Lindsborg USGS gage reference stage time-series plot is shown in Figure 4.3a. The data obtained and plotted by [Juracek \(2001\)](#) indicate that bed aggradation may have occurred over the time period from 1937 to 1965. The ARIMA analysis of this time period confirmed this observation with a 5% confidence level. Furthermore, the MCMC change-point analysis

found that aggradation rate reduced after 1947, one year prior to dam completion. This finding suggests that the accelerated degradation that began soon after the dam construction process started (December 1940) at the Langley USGS gage upstream may have deposited at or near this USGS gage over the next five years. This observation, upstream degradation resulting in downstream aggradation, follows the longitudinal view of the [Schumm et al. \(1984\)](#) channel evolution model (Fig. 1.1).

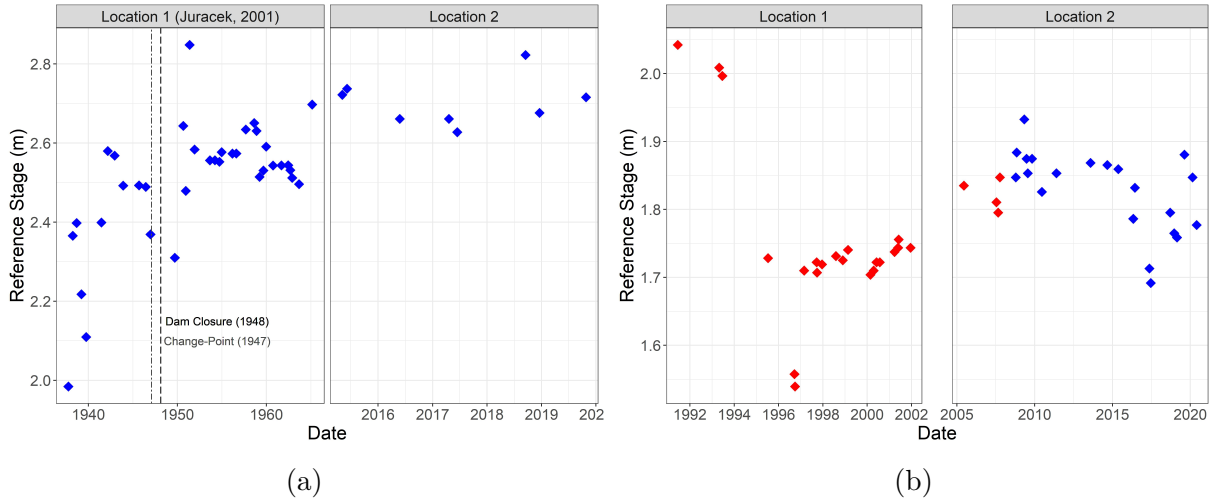


Figure 4.3: (a) Reference Stage (m) at 11.3 cms at the Lindsborg USGS Gage, 63.9 km downstream of Kanopolis Dam from 1937 to 1965 at Gage Location 1 (Adapted from [Juracek \(2001\)](#)) and 2015 to 2020 at Gage Location 2. (b) Reference Stage at 11.3 cms at the Langley USGS Gage, 111.4 km downstream of Kanopolis Dam from 1991 to 2002 at Gage Location 1 and 2003 to 2020 at Gage Location 2. *Note:* Red dots represent the *mean* daily stage based on a mean daily discharge of 11.3 cms, while blue represents the instantaneous measured stage at 11.3 cms.

Since recent reference stage data are limited at the new Lindsborg USGS gage site (Location 2 in Fig. 4.3a), an ARIMA analysis could not be conducted. However, based on observation of the plot and a simple linear regression analysis of the 2015 to 2020 reference stage data, the bed appears to have remained relatively stable over the past six years, as reflected by a linear regression slope of nearly zero.

Finally, the Mentor USGS reference stage time-series plot is shown in Figure 4.3b. As noted in Section 4.1, the mean daily stage, instead of the instantaneous stage, was used and graphed prior to 2008 as indicated by the red dots in Figure 4.3b. Mean daily stage results

may be higher or lower than what they would be if recorded in real-time, which would affect the observations made here.

Based on Figure 4.3b, the plot for Mentor USGS indicates the channel bed elevation has remained relatively stable at both Mentor USGS gage locations. The ARIMA analysis of the instantaneous measured reference stages from 2008 to 2020 did not identify a statistically significant time trend. A simple linear regression analysis supported this finding as well, having a slope of nearly zero.

Some anomalies to this observation of bed stability at the Mentor USGS gage do exist, however. Prior to 1994, stage at the reference discharge was about 0.3 m higher than the average stage in the following years at this location. When reviewing the Mentor USGS gage measured flows (Fig. 3.5), this was the representative stage during the highest flows recorded in the 1991 to 2020 time period (i.e., the flood of 1993, >1.3-yr return interval, see Fig. A.1 in Appendix A). This observation suggests that the bed at the Mentor USGS gage may temporarily deposit during high flow events with a long duration; an observation further supported by review of the measured stage in Figure 4.3b during periods of higher flows (e.g., 1998, 2007-2009, and 2019). A decrease in stage of about 0.2 m in the 1996 to 1997 time range is also observed (Fig. 4.3b). Low flow conditions (less than the median daily flow of 4 cms) from October 1993 to May 1996 were observed in Figure 3.5, with the exception of one high flow event in May 1995. This observation suggests that during periods of low flow conditions, the streambed at the Mentor USGS gage temporarily scours and is further supported by observing recorded stage during times where low flow dominates (e.g., 2006, 2012, 2017).

The observation of rising bed during high flows and lowering bed during low flows is supported by research conducted by Shahmohammadi et al. (2021) on estimating threshold velocity of sand particles and Colby (1964) evaluating scour and fill patterns observed in sand-bed streams. For example, Shahmohammadi et al. (2021) showed that medium to course sand (0.25-1 mm) has a threshold near-bed velocity of 18 to 20 cm/s to entrain and move sand particles of this size. Average stream velocity measured at flows as little as 0.4 cms at the Mentor USGS gage were within 18 to 20 cm/s meaning that medium to course sand

found in the monitoring reaches could be entrained and moved during low flows. Continued entrainment and movement of sand during low flows would result in measurable scour over long periods of time.

Furthermore, [Colby \(1964\)](#) shows that in sand-bed streams in a state of uniform slope, bed material, and roughness, the bed rises during floods and scours during low flows as a result of an imbalance of sediment discharge of sand coming into and going out of a reach of interest. Floods and rising limbs have the ability to entrain and move more sand than low flows and therefore contain a higher sediment discharge coming into the reach than going out, resulting in a temporary rise in bed elevation. The opposite occurs during the receding limb and low flows, more sand particles are leaving the site than are being replenished. This constant shift in bed elevation allows for the maintenance of a common, stable bed elevation in stream reaches of uniform slope, bed material, and roughness.

4.3 Summary

The purpose of this chapter was to evaluate bed stability near Kanopolis Dam since its installation in 1948. Smoky Hill River USGS gage rating curves were used to plot and assess changes in a reference, low flow stage overtime. The results from this analysis are summarized as follows:

- Overall, streambed aggradation has occurred up to 47.9 km upstream of Kanopolis Dam at the Ellsworth USGS gage since installation, while degradation has occurred directly downstream (0.8 km) at the Langley USGS gage.
- Degradation and sediment removal at the Langley USGS gage appears to have aggradated the channel near the Lindsborg gage (63.9 km) since dam construction began in 1940, with aggradation rates decreasing soon after dam closure.
- The Smoky Hill River bed appears to have been stabilizing within the last decade at all USGS gage locations near Kanopolis Dam.

These results will be used in Chapter 6 to answer the questions related to if and how bank erosion process and rates have changed following dam installation and if a readily-available numerical modeling tool can accurately model these observed bed changes (or lack thereof). The next chapter provides methods and results of the channel planform analysis up to 111 river km downstream of Kanopolis Dam.

Chapter 5

Objective 2: Comparing Channel Planform Pre to Post-Kanopolis Dam

In this chapter, channel planform was compared pre and post-Kanopolis Dam installation to 111 river km downstream using recent aerial imagery, as well as historic Smoky Hill River channel locations presented by [Dort \(2009\)](#) from the mid-1800s to the mid-1900s. Two metrics were considered: (1) channel migration rates and (2) channel sinuosity. Channel migration is defined here as the “area of floodplain reworked per channel length per year (pg. 54, [Shields Jr. et al., 2000](#)).” while sinuosity is defined as the ratio of stream length to the valley length. In addition to these two metrics, the effect of Kanopolis Dam closure on lateral migration rates were also discussed. In this assessment, lateral migration rates differ from channel migration rates by the geomorphic process that controls the migration. Channel migration can be influenced by both lateral movement of the channel (e.g., from bank erosion), as well as avulsions (or meander cut-offs) and is what is described and evaluated in [Shields Jr. et al. \(2000\)](#), [Kong et al. \(2020\)](#), [Wellmeyer et al. \(2005\)](#), and [Friedman et al. \(1998\)](#). Alternatively, lateral migration is defined here as the lateral movement of the stream in its floodplain controlled by streambank erosion and deposition, similar to what is described in [Hickin & Gerald \(1984\)](#) and [Bufe et al. \(2019\)](#). The following sections provide more detail on the methods used and the results obtained.

5.1 Methods

To obtain both channel migration rates and sinuosity of the Lower Smoky Hill River downstream of Kanopolis Dam overtime, a combination of data sources were used. The portion of the Lower Smoky Hill River assessed stretches from the location of the Kanopolis Dam to the intersection of the Smoky Hill River and Kansas State Highway 4, which is the location of the present-day Mentor USGS gage (Table 2.1 and Fig. 2.1). Assessed time periods were chosen based on (1) the available data and its reliability and (2) similar year sequences between assessment periods (≈ 30 years) and are summarized in Table 5.1. Channel locations of the Smoky Hill River presented in the National High Altitude Program (NHAP) and the National Agricultural Imagery Program (NAIP) imagery are of high resolution and reliability. Both the NAIP and NHAP imagery are georeferenced aerial images obtained in the year shown in Table 5.1. The channel locations of the Smoky Hill River in Dort (2009) were digitized into GIS software based on analysis and interpretation of historical records. Historical records included surveyed USGS topographic and Public Land Survey System (PLSS) maps from the late 1800s to early 1900s, as well as aerial photographs starting in the 1940s (Dort, 2009). Therefore, possible sources of error with this channel location information come from the digitization of these maps into modern-day GIS software for publication in Dort (2009), but also from the surveying techniques and note-taking that occurred during the late 1800s and early 1900s to create topographic and PLSS maps.

Table 5.1: Summary of Channel Migration Time Periods and Data Sources for the Lower Smoky Hill River, Downstream of Kanopolis Dam.

Period	Year	Δ Time (yrs)	Data Source
Pre-Dam	1884		Dort (2009)
	1920	36	
	1947	27	
Post-Dam	1985	38	NHAP Imagery
	2019	34	NAIP Imagery

Obtaining channel migration rates of the NHAP and NAIP imagery was completed via a three-step process using ArcMap software. In the first step, a polyline of the locations of the perceived top of the left and right streambanks was digitized based on the assessment of the

aerial image. The next step was completed using a Cartography tool in ArcGIS to collapse the left and right bank polylines into a single polyline representing the centerline between the two lines, or the centerline of the channel. Finally, the change in area between centerlines of two given time periods was obtained using a Data Management tool in ArcMap to convert two polylines into a polygon. The areas of the created polygons were summed and used in the following equation to estimate channel migration rates (M_{1-2} ; m/yr):

$$M_{1-2} = \frac{\Delta A_{1-2}}{(L_1)(\Delta T_{1-2})} \quad (5.1)$$

where ΔA_{1-2} represents the change in area (m^2) between the centerline for time period 1 and 2, L_1 represents the length (m) of the centerline for time period 1, and ΔT_{1-2} represents the change in time (yrs) between the two time periods. Figure 5.1 shows an example of this process. This described channel migration method is similar to that presented by [Shields Jr. et al. \(2000\)](#) and [Kong et al. \(2020\)](#).

Obtaining centerlines from the channel location information provided by [Dort \(2009\)](#) was not as straightforward due to the lower levels of accuracy and reliability of the maps themselves. Because of less accuracy, the location of the Smoky Hill River during the years of 1884 through 1973 presented in [Dort \(2009\)](#), were all considered when digitizing in a centerline for a specific year of interest (e.g., 1884, 1920, or 1947). The reason for digitizing the centerline only is due to the occasional gaps in data presented in the GIS polygons that were created to represent the channel through time as mapped in [Dort \(2009\)](#). Figure 5.2 provides examples of the channel polygons provided in [Dort \(2009\)](#). Polygons of specific time periods were typically not connected, except for the year 1973, which would have been based on aerial photographs. Therefore, if it was not clear where the channel went after a certain point, the centerline was drawn in to follow the center of the 1973 channel polygon. When channel location information from a specific year of interest or from years closest to the year of interest were present in a localized area, the stream path was from the 1973 channel polygon and followed the centerline of polygons representing closer time periods instead. For example, referring to Figure 5.2, when digitizing the 1947 centerline of the

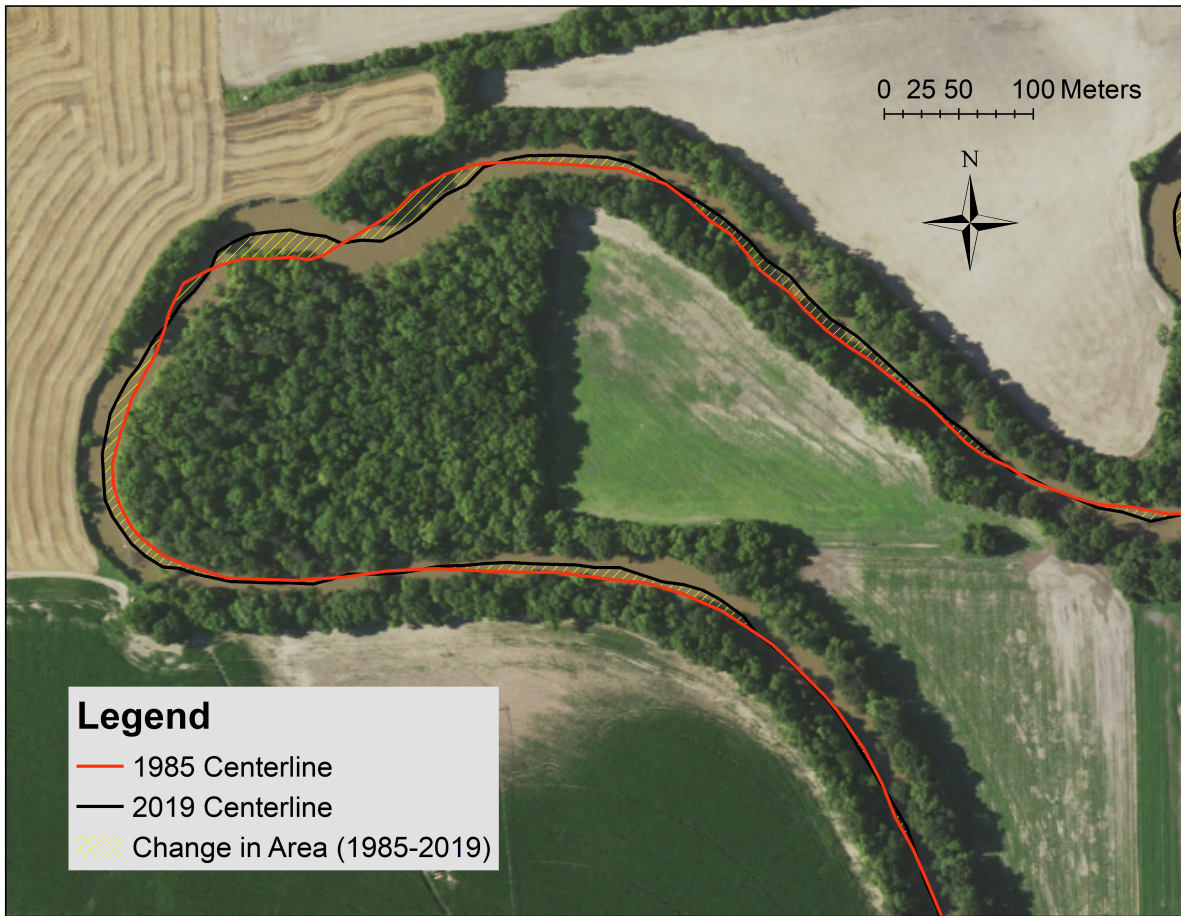


Figure 5.1: Example of Change in Centerline Area Between 1985 and 2019 at Site 1, Downstream of Kanopolis Dam. *Note:* Aerial image is the 2019 NAIP.

channel (green line), the centerline polyline begins in the 1973 channel on the left side of the aerial image and is directed from the 1973 channel to go through the 1947 meander bend, and then is directed to the 1973 channel, in which it remains until another 1947 channel polygon is provided. In Figure 5.2, the 1947 channel centerline is directed from the 1973 channel polygon twice. There is certainly subjectivity that goes with digitizing these paths, but it is expected that if the same instructions were given and demonstrated to another individual, the final stream paths would be similar.

Once centerlines were determined using the Dort (2009) location information, polygons depicting the area between centerlines were created just as was described for the NHAP/NAIP imagery assessment and used to determine the change in area (example shown

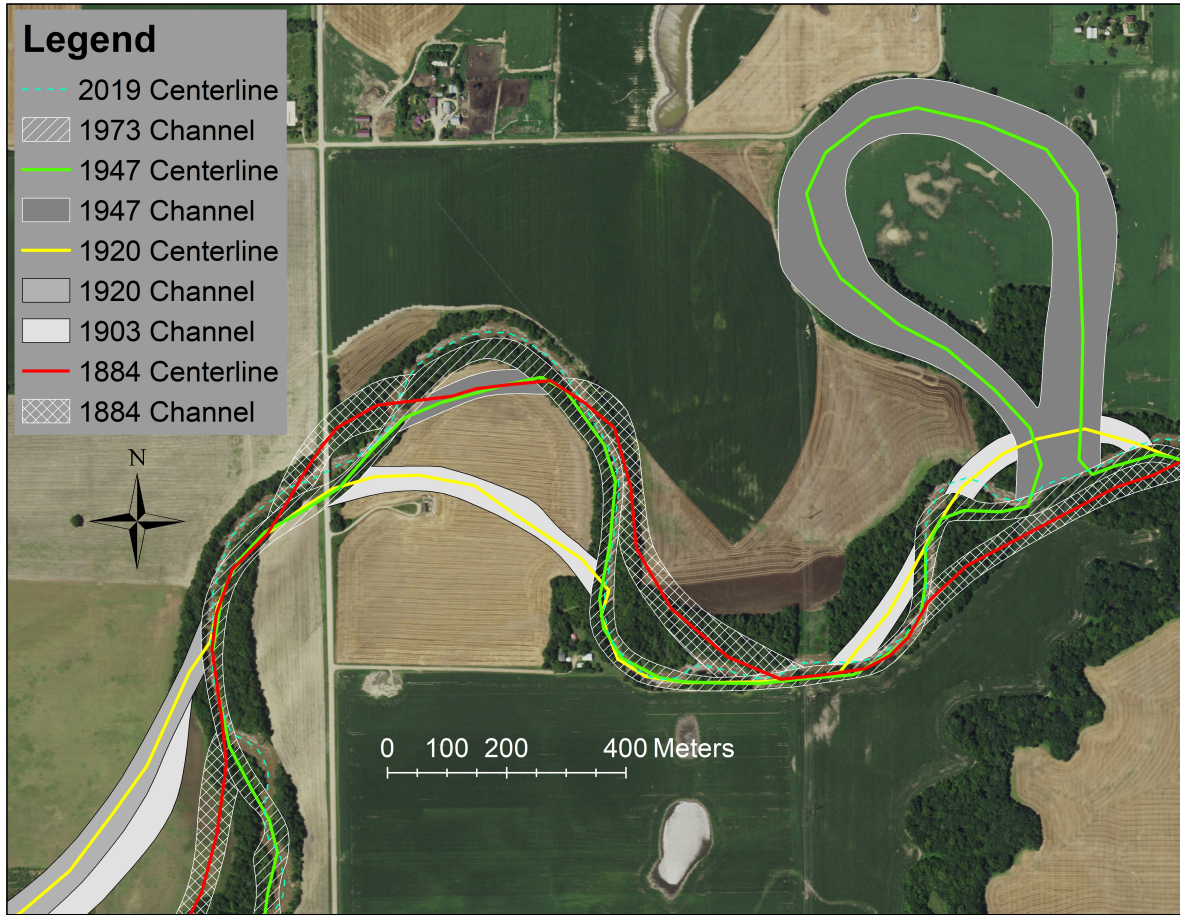


Figure 5.2: Example of Determining Centerline Location Using Dort (2009) Channel Location Information Southwest of Lindsborg, Kansas, Downstream of Kanopolis Dam. *Note:* Aerial image is the 2019 NAIP.

in Fig. 5.1). Equation 5.1 was then used to estimate channel migration rates between two time periods. Once channel migration rates were obtained, a ratio of the average pre to post-dam channel migration rates were calculated and compared to results presented in other below dam channel migration rate studies.

In terms of sinuosity, stream length with each time period was assumed equal to the centerline length (L) determined for the same period in the channel migration analysis. Valley length was measured using a combination of aerial imagery and the overlay of created centerlines for the time periods of interest. The length of the valley was determined by drawing a polyline in the center of the perceived valley in the entirety of the 111 river km

reach. Vertices of the polyline mapping the valley length were, on average, 4 km apart. Total valley length from Kanopolis Dam to the Kansas Highway 4 bridge was found to be 55.3 km. Sinuosity measurements between time periods were computed and compared. Finally, lateral migration that occurs only through streambank erosion and deposition will be discussed by comparing stream lengths through time.

Finally, methods and results from Chapter 3's flow assessment were used to assist in describing observed differences in channel planform. The Langley USGS gage was used to understand flow rates and patterns across the channel migration time periods shown in Table 5.1. As described in Section 3.1.1, the non-parametric Mann Whitney U Test was used to compare differences in median flows for the channel migration time periods, where flow data is available (i.e., 1941-2020 at the Langley USGS gage). A 5% significance level was used to detect significant differences.

5.2 Results and Discussion

The results from the channel planform analysis are presented in Table 5.2. Channel migration rates have progressively decreased on the Smoky Hill River since Kanopolis Dam closure in 1948. The mean pre-dam channel migration rate was 1.19 m/yr and post-dam was 0.37 m/yr. In other words, channel migration rates were on average 3.2 times higher prior to dam installation than post on the Lower Smoky Hill River. This result echoes that of [Friedman et al. \(1998\)](#) where it was found that meandering Great Plains streams decrease in channel migration rates following dam installation. Furthermore, [Shields Jr. et al. \(2000\)](#) reports channel migration rates were 3.4 to 4.3 times greater prior to dam installation on meandering streams downstream of large dams on the Missouri and Milk Rivers in the Great Plains, USA, and the Hanjiang River in China, while an analysis by [Kong et al. \(2020\)](#) indicated pre to post-dam channel migration rates decreased by 2.3 times on the Yellow River in China. Furthermore, [Wellmeyer et al. \(2005\)](#) showed a 42% decrease (ratio of 1.7), following dam installation on the Trinity River in Texas, USA.

Observed peak flows, as well as the observed reduction in peak flows following dam

Table 5.2: Estimated Stream Lengths (km), Sinuosity, and Channel Migration Rates (M, m/yr) Pre-Dam (1948 Closure) to Post-Dam, Up to 111 River km Downstream of Kanopolis Dam on the Lower Smoky Hill River.

Period	Time Range	Avg. Stream Length (km)	Avg. Sinuosity	Channel Migration Rate (M, m/yr)
Pre-Dam	1884-1920	103.4	1.87	0.86
	1920-1947	105.5	1.91	1.53
Pre-Post	1947-1985	108.5	1.96	0.57
Post-Dam	1985-2019	110.5	2.00	0.18

installation, as described in Chapter 3 and shown in Figure 3.1, help explain the reduction in channel migration rates following dam installation. The highest flow shown for Ellsworth USGS gage was in 1938, nearing 1,200 cms, a 200-year return interval event (0.5% annual exceedance probability (AEP), see Table A.2). As described in Leopold et al. (1964), an abundance of geomorphic work can occur during rare floods, like the one that occurred in 1938, as shown in Table 5.2 with channel migration rates being the highest during this time period (1920-1947). The next highest flow event at the Ellsworth USGS gage was 850 cms and occurred in 1993, but represented only a 30-year flood event (3.3% AEP) at the Ellsworth USGS gage (Fig. A.2). Furthermore, the highest flow measured since dam construction at the Langley USGS gage was only 190 cms in 1993. In other words, the dam was able to reduce the 1993 peak flow by 78%.

Results in Table 3.2 further support the flow and channel migration relationship showing a significant difference in median annual peak discharge pre to post-Kanopolis Dam closure, while the change in median daily discharge was found not to be significant. As described in Chapter 3, the reduction of peak flows resulted in a step-like hydrograph, meaning dampened peak flows with a longer duration, as shown by the significant increase in median high pulse duration at the Langley USGS gage following dam closure (see Table 3.2). Therefore, a combination of lower overall observed flows throughout the post-dam time frame, as well as the peak flow dampening ability of Kanopolis Dam, explain the reduction in channel migration rates observed downstream following dam installation.

When evaluating the differences in channel migration rates between the two post-dam

periods (e.g., 1947-1985 vs. 1985-2019), the lower channel migration rate during the latter time period may also be related to the flows experienced during these two time periods. Median daily discharge was significantly higher during the 1947 to 1985 time period (2.15 cms) versus the 1985 to 2019 time period (2.12 cms) at a 5% significance level. Furthermore, variability in flows experienced in the 1947 to 1985 time period was greater as compared to the latest time period. Finally, average annual volume of runoff (cms/yr) was 16.1% less during the 1985 to 2019 time period versus the 1947 to 1985 time period. Therefore, a lower overall channel migration rate during the 1985 to 2019 time period is a plausible response to a lower runoff volume and rate.

By contrast, sinuosity of the Smoky Hill River downstream of Kanopolis Dam has progressively increased overtime, with measured sinuosity being 5% greater than it was prior to dam closure. This finding is similar to that of [Kong et al. \(2020\)](#) downstream of a large dam on the Yellow River in China in which sinuosity increased 12% in the reach closest to the dam, just 18 years after dam closure. It is also worth noting that sinuosity was fairly stable at around 1.9 for this portion of the Smoky Hill River for the 63-year period (1884-1947) that was assessed prior to dam closure, even though channel migration rates were higher.

As quoted from [Leopold et al. \(1964\)](#), “grade is generally considered synonymous with equilibrium (pg. 267).” In other words, prior to Kanopolis Dam installation in 1948, this portion of the Smoky Hill River may have been near dynamic equilibrium, depicted as Stage 1 or Stage 5 in the [Schumm et al. \(1984\)](#) channel evolution model (Fig. 1.1). Dynamic equilibrium refers to a condition in which a “river channel at a particular location over a period of time similarly receives an inflow of sediment and water, which is discharged downstream, while the channel itself remains essentially unchanged (pg. 267, [Leopold et al., 1964](#))”. Equilibrium, however, does not mean that the channel does not move within its channel boundaries, rather, it means that while bed scour and fill and streambank erosion and deposition are occurring, it does so in a way that is balanced, and thus the channel size and slope remain relatively constant over this time of equilibrium. The channel migration rates and sinuosity observed prior to Kanopolis Dam closure point to dynamic equilibrium, showing that bank erosion, deposition, and overall channel movement via avulsions occurred,

but channel slope remained fairly stable, as shown by similar sinuosity measurements. In other words, stream length did not change significantly, even though meanders were created and/or cut-off during this pre-dam time period (e.g., Fig. 5.2).

However, following Kanopolis Dam closure, sinuosity has continued to increase overtime while channel migration rates have decreased. There are several reasons for this observation but the overall driver of this is simply channel instability brought on by an anthropogenic modification to both the flow and sediment regime. In other words, the range of flows and sediment load that this portion of the Smoky Hill River once received are no longer, and thus the channel is adjusting to the new flow and sediment regime regulated by the installation and operation of Kanopolis Dam through degradation, aggradation, and bank erosion processes (example shown in Fig. 1.1). Because flows have been dampened by 72% (see Ch. 3) and the sediment load has been reduced by 95% (Juracek, 2011), the stream slope must adjust to become flatter to account for these reductions in flow and sediment. This occurs through both degradation, as observed and discussed in Chapter 4, and by increasing stream length, and thus sinuosity, through streambank erosion. It is therefore hypothesized that lateral migration rates have increased throughout the extent of the 111 river km reach of the Lower Smoky Hill River following Kanopolis Dam installation, but especially at meander bends. Figure 5.1 provides an example of lateral migration. Note that while change in area is greatest at meanders in Figure 5.1, there is also lateral migration occurring on straight reaches too. In layman terms, it is believed that Smoky Hill River is subtly becoming more wiggly, which can be supported by an observed stream length increase of 5 km and corresponding increase in sinuosity since Kanopolis Dam was closed in 1948.

5.3 Summary

A stream channel's size, slope, and planform are controlled by the incoming sediment and flow regime. This chapter evaluated the effects of Kanopolis Dam installation on two channel planform metrics: (1) channel migration rates and (2) sinuosity. Based on GIS analysis of historical aerial imagery and mapping, channel migration rates along the Smoky Hill River

have decreased by 69% while channel sinuosity has increased by 5% since Kanopolis Dam closure in 1948 up to 111 river km downstream of the dam. These observed planform changes are likely in response to changes in both the flow and sediment regime following Kanopolis Dam installation that resulted in the need for a flatter channel slope, which can occur via degradation and/or increasing overall stream length through lateral migration, and thus sinuosity. The following chapter tests the hypothesis that streambank erosion rates and processes have shifted following Kanopolis Dam closure by using a 1D numerical model.

Chapter 6

Objective 3: Effects of Kanopolis Dam Flow Regulation on Downstream Streambank Erosion Processes and Rates

As noted in Chapter 2, annual geomorphic surveys were conducted along three stream monitoring reaches (Sites 1, 2 and 3 in Figure 2.1; Bigham et al., 2020) between 2016 and 2020. The purpose of these surveys was to monitor woody revetment projects installed in 2016 on Sites 2 and 3. Site 1 was set-up as an unmodified control site for the entirety of the study period. During the flooding that occurred in 2019 along the reaches (see Fig. 3.5), the woody revetments failed. The question was posed as to why failure occurred. There are several potential reasons, such as poor anchoring, lack of vegetative cover following installation, overall construction disturbance, and loosening of the streambank profile. However, the operation of Kanopolis Dam upstream to control flooding may have also played a role. Flow was near the top of the bank for most of the summer of 2019, resulting in a step-like hydrograph versus a more flashy hydrograph typical of unregulated and storm flow dominated streams, as discussed and shown in Chapter 3. Based on the literature review provided in

Chapter 1, studies investigating downstream dam effect on bank erosion are rare, but a few studies have noted changes in bank erosion rates and processes following dam installation (e.g., Gilliam, 2011; Hupp et al., 2009; Mohammed-Ali et al., 2020). As a result of these findings, Mohammed-Ali et al. (2020) and Gilliam (2011) suggest dam operation changes to lessen the impact, however no study provides recommendations on managing streambank erosion through river engineering and restoration techniques downstream of dams.

The purpose of this chapter is to investigate if and how streambank erosion rates and processes have changed following Kanopolis Dam installation. Streambanks erode in three general ways: via subaerial processes from weakening and weathering, fluvial erosion, and/or geotechnical failure. Streambank erosion analysis focuses on assessing the relative contributions of two commonly dominant mechanisms of streambank erosion: fluvial erosion and geotechnical failure. Results from this analysis will add to both the downstream dam effect and streambank erosion literature, as well as provide insight to stream practitioners on how to manage accelerated streambank erosion caused by anthropogenic manipulation of the landscape.

It is hypothesized that streambank erosion rates are higher following dam installation than prior to installation. This hypothesis is based on observed changes in the post-dam hydrograph form from flashy to step-like (see Ch. 3). It is expected that banks will saturate overtime, resulting in positive pore-water pressure followed by mass wasting due to geotechnical failure during or soon after the receding limb of the flow-regulated hydrograph. Furthermore, moderate flow events (i.e., 5 to 40% daily exceedance probability) that are held within the stream channel for longer periods of time due to flow regulation are expected to result in a sustained high applied shear stress, and thus increase fluvial erosion at the toe of the streambank. To assess the effects of flow regulation on streambank erosion and test this hypothesis, a 1D hydraulic, bank erosion, and sediment transport model, in combination with field geomorphic surveys at Site 1 (the control site located 86 river km downstream of Kanopolis Dam) from Biggam et al. (2020) was used. Details about this model are provided in the following section.

6.1 Methods

Given that bank erosion is at the forefront of this research objective, a physically-based numerical modeling tool was selected that could assess both bank erosion rates and processes. The Bank Stability and Toe Erosion Model (BSTEM), developed by the USDA-ARS, is one tool that has been utilized to predict lateral retreat rates by incorporating shear strength and fluvial erosion parameters of varying streambank layers while also assessing average applied shear stresses to determine whether bank erosion occurs because of (1) geotechnical failure (2) planar failure caused by fluvial erosion near the toe of the streambank, or (3) both (Langendoen & Simon, 2008). While a comprehensive streambank erosion model that can predict and estimate rates of all three bank erosion processes does not yet exist, BSTEM's ability to model two of the three erosion processes places it among the most advanced bank erosion numerical models available and it has been employed by researchers from around the globe (Klavon et al., 2017). In addition, it has been integrated into one- and two-dimensional (1D and 2D) models such as CONCEPTS (Langendoen et al., 1999), HEC-RAS (Gibson et al., 2015), TELEMAC (Langendoen et al., 2016), RVR Meander (Motta et al., 2012, 2014), and SRH-2D (Lai et al., 2015). If properly calibrated, 2D sediment transport models would provide the most accurate representations of velocity distribution, sediment transport, and bed evolution (Bhuiyan et al., 2009; Khosronejad et al., 2017; McCoy et al., 2007). However, 2D models can be cost-prohibitive for design purposes as they require more data for calibration and are computationally expensive, requiring more time than 1D models to run (Enlow et al., 2018).

Of the aforementioned modeling software that incorporates BSTEM, HEC-RAS, developed by the USACE, is a free software that has been widely utilized by practitioners for river design applications and floodplain mapping (Klavon et al., 2017). Given its popularity and applicability to the hypothesis posed herein, HEC-RAS (Version 6.1) was chosen to integrate and model flow, bank erosion, and sediment transport. While the current version of HEC-RAS has the ability to model sediment transport in 2D, BSTEM has not yet been incorporated with this higher dimension, therefore the 1D model was used. Details

regarding site characterization, model set-up, calibration, and simulations are presented in the following sections.

6.1.1 Site Characterization and Model Set-Up

To set up a 1D hydraulic, bank erosion, and sediment transport model in HEC-RAS, the following inputs are required: (1) river geometry of the reach(es) of interest, (2) a flow hydrograph, (3) sediment transport characteristics, such as bed composition and depth, as well as water temperature, and (4) streambank stratigraphy, including groundwater and alluvial aquifer characteristics. Figure 6.1 provides a flow chart of the modeling process, including inputs and the final output. Model inputs are discussed in the following sections.

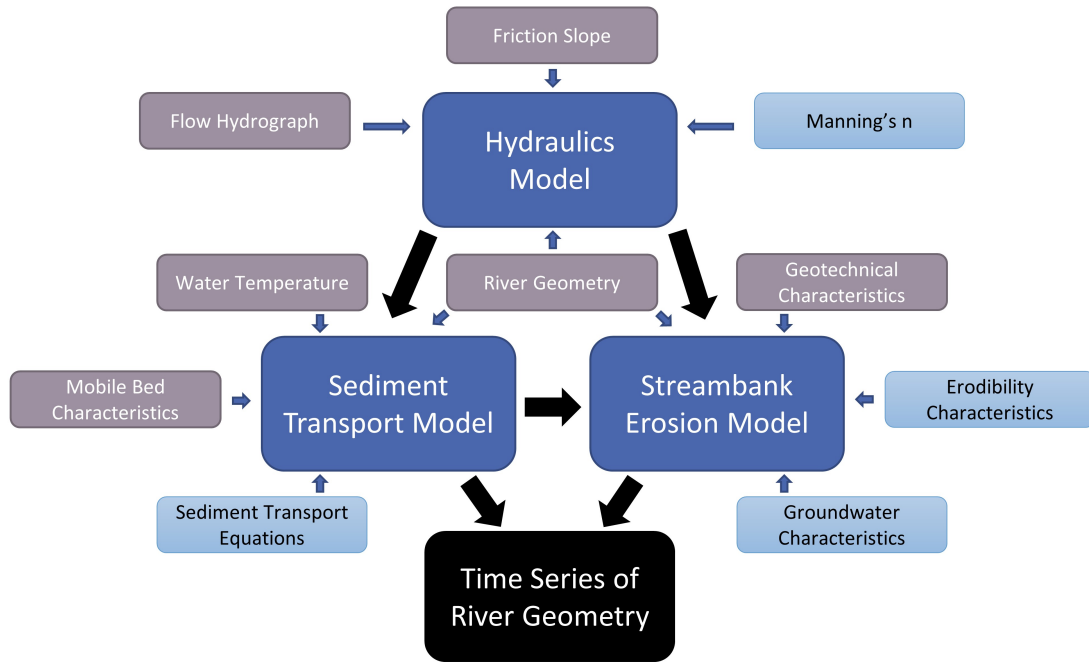


Figure 6.1: Flow Chart of the HEC-RAS 1D Hydraulic, Bank Erosion, and Sediment Transport Modeling Process. *Note:* Dark blue cells and white text represents major model input components. Black cell and white text represents model output. Light gray cells with white text represents model inputs. Light blue cells with black text represents model inputs that are also used for model calibration.

River Geometry

Geomorphic Surveys

Prior to modeling, Site 1 was surveyed with total station surveying equipment three times between March 2016 and March 2018. These annual geomorphic surveys included measurements of channel dimensions and profile. Channel dimensions were obtained through repeated river cross section surveys, monumented on the top of each bank with 1.2 cm by 0.6 m long steel rebar hammered nearly flush with the ground. In addition, repeated longitudinal profile surveys were conducted along the channel thalweg and water surface to assess both channel and water surface slope overtime. The total length of the surveyed Site 1 reach was 830 m (Fig. 6.2).

HEC-RAS River Geometry

The 1D reach length was determined by running a 2D, unsteady hydraulic model in HEC-RAS. The 2D underlying terrain mesh was developed using 2011 Lidar (1 m resolution) and improved with 2016 cross sections and longitudinal profile survey data at Site 1 to incorporate bathymetry data and to enhance resolution of channel geometry. The unsteady flow event modeled was an overbank flood to allow for visualization of flow direction once water reaches the terrace. Since 1D models assume that flow is moving in a 1D direction (i.e., downstream), the reach length was selected where flow was moving mainly in the downstream direction during the flood event and not laterally across the floodplain. Given the field monitoring period of 2016 through 2020, the highest expected natural (no dam) flow event was selected to be modeled from this time period. The natural flow model employed in Chapter 3 indicated this event would have occurred on May 28, 2019 and reached a peak daily flow of 446 cms, as shown in Figure 3.5. Flow direction arrows output from the 2D hydraulic model were used to select (1) the streambank erosion cross section to be modeled and (2) the total length of the 1D reach (see Fig. 6.3). The selected length of the 1D reach, in comparison to the geomorphic survey length, is shown in Figure 6.2.

Once the location of the model reach length was known, 1D river geometry required by HEC-RAS was developed using the software's built-in GIS software tool, RAS Mapper. The

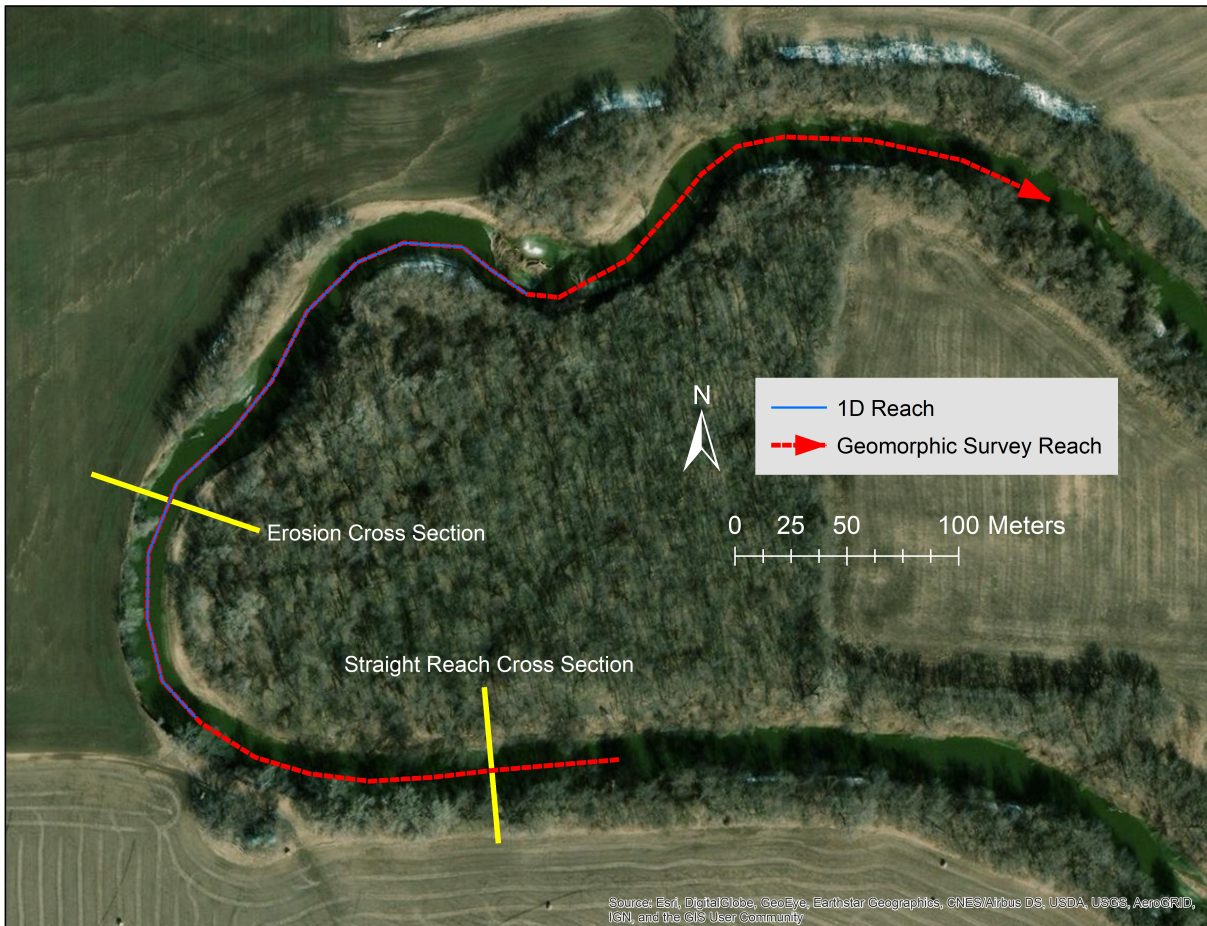


Figure 6.2: Site Map of Site 1 Showing Geomorphic Survey with 1D HEC-RAS Reach. *Note:* “Erosion Cross Section” survey results are shown in Fig. 6.5, “straight reach cross section” survey results are shown in Fig 6.11. Arrow represents direction of flow.

final 1D river reach was 320 m long (start location of the 1D reach is 206 m downstream from the start of the surveyed longitudinal profile, Fig. 6.2) and represented by 18 cross sections lines (Fig. 6.3). The underlying terrain established for the 2D hydraulic model was used to develop the river geometry at each cross section along the reach. Additional information on the process to develop river geometry for HEC-RAS modeling can be found in the HEC-RAS User’s Manual (USACE, 2020d).

HEC-RAS Streambank Erosion Geometry

Based on the 1D reach length, the streambank erosion cross section of interest, shown in Figures 6.2 and 6.3, was selected to model bank erosion along a meander bend, as the outside

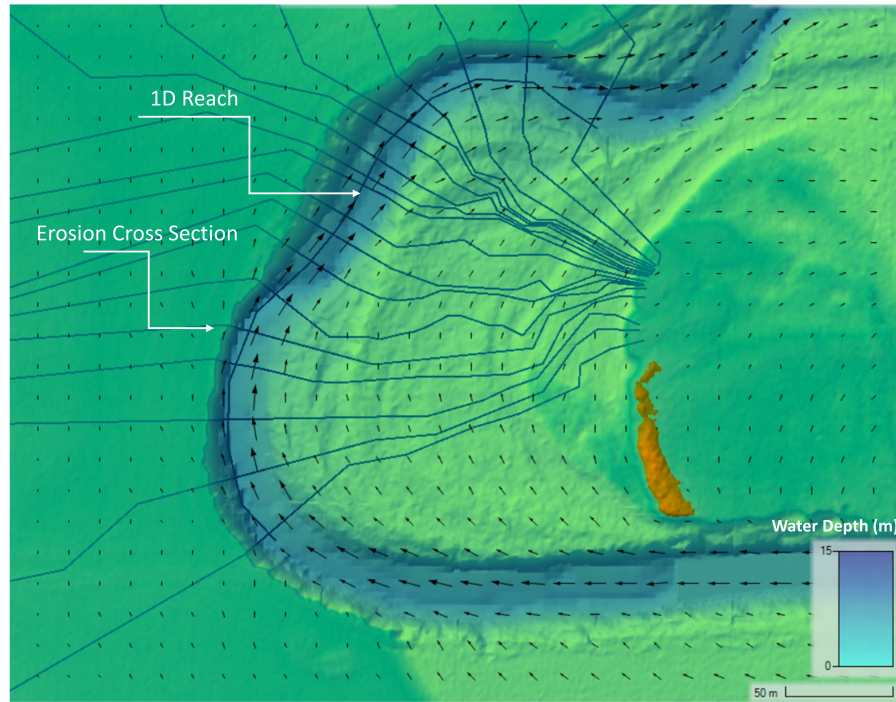


Figure 6.3: HEC-RAS 2D Unsteady Hydraulic Model Used to Specify the 1D Model River Reach. *Note:* Erosion Cross Section is the cross section of interest used in bank erosion modeling. Black arrows represent the direction of flow.

of a meander bend is most susceptible to bank erosion due the formation of secondary currents (Papanicolaou et al., 2007). The erosion cross section was located 309 m downstream from the start of the surveyed longitudinal profile. Figures 6.4 and 6.5 provide a photograph of the streambank and the annual cross section monitoring results, respectively, for the streambank erosion cross section of interest. Finally, the “straight reach cross section” shown in Figure 6.2, was another surveyed cross section located on a straight reach of the Smoky Hill River and was used as a comparison cross section when discussing the BSTEM modeling results later in this chapter.

Flow

HEC-RAS 1D Hydraulics Description

Before sediment is routed, HEC-RAS first calculates the hydraulics given a selected computational interval. Sediment transport and bank erosion can be modeled with either quasi-unsteady or unsteady flow. Quasi-unsteady flow converts a hydrograph into a flow



Figure 6.4: Photograph of the Streambank Erosion Cross Section of Interest along Site 1 on the Lower Smoky Hill River, March 2017. *Note:* Above the dark, thin layer at about 3m in depth represents the Honey Creek Member while below represents the Gunder Member of the DeForest Formation of Holocene Alluvium.

histograph that represents steps of steady flow based on the selected computational interval for a range of flow events. Assuming subcritical flow, steady flow solves the water surface elevation using Bernoulli's equation (the physical principle of the conservation of energy) through an iterative process between cross sections starting on the downstream end of the reach. Alternatively, unsteady flow incorporates the physical principles of the conservation of mass and momentum through the use of the Saint-Venant equations ([USACE, 2021a](#)). While unsteady flow provides the highest computational accuracy, it is more susceptible to model instabilities due to its complexity and also requires more run-time to perform computations. This study compares water surface elevation results to select the hydraulic

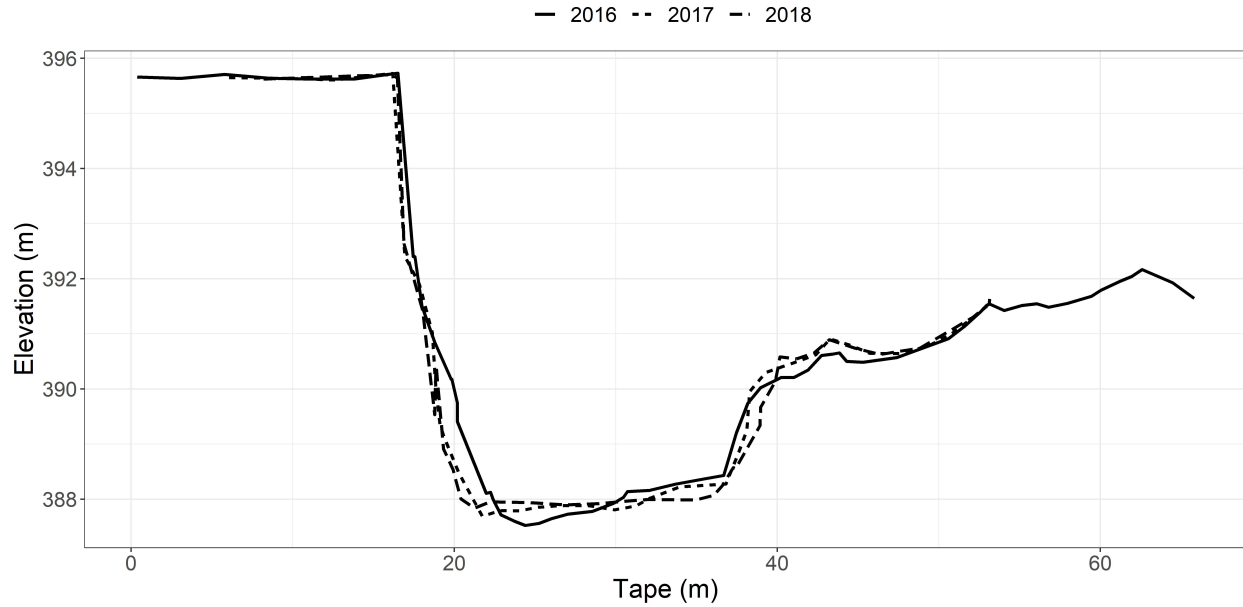


Figure 6.5: Site 1 Streambank Erosion Monitoring Cross Section Survey Results, 2016-2018. *Note:* Bankfull elevation is located at 391.5 m and will be discussed more in Chapter 7. Bankfull cross sectional areas range from 77 m^2 in 2016 to 80 m^2 in 2018.

simulation assumption (discussed later).

HEC-RAS Modeled Flow Events

The flows of interest for the model set-up and simulation were within the range of flows that occurred, or could have occurred under a no-dam condition, during the geomorphic survey assessment period of March 2016 to September 2020. Figure 6.6 shows the majority of these flows, from April 1, 2016 to December 31, 2019 and highlights the dates of annual geomorphic surveys conducted at Site 1. It is noted that due to loss of control pins after 2019 flooding, Site 1 was not re-surveyed in 2020 (see Table 2.1 for more information).

During the erosion assessment period, two days were found with negative expected natural (without Kanopolis Dam) flows from the analysis described in Chapter 3, with the lowest negative discharge value of only 0.4 cms. These two dates were increased to a positive value, as HEC-RAS 1D will not allow a flow of zero cms. Therefore, a low flow value was used to replace negative values. This low flow value was set to the lowest recorded actual discharge during the study period, which was found to be 0.5 cms. In addition to changing the negative discharge values to 0.5 cms, dates where flow was between 0 to 0.5 cms in the

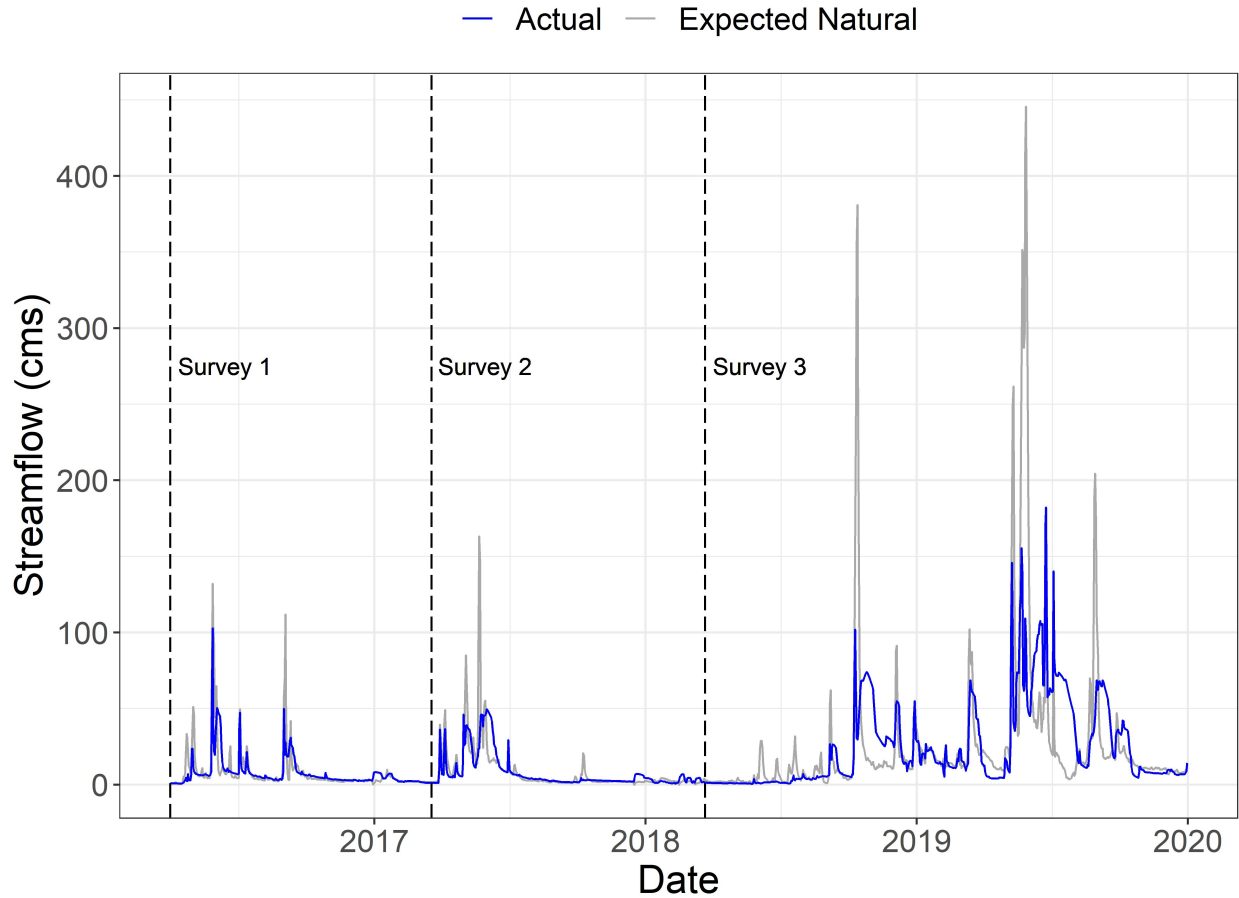


Figure 6.6: Mean Daily Flow (cms) at the Mentor USGS Gage for the Actual/Observed (With Dam) Conditions and for the Expected Natural (Without Dam) Conditions (adjusted to remove negative discharge values), April 1, 2016 to December 31, 2019. Note: Surveys 1, 2 and 3 represents geomorphic survey dates for Site 1.

expected natural hydrograph were also increased to the minimum of 0.5 cms. This occurred on an additional four days of the study period. By converting these dates (6 days in all) to 0.5 cms, very little flow was added to the system ($1.1 \times 10^{-4} \text{ km}^3$ total). The adjusted expected natural flow (cms, with no negative values) versus actual/observed flow (cms) at the Mentor USGS gage is also presented in Figure 6.6.

HEC-RAS Hydraulic Boundary Condition Assumptions

A 1D hydraulics model requires an upstream boundary conditions of flow at the most upstream cross section of the 1D reach, as well as a downstream boundary condition at the most downstream cross section to obtain a solution. For the upstream boundary condition,

the daily flow events that occurred between the first geomorphic survey (April 1, 2016) and ending December 20, 2019 were selected to be simulated. The simulated flow was cut-off on December 20, 2019, as flows began to increase again following that date (see Fig. 6.6). Normal depth was selected as the downstream boundary condition and is calculated using the quasi-equilibrium friction slope of the reach. Over the three years of geomorphic surveys, Site 1 continued to have a baseflow water surface slope of 0.0004 m/m. This measured slope was used as the friction slope of the reach to calculate normal depth.

Sediment Transport

HEC-RAS 1D Sediment Transport Description

Sediment transport is modeled in HEC-RAS as a combination of both bedload and suspended load. The 1D sediment transport model simulates vertical changes of the bed, such as degradation (channel lowering) or aggradation (channel rising). As mentioned in the Chapter 1, channel bed stability is important, as bed changes can affect bank erosion rates (see also Fig. 1.1). HEC-RAS has the ability to model a mobile bed, route sediment, and adjust cross sections, as needed, based on a specified computation interval. To route sediment, HEC-RAS utilizes the sediment continuity equation, or Exner equation (USACE, 2020c):

$$(1 - \lambda_p)B \frac{\delta \eta}{\delta t} = -\frac{\delta Q_s}{\delta x} \quad (6.1)$$

Where B is the channel width, η is the channel elevation, λ_p is the active layer porosity, t is the time, x is the longitudinal distance, and Q_s is the transported sediment load. The Exner equation is simply a sediment balance; if sediment entering does not equal sediment exiting, then a bed change occurs (deposition or erosion). The Exner equation requires the calculation of sediment transport capacity, or the stream's ability to move the present sediment, for a given cross section to determine if aggradation or degradation might occur. To calculate sediment transport capacity, the grain size distribution is required and a sediment transport function is selected that best fit the site conditions. In addition to the Exner equation, the HEC-RAS 1D sediment transport model also considers sediment fall velocity and sorting

and armoring of the bed, which provides a physical constraint to how much the bed may be able to actually erode (sorting and armoring) or deposit (fall velocity).

With these equations in mind, fluvial sediment data required for modeling include channel geometry, depth of the mobile bed and its limits, water temperature, and streambed gradation, in combination with the upstream assumed sediment transport boundary condition. In addition, HEC-RAS provides eight sediment transport capacity/potential functions, three sediment sorting and armoring methods, and seven sediment fall velocity methods that the user must select that best fits the site conditions. Given the variability and sensitivity inherent to sediment transport modeling, field collected data - including repeated cross sections - are invaluable when selecting sediment transport methods and calibrating the model. Most of the required input data for the 1D sediment transport model was previously collected for Site 1 on the Lower Smoky Hill River (see [Bigham et al., 2020](#)). If required data were unavailable, those data were obtained from previously published literature for streams in a similar hydrophysiographic region to develop estimates of these values for the modeled Lower Smoky Hill River reach. The following section provides more information on how these data were collected, as well as data assumptions made.

HEC-RAS Sediment Transport Data Collection and Assumptions

Initial river geometry and streambed gradation is imperative in setting up a 1D sediment transport model in HEC-RAS. The same cross sections that were used to develop the hydraulic model (Fig. 6.3 and 6.1) were used to model sediment transport. Mobile bed boundaries were determined based on cross sectional grade changes within the low flow channel, as well as field observation where the mobile bed ends and the cohesive bank begins. Mobile bed depth (in meters) was estimated using an electrical resistivity tomography (ERT) survey.

An ERT survey measures the spatial distribution of apparent resistivity of soil material by introducing a current into the subsurface and measuring the voltage potential across electrodes. Apparent resistivity measurements are then inverted to determine spatially-distributed true resistivity. Resistivity information provides insight on the type and location of subsurface material up to a certain depth, which is determined by electrode spacing and

array configuration (Lucius et al., 2007). Common resistivity measurements for streambank materials are 1-20 ohm-m for clay and 20-200 ohm-m for sand, with the presence of water and salt causing values to decrease (Everett, 2013; Long et al., 2012).

An ERT survey was conducted at the streambank erosion cross section of interest using a SuperSting R8/IP multichannel imaging system with a 28 electrode dipole-dipole array and electrode spacing of 2 m. The dipole-dipole array was chosen as it has a high sensitivity to horizontal changes in electrical resistivity (Loke et al., 2003), and thus has the potential to provide insight on streambed and streambank stratigraphy. The field collected data was analyzed using the commercial software, EarthImager 2D (AGI, 2009) to invert the measured apparent resistivity using a newly developed geophysical inversion process as described in Koehn (2020). The ERT line of electrodes was placed parallel with the streambank so that the center of the line was at the erosion cross section of interest. ERT results from Site 1 are provided in Appendix B. Depth of mobile sand was estimated to be 5 m below the bed of the Smoky Hill River at Site 1, based on a resistivity reading of about 40 ohms-m in Figure B.1.

Bed gradation was obtained by averaging annual reach Wolman (1954) pebble count surveys, which was done as part of the Lower Smoky Hill River stream monitoring program (Biggam et al., 2020). The average D_{50} based on three years of bed sampling for Site 1 was found to be 0.52 mm which classifies as a course sand on the Wentworth (1922) scale. The distribution of bed sediment is shown in Appendix B as a linear graph in Figure B.2 and a bar graph in Figure B.3. Bed gradation is important for selecting the proper upstream sediment transport boundary condition, which is the only boundary condition required for sediment transport modeling. While the HEC-RAS 1D Sediment Transport Manual (USACE, 2020c) recommends using a sediment rating curve (i.e., sediment discharge vs. flow curve), measuring sediment transport is difficult and rarely done, and the Smoky Hill River is no exception to this. The more common approach is to use the “equilibrium load” upstream sediment transport boundary condition in HEC-RAS, as it is the easiest in terms of the data required; however, its application is limited. The equilibrium load condition computes the sediment load at the upstream cross section using the bed gradation and

selected sediment transport function to calculate sediment transport capacity. The sediment load calculated at this cross section is introduced as a sediment load time series at the next cross section, therefore the first cross section neither aggrades or degrades. In other words, by using equilibrium load as the upstream boundary condition, it is assumed that the bed is stable at the upstream cross section. Often times, this is not the case. However, after review of the streambed on the annual longitudinal profile (see Fig. B.4) and cross section surveys (example shown in Fig. 6.5), as well as the results presented in Chapter 4, bed stability is a reasonable assumption for the Lower Smoky Hill River in its current state. Therefore, equilibrium load was selected as the upstream boundary condition for the purpose of this study.

Finally, the last required sediment transport input is water temperature. Water temperature affects sediment transport relationships, especially the Toffaleti approach that can be used for both sediment transport and fall velocity calculations (USACE, 2020c). Only one Smoky Hill River USGS gage has a temperature sensor and is located near Arnold, KS (USGS 06861000), over 200 km west of these sites with just one month of temperature data within the 2015 to 2019 assessment period. Therefore, another nearby USGS stream gage was identified and used to estimate daily water temperature that (1) was closer to the site and having a similar climate and (2) had daily temperature data for the entire study period. The Little Arkansas River USGS gage near Halstead, Kansas (USGS 07143672) fits this description. This USGS gage is located 70 km south of the sites (Lat. 38.028590, Long. -97.541218), and therefore has similar climate and precipitation.

The author has conducted geomorphic surveys in the Little Arkansas River (e.g., Bigham, 2016). Similarities in site characteristics include that both the Little Arkansas River and the Smoky Hill River (1) are sand-bed, meandering rivers with cohesive banks and (2) have similar channel width, water depths, and riparian vegetation. Differences in site characteristics include (1) the Smoky Hill River watershed at Mentor is 12 times that of the Little Arkansas River watershed at Halstead, (2) the Little Arkansas River is not regulated by a large dam, and (3) the Little Arkansas River runs mainly in the north to south direction while the Lower Smoky Hill River runs from west to east and then south to north. While

varying hydrographs produced by storms and dam releases, as well as differences in general flow direction may result in varying water temperatures along these two rivers, in absence of better water temperature data, the mean daily water temperature data were downloaded for the period of interest and was used in the HEC-RAS 1D sediment transport model. Furthermore, [USACE \(2020c\)](#) states that, while a varying water temperature is better than a single, selected water temperature, the sediment transport model is not as sensitive to temperature compared to other sediment transport model components. The temperature data is graphed in Appendix B in Figure B.5.

Selecting HEC-RAS Sediment Transport Equations

The HEC-RAS 1D Sediment Transport Manual ([USACE, 2020c](#)) was used to provide guidance on selecting the proper sediment transport function, bed sediment sorting method, and suspended sediment fall velocity method. The results are highly sensitive to the chosen transport method ([USACE, 2020c](#)), therefore, all sediment transport functions that were developed for or from sand-bed systems were evaluated in the calibration phase (to be discussed later). Given their applicability to sand-bed streams, the Copeland method was selected as the bed sediment sorting method, while the Toffaleti method was selected for the suspended sediment fall velocity method. See [USACE \(2020c\)](#) for more information about these methods and their application.

Streambank Stratigraphy and Characteristics

HEC-RAS BSTEM Description

As noted previously, BSTEM is able to compute both geotechnical failure and failure due to fluvial erosion of the streambank toe. To model geotechnical failure, BSTEM uses the Mohr-Coulomb criterion for saturated conditions and the [Fredlund et al. \(1978\)](#) criterion for unsaturated conditions. BSTEM determines a linear, or wedge-type, failure plane of a layered streambank by calculating a factor of safety (F_s) using the horizontal layer method, developed specific for streambank analyses ([Simon et al., 2000](#)):

$$F_s = \frac{\sum_{i=1}^I (c'_i L_i + (\mu_a - \mu_w)_i L_i \tan \phi_i^b + [W_i \cos \beta - \mu_{ai} L_i + P_i \cos(\alpha - \beta)] \tan \phi'_i)}{\sum_{i=1}^I (W_i \sin \beta - P_i \sin[\alpha - \beta])} \quad (6.2)$$

Where c' is the effective cohesion (kPa), L is the length of the failure plane (m), μ_a is the pore-air pressure (kPa), μ_w is the pore-water pressure (kPa), ϕ^b is the unsaturated internal friction (degrees), W is the weight of the streambank (kN); P is the hydrostatic confining force due to external water level (kN/m); ϕ' is the effective angle of internal friction (degrees), β is the failure plane angle (degrees from horizontal), and α is the local bank angle (degrees from horizontal), as are characteristic of the i th layer of the streambank stratigraphy. If F_s is less than one, then geotechnical failure occurs. The 1D hydraulic model provides water surface elevations at selected time steps to estimate confining pressure. Pore-water pressure is estimated based on a bank reservoir groundwater model that requires an estimate of alluvial aquifer reservoir length and saturated hydraulic conductivity (K_{sat}) using the following equation:

$$E_2 = E_1 + \sum_{i=1}^I \frac{K_i \Delta t (W - S_i)}{L} \quad (6.3)$$

Where E is the groundwater elevation at times 1 and 2 (m), K_i is the K_{sat} of the i th layer (m/min), Δt is the time (min) of a given water surface elevation (m), S_i is the elevation of the i th layer subject to water flux, and L is the reservoir length (m).

For fluvial erosion, BSTEM utilizes the linear excess shear stress equation ([Partheniades, 1965](#)) to estimate the lateral fluvial erosion (E_d , m) of cohesive banks:

$$E_d = k_d \Delta t (\tau_0 - \tau_c) \quad (6.4)$$

Where k_d is the erodibility coefficient (m^3/Ns), Δt = time step (s), τ_0 is the average boundary shear stress (Pa), and τ_c is the critical shear stress (Pa). HEC-RAS provides the average applied shear stress for each inundated cross section node using a radial shear distribution method ([USACE, 2020c](#)).

HEC-RAS BSTEM Data Collection and Assumptions

With Eq. 6.2 through 6.4 in mind, streambank stratigraphy properties were measured (when possible) at what appeared to be two distinct layers of varying soil material on the erosion cross section of interest at Site 1, one up to 3 m in depth and another from 3 m depth to the streambank toe (approx. 8 m). In Figure 6.4, these layers are separated by the dark, thin layer of sediment, with sediment above this point representing the Honey Creek Member and below, the Gunder Member (A. Layzell, KGS, personal communication, January 31, 2021), both of which are from the lithostratigraphic unit of Holocene deposits called the DeForest Formation (Layzell & Mandel, 2014). The Honey Creek member has also been referred to as post-settlement alluvium (Layzell & Mandel, 2014).

Soil properties obtained include USDA-classified soil texture, bulk density, cohesion, friction angle, critical shear stress, and erodibility. A Gidding's soil probe was used to obtain a core sample of streambank material down to a depth of 3.5 m at Site 1. USDA-classified soil texture and bulk density were obtained in the laboratory from these samples. Bulk density was then converted to saturated unit weight (γ_{sat}), assuming a particle density of 2.65 g/cm^3 and a water density of 997 kg/m^3 . Results from this analysis are summarized in Figure 6.7.

A Borehole Shear Tester (BST), developed by Handy Geotechnical Instruments, was used to obtain *in situ* shear strength measurements of cohesion and friction angle to be used in the F_s equation (Eq. 6.2). The BST was selected to obtain shear strength measurements as it is the most popular technique used when obtaining these values for use in BSTEM (Klavon et al., 2017). A BST test was performed once at each streambank layer, at depths of 1.5 and 3.5 m. During the test, the BST was operated at 20, 30, 40, and 50 kPa normal stress to obtain the resultant maximum shear stress (kPa). Data were then graphed with normal stress on the x-axis and shear stress on the y-axis to obtain the Mohr-Coulomb Failure criterion using linear regression. The fitted line through the measured points provides the cohesion (c' , y-intercept) and the friction angle (ϕ' , slope of the line in degrees). Figure 6.7 summarizes the results obtained for this cross section.

Erodibility parameters of the two streambank layers were also obtained *in situ* using a mini-JET. Similar to the use of the BST, the mini-JET is the most popular technique in

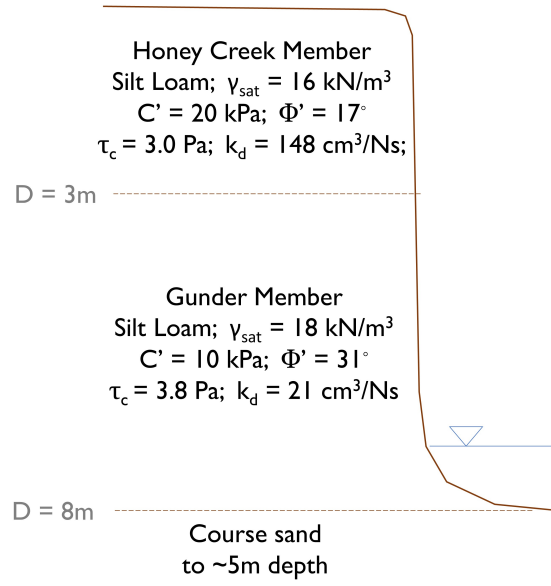


Figure 6.7: Streambank Stratigraphy and Streambed Sediment Profiles for Site 1 on the Smoky Hill River, Kansas used in BSTEM Modeling. *Note:* Soil texture: USDA classification; γ_{sat} : Saturated unit weight; ϕ' : Effective angle of friction; c' : Effective cohesion; k_d : Erodibility coefficient; τ_c : Critical shear stress; D: Depth

obtaining erodibility parameters for use in BSTEM (Klavon et al., 2017). The mini-JET is a smaller version of the original Jet Erosion Test (JET), developed by Hanson (1990), and can obtain *in situ* estimates of erodibility (k_d) and critical shear stress (τ_c) of the excess shear stress equation (Eq. 6.4) by impinging a jet of a known pressure perpendicular to an erodible surface and measuring the scour hole depth the jet creates overtime until an equilibrium depth is obtained. Two mini-JET tests were conducted per streambank layer at 13.8 kPa to obtain average erodibility parameters. Scour over time measurements were then converted to erodibility parameters using the Blaisdell method (Hanson & Cook, 2004). While there are two other techniques that can be used to calculate erodibility parameters, the scour depth solution (Daly et al., 2013) and the iterative solution (Simon et al., 2010), the Blaisdell method was selected for multiple reasons: (1) it is the most popular solution used (Klavon et al., 2017), (2) it has been used in estimating erodibility in other Kansas streams (Layzell & Mandel, 2014), and (3) Daly, Fox, et al. (2015) showed that k_d of Equation 6.4 needed to be significantly reduced to match observed bank retreat if either the iterative or

scour depth approach was used. Figure 6.7 summarizes the results of the mini-JET tests.

Finally, pore water pressure of cohesive streambanks plays a significant role in overall shear strength (Simon et al., 2000). Furthermore, Lammers et al. (2016) found groundwater elevation to be a sensitive BSTEM parameter, especially for non-cohesive streambanks. Pore water pressure varies over space and time as influenced by hydrologic properties such as precipitation, infiltration, evaporation, and soil water movement, as well as the alluvial aquifer associated with the river. While pore-water pressure was not obtained over the study period at Sites 1 through 3, groundwater table elevations are available and were obtained from USGS Water Well 383955097351901 located 0.9 km northwest of Site 2 from March 1997 to March 2020. Groundwater levels at this well from 2015 to 2020 (see Table B.2 in Appendix B) were used to assess water level fluctuations and assist in calibrating the K_{sat} and the aquifer reservoir length (to be discussed later). The initial K_{sat} estimate was 0.8 m/day for silt loam, obtained for Site 1 from the USDA Web Soil Survey. The initial reservoir length was assumed to be the entire length of the left bank of the HEC-RAS 1D cross section at the erosion cross section of interest, which was 340 m. Finally, the initial groundwater level was determined by subtracting the March 2016 well depth from the top of bank and adding 1 m to make sure the groundwater level was above baseflow in the channel. Therefore, the groundwater elevation was set to 388.7 m at the erosion cross section of interest.

6.1.2 Numerical Model Calibration

The 1D flow, sediment transport, and bank erosion model was calibrated using the surveyed cross sections and measured flow data. The model calibration process is discussed in the following sections.

Hydraulics Calibration

A three-step process was used to calibrate the 1D hydraulic model and select the flow assumption (i.e., unsteady or quasi-unsteady flow). A flow chart of the process used to calibrate and select the 1D hydraulics model is presented in Figure 6.8. To calibrate the 1D model,

the outline of the overbank flood damage from June 24, 2019 (191 cms) was used to calibrate Manning's n for the channel and floodplain. Google satellite imagery was used as a basemap in RAS Mapper and was captured three days after the this event on June 27, 2019. Manning's n was calibrated so that the depth would follow close to the flood damage line. Since flood damage was not obvious within the 1D reach at Site 1, a 2D unsteady hydraulics model using the Eulerian-Lagrangian method to solve for depth and velocity was used.

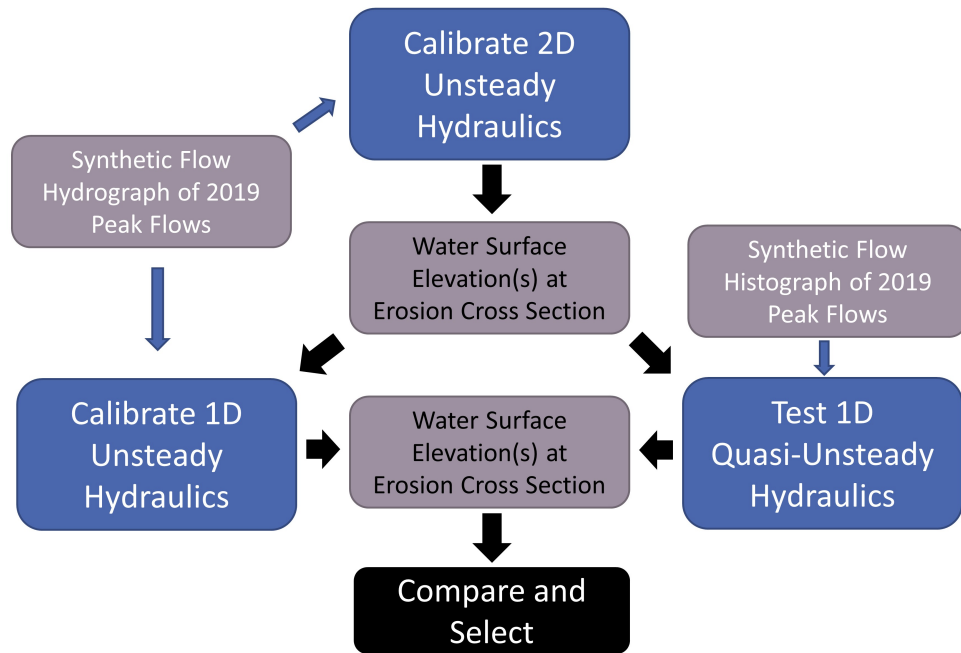


Figure 6.8: Flow Chart of the HEC-RAS Hydraulic Model Calibration Process. *Note:* Dark blue cells and white text represents major calibration steps. Light gray cells with white text represents model inputs. Light gray cells with black text represents model outputs. Black cell with white text represents the final 1D hydraulics model selection. 2019 Peak Flows were 191 cms (observed, with dam) and 446 cms (expected, without dam).

The hydraulics model was calibrated using a synthetic hydrograph having a peak flow value equal to the 2019 flow event of 191 cms. The synthetic hydrograph was based on a representative rise and fall of flow measured at the Mentor USGS gage from May 7, 2019 to May 11, 2019, with a peak discharge of 148 cms (Fig. B.6 in Appendix B). A flow multiplier of 1.29 was used to increase the peak flow to 191 cms. The computational time step for the 2D unsteady simulation was computed based on satisfying the Courant condition using the variable time step option that HEC-RAS provides. Time steps were typically less than

5 seconds when running the 2D unsteady model. See the HEC-RAS Hydraulics Reference Manual (USACE, 2021a) for more information about these computational methods and settings. Manning's n values for the 2D unsteady hydraulics model were finalized as 0.025 in the channel, 0.15 for the wooded area, and 0.07 for the cropland at Site 1, which were within reason to Manning's n estimation methods and values provided in USGS (1989) and USACE (2020b). Figure 6.9 shows an example of the results from modeling this high flow event after successful calibration.



Figure 6.9: Example of 2D Hydraulic Model Calibration using June 27, 2019 Google Imagery showing May 2019 Flood Damage, Site 1.

After 2D model calibration, the highest expected flow during the assessment period was simulated (446 cms) through the 2D terrain using the same synthetic hydrograph (Fig. B.6) and a flow multiplier. The water surface elevations during both flows were recorded at the erosion cross section of interest along the 1D reach to be used in calibrating a 1D, unsteady hydraulic model at the same flow rates using the same synthetic hydrograph. Similar to the 2D unsteady model, the computational time step was computed based on satisfying the Courant condition and were typically less than 15 seconds when running the

1D, unsteady model. After calibration of the 1D, unsteady model, Manning's n values were finalized at 0.052 in the channel and 0.5 in the wooded area, and 0.02 in the cropland area at Site 1, which again were assumed to be reasonable based on review of [USGS \(1989\)](#) and [USACE \(2020b\)](#). Accuracy of water surface elevations through the 1D, unsteady model reach following calibration were within 0.03 m below the 2D modeled stage during the 191 cms flow event and within 0.3 m above the 2D modeled stage during the 446 cms flow event. Differences in Manning's n are likely attributed to the inherent simplicity of the 1D simulation versus 2D.

Furthermore, the right out-of-bank on the 1D cross section for Site 1 was not able to contain the 446 cms flow event and it was not possible to extend the cross sections to a point where it could be contained. When this occurs, HEC-RAS extends the right out-of-bank endpoint vertically to contain the flow, resulting in higher than expected stage due to a decrease in overall available cross sectional area. Using the dynamic water surface elevation tool in cross section view in HEC-RAS, it was noted that flow up to 415 cms could be contained in the actual cross section boundary. When evaluating the expected natural hydrograph shown in Figure 6.6, only one day (446 cms, highest flow) was above this threshold. Therefore, this vertical increase in the right out-of-bank end point is an acceptable assumption; however, it will cause inaccuracies of sediment transport and bank erosion estimates during the highest expected natural flow event of 446 cms.

The final step in the hydraulics calibration process was determining if the quasi-unsteady flow would be acceptable for the purpose of this study. A flow histogram was built for the actual flow conditions and allowed to run under quasi-unsteady flow conditions. Computation increments (or time steps) ranged from 24-hours to 1-hour, based on flow rate, as recommended by the HEC-RAS 1D Sediment Transport Manual ([USACE, 2020c](#)). A 24-hour computation interval was used when flows were less than the 40% exceedance probability (5 cms, see Fig. 3.4), and then decreased gradually to 1 hour for flows ranging from 240 cms to 480 cms. Table B.1 in Appendix B provides the computation increments used for selected flow intervals. Water surface elevations at the streambank erosion cross section of interest for the calibration flow (191 cms) and for the high flow (446 cms) using the quasi-

unsteady assumption were compared to the 1D unsteady flow assumption. Predicted water surface elevations under the quasi-unsteady flow assumption was comparable to those predicted under the unsteady flow assumption, with elevation within 0.5 m for the 191 cms peak flow event (<0.03 m for unsteady) and 0.03 m for the 446 cms peak flow event (<0.3 m unsteady). Therefore, given its faster run time, stability, and water surface approximation, quasi-unsteady hydraulic modeling was selected and used for the purpose of this study.

Sediment Transport Calibration

Sediment transport calibration is key in understanding streambank erosion rates, as localized bed scour and deposition can alter bank height and thus change the gravitational component of shear strength to bank erosion, as well as the localized applied shear stress. To calibrate the 1D sediment transport model in HEC-RAS, measured repeated cross section data were used at the erosion cross section of interest (Fig. 6.5). Flows were obtained between survey dates. For Site 1, daily flow data between geomorphic surveys 1 through 3 were obtained and converted into a flow histogram for the quasi-unsteady flow analysis. Sediment transport was computed at the same computation intervals used for quasi-unsteady flow as described in the previous section (see Table B.1).

The first step in calibrating the sediment transport model is selecting the proper sediment transport function that best describes observed conditions. All eight sediment transport functions can be applied to sand-bed streams. Therefore, it is necessary to evaluate the cross section change to the bed during the calibration time period to help select the proper function. For Site 1, 0.2 m of deposition occurred between survey 1 and 2, followed by another 0.2 m of deposition between surveys 2 and 3 (Fig. 6.5). When applying the various sediment transport models to Site 1, there were three transport functions that came within ± 0.4 m to the observed: the Toffaleti method, the Meyer Peter Muller (MPM) method, and the MPM-Toffaleti method. The Toffaleti method was developed primarily for large, sand-bed streams. The Toffaleti approach combines hydraulics, bed and suspended sediment conditions, and water temperature to estimate sediment discharge. The MPM method is one

of the most widely used sediment transport equations and uses an excess shear relationship (similar to Eq. 6.4) to calculate bedload transport of sand and gravel bed streams. Finally, the MPM-Toffaleti method combines the Toffaleti and MPM methods to calculate sediment transport potential by using the MPM excess shear equation as the bed component in the Toffaleti method. See the HEC-RAS 1D Sediment Transport Manual ([USACE, 2020c](#)) for more information about these three approaches and their equations.

Evaluating the bed change calculations further for these three sediment transport functions, all three methods showed scour at the location of the erosion cross section of interest throughout the entire calibration period, with the MPM method showing the greatest amount of scour of -0.18 m between 2016 through 2018. Based on this prediction and what was observed (deposition), the MPM method was removed as a possible transport function. The final two methods, Toffaleti and the MPM-Toffaleti, produced nearly identical results for the calibration period and thus were further evaluated by observing bed change from the last survey (March 2018) to the end of the assessment period (December 20, 2019). Based on this assessment, the MPM-Toffaleti method showed a maximum deposition during the high flow on June 24, 2019 in the cross section of interest of 0.43 m, while the Toffaleti method showed only a maximum deposition of 0.02 m.

Using acoustic doppler current profiler (ADCP) data from the Mentor USGS gage station collected by the USGS to calibrate the flow rating curve of the gage, maximum flow depths from two measured flow events were obtained: August 10, 2018 at 3.5 cms and June 24, 2019 at 184 cms. Based on the ADCP maximum depth measurements, the bed elevation was 0.2 m higher during the high flow (184 cms), compared to the low flow condition (C. Painter, USGS, personal communication, December 22, 2021). Furthermore, overall deposition was measured between surveys 1 and 3 at the erosion cross section of interest at Site 1, totaling 0.4 m between the 2016 and 2018 surveys (Fig. 6.5). Deposition at high flows can be further supported based on the discussion provided in Section 4.2.3 of research conducted by [Colby \(1964\)](#) that showed similar patterns of bed deposition during high flows and scour during low flows for sand-bed streams having a stable average bed elevation. Given the measured deposition in the local region of the Mentor USGS gage during high flows, as shown by

both the ADCP measurements and the reference stage analysis provided in Section 4.2.3, as well as the observed deposition at the cross section of interest, the MPM-Toffaleti method, which showed the greatest bed deposition during the study period, was selected as the most appropriate sediment transport function for the section of the Lower Smoky Hill River.

Bed change can be altered to more closely model actual conditions in the second step of calibration by adjusting the MPM-Toffaleti transport function with scaling factors. However, when evaluating the bed change differences between predicted and observed and comparing these results to the annual longitudinal profile surveys of the Site 1 reach (see Fig. B.4 in Appendix B), the bed change estimates made by the unadjusted MPM-Toffaleti method are within the range of what was observed throughout the entirety of the longitudinal profile where a combination of scour and deposition occurred spatially and temporally along the reach, with minimal overall average bed change. This again suggests that the streambed is in a dynamic equilibrium. Therefore, no adjustments were made to the sediment transport function to further calibrate the sediment transport model.

Streambank Erosion Calibration

In several BSTEM application studies, researchers adjust k_d and τ_c to calibrate the model to observed bank retreat conditions (e.g., Daly, Miller, & Fox, 2015; Enlow et al., 2018; Klavon et al., 2017; Langendoen & Simon, 2008). Adjusting erodibility parameters to calibrate the BSTEM model was also incorporated here. But before bank erosion parameters were finalized through calibration, the groundwater model was calibrated using depth measurements presented in Table B.2 in Appendix B.

While groundwater table elevation is a critical piece of the FS equation (see Eqs. 6.2 and 6.3), Lammers et al. (2016) showed that groundwater table elevation is not as sensitive as other parameters in the BSTEM model. However, since groundwater table information is available near these sites, it was calibrated to the observed rise and fall of the Lower Smoky Hill River alluvial aquifer water table during the study period.

In terms of calibrating the groundwater model, the goal was to adjust the groundwater

reservoir length (L), and K_{sat} of Eq. 6.3 within reason so that groundwater fluctuations mimicked those observed in Table B.2, particularly the change that occurred between March 2018 to October 2019 (an increase in groundwater elevation of 2.6 m). Furthermore, given the constant groundwater table elevation between March 2015 to March 2016, the initial groundwater depth was also checked to make sure that no drastic changes in elevation were occurring. If there were, initial groundwater elevation would need to be adjusted to minimize fluctuations early on. Following groundwater elevation calibration, the initial groundwater elevation was kept at 388.7 m; however, the reservoir length was decreased from 340 m to 100 m and the K_{sat} value was increased to 2 m/day. This resulted in a fluctuation in March 2018 to October 2019 of 2.3 m, with minimal fluctuation in groundwater level occurring in the early months of the model simulation. It is noted that while 2 m/day for K_{sat} may be on the higher end for silt loam soils, given the purpose of the K_{sat} in modeling groundwater level fluctuations, this value was used for calibration only and thus is assumed to be appropriate for this scenario.

The three cross section surveys obtained in 2016 through 2018 and shown in Figure 6.5 were used to calibrate the bank erosion model. Between 2016 and 2017 (surveys 1 and 2), 4.3 m^2 of erosion occurred, followed by 1.4 m^2 from 2017 to 2018. Initial streambank erosion parameters provided in Figure 6.7 were used first. BSTEM model results indicated no erosion occurred between surveys 1 and 2 and 2 and 3 using these measured values. Therefore, values of k_d and τ_c were adjusted and finalized as 200 cm^3/Ns and 3 kPa, respectively, for the upper layer and 200 cm^3/Ns and 2.7 kPa, respectively, for the lower layer. This adjustment resulted in toe erosion in 2016 through 2017 of 4.1 m^2 and 0.0 m^2 from 2017 to 2018, comparable to what was observed. These selected toe erosion erodibility values were cross-checked using measured data from other Lower Smoky Hill River sites (see Figure B.7 in Appendix B), as well as the northeast Kansas stream data presented in Layzell & Mandel (2014) and are within the range of other measured locations of similar soil type.

Since the streambank erosion portion of the described 1D quasi-unsteady hydraulics, sediment transport, and bank erosion model is the focus of this chapter, additional model evaluation statistics were computed to confirm the level of calibration previously discussed.

[Moriassi et al. \(2007\)](#) recommended using the following three quantitative statistics to evaluate model prediction accuracy: (1) Nash-Sutcliffe efficiency (NSE), (2) percent bias (PBIAS), and (3) ratio of the root means square error to the standard deviation of measured data (RSR). If the $NSE > 0.5$, $RSR \leq 0.7$, and $PBIAS \pm 55\%$ (for sediment), then the calibrated model is deemed satisfactory ([Moriassi et al., 2007](#)). For the aforementioned calibrated bank erosion model, the NSE was found to be 0.56, the RSR was 0.66, and the PBIAS was 28%, which deems the BSTEM calibration and model prediction satisfactory. Equations for the NSE, PBIAS, and RSR are provided in [Moriassi et al. \(2007\)](#).

6.1.3 Model Simulations and Assessment

Several simulations were conducted to assess bank erosion processes and rates with and without dam regulation on the Smoky Hill River, 86 river km downstream of Kanopolis Dam. The first analysis was to determine the dominant mechanisms of failure (toe erosion or geotechnical failure) with and without flow regulation. In addition, median daily erosion volumes were compared with and without flow regulation and statistically analyzed using the non-parametric Mann Whitney U Test at a 5% significance level (as described in Section 3.1.1). Finally, sediment transport results are compared with and without flow regulation and discussed.

[Lammers et al. \(2016\)](#) conducted a sensitivity analysis on BSTEM and observed cohesion was a highly sensitive parameter. Therefore, in addition to these aforementioned simulations, cohesion was manipulated at the erosion cross section of interest to assess sensitivity to the parameter across the range of values measured on the Lower Smoky Hill River. In the model's calibrated form, cohesion was measured and set at 20 kPa at the upper soil layer and 10 kPa at the lower (Fig. 6.7). For the purpose of simplicity, cohesion was set to be the same for each layer to check parameter sensitivity. Figure B.7 in Appendix B was used to determine a range of cohesion values to test. All four streambanks measured on the Lower Smoky Hill River had some level of cohesion, ranging from 1 kPa for a silt loam on Site 2B to 20 kPa for the silt loam soil found on the upper layer of the erosion cross section of

interest on Site 1. Therefore, cohesion at the erosion cross section of interest was dropped to 1 kPa and increased to 20 kPa to observe changes in both bank erosion processes and rates for both with and without flow regulation.

6.2 Results and Discussion

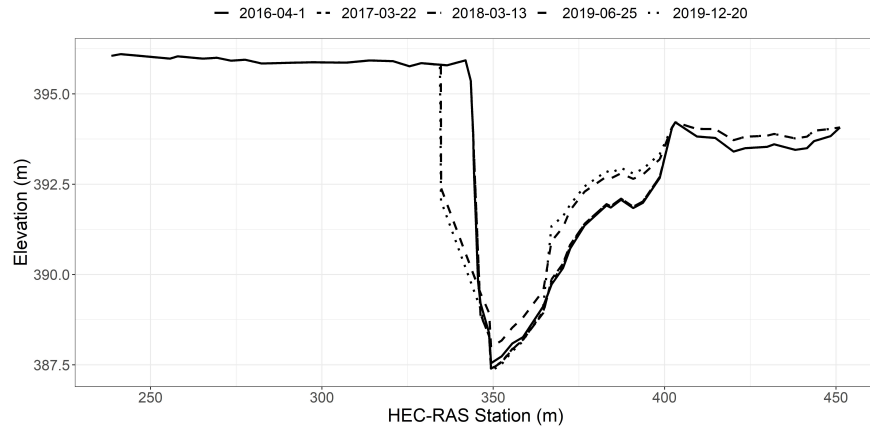
6.2.1 Effect of Flow Regulation on Modeled Bank Erosion

The first simulation was to compare dominant bank erosion processes and rates with and without flow regulation for the entire period of April 1, 2016 to December 20, 2019 (Fig. 6.6). Figure 6.10 shows the cross sections overtime under dam (a) and no-dam (b) scenarios. The results using the calibrated 1D hydraulic, sediment transport, and bank erosion model described in Section 6.1.2 (denoted as EL^1) are shown in Table 6.1. Results in Table 6.1 are also shown for an uncalibrated model in which a sediment rating curve was used as the sediment transport boundary condition (SRC^2) and will be discussed later in this section. Key results presented in Table 6.1 include:

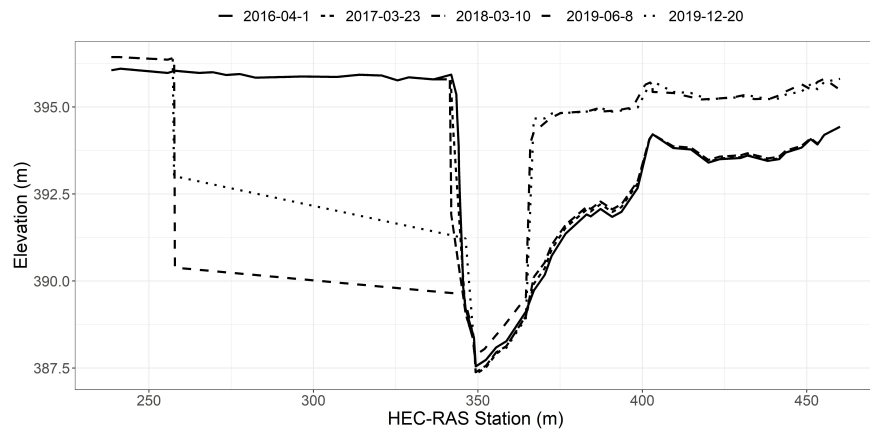
- Toe erosion was the most dominant bank erosion process at Site 1, regardless of the flow condition modeled.
- Toe erosion rates were significantly higher ($\alpha < 0.05$) under no-dam flow conditions, versus with dam flow conditions. However, fewer days of toe erosion were predicted under the no dam conditions versus the with dam conditions.

In accordance with results obtained in the channel planform analysis described in Chapter 5, the hypothesis that inspired this model set-up and comparison was that bank erosion rates would be less when flows were allowed to follow a more natural, unregulated flow regime. Therefore, additional model calibration and assessment was necessary to help further evaluate and explain these results.

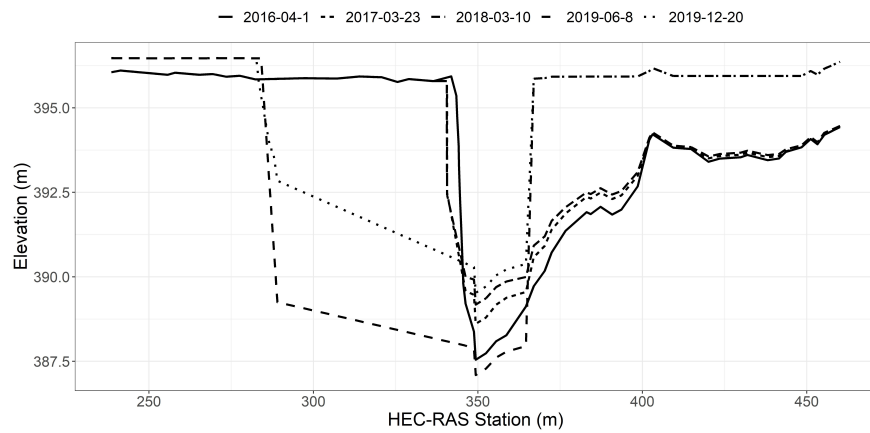
As mentioned, cross section surveys at Site 1 were not conducted in 2020 following 2019 high flows, as control points were lost. Due to this lack of measured data, NAIP aerial



(a) With Flow Regulation - Equilibrium Load



(b) Without Flow Regulation - Equilibrium Load



(c) Without Flow Regulation - Sediment Rating Curve

Figure 6.10: Modeled Bank Erosion and Sediment Transport Results at the Erosion Cross Section of Interest along Site 1 (a) With Flow Regulation using the Calibrated Equilibrium Load as the Sediment Transport Boundary Condition, (b) Without Flow Regulation using Equilibrium Load, and (c) Without Flow Regulation using the Uncalibrated Sediment Rating Curve.

Table 6.1: Summary of Bank Erosion Processes, Rates, and Number of Days for the Erosion Cross Section of Interest on Site 1 of the Lower Smoky Hill River, April 2016 - December 2019.

Bank Erosion Process	Bank Erosion Volumes (m^3)			Median m^3/day			Total Erosion Days		
	With Dam EL^1	No Dam EL^1	No Dam SRC^2	With Dam EL^1	No Dam EL^1	No Dam SRC^2	With Dam EL^1	No Dam EL^1	No Dam SRC^2
Toe Erosion	930	7,835	5,030	24.7	279.7*	115.2*	23	14	14
Geotechnical Failure	160	305	754	5.7	4.7 ^N	10.9 ^N	16	29	29
Total	1,090	8,140	5,784	-	-	-	27	35	31

EL^1 : Results under a calibrated equilibrium load sediment transport boundary condition model.

SRC^2 : Results under a sediment rating curve sediment transport boundary condition model that has not been calibrated. *Significant difference between with dam scenario and without ($\alpha = 0.05$); ^NComparison conducted but a significant difference was not detected ($\alpha = 0.05$).

imagery collected on August 17, 2017 to July 12, 2019 was used as a surrogate. While pixel resolution of these images are low, it was estimated that the retreat of the top of bank at the erosion site of interest between these two dates was about 4 to 5 m. The results from the HEC-RAS 1D hydraulic, sediment transport, and bank erosion model showed a top of bank retreat of 7.5 m between these two dates (see Fig. 6.10a). Another consideration is overbank deposition on the right bank. Figure 6.10a shows up to 1.5 m of deposition on the right bank at the end of the flow regulation simulation. Again, it is difficult to know if that is a believable deposition rate. However, cross section data were collected along Site 2 (see Fig. 2.1) after the 2019 high flows in September 2020. Site 2B is the most similar to Site 1 in terms of bank stratigraphy and geotechnical properties (see Fig. B.7 in Appendix B). Deposition at a cross section along Site 2B was up to 2 m from March 2016 to September 2020 (see Fig. B.8 in Appendix B). Therefore, the differences in both left top of bank retreat and right overbank deposition observed under the flow regulation scenario seem to be a reasonable representation of true conditions.

Figure 6.10b also shows modeled cross sections at various dates under the no flow regulation scenario. During what could have been two major flood events in October of 2018 and May of 2019, the left top of bank retreated a total of 84 m, with toe erosion being the primary cause for this retreat. Furthermore, the right overbank area deposited up to 5 m of sediment. However, the channel, given the equilibrium load boundary condition used (denoted as EL^1 in Table 6.1), remains relatively stable throughout the simulation period. This assumption points to a problem with this simulation set-up.

The original intent of these two model simulations was to compare observed bank erosion with Kanopolis Dam flow regulation (thus, observed conditions) and then again assuming Kanopolis Dam was not installed. However, channel evolution was not thoroughly considered when setting up this experiment. At the time of the first survey of Site 1, 68 years had passed since Kanopolis Dam was installed 86 river km upstream. In other words, the Lower Smoky Hill River has had 68 years to adjust to a new sediment and flow regime. Results from Chapters 4 and 5 show that the Lower Smoky Hill River has changed in both profile and pattern following dam installation. Furthermore, [Williams & Wolman \(1984\)](#) show that between 1946 and 1952, a Lower Smoky Hill River cross section located 92 river km downstream of Kanopolis Dam had deepened (-0.2 m) and widened (5 m) in the four years following dam installation (Site 1 modeled in this study is located at 86 river km). [Legleiter \(2015\)](#) also noticed a shift in channel evolution downstream of a recently installed dam in Wyoming. These measurements and observations provide additional evidence that channel dimensions have also changed due to Kanopolis Dam installation upstream.

Instead of modeling what bank erosion could have looked like had Kanopolis Dam not been installed, what was actually modeled is what could happen if either (a) Kanopolis Dam is removed or (b) flows are drastically changed to mimic more natural (or flashy) conditions. If either of these were to happen, the channel evolution process (e.g., Fig. 1.1) would circle back to a “disturbed” condition where, in this case, disturbance would be the removal of the dam or drastic changes to flow releases. What is likely being shown in Figure 6.10b is the result of disturbance: erosion and deposition to allow for the channel to adjust to a new flow and sediment regime. Therefore, these results are no longer surprising, as one would expect more erosion under the expected natural (without dam) conditions given the current state of the channel’s pattern, profile, and dimensions, which reflect geomorphic adjustments to the current flow and sediment regimes. Results from [Wells et al. \(2007\)](#) agree with these findings, in which the researchers conducted a 1D flow, sediment transport, and bank erosion model using the numerical modeling tool, CONCEPTS, to evaluate changes to sediment loads following removal of low-head dams on a river in Michigan. The results showed excessive erosion of the banks and bed following dam removal, however this was mostly attributed to

the upstream migration of headcuts or knickzones following dam removal (see Fig. 1.1 for example). In the case of the Smoky Hill River, excessive erosion modeled is brought on only by the uncontrolled flows that the stream is no longer capable of transporting. Therefore, the best way, and perhaps the only way, to compare without dam or pre-dam bank erosion rates is to have measured data of pre-dam channel conditions and modeling, if possible, the flows that could have occurred in 2016 through 2019 through a pre-dam reach with similar boundary conditions.

6.2.2 Modeling Streambed Instability and Its Effect on Bank Erosion

Given the disturbance of removing Kanopolis Dam, re-running the calibrated 1D hydraulic, sediment transport, and bank erosion model with the unregulated (no dam) flow condition and the equilibrium load boundary condition is no longer a valid assumption, as the streambed would no longer be in dynamic equilibrium. When the equilibrium load is assumed, if there is any material that is able to deposit, it does so only in the overbanks, which can cause the stream to decrease in cross sectional area, therefore causing velocity and water depth to locally increase and thus increasing applied shear stress to the bank, exacerbating toe erosion rates, as shown in Figure 6.10b and summarized in Table 6.1.

Instead, a sediment load rating curve would be a better assumption. Ideally, a sediment load rating curve would be created using sediment load data collected prior to dam installation, given the high sediment trapping efficiency of Kanopolis Lake (95%) described by Juracek (2011). In addition to a sediment load rating curve, suspended load gradation for various flows is also required for HEC-RAS 1D sediment transport modeling using this boundary condition (USACE, 2020c). Using a sediment load rating curve over the equilibrium load assumption would allow for the streambed to also erode and/or deposit and could drastically change the bank erosion rates and processes observed here.

To demonstrate this, a simple sediment rating curve was developed using 23 total suspended sediment (TSS) measurements from the Lindsborg and Mentor USGS gages between

1951 to 1981 by the USGS. The top three TSS measurements were removed from the dataset as they were considered outliers (i.e., $>[\text{upper quartile} + 1.5 \times \text{interquartile range}]$). Flows were between 14.7 to 227 cms when TSS samples were retrieved. A simple linear regression (normally distributed data, $R^2 = 0.34$) was conducted to develop the following equation to obtain a sediment rating curve:

$$TSS = 6.31Q + 388.74 \quad (6.5)$$

Where TSS is total suspended sediment concentration (mg/L) and Q is the flow (cms). Suspended sediment gradation was estimated using data from the Kansas River at Wamego, KS (Albert, 1973) and is presented in Table B.3 in Appendix B. The Smoky Hill River creates the Kansas River when it joins the Republican River near Junction City, Kansas. The Kansas River is also a sand-bed meandering stream with cohesive banks. Therefore, it is assumed that the Kansas River provides a reasonable estimate of suspended sediment gradation for the Smoky Hill River.

Estimated sediment rating curve information was input to the calibrated 1D hydraulic, sediment transport, and bank erosion model. The model was not re-calibrated for sediment transport, as it is only intended to show how bank erosion can change under different boundary conditions. It is noted that the model using this sediment rating curve showed 1.2 m and 2 m of deposition on the bed from the 2016 to 2017 survey and from the 2017 to 2018 survey at the erosion cross section of interest, respectively, which is higher than the observed 0.4 m total. Additionally, the bed was 0.5 m higher from the measured 2016 elevation after the 2019 high water conditions were modeled.

Without calibrating the model, toe erosion was the dominant streambank erosion process under the no flow regulation scenario with a total volume of erosion approaching $5,800 \text{ m}^3$ and the top left of bank retreating 55 m (see Fig. 6.10c). For comparison, bank erosion rates under the sediment rating curve boundary condition showed that geotechnical failure dominated flow regulation erosion processes with a total erosion volume of 880 m^3 and 10 m of retreat of the left top of bank. This total volume and retreat rate was similar to that

predicted under the equilibrium load boundary condition, although the dominant mechanism of erosion was reversed. Streambank erosion data for the sediment rating curve boundary condition under no flow regulation is presented in Table 6.1, denoted as *SRC*². Note that both the volume of sediment eroded (Table 6.1) as well as the left top of bank retreat rate decreased when the bed was allowed to erode and deposit overtime, given the assumed sediment rating curve and suspended sediment gradation. Also note that toe erosion was still the dominant streambank erosion process for the no flow regulation condition.

6.2.3 Effect of Flow Regulation on the Number of Erosion Days

The number of erosion days per simulated scenario is also provided in Table 6.1. It is interesting to note that while toe erosion is significantly higher under the unregulated (no dam) flow condition, the number of days in which toe erosion occurred is nearly half of the number of days predicted to have occurred under flow regulation (with dam) conditions, suggesting that there is more opportunity for geomorphic work under a flow regulation regime. This is a significant observation, as it may provide insight as to why sinuosity has increased following Kanopolis Dam closure, as presented in Chapter 5. In Chapter 3, Figure 3.4 shows that moderate daily flows (between 5 to 40% daily exceedance probability) have increased following Kanopolis Dam installation. Furthermore, hydrograph rise and fall rates are slower than they were prior to Kanopolis Dam installation. In other words, moderate flow events occur more often and for longer periods of time than what would have occurred prior to dam installation, resulting in more opportunity for toe erosion to occur during these flow flows. Results from Brandt (2000) and Ma et al. (2012) echo this finding, as the researchers note that moderate flows just below the low-lying floodplain (i.e., less than the bankfull discharge) do a considerable amount of work downstream of dams as a result of the step-like nature of the flow hydrograph.

In addition, with higher and longer flows comes opportunity to drown out low-lying riparian vegetation and/or maintain a higher vegetation line, creating more exposed toe surface area. Vegetation can reduce bank erosion by increasing shear strength due to the

presence of the root structure (Pollen, 2007; Simon & Collison, 2002) and by decreasing applied hydraulic forces by increasing surface roughness (Gyssels et al., 2005). Depending on the critical shear stress and erodibility of the material of which the more exposed toe is composed of, toe erosion under a flow regulated stream may be occurring not only at meander bends, but along straight segments of the stream as well. In other words, it is believed that flow regulated streams have more areas of active toe erosion, perhaps system wide in the portion of the Smoky Hill River affected by Kanopolis Dam (which is at least 111 river km downstream, as shown in this dissertation), versus what would have occurred under more natural and flashy hydrograph conditions. Friedman et al. (1998) found that in Central Plains rivers of the United States downstream of large dams where channel widening occurred (as is the case for the Smoky Hill River), there was a decrease in woody riparian pioneer species (e.g., willows) with a concomitant reduction in channel migration rates, or the area of the floodplain reworked per length and per time as described in Chapter 5. Results in Chapter 5 show that channel migration rates have decreased, while lateral migration (through streambank erosion and deposition) have increased on the Lower Smoky Hill River following dam installation (as shown by an increase in stream length and sinuosity). Therefore, it is reasonable to presume that toe erosion is now a system wide issue that may be further amplified by the reduction in riparian pioneer species.

Toe material exposure and toe erosion have also been observed throughout the surveyed stream reaches. For example, while the purpose of this numerical modeling experiment was to observe changes in streambank erosion along meander bends, toe erosion is also apparent on the inside of meander bends, as shown in Figures 6.5 and B.8 on the depositional side of the bend, which in these examples was the right bank. Evidence of toe erosion was observed on the straight reaches of the Smoky Hill River as well. Figure 6.11 depicts repeated survey data for a cross section located 251 m upstream of the erosion cross section of interest at Site 1, along the straight reach upstream of the meander bend (see Fig. 6.2 for location). Despite this cross section being along a straight reach, toe erosion is evident on the left bank of the cross section with deposition on the right, an indication that the meander directly downstream may eventually increase in length. Figure 6.12 provides photographs of all

of these aforementioned cross sections, showing system wide evidence of exposure of toe material, resulting in an increased risk of toe erosion along the Lower Smoky Hill River, downstream of Kanopolis Dam. Increased toe erosion, and as a result, widening of the low flow channel that contains the median daily flow has been observed in several rivers downstream of large dams (e.g., [Chen et al., 2019](#); [Gilliam, 2011](#); [Graf, 2006](#); [Hupp et al., 2009](#); [Ma et al., 2012](#)).

With every possible bank erosion event under flow regulation conditions, a small sediment load brought on by toe erosion, and at times, geotechnical failure, is being introduced at a small spatiotemporal scale (i.e, cross section and daily scale). However, given the observed evidence that this is occurring at a system wide scale and over a long duration, the cumulative effect of toe erosion is concerning. As mentioned in Chapter 5 following the channel planform analysis, it was stated that it appears that the Smoky Hill River is “subtly becoming more wiggly,” and system wide toe erosion occurring over a long duration is likely the culprit.

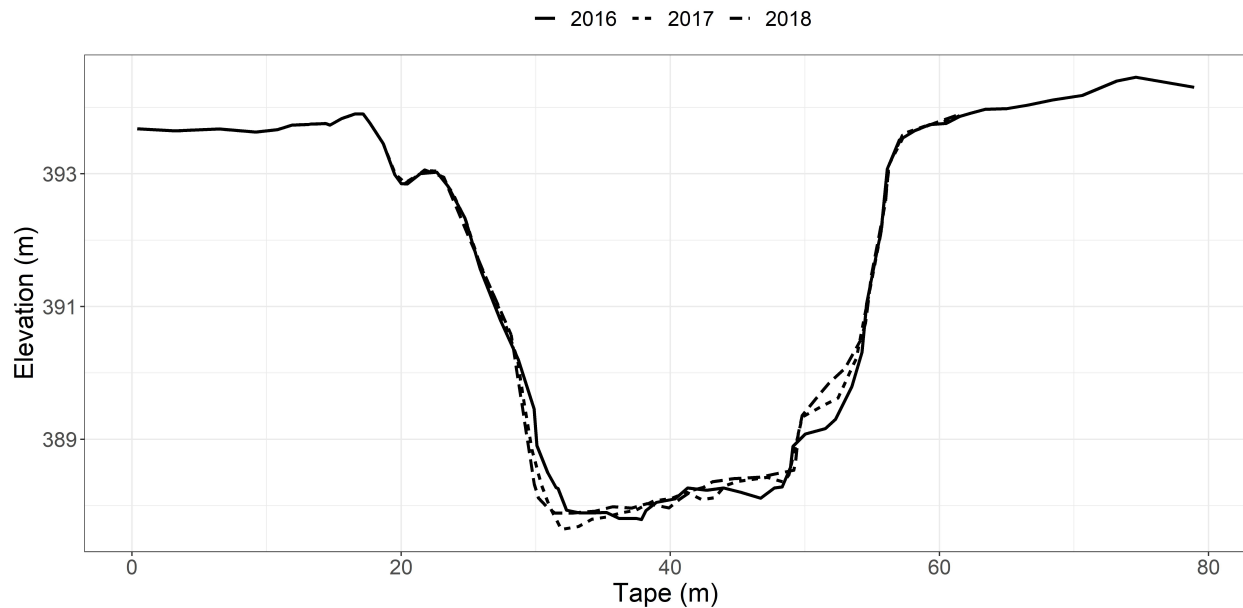


Figure 6.11: Site 1 Straight Reach Cross Section Results, 2016-2018. *Note:* See Fig. 6.2 for location in reference to the erosion cross section of interest. Bankfull elevation is located at 391.5 m and will be discussed more in Chapter 7. Bankfull cross sectional areas range from 49 m^2 in 2016 to 51 m^2 in 2018.

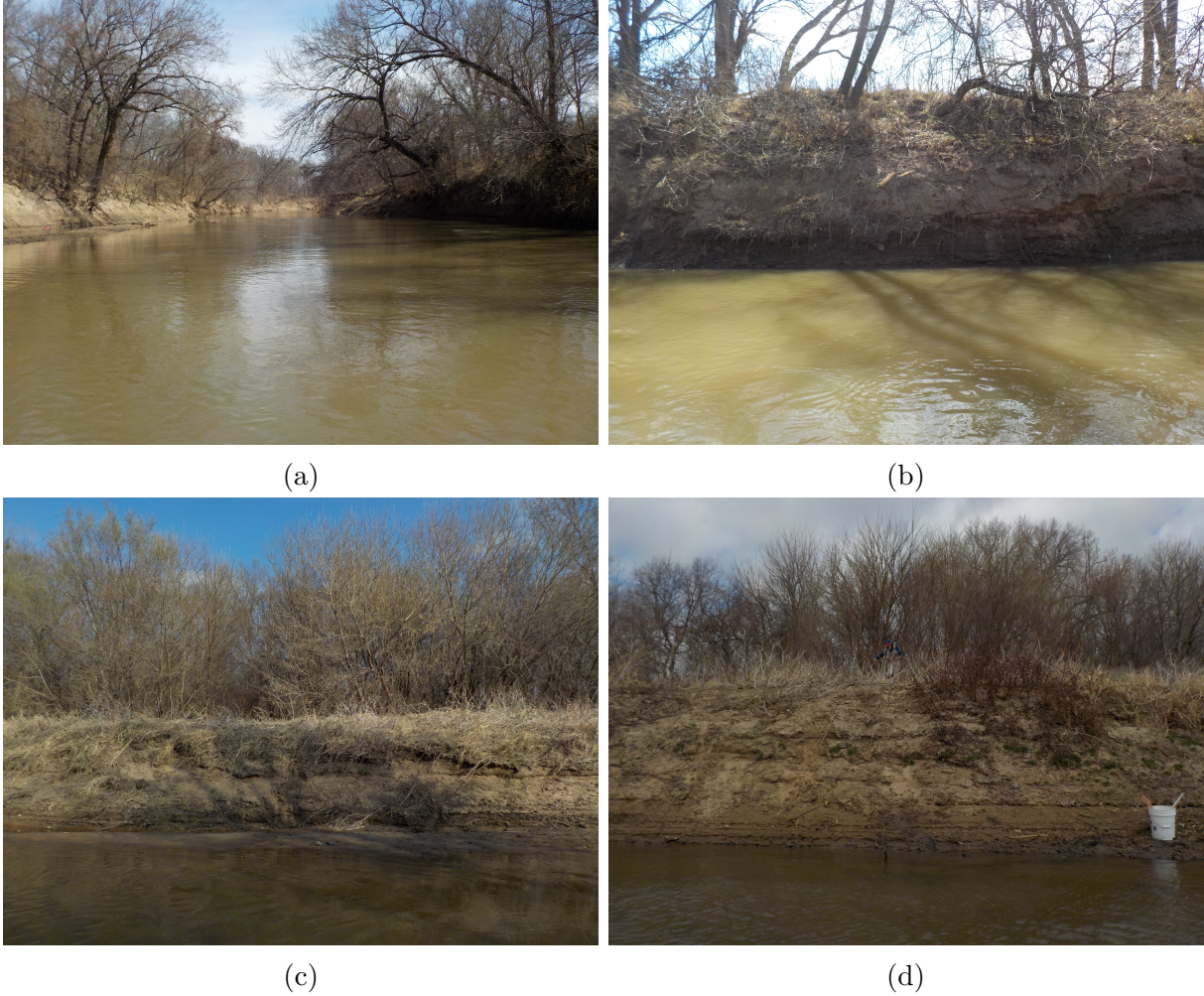


Figure 6.12: Photograph Examples of Toe Erosion Observed throughout the Lower Smoky Hill River. Photos taken looking (a) Upstream of the Cross Section presented in Fig. 6.11 on Site 1, (b) At the left bank of the cross section presented in Fig. 6.11, (c) At the right bank of the erosion cross section of interest at Site 1 (Fig. 6.5), (d) At the left bank of the Site 2B cross section shown in Fig. B.8 in Appendix B.

6.2.4 Effect of Cohesion on Modeled Bank Erosion

Since cohesion is a sensitive parameter of BSTEM (Lammers et al., 2016), cohesion was manipulated from 1 kPa to 20 kPa (reflecting the range in measured BST data at multiple sites on the Lower Smoky Hill River as shown in Fig. B.7), while holding all other parameters constant in the calibrated 1D hydraulics, sediment transport, and bank erosion model. Based on this analysis, it was found that, regardless of the cohesion value, toe erosion was the dominant bank erosion process for the unregulated (no dam) flow condition and exceeded

geotechnical failure rates, usually by one order of magnitude. Similar results were observed when the uncalibrated sediment rating curve was set as the boundary condition.

Alternatively, adjusting the cohesion value at the erosion cross section of interest on Site 1 resulted in a shift of dominant erosion processes for the regulated (with dam) flow condition. The point of inflection occurred between 10 kPa and 11 kPa, with geotechnical failure dominating when cohesion values were less than or equal to 10 kPa and toe erosion dominating for cohesion values between 10 and 20 kPa. The total volume of eroded sediment, however, stayed about the same with each adjustment, averaging at $1,243 \text{ m}^3$ of eroded sediment with a standard deviation of 166 m^3 . This is a significant finding, as the lower layer at this site has a measured cohesion of 10 kPa. However, by decreasing the upper layer to 10 kPa (instead of 20 kPa), the retreat rate of the left top of bank increases from 7.5 m to 10.1 m, which deviates further from the aerial image measurement of 4 to 5 m retreat for this time frame. Therefore, the original cohesion input of 20 kPa for upper layer provides a more accurate result of true conditions. This streambank also has woody brush at the top of the streambank (as pictured in Fig. 6.4), which can cause shear strength to increase slightly within the root zone (Pollen, 2007; Simon & Collison, 2002), further justifying a higher cohesion value for the upper layer of this streambank.

When evaluating Figure B.7, the median cohesion value is 7 kPa, with Site 1 having the highest measured cohesion values overall. Based on the results of this analysis, one might expect geotechnical failure to dominate on these other measured streambanks at Sites 2 and 3; however, this is likely not the case, given the low measured critical shear stress values obtained, all of which were less than 0.5 Pa. In this case, toe erosion can occur even during extended low to moderate flows (<1.3-year return interval), further supporting the observation of system wide toe erosion discussed previously.

Therefore, based on the field measurements conducted for this study and supported by numerical modeling, toe erosion was found to be the dominant streambank erosion process on the Lower Smoky Hill River up to 111 river km downstream of Kanopolis Dam. In rare cases where cohesion is 10 kPa or less but critical shear stress of the toe material is at least 2.7 Pa, geotechnical failure may dominate. However, geotechnical failure is likely to only

occur on steep-sloped, tall streambanks with minimal sand lenses and little vegetation on or near the streambank along the outside of meander bends.

6.3 Model Limitations and Future Research

With any numerical model, and especially for a 1D, quasi-unsteady hydraulic, sediment transport, and bank erosion simulation, there are limitations. The hydraulics portion of the model has several limitations. The first limitation is the lack of measured flow and/or stage data at the site of interest to more accurately calibrate the model. As described in Section 6.1.2, instead of using measured stage data, the visible flood damage line shown on aerial imagery was used to “calibrate” the 2D, unsteady model and thus inform and calibrate the 1D model. A second limitation to the hydraulic model is that flow in channels is never 1D. The 1D assumption would directly influence velocity and stage predictions. Finally, during the highest peak flow event (446 cms) predicted during the assessed time range, the 1D model was not able to contain the entire volume of floodwater moving downstream in some cross sections. Therefore, assumptions in terrain data were required to allow for flood volume to be transported downstream, as described in detail in Section 6.1.2.

In terms of the sediment transport portion of the model, many assumptions have to be made when measured data are unavailable, and even when measured data are available, selecting the appropriate sediment transport equations (e.g., capacity potential, sorting and armoring, and fall velocity) can also be difficult. The available sediment transport equations will never perfectly predict sediment transport through the system, as sediment transport continues to be an extremely difficult process to numerically model. While the sediment rating curve boundary condition would have been ideal for this particular stream, measured data were limited and quite variable, as shown by the low measured R^2 value presented with Eq. 6.5. Furthermore, suspended sediment gradation, as well as stream temperature, had to be approximated using data from other nearby streams. While general characteristics of these streams are similar, it still does not represent actual stream temperatures and suspended sediment gradation for the Lower Smoky Hill River. Calibration of the sediment

rating curve boundary condition was attempted; however, due to the lack of data and the many “knobs” of calibration available for this condition, it was decided not to attempt to make up sediment transport data just to get the model to fit observed conditions. Even if the sediment rating curve boundary condition was calibrated to observed conditions, it would likely not be representative of what the sediment rating curve would look like if Kanopolis Dam was removed or flow and/or sediment releases were dramatically changed, due to a shift in overall channel evolution.

The BSTEM model in HEC-RAS also has limitations. Flow around meander bends is never 1D in nature, resulting in higher applied shear stresses on the outside of the bend. Higher shear stresses around bends applies to sediment transport as well. To compensate for this, the HEC-RAS 1D Sediment Transport Manual (USACE, 2020c) recommends reducing the measured critical shear stress of the toe material. Alternatively, Daly, Miller, & Fox (2015) multiplied τ in the excess shear equation (Eq. 6.4) by an adjustment factor α to account for the simplified nature of 1D hydraulics. This research followed the advice of USACE (2020c) and reduced the critical shear stress at the erosion cross section of interest; however, it is still an approximation and limitation of the bank erosion model. In addition, bank erosion was only modeled on one side of the river. As shown in Figures 6.5, 6.11, and B.8, as well as the photographs presented in Figure 6.12, both sides should be considered in future bank erosion modeling studies downstream of flood control dams, as it appears that toe erosion can occur on any side of the stream, as well as any location of the stream, including meanders bends and straight reaches.

This study showed that the dominant bank erosion process under flow regulation conditions was toe erosion, a result that will assist stream practitioners in selecting and/or developing practices to best manage bank erosion downstream of flood control dams. However, the primary goal of this experiment was to determine if bank erosion rates were less prior to dam installation. Unfortunately, this was never determined given the model set-up and the bank erosion and sediment transport data available. In absence of physical data, Petts (1979) may provide some insight to this question. The researcher predicted that low cohesive or non-cohesive streambanks may erode more due to the increase in base flow vol-

ume and duration brought on by reservoir operation. In other words, prior to Kanopolis Dam installation, toe erosion may have been higher on the Lower Smoky Hill River during high, unregulated peak flows along specific locations but the cumulative annual contribution of toe erosion may have been less compared to regulated conditions. This pre versus post-dam bank erosion rate hypothesis is supported by the results of the unregulated (no dam) numerical model simulation, with high predicted toe erosion rates, but less predicted toe erosion days (Table 6.1).

Of course, extending this study to more reaches throughout the entirety of the 111 km river reach between Kanopolis Dam and the Mentor USGS gage, as well as obtaining field measurements over longer periods of time, would also be beneficial. While expected dominant erosion processes at Sites 2 and 3 were discussed briefly, it could be beneficial to set up a full-scale model of these sites, similar to what was done here, to confirm, as water surface slope is lower (0.0002 m/m instead of 0.0004 m/m) and cross section dimensions are slightly different (channel area is within 10 m^2 across sites). Additionally, below a flood control dam in North Carolina, Hupp et al. (2009) observed lower toe erosion rates closest to the dam, compared to that observed in the middle reaches, suggesting that increased bank erosion rates may shift downstream overtime. Therefore, it is suggested that future streambank erosion field measurements should include sites closer to the dam to compare rates to those observed in this study. Furthermore, as noted by Petts & Gurnell (2005) and Graf (2005), there is a need for long-term case studies (>10 years) to provide a thorough evaluation of the measured adjustments along specific reaches.

The current form of BSTEM in HEC-RAS is unable to model effects of vegetation, while the latest Excel spreadsheet version (Version 5.5) of BSTEM can (E. Langendoen, USDA-ARS, personal communication, June 7, 2021). Given that the modeled lateral retreat of the left top of bank on the erosion cross section of interest for this study was slightly higher than what was estimated to have actually occurred based on NAIP aerial imagery analysis, the effects of the woody brush shown in Figure 6.4 on top of the left bank may have increased shear strength slightly within the root zone, resulting in decreased lateral retreat rates. Therefore, it would be beneficial for the vegetation portion of BSTEM Version 5.5 to be

incorporated into HEC-RAS. If this were to be incorporated, it would also be beneficial if the length of the vegetative root zone could be an input, allowing for decreased shear strength after riparian vegetation has been completely eroded from the bank of interest.

Several studies have mentioned the reaction of riparian vegetation to dam operation (e.g., Collier et al., 1996; Friedman et al., 1998; Petts & Gurnell, 2005; Schmidt & Wilcock, 2008; Williams & Wolman, 1984), in which both loss and encroachment have been observed. Based on field observation, this study suggests that riparian pioneer species are being damaged and drowned out during long duration releases of moderate to high flows, in an attempt to reduce peak flows and flooding on the Smoky Hill River. Figure 6.13 provides an example of the drowning out of willow trees (*Salix* sp.) following nearly 14 weeks of high, extended flows in summer of 2019. The loss of low-lying pioneer riparian vegetation should be further evaluated to assess the unintended impacts of flow regulation.

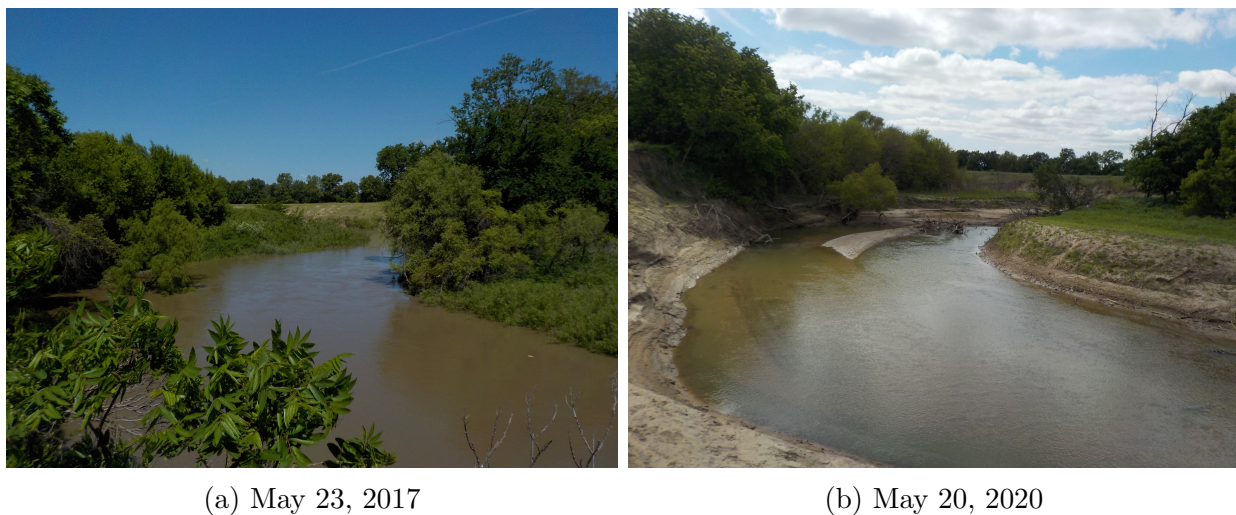


Figure 6.13: Photograph Examples of Loss of Willow Trees (*Salix* sp.) on the Inside of the Meander Bends at Site 3 following nearly 14 Weeks of High (>1.3-year return interval) and Extended Flows in Summer of 2019.

While BSTEM is one of most comprehensive physical models available to simulate bank erosion, it still is not able to simulate subaerial processes (Klavon et al., 2017). Subaerial processes include dry ravel production from wetting and drying and freeze/thaw action, as well as seepage. Subaerial processes weaken bank material, making it more susceptible to failure. Dry ravel production has been observed on the Lower Smoky Hill River, with an

example shown in Figure 6.4 (loose material at toe). While it is unlikely that dry ravel production is more dominant than toe erosion along the the Smoky Hill River downstream of Kanopolis Dam, it should not be ignored. If numerical modeling of sediment transport is difficult, modeling dry ravel production is even more so; however, doing so may provide additional insights of the effect of flow regulation on streambank erosion processes and rates.

Another consideration is the effect of future reservoir operation changes due to human-induced climate change and/or reservoir sedimentation. A recent study by Brookfield & Layzell (2019) showed that fluvial erosion downstream of water supply dams will increase under any changing climate scenario. The researchers note the need to determine which is more important or of higher value: reducing fluvial erosion and/or reservoir sedimentation or protecting against downstream water shortages. The results found by Brookfield & Layzell (2019) are concerning, given that (1) fluvial erosion is the dominant bank erosion process downstream of Kanopolis Dam, (2) annual precipitation is increasing under a changing climate in this region (Rahmani et al., 2015), and (3) Kanopolis Dam flow releases recently changed in 2012 to guarantee water supply downstream (D. Hays, USACE, personal communication, July 9, 2021). Therefore, toe erosion rates should continue to be monitored under a changing climate.

In recent years, reservoir sedimentation and loss of runoff storage has been a major concern worldwide, but especially in the United States, as many dams and their produced reservoirs are nearing their design lives (Fox et al., 2016). To reduce and/or prevent reservoir sedimentation, many new dams are being designed to route or bypass sediment through the dam, while some existing dams are being retrofitted to allow for sediment routing. Techniques may include sediment bypass systems, such as off-channel reservoirs and flood bypass designs, or sediment pass-through systems, such as turbid density currents or drawdown routing or sluicing (Kondolf et al., 2014). In Kansas, government agencies and other stakeholders, are assessing the feasibility to develop a turbid density current pass-through system at the largest reservoir in Kansas, Tuttle Creek Lake (KWO, 2018). This USACE-operated dam was closed for flood control in 1959 and is projected to have lost 90% of its 0.52 km^3 storage capacity by 2070 (KWO, 2021). Given the results from this study showing channel instability

under a no-dam flow condition, the question remains that after over 60 years of sediment-hungry flow releases from Tuttle Creek Dam, what will be the implications of retrofitting and incorporating flow releases containing fine sediment from the dam moving forward? Results from [Zahar et al. \(2008\)](#) below Sidi Salem Dam in Tunisia of northern Africa suggest that channel capacity might decrease downstream, resulting in increased flooding at lower flow releases. Quantitative long-term monitoring following retrofit installation may be the best and only way to answer this question.

Dams provide humanity with many resources: flood control, energy, water supply, recreation, and much more. Dam operation certainly plays a role in the food-water-energy nexus and with a growing global population, there is a higher demand for all three of these. Dams can provide water supply for drinking and growing food, and can also be designed to produce hydropower. In fact, [Kondolf et al. \(2014\)](#) states the majority of large dams being built today are for hydropower in countries such as Asia, Africa, and South America. But there are always environmental costs to exploiting natural resources. Using bank erosion as an example, the process of bank erosion brought on by flow regulation can eat away at arable land while polluting water with excess fine sediment. The Smoky Hill River feeds into the Kansas River, which is a major source of water for large cities downstream. It is imperative that we continue to assess and value damages, especially lost and/or damaged ecosystem services, and adapt our management and operation strategies moving forward.

6.4 Summary

Based on the 1D quasi-unsteady hydraulic, sediment transport, and bank erosion modeling analysis, it has been shown that toe erosion is the dominant streambank erosion process for Site 1, with and without Kanopolis Dam. Furthermore, this analysis shows that bank erosion would be higher under the expected natural (no dam) conditions versus the actual (with dam) conditions. Higher predicted bank erosion rates are likely due to the channel evolution process in which disturbance to the flow and/or sediment regime can cause degradation and/or aggradation and thus, increase bank erosion rates. The prediction of higher bank

erosion rates under an unregulated flow regime for the current channel dimension and profile is especially interesting given that previous streambank erosion studies downstream of dams have recommended altering flow releases from dams to mimic more natural flows (e.g. [Gilliam, 2011](#); [Mohammed-Ali, 2020](#)). Based on the findings of this study, changing the regulated flow regime is not advised.

In addition to toe erosion being the dominant process, it was also shown that this process is occurring in other locations of the stream where bank erosion is typically less common, such as the inside of meander bends and along straight reaches. Toe erosion that is occurring throughout the reach is likely due to the dampened flow hydrograph brought on by flood flow regulation, resulting in a step-like instead of a flashy hydrograph form, as shown in Chapter 3. Moderate flows (5-40% daily exceedance probability) have increased and are occurring for longer periods of time throughout the entirety of the study reach of interest due to flow regulation, resulting in more opportunity for toe erosion on a daily basis. Furthermore, it has been observed that pioneer riparian vegetation has difficulty colonizing due to extended high flow events (>1.3 -year return interval), resulting in more exposed streambank and potentially amplifying toe erosion rates. In some cases, toe erosion may be occurring even during low flows ($<$ median daily flow), due to the erodibility of the toe material, similar to that shown for Sites 2 and 3. This observation is invaluable, as it helps explain the increased sinuosity, observed in Chapter 5, that has occurred since Kanopolis Dam installation. Therefore, this portion of the Lower Smoky Hill River affected by flow regulation is “subtly becoming more wiggly,” likely due to increased toe erosion occurring at a larger spatial scale and longer temporal scale.

The following chapter summarizes the results of this dissertation and answers the overarching question: Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how?

Chapter 7

Managing the Downstream

Geomorphic Effects of a Flood

Control Dam

On February 17, 1948, the USACE-owned Kanopolis Dam was closed on the Smoky Hill River in Ellsworth County, Kansas to begin storage, as well as flow regulation downstream. While this dam has prevented nearly 2.5 billion dollars in flood damage ([USACE, 2020a](#)), as the saying goes “no good deed goes unpunished.” In this case, the punishment is downstream channel instability and increased erosion due to channel evolution processes following dam installation. The installation of large dams peaked in the mid-1900s and the research of the effects of these flow control structures soon followed. As a result of these studies, there is scientific consensus that dams affect both the flow and sediment regimes, and thus also affect the downstream channel geomorphology. While previous research on the Smoky Hill River downstream of Kanopolis Dam has shown changes to the flow and sediment regime, as well as changes to the bed stability and channel width (e.g., [Juracek, 2001, 2011](#); [Williams & Wolman, 1984](#)), effects on channel planform and streambank erosion rates and processes were unknown. Furthermore, very few studies have focused their analyses on if and how dam installation affected bank erosion processes and rates downstream.

In 2016 and 2017, woody revetments were installed along the toe of eroding streambanks of the Smoky Hill River, up to 94 river km downstream (see Fig. 2.1). These projects were monitored through annual geomorphic surveys by Bigham et al. (2020) to evaluate their effectiveness. Following over 14 weeks of high flow (>1.3 -year return interval, post-dam, see Fig. A.1) in the summer of 2019 (Fig. 3.1), the woody revetments failed and were washed downstream. While several potential design and/or installation issues exist, it was unknown if and how the operation of Kanopolis Dam may have influenced woody revetment failure. The overarching question that drove this research was:

Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how?

Addressing this overarching question, the following research questions were posed:

1. Is there a shift in downstream streambank erosion process following large dam installation?
2. Is there a bankfull condition downstream of a large dam?
3. Can stream practitioners use available numerical modeling tools to assess downstream geomorphic change following dam installation and thus assist in channel erosion reduction strategies?

These questions were answered after conducting the following research objectives:

- Extend previous evaluations of the effects Kanopolis Dam closure has had on flow and channel bed stability in space (up to 111km downstream) and time (through 2020).
- Assess the impact of Kanopolis Dam closure on downstream channel planform.
- Evaluate the effects flow regulation has had on streambank erosion processes and rates.

This chapter summarizes the results from these three research objectives and answers the questions posed here.

7.1 Application of Results

Table 7.1 provides a summary of the results obtained from research objectives 1 through 3 (Ch. 3 through 6) regarding the effects of Kanopolis Dam installation on the Smoky Hill River up to 111 river km downstream. These results, along with supplemental analyses, inform answers to the aforementioned research questions.

Table 7.1: Summary of Results from Research Objectives 1 through 3 Evaluating the Effect of Kanopolis Dam Installation on the Smoky Hill River up to 111 river km Downstream.

Note: EP: daily exceedance probability

Objective 1a (Ch. 3) Effect on Flow	Objective 1a (Ch. 4) Effect on Bed Stability
- Dampened peak flows - Step-like hydrograph - Moderate flows (5-40% EP) are higher with longer duration - Annual peak flow occurs at the same time of the year	- Aggradation up to 48 river km upstream - Degradation close to the dam - Some aggradation occurred at least 64 river km downstream - Bed is stabilizing since 2010
Objective 2 (Ch. 5) Effect on Channel Planform	Objective 3 (Ch. 6) Effect on Bank Erosion
- Channel migration has decreased 69% - Sinuosity and stream length has increased 5%	- Toe erosion is the dominant process and occurring throughout the Lower Smoky Hill River - Bank erosion rates would be higher if dam was removed

7.1.1 Is there a shift in downstream streambank erosion process following large dam installation?

Based on the results of Chapters 5 and 6, it is clear that toe erosion is the dominant bank erosion process up to 111 river km downstream of Kanopolis Dam. It was observed that toe erosion is occurring not only along the outside of meander bends of the Lower Smoky Hill River, but also on the inside of bends and along straight reaches. Based on the results of the bank erosion model (Ch. 6), as well as measured cross section data (e.g., Figures 6.5, 6.11 and B.8), the low to moderate flow channel (<5% daily exceedance probability (EP))

of the Lower Smoky Hill River has widened and/or is widening to support more frequent moderate flows (5-40% EP) with a longer flow duration, as shown in Chapter 3. The channel widening hypothesis can be further supported by other studies in which extended moderate flows coincided with channel widening (e.g., [Chen et al., 2019](#); [Gilliam, 2011](#); [Graf, 2006](#); [Hupp et al., 2009](#); [Ma et al., 2012](#)).

The increased duration and magnitude of moderate flows may be affecting the recruitment and colonization of pioneer woody riparian species, such as willow (*Salix* sp.), that have the ability to protect against both fluvial erosion, as well as geotechnical failure. An example of this loss is shown in Figure 6.12 and Figure 6.13. Similarly, [Friedman et al. \(1998\)](#) found that in Great Plains rivers of the United States downstream of large dams where channel widening was observed, there was a decrease in woody riparian pioneer species consistent with a reduction in channel migration rates. Results in Chapter 5 show that channel migration rates, which includes both streambank erosion and avulsions, have decreased on the Lower Smoky Hill River following dam installation, while lateral migration rates, which includes only streambank erosion, have increased. Additionally, Figure 6.13 provides evidence of how extended high flows that occurred in 2019 can result in loss of previously established woody pioneer species. [Friedman et al. \(1998\)](#) believed that a decrease of woody pioneer species was occurring due to fewer avulsions and mass erosion events exposing raw surfaces, as well overall channel degradation caused by dam installation (as shown in Chapter 4). In addition to these hypotheses by [Friedman et al. \(1998\)](#), the observations made here point to the step-like hydrograph causing newly established pioneer species to drown out during times of high and extended flows. In Figure 6.13, note the exposure of the toe material after over 14 weeks of extended high flows (>1.3-year return interval) in 2019, as well as the exposed toe material shown in Figure 6.12 prior to the 2019 event.

Based on the results of Chapter 6, in the zone of the Lower Smoky Hill River affected by Kanopolis Dam operation, toe erosion is occurring at low, but cumulative, rates on banks made of more cohesive materials, and is higher on banks composed of less fluvial-resistant materials (e.g., Sites 2 and 3, see Figure B.7), similar to the hypothesis made by [Petts \(1979\)](#). This research found that on highly resistant, steep-sloped banks lacking vegetation

on the outside of meander bends, geotechnical failure may be the more dominant erosion process. However, based on field observation of bank characteristics and erosional features, the described scenario is uncommon in comparison to system wide toe erosion.

It is also unclear if these bank erosion observations apply to all rivers downstream of flood control dams. As mentioned by [Petts \(1979\)](#), [Schmidt & Wilcock \(2008\)](#), and [Williams & Wolman \(1984\)](#), dominant drivers are likely dependent more so on the sediment boundary conditions. For example, in deeply-incised streams of highly cohesive material, geotechnical failure may dominate, as demonstrated in this study. However, results from downstream of a flood control dam in North Carolina ([Hupp et al., 2009](#)) and a water supply dam in Wyoming ([Gilliam, 2011](#)) provide support that toe erosion might dominate regardless, at least at a system wide scale affected by flow regulation, given the differences in hydrophysiographic region and stream channel size, slope, and boundary conditions inherent to these three sites.

Finally, the question remains as to if and how bank erosion processes and rates may have changed following Kanopolis Dam installation. The results from Chapter 5 suggest that lateral migration rates have increased, even though channel migration rates have decreased, given that stream length, and thus sinuosity, have increased following Kanopolis Dam closure. Chapter 6 shows that toe erosion rates are low at a cross section scale, especially on banks with a higher critical shear stress (τ_c , Eq. 6.4) of the toe material. Low rates of bank erosion as well as an increase in lateral migration seems counterintuitive. But when both the spatial and temporal scale at which toe erosion is occurring is considered, a subtle increase in stream length over time seems possible. This suggests that, at the very least, the scale and frequency at which bank erosion is occurring has likely shifted following Kanopolis Dam installation.

As mentioned in Chapter 6, [Petts \(1979\)](#) also provides insight towards answering the question as to if and how bank erosion processes and rates may have changed following dam installation. The researcher predicted that streambanks having low to no cohesion may erode more due to the increase in base flow volume and duration caused by reservoir operation. In other words, prior to Kanopolis Dam installation, toe erosion may have been higher on the Lower Smoky Hill River during high, unregulated peak flows but the cumulative annual

contribution of toe erosion may have been less compared to flow regulated conditions. When comparing this prediction to the bank erosion modeling results from Chapter 6, bank erosion rates were expected to be five times higher under an unregulated (no dam) flow condition versus a regulated (with dam) flow condition; however, overall days of toe erosion were less under an unregulated flow condition (see Table 6.1). For a more confident answer to the question posed here, the best way, and perhaps the only way, to compare without dam or pre-dam bank erosion rates to post-dam rates is to have measured data of pre-dam channel conditions.

7.1.2 Is there a bankfull condition downstream of a large dam?

It is unclear in the literature how flow regulation affects the bankfull or channel forming discharge below dams. Leopold et al. (1964) explains that, at least on unregulated streams, annual peak flows having a return interval of 1 to 2-years control the dimension, pattern and profile of streams. While high flows are effective at erosion and transporting sediment, they are rare, thus it is these moderate and more frequent flow events that have the greatest effect on stream morphology.

As discussed in Chapter 1, there is disagreement in the literature in regard to how dam installation affects bankfull flow events. Two assumptions are often made when evaluating this parameter downstream of dams:

1. Bankfull discharge (volume/time) stays the same but its annual frequency will increase, decrease, or stay the same (Brandt, 2000; Church, 1995; Page et al., 2005; Petts, 1979; Petts & Gurnell, 2005; Williams & Wolman, 1984)
2. Bankfull discharge changes and its associated daily and/or annual frequency might also change (e.g., Lyu et al., 2020; Ma et al., 2012; Wu et al., 2008)

Ma et al. (2012) argued that bankfull flow may not exist below dams following a 60-year study downstream of four large dams on the Lower Yellow River in China, with varying flow and sediment (i.e., flushing) release schedules across both space and time. Brandt (2000)

obtained a similar observation, noting that higher median daily flows may do a considerable amount of work downstream of dams, rather than bankfull flow. Using a combination of geomorphic surveys, the calibrated HEC-RAS 2D hydraulic model described in Chapter 6, as well as the results presented in Chapters 3 and 4, the question of existence of a bankfull condition below a large dam was answered for the Smoky Hill River downstream of Kanopolis Dam.

To begin, it was assumed that bankfull flow exists, contrary to the findings of [Ma et al. \(2012\)](#), as sediment flushing from Kanopolis Dam does not occur. In unregulated streams, the first step to identifying bankfull stage is to conduct a flood frequency analysis of annual peak flows. This was accomplished using the Log Pearson Type III Distribution and annual peak flow data from 1949 to 2020 (post-dam) at the Mentor USGS gage. The results of the flood frequency analysis are provided in Figure A.1 in Appendix A. This analysis indicated the 1-year to 2-year return interval ranged from 10 to 110 cms, respectively. Often times bankfull flow occurs at a 1.5-year return interval ([Dunne & Leopold, 1978](#)). The 1.5-year return interval flow at the Mentor USGS gage was found to be 60 cms.

The next step was to identify field indicators of possible bankfull flow elevation. The results of Chapter 4 show that the bed appears to be stabilizing over the last decade (2010 to 2020). [Rosgen \(1996\)](#) provides guidance for identifying bankfull in streams based on field indicators, such as a low-lying floodplain, grade change, change in particle distribution, rock staining, etc. However, identifying bankfull indicators in incised streams can be difficult as field indicators can be absent. Fortunately, since the bed elevation appears to be stable at this site, bankfull field indicators should be present. Reviewing data from the erosion cross section of interest (Fig. 6.5), as well as the upstream cross section on the straight reach (Fig. 6.11), a low-lying floodplain, as well as a noticeable grade change appear at elevations 390 m to 392 m.

Using the calibrated 2D hydraulic model as described in Chapter 6, these elevations correlate to a discharge of 45 to 61 cms, which is within the range of flows for the 1-year to 1.5-year return interval flow event. Based on the results of the surveyed cross sections, and based on the flow events experienced in 2016 through 2018, all of which had a peak flow

event around 50 cms, it is believed that a predictable bankfull event occurs, even under flow regulation at a stage of 391.5 m at the erosion cross section of interest, which represents a flow of 51 cms (1.3-yr return interval). This elevation is roughly 3.5 m above the channel, which lies close to the vegetation line and exposed bank toe that is shown Figure 6.12.

The flood frequency analysis of the Ellsworth USGS gage, presented in Figure A.2 in Appendix A, and described in Section 3.1.1, shows that the bankfull flow at this control site (located 46 river km upstream of Kanopolis Dam) is 100 cms and has an annual return interval of 1.3 years. Since this was considered a control site for the assessed Lower Smoky Hill River reach and since there are 92 years worth of flow data available to conduct this flood frequency analysis at the Ellsworth USGS gage, this bankfull flow observation at the Ellsworth USGS gage suggests that bankfull flow has maintained its annual frequency (1.3-year) but has decreased its discharge below Kanopolis Dam. In other words, dam operation continues to follow a similar flow release pattern as what would have occurred naturally, however, because peak flows have been dampened (as shown in Ch. 3), bankfull flow has also dampened. In terms of daily frequency, the bankfull discharge of 51 cms occurs more often than it used to, with post-dam exceedance probability of 5% compared to a pre-dam exceedance probability of 3% (see Fig. 3.4).

Based on the results of this analysis, this study supports assumption (2), that bankfull discharge has decreased following Kanopolis Dam installation. In terms of frequency, the bankfull flow annual frequency remains unchanged, however, its daily frequency has increased slightly. Pre-dam channel geometry data would help confirm these observations and are presented in the following section.

Evolution of a Single-Threaded Channel Downstream of a Flood Control Dam

As described in Chapter 1, channel evolution models (CEMs, also known as stream succession models) exist to assist in explaining how stream channels adjust with changes to the flow and/or sediment regime, or changes to the channel itself. The process of channel evolution begins when a disturbance occurs. While natural disturbances can occur over a geological

time scale, in the present time, most disturbances that result in stream instability and evolution are influenced by mankind, such as the construction of dams (Williams & Wolman, 1984).

The stage of channel evolution is one of the most influential factors on bank erosion processes and rates through time (Rosgen, 2009; Schumm et al., 1984; Simon & Hupp, 1986). The Schumm et al. (1984) model was the example CEM shown in Chapter 1 (see Fig. 1.1) to help describe the basics of channel evolution and its effect on bank erosion rates. However, as noted by Cluer & Thorne (2014) and also shown by the number of stream succession scenarios provided by Rosgen (2009, 2014), channel evolution may be more cyclic in nature and may skip or revisit various stages overtime due to the dynamic nature of watershed processes, resembling more of continuum, rather than a discrete, linear process.

After answering the first two research questions and finding that (1) bank erosion has shifted in both space (entire river length and on both sides of the river) and time (more fluvial erosion days per year) due to the step-like hydrograph and that (2) a bankfull condition still exists for the regulated Lower Smoky Hill River, a new question surfaced:

What channel evolution sequences has the Lower Smoky Hill River undergone following Kanopolis Dam closure in 1948?

Keane & Sass (2017) provide a thorough review of four highly-cited CEMs that have been developed and used over the last 30 years: (1) Schumm et al. (1984) CEM (Fig. 1.1), (2) Simon & Hupp (1986) CEM (Fig. D.1), (3) Rosgen (2009, 2014) Stream Succession Scenarios (Fig. D.2), and (4) Cluer & Thorne (2014) Stream Evolution Model (SEM, Fig. D.3). Table 7.2 compares and contrasts these four models based on the review conducted by Keane & Sass (2017).

To be able to properly apply available CEMs, stream geomorphic data of the Lower Smoky Hill River prior to Kanopolis Dam closure would be required. From 1946 through 1951, the USACE installed and surveyed 17 degradation/range lines (also known as cross sections), located 0.5 to 126 river km downstream of the proposed Kanopolis Dam (completed in 1948). Six out of the 17 range lines and up 30 river km downstream of the dam have been resurveyed up to 2015. The remainder were resurveyed until 1952 to 1961 (K.S. Campbell-

Table 7.2: Comparison of Common Channel Evolution Models (CEMs) as Summarized by [Keane & Sass \(2017\)](#) *Note:* Schumm is the [Schumm et al. \(1984\)](#) CEM, Simon is the [Simon & Hupp \(1986\)](#) CEM, Rosgen is the [Rosgen \(2009, 2014\)](#) stream succession scenarios, and Cluer is the [Cluer & Thorne \(2014\)](#) stream evolution model.

Model	Linear or Cyclic?	# of Stages	Area Created For/ Applied To	Area Characteristics	Model Considerations
Schumm	Linear	5 stages	Yazoo River Basin, Mississippi	- 100-1000 km^2 drainage areas - Upstream of flood control reservoirs - Heavily channelized - Recent land use changes	Heavy reliance on channel width-to-depth ratios.
Simon	Linear	6 stages	3 streams in Western Tennessee	- Heavily channelized - Sediment dredging - Cohesive banks	Bank stability and vegetation analyses are important components.
Rosgen	Linear/ Cyclic	17 scenarios w/ 3-6 stages	Unknown	Unknown	Relies heavily on the Rosgen (1996) stream classification system.
Cluer	Cyclic	8-10 stages	Geomorphic review of anthropogenic effects to streams in the US, Europe, and Africa.	Stream types and sizes across the globe influenced by man.	- Combines Schumm and Simon models - Adds habitat and ecosystem benefits

Patti, USACE, personal communication, February 11, 2022).

Three of these degradation lines were selected and will be discussed further: (1) Range line 2 located 2.5 river km downstream of Kanopolis Dam (Lat. 38.610175, Long. -97.931782), (2) Range line 9 located 30 river km downstream (Lat. 38.551345, Long. -97.849883), and (3) Range line 15 located 85 river km downstream (Lat. 38.623550, Long. -97.614118). Range line 9 is located one river km downstream from a surveyed cross section conducted by [Bigham et al. \(2020\)](#) while range line 15 is located one river km upstream of Site 1 that was modeled in Chapter 6. Determining the current bankfull elevation at these two range lines was established by using the repeated cross section measurements from 2016 to 2019 conducted by [Bigham et al. \(2020\)](#). Analysis of flow events during this time range (2016-2019), in combination with changes in cross sectional dimensions and notable bankfull features (e.g., grade changes) were used to pinpoint the bankfull elevation. Figure B.9 in Appendix B provides the cross section results from [Bigham et al. \(2020\)](#) used to estimate bankfull stage and dimensions at a cross section located 29 river km downstream of Kanopolis Dam in 2018 (near range line 9). Figure 6.11 was used to determine bankfull stage and dimensions in 2018 at Site 1, located 86 river km downstream (near range line 15). Since no recent cross sections have been conducted by the author closer to the dam, bankfull cross sectional

dimensions obtained from range line 9, as well as notable bankfull features, were used to estimate bankfull stage for range line 1.

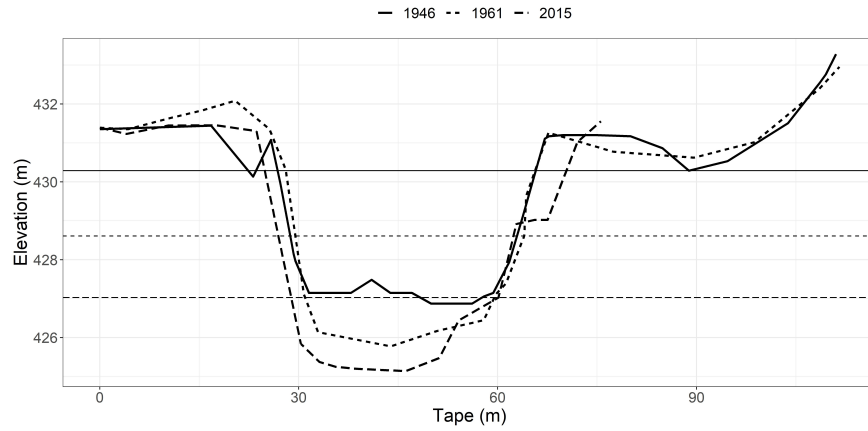
To determine the bankfull cross sectional area in the 1940s and 1960s, the expected pre-dam bankfull discharge (2 times greater than the post-dam bankfull discharge, as previously discussed) as well as notable bankfull features were used to estimate the bankfull stage during this time frame. The cross section overlays, as well as the expected bankfull elevations for these three range lines are provided in Figure 7.1. Table 7.3 provides bankfull dimensions, estimated planform (i.e., estimated sinuosity from Ch. 5), and Rosgen (1996) stream classification for each range line and nearby Bigham et al. (2020) surveyed cross sections overtime.

Table 7.3: Summary of the Cross Sectional and Estimated Profile Change (based on Ch. 5) of the Lower Smoky Hill River from 1946 to 2018 and from 2.5 to 86 river km downstream of Kanopolis Dam. *Note:* River km represents distance from Kanopolis Dam; Bkf means bankfull; XS means cross sectional; ER means entrenchment ratio; W/D means width-to-depth ratio.

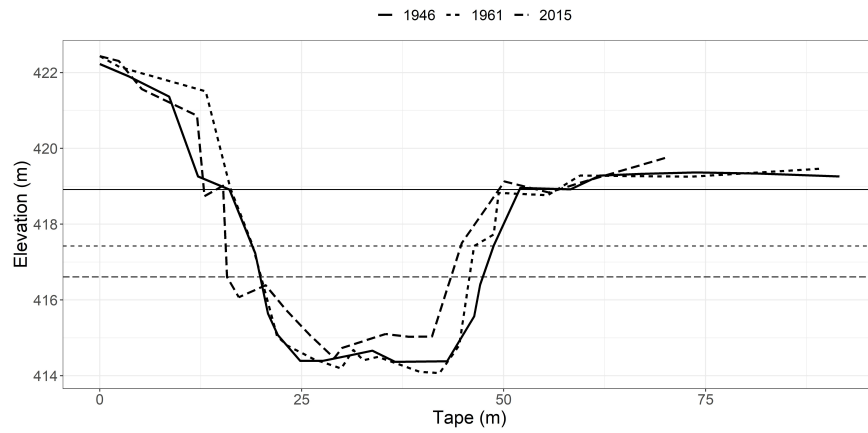
Range Line	River km	Year	Bkf XS Area (m^2)	ER*	W/D**	Estimated Sinuosity	Rosgen (1996) Stream Type
2	2.5	1946	108.9	>2.2	14.8	1.9	C5c-
2	2.5	1961	76.0	>2.2	15.7	1.95	C5c-
2	2.5	2015	42.1	1.2	23.2	2	F5
9	30	1946	80.2	>2.2	11.6	1.9	E5
9	30	1961	74.0	>2.2	10.1	1.95	E5
9	30	2015	34.0	1.2	22.6	2	F5
Bigham et al. (2020)	29	2018	37.6	1.7	26.1	2	B5c
15	85	1951	71.5	>2.2	8	1.95	E5
15	85	1961	67.8	>2.2	7.8	1.95	E5
Bigham et al. (2020)	86	2018	50.5	>2.2	13.3	2	B5c

*ER = channel width at two times the bankfull elevation divided by the channel width at bankfull; **W/D = bankfull width divided by the bankfull mean depth

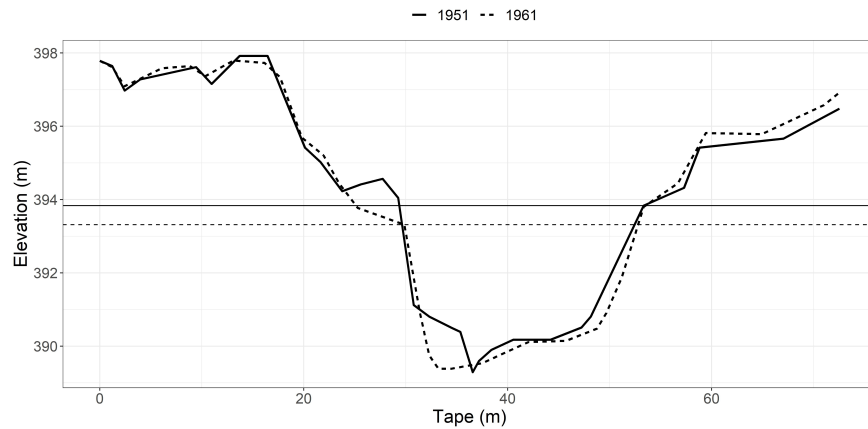
Based on review of Table 7.3, each range line seems to move from a stable Rosgen (1996) stream type (e.g., C5c- or E5) to an unstable stream type (e.g., F5), and in more recent years, back to a more stable stream type (e.g., B5c). Since a B5c is not likely a long term stream type for this low-gradient sand-bed stream (more common on steep gradient, step-pool systems), the Lower Smoky Hill River is likely moving back into a C5c- (meandering stream with sand bars) and may continue to infill to form an E5 stream type (meandering



(a) Range Line 2 - 2.5 river km downstream



(b) Range Line 9 - 30 river km downstream

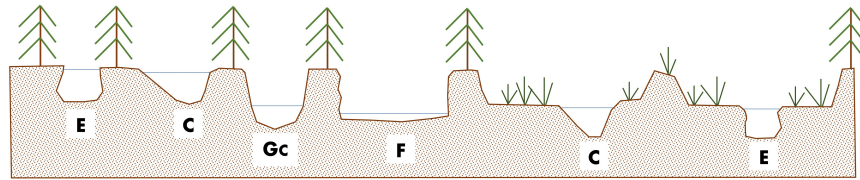


(c) Range Line 15 - 85 river km downstream

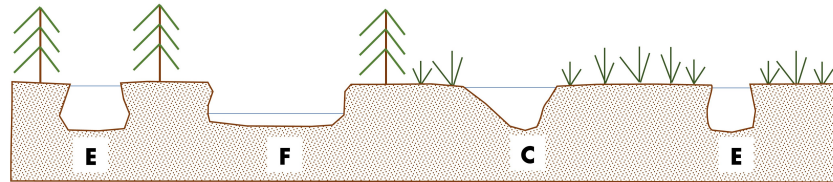
Figure 7.1: USACE Surveyed Degradation/Range Lines: 1946 through 2015. *Note:* Horizontal lines represent estimated bankfull elevation at the time of survey where the line type represents the year shown in the legend.

stream without sandbars) in the long term if sediment supply is sufficient and there are no drastic changes in flow regime or climate. See [Rosgen \(1996\)](#) for detailed information about these stream classifications.

After reviewing the four CEMs presented in Table 7.2 and comparing these models to observed conditions on the single-threaded Lower Smoky Hill River (e.g., Fig. 7.1 and Table 7.3), it appears that the [Rosgen \(2014\)](#) stream succession scenario number 1 (Fig. 7.2a) best describes the channel evolution within the degradational zone directly downstream of Kanopolis Dam (see Fig. 7.1a). Given that the other cross sections downstream of Kanopolis Dam presented in Table 7.3 show E5 channels in 1946 (prior to completion of the dam), it is assumed that range line 2 was an E5 prior to dam construction, which began in 1940. The six years of dam construction could have sparked the channel evolution process near the dam, turning the historic E5 channel into a C5c-, as presented in scenario shown in Figure 7.2a.



(a) [Rosgen \(2014\)](#) Stream Succession Scenario 1 for the Degradational Zone



(b) Proposed Stream Succession Scenario Downstream of the Degradational Zone

Figure 7.2: [Rosgen \(2014\)](#) Stream Succession Scenario 1 (a) and Proposed Scenario (b) Representative of Downstream Evolution of Single-Threaded Stream Impacted by a Flood Control Dam. *Note:* Stream succession scenario 1 shown in (a) represents the degradational zone downstream of Kanopolis Dam while proposed stream succession scenario shown in (b) represents the area of the stream directly downstream of the degradational zone created by Kanopolis Dam.

Based on field observation of fluctuating vegetation downstream of Kanopolis Dam dur-

ing periods of extended, high flows (e.g., Fig. 6.13), one adjustment made to the Rosgen (2014) stream succession scenario 1 was the addition of variable riparian vegetation. Rather than just grasses and forbs as shown in the drawings of the Rosgen (2014) stream succession scenarios (see Fig. D.2), riparian trees were added for pre-dam conditions that then transition into pioneer riparian vegetation (e.g., willows) during later stages of stream succession following dam closure. A transition in vegetation as shown in scenario 1 of Figure 7.2 is necessary to best describe conditions downstream of Kanopolis Dam following closure, as shown by the photographs of the willow grove fluctuation in Figure 6.13 and further discussed in Section 7.1.1. The vegetation adjustment to the Rosgen (2014) stream succession scenario 1 for the degradation zone downstream of large dams is also consistent to observations made by Friedman et al. (1998) downstream of dams on other Great Plains rivers in the United States, in which pioneer woody riparian vegetation appeared to decrease following dam installation.

Williams & Wolman (1984) hypothesized that the degradational zone created by Kanopolis Dam likely ended around 15 river km downstream. After the 2015 resurveys from the USACE, the degradational zone ends between 9 and 11 river km downstream of Kanopolis Dam. Therefore, the remaining two range lines described in Table 7.3 and shown in Figure 7.1 as letters (b) and (c) represent a different channel evolution process. After reviewing the four CEMs presented in Table 7.2, none of the models describe what was observed downstream of the degradational zone and still in the region influenced by flow regulation (up to 111 river km downstream) on the single-threaded Lower Smoky Hill River. In this region, degradation has never been observed (see Fig. 7.1); however, based on the bankfull condition results discussed in this section, bankfull flow has been decreased by 49% due to flow regulation. The Lower Smoky Hill River responded to this reduction by first adjusting to a flatter slope through bank erosion only (see Ch. 5 and 6), followed by sediment deposition/aggradation in the channel to form a smaller cross sectional area to carry this smaller bankfull flow (as shown in Table 7.3). While Rosgen (2014) stream succession scenario 9 shows an adjustment from a stable C channel to an unstable F channel through aggradation, this scenario still does not represent what is described in Table 7.3. Therefore, a new stream

succession scenario is presented in Figure 7.2b. The proposed scenario moves from stable E channel type to an F channel through bank erosion and channel widening brought on by the step-like nature of the regulated hydrograph (Ch. 3) and modeled in Chapter 6. After the stream slope has flattened enough through bank erosion (as shown in Ch. 5), sediment begins to deposit in the channel to form a C channel type and could eventually return to a smaller cross-sectional area E channel type if sediment supply is sufficient and no major changes to flow regime or climate occur. Similar to the vegetation presented in scenario 1 shown in Figure 7.2a, vegetation transitions from riparian trees to riparian pioneer species due to the step-like nature of the flow-regulated hydrograph, resulting in the drowning out of large riparian trees and thus the ease of colonization and re-colonization of pioneer species overtime.

The stream succession scenarios presented in Figure 7.2 for the geomorphic adjustment of a single-threaded channel downstream of a flood control dam are compared to similar studies presented in Chapter 1. Table 1.1 provides a list of case studies that provide a range of channel width response to upstream dam closure, with 37% of the studies reporting channel narrowing and 58% reporting channel widening. Of the flood control dams listed in Table 1.1, 22% reported channel narrowing while the remaining 78% reported channel widening. It is noted that the stream succession scenarios presented in Figure 7.2 allows for both width responses, widening during the G or F channel type and narrowing during aggradation/deposition to form a new stable C or E channel. When reviewing Table 1.1, there is no clear time range of how long it takes to go from one stage to the next. In fact, the only clear difference between the studies presented in Table 1.1 are the dam locations. This research shows that a shift from a natural, flashy to a man-made, step-like hydrograph due to the creation of an upstream dam, whether for flood control, hydropower, or water supply, will result in evolution of a single-threaded stream similar to that described here and shown in Figure 7.2. In other words, for those single-threaded streams described as narrowing due to vegetation encroachment, it is believed that those stream channels are further along in the stream succession stages (Fig. 7.2), taking two (e.g., Tukur & Mubi, 2002) to four decades (e.g., Grams et al., 2007) to reach the C and E channel type stages of scenario 1

and/or proposed scenario. Alternatively, channel widening represents an unstable channel type (e.g., G or F). The channel widening stage could last at least six (e.g., [Mohammed-Ali, 2020](#)) to seven decades (e.g., the Lower Smoky Hill River downstream of Kanopolis Dam as described here). Therefore, the time it takes to shift from one stage to the next is dependent upon the hydrophysiographic region, the local site conditions, dam operation, and the distance from the dam.

Studies that evaluated dam operation on bank erosion downstream of the dam obtained similar results as that obtained in Chapter 6, with increased fluvial erosion and widening due to flood control ([Hupp et al., 2009](#)) and water supply dam operations ([Gilliam, 2011](#)) and increased geotechnical failure and widening due to hydropower operations ([Mohammed-Ali, 2020](#)). In other words, these three case studies are examples of unstable channel forms (e.g., G or F) downstream of dams that are still adjusting to flow regulation. Furthermore, [Hupp et al. \(2009\)](#) hypothesized that bank instability attenuates downstream following dam closure, being highest near the dam immediately following closure and then migrating downstream with time. This observation suggests a spatial/longitudinal component of channel evolution downstream of dams, similar to that shown in Figure 1.1.

Finally, a case study downstream of Mangrove Creek Dam, a water supply dam in Australia, provides a geomorphic response similar to that described downstream of Kanopolis Dam. [Sherrard & Erskine \(1991\)](#) state that: “Contraction has occurred by a combination of in-channel bench construction, the formation and bank attachment of longitudinal bars and bank deposition. *Leptospermum polygalifolium* has rapidly colonized these in-channel deposits and will result in the stabilization of benches (pg. 53).” Furthermore, the researchers hypothesized that this once straight, wide, and active sand-bed stream would eventually become small, sinuous, and well-vegetated ([Sherrard & Erskine, 1991](#)), similar to that presented in the proposed scenario (Fig. 7.2b).

Understanding the channel evolution or succession of a stream impacted by man has great value in assisting in a stream’s restoration or repair ([Keane & Sass, 2017](#)). It is expected that these stream succession scenarios could be applied to other downstream reaches of single-threaded streams impacted by flow regulation. However, future research is required to test

the application of these stream succession scenarios in other regions.

7.1.3 Can stream practitioners use 1D HEC-RAS to assess downstream geomorphic change following dam installation and assist in channel erosion reduction strategies?

Using a 1D hydraulic, sediment transport, and/or bank erosion model to assess geomorphic change and possible stream restoration designs is not new and has been applied effectively by many researchers and designers (e.g., [Bischetti et al., 2010](#); [Enlow et al., 2018](#); [Frothingham, 2008](#); [Simon et al., 2009](#); [Wells et al., 2007](#)). However, few studies (e.g., [Kassa, 2019](#); [Mohammed-Ali et al., 2020](#)) have evaluated the use of HEC-RAS 1D since BSTEM was incorporated into the 1D sediment transport model by [Gibson et al. \(2015\)](#). Furthermore, this is the first time to the author’s knowledge that a tool like this has been used for geomorphic and design assessment on a stream in the state of Kansas. Based on the results of Chapter 6, HEC-RAS 1D quasi-unsteady hydraulic, sediment transport, and bank erosion modeling was a useful tool in evaluating geomorphic change along Site 1. However, it is imperative to have site-specific measured data to calibrate and validate each portion of the model. Table 7.4 summarizes the model inputs required to run a sediment transport and bank erosion model, and also highlights sensitive parameters based on [Lammers et al. \(2016\)](#) and [USACE \(2020c\)](#). Figure 6.1 provides a flow chart of these model inputs. Stream stage, which is highly dependent upon channel slope and dimensions, is a sensitive parameter for bank erosion modeling. Furthermore, [USACE \(2020c\)](#) stresses the importance of having a calibrated hydraulics model before performing sediment transport calculations. Therefore, for the hydraulics model, accurate channel geometry and Manning’s n values, as well as flow with stage field measurements are required for proper model set-up and calibration. For the flow rating curve, a minimum of two measured flows, one at low flow and one at high flow would be ideal; however the model set-up shown in Chapter 6 used only one high flow stage and discharge for calibration and obtained reasonable results.

The sediment transport portion of the model has the most model inputs, as shown in

Table 7.4: Required Model Inputs for HEC-RAS 1D Sediment Transport and Bank Erosion Simulation. *Note:* Items with numerical superscripts represent sensitive parameters.

Sediment Transport	Streambank Erosion	
	Fluvial Erosion	Geotechnical Failure
Channel geometry	Channel geometry ²	Bank geometry ²
Channel slope (S)	Channel slope (S) ²	Stratigraphy
Flow hydraulics	Stream stage ²	Effective cohesion (c') ²
Bed material gradation	Erodibility coefficient (k_d)	Effective internal friction angle (ϕ')
Water temperature	Critical shear stress (τ_c)	Saturated unit weight (γ_{sat})
Sediment transport function ¹		Stream stage ²
Sorting method ¹		Groundwater table elevation
Fall velocity method		
Mobile bed depth ¹		
Mobile bed lateral limits ¹		

Sensitive parameter based on ¹Lammers et al. (2016) results or ²USACE (2020c) methodology.

Table 7.4, and, based on experience, seemed to have the greatest level of uncertainty due to both lack and/or variability of available sediment data and the sediment transport equations themselves. Minimum data collection for effective modeling include at least one field-measured cross section (if bank erosion is also being modeled, choose a cross section on an eroding bank) with a reach-scale longitudinal profile of the channel thalweg that have both been surveyed twice, once before a high flow (>1.5-year return interval) and again soon after, a particle size distribution analysis of the bed material, and an estimate of mobile bed depth. An ERT survey to obtain stratigraphy of the bank and bed, similar to that described in Chapter 6 (example shown in Fig. B.1), appears to be a useful tool in estimating mobile bed depth. In terms of the sensitive parameter mobile bed limits, this parameter can be determined and noted during cross sectional surveys.

If channel instability (i.e., aggradation or degradation) is observed during the measurement period, the equilibrium load sediment boundary condition is no longer valid and thus, sediment load data would be required to develop a sediment rating curve or series (load vs. flow), as well as gradation of the varying flows and sediment loads. Sediment load data can be difficult and costly to acquire, so if bed instability needs to be modeled, caution and thoughtful consideration should be made during the data acquisition and calibration phase. Finally, if any historical analysis of cross section change is required, it is also imperative that

channel conditions - including bed stability, channel dimensions, and measured sediment load data (if bed was historically unstable) - are characterized for proper modeling.

As stated in [USACE \(2020c\)](#), selecting proper sediment transport and sorting and armoring functions are highly important for sediment transport modeling. There are currently three sorting and armoring functions in HEC-RAS, and among these three, the one that best represents the stream of interest and its mobile bed configuration can be identified with relative ease. Selecting the sediment transport function is not as straightforward. Without field measured data of channel stability and change, it would be extremely difficult to select the sediment transport function that best represents the site conditions.

In terms of bank erosion modeling, the majority of the sensitive parameters presented in Table 7.4 are related to setting up and calibrating the hydraulics model. However, it is noted that the measured effective cohesion is a sensitive parameter when assessing geotechnical failure and should not be an assumed value. [Lammers et al. \(2016\)](#) recommends obtaining multiple measurements of cohesion and critical shear stress from the streambank erosion site of interest to provide a window of possible values for both parameters. While critical shear stress was not found to be as sensitive as other fluvial erosion parameters, it is imperative that values input into BSTEM are within a reasonable range of site conditions, especially since critical shear stress is often used as a calibration parameter ([Gibson et al., 2015](#); [Langendoen & Simon, 2008](#); [Wells et al., 2007](#)). [Klavon et al. \(2017\)](#) notes that there are many different ways to obtain these values, however, *in situ* measurements using a BST and a mini-JET device, similar to what was used and described in Chapter 6, have been the most popular techniques to obtain these values. Both the BST and mini-JET are rapid tests that have been shown to provide reasonable results of cohesion, friction angle, erodibility, and critical shear stress for the purpose of BSTEM modeling.

Incorporating HEC-RAS 1D hydraulics, sediment transport, and bank erosion modeling into stream restoration design and assessment would certainly increase the initial cost of projects due to required data collection, as well as time spent on model set-up and simulation. However, if the model was developed with sound geomorphic data and engineering reasoning, using this model could decrease long term maintenance and adaptive manage-

ment costs. For example, prior to installation of woody revetments on the Lower Smoky Hill River, it was unknown if the project would work in the long run and thus was designated as a demonstration project with funding provided to conduct short term monitoring (e.g., [Bigham et al., 2020](#)). However, it is believed that if more time and resources were spent early on in the design phase of the project to determine the dominant bank erosion process through modeling and field survey, the final design would have been much different and could have potentially withstood the high flows of 2019. The last part of the question here is: can stream practitioners use HEC-RAS to assist in channel erosion reduction strategies? The short answer is “yes,” assuming that field data collection and modeling set-up and calibration/validation are conducted, as outlined here. A more detailed answer to this question will be discussed in the following section.

7.1.4 Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how?

Managing observed and modeled system wide toe erosion on the Lower Smoky Hill River due to flow regulation (see Ch. 6) would best be addressed by either (a) letting the stream adjust on its own with little anthropogenic manipulation or (b) river restoration measures. These two approaches are sometimes referred to as passive and active restoration, respectively ([Kristensen et al., 2013](#)). [Bigham \(2020\)](#) defines river restoration as “the design and construction of a vertically and laterally stable, floodplain-connected channel that is capable of carrying the bankfull or effective discharge...and its produced sediment load (p. 352).” In the case of the Lower Smoky Hill River, river restoration would be extremely costly and would also require landowner permission from likely hundreds of private landowners, as demonstrated by [Moerke & Lamberti \(2004\)](#) in Indiana. Therefore, system wide river restoration on the Smoky Hill River that is influenced by Kanopolis Dam flow releases is likely not feasible. However, eroding streambanks could be stabilized on a site-by-site scale through streambank stabilization measures.

Bigham (2020) defines streambank stabilization as a “single technique or system of techniques that maximize localized streambank shear strength and/or minimize the forces acting on a streambank with the intent of halting or minimizing lateral retreat (p. 352)”. Streambank stabilization can be implemented with other river restoration techniques (e.g., bed stabilization, channel design and realignment, etc.), however, it is most often installed on a site-by-site basis (Bigham, 2020). This approach has been criticized by some as a “one size fits all” (Rosgen, 1996) or “Band-Aid” (Bernard & Tuttle, 1998) approach to restoration, as it does not address the underlying cause of instability (Florsheim et al., 2008; Rosgen, 1996; Wohl et al., 2005), which in this case is upstream dam installation. However, due to the scale and the cost of river restoration as described above, streambank stabilization is the only active restoration component that could be reasonably applied on the Lower Smoky Hill River.

When reviewing streambank stabilization literature and design manuals, it is highly recommended to address any active or predicted bed incision or degradation before conducting any streambank stabilization measures (Enlow et al., 2018; NRCS, 1996; Shields Jr. et al., 2004). Fortunately, for the Lower Smoky Hill River, the streambed appears to have either remained stable, or is stabilizing and/or aggrading (as discussed in Section 7.1.2), thus streambed stabilization measures are not necessary for the current geomorphic state of this stream. Bigham (2020) provides a thorough review of the literature on streambank stabilization measures. Two designs were evaluated based on (1) the methods provided in Bigham (2020), (2) the woody revetments previously installed on the Smoky Hill River (Bigham et al., 2020), and (3) the author’s experience for common low cost (design constraint) streambank stabilization systems:

- Woody bankfull bench design with 3H:1V bank shaping. The woody bench would consist of root wads, fill from bank shaping cut material, and recycled, broken concrete.
- 3H:1V bank shaping (cut) only.

These two designs were tested using the calibrated 1D, flow, sediment transport, and bank erosion model described in Chapter 6. Design details (not to scale), as well as bank erosion

model assumptions, are provided in Figure 7.3. Bank erosion parameters were adjusted for the woody bench design. Critical shear stress for the woody bench came from the “Vegetation and Protection” tab in the Excel version of BSTEM (Version 5.5). While large woody debris has a permissible shear stress of 192 Pa, 100 Pa was used since a combination of wood and other materials would be used to create the bench. Erodibility was assumed to be very low for the woody bench. Chosen cohesion and friction angle values for the woody bench are average values from the mixture of the two soil types used to fill the voids of the woody bench.

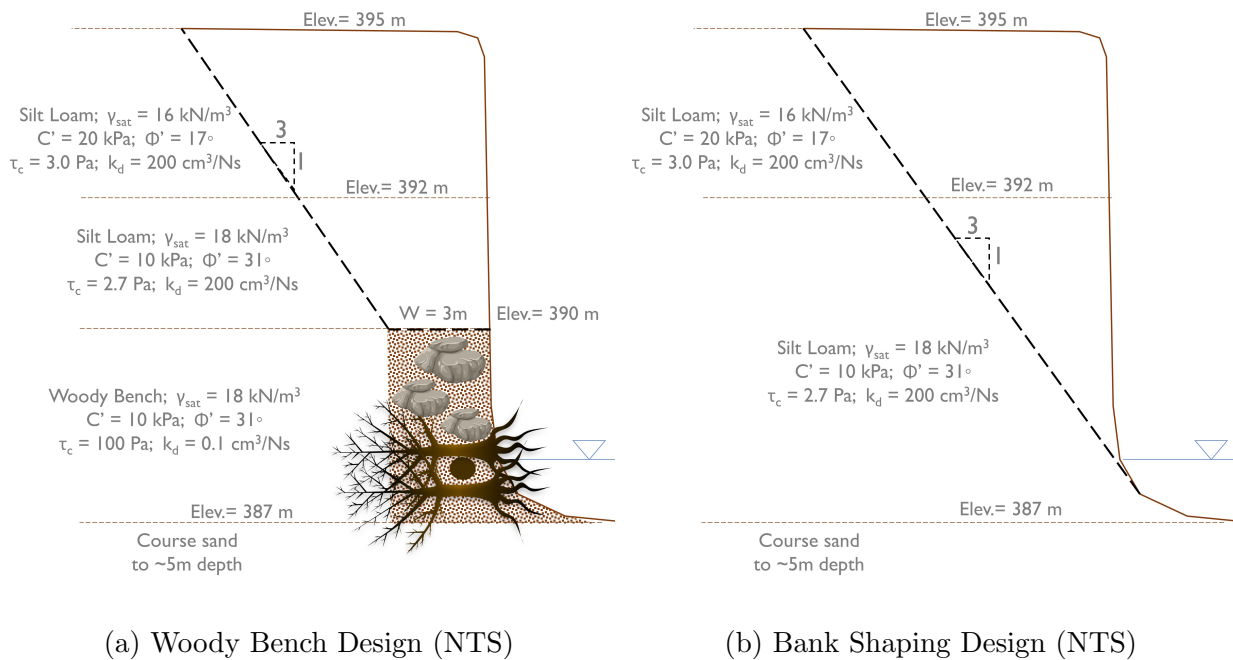


Figure 7.3: Design Details of Two Possible Streambank Stabilization Systems for the Smoky Hill River along Site 1. *Note:* The woody bench design was determined to be the most resistant and resilient to flows. NTS: Not to scale

The HEC-RAS 1D hydraulics, bank erosion, and sediment transport model using the actual (with dam) flow conditions was used to evaluate these designs. Both the channel geometry for the entire meander bend containing the Site 1 erosion cross section of interest and the BSTEM data were manipulated based on the details provided in Figure 7.3. The results from the design simulations from HEC-RAS are presented in Table 7.5. These results show that the woody bench design is the most effective. Possible reasons include a (1)

higher critical shear stress in the zone of the bankfull stage, (2) lowered stream stage due to the increase in cross sectional area (drop of 0.2 m during June 24, 2019 flow), and (3) channel cross section that mimics the stream succession scenario shown in Figure 7.2b (C channel). The bank shaping only design was effective until the 2019 high flows, with the biggest erosion event occurring following the receding limb of the highest flow on June 24, 2019, which resulted in geotechnical failure of the bank. It is interesting to note that total volume of erosion with bank shaping was almost twice of that observed without (Table 6.1, also shown in Table 7.5). While geotechnical failure can introduce a lot of sediment at once (Simon et al., 2000), it is a necessary and natural process that will eventually reduce bank angles to achieve stability in the long term (Hupp et al., 2009). Imposing a stable bank angle, as was done for the bank shaping design, may not be stable for all flow and groundwater levels and could result in greater cumulative erosion rates in the long term, as shown here.

Table 7.5: Bank Erosion Simulation Results from the Two Design Options to Stabilize Site 1 Erosion Streambank of Interest.

Bank Erosion Process	Bank Erosion Volumes (m^3)		
	Woody Bench	Bank Shaping	Existing Condition ¹
Toe Erosion	79	9	930
Geotechnical Failure	0	1,974	160
Total	79	1,938	1,090

¹Results from Table 6.1.

Finally, the woody bench design was tested under the expected natural (no dam) scenario. The design was effective until the receding limb of the highest flow event (446 cms). In other words, the woody bench design would be able to withstand flows up to the 40-year return interval annual peak flow (see Fig. A.1). It would be up to the designer to determine if modifications would be necessary to guarantee success for flow events of a 40-year return interval or greater.

These observations prove the value of (1) evaluating streambank stabilization designs using a calibrated 1D hydraulic, sediment transport, and bank erosion model before implementation and (2) using channel evolution models or stream succession scenarios to help inform design features. In terms of the Lower Smoky Hill River, a woody bankfull bench, as

shown in Figure 7.3a, is one approach to stabilizing Smoky Hill River streambanks that are susceptible to both toe erosion and geotechnical failure. However, this design may not be appropriate for all streams downstream of large dams. The following should be considered when developing a design below flood control dams:

- How has the flow duration curve changed following dam installation (see Chapter 3)?
- Is there a bankfull condition and if so, where is the stage located (see Chapter 3)?
- If bankfull condition exists, what stage of channel evolution is the stream currently in (see Section 7.1.2 for single-threaded channel evolution scenarios)?
- Is the streambed stable (see Chapter 4)?
- What is the dominant bank erosion process (see Chapters 5 and 6)?
- What is the elevation of the vegetation line (see Chapter 6)?
- Is passive over active restoration acceptable?
- Is bank stabilization over river restoration acceptable?
- What maximum design storm is considered to be acceptable for the site of interest?
- What are other design constraints (e.g., cost, maximum loss of land, etc.)?

These are just a few questions that will help guide stream practitioners in selecting appropriate channel instability management strategies.

Chapter 8

Conclusion

For nearly a century, man-made dams have played a major role in protecting and supporting human life by storing runoff for water supply, hydropower, and flood control. However, dam installation does come with an environmental cost. There is scientific consensus that large dams induce geomorphic channel instability both upstream and downstream of the structure. Dams induce aggradation upstream and within the produced reservoir by reducing sediment transport capacity and trapping fluvial sediment. The stream also responds downstream, as the installation and operation of dams can drastically change the flow and sediment regime. Unlike upstream geomorphic response, geomorphic changes downstream are variable and are dependent upon dam operation, time since installation, distance from dam, hydrophysiographic region, local variability of site conditions, and many other factors. This variability makes it difficult to not only predict physical geomorphic change but also manage its secondary impacts to other important abiotic and biotic factors of aquatic ecosystems such as water quality, aquatic and riparian habitats, and species diversity and richness. This research answers the question: Can stream practitioners manage the unintended impact to channel instability downstream of large dams and if so, how?

A case study was conducted at a reach-scale downstream of Kanopolis Dam, a flood control dam located on the Smoky Hill River in central Kansas, USA. To answer this overarching question, the following research objectives were completed:

- Evaluate the effects Kanopolis Dam closure has had on flow and channel bed stability up to 111 river km downstream.
- Assess the impact of Kanopolis Dam closure on downstream channel planform.
- Evaluate the effects flow regulation has had on streambank erosion processes and rates.

In terms of the flow regime, measured flow data from Smoky Hill USGS gages were summarized and statistically analyzed. Based on this analysis it was found that:

- Peak flows have been dampened by 72% with high flows (>100 cms, 1.3-year return interval at the Ellsworth USGS Gage) occurring less often.
- Flow duration curves have not changed substantially from pre to post-dam flow conditions, however, moderate flows (5-40% daily exceedance probability) tended to be higher and high flows tended to be lower.
- Hydrographs have become more step-like versus flashy in shape with general increase in duration of high flows at locations closest to the dam and slower rising and falling limbs at all locations.
- Timing of the annual peak flow event has not changed and continues to occur in the summer months.

Streambed stability was assessed using Smoky Hill River USGS gage rating curves to plot and assess changes in a reference, low flow stage overtime. The results from this analysis are summarized as follows:

- Overall, streambed aggradation has occurred up to 47.9 km upstream of Kanopolis Dam at the Ellsworth USGS gage since installation, while degradation has occurred directly downstream (0.8 km) at the Langley USGS gage.
- Degradation and sediment removal at the Langley USGS gage appears to have aggradated the channel near the Lindsborg USGS gage (63.9 km) since dam construction began in 1940, with aggradation rates decreasing soon after dam closure.

- The Smoky Hill River bed appears to have been stabilizing within the last decade at all USGS gage locations near Kanopolis Dam.

Channel planform was assessed using two metrics: (1) channel migration rates and (2) sinuosity. Based on GIS analysis of historical aerial imagery and mapping, channel migration rates, or the area of floodplain reworked per river length per year, have decreased by 69% while channel sinuosity has increased by 5% since Kanopolis Dam closure along the Lower Smoky Hill River. These observed planform changes are likely in response to changes in both the flow and sediment regime following Kanopolis Dam installation that resulted in the need for a flatter channel slope, which can occur via degradation and/or increasing overall stream length through lateral migration caused by streambank erosion and deposition.

Finally, using a HEC-RAS 1D quasi-unsteady hydraulic, sediment transport, and bank erosion model, it has been shown that toe erosion is the dominant streambank erosion process for a site 86 river km downstream of Kanopolis Dam. Furthermore, if the dam was removed, bank erosion would be higher under the expected natural (no dam) conditions versus the actual (with dam) conditions, likely due to the channel evolution process in which disturbance to the flow and/or sediment regime can cause an increase in bank erosion rates. Coupling model results with measured cross sections and observations, it was also demonstrated that toe erosion is occurring in other locations of the stream where bank erosion is typically less common, such as the inside of meander bends and along straight reaches. This is likely due to the step-like flow hydrograph brought on by flow regulation. Moderate flows, between 5 to 40% daily exceedance probability, have increased and occur for longer periods of time, resulting in more opportunity for geomorphic work. Furthermore, extended high flow events (>1.3-year return interval) have been observed to drown out pioneer riparian species causing exposure of more raw streambank and amplifying bank erosion rates. In some cases, toe erosion may be occurring even during low flow (>40% daily exceedance probability) due to the erodibility of the toe material, similar to that shown for sites located further downstream. System wide toe erosion occurring over longer periods of time in the reach assessed in this study (up to 111 river km downstream) may also explain the observed increase in stream

length and sinuosity.

Based on the results of these three study objectives, several questions related to stream-bank erosion management in the reach of the Smoky Hill River affected by flow regulation were answered. The results from the bank erosion assessment provide insight on the dominant mechanism, as well as the spatiotemporal scale of bank erosion occurring on the Lower Smoky Hill River. In unregulated streams, the bankfull discharge typically controls the stream's dimension, pattern, and profile, and is often an important consideration when designing stream restoration or bank stabilization projects. However, based on review of the literature it was unclear if bankfull conditions even exist downstream of dams. This research shows that a bankfull condition does exist, at least on the Lower Smoky Hill River. It was also found that the bankfull discharge occurs at the same annual frequency (1.3-year return interval); however, the flow rate has been reduced by 49% since Kanopolis Dam closure. Furthermore, the daily frequency of the new bankfull discharge has also increased following Kanopolis Dam installation, meaning that this discharge is occurring more often on a daily basis than it did historically.

Once bankfull condition was confirmed, two stream succession scenarios were observed: (1) [Rosgen \(2014\)](#) stream succession scenario 1 within the degradational zone downstream of Kanopolis Dam (up to 11 river km downstream) and (2) a proposed stream succession scenario downstream of the degradational zone (up to 111 river km downstream). These stream succession scenarios can be applied to pre-dam single-threaded channels to assist in channel erosion management strategies and techniques following dam closure.

Since toe erosion appears to be occurring system wide within the assessed reach of the Lower Smoky Hill River, providing evidence of continued channel instability, full-scale river restoration is likely not feasible due to cost and the need for private landowner support. Instead, streambank stabilization may be a more feasible option. Given the success of developing and calibrating a HEC-RAS 1D hydraulic, sediment transport, and bank erosion model to assess dominant bank erosion processes and rates on the Smoky Hill River, this same model was used to assess two low cost streambank stabilization designs. Modeling results suggest that a woody bankfull bench, in combination with 3H:1V bank shaping,

would provide stability for at least a 40-year return interval flow event on the Lower Smoky Hill River. Assessing river restoration designs prior to installation with calibrated numerical modeling tools could reduce long term maintenance and adaptive management costs.

Dams provide humanity with many resources: flood control, energy, water supply, recreation, and much more. Unfortunately, there are always environmental costs to exploiting natural resources. Excessive toe erosion brought on by flow regulation can eat away at arable land while polluting water with excess fine sediment. Poor water quality affects downstream water supply, as the Smoky Hill River feeds into the Kansas River, a major source of water for many Kansas communities downstream. Poor water quality and channel instability can also impact aquatic and riparian habitats. Furthermore, longer high flows caused by dam operation can drown out native pioneer riparian species, impacting habitat and amplifying bank erosion rates. Human-induced climate change and upstream reservoir sedimentation further complicate these issues. Therefore, future research should continue to assess and value damages induced by dam operation under a changing climate, especially lost and/or damaged ecosystem services, to inform and adapt river management and dam operations through time.

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Appendix A

Log Pearson Type III Flood Frequency Analysis

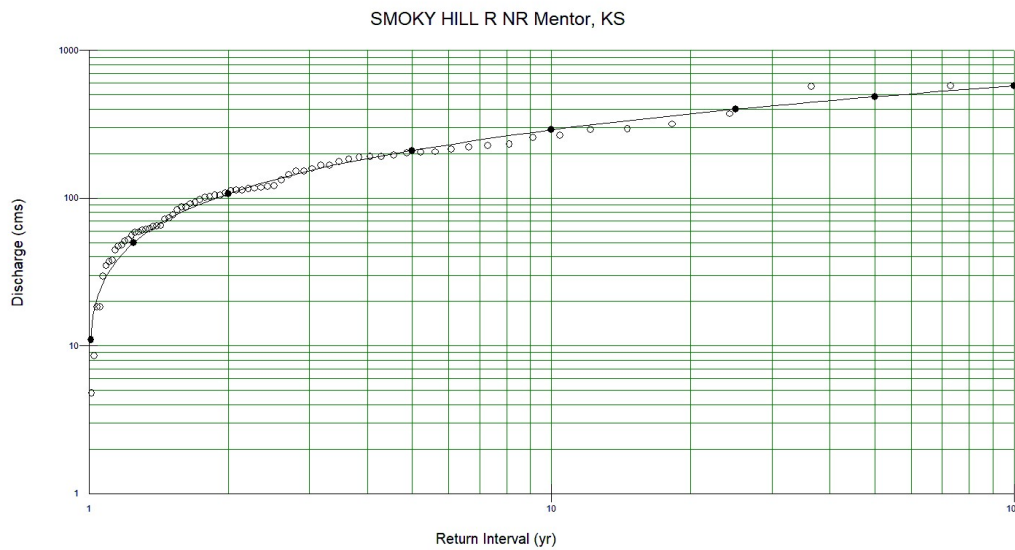


Figure A.1: Log Pearson Type III Flood Frequency Plot for USGS Gage 06866500 near Mentor, KS (1949 to 2021).

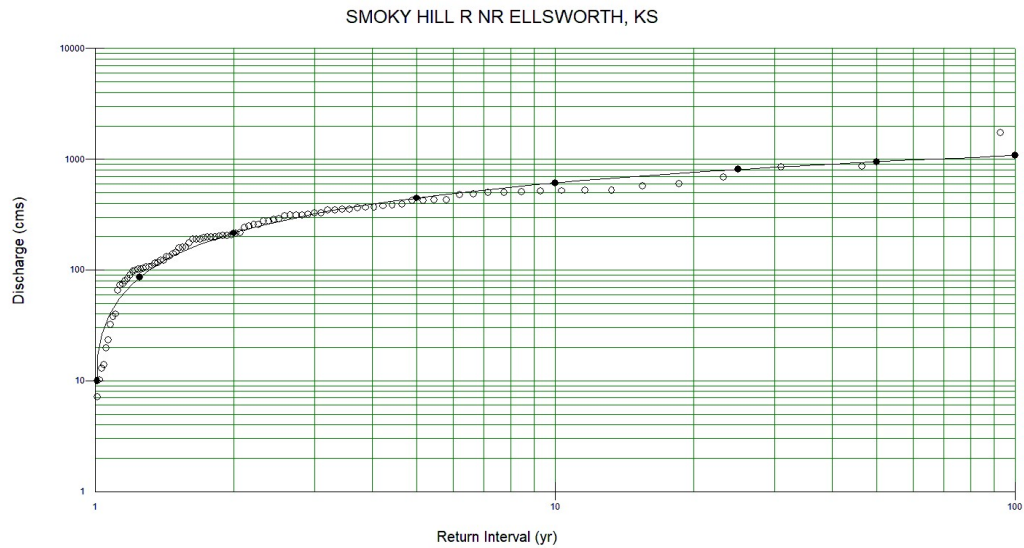


Figure A.2: Log Pearson Type III Flood Frequency Plot for USGS Gage 0686450 near Ellsworth, KS (1929 to 2020).

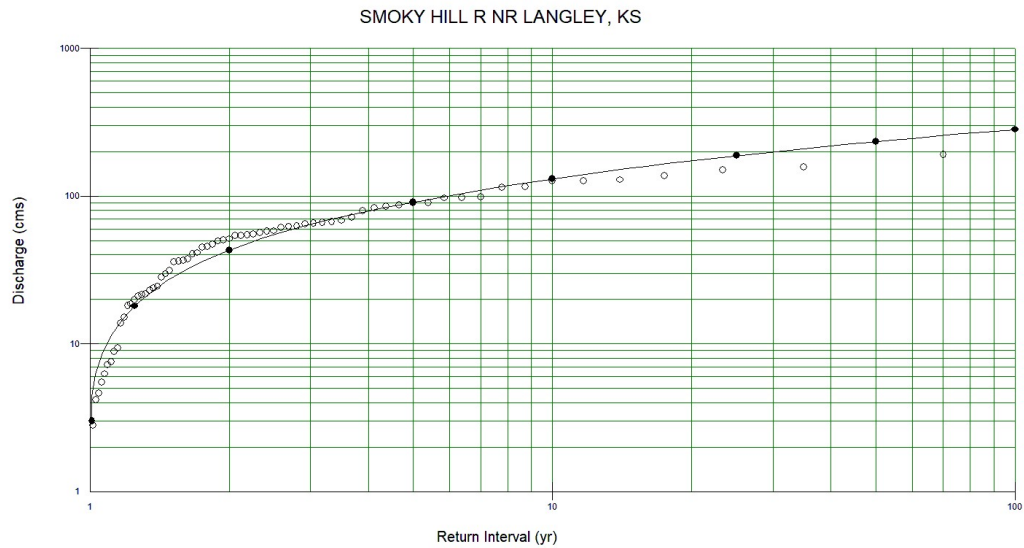


Figure A.3: Log Pearson Type III Flood Frequency Plot for USGS Gage 06865500 near Langley, KS (1949 to 2020).

Appendix B

Supplementary Data Required for HEC-RAS 1D Hydraulic, Sediment Transport, and Bank Erosion Modeling

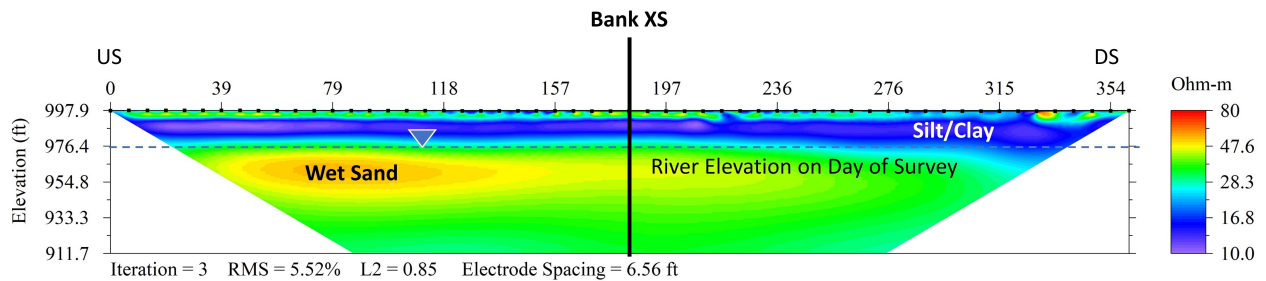


Figure B.1: ERT Results from Site 1 to Estimate Mobile Bed Depth. *Note:* DS: Downstream, US: Upstream, Distance and elevation units are in feet. Resistivity units are in ohms-m.

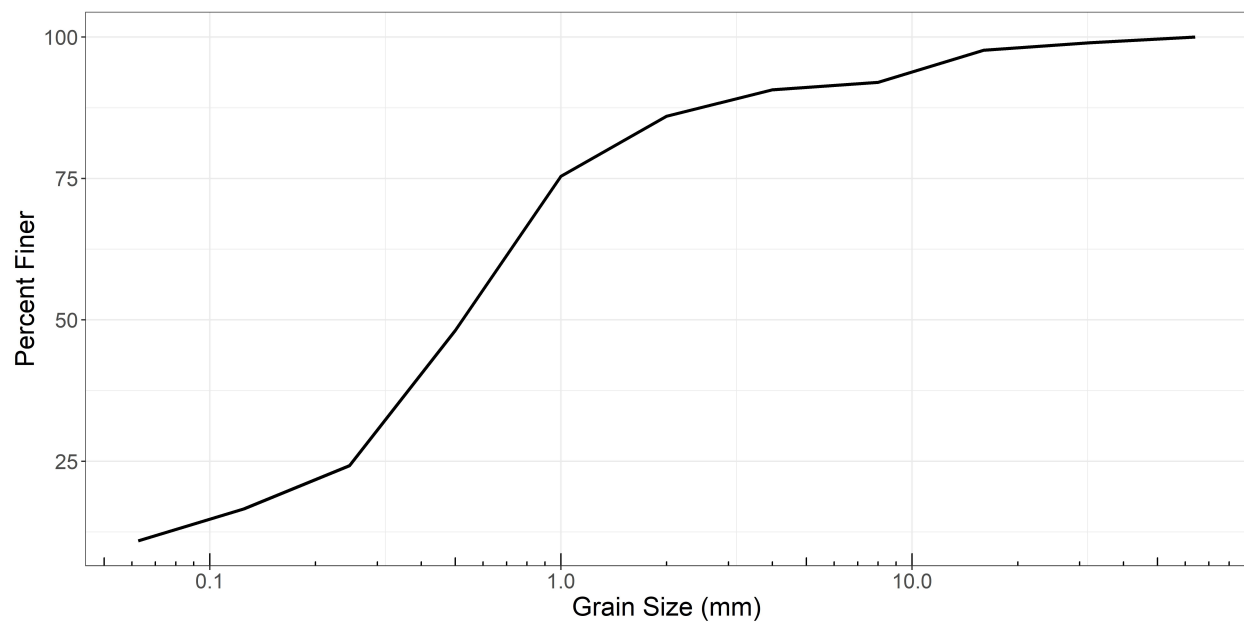


Figure B.2: Linear Graph of Particle Size Distribution of Site 1's Streambed Based on Annual [Wolman \(1954\)](#) Pebble Count Surveys, 2016-2018.

Table B.1: Computation Increments (CI) for Various Expected Flow Intervals used in HEC-RAS 1D Quasi-Unsteady Hydraulic Model

CI Interval	Low cms	High cms	CI (hr)
1	0	5	24
2	5	15	18
3	15	30	12
4	30	60	6
5	60	120	3
6	120	240	2
7	248	480	1

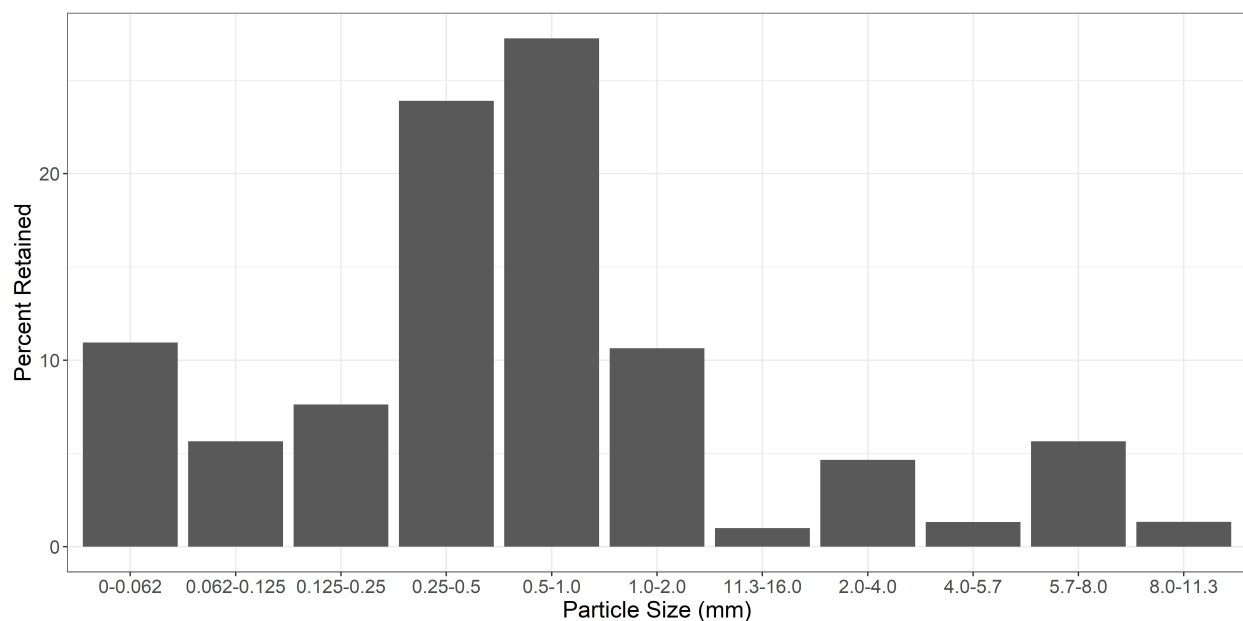


Figure B.3: Bar Graph of Particle Size Distribution of Site 1's Streambed Based on Annual [Wolman \(1954\)](#) Pebble Count Surveys, 2016-2018. *Note:* Particle size ranges are based on the [Wentworth \(1922\)](#) scale.

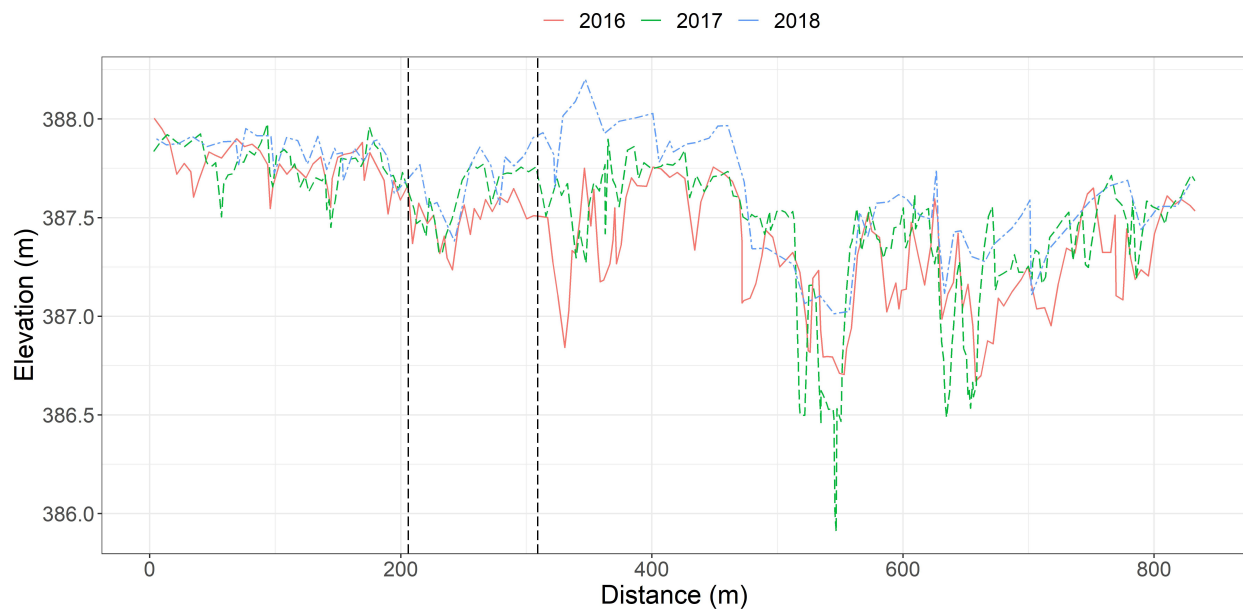


Figure B.4: Longitudinal Profile Survey Results of the Thalweg at Site 1, 2016-2018. *Note:* 2016 is a solid red line, 2017 is a dashed green line, and 2018 is a two-dash blue line.

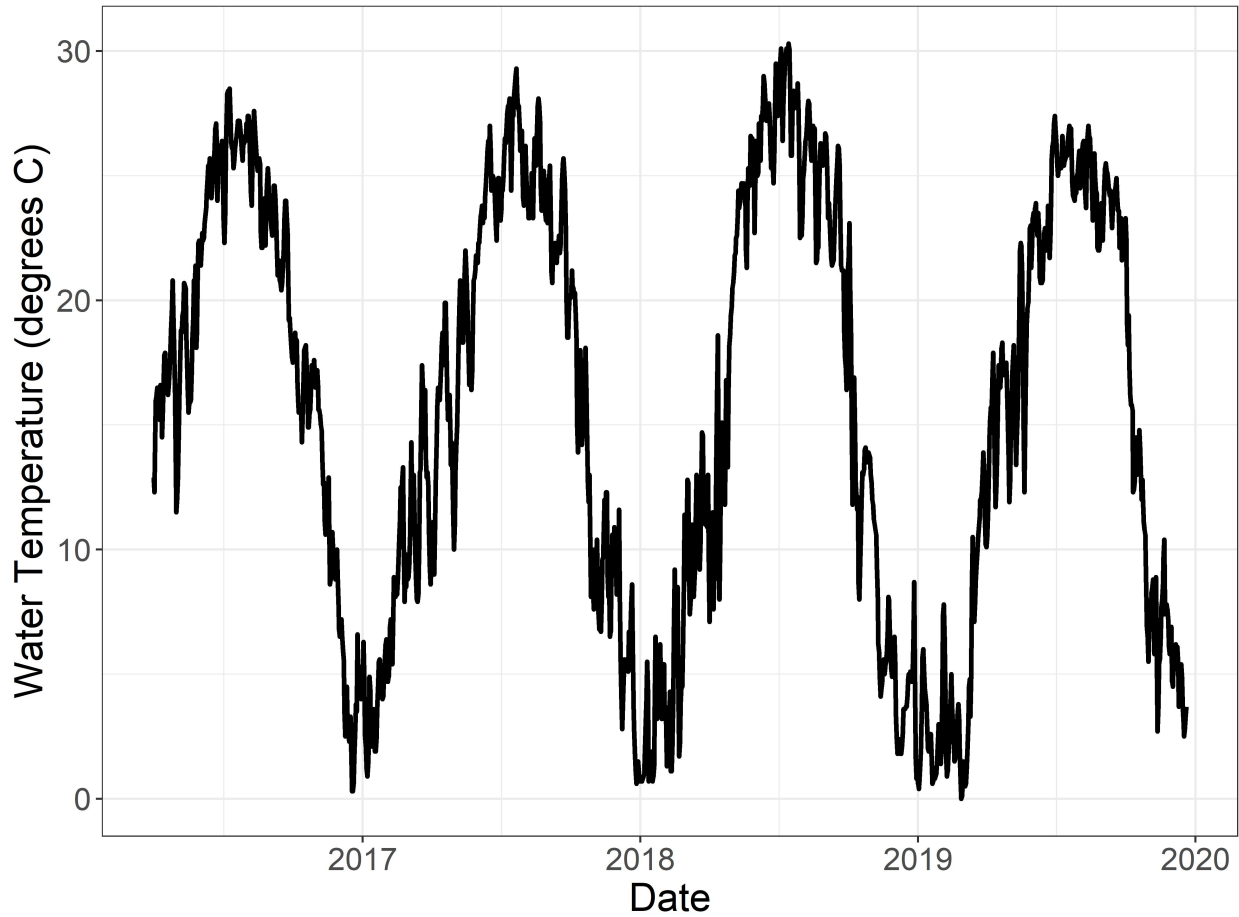


Figure B.5: Water Temperature Data from the Little Arkansas River USGS gage near Halstead, Kansas (USGS 0714367) used in 1D Sediment Transport Modeling of the Lower Smoky Hill River.

Table B.2: Measured Groundwater Depths Near Smoky Hill River Site 2 at USGS Water Well 383955097351901

Date	Measured Depth to Water Table (m)
3/24/2015	8.12
10/8/2015	8.17
3/11/2016	8.16
10/12/2016	7.73
3/14/2017	7.95
10/11/2017	8.13
3/13/2018	8.15
10/19/2018	7.84
10/9/2019	5.52
3/11/2020	6.75

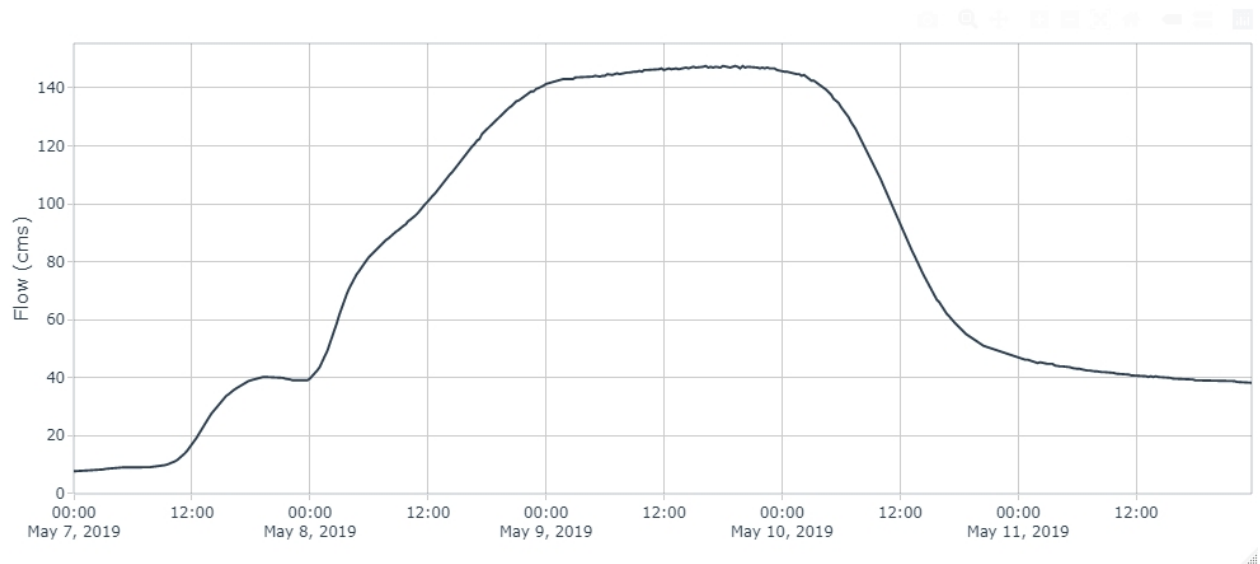


Figure B.6: Synthetic Hydrograph Used for Flow Calibration, Measured Flow at the Mentor USGS Gage from May 7-11, 2019.

Table B.3: Estimated Suspended Sediment Gradation for the Smoky Hill River using Data from the Kansas River near Wamego, Kansas ([Albert, 1973](#)).

Grain Size (mm)	Kansas River (%) Albert (1973)
0.002-0.004	51
0.004-0.008	10
0.008-0.016	10
0.016-0.032	8
0.032-0.0625	11
0.0625-0.125	3
0.125-0.25	3
0.25-0.5	4

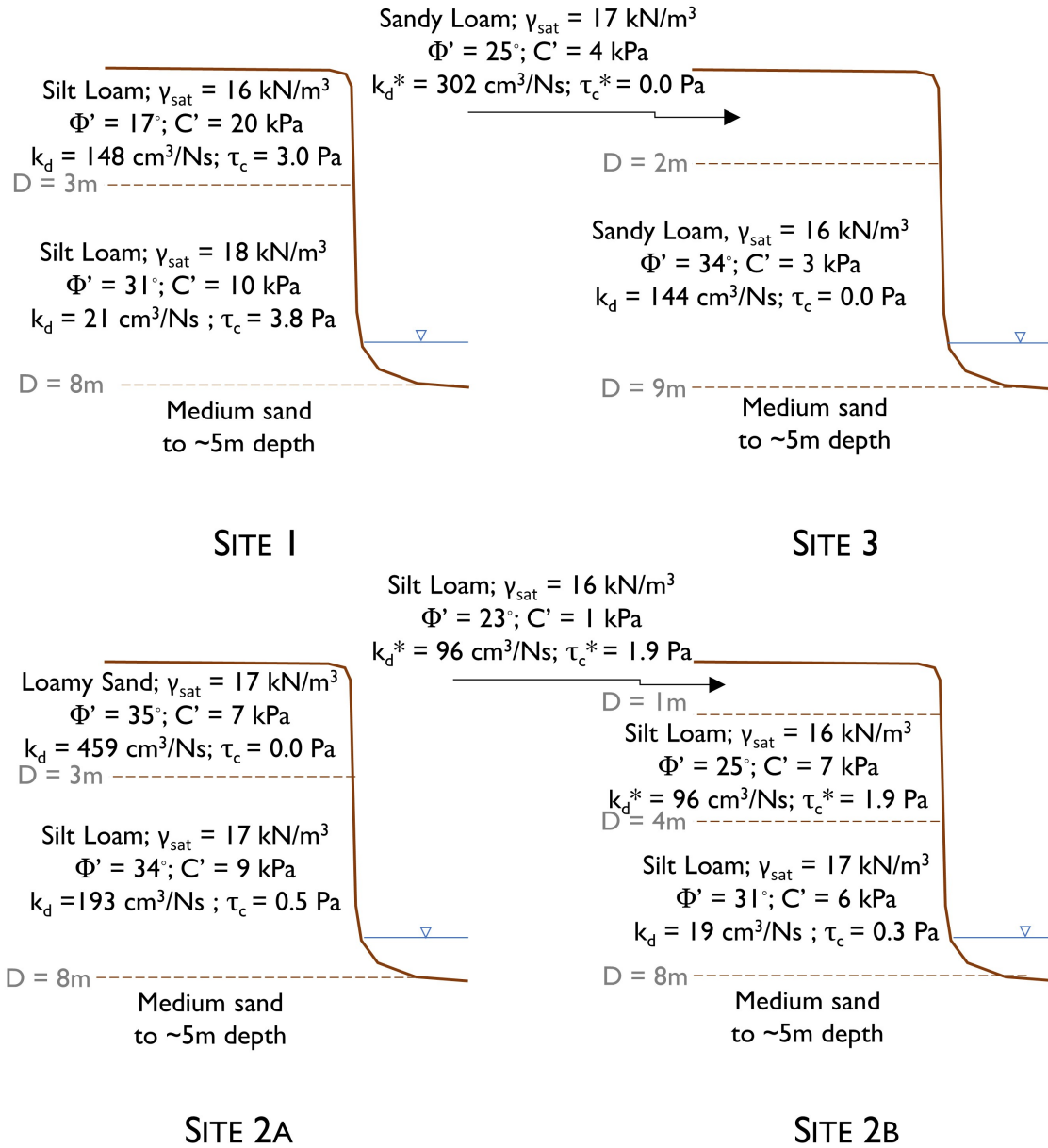


Figure B.7: Streambank Stratigraphy and Streambed Sediment Profiles for Site 1 through 3 on the Lower Smoky Hill River. *Note:* Soil texture: USDA classification; γ_{sat} : Saturated unit weight; ϕ' : Effective angle of friction; c' : Effective cohesion; k_d : Erodibility coefficient; τ_c : Critical shear stress; D: Depth

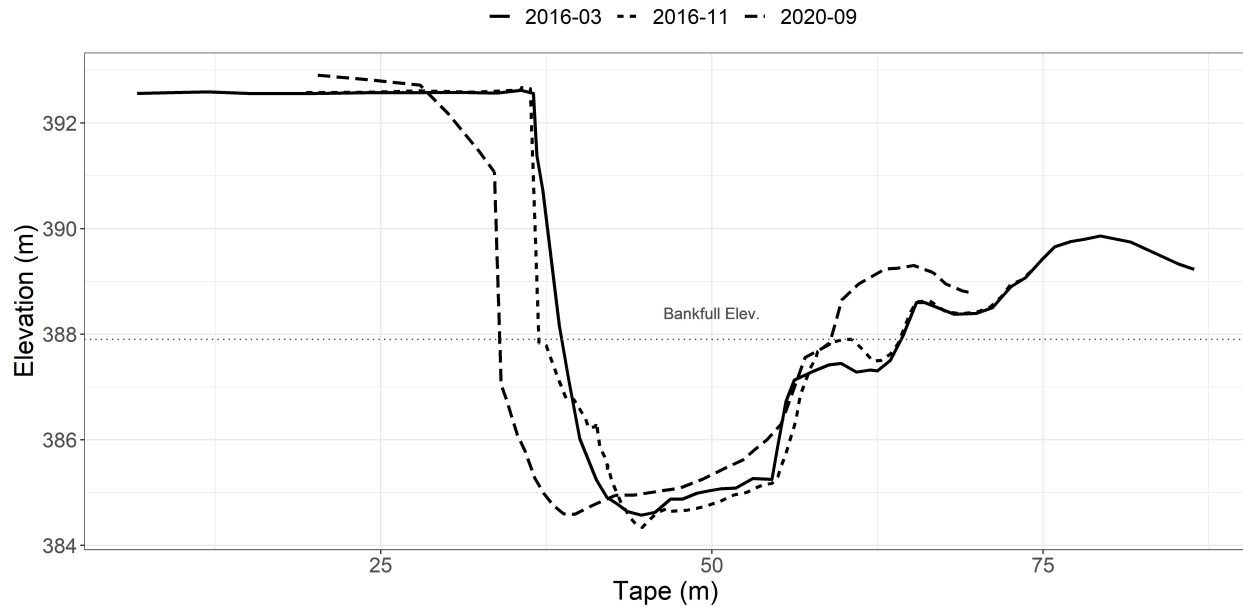


Figure B.8: Site 2B Streambank Erosion Monitoring Cross Section Results, 2016-2020. *Note:* The left bank was stabilized with woody revetments in December of 2016. Deposition on the top of the left bank is from construction of berm after bank shaping. Bankfull cross sectional area ranged from 51 to 55 m^2 in 2016 and 2020, respectively.

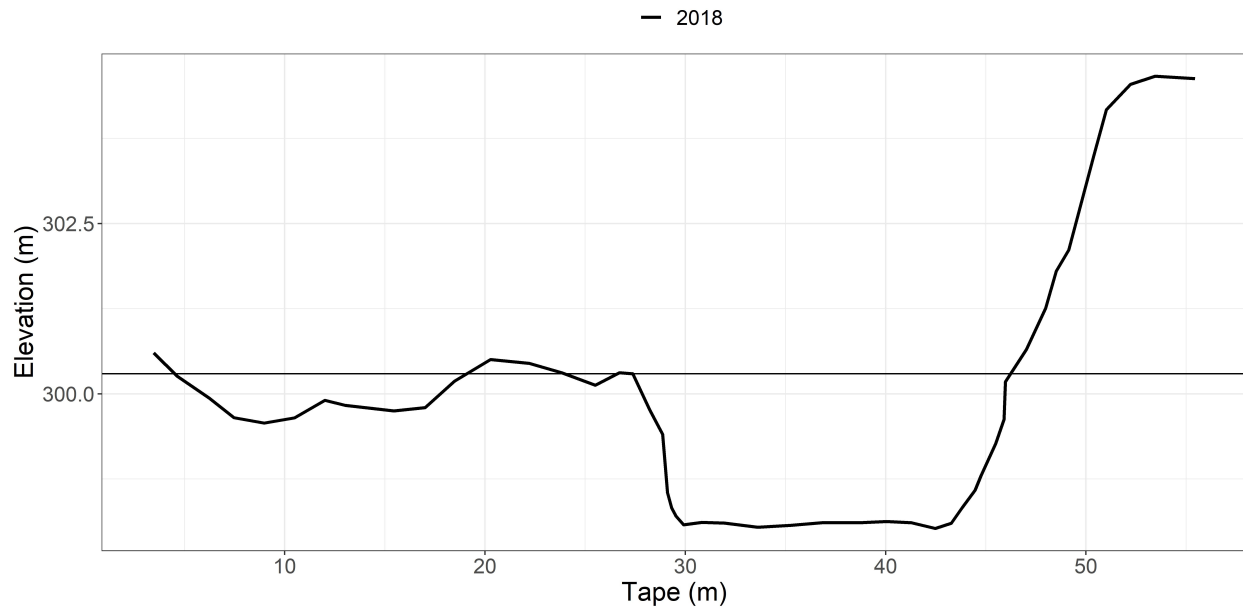


Figure B.9: Streambank Erosion Monitoring Cross Section, 29 river km downstream of Kanopolis Dam, 2018. *Note:* Solid black line represent bankfull elevation.

Appendix C

SAS Code Used in Chapters 3 and 4

C.1 Example Mann Whitney U Test SAS Code

Note: Example shown below evaluates pre to post-dam median daily flow at the Ellsworth USGS gage

*Begin SAS Code:

*Produce descriptive statistics;

```
proc means data=EllDay n nmiss mean std stderr median min max qrange maxdec=4; class
Period; var cms; run;
```

*Test for normality and produce confidence intervals on the median;

```
proc univariate data=EllDay normal cipctldf; class Period; var cms; histogram cms /normal;
qqplot /normal (mu=est sigma=est); run;
```

*if p-values are <0.05 then my data is not normally distributed

*Produce boxplots;

```
proc sgplot data=EllDay; title 'Boxplot of Daily Flow Data at Langley USGS Gage'; vbox
cms /category=Period; run;
```

*Perform the Mann-Whitney U Test;

```
proc npar1way data=EllDay wilcoxon; class Period; var cms; run;
```

*if p-values are <0.05 then significant difference between medians of flow

C.2 Example ARIMA Analysis SAS Code

Note: Example shown below evaluates streambed elevation changes for Ellsworth USGS gage

*Begin SAS Code:

*Equally-spaced intervals;

```
proc expand data=ellbed out=ellbedLI; convert Stage=StageNew / method=join; id Year;
run;
```

*Plot Data;

```
proc sgplot data=ellbedLI; scatter y=StageNew x=Year; run;
```

*Identify: The analysis of the IDENTIFY statement output usually suggests one or more ARIMA models that could be fit. Options enable you to test for stationarity and tentative ARMA order identification;

```
proc arima data=ellbedLI ; identify var=StageNew nlag=18; run;
```

```
proc arima data=ellbedLI ; identify var=StageNew(1); estimate p=1; forecast lead=10 in-
terval=year id=year out=results; run;
```

```
proc sgplot data=ellbedLI; scatter x=Year y=StageNew; run;
```

C.3 Example MCMC Change Point Analysis SAS Code

Note: Example shown below evaluates streambed elevation changes for Ellsworth USGS gage

*Begin SAS Code:

*Calculate summary statistics;

```
proc summary data=ellbedLI missing nway; var Year Stage; output out=ellbedstat(drop=_TYPE_
_FREQ_) mean(Year) = meanyr std(Year) = stdyr mean(Stage) = meanst std(Stage) = stdst;
run;
```

* standardize year and stage using summary statistics;

```
data ellbed_z;
```

```

merge ellbedLI(in=a) ellbedstat(in=b); if a; x = (Year-meanyr)/stdyr; y = (Stage - meanst)/stdst;
run;

* changepoint model using Markov Chain Monte Carlo *;
proc mcmc data=ellbed_z outpost=postout seed=24860 ntu=1000 nmc=20000
    * use 20000 iterations after burn-in of 1000
statistics=summary; ods select PostSummaries; ods output PostSummaries=test1;
array beta[2]; parms alpha cp beta1 beta2; parms s2;
prior cp unif(-1.3, 1.1); prior s2 uniform(0, 5); prior alpha beta: normal(0, v = 1e3);
j = 1 + (x >= cp); mu = alpha + beta[j] * (x - cp); model y normal(mu, var=s2); run;
*Visualize data;
data _null_; set test1; call symputx(parameter, mean); run;
data b; missing A; input x1 @@; if x1 eq .A then x1 = &cp; if _n_ <= 2 then y1 = &alpha
+ &beta1 * (x1 - &cp); else y1 = &alpha + &beta2 * (x1 - &cp); datalines; -1.5 A 1.2 run;
proc kde data=postout; univar cp / out=m1 (drop=count); run;
data m1; set m1; density = (density / 3) - 4; run;
data all; set ellbed_z b m1; *rename x=Year_Standardized y=Stage_Standardized; run;
title "Change-Point Model of Ellsworth Bed"; proc sgplot data=all noautolegend; scatter
x=x y=y; series x=x1 y=y1 / lineattrs = graphdata2; series x=value y=density / lineattrs
= graphdata1; run;

```

Appendix D

Additional Channel Evolution Models (CEMs)

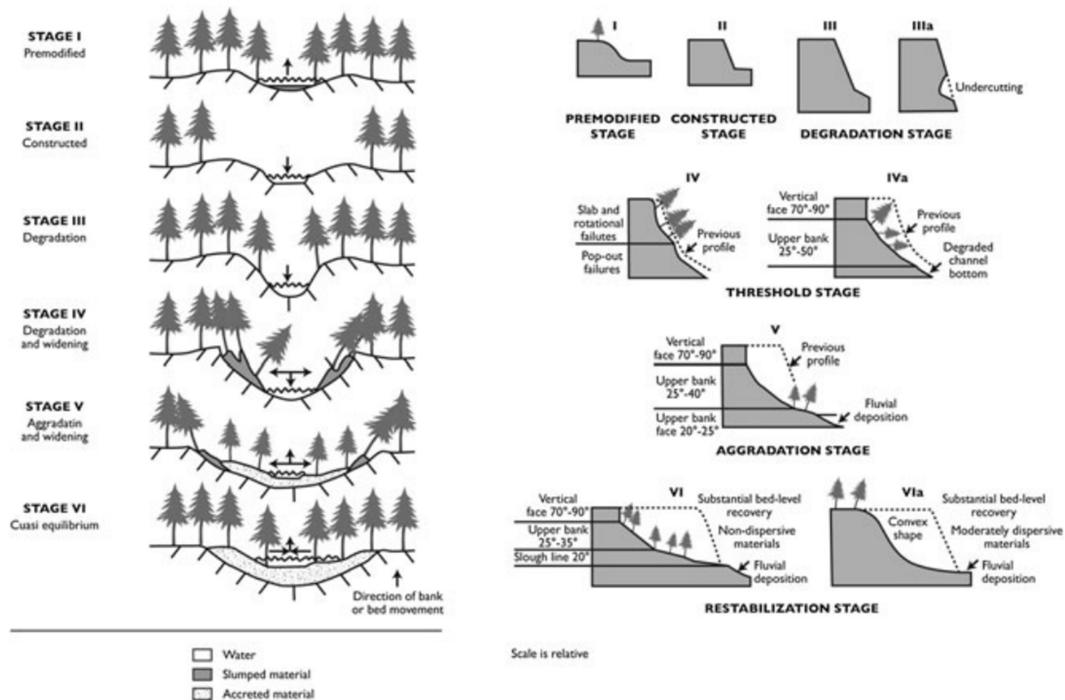


Figure D.1: [Simon & Hupp \(1986\)](#) Channel Evolution Model

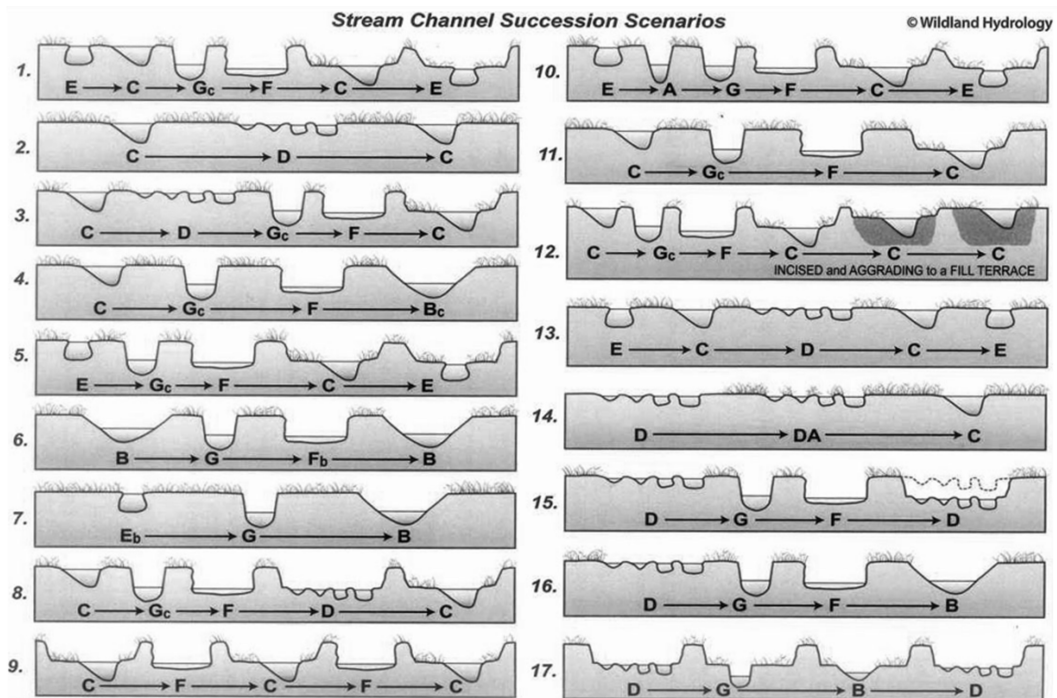


Figure D.2: [Rosgen \(2014\)](#) Stream Succession Scenarios

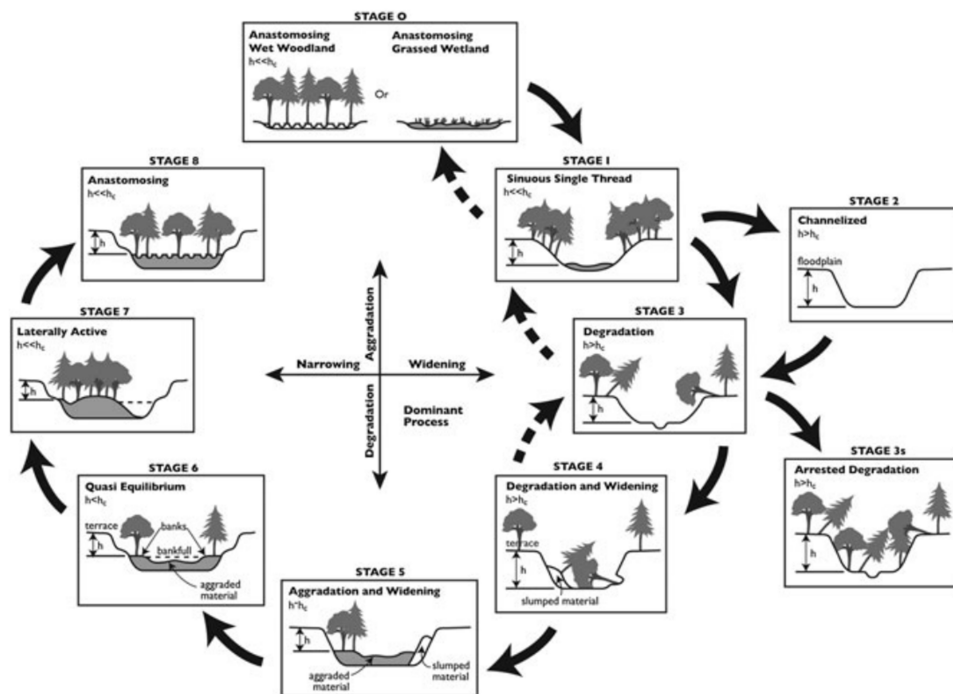


Figure D.3: [Cluer & Thorne \(2014\)](#) Stream Evolution Model