WATERSHED PROJECT FINAL REPORT
SECTION 319 NONPOINT SOURCE
POLLUTION CONTROL PROGRAM

Topical Report RSI-2091

prepared for

Belle Fourche River Watershed Partnership
1837 5th Avenue South
Belle Fourche, South Dakota 57717

December 2009

RESPEC
CONSULTING & SERVICES

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by

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This project was conducted in cooperation with the South Dakota Department of Environment and Natural Resources and the United States Environmental Protection Agency, Region VIII.

Grant # 999818505

EXECUTIVE SUMMARY

Project Title:	Belle Fourche River Watershed Management and Project Implementation Plan Segment III
Section Grant Number(S):	999818505
Project Start Date:	January 2006
Project Completion Date:	December 2009
Funding	
Total EPA Grant Budget:	$1,728,800
Total Matching Funds Budget:	$3,318,326
Total Nonmatching Funds Budget:	$2,265,784
Total Budget:	**$7,312,910**
Budget Revisions:	Total 319 Funds did not change
Total Expenditures of EPA Funds:	$1,728,800
Total 319 Matching Funds Accrued:	$2,297,151
Total Nonmatching Funds Accrued:	$2,287,151
Total Expenditures:	**$6,313,102**

Belle Fourche River Watershed Management and Project Implementation Plan Segment III was sponsored by the Belle Fourche River Watershed Partnership (BFRWP) with support from agricultural organizations, federal and state agencies, local governments, South Dakota State University (SDSU), and South Dakota School of Mines & Technology (SDSM&T). This project continued implementation of the best management practices (BMPs) identified in the Total Maximum Daily Load (TMDL) report for the Belle Fourche River. The objectives of this project segment were:

- Continue implementation of BMPs in the watershed to reduce total suspended solids (TSS) (46.6 milligrams per liter (mg/L) reduction below the Belle Fourche Reservoir (43 percent of goal); 41.6 mg/L reduction above the Belle Fourche Reservoir (22 percent of goal)).
- Conduct public education and outreach to stakeholders within the Belle Fourche River Watershed.

- Track progress made toward reaching the goals of the TMDL to help ensure that the BMPs are being implemented.

Several of the completed activities resulted in a reduction of sediment-laden irrigation waste water discharged from the Belle Fourche Irrigation District (BFID) delivery system into surrounding water by 3,455 acre-feet per year. This brings the total acre-feet reduction to 5,655, or 33 percent of the 10-year goal. Twenty-five real-time stage control units installed on the gates of check structures on both the north and south canals reduced nonused irrigation water by more precisely maintaining the level within the canals and laterals. The upgraded water card and water order system was completed to help check for mathematical errors associated with hand calculations. Data from permanent stage/flow-measuring devices, flow automation units, and portable stage-measuring units were used to calibrate and validate a canal operational model. The BFID lined 4,660 feet of the inlet canal and 2,600 feet of the Welke Lateral along with installing 4,946 feet of pipeline that delivers water from the BFID to the producers.

Several activities were completed to improve irrigation efficiencies after water was delivered to producers. A total of 31,732 feet of pipeline was installed by 20 producers to convey water to center pivot irrigation systems or to gated pipe that replaced open ditches. Seventeen center-pivot sprinkler systems were installed to replace existing surface irrigation.

Grazing/riparian areas were improved significantly within the watershed. Approximately 41 miles of pipeline, 56 watering facilities, 10 wells, and 3 miles of cross fence were installed using 319 dollars to provide off-stream livestock water and improve grazing distribution. These projects involved 30 producers on over 200,000 acres, resulting in 18,138 acres of riparian vegetation improvements. Conservation plans were written for over 120,000 acres of grazing lands.

Approximately 45 public education and outreach events were completed during this project segment. Outreach activities were in the form of public meetings, informational booths, Web site maintenance, radio sound bites, and watershed tours. It is estimated that outreach and education efforts reached approximately 10,000 people. A new brochure was developed for use at informational booths to showcase some of the BFRWP's projects and to explain their purpose and mission. The Butte County, Lawrence County, and Elk Creek Conservation Districts each sent out newsletters which included project updates. The BFRWP hosted 12 meetings to provide updates on project work and progress being made. The BFID sent out a newsletter called the *Ditch Writer* to over 480 irrigators in the BFID informing them of the status of the projects going on throughout the BFID. The BFRWP Web site continues to be updated with happenings and project status and is located at <*www.bellefourchewatershed.org*>. Outreach activities have helped increase participation and support in the BFRWP and also gave the BFRWP several contacts for BMP installation.

Preliminary estimates based on BMP installation indicate that TSS load was reduced by 83,833 tons per year, which is 12,105 tons per year greater than what was estimated to be accomplished in this project segment.

ACKNOWLEDGEMENTS

The Belle Fourche River Watershed Partnership would like to thank all those involved with this segment of the implementation of practices recommended from the Belle Fourche River Watershed Total Maximum Daily Load. The efforts of all those involved from the following organizations are greatly appreciated and have been essential to the success of this project:

Belle Fourche Irrigation District

Butte County Conservation District

Crook County Conservation District

Elk Creek Conservation District

Individual ranchers, farmers, and landowners within the watershed

Lawrence County

Lawrence County Conservation District

Natural Resources Conservation Service

South Dakota Association of Conservation Districts

South Dakota Conservation Commission

South Dakota Department of Agriculture

South Dakota Department of Environment and Natural Resources

South Dakota Game Fish and Parks

South Dakota Grassland Coalition

South Dakota School of Mines and Technology

South Dakota State University

United States Army Corp of Engineers

United States Bureau of Reclamation

United States Environmental Protection Agency

United States Fish and Wildlife Service

United States Geological Survey

Wyoming Department of Environmental Quality.

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1.0 INTRODUCTION

The Belle Fourche River is a natural stream that drains parts of Butte, Lawrence, and Meade Counties in South Dakota. The headwaters are located in Wyoming. The river flows into the Cheyenne River (Figure 1-1) in southern Meade County and ultimately to the Missouri River. The Belle Fourche River Watershed encompasses approximately 2,100,000 acres (3,300 square miles) in South Dakota and includes Hydraulic Units 10120201, 10120202, 10120203. The city of Spearfish (population 8,606) is the largest municipality located in the South Dakota portion of the watershed. Other South Dakota communities in the watershed include Deadwood (population 1,380), Lead (3,027), Sturgis (4,442), Belle Fourche (4,565), Fruitdale (62), Nisland (204), and Newell (646).

Land in the watershed is used primarily for grazing with some cropland and a few urban areas. Wheat, alfalfa, native and tame grasses, and hay are the main crops. Some corn is grown in the Belle Fourche Irrigation District (BFID). Gold mining, while reduced in scope from the past, and silviculture occur in the Black Hills portion of the watershed. Approximately 15 percent of the watershed is federally owned. Of this, 11 percent is managed by the U.S. Forest Service and four percent by the Bureau of Land Management.

The Belle Fourche River is identified in the 1998 and 2002 *South Dakota 303(d) Waterbody Lists* and the 2004 and 2006 *Integrated Report for Surface Water Quality Assessment* (IR) as impaired because of elevated total suspended solids (TSS) concentrations. According to the 2006 IR, the Belle Fourche River from the Wyoming border to the Cheyenne River, South Dakota, failed to support its assigned uses because of high TSS concentrations. In the report, agricultural activities were listed as a likely source of occasional impairment. This report also states that a natural source of TSS may be the erosion of exposed shale beds that lie along the river and its tributaries. The 2008 IR shows all segments of the Belle Fourche River, with the exception of the reach from the Wyoming border to Fruitdale, South Dakota, were delisted after water-quality standards for TSS were met. Table 1-1 contains a summary of 12 impaired TMDL segments within the Belle Fourche River Watershed. The table also lists the impaired beneficial use, impairment parameter, water-quality data, and possible source.

Horse Creek was listed in the 1998 impaired waterbody list for total dissolved solids (TDS) that was later determined to be a listing error. The Horse Creek listing was corrected to conductivity during 2002. During this assessment, approximately 10 percent of the samples collected from Horse Creek exceeded the water-quality standard for TSS. The 2008 IR lists Horse Creek as nonsupporting for conductivity and delisted for TSS. The TMDL report for Horse Creek includes both TSS and conductivity.

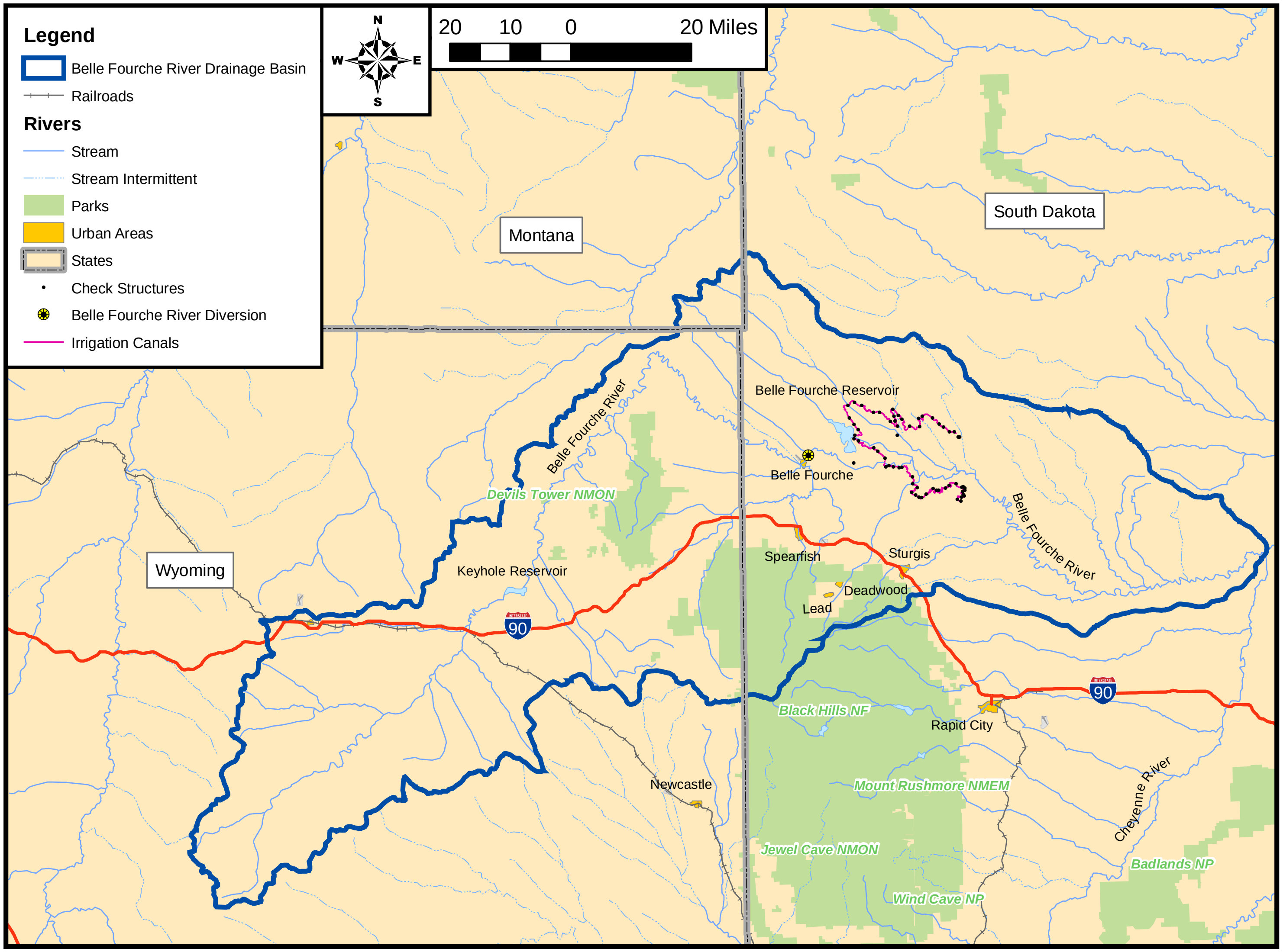

Figure 1-1. Belle Fourche River Watershed.

Table 1-1. Summary of Belle Fourche River Watershed Exceedance Water-Quality Data (Page 1 of 2)

Stream	WQM/ USGS[a]	Beneficial Use	Impairment Parameter	Water Quality Criteria	Source
Bear Butte Creek[b]	460126	Cold-Water Permanent Fish Life	Water Temperature ($^{\circ}$F)	<65°F	Natural Source
Bear Butte Creek[c]	460125	Cold-Water Permanent Fish Life	Water Temperature ($^{\circ}$F)	<65°F	Natural Source
Belle Fourche River[d]	460130	Immersion Recreation	Fecal Coliform (per/100 mL)	200[e]/400[f]	Riparian Grazing/Wildlife
		Warm-Water Permanent Fish Life	TSS (mg/L)	90[e]/158[f]	Crop Production/ Livestock
Horse Creek[g]	6436760	Irrigation Waters	Conductivity (mohms/cm @ 25°C)	2,500[e]/4,375[f]	NA
Redwater River[h]	6430500	Cold-Water Permanent Fish Life	Water Temperature ($^{\circ}$F)	<65°F	Natural Source
Strawberry Creek	460116	Cold-Water Marginal Fish Life	pH	6.5–8.8	Mining Impacts
		Fish/Wildlife Prop. Rec. Stock Waters	Cadmium (mg/L)	(i)	Mining Impacts
West Strawberry Creek	460675	Cold-Water Permanent Fish Life	Water Temperature ($^{\circ}$F)	<65°F	NA
		Limited Contact Recreation	Fecal Coliform (per/100 mg/L)	1,000[c]/2,000[f]	NA
Whitewood Creek[j]	460686	Cold-Water Permanent Fish Life	Water Temperature ($^{\circ}$F)	<65°F	NA

Table 1-2. Summary of Belle Fourche River Watershed Exceedance Water-Quality Data (Page 1 of 2)

Stream	WQM/ USGS	Beneficial Use	Impairment Parameter	Water Quality Criteria	Source
Whitewood Creek[k]	460123	Immersion Recreation	Fecal Coliform (per/100 mg/L)	200[c]/400[f]	Combined Sewers/Grazing
Whitewood Creek[l]	460684	Cold-Water Marginal Fish Life	pH	6.5–8.8	Natural Sources
Whitewood Creek[m]	460652	Warm-Water Permanent Fish Life	pH	6.5–9.0	Natural Sources
Whitewood Creek[n]	NA	Irrigation Waters	Conductivity (mohms/cm @ 25°C)	2,500[c]/4,375[f]	NA

(a) WGM/USGS is water-quality monitoring/U.S. Geological Survey

(b) Headwaters to Strawberry Creek.

(c) Strawberry Creek to mouth.

(d) Wyoming border to near Fruitdale, South Dakota.

(e) 30-day average.

(f) Daily maximum.

(g) Indian Creek to mouth.

(h) Wyoming border to US HWY 85.

(i) Cadmium Concentration $< (1.136672 - ((\ln(\text{hardness}) \times 0.041838) \times \exp(1.128 \times (\ln(\text{hardness})) - 3.828)$.

(j) Whitetail Summit to Gold Run Creek.

(k) Deadwood Creek to Spruce Gulch.

(l) Sandy Creek to I-90.

(m) I-90 to Crow Creek.

(n) Near Vale, South Dakota.

The Belle Fourche River Watershed Partnership (BFRWP) completed a water-quality assessment project which led to development of a TSS Total Maximum Daily Load (TMDL) for the Belle Fourche River and Horse Creek. The project period extended from April 2001 through 2003. Six TMDLs were approved by the U.S. Environmental Protection Agency (EPA) for the Belle Fourche River and Horse Creek in 2005. Based on the results of the watershed study, the main sources of TSS were determined to be rangeland erosion, irrigation return flows, free cattle access to streams, riparian degradation, natural geologic processes, hydraulic alteration by irrigation, and reduced stream miles. The *Ten-Year Belle Fourche River Watershed Strategic Implementation Plan* [Hoyer, 2005] developed to implement the TMDL includes recommendations for reducing TSS concentrations using practices that include irrigation water management, riparian rehabilitation, and grazing management. TMDLs are in review for Whitewood Creek and Bear Butte Creek. These TMDLs include Whitewood Creek listings for pH, fecal coliform, and temperature and Bear Butte Creek listings for temperature and TSS.

During the winter 2004, the BFRWP applied for and received a Clean Water Act Section 319 Grant to begin implementation of the best management practices (BMPs) recommended in the TMDLs for the Belle Fourche River. Currently, the BFRWP is in its fifth year of implementing BMPs in the watershed and has been funded through Fiscal Year 2011 with the Segment IV proposal. The project is supported by agricultural organizations, federal and state agencies, local governments, South Dakota State University (SDSU), and the South Dakota School of Mines and Technology (SDSM&T).

Funding for the project included support from local ranchers and farmers, BFRWP, South Dakota Department of Environment and Natural Resources (DENR), United States Fish and Wildlife Service (USFWS), Lawrence County, BFID, Wyoming Department of Environmental Quality (WYDEQ), Natural Resources Conservation Service (NRCS), Corps of Engineers, Bureau of Reclamation (BOR), United States Geological Survey (USGS), and the Clean Water Act Section 319 Grant. Products of the first implementation project segment were the *Ten-Year Belle Fourche River Watershed Strategic Implementation Plan* [Hoyer, 2005] and the *Belle Fourche Irrigation District Water Conservation Plan* [Rolland and Hoyer, 2005]. These plans outline BMP installation activities to be completed in this project for a 10 year time frame, and associated TSS and nonused water savings are presented for each action planned. BMPs recommended by the TMDLs and the 10-year plan installed during this project segment include flow automation units, real-time stage/flow-measuring devices, upgraded water card and water ordering system, updated canal operational model, replacing open irrigation ditches with pipeline, lining open irrigation ditches, installing pipelines to deliver water from the BFID system to the fields, installation of irrigation sprinkler systems within the BFID, and managed grazing. These BMPs were installed in the South Dakota portion of the Belle Fourche River Watershed (Figure 1-1).

2.0 PROJECT GOALS AND OBJECTIVES

The goal of the Belle Fourche River Watershed Management Project is to bring the Belle Fourche River and Horse Creek into compliance with TSS water-quality standards within 10 years. To accomplish the goal, a reduction of 55 percent (289,910 tons/year) in TSS is required. A reduction of 41 percent (2,033 tons/year) in TSS is required for Horse Creek.

In this project segment, the load reduction goal is 71,425 tons per year. To accomplish this goal, this project segment had three objectives:

1. Continue implementation of BMPs in the watershed to reduce TSS 46.6 mg/L below the Belle Fourche Reservoir and 41.6 mg/L above the Belle Fourche Reservoir.
2. Conduct public education and outreach to stakeholders within the Belle Fourche River Watershed.
3. Track progress toward meeting TMDL goals to help ensure that the BMPs are effective and that the proper BMPs are being implemented.

2.1 PLANNED AND ACTUAL MILESTONES, PRODUCTS, AND COMPLETION DATES

Objective 1. Implement BMPs Recommended to Reduce TSS. This objective was comprised of two tasks: improving irrigation water management and implementing riparian vegetation improvements. The products of this objective included 25 real-time stage control units; a water card/water ordering system; 9 real-time stage/flow-measuring devices; a canal operational model for the BFID north canal; replacement of canals, laterals, and/or ditches with 4,946 feet of pipelines; 4,660 feet of inlet canal lining and 2,600 feet of lateral lining; 31,995 feet of pipeline installed to convey water to center-pivot irrigation systems or to gated pipe that replaced open ditches; installing of 17 sprinkler irrigation systems; replacing existing flood irrigation; and rangeland implementation projects benefiting 18,138 riparian acres. Implementation of the BMPs is discussed further in Chapter 3.0.

Objective 2. Conduct Public Education and Outreach. There were approximately 45 outreach activities that are further discussed in Chapter 5.0 of this report.

Objective 3. Tracking Progress Toward Meeting Goals. Water-quality samples were collected by USGS at real-time stream gauging sites and DENR at several water-quality monitoring (WQM) sites in the watershed. A detailed statistical analysis is included in Chapter 4.0 of this report. Midyear and annual Grant Tracking and Reporting System (GRTS) reports were completed on schedule along with this final project report.

Table 2-1 lists the project objectives along with their products, planned milestone completion date, and actual milestone completion date. An extension of time from June 2009 to December 2009 was requested from and granted by the SD DENR. The extension of time was needed by agricultural producers to complete installation of BMPs because of extremely wet conditions in the area in the spring of 2009.

Table 2-1. Planned Versus Actual Milestone Completion Dates

Belle Fourche River Watershed Partnership Implementation	**Planned Completion**	**Actual Completion**
Objective 1. Implement BMPs Recommended to Reduce TSS		
Product 1. Improve Irrigation Delivery	June 2009	June 2009
Product 2. Improve Irrigation Application	June 2009	December 2009
Product 3. Complete and Install Riparian Vegetation Improvements	June 2009	December 2009
Objective 2. Conduct Public Education and Outreach		
Product 4. Supplement Existing Outreach Programs	June 2009	June 2009
Objective 3. Tracking Progress Toward Meeting Goals		
Product 5. GRTS and Final Reports	June 2009	December 2009

2.2 EVALUATION OF GOAL ATTAINMENT

Project success was evaluated by comparing project outputs and outcomes with the planned milestone. All objectives established for this project were reached:

- Implementation of several BMPs from Phase I Watershed Assessment Final Report and TMDL [Hoyer and Larson, 2004].
- Load reductions, estimated as a result of BMP installation, of 83,833 tons per year which is 12,105 tons per year greater than the goal for the project.

- Completion of approximately 45 successful education and outreach activities which led to greater public participation in the project.
- Completion of midyear and annual GRTS reports along with this final report.

This project was very successful. The project goals were exceeded for all of the objectives. BMPs were implemented that are estimated to reduce TSS in the Belle Fourche River by 83,833 tons per year.

3.0 BEST MANAGEMENT PRACTICES

Installation of the BMPs recommended in the Belle Fourche River TMDL was continued during this project segment. The BMP installation included funding from local ranchers and farmers, BFID, BOR, USFWS, and NRCS as well as financial assistance from the 319 project.

The BMPs installed included the following:

- 25 real-time stage control units.
- 9 real-time stage/flow-measuring devices.
- A north canal operational model.
- A water card/water ordering system.
- Replacement of open irrigation ditch with 4,946 feet of pipeline.
- 7,260 feet of inlet and lateral lining.
- 31,732 feet of pipeline installed by 19 producers to convey water to center-pivot irrigation systems or to gated pipe that replaced open ditches.
- Installed 17 irrigation sprinkler systems, approximately 41 miles of pipeline, and 56 watering facilities, 10 wells, and 3 miles of cross fence installed to provide off-stream livestock water and improve grazing distribution involving 30 producers on over 200,000 acres resulting in 18,138 acres of riparian vegetation improvements.
- Wrote conservation plans for over 120,000 acres of grazing lands.

Table 3-1 provides a track of BMP implementation planned and implemented to date.

Table 3-1. BMPs Implemented

Best Management Practice	10-Year Plan	Planned This Segment	Installed This Segment	Installed to Date
Flow Automation Units	42	25	25	44
Real-time Stage/Flow-Measuring Devices	15	0	9	24
Canal Operational Model	2	1	1	2
Water Card Ordering System	1	1	1	1
Line Open Canals and Laterals (Feet of Lining)	26,560	3,200	7,260	9,060
Replace Open Canals and Laterals With Pipeline (Feet of Pipeline)	25,000	4,000	4,946	7,796
Sprinkler Irrigation Systems	36	10	17	23
Managed Riparian Grazing (Acres)	34,000	9,000	18,138	19,638

3.1 REDUCING NONUSED IRRIGATION WATER AND IMPROVING EFFICIENCY

To reduce return flows of nonused irrigation waters, BMPs that will improve precision in water quantity delivered to irrigators were installed. The installation of nine real-time stage control units, coupled with the ten real-time, flow-measuring devices within the BFID delivery system, enables water levels to be measured, monitored, and adjusted from the BFID office in Newell, South Dakota. Six portable stage-measuring devices were also purchased to aid in developing the north canal operational model.

Over the life of this implementation project, there have been 27 gate control units and 18 flow-measuring devices installed (Figure 3-1). These automated units provide continual oversight of canal water levels and the ability to immediately adjust levels when necessary, thereby reducing waste and improving efficiency (Figure 3-2). Water-level data at each site are recorded every 10 minutes and stored in a database. This allows for easy summation of the total volume of water delivered during any given time period and calculation of efficiencies.

An upgraded water card ordering system was also implemented. The system allows BFID personnel to enter the timing and amount of water ordered for individual farmers on a given ride (or section of the irrigation district). Once this information is entered, the upgraded water card ordering system generates daily water delivery cards for the ditch riders that deliver the water to the fields. It also calculates the amount of lag time that it takes for the water to travel from the dam to all fields within the BFID and provides a daily estimate of the amount of water to release from the dam to meet the water order demands. This system eliminates mathematical and transcription errors from manual data entry.

Currently, the entire north canal is set up in the **Storm Water Management Model** (**SWMM**), an EPA model capable of simulating all the conditions within the north canal. The model was calibrated and validated using data collected at automated checks and portable stage-measuring devices as well as manual field measurements collected during the summers of 2006 through 2008. The hydraulic model is capable of assisting with irrigation delivery system settings and improving irrigation efficiency during future irrigation season. To help validate the **SWMM** model, operational curves, charts, and spreadsheets were developed for five automated check structures within the BFID. These tools provide BFID personnel with a better understanding of how to optimally operate automated check structures and offer flow measurements based on the check settings and upstream water levels. Using the operational curves, charts, and spreadsheets along with the developed **SWMM** model will help BFID personnel understand the dynamic irrigation system. This understanding will reduce irrigation return flows and, in turn, TSS levels in the Belle Fourche River.

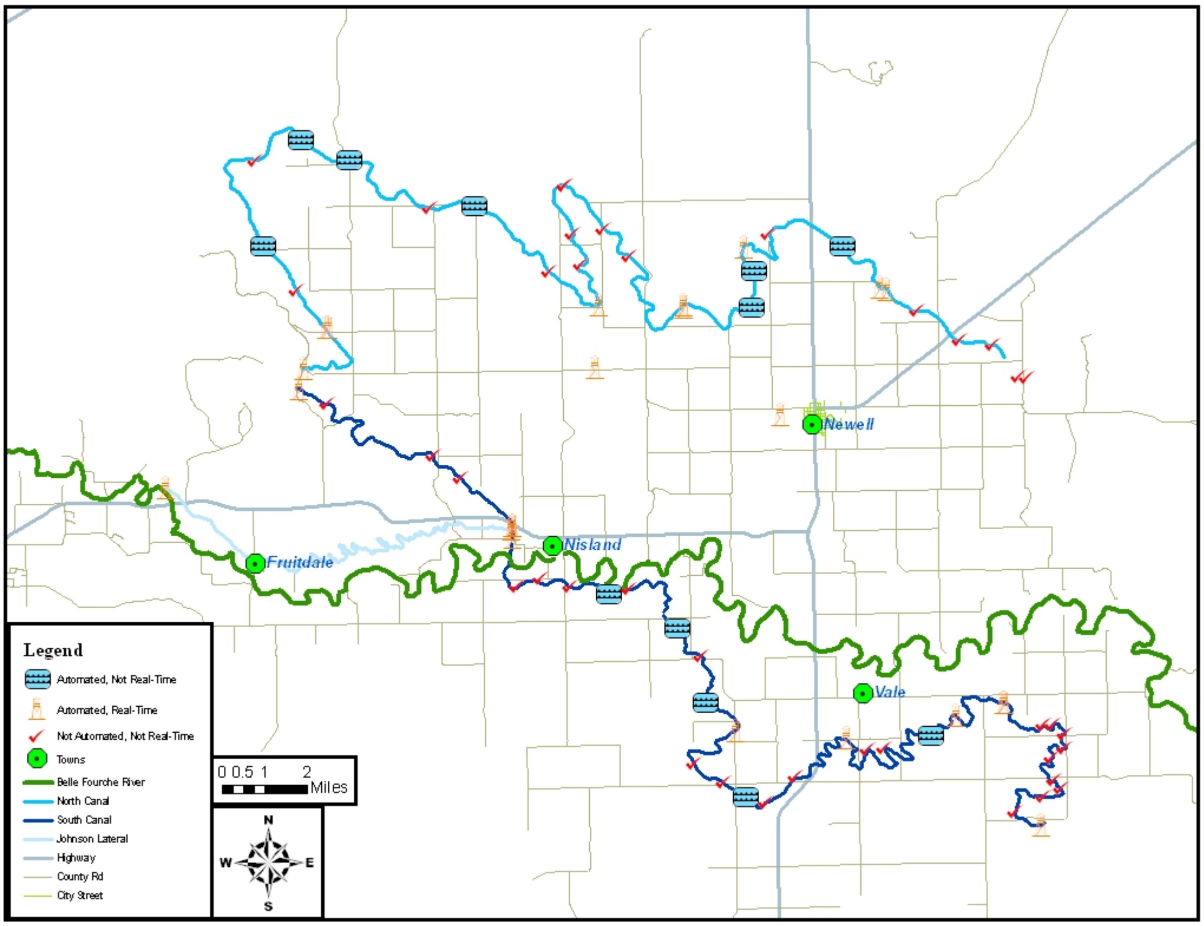

Figure 3-1. Location of Automated Sites in the Belle Fourche Irrigation District.

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Figure 3-2. Gate Automation Unit Installed in the Belle Fourche Irrigation District.

A total of 31,732 feet of pipeline was installed by 19 producers to convey water to center-pivot irrigation systems or to gated pipe that replaced open ditches. Seventeen center-pivot sprinkler systems were installed to replace existing surface irrigation (Figure 3-3). Locations of producer irrigation BMPs is shown in Figure 3-4.

RSI-1870-09-010

Figure 3-3. Center Pivot Installed in the Belle Fourche Irrigation District.

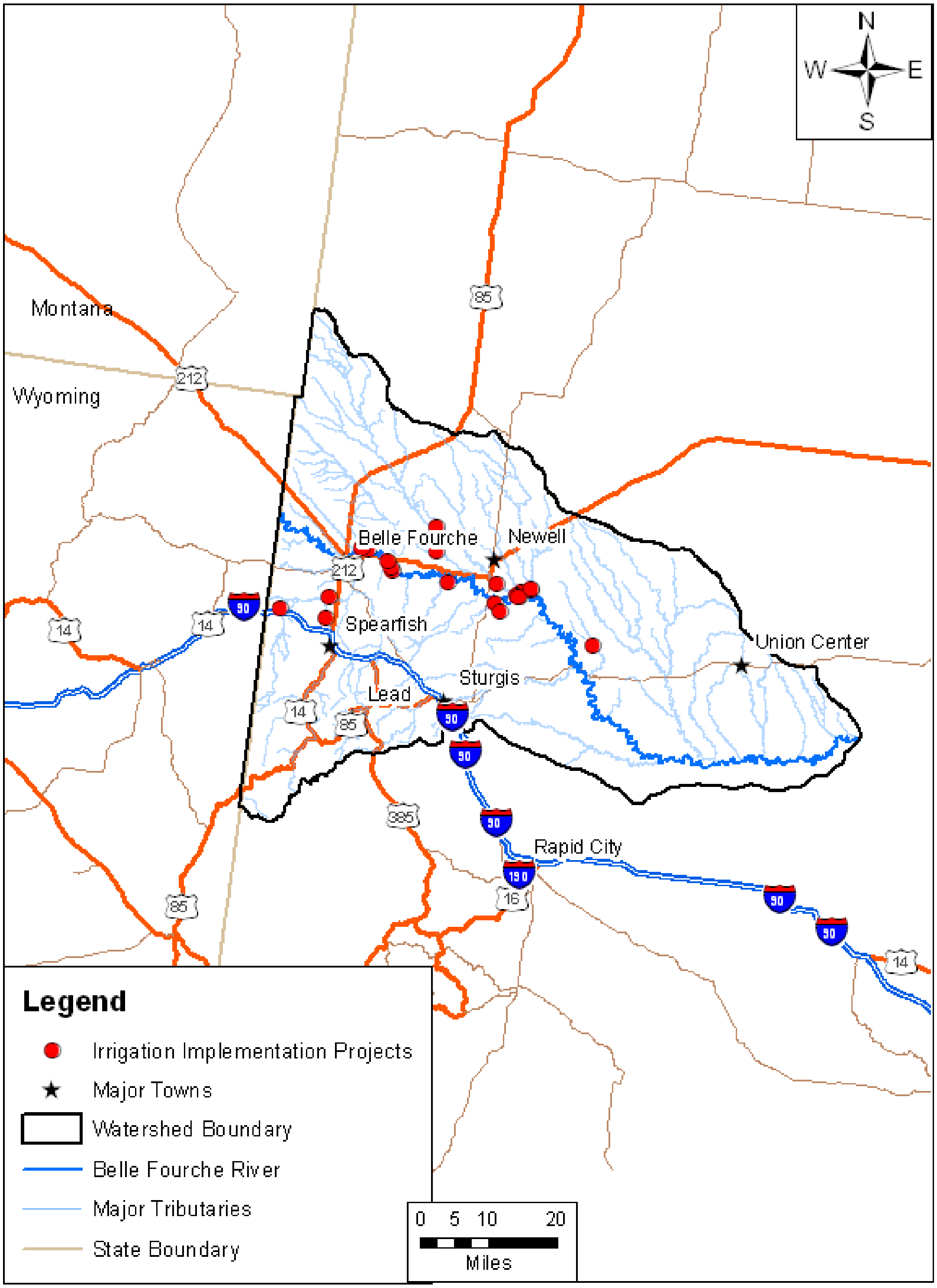

Figure 3-4. Location of Producer Irrigation Implantation Project in Segment III.

Over 2,600 feet of lateral lining was completed by the BFID on the wilson lateral and 4,660 feet of lining on the inlet canal, totaling 7,260 feet of lining. The inlet canal lining is shown in Figure 3-5. A total of 4,946 feet of canal and open laterals within the BFID were replaced with pipeline. Installation of pipeline eliminated water losses from infiltration and evaporation along these sections.

RSI-1870-09-012

Figure 3-5. Lining of the Inlet Canal.

3.2 MANAGED GRAZING

Information from resource inventories of several ranches located in the watershed were used to plan and install BMPs that significantly improved grazing/riparian areas within the watershed. Grazing/riparian areas were improved significantly within the watershed. Approximately 41 miles of pipeline and 56 watering facilities, 10 wells, and 3 miles of cross fence were installed using 319 dollars to provide off-stream livestock water and improve grazing distribution. Improved grazing distribution maintains or improves the integrity of the riparian corridor of the watershed. Healthy riparian areas are integral to trapping sediment from rangeland runoff, reducing TSS entering the Belle Fourche River. These projects involved 30 producers on over 200,000 acres resulting in an estimated 18,138 acres of riparian vegetation improvements (Figure 3-6). In addition to practices installed, conservation plans were written for over 120,000 acres of grazing lands in the watershed. A ranch with a planned grazing system is shown in Figure 3-7. The locations of the implementation projects are shown in Figure 3-8.

RSI-1870-09-013

Figure 3-6. Off-Stream Livestock Water Development in the Watershed.

RSI-1870-09-014

Figure 3-7. Planned Grazing System in the Watershed.

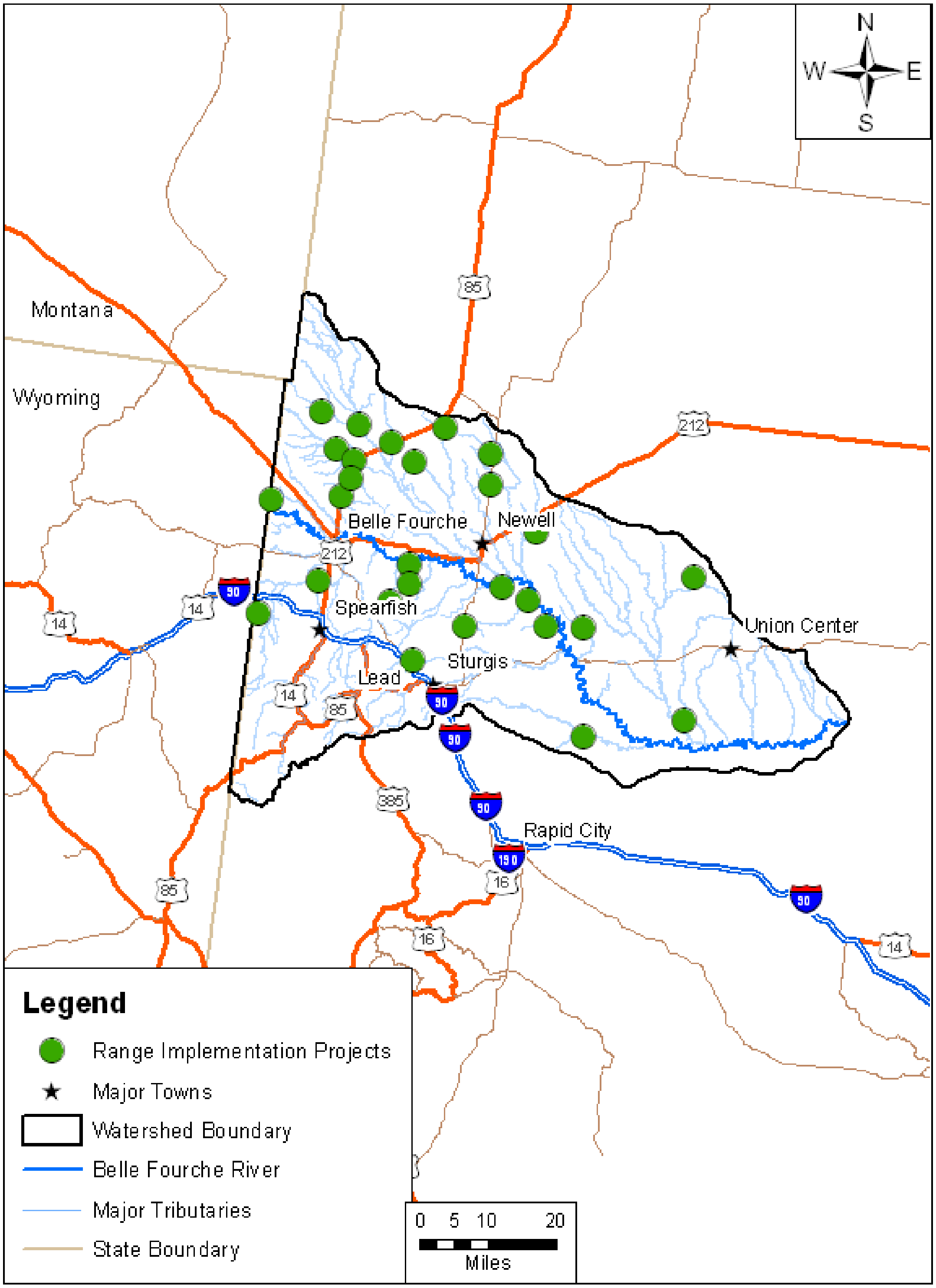

Figure 3-8. Location of Producer Range Implementation Projects in Segment III.

Along with the riparian vegetation BMPs a graduate student from SDSU was funded to complete a 2-year study quantifying rainfall infiltration rates and sediment loss associated with runoff. Vegetation, infiltration, runoff, and sediment data for predicting load and factors contributing to load reduction using rainfall simulations were developed for two prevalent ecological sites (dense clay and clayey) during the summers of 2007 and 2008. Data showed that no differences occurred between the two ecological sites for infiltration rates, runoff rates, and time to runoff. However, sediment yield and sediment concentrations were twice as high from dense clay sites than clayey sites. For each ecological site, models for time to runoff, infiltration rate, sediment concentration, and sediment yield were developed using 19 site and vegetation variables for both antecedent moisture conditions as well as 24 hours after a rainfall simulation on wet soils. Models, as well as collected data, provide not only a better understanding of rangeland hydrology but also information specific to the Belle Fourche watershed, which can be used in identifying areas of concern, prediction, and in grazing management to favorably impact vegetation characteristics important to load reduction for the Belle Fourche River. A complete version of this 2-year study will be available in the form of a master's thesis in spring 2010. Figure 3-9 shows a rainfall simulation event on one of the selected sites.

RSI-1870-09-016

Figure 3-9. Rainfall Simulators for Infiltration and Runoff Study.

4.0 SUMMARY OF PUBLIC PARTICIPATION AND OUTREACH

Approximately 45 public education and outreach events were completed (Table 4-1) during this project segment. Outreach activities were in the form of public meetings, informational booths, Web site, radio sound bites, and watershed tours. It is estimated that outreach and education efforts reached approximately 10,000 people. A new brochure was developed for use at informational booths to showcase some of the BFRWP's projects and to explain their purpose and mission. The Butte County, Lawrence County, and Elk Creek Conservation Districts each sent out newsletters which included project updates. The BFRWP hosted 15 meetings to provide updates on project work and progress being made. The BFID sent out a newsletter called the *Ditch Writer* to approximately 480 producers in the BFID informing them of the status of the projects throughout the BFID. The BFRWP Web site, located at <*www.bellefourchewatershed.org*>, continues to be updated with events and project status. The BFRWP purchased a soil-quality demonstration trailer, shown in Figure 4-1, used to educate audiences of all ages about the importance of good stewardship on soil health.

The BFRWP sponsored/cosponsored four tours in the watershed during Segment III. These tours included local producers; state and federal agency staff; local, state, and federal government officials; and the interested public (Figure 4-2). Partners in these tours included Butte, Lawrence, and Elk Creek Conservation Districts, South Dakota Association of Conservation Districts, South Dakota State University Cooperative Extension, and Bureau of Reclamation. These tours showcased projects sponsored by the BFRWP, including irrigation demonstrations in the BFID and rangeland demonstrations on ranches in the watershed. These outreach activities helped increase participation and support for the BFRWP and also gave the BFRWP several contacts for BMP installation.

Table 4-1. Summary of Public Outreach and Education During Segment III

Type of Education and Outreach	Date	Number of Participants
Belle Fourche River Watershed Partnership Meetings (12 Meetings)	June 2006–June 2009	240
Belle Fourche Irrigation District Annual Meeting	2007	45
Newell Field and Home Show	2007, 2008, 2009	900
Eastern South Dakota Water Conference	2007	150
Ditch Writer Publication	2007, 2008, 2009	480
Black Hills Hydrology Conference	2007	100
Belle Fourche Capital for a Day Tour	2007	60
South Dakota Conservation Commission	2007	60
Butte-Lawrence County Fair	2007, 2008	1,300
Watershed Tour	2008	60
Tri State Expo	2008, 2009	1,000
Society for Range Management Presentation	2009	150
Western Resource Conservation & Development Conference	2009	200
Key City Pen of Three	2009	300
Cheyenne River Watershed Meeting (Rapid City)	2008	30
Soil and Water Conservation Society Convention	2009	150
South Dakota High School Range Camp	2008	75
Nonpoint Source Task Force Meeting	2006, 2007, 2008	135
Belle Fourche Irrigation District Tour	2007, 2008	65
Informational Radio Sound Bites	2009	5,000
Soil Quality Demonstrations (2 events)	2009	200

RSI-1870-09-017

Figure 4-1. Soil Quality Demonstration.

RSI-1870-09-018

Figure 4-2. Tour in the Belle Fourche Irrigation District.

5.0 MONITORING RESULTS

5.1 WATER-QUALITY ANALYSIS

To understand the effectiveness of the current implementation plan, a rigorous statistical analysis was performed on TSS data collected at the five WQM sites located on the mainstem of the Belle Fourche River (Figure 5-1). The data was grouped into two categories: Pre-BMP implementation, or before Year 2005 and Post-BMP implementation, or including and after Year 2005.

Table 5-1 displays the basic summary statistics of TSS for the WQM sites analyzed from upstream to downstream. The mean concentrations at all sites dropped after significant BMP implementation began in 2005 (post-BMP). On average, the mean TSS concentration has decreased 40 percent at all sites with the largest percent reduction taking place at WQM 130 (63.4 percent) and the least percent reduction occurring at WQM 21 (16.2 percent).

Simply analyzing the mean of any dataset can be misleading, however, as one sample with an unusually high concentration can skew the entire set of samples. For this reason, it is appropriate to analyze the sample set medians to understand if a similar trend exists. Figure 1 displays the median TSS concentrations at each WQM site pre- and post-BMP implementation. Median TSS values have decreased at WQM 83, 21, and 76 with no change occurring at WQM 130 and a slight increase being observed at WQM 81.

To understand the statistical significance of any of these changes in median values, either positive or negative, a Mann-Whitney test was performed using **Minitab** 14 statistical software. A two-sample rank test, such as the Mann-Whitney, tests the equality of two population medians. Datasets were once again separated into pre-BMP ($\eta 1$) and post-BMP ($\eta 2$) at each of the sites. The null hypothesis was that the median TSS concentration at each of the sites pre-BMP implementation was equal to the median TSS concentration at each of the sites post-BMP implementation. The alternate hypothesis was that the median TSS concentration at each of the sites pre-BMP implementation was not equal to the median TSS concentration at each of the sites post-BMP implementation. This is expressed mathematically as follows:

$$\begin{aligned} &H0{:}\eta 1 \neq \eta 2 \\ &H1{:}\eta 1 \neq \eta 2. \end{aligned} \tag{5-1}$$

The result of the Mann-Whitney test for all sites indicated the data not support the hypothesis that there is a difference between the population medians at the 95 percent confidence interval. In other words, although there appears to be a difference between the population medians, the lack of samples in the post-BMP datasets does not allow us to declare that this difference is statistically significant.

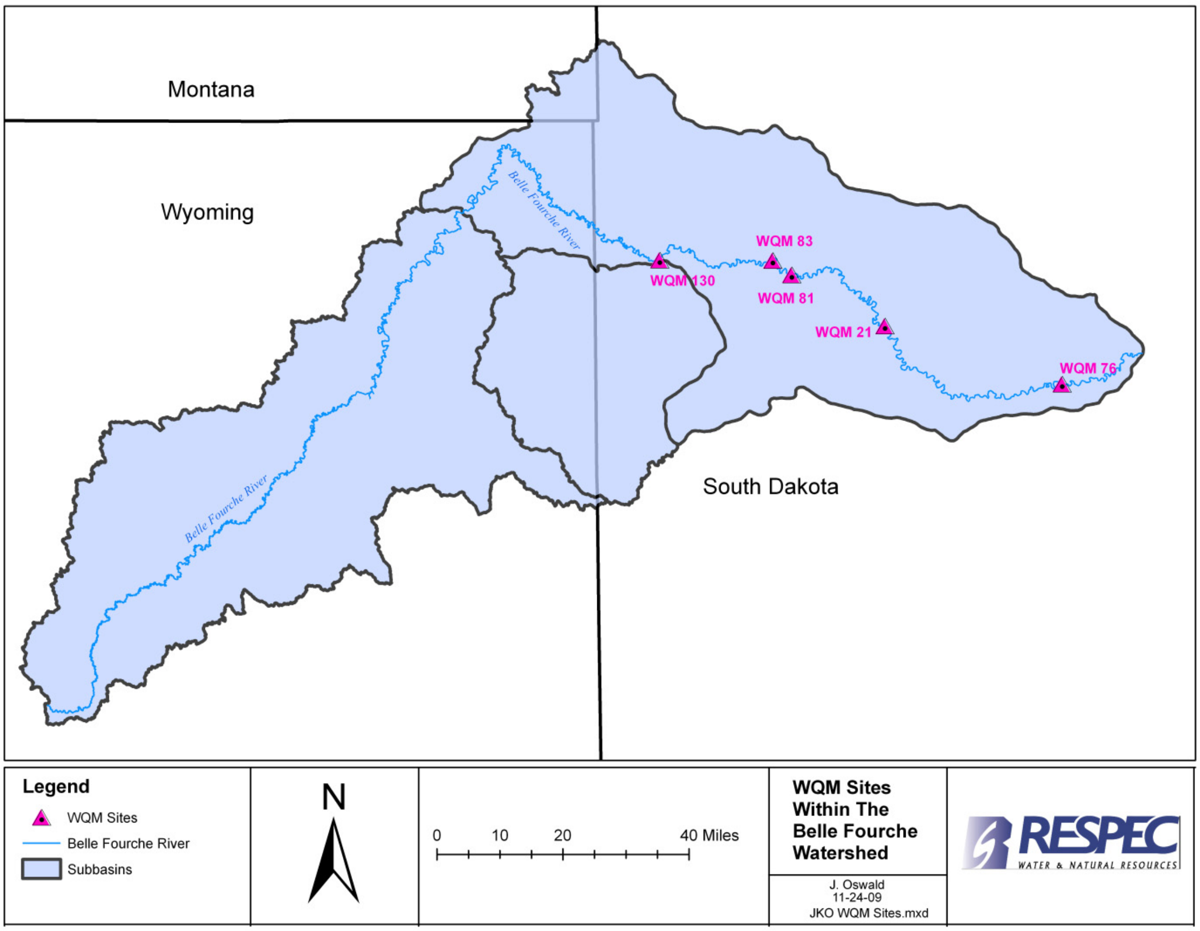

Figure 5-1. Location of the Five Water-Quality Monitoring Sites Within the South Dakota Portion of the Belle Fourche Watershed.

Table 5-1. Summary Total Suspended Solids Statistics for Mainstem Water-Quality Monitoring Sites on the Belle Fourche River in South Dakota

Site	BMP Status	Mean	Standard Deviation	Q1	Median	Q3	Min	Max	*n*
WQM 130	Post-BMP	155.8	370.6	5.0	8.0	102.0	1	2,000	41
WQM 130	Pre-BMP	245.7	781.6	5.0	8.0	87.0	5	4,520	37
WQM 81	Post-BMP	47.7	84.9	5.0	20.5	48.8	5	350	16
WQM 81	Pre-BMP	192.2	890.8	7.0	18.0	44.0	1	6,885	105
WQM 83	Post-BMP	31.1	34.9	5.0	19.0	48.0	5	130	16
WQM 83	Pre-BMP	77.8	154.5	9.3	34.5	68.8	1	885	104
WQM 21	Post-BMP	85.3	179.0	7.0	25.0	60.0	5	700	15
WQM 21	Pre-BMP	527.2	1,517.7	11.0	41.5	255.8	0	14,977	198
WQM 76	Post-BMP	196.7	875.2	5.3	32.5	58.8	1	5,800	44
WQM 76	Pre-BMP	350.0	1,280.8	8.5	35.0	110.0	1	11,000	135

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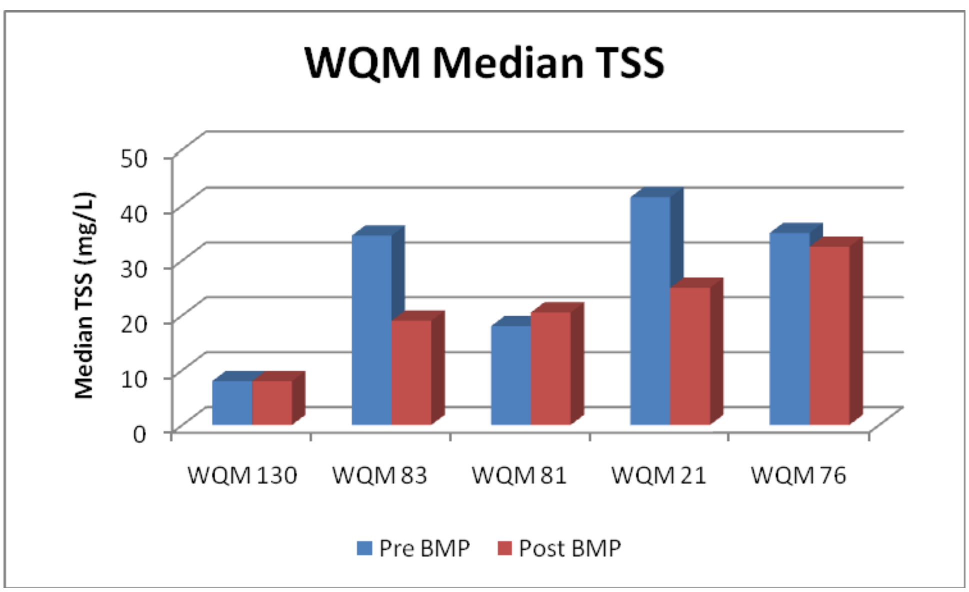

Figure 5-2. Median Total Suspended Solids Observed at Water-Quality Monitoring Sites Pre- and Post-Best Management Practice Implementation.

The end result for any 319 implementation project is for the designated waterbody to come into compliance with water quality standards. The water-quality standard for TSS with a stream assigned the designated beneficial use of warm-water fishlife propogation (the designated beneficial use of the mainstem of the Belle Fourche River) is 158 mg/L. Figure 5-3 displays the percent exceedances of the water quality standard at each of the WQM sites on the mainstem of the Belle Fourche River before and after BMP implementation. The percent of exceedances decreased substantially at all the WQM sites except WQM 130 that monitors water quality in the reach from the Wyoming border to the city of Fruitdale. This makes practical sense since a majority of the implementation projects thus far have been focused in and around the Belle Fourche Irrigation District, whose drainage is downstream of WQM 130.

5.2 HORSE CREEK FLOW ANALYSIS

Real-time discharge data collected by the USGS at Horse Creek was analyzed from April 1962 to October 2009. It should be noted that data from October 1, 2008, to October 30, 2009, is designated by the USGS as “Provisional Data Subject to Revision.” BMPs implemented within the Belle Fourche Irrigation District delivery system, along with on-farm improvements, are designed to reduce the volume of sediment-laden return flows impacting Horse Creek and ultimately the Belle Fourche River. Figure 5-4 shows the relation of the Horse Creek to the delivery system and fields located within the BFID.

The influence that waste from the BFID delivery system and fields has on flows in Horse Creek is evident when observing a boxplot of historic monthly flows at the sight (Figure 5-5). The boxplot shows 95 percent of the data (the highest and lowest 2.5 percent of values are considered outliers). Median value of the average daily flow are labeled in blue, the boxes delineate the inner quartile range (the range bounded by the 1st and 3rd quartiles), and the whiskers mark the extents of 95 percent of the data. The typical irrigation season in the BFID begins in June and lasts until the end of September. This is demonstrated in the boxplot as the median flow jumps from 18 cubic feet per second (cfs) in May to 41.5 cfs in June. The median flow then drops from 45 cfs in September to 5.9 cfs. Since the region receives very little precipitation during the irrigation season, nearly all of the increase in flow can be attributed to losses or waste within the irrigation system.

The impact of the BMPs being implemented within the BFID is becoming evident. Specifically, BMPs implemented in this watershed are automated gate controls and flow monitoring, underground pipelines replacing open ditches, sprinkler irrigation replacing flood irrigated fields, and irrigation scheduling. A box plot of the flows in Horse Creek before and after BMP implementation began (Figure 5-6) demonstrates that median flows during the irrigation season (June–September) are being dramatically reduced, especially in July and August, which are typically months with the greatest amount of irrigation deliveries.

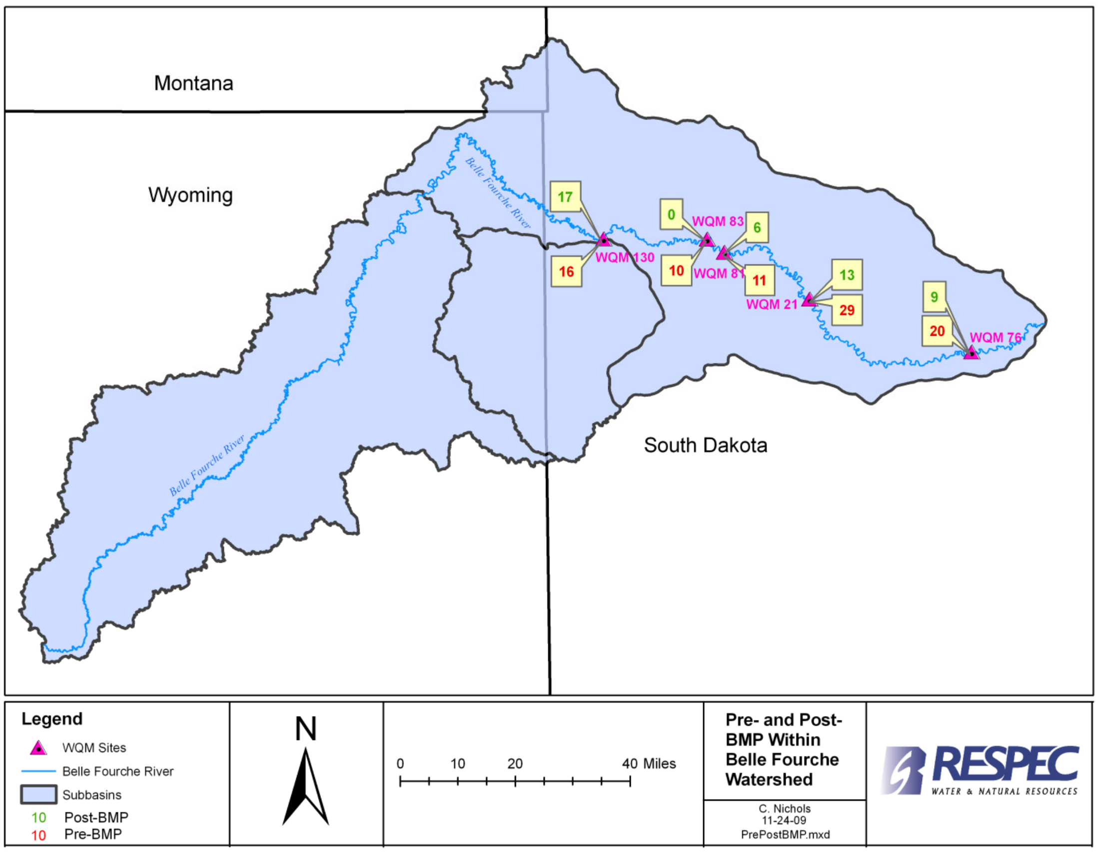

Figure 5-3. Percent Exceedances of Total Suspended Solids Water-Quality Standard Pre- (red) and Post- (green) Best Management Practice Implementation at the Water-Quality Monitoring Sites on the Mainstem of the Belle Fourche River in South Dakota.

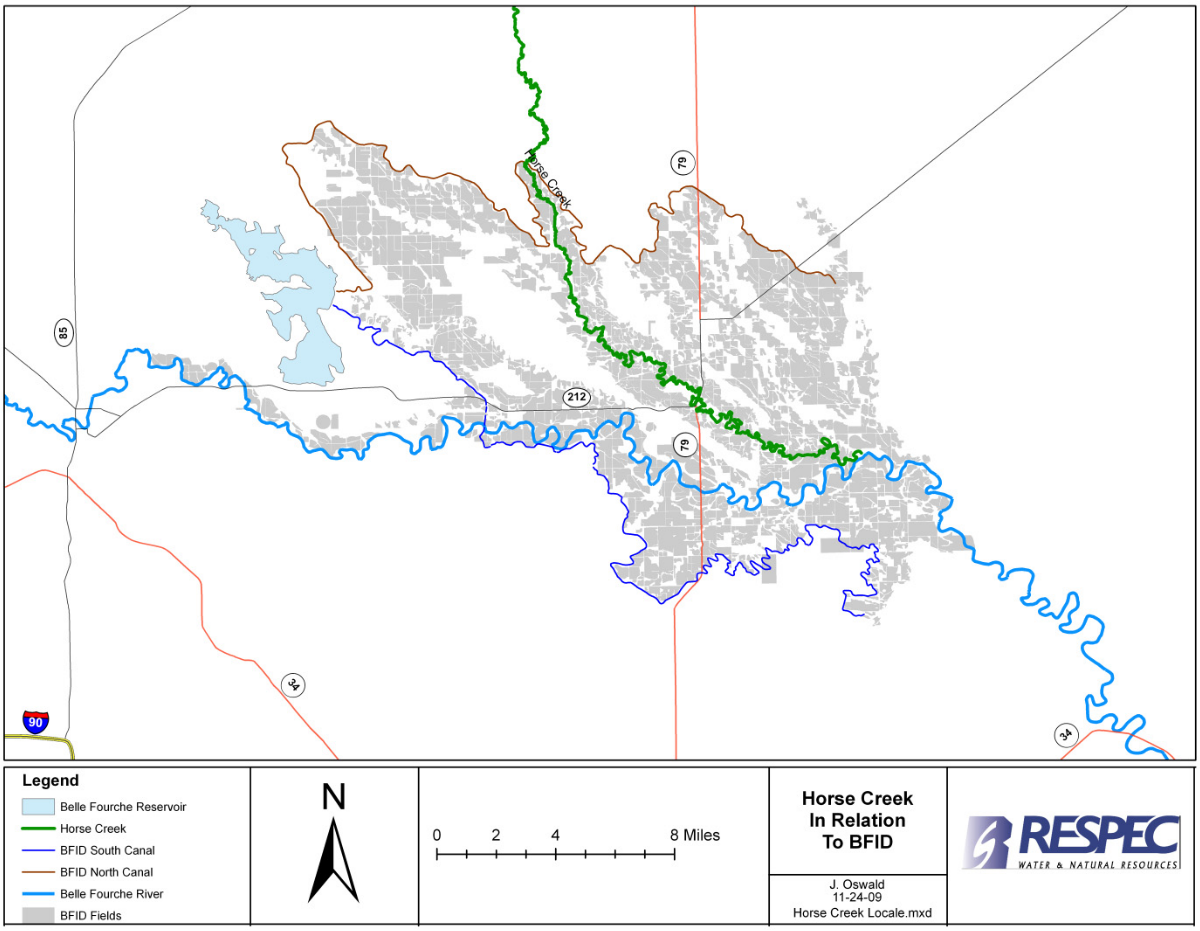

Figure 5-4. Location of Horse Creek in Relation to the Fields and Main Delivery System of the Belle Fourche Irrigation District.

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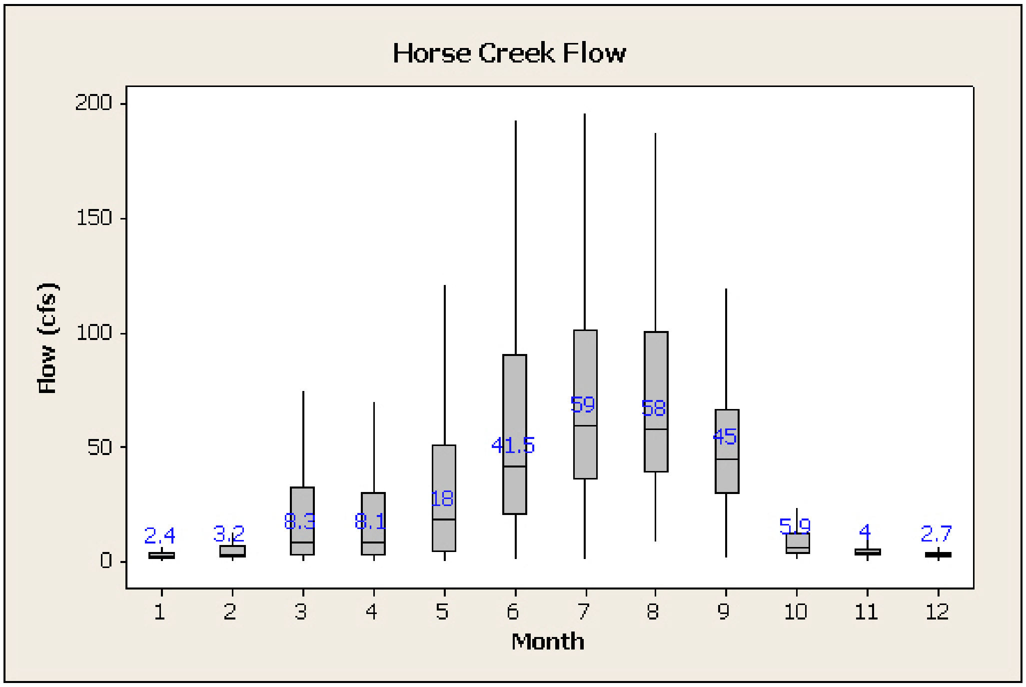

Figure 5-5. Box Plot of Historic Monthly Flows at the Mouth of Horse Creek.

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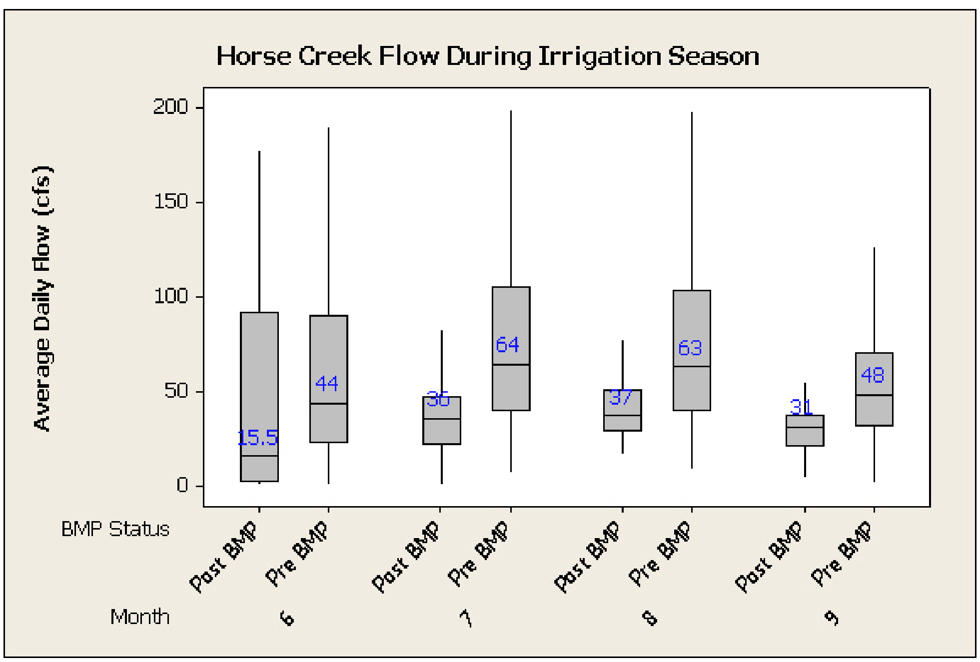

Figure 5-6. Box Plot of Average Daily Flow of Horse Creek During the Belle Fourche Irrigation District Irrigation Season Before and After Best Management Practice Implementation.

5.3 EVALUATION OF GOAL ATTAINMENT

Project success was evaluated by comparing planned versus actual project outputs and outcomes. The goal was attained by reaching the objectives as follows:

- Implementation of several BMPs from the 10-year BFRWP Strategic Implementation Plan.
- Load reductions, estimated as a result of BMP installation, of 83,833 tons per year which is 12,105 tons/year greater than the goal for the project.
- Completion of nearly 45 successful education and outreach activities which led to greater public participation in the project.
- Completion of midyear and annual GRTS reports along with this final report.

This project was successful in that project goals were attained and BMPs were implemented that are estimated to reduce total suspended solids in the Belle Fourche River and Horse Creek.

6.0 ASPECTS OF THE PROJECT THAT DID NOT WORK WELL

Extremely wet weather precluded many of the producer implementation projects from being completed by the original June 1, 2009, deadline prompting an extension to December 31, 2009, to complete the projects. With this extension, all projects were able to be completed as planned.

7.0 PROJECT BUDGET/EXPENDITURES

The BFRWP received a $1,728,800 EPA section 319 Grant through DENR to continue installation of the BMPs recommended in the *Phase I Watershed Assessment Final Report and TMDL* [Hoyer and Larson, 2004]. Tables 7-1 and 7-2 show the budgets of 319/matching funds and nonmatching funds respectively. The budgets were the final budgets after the approval of the Segment III amendment and the additional documented changes to the budget after the Segment III amendment. Tables 7-3 and 7-4 are the final expenditure budgets for 319/matching funds and nonmatching funds, resepectively.

7.1 319 BUDGET

The total 319 budget remained the same with some minor changes between tasks. From Task 1d, $136 was transferred to Task 2 to match the allotted money in the Tracker. Remaining money left over in Task 1d and Task 2, after all of the producer cost share was paid out, was transferred into Product 3. From Task 1d, $2,652 was transferred to Product 3 and $7 dollars was transferred from Task 2 to Product 3. No other changes were made to the 319 budget.

7.2 MATCHING FUNDS BUDGET

All federal match requirements were met in this project. Final match dollars were not as high as originally estimated. Producer cash match for Task 1d was not as high as estimated. Producer cash that would have been matched in this task was used to match another grant in the watershed, reducing cash match for the 319 project. SD DENR Water Rights cash match column was taken out of the final matching funds budget and put into the federal nonmatching funds budget as per the request of SD DENR. Conservation Commission, SRF Loan Lead, and SRF Nisland cash match columns were also taken out of the matching funds budget. These projects were expected to take place but did not occur. Other differences reflect minor adjustments to what was originally estimated.

7.3 NONMATCHING FEDERAL FUNDS BUDGET

The Bureau of Reclamation 2025 Grant was taken out of the final budget. This money was part of the inlet canal lining project and was recorded under the Bureau of Reclamation federal dollar column. The NRCS Wildlife Habitat Incentive Program (WHIP) column was taken out; very few WHIP projects take place in the watershed. Other differences reflect minor adjustment to what was originally estimated.

Table 7-1. Planned Belle Fourche River Watershed 319 and Matching Funds Budget

EPA 319 and Matching Funds Budget	EPA 319 ($)	Matching Funds ($)										Sum of Matching Funds ($)
		Producer (Cash and In-kind) ($)	BFRWP (Cash and In-kind) ($)	SD DENR Water Rights (Cash) ($)	Lawrence County (Cash) ($)	BFID (Cash and In-kind) ($)	WY DEQ (Cash)	Conservation Commission (Cash) ($)	DENR CWSRF Water Quality Grant ($)	SRF Loan Lead (Cash) ($)	SRF Loan Nisland (Cash) ($)	
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL												
Task 1. Reduce Nonused Water												
Product 1. Improved Irrigation Water Delivery and Application 3,400 Ac-ft Reduction of Nonused Water												
1a. 27 Stage Control Automation Projects	368,800	–	–	–	–	48,000	–	–	7,200	–	–	55,200
1b. Phase II of Canal Operational Model	221,604	–	6,000	–	–	120,000	–	–	25,000	–	–	151,000
1c. Line and Pipe Open Canals and Laterals	–	–	–	–	–	120,000	–	–	–	–	–	120,000
1d. Install Sprinkler Systems	252,599	2,037,500	–	–	–		–	–	–	–	–	2,037,500
Task 2. Install Riparian Vegetation Improvements												
Product 2. Grazing/Rangeland/ Riparian Management	751,972	401,000	–	–	–	–	–	82,500	7,800	–	–	491,300
Objective 2. Conduct Public Outreach, Complete Essential Water Quality Monitoring, and Write Reports												
Task 3. Conduct Public Outreach Program, Monitor Water Quality and Write Reports												
Product 3. Public Education and Outreach, Monitor Water Quality, Write Reports	133,825	–	12,000	78,275	15,655	11,741	15,655	–	50,000	–	–	183,326
Other Watershed Improvement Projects	–	–	–	–	–	–	–	–	–	210,000	70,000	280,000
Total	**1,728,800**	**2,438,500**	**18,000**	**78,275**	**15,655**	**299,741**	**15,655**	**82,500**	**90,000**	**210,000**	**70,000**	**3,318,326**

Table 7-2. Planned Belle Fourche River Watershed 319 and Nonmatching Funds Budget

EPA NonMatching Funds Budget	Nonmatching Funds ($)											Sum of Nonmatching Funds ($)
	USFWS (Federal) ($)	SDGF&P (Nonfederal) ($)	NRCS CIG Grant (Federal) ($)	NRCS CCPI Grant (Federal) ($)	NRCS RWA Grant (Federal) ($)	WHIP (Federal) ($)	NRCS EQIP (Federal) ($)	COE (Federal) ($)	BOR (Federal) ($)	BOR 2025 Grant (Federal) ($)	USGS (Federal) ($)	
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL												
Task 1. Reduce Nonused Water												
Product 1. Improved Irrigation Water Delivery and Application 3,400 Ac-ft Reduction of Nonused Water												
1a. 27 Stage Control Automation Projects	–	–	–	–	–	–	–	–	–	–	–	–
1b. Phase II of Canal Operational Model	–	–	–	–	–	–	–	–	–	–	–	–
1c. Line and Pipe Open Canals and Laterals	–	–	–	–	–	–	–	–	120,000	–	–	120,000
1d. Install Sprinkler Systems	–	–	–	–	–	–	412,500	–	–	–	–	412,500
Task 2. Install Riparian Vegetation Improvements												
Product 2. Grazing/Rangeland/ Riparian Management	137,500	50,000	–	200,000	85,000	104,000	608,700	–	–	–	–	1,185,200
Objective 2. Conduct Public Outreach, Complete Essential Water Quality Monitoring, and Write Reports												
Task 3. Conduct Public Outreach Program, Monitor Water Quality and Write Reports												
Product 3. Public Education and Outreach, Monitor Water Quality, Write Reports	–	–	–	–	–	–	–	15,655	7,828	–	199,601	223,084
Other Watershed Improvement Projects	–	–	200,000	–	–	–	–	–	–	125,000	–	325,000
Total	**137,500**	**50,000**	**200,000**	**200,000**	**85,000**	**104,000**	**1,021,200**	**15,655**	**127,828**	**125,000**	**199,601**	**2,265,784**

Table 7-3. Actual Expenditures of Belle Fourche River Watershed 319 and Matching Funds Budget

EPA 319 and Matching Funds Budget	EPA 319 ($)	Matching Funds ($)						Sum of Matching Funds ($)
		Producer (Cash and In-kind) ($)	BFRWP (Cash and In-kind) ($)	Lawrence County (Cash) ($)	BFID (Cash and In-kind) ($)	WY DEQ (Cash)	DENR CWSRF Water Quality Grant ($)	
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL								
Task 1. Reduce Nonused Water								
Product 1. Improved Irrigation Water Delivery and Application 3,400 Ac-ft Reduction of Nonused Water								
1a. 27 Stage Control Automation Projects	368,800	–	–	–	66,928	–	7,200	74,128
1b. Phase II of Canal Operational Model	221,604	–	0	–	0	–	25,894	25,894
1c. Line and Pipe Open Canals and Laterals	–	–	–	–	117,307	–	–	117,307
1d. Install Sprinkler Systems	249,811	1,275,498	–	–	–	–	–	1,275,498
Task 2. Install Riparian Vegetation Improvements								
Product 2. Grazing/Rangeland/Riparian Management	752,101	689,565	–	–	–	–	7,773	697,338
Objective 2. Conduct Public Outreach, Complete Essential Water Quality Monitoring, and Write Reports								
Task 3. Conduct Public Outreach Program, Monitor Water Quality and Write Reports								
Product 3. Public Education and Outreach, Monitor Water Quality, Write Reports	136,484	–	12,000	16,250	13,354	16,250	49,133	106,987
Other Watershed Improvement Projects	–	–	–	–	–	–	–	–
Total	**1,728,800**	**1,965,063**	**12,000**	**16,250**	**197,589**	**16,250**	**90,000**	**2,297,152**

Table 7-4. Actual Expenditures of Belle Fourche River Watershed 319 and Nonmatching Funds Budget

EPA Nonmatching Funds Budget	Nonmatching Funds ($)										Sum of Nonmatching Funds ($)
	USFWS (Federal) ($)	SDGF&P (Nonfederal) ($)	SD DENR Water Rights (Nonfederal) ($)	NRCS CIG Grant (Federal) ($)	NRCS CCPI Grant (Federal) ($)	NRCS RWA Grant (Federal) ($)	NRCS EQIP (Federal) ($)	COE (Federal) ($)	BOR (Federal) ($)	USGS (Federal) ($)	
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL											
Task 1. Reduce Nonused Water											
Product 1. Improved Irrigation Water Delivery and Application 3,400 Ac-ft Reduction of Nonused Water											
1a. 27 Stage Control Automation Projects	–	–	–	–	–	–	–	–	–	–	0
1b. Phase II of Canal Operational Model	–	–	–	–	–	–	–	–	–	–	0
1c. Line and Pipe Open Canals and Laterals	–	–	–	–	–	–	–	–	345,000	–	345,000
1d. Install Sprinkler Systems	–	–	–	–	–	–	257,938	–	–	–	257,938
Task 2. Install Riparian Vegetation Improvements											
Product 2. Grazing/Rangeland/ Riparian Management	121,655	19,751	–	–	200,000	–	787,925	–	–	–	1,129,331
Objective 2. Conduct Public Outreach, Complete Essential Water Quality Monitoring, and Write Reports											
Task 3. Conduct Public Outreach Program, Monitor Water Quality and Write Reports											
Product 3. Public Education and Outreach, Monitor Water Quality, Write Reports	–	–	–	–	–	–	–	16,250	–	202,479	218,729
Other Watershed Improvement Projects	–	–	42,250	200,000	–	85,000	–	–	8,903	–	336,153
Total	**121,655**	**19,751**	**42,250**	**200,000**	**200,000**	**85,000**	**1,045,863**	**16,250**	**353,903**	**202,479**	**2,287,151**

8.0 FUTURE ACTIVITY RECOMMENDATIONS

During the next 5 years, additional projects segments are planned to finish installing the BMPs outlined in the *Phase I Watershed Assessment Final Report and TMDL* [Hoyer and Larson, 2004] and the *Ten-Year Belle Fourche River Watershed Strategic Implementation Plan* [Hoyer, 2005]. This will ensure that the overall goal for the watershed is met, which is to bring the Belle Fourche River and Horse Creek into compliance with state TSS standards. As additional TMDLs are completed for other lakes and tributaries in the watershed, implementation of TMDLs developed should be added to the Belle Fourche River Watershed project.

9.0 REFERENCES

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Rolland, C. and D. P. Hoyer, 2005. *Belle Fourche Irrigation District Water Conservation Plan*, RSI-1824, prepared by RESPEC, Rapid City, SD, for Belle Fourche Irrigation District, Newell, SD.

Hoyer, D. P., 2005. *Ten-Year Belle Fourche River Watershed Strategic Implementation Plan*, RSI-1821, prepared by RESPEC, Rapid City, SD, for Belle Fourche Irrigation District, Newell, SD.

APPENDIX A

HYDRAULIC MODEL OF THE BELLE FOURCHE IRRIGATION DISTRICT NORTH CANAL AND AUTOMATED CHECK STRUCTURE OPERATIONAL CURVES AND CHARTS

Hydraulic Model of the Belle Fourche Irrigation District North Canal and Automated Check Structure Operational Curves and Charts

by

Lacy Marie Pomarleau

A thesis submitted to the Graduate Division
in partial fulfillment of the requirements for the degree of

Master of Science in Civil Engineering

South Dakota School of Mines and Technology
Rapid City, South Dakota

2008

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Approved by:

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Dr. Scott Kenner, Major Professor

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Dr. Henry Mott, Chair, Department of Civil and Environmental Engineering

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Dr. John Helsdon, Dean, Graduate Division

Abstract

The Belle Fourche River total suspended solids (TSS) total maximum daily load (TMDL) study, sponsored by the Belle Fourche River Watershed Partnership, found that irrigation return flows were one of several main contributors of TSS. It was determined that the TSS exceedance in the Belle Fourche River could be improved by reducing nonused irrigation return flows from the Belle Fourche Irrigation District (BFID), which contributed approximately 20 percent of the TSS loading in the Belle Fourche River. A set of best management practices was developed to bring the Belle Fourche River back into compliance; one of these includes the development and implementation of a hydraulic model of the BFID.

The overall goal of this research was to increase the operational efficiency of the BFID by creating a hydraulic model of the North Canal supplemented by operational curves, charts, and spreadsheets developed for key check structures. Operational curves, charts, and spreadsheets were developed for key automated checks based on field measurements which included gate openings, manual weir settings, upstream and downstream water levels, and discharge, which was measured using the Acoustic Doppler Current Profiler (ADCP). Based on these field measurements, discharges were calculated using weir and orifice discharge equations. The orifice and weir discharge coefficients were adjusted in the weir and orifice equations until the sum of the calculated discharge matched the discharge measured in the field. The discharge coefficients that minimized the error between the calculated flows and the measured flows were used to develop charts, curves and spreadsheets. These tools can be used to optimally operate automated check structures, turn automated checks into flow measuring devices, increase the understanding of BFID operations, and ultimately increase the operational efficiency of the BFID.

A North Canal model was developed using EPA SWMM for the entire North Canal. The North Canal was divided into four reaches. Of these, the first three were calibrated based on recorded water orders, field measurements collected during the 2007 and 2008 irrigation seasons, field dataloggers, and rating curves developed from the field measurements. Various methods of simulating automated gates at check structures were investigated. Due to limited field data and time constraints, the North Canal model and the operational curves/charts were not validated. The error between the simulated results and the measured values was minimized to ±10 percent at the majority of the check structures. Differences between the observed data and the simulated results are estimated to be primarily due to uncertainty in water orders, water deliveries, insufficient physical characteristics of the canal, and assumptions pertaining to manual structure adjustments. The model is fully capable of simulating the entire BFID irrigation system if the appropriate amount of data is collected. The BFID can use this model as a tool for ditch-rider training and for understanding the complexities of the North Canal, and as a decision-making tool concerning system operation and structure adjustments. By using the developed SWMM model and the operational curves, charts, and spreadsheets, the BFID could reduce non-used irrigation return flows by improving the operational efficiency, which would in turn reduce the TSS in the Belle Fourche River.

Acknowledgements

First and foremost, I would like to extend my deepest appreciation to Dr. Scott Kenner for generously offering me an opportunity to come to Rapid City. It was his contagious enthusiasm for water resources and his steady guidance that directed me to be the Water Resource Engineer that I am today. His mentoring is priceless and can be replaced by no other. He taught me to never substitute for less than the best and to always ask myself what I can do to make my projects better. Along with Dr. Kenner's support I would like to thank the employees at RESPEC, with particular thanks to Mr. Jared Oswald for sharing his expertise in project management and irrigation. His calm personality always made it easy to ask questions and understand new concepts. I would also like to thank my current committee members, Dr. Arden Davis and Dr. Thomas Fontaine, for supporting my research and educational efforts, as well as Dr. Dan Hoyer, a former RESPEC employee and committee member.

There have been many project partners that have helped provide me with background information and field assistance. Mr. Tim Olson and Mr. Curt Shoenfelder both passed on endless knowledge that they gained in the Belle Fourche Irrigation District (BFID). One point of advice that has helped students following in their path was to always bring coffee to the District Manager to keep him smiling and cookies to BFID personnel if you happen to get stuck and need their help. Mr. Shoenfelder also provided exceptional field assistance during the 2006 irrigation season as well as SWMM advice after he had started his career in Washington. His easygoing personality made my work even more enjoyable. Other students that I owe appreciation to are Mr. Jeremy Sanson, Mr. Brett Morlok, and Ms. Quana Higgins. Mr. Sanson helped me collect field data in

2007 and understand the development of operational charts for check structures. Mr. Brett Morlok and Ms. Quana Higgins collected field data during the 2008 irrigation season which helped finish my research. Mr. "MacGuyver" Morlok's ideas and contraptions always seized to amaze me. I also would like to thank my many other friends that I acquired during my time at SDSM&T. Their friendships made graduate life much easier.

I would like to applaud the BFID personnel on their generosity, cooperation, and open-mindedness, which ultimately allowed for a successful project. In particular, Mr. Clint Pitts and Mr. Bill Anderson were exceptional at teaching a city girl everything she needed to know about irrigation and farming operations. Although Mr. Pitts' North Dakota jokes were usually less than appealing, they always have made me shake my head and smile. Mr. Anderson was my "go to guy" at the BFID. His wealth of knowledge and ability to explain things in a down to earth matter is not only valued by myself, but by other students that I have sent his way. My appreciation also goes out to the Belle Fourche River Watershed Partnership for providing funding and support for this project.

I would like to thank my family and friends who may not have understood what I was talking about most of the time, but always nodded their heads and supported me. I am forever grateful to my parents, Tony and Margie Metz who raised me to be a self-starter and always said they were proud of me. Without them I would not be where I am today. And finally, I would like to shout "Thank You" to my loving husband, Matthew, for holding my head out of the water for the last eight years. I admire him for always supporting his poor college girl. He is my pedestal and rock that I stand on.

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1.0 Introduction

1.1 Project Background

1.1.a Belle Fourche River Watershed

The Belle Fourche River watershed is composed of over 5 million acres of land located in Wyoming, South Dakota and Montana (Figure 1). Approximately 40 percent of the watershed is located in South Dakota. Land use is primarily livestock grazing, cropland and a few urban and sub-urban areas (Belle Fourche River Watershed Partnership, 2005). The Belle Fourche River is one of the major tributaries of the watershed. The Belle Fourche River runs through Keyhole Reservoir in Wyoming and into South Dakota where it meets the Cheyenne River. Prior to the Cheyenne River, the Belle Fourche River receives non-used irrigation return flows produced by the Belle Fourche Irrigation District (BFID) which is located within the watershed (Figure 1).

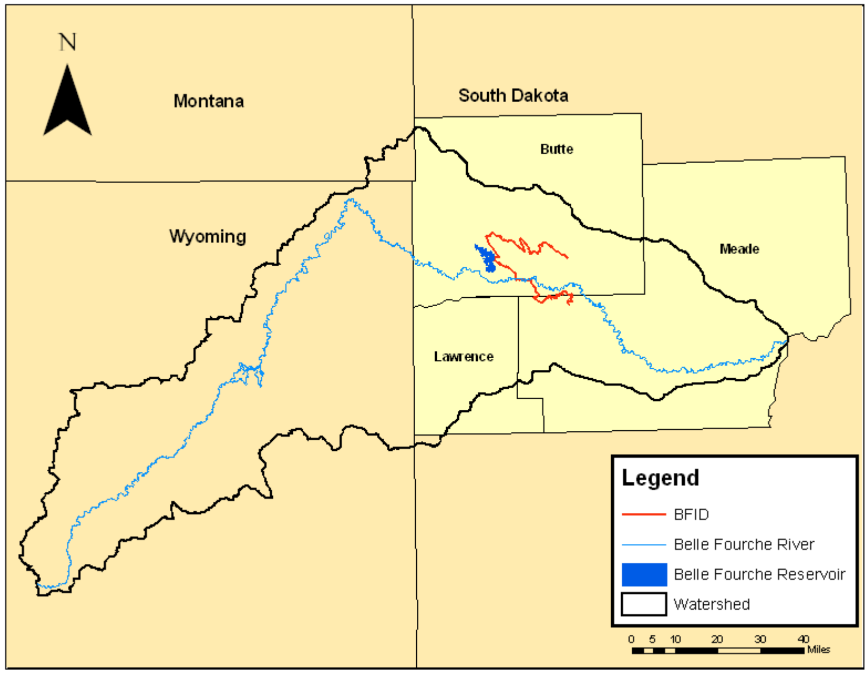

Figure 1. Belle Fourche Watershed and Belle Fourche Irrigation District (Olson, 2006).

1.1.b Belle Fourche Irrigation District

The BFID provides irrigation services to 57,183 acres in southwestern South Dakota for the U.S. Bureau of Reclamation (BOR). Alfalfa and hay make up approximately 65% of the crop production in the BFID. The remaining crop land is composed of small grains and corn. There is also some livestock and dairy production within the BFID. The soil in the northern region of the BFID consists of heavy clay with some silt and gravel, whereas the southern region of the BFID consists of clay/sand soils (Rolland, 2005). The average yearly water allocation to the BFID is approximately 15 inches, which is twice the amount of water that would be received from the average precipitation of the watershed (Belle Fourche River Watershed Partnership, 2005). The water allotment is the total available water that can be distributed to each farmer based on the capacity of the Belle Fourche Reservoir.

1.1.c BFID Flood Irrigation

Most of the irrigation in the BFID is done by means of surface flooding (or flood irrigation) with some sprinkler or pivot irrigation. Flood irrigation consists of flowing water over a field by means of gravity. This type of irrigation is typically used where water is inexpensive because of the inefficiency of flood irrigation. Roughly 64 percent of the water released onto the BFID is delivered to the field. The remaining 36 percent is consumed by transportation losses (seepage and evaporation) and operational losses (Belle Fourche River Watershed Partnership, 2005). Operational losses include overflow from the canals, laterals, and gates or valves into adjacent waterways (Schoenfelder, 2006). The crops use approximately 32 percent of the water that is delivered to the field. The other 68 percent (of the water delivered to the field) is either evaporated or becomes nonused water that runs to adjacent waterways and is considered nonused irrigation return flows (Schoenfelder, 2006).

Sediment mobilization occurs as a result of flood irrigation. As described by Schoenfelder (2006), there are three different processes that can cause sediments to become mobilized during flood irrigation. The sediments are mobilized (1) as tail water/runoff crosses the field, (2) in the canals and laterals, and (3) in the intermittent streams carrying tail water/runoff to the perennial streams within the watershed. Flood irrigation can adversely affect the watershed due to low efficiency and high sediment loading.

1.1.d Belle Fourche River Impairment

The *1998 South Dakota 303(d) Waterbody List* (Meyers, 2005), the *2002 South Dakota 303(d) Waterbody List* (Pirner, 2005), and the *2004 Integrated Report for Water Quality Assessment* (South Dakota Department of Environmental and Natural Resources, 2003) all listed the Belle Fourche River as impaired due to elevated total suspended solids (TSS) concentrations (Belle Fourche river Watershed Partnership, 2005). The Belle Fourche River watershed total maximum daily load (TMDL) study (Hoyer and Larson, 2004) identified the following key points. The Belle Fourche River TSS impairment is primarily due to natural bank sloughing, the quantity of nonused irrigation water discharged to the natural waterways, and riparian habitat impairment. Seventy-five percent of the TSS in the Belle Fourche River is a consequence of stream entrenchment and bank failure in the eastern portion of the watershed. This area is composed of high clay banks that are erodible. When erosion occurs, suspended solids are delivered to the channel. Nonused irrigation return flows increase the quantity of water in the channels and are the major driver causing channel incising, bank failures and, therefore, suspended solids. Irrigation return flows are responsible for approximately 20 percent of the TSS in the Belle Fourche River system.

The Ten-Year Implementation Plan (Hoyer and Schwickerath, 2005) recommended several best management practices (BMPs) with the goal of bringing the Belle Fourche River into TSS compliance. The BMPs are defined in *Segment III of the Belle Fourche River Watershed Management and Project Implementation Plan* (Belle Fourche River Watershed Partnership, 2005). One of the BMPs recommended was to improve the delivery and application efficiency of irrigation waters by the development and implementation of an operation model for the BFID. The operation model (Olson, 2006) has a hydraulic model component. The focus of this paper will be on the development and applications of the North Canal hydraulic model as well as automated check operational curves that can be used to improve the operation efficiency of the BFID. By improving the operational efficiency of the BFID, the amount of non-used irrigation return flows will be decreased which will reduce the TSS concentration in the Belle Fourche River.

1.2 System Characteristics

1.2.a Canals and Laterals

The BFID consists of three main canals: the inlet canal, the North Canal and the South Canal. The inlet canal carries water that is diverted from the Belle Fourche River into the Belle Fourche Reservoir (also known as Orman dam). Water that is stored in the Belle Fourche Reservoir is distributed to the fields by the North and South Canals (Figure 2). The South Canal is 44 miles long with a design capacity at the dam outlet of 400 cubic feet per second (cfs). The North Canal is 43 miles long and has a 600 cfs design capacity at the dam. In addition to the three major canals, there is also a network of approximately 450 miles of smaller canals, also known as laterals, which deliver water to fields. Water is released to laterals and fields through structures known as turnouts. Turnouts are typically on the side of the canal, and can be opened or closed by a headgate to regulate the discharge flow rate (ASCE, 1987). Farmer turnouts typically deliver water to a single farmer’s field off of a main canal or lateral.

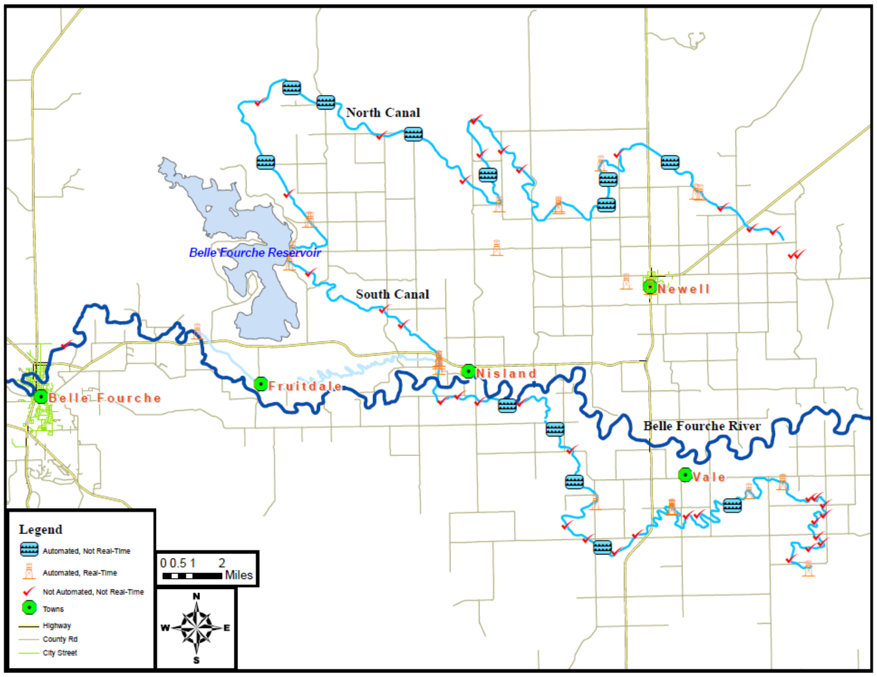

Figure 2. Illustration of key structures located on the BFID North and South Canals.

1.2.b Check Structures

A series of check structures are located along the main canals and laterals (Figure 2). The water located in the portion of canal between two check structures is known as the pool (ASCE, 1987). A check structure acts like a small dam that builds up the upstream pool level in order to produce the required delivery head at each of the turnouts upstream of the check. Check structures are installed in canals to maintain a constant upstream water level (or target depth), regardless of the flow rate through the structure (ASCE, 1987). A check structure typically consists of a series of undershot sluice gates (or orifices), adjustable boards (or weirs) and overflow weirs (or automatic weirs also referred to as auto's) (Figure 3). Sluice gates are adjusted to allow water to flow under the gate, whereas adjustable weirs allow flow to overtop the boards. Weirs are adjusted by either adding or taking out boards. The overflow weirs were designed for emergency use. When the water rises over the design water surface, water will overtop the overflow weirs. Typically, gates and weirs are manually adjusted by a ditch rider (system operator) to maintain a constant upstream pool level as fluctuations in flow pass through the structure.

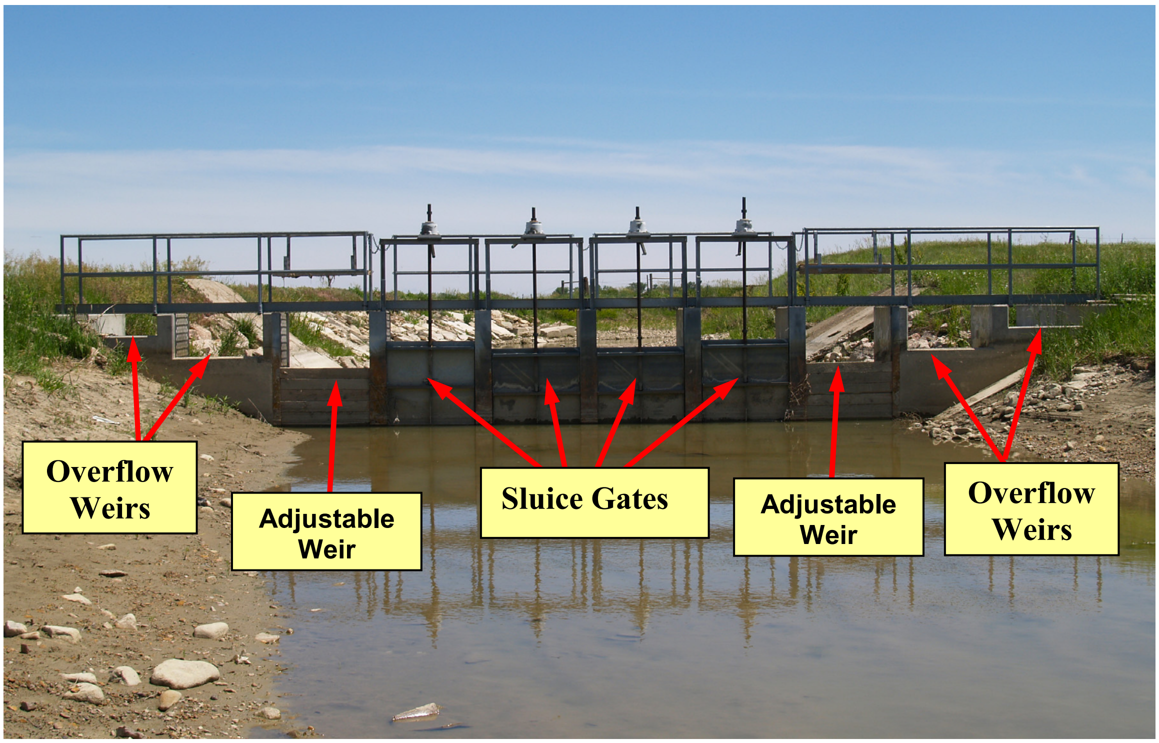

Figure 3. Typical unautomated check structure consisting of overflow weirs, adjustable weirs and sluice gates.

1.2.c Flow Measurement

Flow measurement is a key component in the decision making process of the BFID. Each ditch rider is responsible for checks, laterals, and farmer turnouts along a certain section of the BFID. These sections are called rides. Typically, there is a flow measuring device at the beginning and at the end of each ride. Discharge is monitored throughout the BFID using parshall flumes, sharp-crested weirs, cippolleti weirs, constant-head orifices (CHO's) and diversion box weirs. Measuring devices are located at the end of each ride on the major canals, laterals and some farmer turnouts. Laterals and turnouts without a downstream weir box are monitored using a portable flow meter or the ditch rider's best judgment. The amount of flow discharged into large laterals is often measured using a constant-head orifice (CHO). A CHO is a water measuring device frequently used in irrigation as a combination regulating gate and measuring gate structure (BOR, 2001). This device uses an adjustable rectangular gate opening as a submerged orifice for discharge measurement and a less expensive circular gate downstream. It is operated by setting and maintaining a constant head differential across the orifice (BOR, 2001). Discharges are set and varied by changing the gate openings.

1.3 System Automation

According to Segment III of the Watershed Implementation Plan, 25 flow automation units were scheduled to be installed on the BFID during the 2005 and 2006 irrigation seasons to improve the irrigation delivery system (Belle Fourche River Watershed Partnership, 2005). The types and description of automated equipment are defined by Olson (2006).

Check structures were automated by installing equipment that would control the pool level upstream of the structure. The purpose of the automated gate is to maintain a constant upstream pool level which allows for continuous and consistent flow releases to the upstream laterals and farmer turnouts. Without automation, the constant variability of the pool level (and hence the head pressure at laterals and turnouts) can consume water meant for downstream orders. It can also deprive upstream farmers of ordered water if the pool level is too low.

Check automation in the BFID consists of only one gate being automated (all of the other weirs and gates must be adjusted manually). Initially, an upstream target level is entered into a datalogger by a BFID technician. A pressure transducer (PT) located on the upstream side of the check then reads the actual water level. Every ten minutes, the upstream water level is averaged. If the average actual water level differs from the target level, a signal is sent to the automated gate telling it to either raise or lower in order to obtain the target level. An automated gate can only account for fluctuations of approximately 50 cfs. Optimally, the automated gate should be set at 50% open in order to give it the largest range of motion. If the automated gate begins to near 100 percent open or closed, the manual gates and weirs (also known as the manual settings) must be adjusted by an operator to give the automated gate the maximum range of motion and allow for the proper operation of the check.

Some of the check structures were also set up to be “real-time” sites. This means that the data is sent directly to the district office by means of a radio tower. This allows management to see the gate position and water level occurring in the field. If a gate is getting maxed out, a ditch rider can be sent out to adjust the manual settings. Real-time sites also allow district personnel to adjust the target level, gate position and detect mechanical problems without being in the field.

Flumes, weirs, and CHO’s located along the main canals and laterals were also automated to be real-time with dataloggers, PT’s, and radio towers. This allows the BFID to access the flow discharges at major flow structures remotely from the district office (or a mobile base station) by means of a computer. Automation decreases the amount of manpower that is needed to operate the system and allows for more accurate deliveries which improves the operational efficiency of the BFID.

1.4 North Canal Background

The North Canal (Figure 4) is a gravity fed system that has a 600 cfs capacity at the dam. Natural unlined open channels with a design side slope of 2:1 compose the majority of the canal. The North Canal consists of 26 check structures: 11 automated and 15 unautomated. Along the 43 mile canal, there are 3 siphons, approximately 20 bridges/road crossings, and 120 farmer turnouts and lateral head gates. Indian Creek Lateral and Deer Creek Lateral are two major laterals that are controlled by automated CHO's. Flow is measured along the canal with the aide of two parshall flumes and one cippolletti weir. The flow measuring devices are located at the beginning and end of each ride.

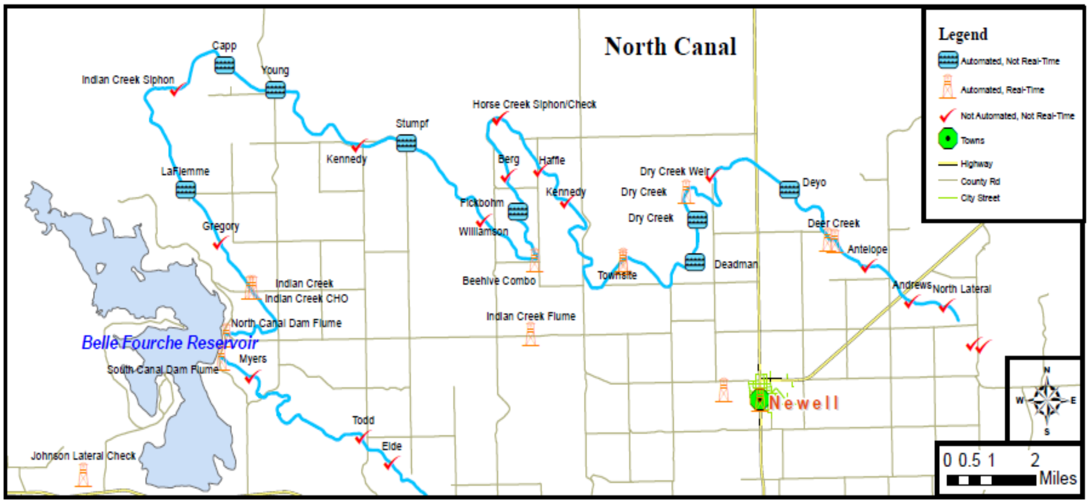

Figure 4. Illustration of key structures along the North Canal.

1.5 System Operation

The operation of the BFID was described in detail by Olson (2006). He states that the operation of the BFID is governed by a series of dependant components. The three components to the demand/delivery system are: water call cards, Water Master sheets, and billing cards. The water call cards link the farmer and the ditch riders. They include the amount of water needed for proper delivery and system operation which accounts for the farmer's water order, operational and transportation losses. The Water Master sheets include the total water orders summed from all of the water call cards and are used to decide what changes need to be made at the dam on a daily basis. The billing cards include the amount of water allocated and the total water delivered to each farmer.

Olson (2006) also described the operational processes that occur on a daily basis. These processes are a combination of human interaction and information transfer. The process begins with water orders and ends with delivery of water to the farmers. The following processes and interactions occur daily (Olson, 2006):

1. ***Farmer/Ditch Rider****:* Ditch rider records the amount of water ordered by the farmers onto the water call card.
2. ***Ditch Rider/Data Entry:*** Data from the water call card is entered into a database known as the real-time irrigation system. The real-time irrigation database calculates the total water ordered along with the transportation and operation losses and creates the Water Master sheet.
3. ***Data Entry/Water Master:*** Every morning at approximately 9:00 am, the Water Master analyzes the Water Master sheet and makes the necessary adjustments at the dam.
4. ***Data Entry/Ditch Rider:*** A check structure demand schedule is produced using the water call cards. The irrigation demands upstream of each check, along with available flow measurements, are used to make decisions about the system operation.
5. ***Ditch Rider/Farmer:*** The ditch rider tells the farmer when his water will be delivered based on travel time resulting from the appropriate headgates distance from the dam. The ditch rider releases water to the farmer turnouts and laterals when it is available. The farmer makes changes to water orders if necessary, and the process repeats.

1.6 Previous BFID Research

Previous hydraulic modeling on the BFID was focused on the South Canal. Two different types of hydraulic modeling software were initially considered for the BFID: U.S. Environmental Protection Agency's Storm Water Management Model Version 5.0 (EPA SWMM 5.0) (EPA, 2008) and RootCanal (Utah State University). An investigation of the hydraulic models along with a comparison of trials on the South Canal was previously conducted by Rolland (2005). Issues being compared included initial steady state conditions, simulation time, modeling turnouts, modeling gates, weir and gate equations used, time series entry, and modeling check structure automated gates. Both models had advantages and disadvantages.

Both models produced fairly similar results and conclusions when they were applied to the first eight miles of the South Canal (Rolland, 2005). The initial results showed that the time required for a change in discharge at the Dam to reach the Belle Fourche River Siphon Flume was longer than originally assumed by the BFID. Rolland concluded that the problem of "missing water" could simply be that the water had not arrived yet. Although RootCanal was being developed specifically for irrigation purposes, it was still in the development phase. Therefore, Rolland found that SWMM would be more appropriate for the BFID because of its greater capabilities for unsteady flow computations and its reliability. Rolland's research was also focused on determining gate and weir discharge coefficients at the Vale Check on the South Canal. The gate and weir discharge coefficients were determined to be 0.65 and 3.0, respectively.

A hydraulic model of the South Canal was completed by Schoenfelder (2006) using EPA SWMM 5.0 as per Rolland's (2005) recommendations. The model was focused on improving the operational efficiency of the BFID. During the 2006 validation period of the South Canal model, 94% of the simulated depths were within ±10% of the observed depth, 58% within ±5%, and 40% within ±2.5%. The model proved to be fully capable of simulating the entire BFID irrigation system and all of its structural components, including automated check structure gates (Schoenfelder, 2006). Schoenfelder (2006) explained the assumptions that were made for the South Canal model and recommended various ways to improve the hydraulic model. These assumptions and recommendations were followed while modeling the North Canal.

Sanson (2008) developed operational curves and charts for the Fickbohm check (North Canal) and the Simmons check (South Canal), both being unautomated checks. A model was developed to predict the flow passing through the check structures using stage, flow, and structure setting data collected during the 2007 irrigation season. The model predicted the flow for the Simmons Check between -5.2% and +12.2% with an average percent difference of ±2.7% and for the Fickbohm Check between -3.7% and +4.1% with an average percent difference of ±2.2%. The operational charts developed in Sanson's (2008) research can be used to improve the operation of the study reaches and other unautomated reaches.

1.7 Objectives

The overall goal of the Belle Fourche River Watershed Management and Project Implementation Plan is to bring the Belle Fourche River back into TSS compliance. One way that this goal will be accomplished is to reduce the nonused irrigation return flow by increasing the operational efficiency of the BFID. By doing this, approximately 37 percent of the overall TSS reduction will be achieved. The objectives of this research are as follows:

- Develop a hydraulic model using EPA SWMM 5.0 that will simulate many possible flow scenarios which can be used as a decision making tool for the BFID.
- Develop Operational curves for key automated check structures to calibrate the North Canal hydraulic model. The Operational curves will also provide additional flow measurements for the BFID which will aide in the decision making process and system operation.

1.8 Scope and Approach

This research is directed towards the North Canal of the BFID. It is intended to provide a part of the hydraulic component of the operational model of the BFID in hopes of increasing the operational efficiency of the BFID. By increasing the operational efficiency, the nonused irrigation return flows will be decreased resulting in a reduction of the TSS levels in the Belle Fourche River. The hydraulic model, along with the operational curves, will provide tools to analyze, predict, and assess operational changes and their effects throughout the system (Schoenfelder, 2006).

The hydraulic model was developed for the entire 43 miles of the North Canal. Calibration efforts were focused on the canal from the Dam outlet to the Dry Creek Weir. The model was calibrated using field measurements and automated check operational curves that were developed. This report will include a discussion on the hydraulic model development, calibration, a sensitivity analysis, applications of the hydraulic model, and recommendations for future modeling efforts. It will also address automated check operational curves including the development process, calculations of weir and gate discharge coefficients, and applications of the operational curves.

2.0 Automated Check Operational Curves and Charts

2.1 Development Process

The objective of this process was to create a set of curves for a specified stage that would relate the automatic gate percent opening to the amount of flow in the channel at various manual settings. These curves could then be used to calibrate the SWMM model where permanent flow measuring devices were not available. In addition to calibrating the SWMM model, these curves could also be used during the irrigation season as additional flow measuring devices which would aide in the management and operation of the BFID. Five key automated checks were chosen to be analyzed which include: Indian Creek check, Young check, Stumph check, Beehive check, and Townsite check (Figure 5). The development of operational charts for unautomated checks was described by Sanson (2008). Sanson's findings were used as a basis for the development of the automated check operational curves.

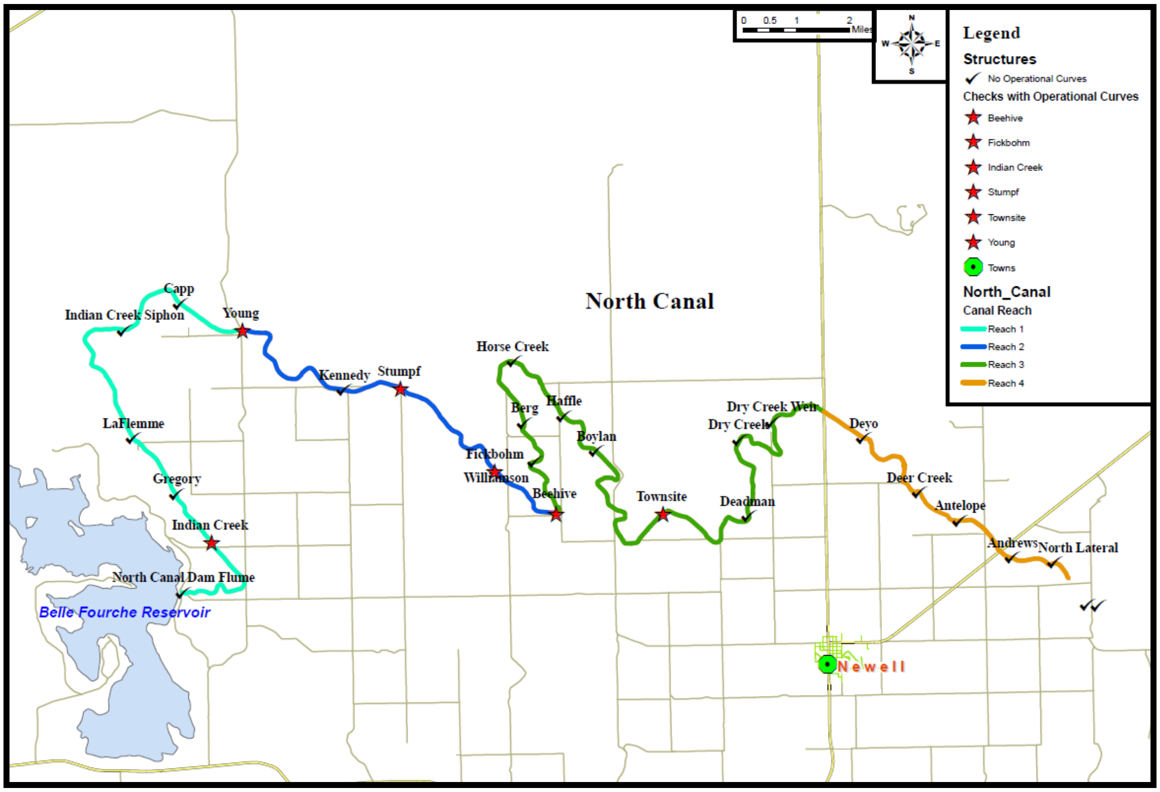

Figure 5. Check locations on the North Canal with Operational Curves.

2.1.a Data Collection

During the summer of 2008, flow measurements were collected at key automated checks using an acoustic doppler current profiler (ADCP) (Figure 6). The ADCP uses sound waves to detect the movement of dissolved material in water which correlates to the velocity of the water. It is also capable of measuring the depth of the water. With these measurements, it outputs the calculated flow. The ADCP was used opposed to a Marsh-McBirney flow meter because the depth of water and/or high velocities at most of the checks made the channel unwadable. The ADCP was also favored because it has the capability of measuring flow within ± 4 percent in a short amount of time. It also requires a minimum of four flow measurements which minimizes measurement errors.

Along with flow measurements, the corresponding check settings including the automatic gate position (recorded as percent open), were collected. Check settings were collected prior to and succeeding the measurement of flow in order to account for any fluctuations in stage or the automatic gate position that may have occurred while flow was being measured. Since the checks being analyzed are automated, the upstream stage should be constant throughout the summer (±0.05 ft). Any deviations that could alter the effectiveness of the operational curves were recorded including: variable upstream stage, weir submergence, automatic gate out of water, and automatic gate 100 percent open or closed.

The ADCP was purchased and delivered to SDSMT in mid July, 2007. For this reason, measurements were primarily collected during the last part of July and the month of August. During this time frame, the flow in the North Canal was at its peak and going into recession for the season. Low flow conditions were not detected at most of the checks due to time constraints. The checks that were monitored included the Indian Creek check, Young check, Stumph check, Beehive check and the Townsite check.

Figure 6. Using the Acoustic Doppler Current Profiler.

2.1.b Measurement Errors

Errors in measurement of flow and check settings were inevitable. The water surface to the top of the concrete, gate stem heights, and weir settings were measured using a tape measure and/or a survey rod. Although care was taken in obtaining data, human errors in reading measurements along with the natural fluctuations in the water surface due to waves and wind, create variability in the data collected at the checks. Collecting the flow measurements with the ADCP took, on average, about 30 minutes. During this time, a minimum of four flow measurements were taken. For this reason, only the average flow that was passing through the channel during the time of measurement could be collected. Weeds and other large debris could have altered the data collected by the ADCP. The operational curves will need to be validated with more measurements in coming seasons to confirm their effectiveness.

2.1.c Model Development: Discharge Coefficients

In order to develop a series of operational curves for each check, a spreadsheet model was created. The model calculated flow through each of the orifices and weirs using Equation 1 and Equation 2 (Gupta, 2001):

$$Q_{gate} = Cd_g * a * b * \sqrt{2gh}$$ **Equation 1**

Where:

Q_{gate} = Flow under the gate (cfs)

a = sluice gate opening (ft)

b = sluice gate width (ft)

g = gravitation constant (32.2 ft/s)

h = upstream head (from the centroid of the gate opening) (ft)

Cd_g = discharge coefficient of sluice gate (unit less)

$$Q_{weir} = Cd_w * L * h^{1.5}$$ **Equation 2**

Where:

Q_{weir} = Discharge over the weir (cfs)

L = length of weir (ft)

h = head over the weir (ft)

Cd_w = discharge coefficient of weir (unit less)

The total flow was estimated by the model as the sum of the flow through all of the orifices and weirs as shown in Equation 3.

$$Q_{total} = \sum Q_{weirs} + \sum Q_{gates} \qquad \textbf{Equation 3}$$

An initial gate discharge coefficient and weir discharge coefficient was assumed. The most appropriate gate and weir discharge coefficients for each check were calculated by minimizing the explained error in the regression through an iterative process. The explained error (also known as the residual) was calculated by finding the sum of the square of the error (SSE) as shown in Equation 4 (Montgomery and Runger, 2007). Microsoft Excel's solver was used to minimize the SSE of the model by changing both the weir and gate discharge coefficients as was recommended by Sanson (2008). The final discharge coefficients that were found to best represent the flow for each check can be seen in Table 1.

$$SSE = \sum_{i=1}^{n} \left(y_i - \hat{y}_i \right)^2 \qquad \textbf{Equation 4}$$

Where:

SSE = sum of squares of the errors

y_i = measured flows (cfs)

$\hat{y}_i$ = predicted (modeled) flows (cfs)

Table 1. Weir and gate discharge coefficients developed by the model.

Check	Cd-gates	Cd-weirs	SSE
Indian Creek	0.462	4.156	337
Young	0.484	3.417	60
Stumph	0.501	3.274	183
Beehive	0.168	3.438	174
Townsite	0.399	1.577	75

Some data points were eliminated from the analysis for various reasons including: gate above the water surface, automated gate one hundred percent open or closed, variable target levels, and faulty data collected by the ADCP due to mechanical errors.

2.1.d Degree of Submergence and Discharge Coefficients

The variability in the discharge coefficients could be related to downstream backwater effects which typically increase with distance from the dam. Backwater effects vary from check to check. The degree of submergence of the check structure was calculated by dividing the downstream water depth (stage) by the upstream water depth. Table 2 shows that as the degree of submergence increases, the gate discharge coefficient decreases. The Beehive check had the smallest average difference between upstream and downstream stage and the highest degree of submergence. Townsite check also had backwater effects but not to the extent of the Beehive check. This shows an inverse relationship between the amount of backwater and the discharge coefficient. As backwater effects increase, the gate discharge coefficient decreases.

Table 2: Effects of submergence on discharge coefficients

Check	Cd-gates	Cd-weirs	Stage Difference	Degree of Submergence
Indian Creek	0.462	4.156	2.51	0.47
Young	0.484	3.417	2.44	0.56
Stumph	0.501	3.274	2.24	0.5
Beehive	0.168	3.438	0.74	0.86
Townsite	0.399	1.577	0.96	0.72

2.2 Model Results

Overall, the model predicted the measured flow reasonably well. Comparisons of the measured flow to the predicted flow estimated by the model for each check are shown in Figure 7 to Figure 11. The average percent difference between the predicted flow and the measured flow for each check can be seen in Table 3. The average percent differences ranged from 3.17 to 1.22. The largest difference came from the Townsite check model which only used seven relevant data points. The Young check model resulted in the lowest percent difference and used 9 data points. This shows that more data points could improve the models predictions.

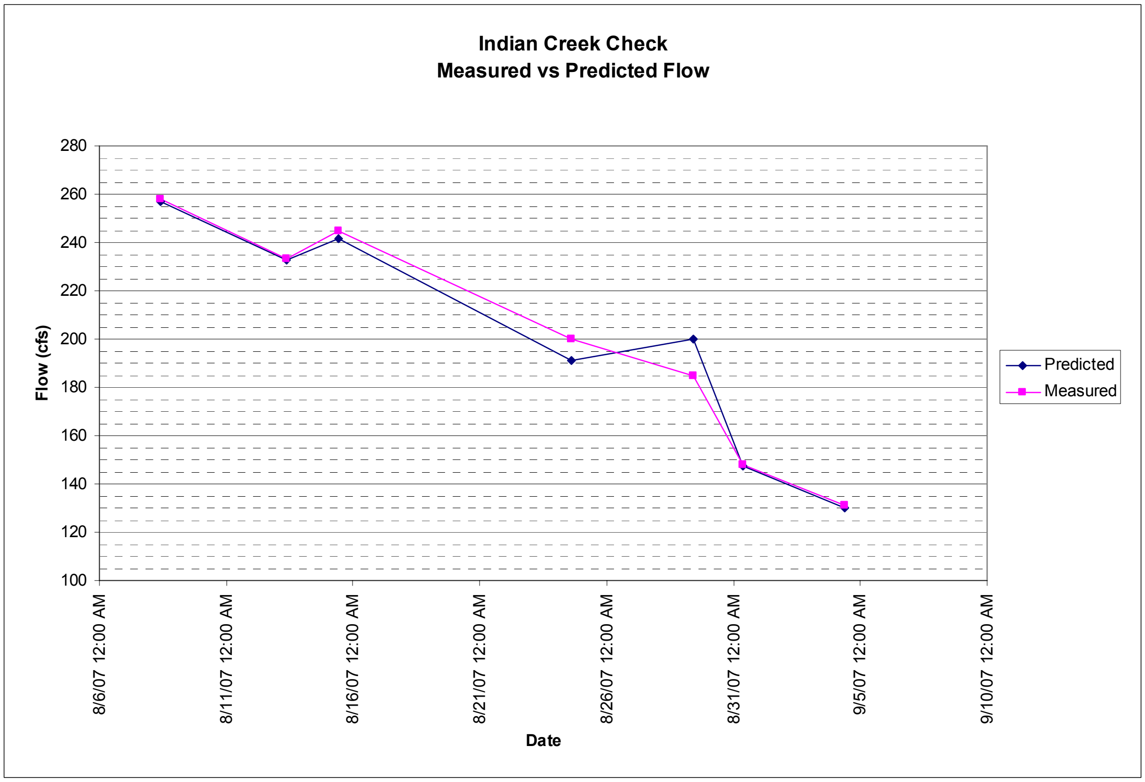

Figure 7. Comparison of measured flow to the predicted flow estimated by the model for the Indian Creek check.

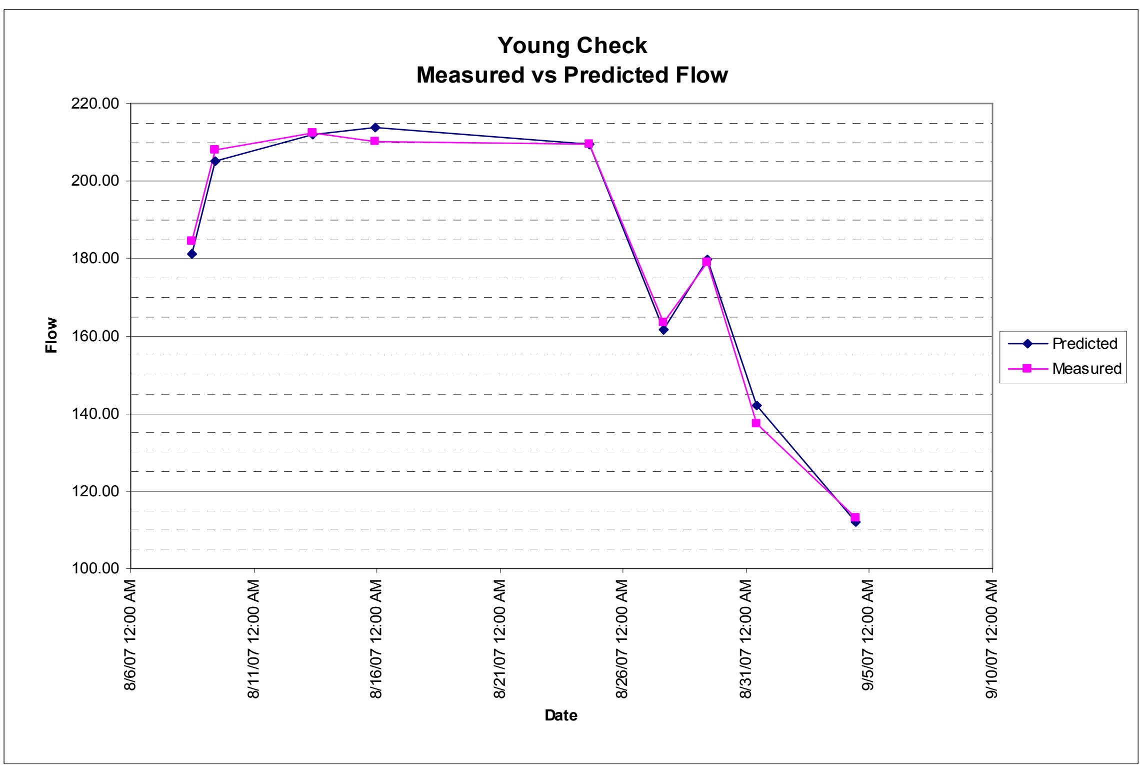

Figure 8. Comparison of measured flow to the predicted flow estimated by the model for the Young check.

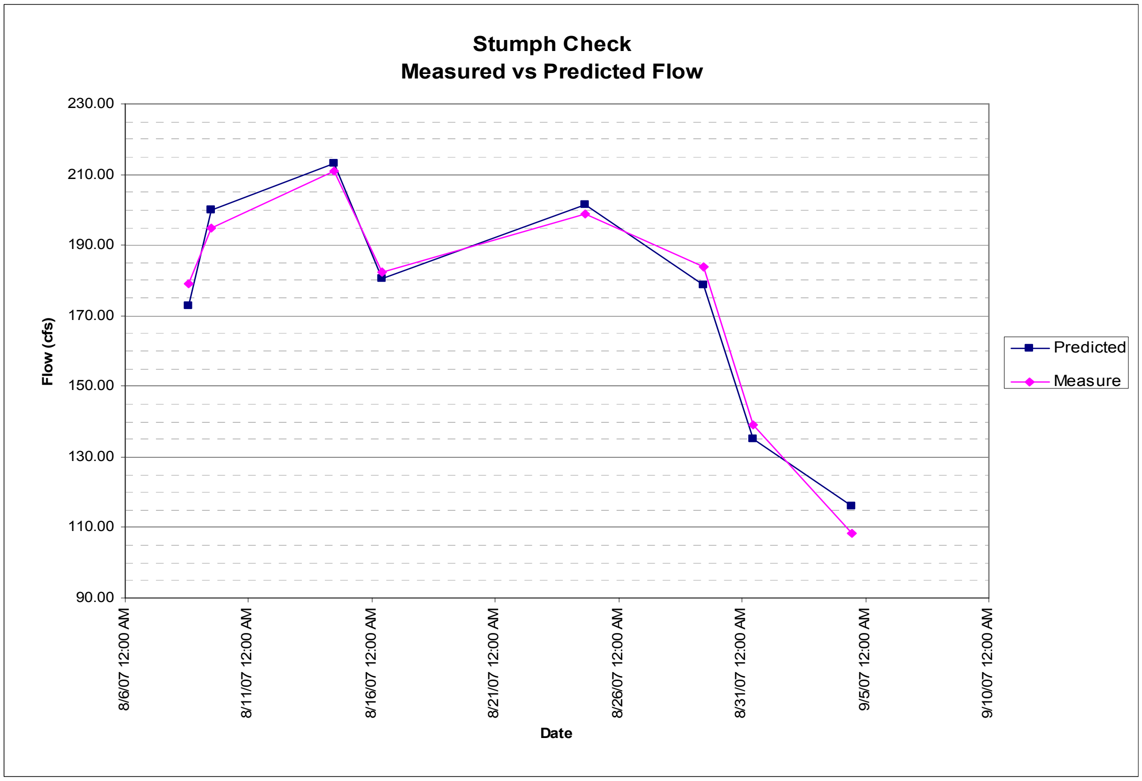

Figure 9. Comparison of measured flow to the predicted flow estimated by the model for the Stumph check.

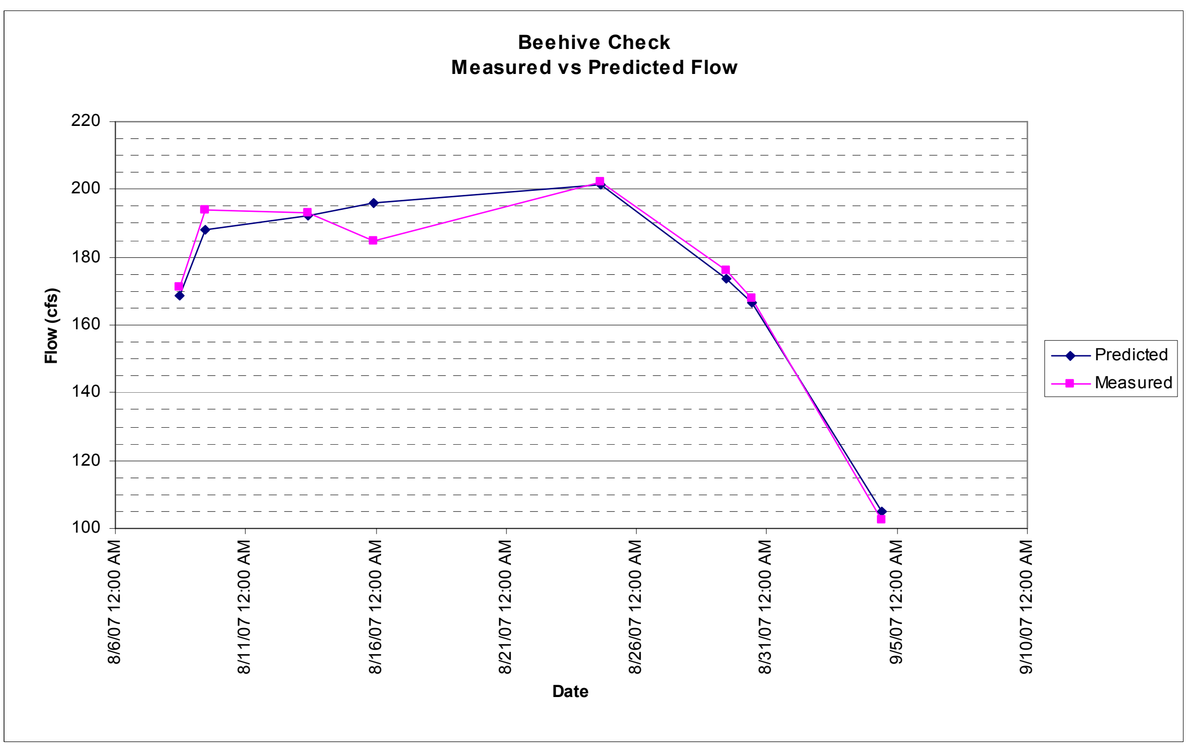

Figure 10. Comparison of measured flow to the predicted flow estimated by the model for the Beehive check.

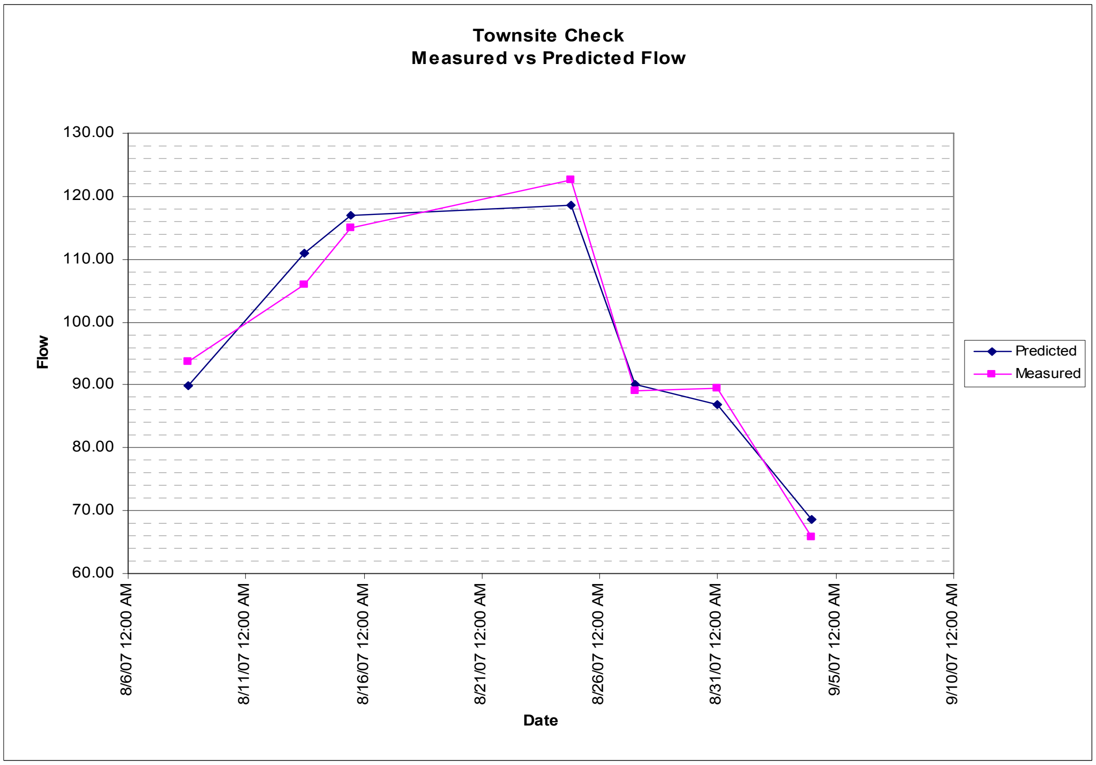

Figure 11. Comparison of measured flow to the predicted flow estimated by the model for the Townsite check.

Table 3. Average percent differences between the measured flow and the predicted flow estimated by the model.

Check	Average Percent Difference	Number of Data Points Collected
Indian Creek	2.27	7
Young	1.22	9
Stumph	2.79	8
Beehive	1.97	8
Townsite	3.17	7

2.3 Development of Operational Curves and Charts

Operational curves were developed for the automated checks using the discharge coefficients found in Table 1. The variability in check settings made it impossible to develop operational curves for each check setting. Rather, a set of operational curves was developed to give the BFID a tool to estimate the optimum check setting for various flow conditions. For instance, the flow at the Young check during the monitoring period ranged from 137 cfs to 212 cfs. One check setting could probably handle this range of flow if there were no abrupt fluctuations in the canal which is unlikely. The automatic gate is used to keep a constant stage. This task is carried out by the gate opening or closing when there are flow fluctuations in the canal. It is optimum to have the gate approximately fifty percent open to allow for the maximum range of motion. For this reason, various different check settings may be needed for this range of flow in order to keep the automatic gate at the most favorable position.

The curves were developed using the same stem height (gate opening) for all of the manual gates. The curves are based on the target level and weir settings that were used during the 2007 irrigation season. The curves are only applicable at these conditions although changes can be made to satisfy any adjustable weir settings, target levels, and manual gate settings. Operation charts were also developed in the same manner. The same data can be obtained from the curves and charts. Developed operational curves and charts are shown in Figure 12 to Figure 16 and Table 4 to Table 8.

Table 4. Operational chart developed for the Indian Creek check. Applicable for: Target level = 3.7' (3 tenths below the overflow weirs), weirs to top of concrete = 4.1'.

Manual Gate Stem Heights (ft)

Automatic Gate Percent Open	0.0'	0.2'	0.4'	0.6'	0.8'	1.0'	1.2'	1.4'	1.6'	1.8'	2'	2.2'	2.4'	2.6'	2.8'	3.0'	3.2'	3.4'	3.6'	3.8'	4.0'
0	80	99	118	136	154	171	188	205	221	236	251	266	280	293	306	319	331	342	353	363	372
5	88	107	126	144	162	179	196	212	228	244	259	273	287	301	314	326	338	350	360	370	380
10	95	114	133	151	169	186	203	220	236	251	266	281	295	308	321	334	346	357	368	378	387
15	102	121	140	158	176	193	210	227	243	258	273	288	302	315	328	341	353	364	375	385	394
20	109	128	147	165	183	200	217	234	250	265	280	295	309	322	335	348	360	371	382	392	401
25	116	135	154	172	190	207	224	240	256	272	287	301	315	329	342	354	366	378	388	399	408
30	122	142	160	178	196	214	230	247	263	278	293	308	322	336	349	361	373	384	395	405	415
35	129	148	166	185	202	220	237	253	269	285	300	314	328	342	355	367	379	390	401	411	421
40	135	154	173	191	209	226	243	259	275	291	306	320	334	348	361	373	385	397	407	417	427
45	141	160	178	197	214	232	249	265	281	297	312	326	340	354	367	379	391	402	413	423	433
50	**146**	**165**	**184**	**202**	**220**	**237**	**254**	**271**	**287**	**302**	**317**	**332**	**346**	**359**	**372**	**385**	**397**	**408**	**419**	**429**	**438**
55	152	171	189	208	225	243	260	276	292	308	323	337	351	365	378	390	402	413	424	434	444
60	157	176	194	213	230	248	265	281	297	313	328	342	356	370	383	395	407	418	429	439	449
65	162	181	199	217	235	253	270	286	302	318	333	347	361	375	388	400	412	423	434	444	454
70	166	185	204	222	240	257	274	291	307	322	337	352	366	379	392	405	417	428	439	449	458
75	170	190	208	226	244	262	278	295	311	326	341	356	370	384	397	409	421	432	443	453	463
80	174	194	212	230	248	266	282	299	315	330	345	360	374	388	401	413	425	436	447	457	467
85	178	197	216	234	252	269	286	303	319	334	349	364	378	391	404	417	429	440	451	461	470
90	182	201	219	238	255	273	290	306	322	338	353	367	381	395	408	420	432	443	454	464	474
95	185	204	223	241	259	276	293	309	325	341	356	370	384	398	411	423	435	447	457	467	477
100	188	207	225	244	261	279	296	312	328	344	359	373	387	401	414	426	438	449	460	470	480

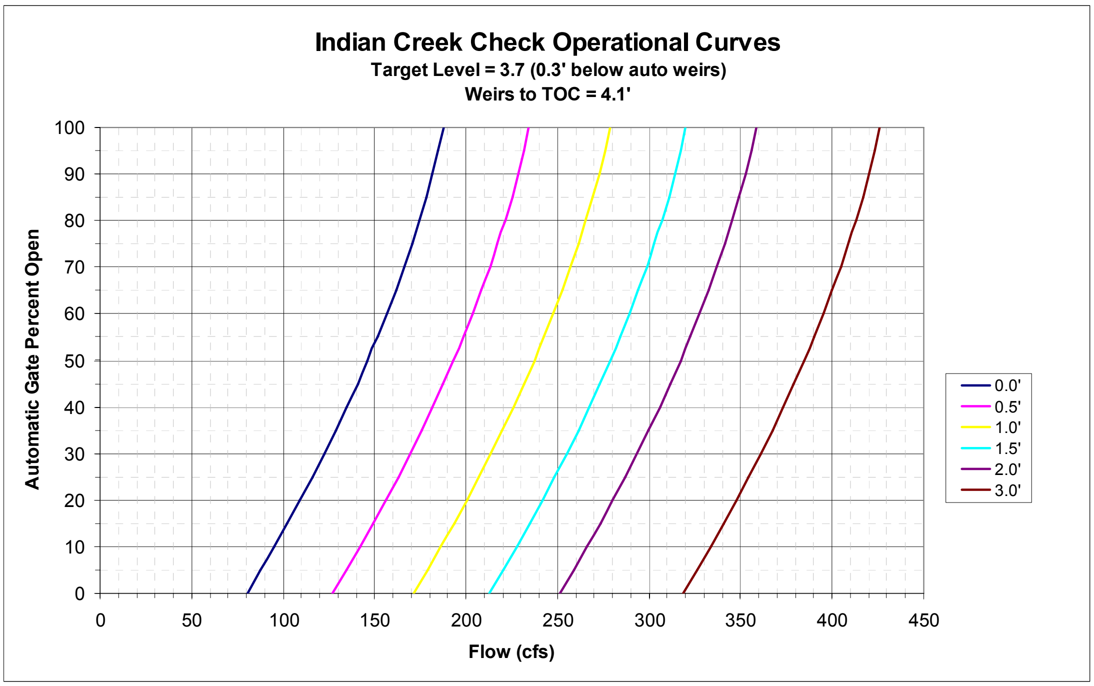

Figure 12. Operational curves developed for the Indian Creek check.

Table 5. Operational chart developed for the Young check. Applicable for: Target level = 3.5' (5 tenths below the overflow weirs), Weirs to top of concrete = 4.5'.

Automatic Gate Percent Open	Manual Gate Stem Heights (ft)								
	0.0'	0.25'	0.5'	0.75'	1.0'	1.25'	1.5'	1.75'	2.0'
0	59	79	97	116	133	150	167	183	199
5	68	88	106	125	142	159	176	192	208
10	76	96	115	133	151	168	184	201	217
15	84	104	123	141	159	176	193	209	225
20	92	112	131	149	167	184	201	217	233
25	100	120	139	157	175	192	208	225	241
30	108	128	147	165	182	199	216	232	248
35	115	135	154	172	190	207	224	240	256
40	123	143	161	179	197	214	231	247	263
45	130	150	168	187	204	221	238	254	270
50	137	157	176	194	211	228	245	261	277
55	144	164	182	200	218	235	252	268	284
60	150	170	189	207	225	242	259	275	291
65	157	177	196	214	231	248	265	281	297
70	163	183	202	220	238	255	271	288	304
75	169	189	208	226	244	261	278	294	310
80	175	195	214	232	250	267	284	300	316
85	181	201	220	238	255	273	289	306	322
90	186	206	225	243	261	278	295	311	327
95	191	211	230	248	266	283	300	316	332
100	196	216	235	253	270	287	304	320	336

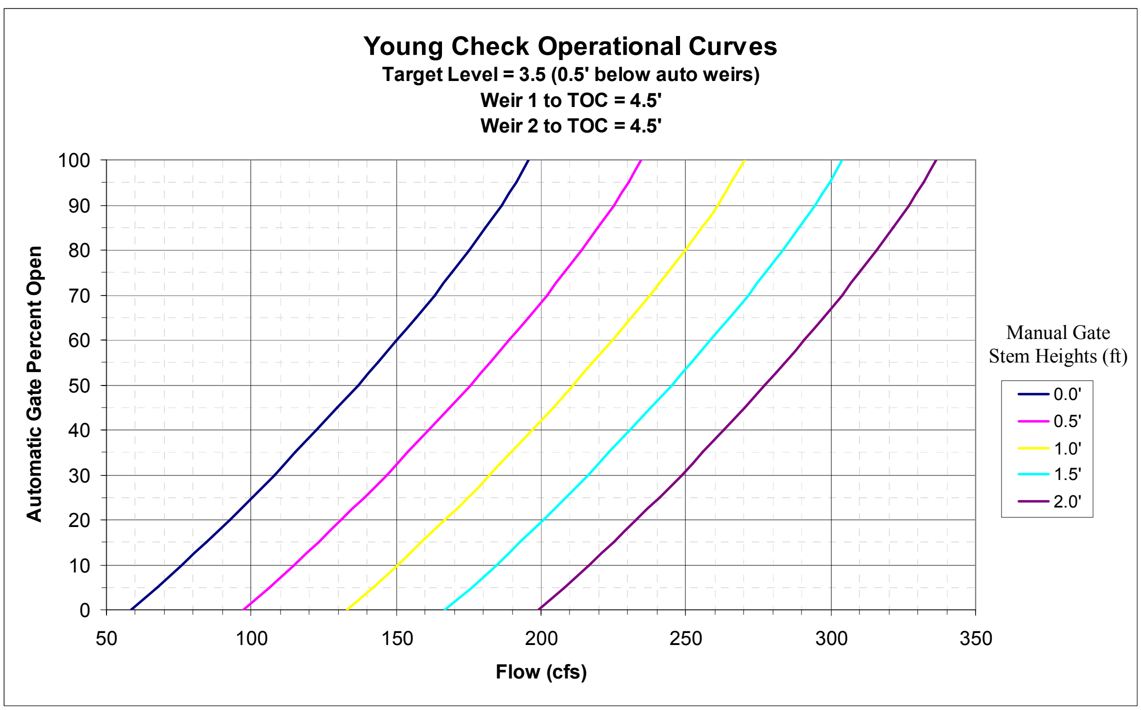

Figure 13. Operational curves developed for the Young check.

Table 6. Operational chart developed for the Stumph check. Applicable for: Target level = 3.4' (6 tenths below the overflow weirs), Weir 1 to top of concrete = 4.3', Weir 2 to top of concrete=3.65'

Manual Gate Stem Heights (ft)

Automatic Gate Percent Open	0.0'	0.25'	0.5'	0.75'	1.0'	1.25'	1.5'	1.75'	2'
0	70	85	100	114	128	141	153	166	177
5	85	100	115	129	142	155	168	180	192
10	99	114	129	143	156	169	182	194	206
15	113	128	142	156	170	183	196	208	219
20	126	141	156	170	183	196	209	221	233
25	139	154	168	182	196	209	222	234	245
30	151	166	181	195	208	221	234	246	258
35	163	178	192	207	220	233	246	258	270
40	174	189	204	218	231	245	257	269	281
45	185	200	215	229	242	255	268	280	292
50	195	210	225	239	253	266	278	290	302
55	205	220	235	249	262	275	288	300	312
60	214	229	244	258	272	285	297	309	321
65	223	238	253	267	280	293	306	318	330
70	231	246	261	275	288	301	314	326	338
75	238	253	268	282	296	309	321	334	345
80	245	260	275	289	302	316	328	340	352
85	251	266	281	295	309	322	334	346	358
90	257	272	286	300	314	327	340	352	363
95	261	276	291	305	319	332	344	356	368
100	265	280	295	309	323	336	348	360	372

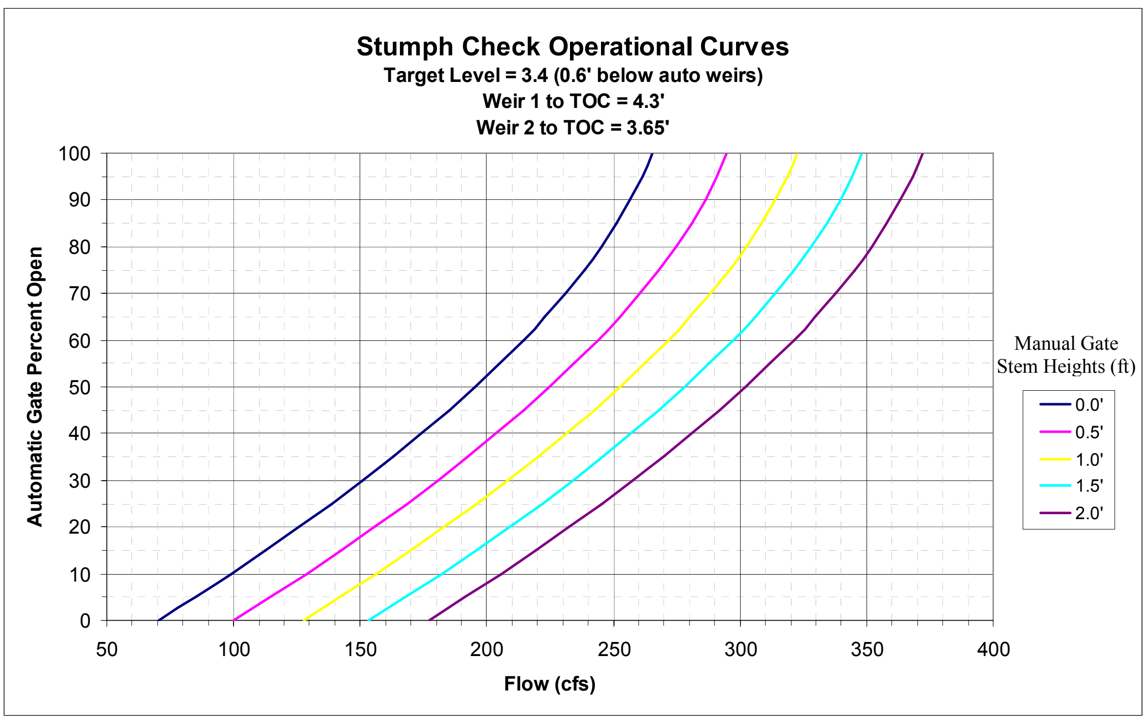

Figure 14. Operational curves developed for the Stumph check.

Table 7. Operational chart developed for the Beehive check. Applicable for: Target level = 3.7' (3 tenths below the overflow weirs), Weir 1 to top of concrete = 4.7’, Weir 2 to top of concrete=4.5'

Manual Gate Stem Heights (ft)

Automatic Gate Percent Open	0.0'	0.5'	1.0'	1.5'	2.0'	2.5'	3.0'	3.5'	4.0'	4.5'
0	107	119	131	142	153	162	171	180	187	194
5	110	122	134	145	155	165	174	182	190	196
10	112	125	136	147	158	168	177	185	192	199
15	115	127	139	150	160	170	179	187	195	201
20	118	130	141	153	163	173	182	190	197	204
25	120	132	144	155	165	175	184	192	200	206
30	122	135	146	157	168	177	186	195	202	209
35	125	137	149	160	170	180	189	197	205	211
40	127	139	151	162	172	182	191	199	207	213
45	129	141	153	164	175	184	193	202	209	216
50	131	144	155	166	177	186	195	204	211	218
55	133	146	157	168	179	189	198	206	213	220
60	135	148	159	170	181	191	200	208	215	222
65	137	150	161	172	183	193	202	210	217	224
70	139	152	163	174	185	194	203	212	219	226
75	141	153	165	176	187	196	205	213	221	228
80	143	155	167	178	188	198	207	215	223	229
85	145	157	168	180	190	200	209	217	224	231
90	146	158	170	181	192	201	210	218	226	233
95	148	160	172	183	193	203	212	220	227	234
100	149	161	173	184	194	204	213	221	229	235

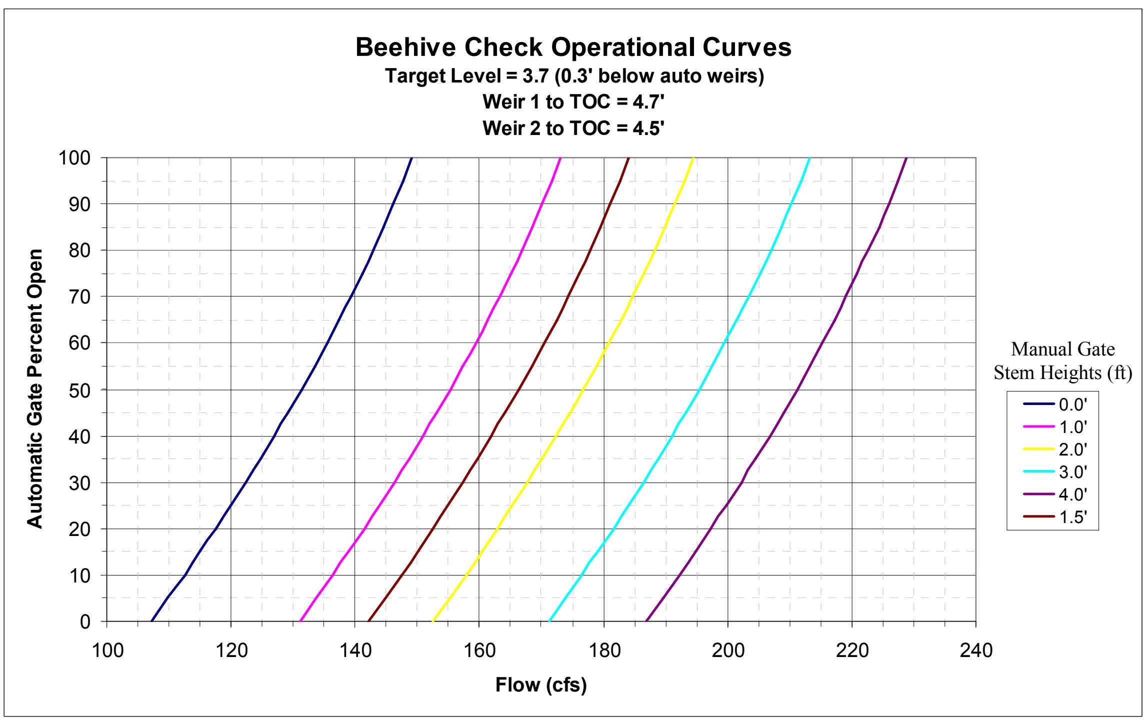

Figure 15. Operational curves developed for the Beehive check.

Table 8. Operational chart developed for the Townsite check. Applicable for: Target level = 3.6' (4 tenths below the overflow weirs), Weirs to top of concrete=3.6’

Manual Gate Stem Heights (ft)

Automatic Gate Percent Open	0.0'	0.25'	0.5'	0.75'	1.0'	1.25'	1.5'	1.75'	2.0'
0	22	34	45	55	65	75	84	92	100
5	26	38	49	59	70	79	88	97	105
10	30	42	53	64	74	83	92	101	109
15	34	46	57	68	78	87	96	105	113
20	38	50	61	71	82	91	100	109	117
25	42	54	65	75	85	95	104	113	121
30	46	57	68	79	89	99	108	116	124
35	49	61	72	82	92	102	111	120	128
40	53	64	75	86	96	105	115	123	131
45	56	67	78	89	99	109	118	126	134
50	59	70	81	92	102	112	121	129	137
55	62	73	84	95	105	115	124	132	140
60	65	76	87	98	108	117	126	135	143
65	67	79	90	100	110	120	129	138	146
70	70	81	92	103	113	122	131	140	148
75	72	83	94	105	115	125	134	142	150
80	74	85	96	107	117	127	136	144	152
85	76	87	98	109	119	128	138	146	154
90	77	89	100	110	121	130	139	148	156
95	79	90	101	112	122	132	141	149	157
100	80	92	103	113	123	133	142	150	158

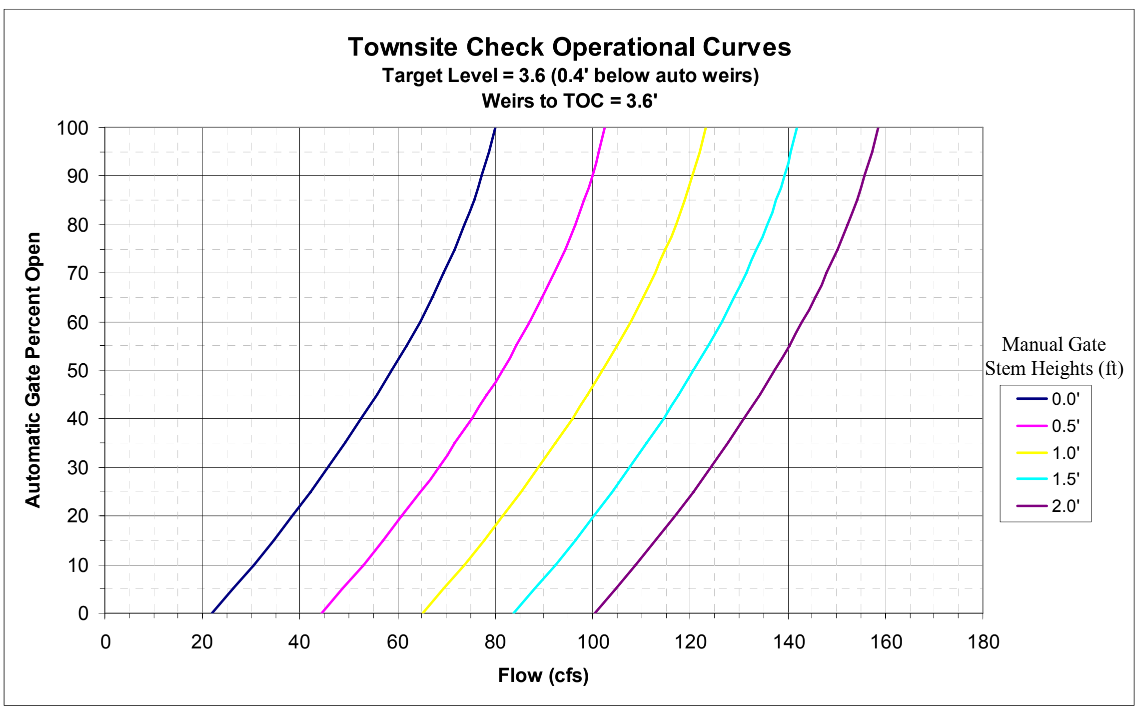

Figure 16. Operational curves developed for the Townsite check.

2.4 Application of Operational Curves and Charts

The operational curves/charts can be used two different ways: as a flow measuring device or as a tool for check operation. If the BFID knows the manual check settings and the automatic gate percent open, a curve or chart can be used to find the estimated flow. An example of this type of application can be seen in Figure 17 and Table 9. If all the manual gates at the automated check are set to a stem height of 1.0 ft, and if the automated gate is 50 percent open, the flow according to the operational chart/curve would be 256 cfs.

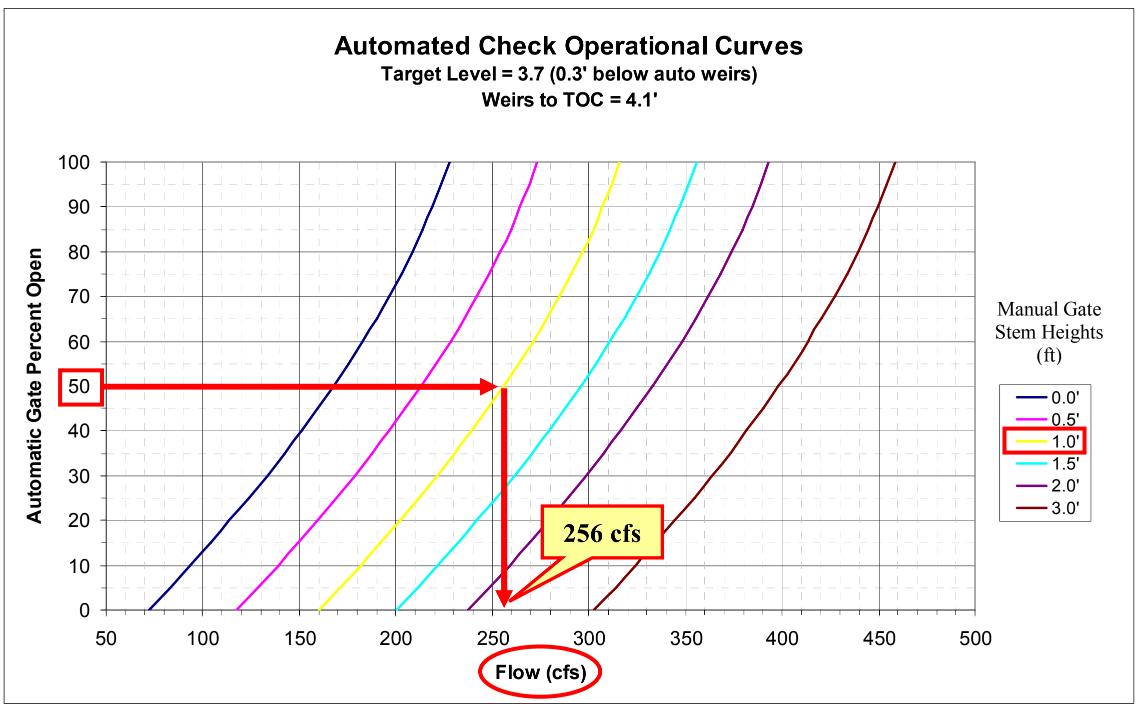

Figure 17: Illustration of how operational curves can be used for flow measurements.

Table 9. Illustration of how operational curves can be used for flow measurements.

Manual Gate Stem Heights (ft)

Automatic Gate Percent	0.0'	0.25'	0.5'	0.75'	1.0'	1.25'	1.5'	1.75'	2'	2.5	3
0	72	95	117	139	160	181	200	219	237	272	303
5	83	106	128	150	171	191	211	230	248	283	314
10	94	117	139	161	182	202	222	241	259	293	324
15	104	127	149	171	192	212	232	251	269	304	335
20	114	137	159	181	202	222	242	261	279	314	345
25	124	147	169	191	212	232	252	271	289	323	354
30	133	156	179	200	221	242	261	280	299	333	364
35	142	165	188	209	230	251	270	289	308	342	373
40	151	174	196	218	239	260	279	298	316	351	382
45	160	183	205	227	248	268	288	307	325	359	390
50	168	191	213	235	256	276	296	315	333	367	398
55	175	198	221	243	264	284	304	323	341	375	406
60	183	206	228	250	271	291	311	330	348	382	413
65	190	213	235	257	278	298	318	337	355	389	421
70	197	220	242	264	285	305	325	344	362	396	427
75	203	226	248	270	291	311	331	350	368	402	433
80	209	232	254	276	297	317	337	356	374	408	439
85	214	237	259	281	302	323	342	361	379	414	445
90	219	242	264	286	307	328	347	366	384	419	450
95	224	247	269	291	312	332	352	371	389	423	454
100	228	251	273	295	316	336	356	375	393	427	458

Another application of the operational curves/charts is to aide in optimally operating automated check structures. An example of this type of application can be seen in Figure 18 and Table 10. If all the manual gates at the Indian Creek check are set to a stem height of 1.0 ft, and the flow expected is 200 cfs, the manual gate will be approximately 17 percent open. The yellow region on the operational charts signifies the "safe operating range" of the automated gate operation. Seventeen percent lies on the border of the safe operational zone. This is not the optimum position of the automated gate because it does not have a large range of motion. A manual gate stem height of 0.5' might be more appropriate to optimize the operation of the automated gate because it will place the automated gate at approximately 45 percent open. Knowing this, the BFID can send a ditchrider out to adjust the manual gate settings to optimize the checks operation.

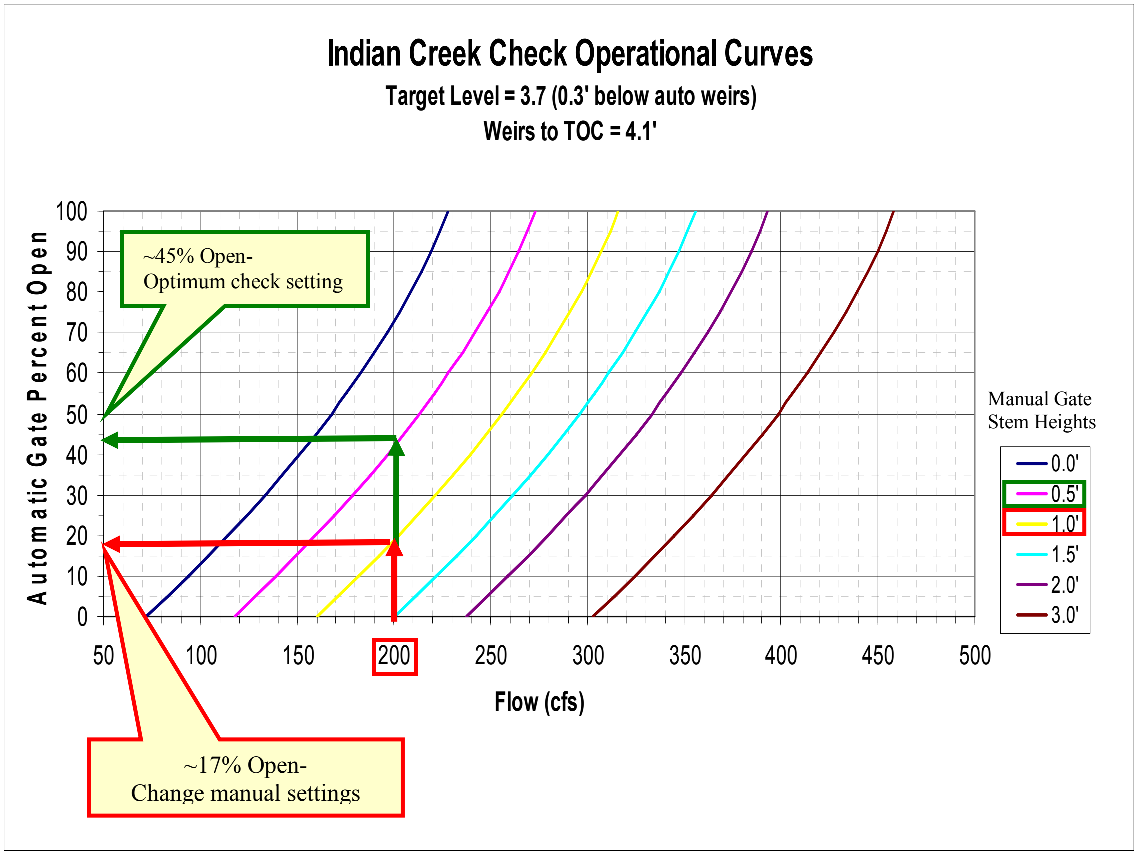

Figure 18. Illustration of how operational curves can be used to optimally operate check structures.

Table 10. Illustration of how operational charts can be used to optimally operate an automated check structure. Applicable for: Target level = 3.7' (3 tenths below the overflow weirs), weirs to top of concrete = 4.1'.

Manual Gate Stem Heights (ft)

Automatic Gate Percent	0.0'	0.25'	0.5'	0.75'	1.0'	1.25'	1.5'	1.75'	2'	2.5	3
0	72	95	117	139	160	181	200	219	237	272	303
5	83	106	128	150	171	191	211	230	248	283	314
10	94	117	139	161	182	202	222	241	259	293	324
15	104	127	149	171	192	212	232	251	269	304	335
20	114	137	159	181	202	222	242	261	279	314	345
25	124	147	169	191	212	232	252	271	289	323	354
30	133	156	179	200	221	242	261	280	299	333	364
35	142	165	188	209	230	251	270	289	308	342	373
40	151	174	196	218	239	260	279	298	316	351	382
45	160	183	205	227	248	268	288	307	325	359	390
50	168	191	213	235	256	276	296	315	333	367	398
55	175	198	221	243	264	284	304	323	341	375	406
60	183	206	228	250	271	291	311	330	348	382	413
65	190	213	235	257	278	298	318	337	355	389	421
70	197	220	242	264	285	305	325	344	362	396	427
75	203	226	248	270	291	311	331	350	368	402	433
80	209	232	254	276	297	317	337	356	374	408	439
85	214	237	259	281	302	323	342	361	379	414	445
90	219	242	264	286	307	328	347	366	384	419	450
95	224	247	269	291	312	332	352	371	389	423	454
100	228	251	273	295	316	336	356	375	393	427	458

An operational spreadsheet for each of the analyzed checks was also created for the BFID which condenses the information in the charts/curves (Figure 19). Within the operational spreadsheet there are two different options that the BFID can use. The first option will calculate flow based on the check settings and the upstream target level. The second option calculates the optimum manual gate settings based on an expected flow that is entered by the user. Both options offer an operational chart specific to the manual gate settings. This allows the BFID to pinpoint where they are within the operational range of the automated gate. The operational spreadsheets developed all provide option one. Due to time constraints, option two was only developed for the Indian Creek check to be used as an example for near future efforts.

An operational spreadsheet could be developed for all check structures (automated and unautomated) if the appropriate data is collected. This could be done by copying and pasting the data into a new spreadsheet. The new checks dimensions and developed coefficients would need to be input into the spreadsheet. The number of gates and weirs within the calculations will need to be adjusted for each check as most checks do not have the same number of weirs and gates. The developed coefficients can also be programmed into the existing BFID real-time automation to provide an instantaneous estimate of flow based on the check settings for each of the analyzed checks. The operational curves could also be used as a training tool for new ditch-riders allowing them to better understand the operation of an automated check structure. These curves, along with the discharge coefficients, would need to be validated during future irrigation seasons to confirm their accuracy.

Indian Creek Check

Option 1- Known: Check settings -- Find: Flow

Field Measurements Needed:	Target Level	Auto Gate Percent Open (%)	Manual Stem Height 1 (ft)	Manual Stem Height 2 (ft)	Manual Stem Height 3 (ft)	Weirs to TOC (ft)
Input Data Here: →	3.7	20	0.5	0.2	0.2	4.1

Flow Estimate: ***138*** ***cfs***

Possible Alarms:

Gate out of water:	OK
Auto gate maxed out:	OK

(this chart is only applicaple for Target Level=3.7 and Weir to TOC = 4.1)

Automatic Gate % Open	Total Flow (cfs)
0	109
5	116
10	123
15	131
20	138
25	144
30	151
35	157
40	163
45	169
50	**175**
55	180
60	185
65	190
70	194
75	199
80	203
85	207
90	210
95	213
100	216

Safe Operating Range

Option 2 - Known: Flow -- Find: Optimum manual gate settings

Expected Flow:	*156.0*	cfs ← *Input Data Here*
Manual Gate Settings:	***0.1***	***ft***

(this setting assumes: Target Level=3.7 and Weir to TOC = 4.1)

Automatic Gate % Open	Total Flow (cfs)
0	90
5	97
10	105
15	112
20	119
25	126
30	132
35	138
40	144
45	150
50	**156**
55	161
60	166
65	171
70	176
75	180
80	184
85	188
90	191
95	194
100	197

Safe Operating Range

Figure 19. Screen snapshot of an operational spreadsheet.

3.0 Hydraulic Model Development

3.1 EPA SWMM 5.0

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality (EPA, 2008). SWMM contains a runoff component and a routing component. The routing portion of SWMM transports runoff and inflows through a system of pipes, channels, storage/treatment devices, pumps, and regulators (EPA, 2008).

EPA SWMM 5.0 is a well documented program that was initially developed in 1971 and is widely used throughout the world for planning, analysis and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well (EPA, 2008).

SWMM's hydraulic modeling capabilities can be used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures. These capabilities allow SWWM to handle a variety of situations including the ability to (EPA, 2008):

- handle drainage networks of unlimited size
- use a wide variety of standard closed and open conduit shapes as well as natural channels
- model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices
- apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows
- utilize either kinematic wave or full dynamic wave flow routing methods
- model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding
- apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels

EPA SWMM 5.0 was chosen to be used based on the reasoning of Rolland (2005). SWMM was also chosen because it has powerful hydraulic capabilities, a graphical user-friendly interface, detailed documentation, and because it is widely accepted and used worldwide (Schoenfelder, 2006). For this application, runoff and pollutant loadings were not accounted for. Only the hydraulic routing capabilities of SWMM were used in the modeling process.

3.2 Modeling of the BFID North Canal

3.2.a Input Data

Data needed for input into the SWMM model was obtained from Bureau of Reclamation (BOR) survey data (collected in the 1980's), contract drawings, and field measurements. The data that was obtained included channel geometry, check structure dimensions, siphon layouts, culvert/bridge dimensions, parshall flume geometry, and cippolletti weir dimensions. The BOR survey data provided: structure stationing (checks, turnouts, bridges/culvers, siphons, flumes, and weirs), canal invert elevations, turnout and lateral pipe invert elevations, top of structure elevations, and channel geometry. Field measurements of structures were also recorded and used to fill in any missing data and verify the survey data. Every structure on the North Canal was mapped using a basic handheld Garmin Global Positioning System (GPS) unit and Geographic Information System (GIS) software. A background map of the structures was uploaded into SWMM by converting the GIS map into a BMP file. The horizontal layout (plan view) assisted in choosing exit/entrance loss coefficients in curved sections of the channel. Although the plan view is not important in SWMM, it does provide a realistic picture of the canal and serves as a visualization tool (Schoenfelder, 2006). The vertical profile was assigned using station and elevation data from the BOR survey. The plan and profile views are shown in Figure 20 and Figure 21.

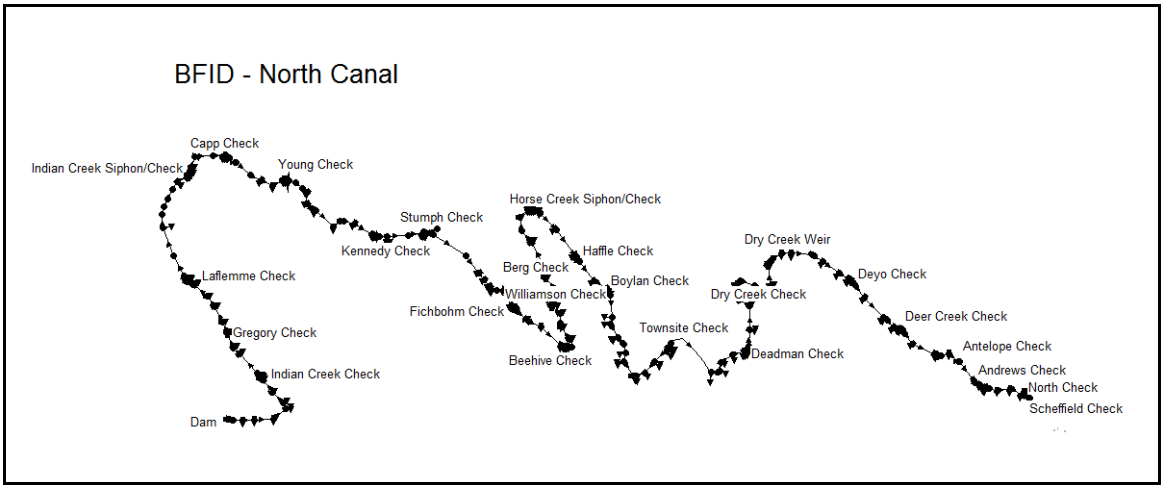

Figure 20. Plan view from SWMM.

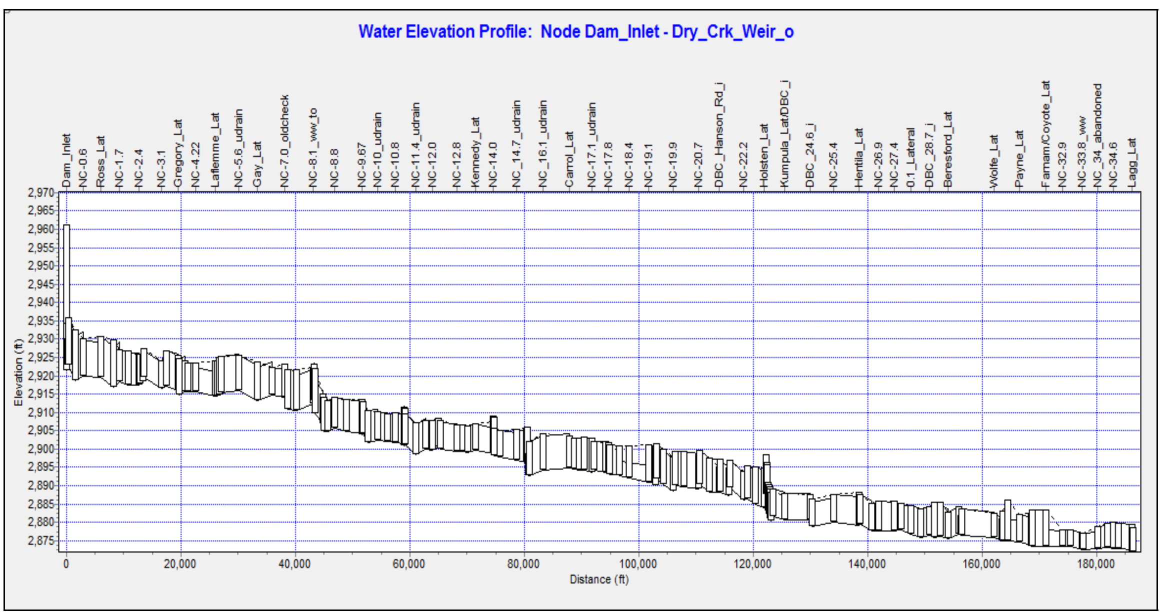

Figure 21.Vertical profile as seen in SWMM model.

3.2.b Simulation of BFID Components

The North Canal components were simulated similar to Schoenfelder's (2006) South Canal model.

3.2.b.i Open Channel Canal

The North Canal was modeled as a series of open channel trapezoidal conduits. All conduits were connected with junction nodes in SWMM with the appropriate centerline invert elevations. The conduits were assigned bottom widths, depths, and lengths according to the BOR survey data. Average depths and widths were assigned to each conduit based on the depth and bottom width of the beginning and ending junction nodes. Channel side slopes were calculated based on BOR survey data. Initial Manning's n values of 0.0145 were assigned to each conduit based on Schoenfelder (2006) and French (1986).

3.2.b.ii Check Structures

Each check structure along the North Canal has unique dimensions. Field measurements of check structure dimensions were collected and used as input into the model. Check structures were modeled as a series of weirs and orifices (under-shot sluice gates) with appropriate dimensions that convey water through the structure. The weirs and orifices were connected with inlet and outlet junction nodes having appropriate invert elevations (Figure 22). Initially, sluice gates and weirs were assigned discharge coefficients based on the results of the operational curves/charts (Table 1). The Fichbohm check was assigned gate and weir discharge coefficients of 3.46 and 0.443 as recommended by Sanson (2008). For checks without operational curves/charts, discharge coefficients were assigned by averaging the coefficients from the nearest check structures with developed discharge coefficients (Table 11). Entrance and exit loss coefficients of 0.4 and 1.0 were also assigned to check structures (Mays, 2001; Sturm, 2001).

Automated check structures that had operational curves developed were assigned discharge coefficients that were established as shown in Table 1. Automated gates were modeled in SWMM by assigning control rules that adjust the gate to hold the pool level at the specified target depth, as discussed in Section 4.3.

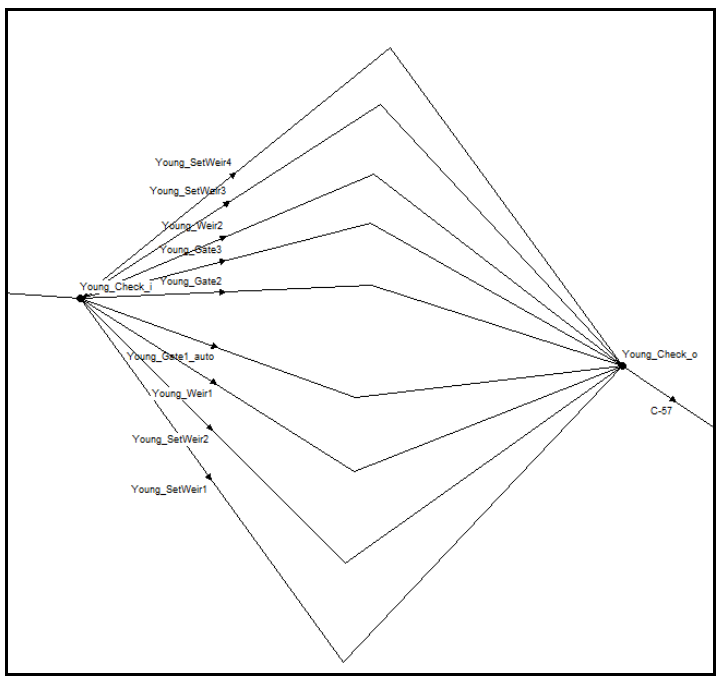

Figure 22. Young Check structure modeled in SWMM

Table 11. Initial discharge coefficients assigned to check structures

Check	Cd-gates	Cd-weirs	
Indian Creek	0.669	3.731	
Gregory	0.669	3.731	*
LaFlemme	0.577	3.574	*
Indian Creek Siphon	0.577	3.574	*
Capp	0.484	3.417	*
Young	0.484	3.417	
Kennedy	0.493	3.346	*
Stumph	0.501	3.274	
Fickbohm	0.443	3.462	
Beehive	0.168	3.438	
Williamson	0.421	2.520	*
Berg	0.421	2.520	*
Horse Creek Siphon	0.421	2.520	*
Haffle	0.421	2.520	*
Boylan	0.399	1.577	*
Townsite	0.399	1.577	

* assumed values

3.2.b.iii Farmer Turnouts and Laterals

Farmer turnouts and laterals were modeled as side orifices at a junction node on the main canal with the appropriate diameter and invert elevation. A discharge coefficient of 0.65 was assigned to every orifice. The orifice was connected to a free fall outfall node which is capable of discharging water from the system. The outfall invert elevations were set to the head gate turnout pipe invert elevations which may not be the case in reality. The downstream conditions of most laterals and turnouts were not modeled, although it is possible with the appropriate channel geometry and elevation data.

3.2.b.iv Siphons

Siphons are pressurized pipe sections that serve to convey irrigation canal water below major waterways or drainages in the canal's path. The North Canal has 3 siphons: Indian Creek Siphon, Horse Creek Siphon, and Dry Creek Siphon. The siphons were modeled as a series of circular pipe conduits with appropriate diameters connected by a junction node. Each section of conduit was assigned a bend loss coefficient of 0.2 and a Manning's n value of 0.013 (Sturm, 2001; Mays, 2001). The elevation and station of each section of conduit comprising the siphon were collected from the BOR survey data. Surcharge depths were specified at the pipe junctions to allow the siphon to function as a pressurized pipe. SWMM defines surcharge depth as the additional depth of water beyond the maximum depth that is allowed before the junction floods (EPA, 2008). The surcharge depth was defined to exceed the maximum depth needed to overcome the head increase (elevation difference from the lowest part of the siphon to the inlet elevation) in order to prevent flooding. Entrance and exit loss coefficient of 0.4 and 1, respectively, were assigned to all siphons.

3.2.b.v Bridges and Culverts

Dimensions of bridges and box culverts were collected in the field which included the height, width, and length. These structures were modeled as rectangular closed conduits with the appropriate dimensions and number of barrels. Bridges and culverts were assigned entrance and exit loss coefficients of 0.5 and a Manning's n value of 0.013 (Sturm, 2001; Mays, 2001) which follows Schoenfelder's (2006) assumptions.

3.2.b.vi Parshall Flumes

All of the flumes on the North Canal are Parshall flumes. Contract drawings were verified by field measurements and used as input into SWMM. Parshall flumes were modeled as rectangular open conduits and assigned a constant throat width to the entire length of the structure. This disregards the actual dimensioning of parshall flumes. Studies were performed by Schoenfelder (2006) with actual flume dimensions. The simplified flumes produced similar results of flow and depth in the flume and were therefore input into SWMM. Each flume was assigned entrance and exit loss coefficients of 0.4 and 1.0, respectively (Sturm, 2001; Mays, 2001).

3.2.b.vii Cipolletti Weir

The Dry Creek cipolletti weir is the only weir on the main North Canal used for flow measurement. The cipolletti weir was modeled as a trapezoidal weir with the appropriate dimensions that were measured in the field. A discharge coefficient of 3.4 was assigned to the cipolletti weir which is representative of a sharp-crested weir (BOR, 2001).

3.2.c Simulation of System Losses

Operational losses account for seepage and evaporation occurring in the system. SWMM 5.0 currently does not have the capabilities of simulating infiltration or evaporation in conduits, thus other means of addressing these losses were employed (Schoenfelder, 2006). Schoenfelder (2006) obtained an estimate of losses from the BOR Belle Fourche Unit Water Management Study (1998). The study indicated an average loss rate of approximately 5.5 cfs per mile for South Canal miles 34-38 which was regarded as significantly high. Schoenfelder (2006) found that a loss rate of 1 cfs per mile was more representative of the losses indicated during his calibration process. Schoenfelder's (2006) assumption of 1 cfs per mile was used and pumped out at every check using pump curves.

3.2.d Simulation Computational Method

The dynamic wave routing option was chosen to simulate the irrigation system. Dynamic wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results (EPA, 2008). Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow (EPA, 2008). It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices (EPA, 2008) which is typical of this irrigation system. Within the dynamic wave routing options, a routing time step of two seconds was used as well as a ten minute reporting time step.

4.0 Model Calibration

4.1 Defining Reaches

In order to collect continuous data, it was decided that the North Canal should be divided into 4 reaches because of its size. The defined reaches are shown in Table 12 and Figure 23. The reaches were split at key locations to provide inflow and outflow values during calibration. Reach 1 ends at the Young automated check structure and therefore needed operational curves/charts developed in order to calibrate the model to both stage and flow. Reach 2 ends at the Beehive flume and automated check combo site. (A combo site refers to an automated site with a combination of an automated check and flume). The Beehive flume had submergence issues due to backwater effects causing inaccurate flow measurements. Therefore, the operational spreadsheet developed for the Stumph Check was used to help calibrate the SWMM model to flow. Reach 3 ended at the Dry Creek Weir which provided accurate flow measurements for calibration. Due to time constraints and the difficult operation of the lower end of the North Canal, Reach 4 data was entered into SWMM but was excluded from the calibration process until further data collection and detailed analysis can be performed outside of this research.

Table 12. Reaches defined for data collection and model calibration.

Reach Number	Starting Structure	Ending Structure
1	North Canal Dam Flume	Young Automated Check
2	Young Automated Check	Beehive Flume/Automated Check Combo Site
3	Beehive Flume/Automated Check Combo Site	Dry Creek Weir
4	Dry Creek Weir	Final Wasteway

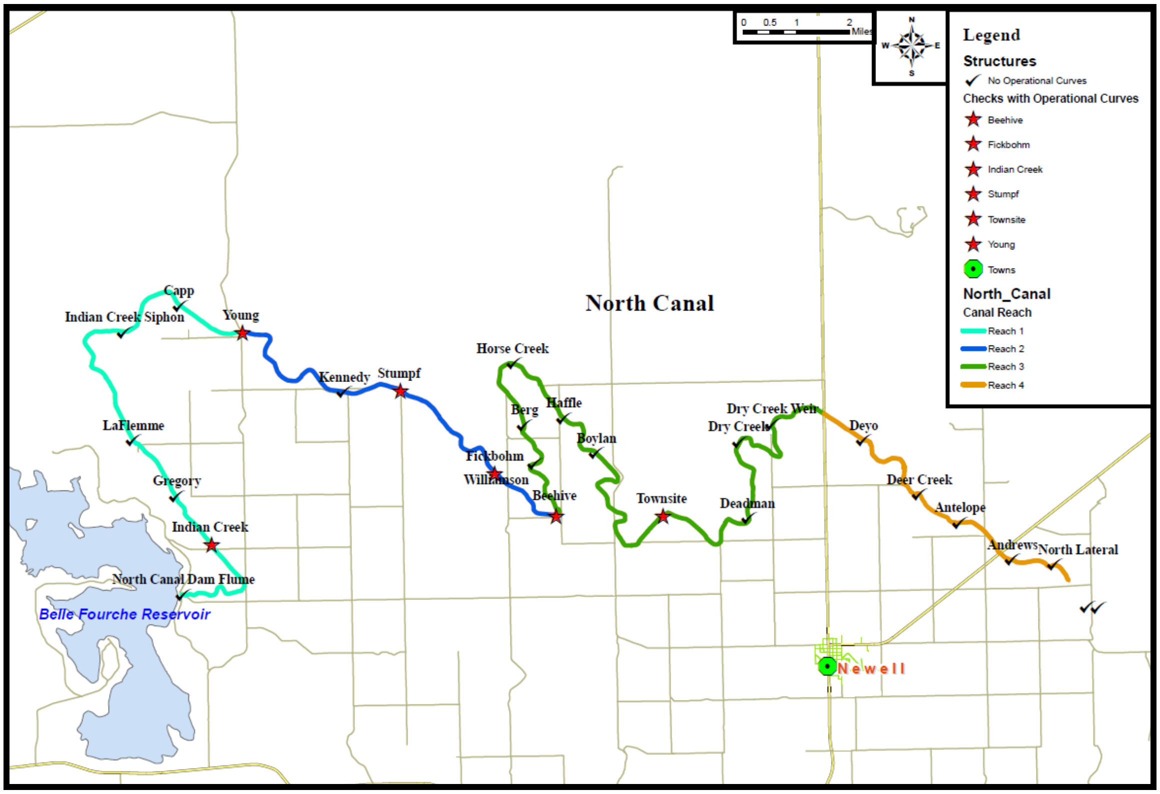

Figure 23. Monitoring and calibration reaches defined for the North Canal.

4.2 Data Collection

Calibration data was collected during the 2006, 2007 and 2008 irrigation seasons. Data for each reach was collected for three to seven continuous days. Data within each reach was collected sequentially starting at the upstream reach structure and proceeded downstream to the structure at the end of the reach. The data collected at laterals and turnouts included: gate stem heights (openings), and relative stage measurements of water surface to top of concrete at the structure (Figure 24). Check structure settings were also collected which included: manual gate stem heights, automatic gate percent open (from the datalogger), adjustable weir relative measurements of top of weir to top of concrete at the structure, upstream water surface to top of concrete at the structure and downstream water surface to top of concrete at the structure (Figure 24 and Figure 25). The reference data collected were later converted to actual depths and settings. Stage data collected at various other automated check structures was also used in calibration. Portable pressure transducers and dataloggers were also placed throughout the North Canal during reach monitoring periods. The top of the overflow weirs was used as a consistent datalogger reference point. This reference point was set at 4.0 for all checks in the BFID for simplicity. Unautomated (Sanson, 2008) and automated operational curves/charts were also used during calibrating where data were available.

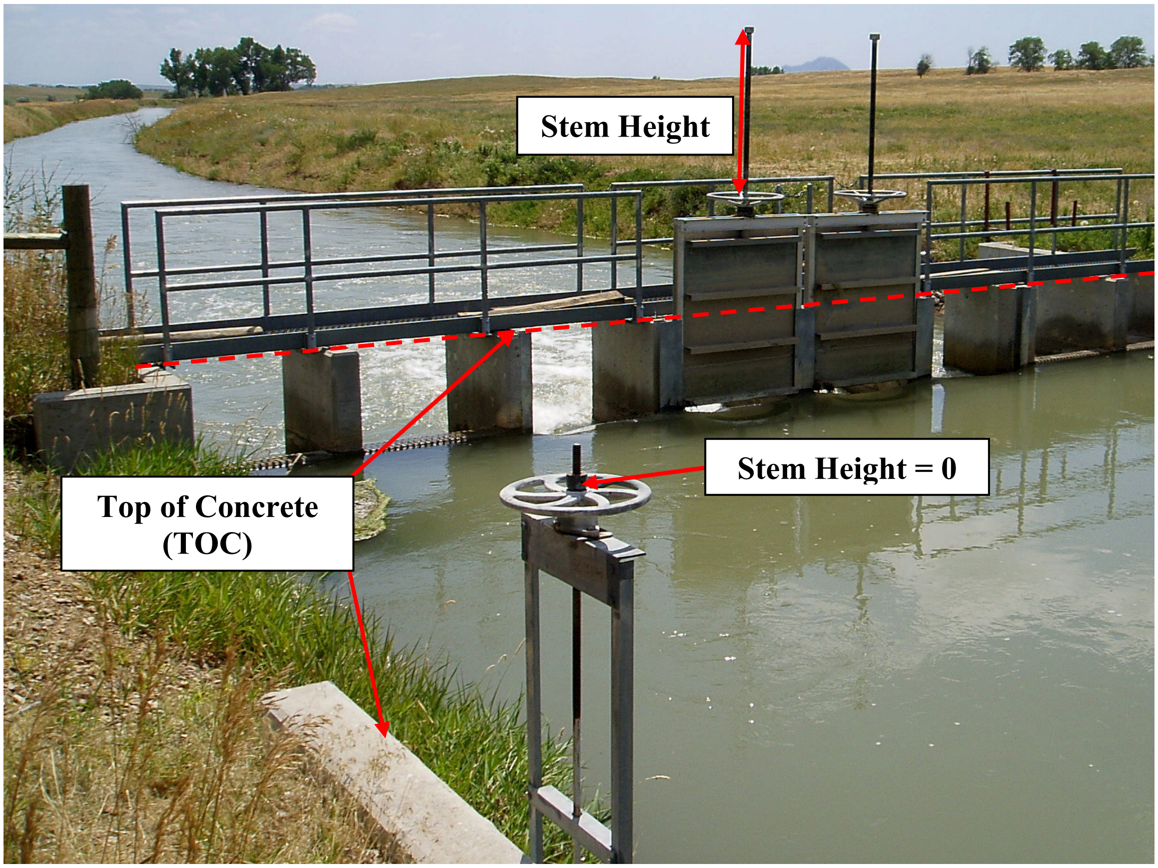

Figure 24. Illustration of manual gate stem height measurement and the location of the top of concrete at check structures and headgates along the canal.

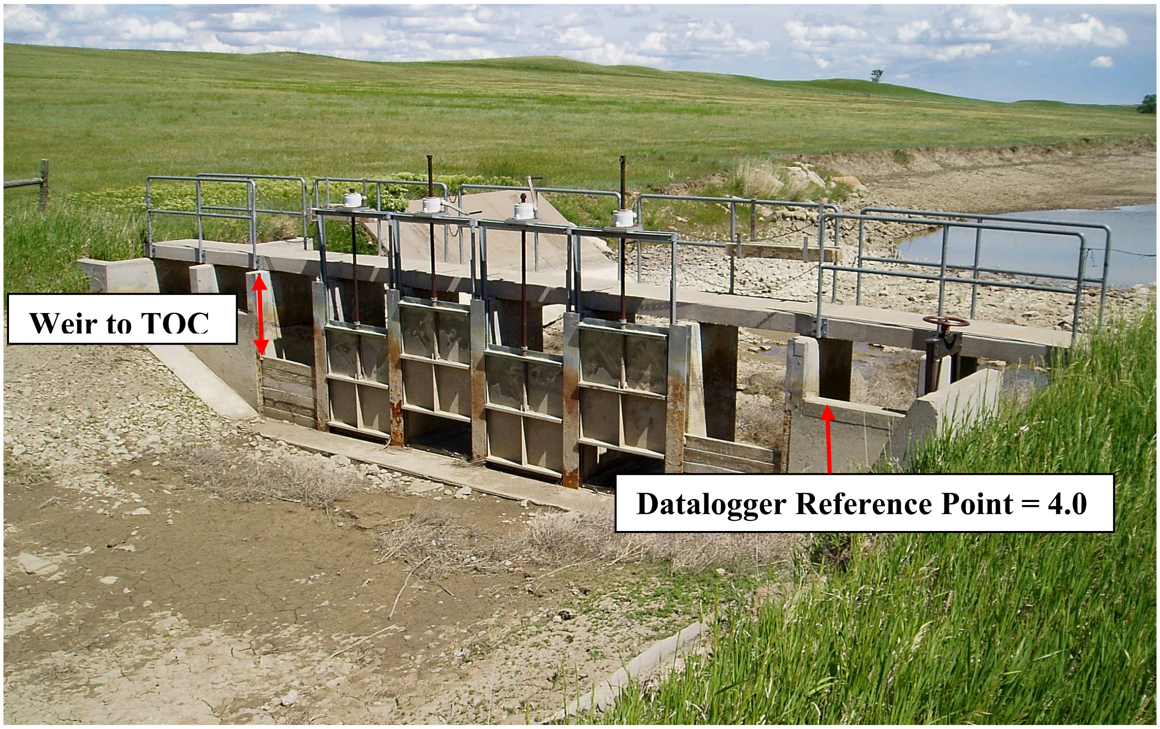

Figure 25. Illustration of weir to TOC measurement and the 4.0 reference point located at the top of the overflow weirs.

4.3 Simulation of Automated Gates

4.3.a Control Rules Based on Water Level

As mentioned previously, the purpose of an automated gate is to maintain a constant upstream pool level. Without the automated gate, the upstream stage would vary depending on the flow being delivered through the check structure. Control rules were used to define the automated gate setting based on the upstream water level at the check structure. Initially, the control rules suggested by Schoenfelder (2006) were used to simulate an automated gate:

Rule c_close_gate_Indian_Crk_check_automation
If NODE Indian_Crk_Check_i DEPTH < 4.65
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.2

Rule o_open_gate_Indian_Crk_check_automation
If NODE Indian_Crk_Check_i DEPTH > 4.75
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.8

With these control rules, a target level of 4.7 feet was used with the application of a deadband of ±0.05 feet. This mimics the automated gate program used in the BFID. Setting a deadband reduces the frequency of gate movement. Figure 26 shows how the modeled water level was kept within the target level deadband. It also shows a large amount of fluctuation occurring during the simulation. Similar results were found for the flow comparison of the Indian Creek Check and the input dam release time series from the 2007 irrigation season (Figure 27). This is because the gate is overshooting the target level by opening or closing the gate too much every time step. In order to reduce the amount of time the gate moved, the time to open/close the automated gate (as a fraction of an hour) was adjusted within a range of 0 to 1.0 hours (Figure 28 to Figure 30). Results show that when the gate movement time was decreased, the fluctuations became larger and the gate was unable to hold the target level because the gate was opening and closing too far and too fast.

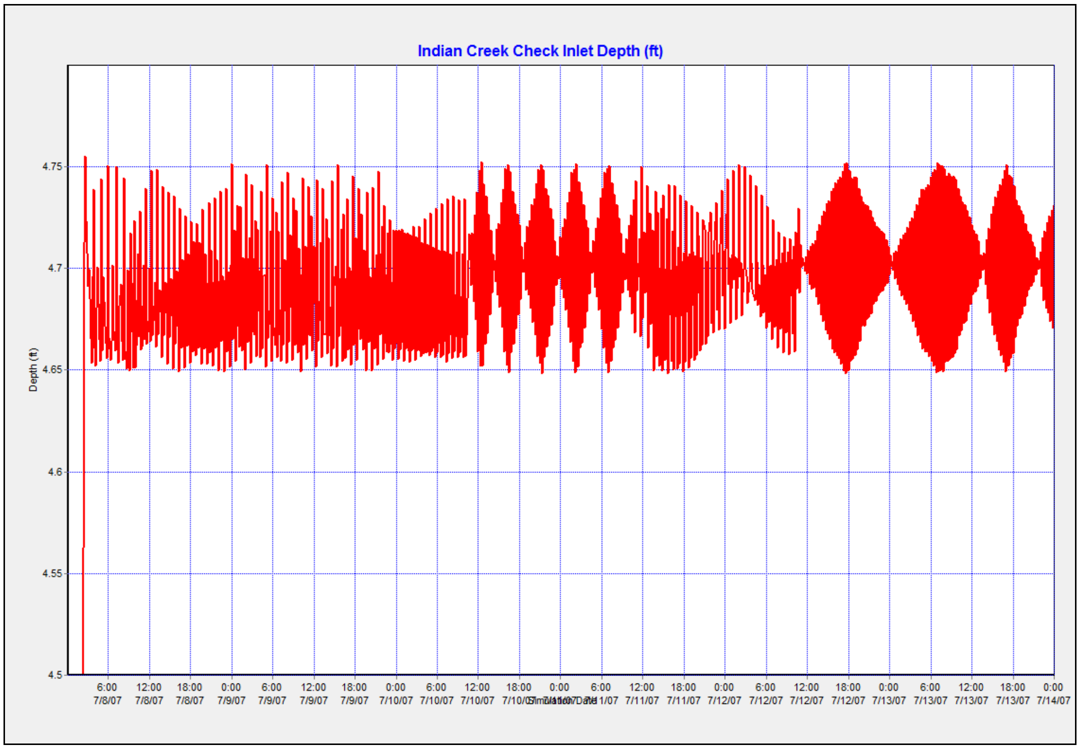

Figure 26. Indian Creek check structure depth using the control rules recommended by Schoenfelder (2006) with the time to open/close the automated gate set to 0.75 hours.

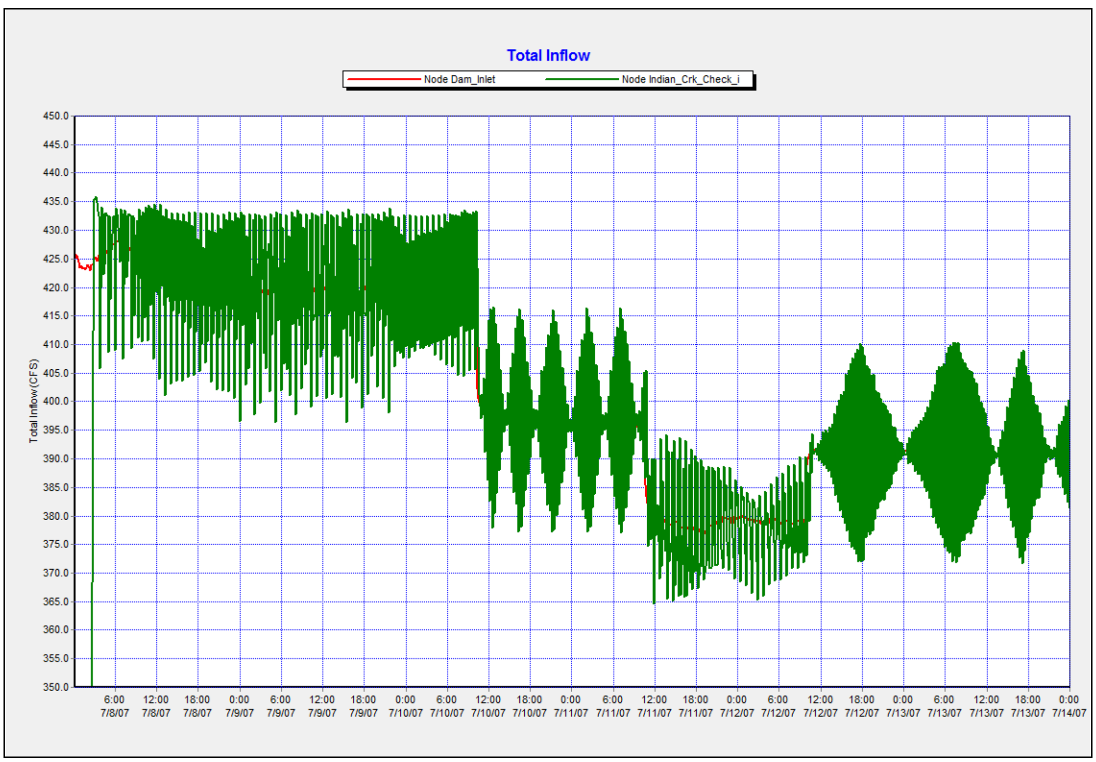

Figure 27. Total flow at the Indian Creek check and the input Dam releases using the control rules recommended by Schoenfelder (2006) and a time to open/close the automated gate of 0.75 hours.

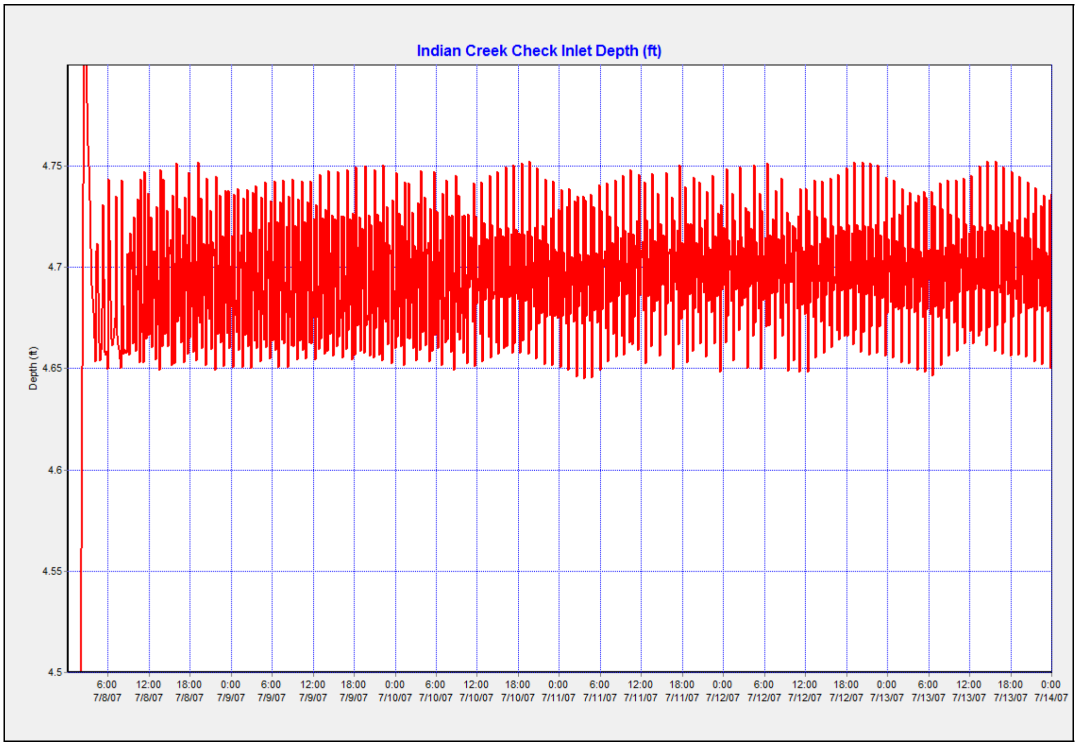

Figure 28. Figure 20. Upstream stage at the Indian Creek check using the control rules recommended by Schoenfelder (2006) and a time to open/close of 1.0 hrs.

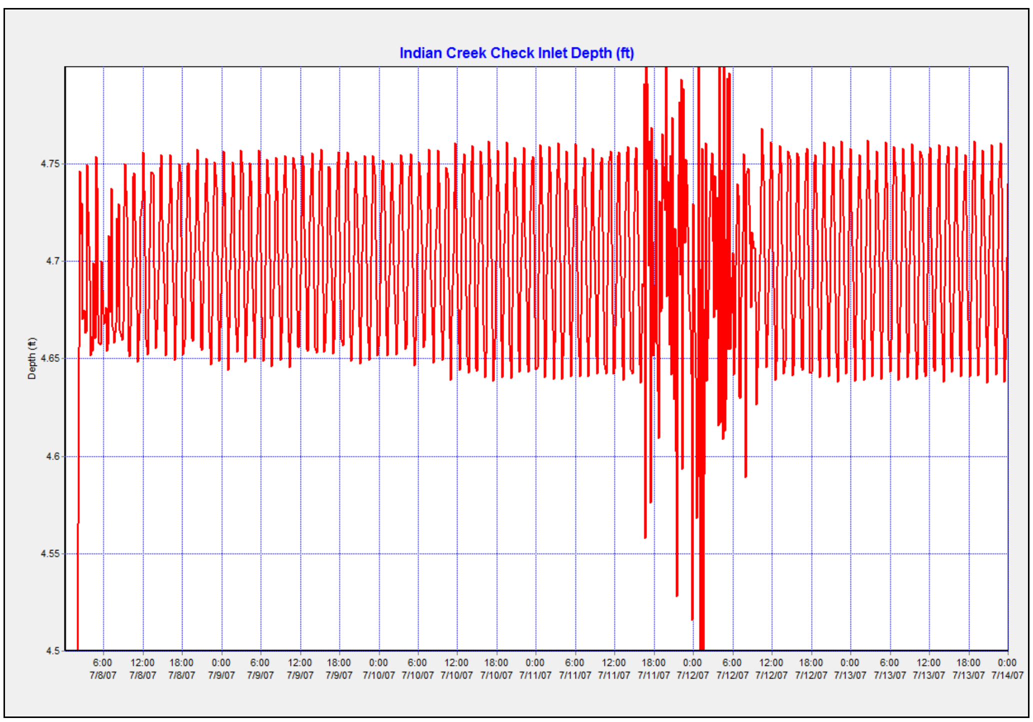

Figure 29. Upstream stage at the Indian Creek check using the control rules recommended by Schoenfelder (2006) and a time to open/close of 0.5 hrs.

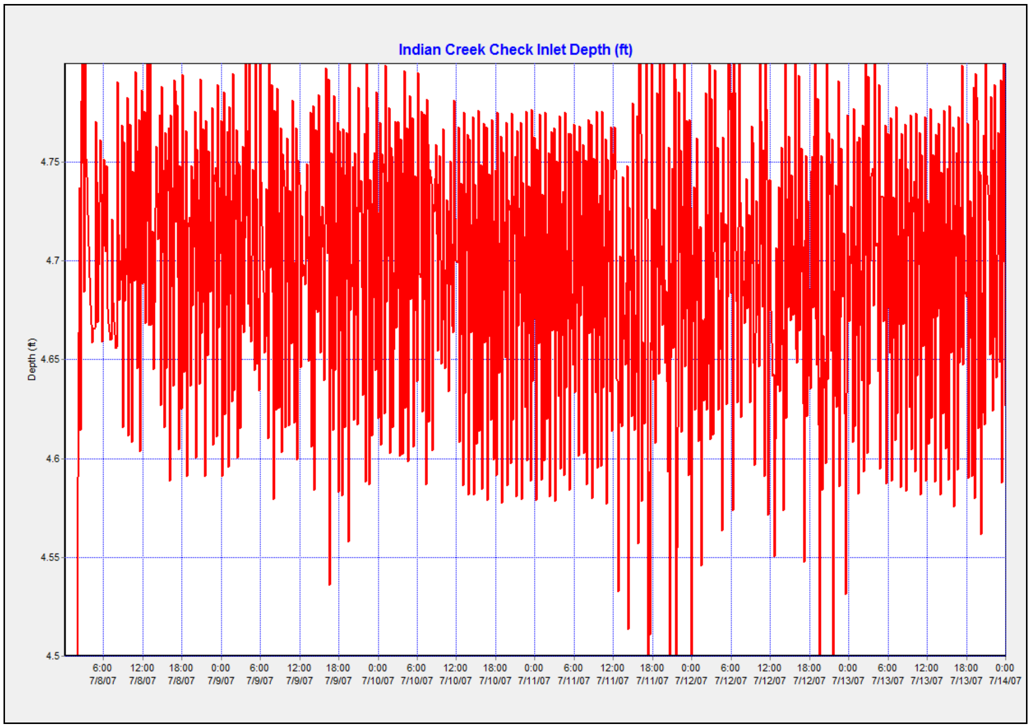

Figure 30. Upstream stage at the Indian Creek check using the control rules recommended by Schoenfelder (2006) and a time to open/close of 0.25 hours.

New control rules were developed in an attempt to reduce the distance the gate would open and the frequency of gate adjustments. These control rules were again based on the upstream water level at the Indian Creek Check. Gate settings were adjusted using trial and error until the results were reasonable. The following control rules were found to keep a relatively constant upstream water level and simulated the input dam releases very well (Figure 31 and Figure 32). Even though the time to open/close the automated gate was set to 0.75 hours, the automated gate moved at a frequency of approximately 1 minute or less.

Rule d_close_Indian_Crk_check_automation
If NODE Indian_Crk_Check_i DEPTH < 4.6
AND NODE Indian_Crk_Check_i DEPTH > 4.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.3
PRIORITY 1

Rule e_down_a_bit_Indian_Crk_check_automation
If ORIFICE Indian_Crk_gate3_auto SETTING >= 0.3
AND NODE Indian_Crk_Check_i DEPTH < 4.5
AND NODE Indian_Crk_Check_i DEPTH > 4.4
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.2
PRIORITY 2

Rule f_down_a_bit_more_Indian_Crk_check_automation
If ORIFICE Indian_Crk_gate3_auto SETTING >= 0.2
AND ORIFICE Indian_Crk_gate3_auto SETTING < 0.3
AND NODE Indian_Crk_Check_i DEPTH < 4.6
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.1
PRIORITY 3

;;;;gate moves up

Rule g_open_gate_Indian_Crk_check_automation
If ORIFICE Indian_Crk_gate3_auto SETTING <= 0.3
AND NODE Indian_Crk_Check_i DEPTH > 4.75
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.4
PRIORITY 4

Rule h_up_a_bit_Indian_Crk_check_automation
If ORIFICE Indian_Crk_gate3_auto SETTING >= 0.4
AND NODE Indian_Crk_Check_i DEPTH > 4.7
AND NODE Indian_Crk_Check_i DEPTH < 4.75
THEN ORIFICE Indian_Crk_gate3_auto Setting =0 .5
PRIORITY 5

Rule i_up_a_bit_more_Indian_Crk_check_automation
If NODE Indian_Crk_Check_i DEPTH >= 4.75
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.8
PRIORITY 6

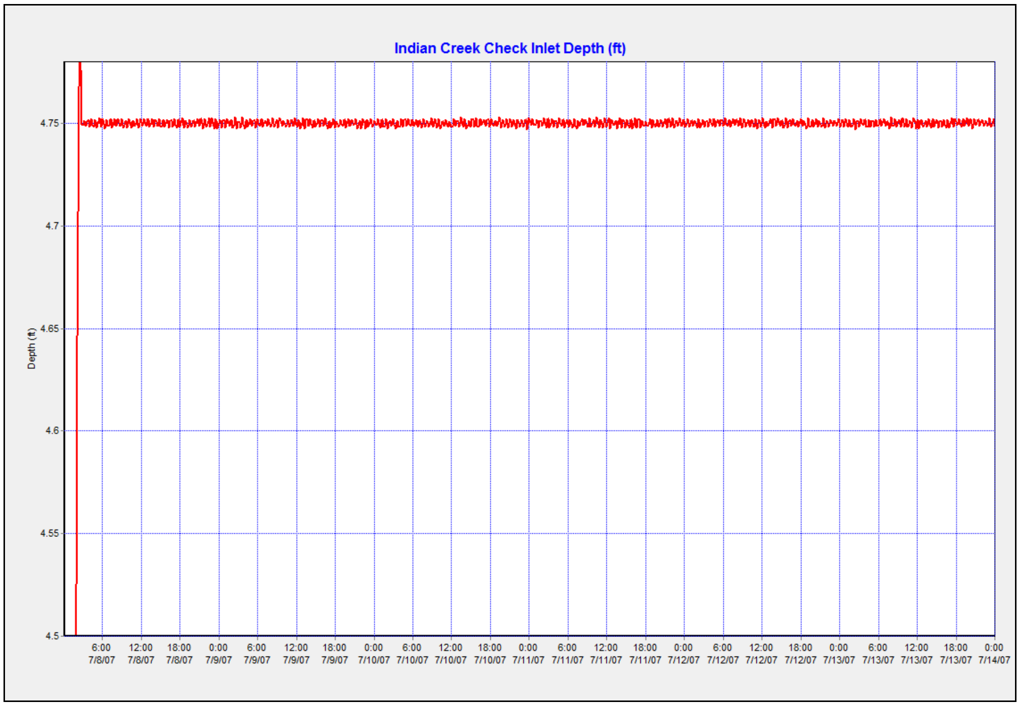

Figure 31. Upstream stage at the Indian Creek check using advanced control rules based on the upstream water level and a time to open/close of 0.75 hours.

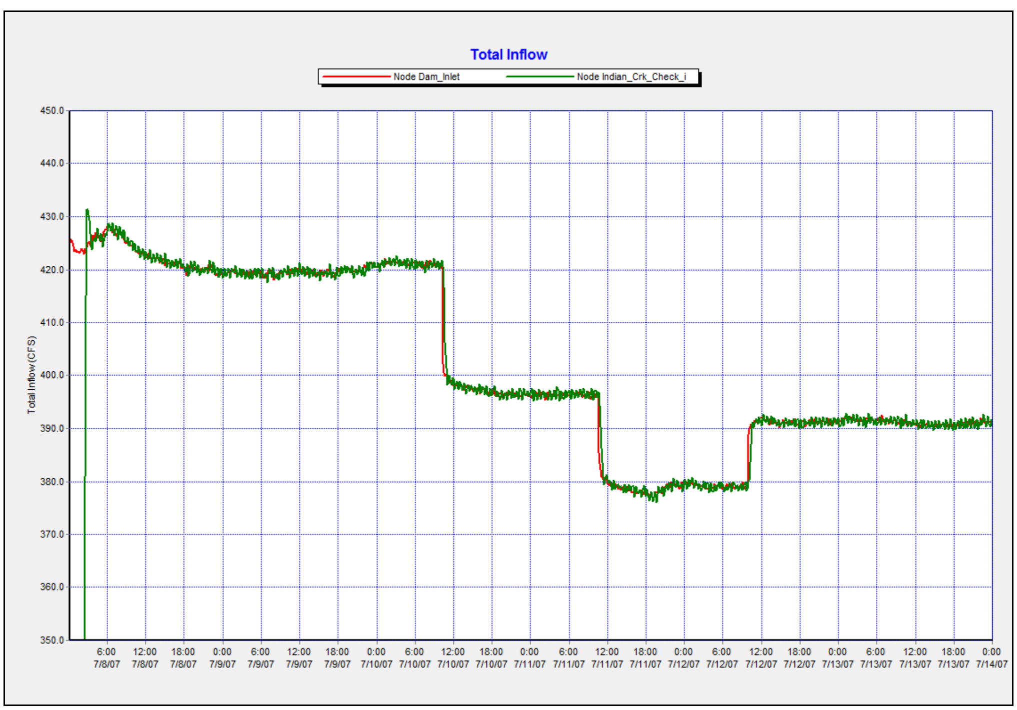

Figure 32. Total inflow from the Dam timeseries data and at the Indian Creek check using advanced control rules based on the upstream water level and a time to open/close of 0.75 hours.

4.3.b Control Rules Based on Flow

Several more attempts were made to control the automated gate settings based on the inflow at the inlet of the check instead of the upstream water level. It was thought that this could reduce the frequency of gate adjustments. The automated gate settings, over a range of flow, were defined using the operational curve/chart relationships. Using the manual gate settings that were measured and recorded for the calibration period in 2007 along with a set upstream water level of 4.7 ft, the flow for various automated gate settings was obtained. The settings and flow data were converted into control rules as shown below. The flow and depth results are very similar to previous results with the stability somewhat less but still not acceptable (Figure 33 and Figure 34).

```
Rule d_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 234.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.0

Rule e_open_Indian_Crk_check_automation
If LINK US_IC_check Flow < 245.5
AND LINK US_IC_check Flow >= 234.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.05

Rule f_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 256
AND LINK US_IC_check Flow > 245.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.1

Rule g_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 266
AND LINK US_IC_check Flow > 256
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.15

Rule h_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 276
AND LINK US_IC_check Flow > 266
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.2
```

Rule i_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 285.5
AND LINK US_IC_check Flow > 276
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.25

Rule j_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 294.5
AND LINK US_IC_check Flow > 285.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.3

Rule k_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 303.5
AND LINK US_IC_check Flow > 294.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.35

Rule l_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 312.5
AND LINK US_IC_check Flow > 303.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.40

Rule m_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 321.0
AND LINK US_IC_check Flow > 312.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.45

Rule n_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 328.5
AND LINK US_IC_check Flow > 321.0
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.50

Rule o_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 336
AND LINK US_IC_check Flow > 328.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.55

Rule p_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 343.5
AND LINK US_IC_check Flow > 336
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.60

Rule q_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 350.5
AND LINK US_IC_check Flow > 343.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.65

Rule r_open_Indian_Crk_check_automation

If LINK US_IC_check Flow <= 357
AND LINK US_IC_check Flow > 350.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.70

Rule s_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 363
AND LINK US_IC_check Flow > 357
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.75

Rule t_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 368.5
AND LINK US_IC_check Flow > 363
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.80

Rule u_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 373.5
AND LINK US_IC_check Flow > 368.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.85

Rule v_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 378.5
AND LINK US_IC_check Flow > 373.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.90

Rule w_open_Indian_Crk_check_automation
If LINK US_IC_check Flow <= 383
AND LINK US_IC_check Flow > 378.5
THEN ORIFICE Indian_Crk_gate3_auto Setting = 0.95

Rule x_open_Indian_Crk_check_automation
IF LINK US_IC_check Flow > 383
THEN ORIFICE Indian_Crk_gate3_auto Setting = 1.0

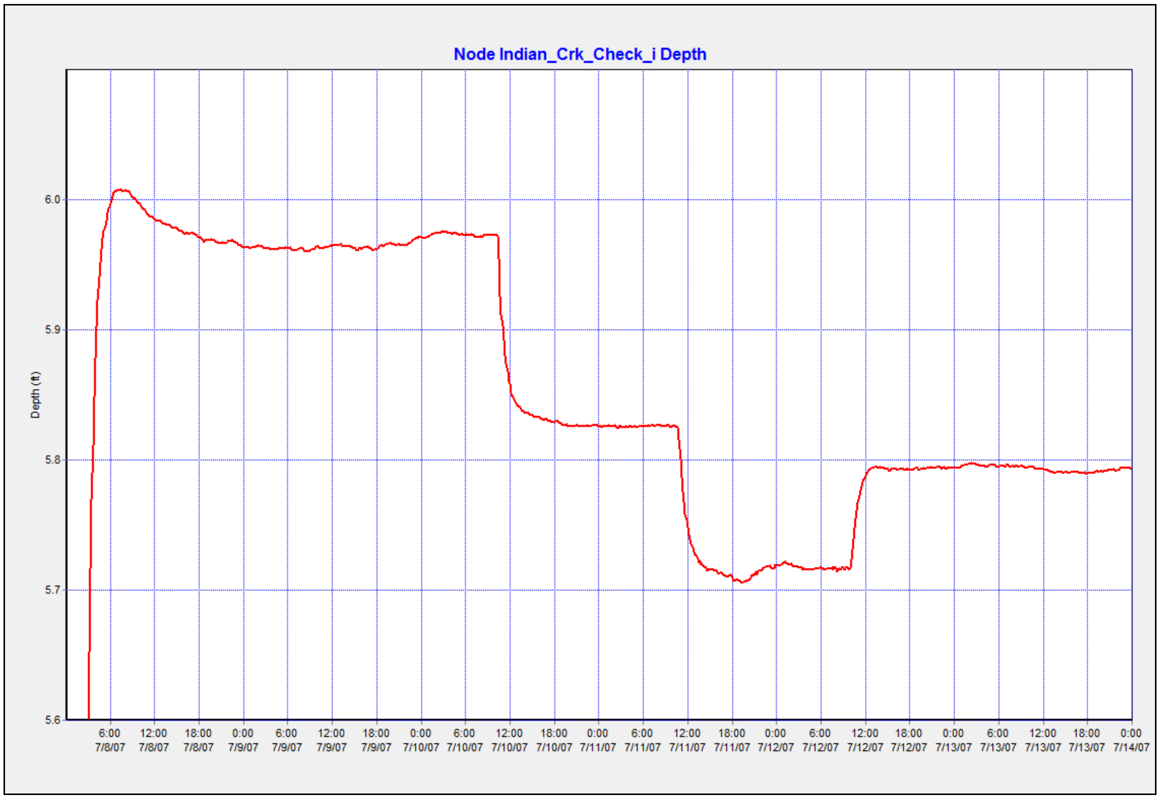

Figure 33. Upstream water level at Indian Creek check using control rules based on flow

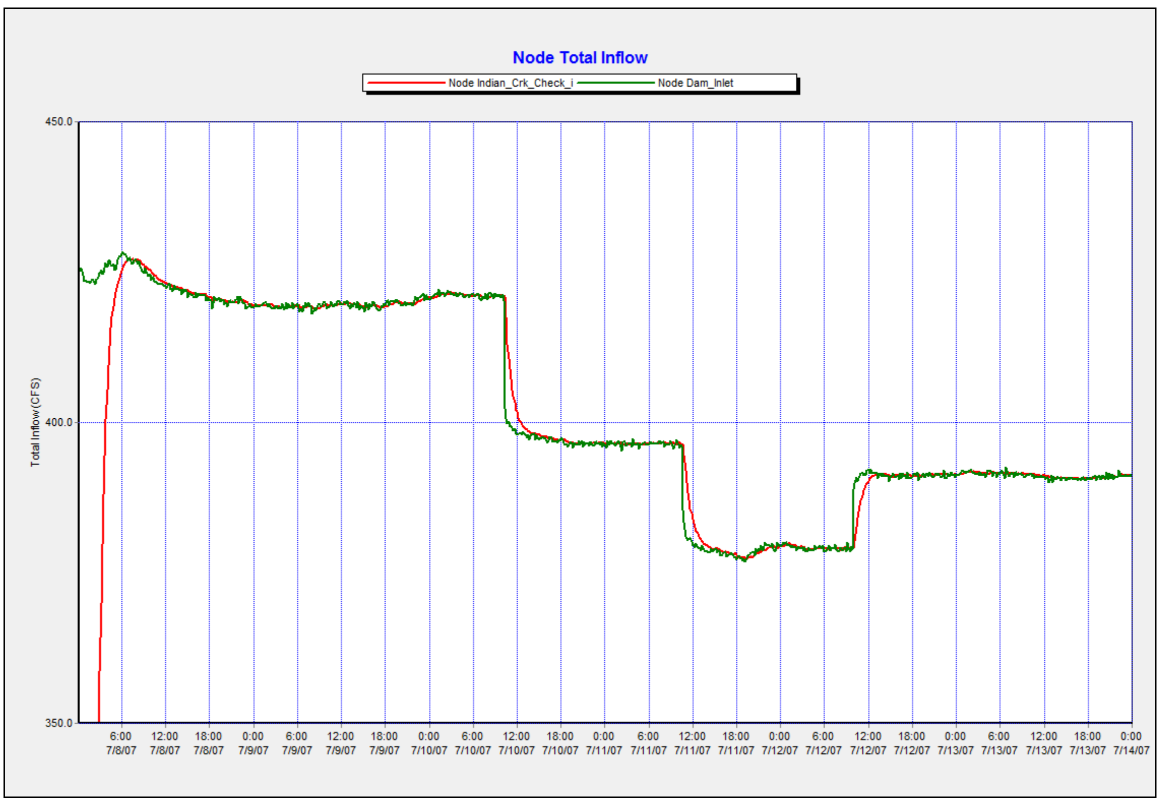

Figure 34. Dam release inflows and flow at Indian Creek check using control rules to define gate setting based on flow.

4.3.c Modulated Control Rule: Control Curve

A control curve can also be used to determine how the setting of a gate varies as a function of a control variable (such as water level or flow at a particular node). A control rule is developed by inputting a controller value and a controller setting into the control curve editor. The controller value is the variable (such as flow or water depth) that determines what the controller setting (or automated gate setting) will be set to. A control curve was developed using flow as the controller value and the automated gate setting (fractional opening) as the controller setting. The automated gate setting and the corresponding flow were again found using the operational curve/chart relationships (Table 13). The control curve is specified using modulated controls (EPA, 2008). Modulated controls are control rules that provide for a continuous degree of control applied to a flow regulator as determined by the controller variable. The modulated control used to specify the control curve is shown below.

```
RULE d_automated_gate_control_curve
If NODE Indian_Crk_check_i DEPTH >= 4.6
THEN ORIFICE Indian_Crk_gate3_auto SETTING = CURVE IC
PRIORITY 1
```

The control rule developed uses flow as the controller variable (the last conditional clause) and depth as a condition that must be met before the curve will be activated (the first conditional clause). This allows the automated gate to be stationary until the appropriate upstream water level is met. Once the upstream water level reaches 4.6 feet, the automatic gate setting will be based on the control curve “CURVE IC” (Figure 35).

The results show that the target depth of 4.6 was not held during the simulation (Figure 36). The flow pattern at the Indian Creek check (green line) is similar to the dam releases (red line), but has a lot of small oscillations from July 7th to July 10th (Figure 37). The status report generated by SWMM did not report any movements of the automated gate during the simulation time. For this reason, the target depth was not sustained.

Table 13. Control curve used to define automated gate setting based on flow directly upstream of the Indian Creek check.

Flow	Gate Setting
0	0.00
100	0.00
200	0.00
229	0.00
240	0.05
251	0.10
261	0.15
271	0.20
281	0.25
290	0.30
299	0.35
308	0.40
317	0.45
325	0.50
332	0.55
340	0.60
347	0.65
354	0.70
360	0.75
366	0.80
371	0.85
376	0.90
381	0.95
385	1.00
400	1.00
425	1.00
450	1.00
475	1.00

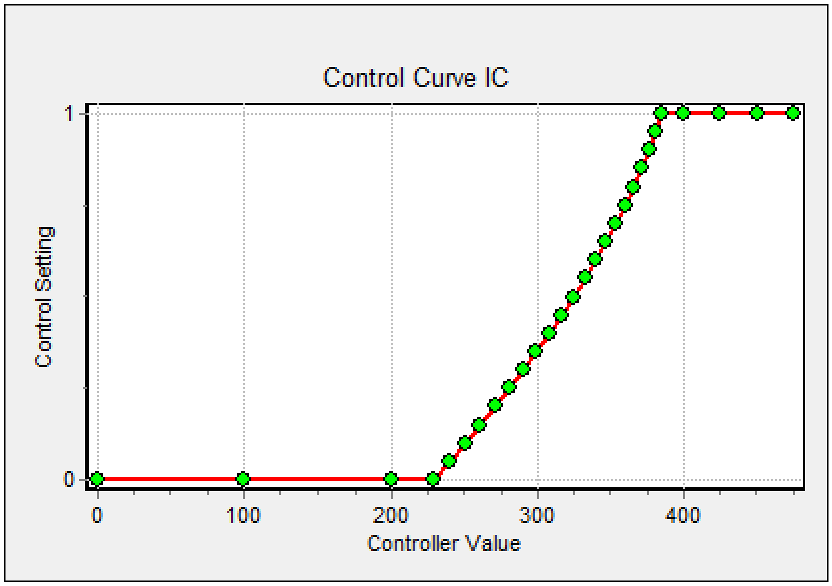

Figure 35. Control curve used to define the automateic gate setting based on flow.

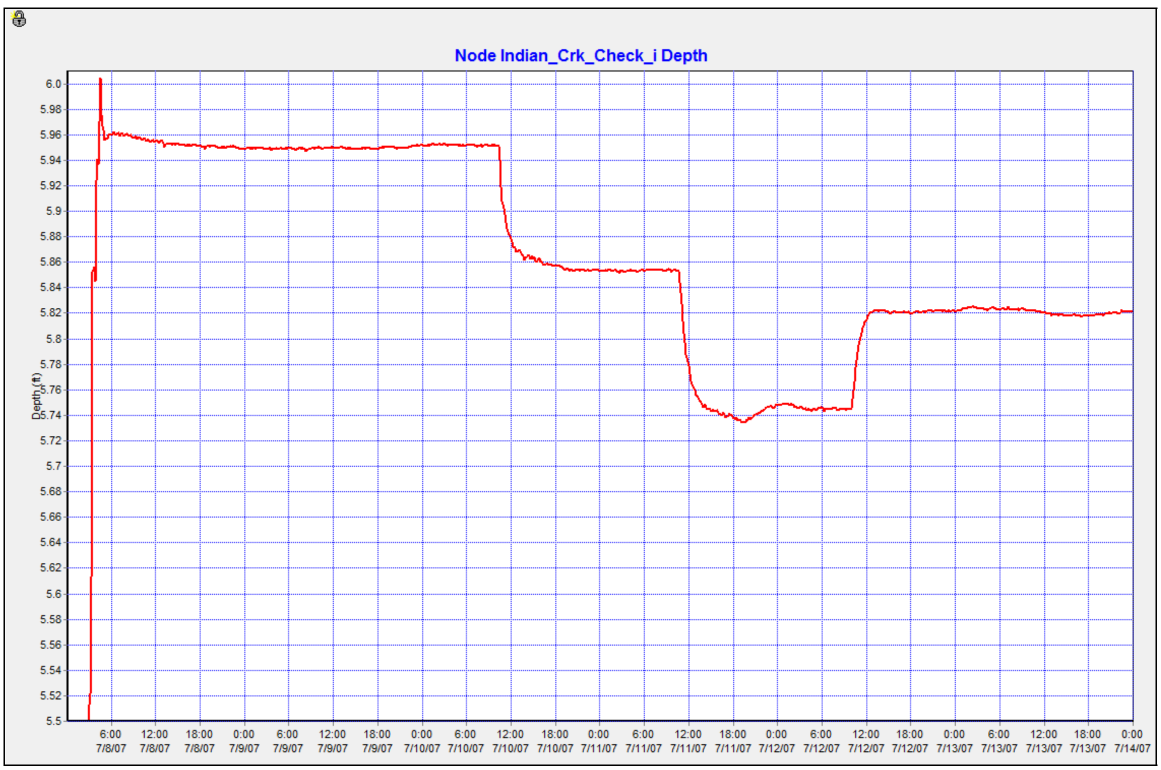

Figure 36. Upstream stage using control curve to define gate setting based on flow.

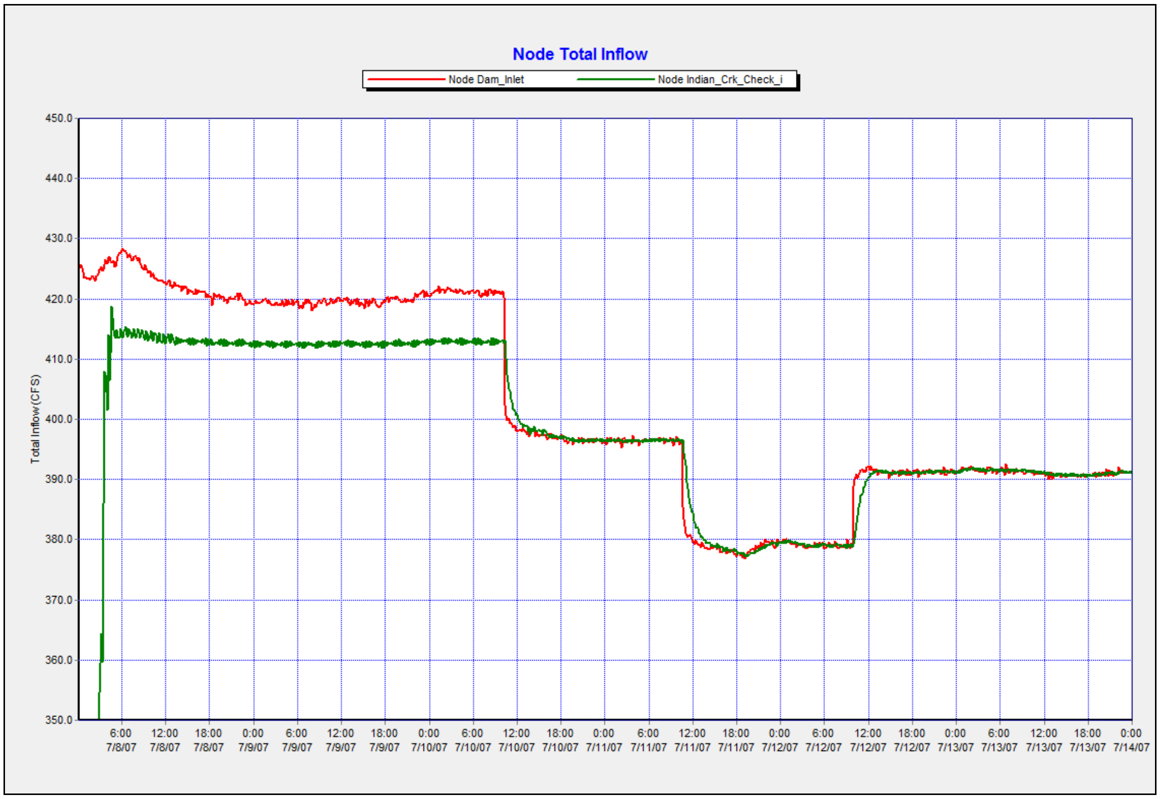

Figure 37. Dam release inflows and flow at Indian Creek check using control curve to define gate setting based on flow.

4.3.d Modulated Control Rule: PID Controller

A Proportional-Integral-Derivative (PID) controller is a generic closed-loop control scheme that tries to maintain a desired set-point on some process variable by computing and applying a corrective action that adjusts the process accordingly (EPA, 2008). A set-point is synonymous to a fixed target level. A PID controller can be used to adjust a gate setting in order to maintain a constant depth at a specified node. This feature was only available after March 2008 at which time the newest version of SWMM was released. For this reason, using a PID controller was not considered during previous modeling efforts.

The PID controller calculation (algorithm) includes three different terms: the proportional, the integral and the derivative (Equation 5). The sum of these three terms equals the output of the PID controller, m(t).

$$m(t) = P_{out} + I_{out} + D_{out}$$ **Equation 5**

Where:

m(t) = PID controller output –automated gate setting

P_{out} = proportional term output

I_{out} = integral term output

D_{out} = derivative term output

The classical PID controller has the form of Equation 6. The performance of a PID controller in SWMM is determined by the values assigned to the coefficients Kp, Ti, and Td.

$$m(t) = K_p \left[e(t) + \frac{1}{T_i} \int e(\tau) d\tau + T_d \frac{de(t)}{dt} \right]$$ **Equation 6**

Where:

m(t) = PID controller output – automated gate setting

K_p = proportional coefficient

e(t) = error (difference between setpoint and observed value)

T_i = integral time (minutes)

T_d = derivative time (minutes)

t = time or instantaneous time

τ = time in the past contributing to the integral response

4.3.d.1 Proportional Term

The proportional term makes changes to the output that is proportional to the current error term. The proportional term adjusts the output by multiplying the current error term by the proportional coefficient as shown in Equation 6. For the case in hand, the automated gate setting would be the controller output (or m(t)), the error (or e(t)) would be the difference between the upstream target level (defined by the BFID) and the actual upstream water level measured at a specified check structure.

$$P_{out} = K_p\left[e(t)\right] \qquad \textbf{Equation 6}$$

Where:

P_{out} = proportional term output

K_p = proportional coefficient

e(t) = error (difference between setpoint and observed value)

t = time or instantaneous time

Tuning theory and industrial practice both indicate that the proportional term should be the largest contributor to output change (Cooper, 2008). Fine tuning of the proportional coefficient must be done in order to optimize the PID controller. A high proportional coefficient will result in a faster response and a larger change in the automated gate setting output (or m(t)). If the proportional coefficient is too high, the system can become unstable resulting in oscillations around the setpoint. A low proportional coefficient will result in a smaller output change and the controller would therefore be less responsive to system changes (Cooper, 2008).

4.3.d.2 Integral Term

The integral term and the derivative term both offer finer tuning of the controller output. The integral term tries to correct the accumulated error that should have been corrected previously as shown in Equation 7. This can cause the present value to overshoot the setpoint by crossing over the setpoint and then creating an error in the opposite direction. Fine tuning must also be done to the integral term to stabilize the PID controller.

$$I_{out} = K_p\left[\frac{1}{T_i}\int e(\tau)d\tau\right] \quad or \quad I_{out} = K_i\int e(\tau)d\tau \qquad \textbf{Equation 7}$$

Where:

I_{out} = integral term output

K_i = integral coefficient = K_p/T_i

T_i = integral time (minutes)

$e(\tau)$ = error (difference between setpoint and observed value)

τ = time in the past contributing to the integral response

4.3.d.3 Derivative Term

The derivative term in the PID controller slows the rate of change of the controller output by using Equation 8. The effect of the derivative term is most noticeable when the present value is close to the setpoint. This helps reduce the magnitude of overshooting the setpoint produced by the integral component. It also helps improve the combined PID controller stability. The differentiation amplifies any noise within the system. This makes the controller much more sensitive to noise in the error term and can cause the process to become unstable (Cooper, 2008).

$$D_{out} = K_p \left[T_d \frac{de(t)}{dt} \right] \qquad or \qquad D_{out} = K_d \frac{de(t)}{dt} \qquad \textbf{Equation 8}$$

Where:

D_{out} = derivative term output

K_d = derivative coefficient (gain) = K_p* T_d

T_d = integral time (minutes)

$e(\tau)$ = error (difference between setpoint and observed value)

τ = time in the past contributing to the integral response

4.3.d.4 Tuning PID parameters

A PID controller can be adjusted to suit specific processes by adjusting the coefficients in each of the three PID parameters. This is referred to as tuning the control loop. In SWMM, the coefficients that are adjusted are Kp, Ti, and Td. It is possible to use only one or two of the PID parameters. If this is the case the controller is called either a

PI, PD, P or I controller. Unstable or oscillating results can arise as a consequence of incorrect PID parameters.

There are various different ways to tune a control loop. Manual tuning methods can be inefficient and very time consuming even with experienced personnel. There are various other methods and software that are used in tuning PID parameters some of which have costs and involve training. Table 14 gives a summary of the resulting factors that arise from adjusting the PID parameters. Within the BFID, a P controller is currently programmed into the dataloggers to adjust the automated gate settings. As a means of simplification, a P controller was chosen to be developed and manually tuned for this SWMM model. Manual tuning was accomplished by setting both the I and D parameters to zero and increasing the Kp value to a point where oscillations just start to occur in the results. This Kp value was then divided by two to obtain the final Kp value. This is a common means of tuning the Kp value. Note that the Kp value is negative since there is an inverse relationship between water level and the automated gate setting (i.e. as the automated gate opens, water level decreases).

Table 14. Summary of simulation responses to the adjustments of PID coefficients, Kp, Ki and Kd.

Coefficient	Adjustment	Advantages	Disadvantages
Proportional (Kp)	Larger	faster response	instability and oscillation
	Smaller	smaller output change	less responsive
Integral (Ki)	Larger	steady state errors eliminated quicker	larger overshoot
Derivative (Kd)	Larger	decreases overshoot	slows down response, instability due to noise

4.3.d.5 PID Control Rule

A PID controller is signaled by defining a PID parameter set in a modulated control rule within SWMM. A PID parameter set contains three values: a proportional coefficient, an integral time (in minutes), and a derivative time (in minutes). Also, by convention the controller variable used in a Control Curve or PID Controller will always be the object and attribute named in the last condition clause of the rule (EPA, 2008). The following is an example of a PID modulated control rule.

```
RULE d_PID_Indian_Creek_automated_gate
   IF NODE Indian_Crk_check_i DEPTH <= 4.65
   OR NODE Indian_Crk_check_i DEPTH >= 4.75
   THEN ORIFICE Indian_Crk_gate3_auto SETTING = PID -70.0 0.0 0.0
```

This control rule is for the Indian Creek Check in which the current upstream target level is 4.7 feet. A deadband of ±0.05 feet is defined which leaves two setpoints: 4.65 feet and 4.75 feet. The PID parameter set contains a value of -70.0 for the proportional gain and 0.0 minutes for both the integral time and derivative time as they were not considered in this analysis. This control rule was tested on the reach from the Dam outlet to the Indian Creek automated check which is the first check downstream of the Dam. This reach was unhooked from the rest of the system to reduce interferences from the rest of the system. Figure 38 and Figure 39 show the modeled flow and upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -10.0. The first six hours of the simulation consists of a stabilization period as the canal storage fills. Figure 40 and Figure 41 show the modeled flow and upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 1.0 hours and a Kp value of -70.0. Figure 42 and Figure 43 show the modeled flow and upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -70.0. The results for flow are very similar for each scenario. Differences can be noticed in the upstream depths. In order to appropriately simulate the depths, the time to open/close each automated gate should be measured in the field and entered into the property dialog box in SWMM. The Kp value would be a parameter to tweak during the calibration process

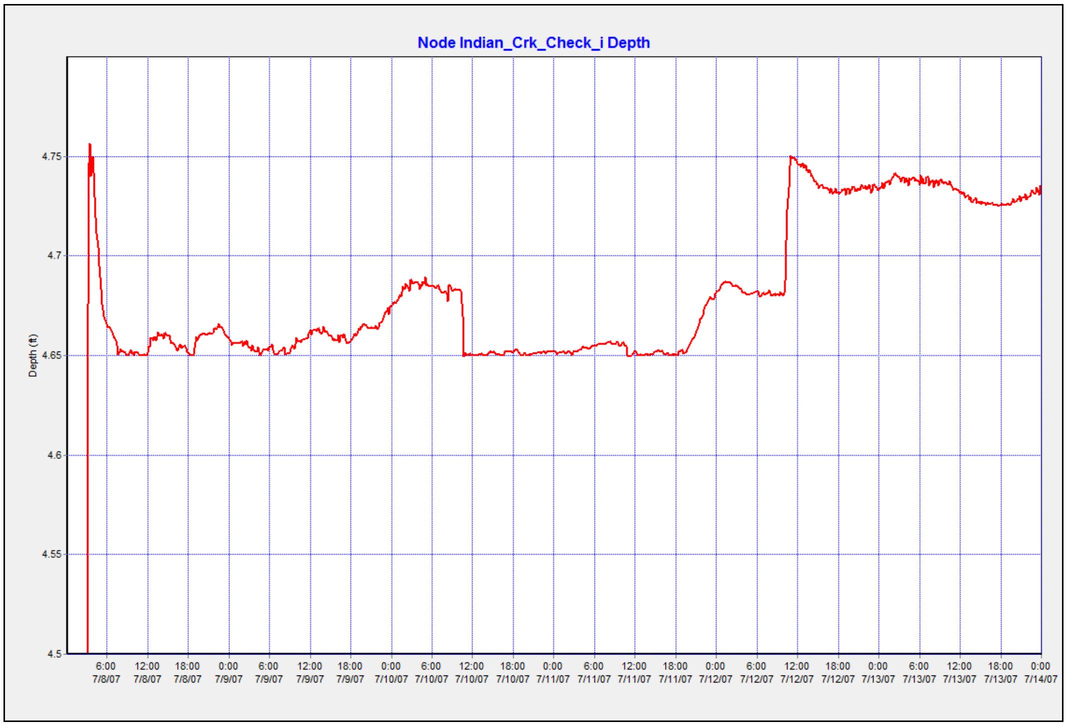

Figure 38. Modeled upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -10.

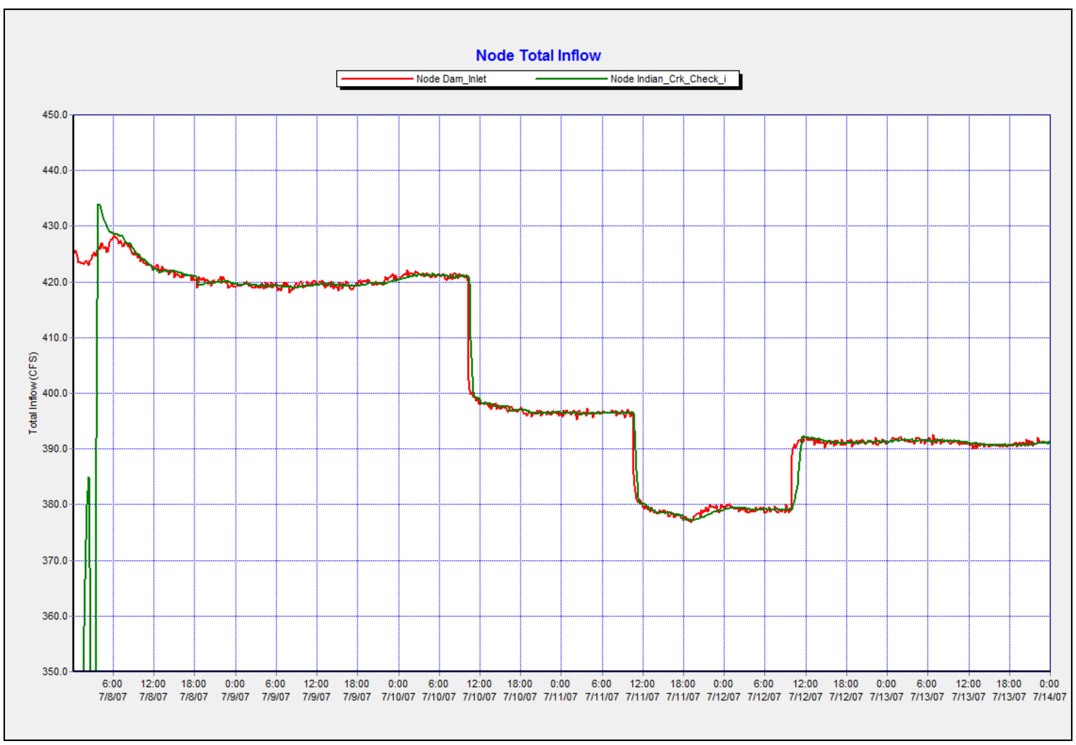

Figure 39. Modeled flow as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -10.0.

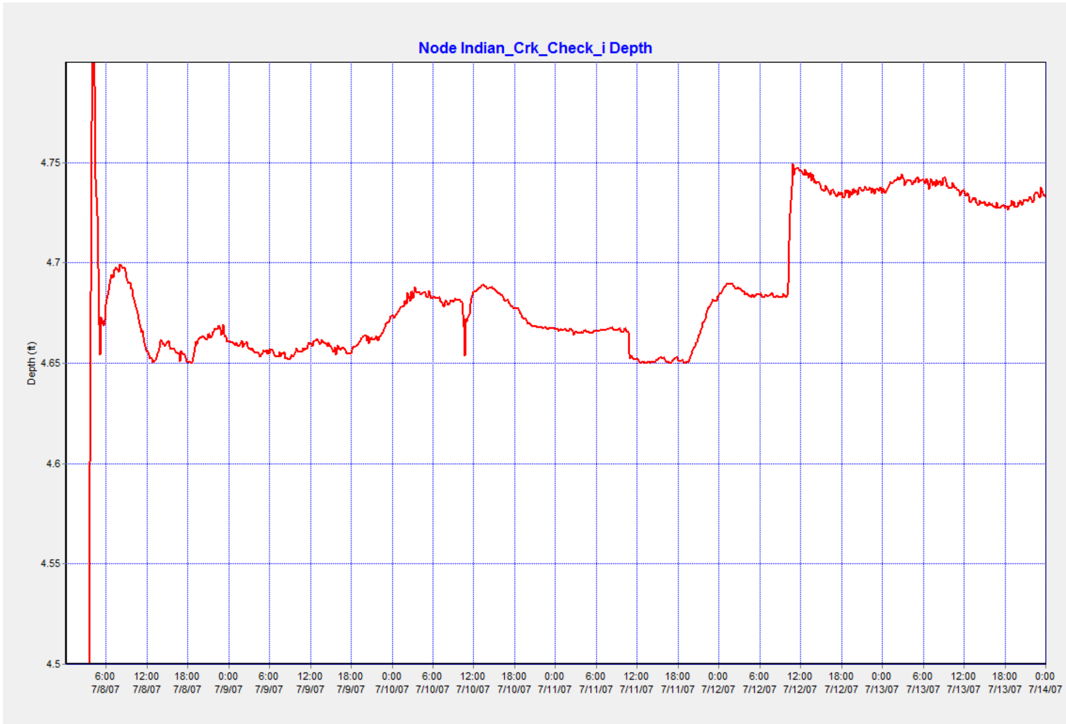

Figure 40. Modeled upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 1.0 hours and a Kp value of -70.0TTOC = 1.0 hrs, Kp = -70.0

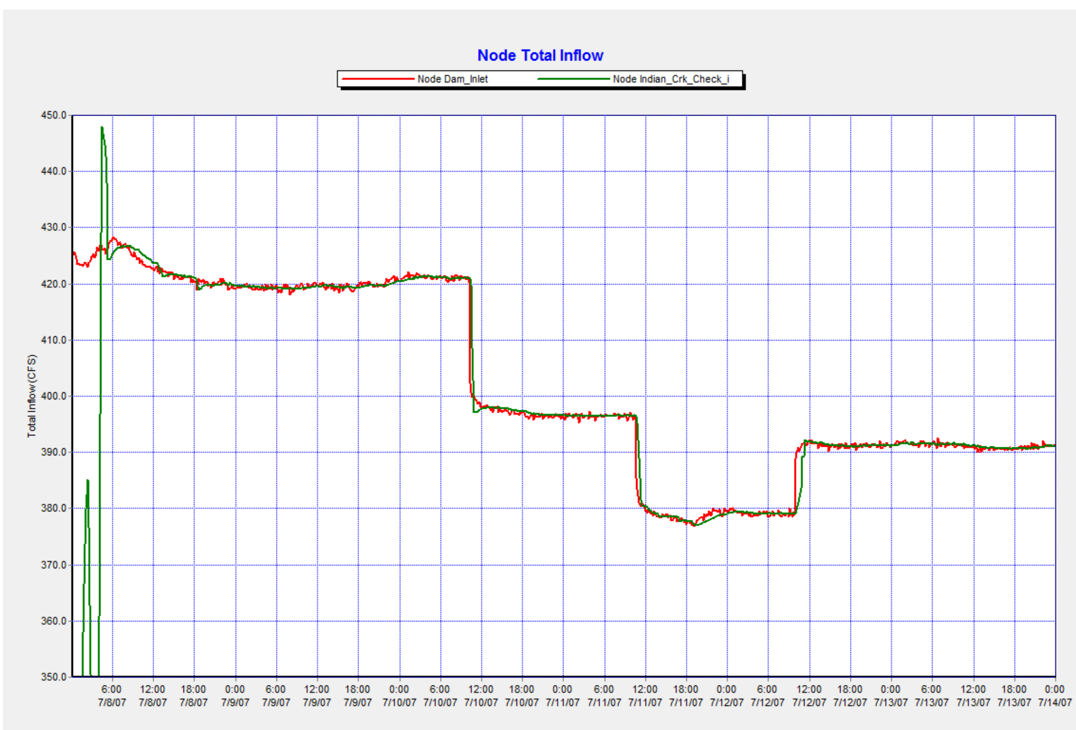

Figure 41. Modeled flow as a result of the PID modulated control rule with a time to open/close the automated gate equal to 1.0 hours and a Kp value of -70.0.

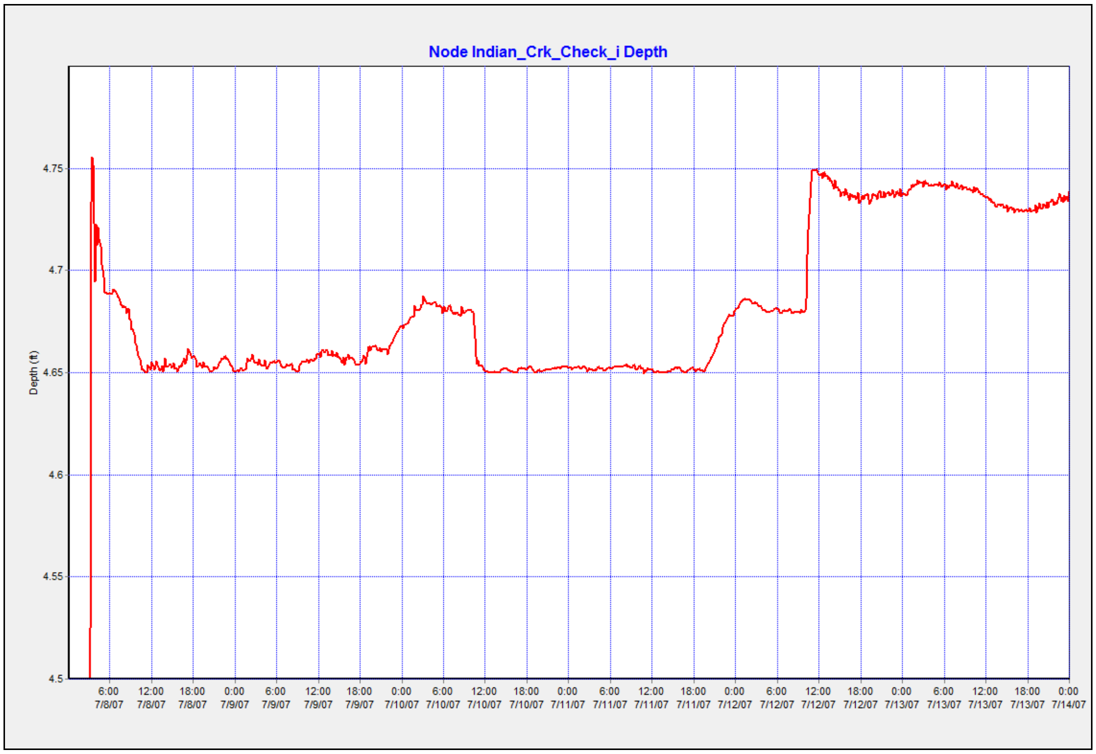

Figure 42. Modeled upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -70.0

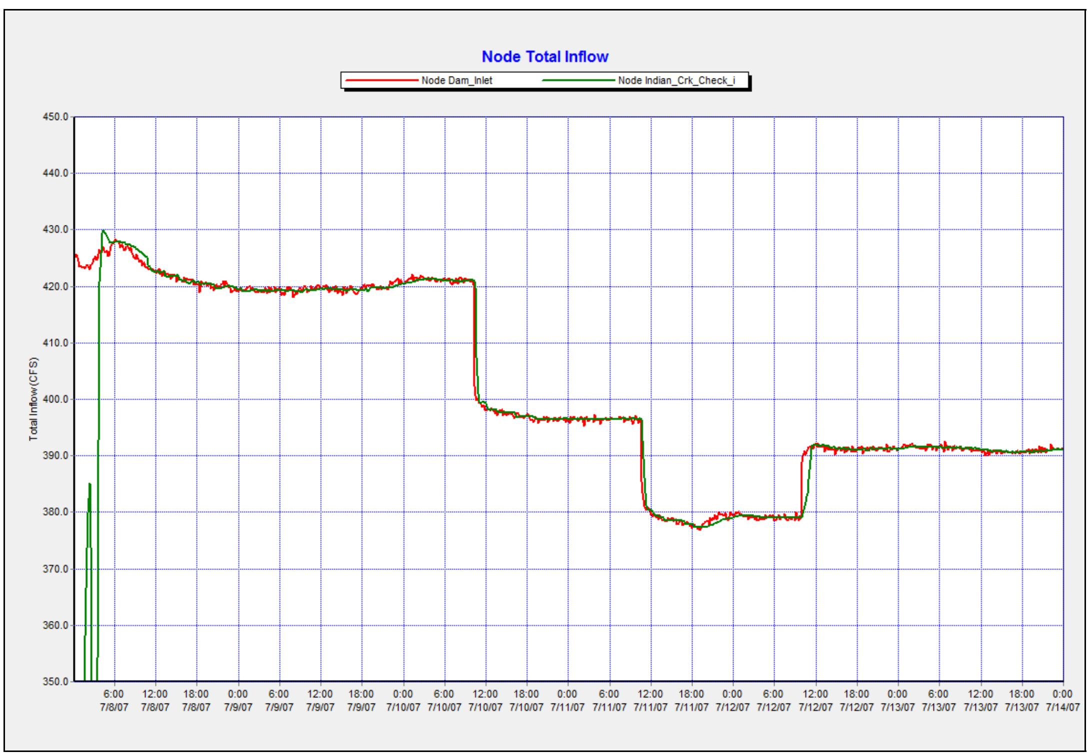

Figure 43. Modeled flow upstream water depth as a result of the PID modulated control rule with a time to open/close the automated gate equal to 0.25 hours and a Kp value of -70.0.

4.3.e Summary of Automated Gate Simulation Options

After analyzing the results from all of the automated gate simulation options, the decision was made to simulate automated gates using PID modulated control rules. The PID modulated control rule seemed to be the most reliable and realistic way to represent the gate operations in the BFID. It also provided the best results with the smallest amount of control rule manipulation. The time to open/close the gate was set to 0.25 hours which represents typical field operations in the BFID. The Kp values were initially set to 50 which seemed to be an intermediate point in preliminary investigations. Adjustment of the Kp value could be used to calibrate the model.

4.4 Contributing Factors to Simulation Instability

The flow in some links and/or water depths at some nodes may fluctuate or oscillate significantly at certain periods of time as a result of numerical instabilities produced by the solution method which is due to the explicit nature of the numerical methods used for Dynamic Wave routing (EPA, 2008). Reducing numerical instabilities was one of the largest obstacles to overcome while developing this model. There are a number of factors that can contribute to numerical instability. It was found that several different factors can simultaneously contribute to numerical instability. The frequency and amplitude of the oscillations vary depending on the number of contributing factors and the severity of each contributing factor. Numerical instabilities were minimized by:

- Defining an initial flow of 100 cfs in conduits
- Defining an initial depth of 2 feet at junctions

- Defining the default minimum node surface area in the dynamic wave routing options (EPA, 2008)
- Using the smallest routing time step necessary to avoid oscillations (2 sec)
- Selecting to ignore the inertial terms of the momentum equation producing what is essentially a Diffusion Wave solution (EPA, 2008)
- Adjusting ("tuning") the Kp values of automated check structures
- Selecting the option to lengthen short conduits within the dynamic wave options (EPA, 2008)
- Combining short conduits with adjacent conduits with appropriate lengths (which involved adjusting the location of some turnouts and laterals)

4.5 SWMM Model Calibration Process and Results

Calibration of the SWMM model was done in two different components. The first component consisted of conducting a water balance to correctly model the observed flows within the particular reach being calibrated. The second step was to correctly model the observed stages, upstream and downstream at each check structure, within the reach. The overall goal was to model stage and flow within an acceptable range of ±10 percent (centered around zero percent) of the observed values.

The first step was completed using the operational curves/charts developed for key automated and unautomated check structures, existing flow measuring devices, and water orders recorded or observed on the North Canal. The Beehive flume real-time data was only used as a comparison due to the high degree of submergence that occurred during the 2007 calibration year. The North Canal flume real-time data was used to specify the initial inflows into the model. Because changes to the system are continually being made and field measurements were only collected at one particular time of the day, assumptions were required. It was assumed that changes in Dam releases were made at 9 am each morning and that all the structure settings were made at 9 am the previous day (Schoenfelder, 2006) unless circumstances proved differently. The control rules in SWMM were set according to these assumptions.

Discharges from each reach were estimated based on field measurements and the real-time irrigation system which provides specific delivery amounts for *some* of the larger laterals and a sum of the remaining water orders for each ride. While developing the South Canal model, Schoenfelder (2006) used a water call spreadsheet for the South Canal and field measurements to estimate the flow going out of each turnout. A detailed water call spreadsheet for all turnouts and laterals on the North Canal was not available. Therefore, the real-time irrigation system was used to define flow out of large laterals but not the entire system. The real-time irrigation system report what *should* have been delivered based on the water cards, which may not have been the case in reality according to discussions with BFID personnel. Field measurements of headgate openings were used to estimate the amount of flow through smaller laterals and turnouts. These are rough estimates and could affect the results of the model. The deliveries out of large laterals (reported by the real-time irrigation system) were pumped out at the appropriate locations. A type 4 pump curve was developed for a constant flow and varying inlet node depth.

Water orders from the larger laterals were simulated using a pump and a set of control rules. All pumps were assigned a TYPE 4 pump curve which assigns flow to varying depths. A single pump curve with a constant flow of 1 cfs was assigned to all laterals (Figure 44). The control rules that were developed define a setting (or multiplier) for a specified pump curve based on the date and time, as shown below. Control rules were used as opposed to developing a different pump curve for daily water orders which could be time consuming to the user. The first control rule named "Rule m_Young_Lat_water_orders" takes 12 cfs from the Young lateral starting on July 10 at nine o'clock. This flow will continue to be taken out of the system until July 11 when the control rule named "Rule m_Young_Lat_water_orders" defines a new flow of 15 cfs to be taken out of the system.

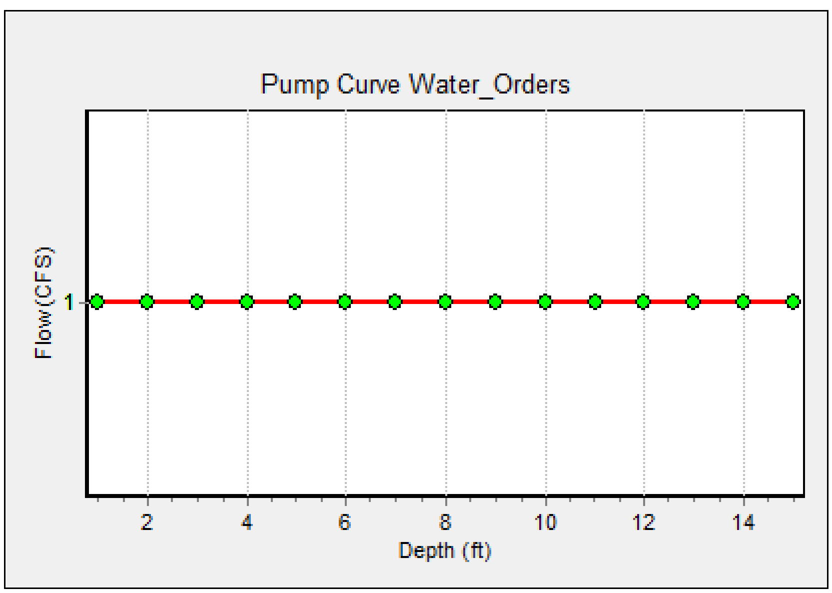

Figure 44. Pump curve used to simulate water orders from larger laterals.

Rule m_Young_Lat_water_orders
IF SIMULATION DATE = 7/10/2007
AND SIMULATION CLOCKTIME = 09:00
THEN PUMP Pump_Young_Lat SETTING = 12

Rule n_Young_Lat_water_orders
IF SIMULATION DATE = 7/11/2007
AND SIMULATION CLOCKTIME = 09:00
THEN PUMP Pump_Young_Lat SETTING = 15

Some water orders on laterals with real-time flow data during the calibration period were simulated using modulated control rules. The real-time flow data was entered as a time series. The pump was assigned a setting based on the input time series, as shown below. This control rule would only be useful if the user wanted to simulate past events otherwise a constant setting defined within a control rule would be more appropriate.

Rule h_Indian_Crk_Lat_water_orders
IF SIMULATION DATE = 7/10/2007
AND SIMULATION CLOCKTIME = 09:00
THEN PUMP Pump_Indian_Crk_Lat SETTING = TIMESERIES Indian_Crk_Lat

Water orders out of farmer turnouts and smaller laterals were simulated by adjusting the orifice setting (or percent open). Initially SWMM defaults to a setting of 1 (or 100 percent open) until a control rule specifies differently. The orifice settings were set to 0 at the start of the simulation with the use of a control rule. Once steady-state conditions were obtained, new control rules were engaged that assigned new orifice settings to correspond with the stem heights recorded in the field during the calibration period.

4.5.a Reach 1

4.5.a.i Issues and Assumptions

Calibration efforts on reach one began by further separating the reach into smaller "check reaches". This was done by breaking reach one apart after each check structure. This allowed any instability issues or errors within the check reaches to be identified. Flow recorded at the North Canal Dam flume during the 2007 calibration period was input at the most upstream node of each check reach. The flow results were analyzed and an appropriate Kp value was assigned to each automated check to minimize oscillations (Table 15).

Table 15. Kp values assigned to automated check structures in Reach 1.

Automated Check	Kp Value
Indian Creek	-30
Laflemme	-10
Capp	-10
Young	-10

Numerical instabilities due to small conduit lengths were also addressed. After completing the South Canal model, Schoenfelder (2006) recommended that model development data be as complete as possible. Following Schoenfelders' recommendations, more detailed information was collected from the BOR survey data. This included recording channel side slopes as opposed to assuming a side slope for the entire canal. Data was also collected at more frequent stations as opposed to just the laterals, turnouts, and check structures. Although this was a good recommendation, it resulted in smaller conduits and more junctions which caused numerical instability in some areas. Conduits with problematic small lengths were combined with adjacent conduits. The new lengthened conduit was assigned the channel dimensions of the longest conduit being combined. Laterals and turnouts that were affected by these adjustments were moved to the nearest junction. The option to lengthen conduits within the dynamic wave routing options was also selected to correct any minor lengthening that might be necessary. Manual adjustments were made to the system in order to reduce the amount of lengthening required in order to obtain stable results. These adjustments could skew the results slightly, but were necessary to best represent the actual canal length and reduce numeric instability.

Once the Kp values were found and numeric instability was reduced, the small check reaches were connected to represent reach one in its entirety. The same process was followed to identify any remaining errors and instabilities. The real-time dam releases were input at the North Canal dam node and estimated system losses were pumped out of the system upstream of the corresponding check structures.

When the results were stable, control rules were developed to simulate the water orders based on the field measurements obtained during the calibration period. Water orders for some of the larger laterals were taken out of the system using a pump and appropriate control rules. The amount of water released from these laterals was based on the water orders recorded on the water call cards by the BFID. Water orders released from smaller laterals and single farmer turnouts were based on the stem heights recorded during the calibration period as opposed to referencing the water call cards. This was because the water call cards report water orders directly off of the North Canal solely on the farmers name instead of the name of the turnout. The water call cards were used as a comparison to see if the total amount of water SWMM was releasing from the system was similar to the total water orders delivered directly off of the North Canal, as reported on the water call cards. Water orders delivered in the field may not always match what was recorded on the water call cards due to the difficult estimations that ditch riders must make as well as the fluctuations in water depths which creates variable head pressures resulting in inconsistent flow going out the turnouts. Some of the water orders were adjusted slightly to obtain a water balance in the entire reach. A water balance was also performed at automated checks within the reach that had real-time stage data and an operational spreadsheet developed.

The Indian Creek Siphon is the only siphon within Reach 1. Survey data of the siphon was unable to be located therefore the siphons invert elevations was predicted based on model results and visual field estimates. Not having the exact invert elevations proved troublesome during calibration and could have led to inaccuracies in the results.

4.5.a.ii Results and Discussion

The calibration period for reach one in 2007 consisted of only two days: July 12 and July13. These dates were modeled along with a startup period of 6 days in order to fill the canal and build the water levels up at the check structures. The flow that was modeled by SWMM at the Indian Creek Check (which is the first check downstream of the Dam) was consistently higher than the flow that was predicted by the operational spreadsheet (Figure 45). The average percent difference between the modeled and the predicted flows was 2.43 percent with a maximum difference of 5.92 percent and a minimum difference of 1.24 percent. The results at the Indian Creek check are very reasonable considering the amount of flow that passes through that check and the variability in the upstream reach. The results are within the acceptable range of ±10 percent but the values are not centered on zero (i.e., they are consistently higher than the predicted flows). It is important to note that the operational spreadsheets were not validated prior to calibrating the SWMM model which could lead to inaccuracies in the results.

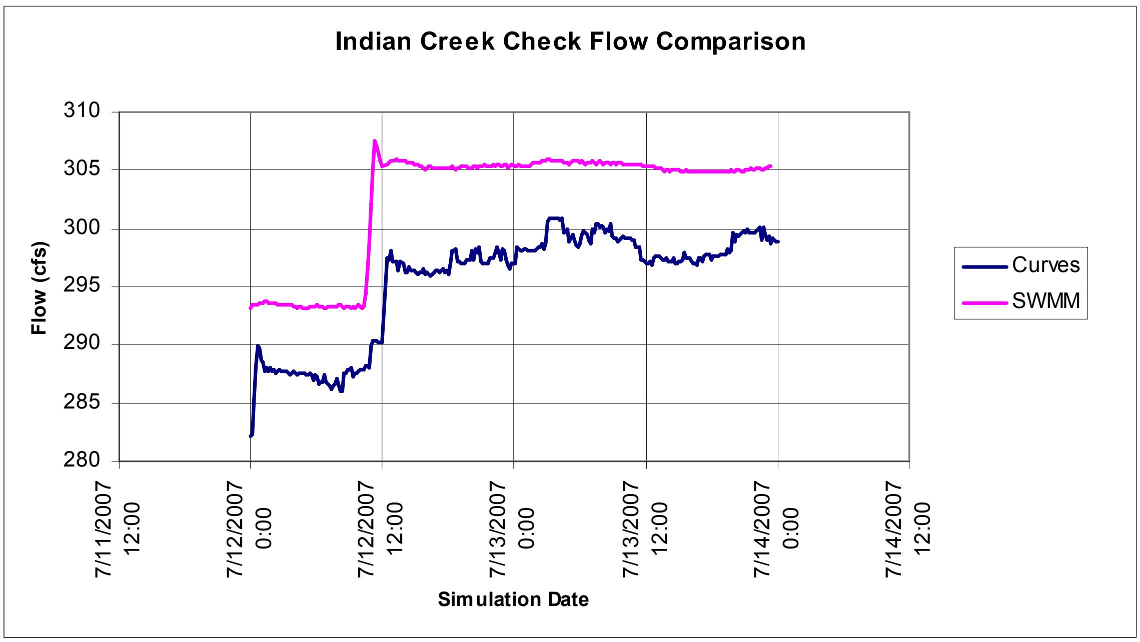

Figure 45. Comparison of flow predicted by the operational spreadsheet and the flow modeled by SWMM at the Indian Creek check structure.

There are two large laterals that are located between the dam and the Indian Creek check which also could have caused variance in the water balance. The first lateral is the Gillette lateral which has a flume located just downstream on the lateral. The flume reading during the calibration period was not reliable due to the high level of submergence. For this reason, the amount of water released from the Gillette lateral was based on the stem height measured in the field. This may have induced some errors as the downstream conditions of the laterals and turnouts were not modeled. The Indian Creek lateral is the largest lateral off of the North Canal and is located just upstream of the Indian Creek check. Indian Creek lateral releases are controlled by a newly installed real-time automated constant head orifice (CHO). The real-time CHO data only provided an estimate of the amount of flow passing through the CHO. A parshall flume provides flow measurement on the Indian Creek lateral, but it takes approximately 12 hours for the water released from the CHO to reach the flume. There were differences between the flume and the CHO readings during the calibration year. The flume data was thought to be more reliable than the CHO estimate. Therefore, a pump was used to simulate a constant flow into the Indian Creek lateral based on the flume data. Flow data from the flume was adjusted by 12 hours to account for the estimated water travel time. The water balance from the dam to the Indian Creek check could have been skewed due to the simplifications and assumptions made for the Gillette and Indian Creek laterals.

The flow predicted by the model at the Young check was consistently lower than the flow predicted by the operational spreadsheets (Figure 46). This opposes the Indian Creek check comparison which was consistently higher. The average percent difference between the modeled and the predicted flows at the Young check was 2.43 percent with a maximum difference of 5.92 percent and a minimum difference of 1.24 percent. The values are within the acceptable range of ±10 percent but the values are not centered on zero.

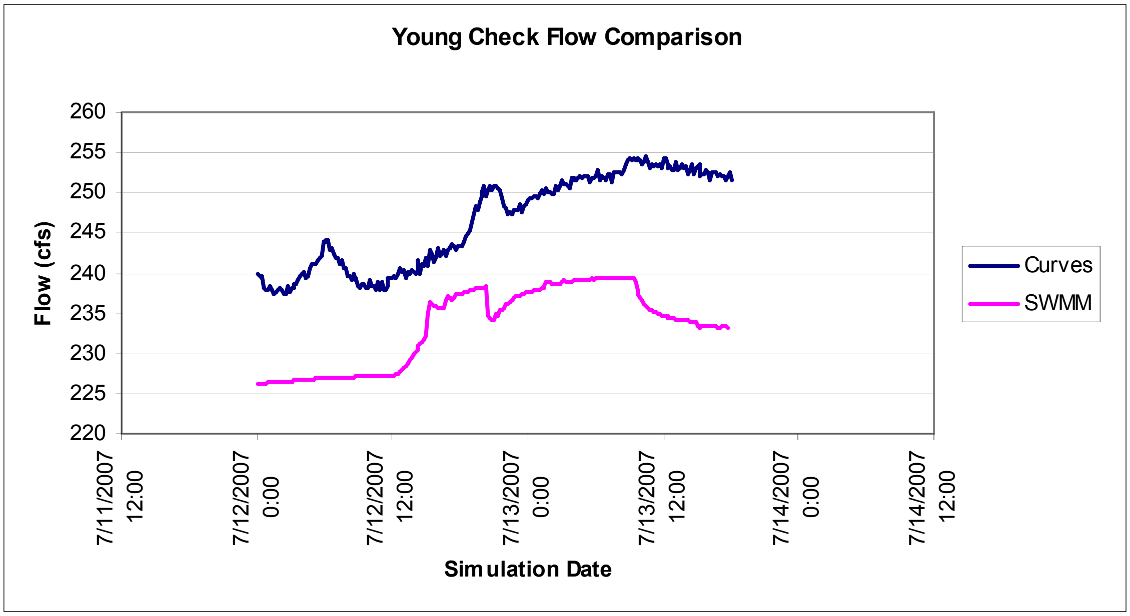

Figure 46. Comparison of flow predicted by the operational spreadsheet and the flow modeled by SWMM at the Young check structure.

The variability in the water balance for Reach one could be due to multiple factors. As discussed previously, the Gillette and Indian Creek laterals could easily have altered the flow at the Indian Creek Check. The amount of water orders that were pumped down laterals were based on the water call cards. These are not always accurate as there are inevitable fluctuations that are not recorded on the water call cards. The water orders that were released based on stem heights measured in the field can be erroneous since the downstream conditions on the laterals and delivery ditches were not modeled. Primed laterals are usually 100 percent open along the main North Canal and are controlled by a downstream lateral control box. Field measurements were only taken along the main North Canal thus flow measurements on the turnout/lateral downstream control box were unavailable. Some primed laterals were identified but others may not have been. This could overestimate the amount of water being released from the North Canal.

Manning’s n values and check structure weir and orifice discharge coefficients were manipulated during the calibration process to match the modeled water levels to the observed water levels. The Manning’s n values for roughness were slightly lower than expected in some areas (Table 16). Some of the weir discharge coefficients are also outside of the expected range of 2.6 to 3.4 (Table 17). These factors may be outside of the expected range because all of the assumption errors as well as changes in the channel geometry are absorbed by these coefficients.

Table 16. Manning's n values assigned to conduits representing canal regions in Reach 1

Canal Section	Manning's n
Dam to Indian Creek	0.0103
Indian Creek to Gregory	0.0114
Gregory to Laflemme	0.0103
Laflemme to Indian Creek Siphon	0.0103
Indian Creek Siphon to Capp	0.013
Capp to Young	0.01

Table 17. Discharge coefficients assigned to checks in Reach 1.

	Discharge Coefficients	
Check Structure	Weir	Orifice/Gate
Indian Creek	4.156	0.5544
Gregory	5	0.669
Laflemme	5	1
Indian Creek Siphon	4	0.7
Capp	3.417	0.484
Young	2.8	0.6

Automated check structures along Reach 1 were all modeled to hold a target range based on the specified target depth with a deadband of 0.05 feet. The target range was held by the automated gate PID controller with specified Kp values (Table 18). Comparisons were made between the modeled upstream depths and the upstream depths recorded by a datalogger for each automated check along Reach 1 (Figure 47 to Figure 50). The target range for each individual check is labeled within the Figures. SWMM reads the PID control rule every time step (two seconds). This provides a much smoother transition compared to the field adjustments which are made every ten minutes (in order to conserve the solar battery power).

Table 18. Kp values entered into the PID control rules for automated checks within Reach 1.

Automated Check Structure	Kp Value
Indian Creek	-70
Laflemme	-20
Capp	-25
Young	-30

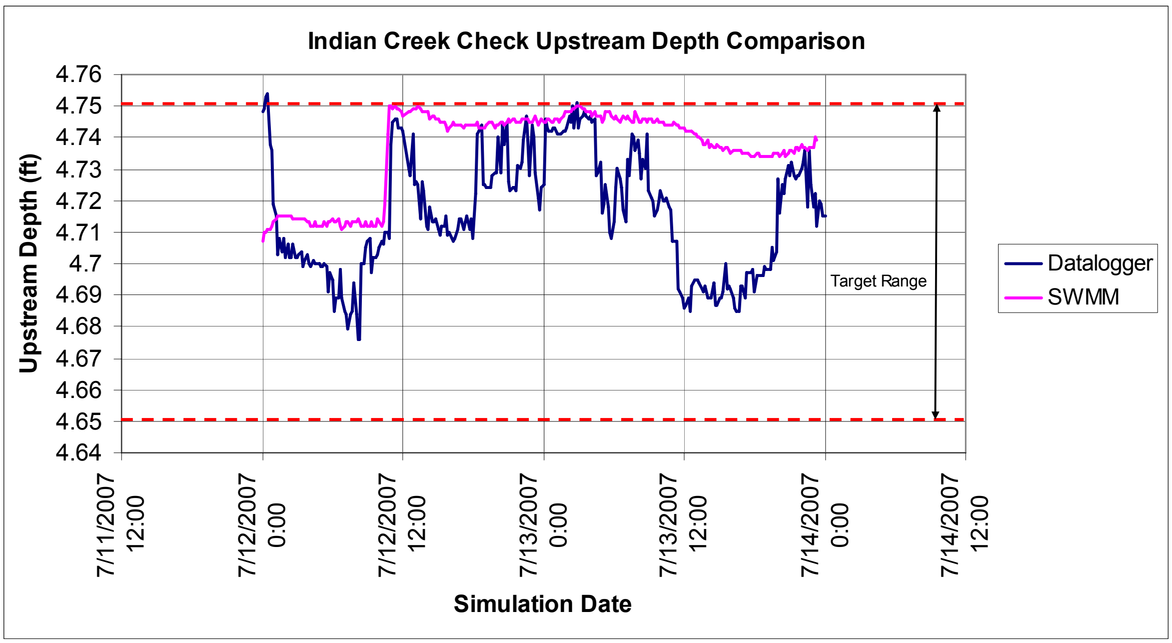

Figure 47. Comparison of upstream water depths recorded in the field by the datalogger and the water depths modeled by SWMM at the automated Indian Creek check.

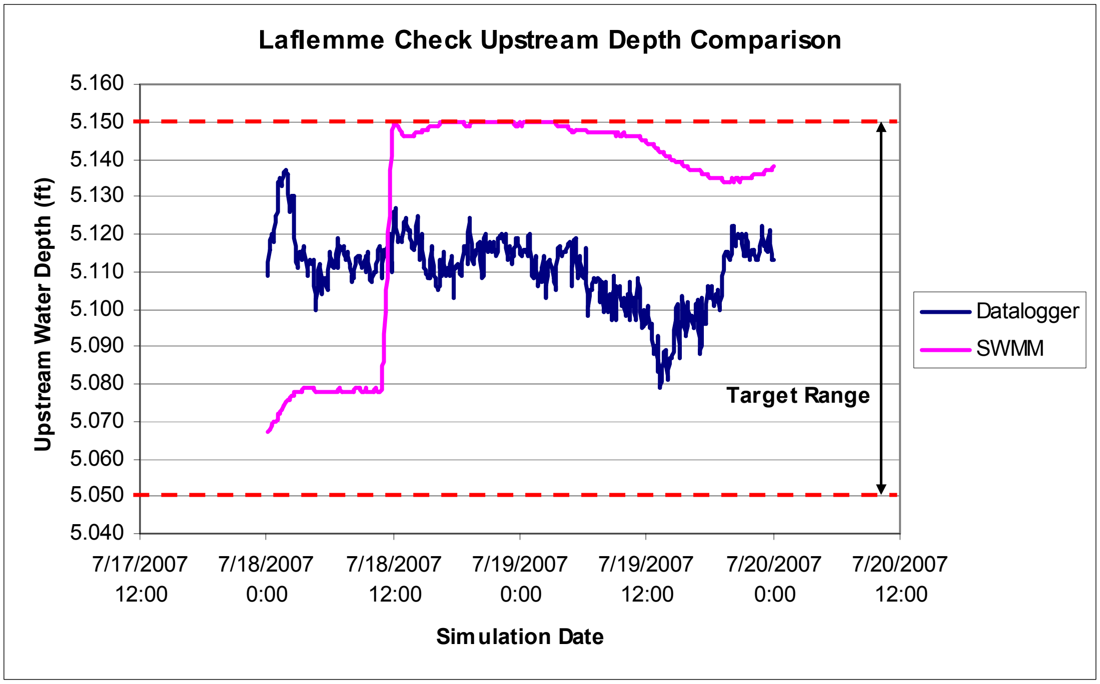

Figure 48. Comparison of upstream water depths recorded in the field by the datalogger and the water depths modeled by SWMM at the automated Laflemme check.

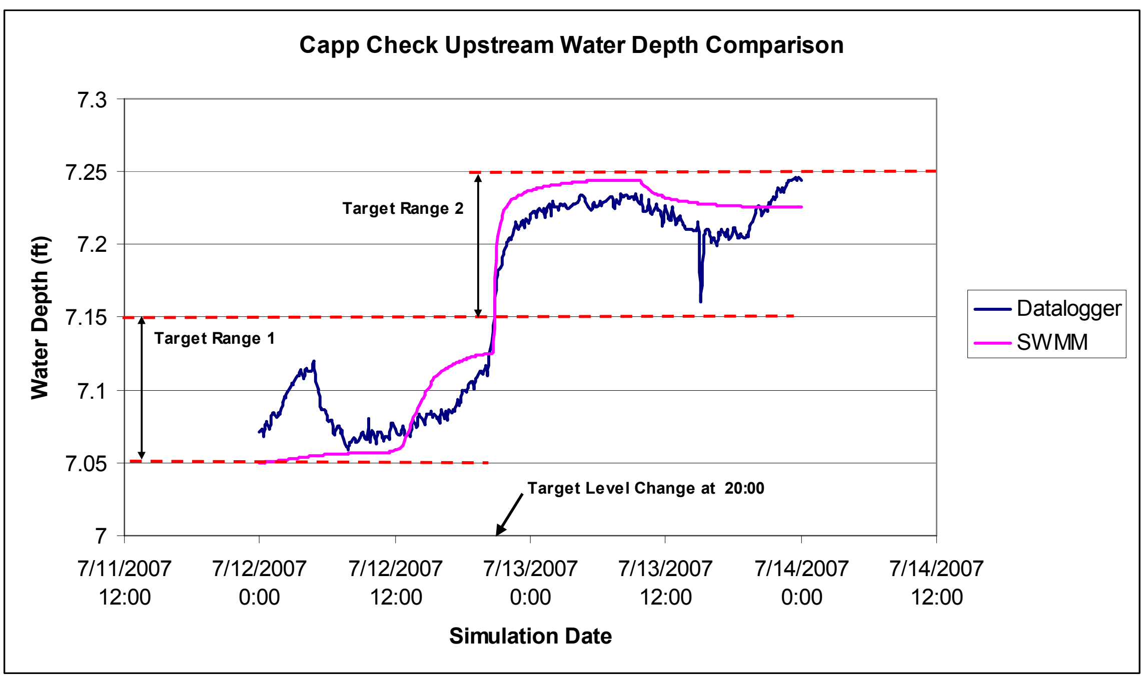

Figure 49. Comparison of upstream water depths recorded in the field by the datalogger and the water depths modeled by SWMM at the automated Capp check.

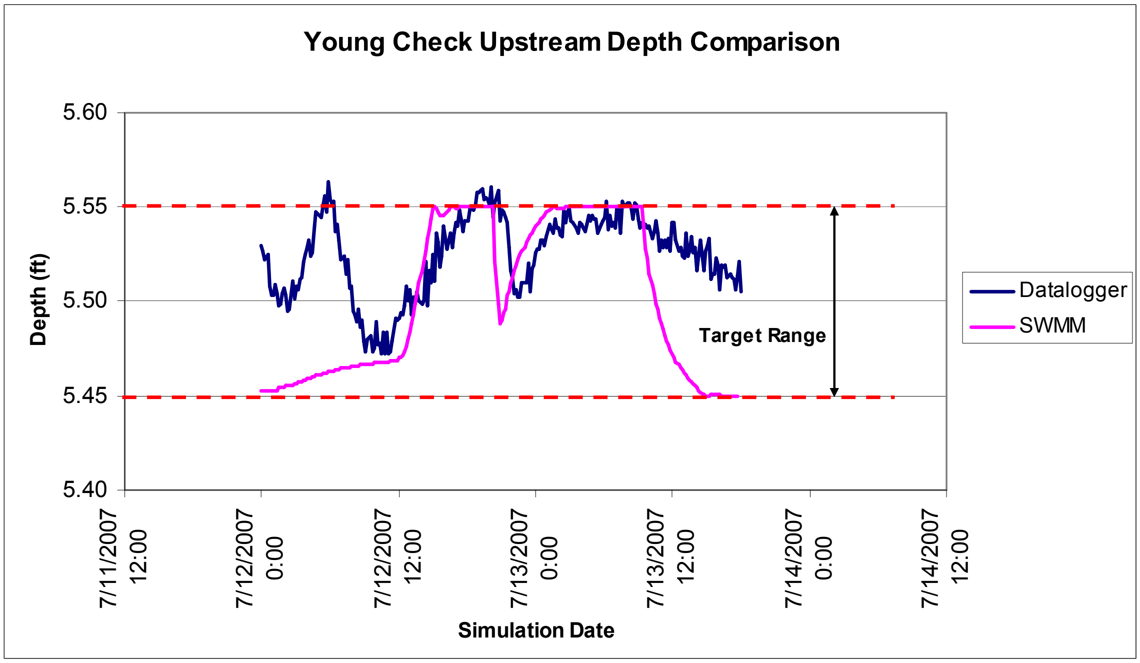

Figure 50. Comparison of upstream water depths recorded in the field by the datalogger and the water depths modeled by SWMM at the automated Young check.

The modeled upstream depths at the check structures in reach one were within the acceptable range of ±10 percent, with the exception of one measurement at the Gregory check which was 11 percent different (Table 19). The downstream depths were more variable than the upstream depths. The downstream check conditions are typically very turbulent which can lead to errors in measurements. Measurement differences of ±0.4 feet could be explained by the measurement accuracy. Within Reach 1, there were two check structures that exceeded the acceptable range of ±10 percent for the downstream depths, the Gregory check and Capp check. The Young check downstream depths were noted but not tightly calibrated within Reach 1 since settings for the Kennedy check (the next downstream check) were not collected in this Reach and would greatly affect the downstream depth.

The Gregory check has by far the largest differences between the observed and modeled depths. The upstream depths differed by 0.66-0.72 feet and the downstream by 1.5-1.67 feet which is very significant. Collecting data at this site was very difficult as there is a beeyard with numerous beehives neighboring the check. This could have influenced erroneous field measurements. It is also possible that the dimensions of the Gregory check were mismeasured which could easily cause large differences in the results. The invert elevation of the check was increased by 0.3 tenths in order to allow the adjacent automated checks to hold the appropriate upstream levels. It is recommended that the canal be re-surveyed (especially in this section of the canal) to account for changes that have occurred since the BOR surveyed in 1989. The Capp check also showed large downstream differences ranging from 1.26 to 1.27 feet which is very noteworthy. Many changes have occurred since then such as channel side slope erosion, buildup of sediment within the canal, and side slope degradation leading to stock pools along the main canal. These factors are critical in determining how the system reacts and representing its physical dimensions.

Table 19. Comparison of field observation and SWMM modeled depths at the check structures within reach one during the calibration period.

Date	Check Structure	Upstream Depth				Downstream Depth			
		SWMM	*Observed*	*Difference*	*% Difference*	*SWMM*	*Observed*	*Difference*	*% Difference*
7/12/2007	Indian Creek	4.75	4.74	0.01	0%	3.01	3.25	-0.24	-8%
7/13/2007	Indian Creek	4.74	4.72	0.02	1%	3.00	3.25	-0.25	-8%
7/12/2007	Gregory	7.31	6.58	0.72	11%	7.17	5.67	1.50	27%
7/13/2007	Gregory	7.31	6.63	0.68	10%	7.17	5.50	1.67	30%
7/12/2007	Laflemme	5.15	5.03	0.11	2%	3.69	3.45	0.24	7%
7/13/2007	Laflemme	5.15	5.03	0.11	2%	3.72	3.45	0.27	8%
7/12/2007	Indian Creek Siphon	4.65	4.85	-0.20	-4%	3.38	3.52	-0.14	-4%
7/13/2007	Indian Creek Siphon	4.85	4.85	0.00	0%	3.66	3.52	0.15	4%
7/12/2007	Capp	7.09	7.08	0.01	0%	4.14	2.87	1.27	44%
7/13/2007	Capp	7.23	7.08	0.16	2%	4.13	2.87	1.26	44%
7/12/2007	Young	5.52	5.68	-0.17	-3%	3.24	3.77	-0.53	-14%
7/13/2007	Young	5.48	5.68	-0.20	-4%	3.40	3.77	-0.37	-10%

4.5.b Reach 2

4.5.b.i Issues and Assumptions

Reach 2 was calibrated using the process described previously for reach 1. To calibrate the automated check structures, appropriate Kp values were entered that reduced oscillations (Table 20). The largest issue in reach two was completing the water balance. Reach two began at the young automated check and ended at the Beehive check/flume combo site. The real-time data collected at the young check during the calibration period was entered into the operational spreadsheet to obtain a flow timeseries. The timeseries was input at a node just upstream of the young check. Since the operational spreadsheet has not been validated there could be some differences from the actual flow during that time. The Beehive combo site consists of an automated check and a parshall flume located just downstream from the check. The flume had a very high degree of submergence during the calibration period therefore the data was disregarded. Initially, the operational spreadsheet was going to be used to provide continuous flow data at the check. After analyzing data from the calibration period it was found that the beehive automation equipment was out of order and the gate was left at 100 percent open the entire time. Since the gate was out of the water, the operational spreadsheet was invalid therefore no flow data was available. Instead, the Stumph check operational spreadsheet was used to develop a continuous water balance for the first half of reach two using the real-time data collected during the calibration period.

Table 20. Automated check structure Kp values entered into the PID control rule.

Automated Check Structure	Kp Value
Young	-30
Stumph	-20

4.5.b.ii Results and Discussion

The calibration period for reach two in 2007 consisted of only two days: July 18 and July 19. These dates were modeled along with a startup period of 6 days in order to fill the canal and build the water levels up at the check structures. The water balance modeled by SWMM was very close to what was expected according to the operational spreadsheet data. The modeled flow at the Stumph check differed from the expected operational spreadsheet estimate by an average value of -0.53 percent and a maximum value of 6 percent (Figure 51). The upstream target range at the Stumph check was between 4.25 and 4.35. The modeled water level versus the water level recorded by the datalogger was very similar (Figure 52). Initially the target range was not held. After analyzing the situation, the check elevation was increased by four tenths in order for the automated gate to hold the correct level. This is probably a reasonable estimate due to the buildup of silt that occurs near the check structures. The modeled upstream and downstream water levels at the check structures were all within the acceptable range which was set at ±10 percent. In fact, all upstream water levels were within ±6 percent and all downstream water levels were within ±8 percent of the observed values for both calibration days (Table 21). The downstream check conditions are typically very turbulent which can lead to errors in measurements. All of the downstream differences were less than 0.3' which could be explained by the measurement accuracy.

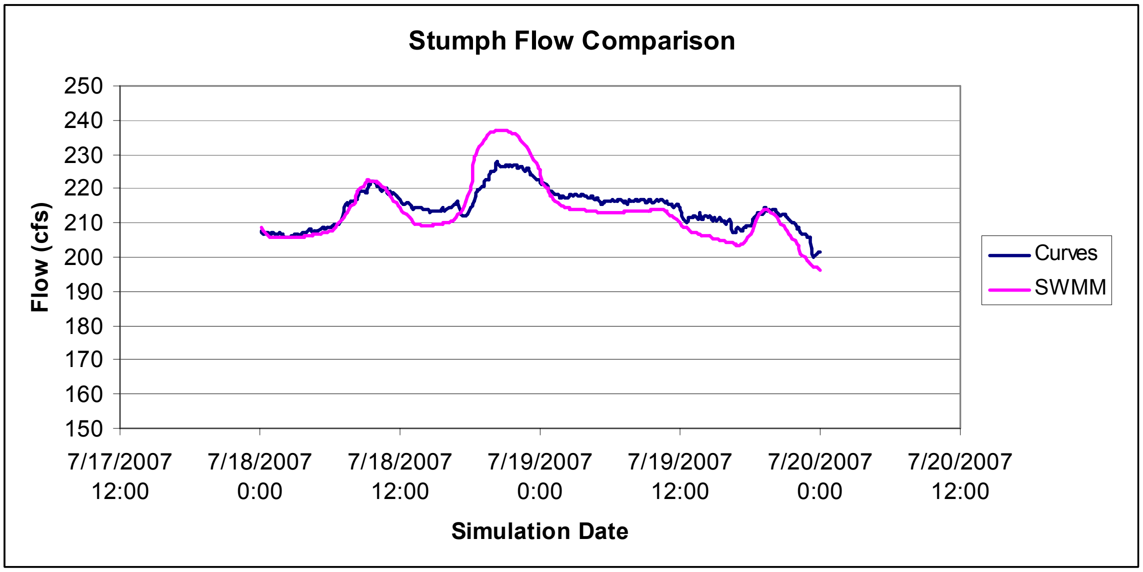

Figure 51. Comparison of the operational curve flow estimate and the results from SWMM at the automated Stumph check.

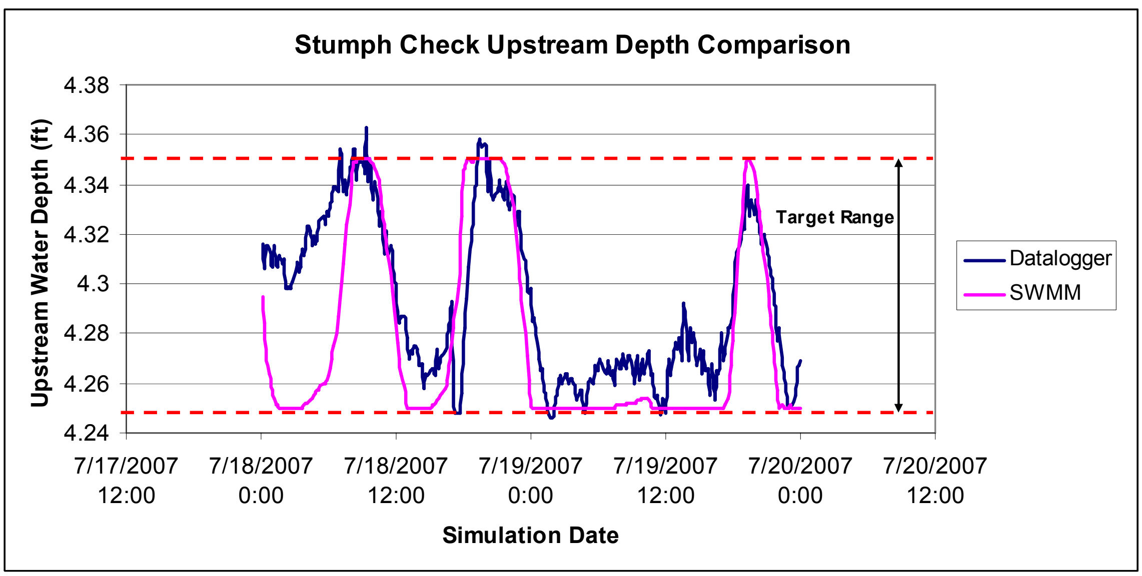

Figure 52. Comparison of water levels recorded in the field by a datalogger and the water levels modeled by SWMM at the automated Stumph check.

Table 21. Comparison of field observations and SWMM modeled depths at the check structures within reach two during the calibration period.

Date	Check Structure	Upstream Depth				Downstream Depth			
		SWMM	*Observed*	*Difference*	*% Diff*	*SWMM*	*Observed*	*Difference*	*% Diff*
7/18/2007	Young	5.46	5.52	-0.06	-1%	3.58	3.85	-0.27	-7%
7/19/2007	Young	5.46	5.52	-0.06	-1%	3.56	3.85	-0.29	-8%
7/18/2007	Kennedy	4.83	4.67	0.16	3%	2.22	2.42	-0.20	-8%
7/19/2007	Kennedy	4.82	4.67	0.15	3%	2.21	2.42	-0.20	-8%
7/18/2007	Stumph	4.25	4.33	-0.08	-2%	2.68	2.83	-0.15	-5%
7/19/2007	Stumph	4.25	4.33	-0.08	-2%	2.66	2.83	-0.17	-6%
7/18/2007	Fickbohm	4.75	4.50	0.25	6%	3.40	3.17	0.24	7%
7/19/2007	Fickbohm	4.71	4.50	0.21	5%	3.38	3.17	0.21	7%
7/18/2007	Beehive	6.36	6.26	0.10	2%	5.58	5.55	0.03	1%
7/19/2007	Beehive	6.33	6.26	0.07	1%	5.55	5.55	0.00	0%

Manning's n values and check structure weir and orifice discharge coefficients were manipulated during the calibration process to match the modeled water levels to the observed water levels. The Manning's n values for roughness were slightly lower than expected in some areas (Table 22). Some of the weir discharge coefficients are also outside of the expected range of 2.6 to 3.4 (Table 23). These factors may be outside of the expected range because all of the assumption errors as well as changes in the channel geometry are absorbed by these coefficients.

Table 22. Manning's n (roughness) values for Reach 2

Canal Region	Manning's n
Young to Kennedy	0.022
Kennedy to Stumph	0.015
Stumph to Fickbohm	0.022
Fickbohm to Beehive	0.0103

Table 23. Discharge coefficients at check structures within Reach 2.

Check Structure	Discharge Coefficients	
	Weir	*Orifice/Gate*
Young	2.8	0.6
Kennedy	4.5	0.7
Stumph	4.5	0.7
Fickbohm	3.5	0.5
Beehive	3.2	0.4

4.5.c Reach 3

4.5.c.i Issues and Assumptions

Insufficient calibration data was collected during the summer of 2007 for Reach 3. The reach begins at the Beehive automated check and flume combo site. The flume was highly submerged; therefore; input flow estimates were not available from the flume. The Beehive check operational spreadsheet was also not valid for estimating flow due to the automated gate being out of order and 100 percent open (and out of the water) during the calibration period. Since accurate inflows were not available during the 2007 calibration period, data was collected during the summer of 2008 to be used for calibration on Reach 3. The calibration dates included July 30, July 31, and August 1, 2008.

A detailed study of the Beehive flume submergence was completed by Morlok and Carlson (Morlok and Carlson, 2008). Downstream channel modifications were recommended to reduce the Beehive flume submergence. Based on these recommendations, a portion on the North Canal between the Beehive flume and the Williamson check (which is the next downstream check) was dredged and maintained by the BFID prior to the 2008 irrigation season. These alterations in the canal could account for some of the differences between the SWMM model and observed field conditions. Following Morlok and Carlson's recommendation, the BFID operated the Williamson check more appropriately in 2008 to reduce backwater effects which had previously resulted in submergence of the Beehive flume. An automated gate was added at the

Williamson check during the 2008 irrigation season to ensure that the proper upstream water depth is maintained at the Williamson Check for specified flows.

During the 2008 calibration period, the Beehive check automation was operable and the flume was not highly submerged. Recorded flows at the Beehive flume were used for calibration input into Reach 3. According to BFID personnel, a 12 hour “slug” of water was sent from the dam to alleviate extreme low flow conditions at the bottom of the North Canal. This slug of water reached Reach 3 in the middle of the 2008 calibration period. Having a range of flows during the calibration period is optimum for calibration but made the Reach more sensitive to the timing of water releases and check adjustments.

Reach 3 was calibrated using the process described previously for both Reach 1 and Reach 2. To calibrate the automated check structures, appropriate Kp values were entered that reduced oscillations (Table 24). The Williamson check was converted to an automated check during the 2008 irrigation season. The automation was not operating during the calibration period. Therefore the Williamson check is not correctly calibrated as an automated check. Correct positioning of the automated gate in the future could alter the discharge coefficients and Manning’s n values of adjacent canal sections and check structures that were found during the 2008 calibration. This issue should be addressed by collecting field data and calibrating the Williamson Check while the automation is operable and adding a PID control rule to hold the target level. This process should be completed before integrating the model into BFID operations.

Table 24 Automated check structure Kp values entered into the PID control rule for Reach 3.

Automated Check Structure	Kp Value
Beehive	-20
Williamson	NA
Townsite	-20
Deadman	-10
Dry Creek	NA

During the initial calibration phase, it was noticed that water depths at the first four check structures were extremely high in comparison to the field measurements and datalogger measurements. After a thorough investigation, it was determined that the reach upstream of the Horse Creek Check and Siphon is highly sensitive to adjustments and conditions at the Horse Creek Check and Siphon. According to BFID personnel, this coincides with field observations. If the Horse Creek Check is checked to high, backwater effects are seen at all checks including the Beehive Check/Flume combo site, which is nearly 4 miles upstream. It should be noted that the Horse Creek Check has two triangular shaped overflow weirs. These dimensions were unable to be simulated in SWMM so the weirs were modeled as rectangular weirs which shouldn't make a significant impact unless the water level is high enough to send water over these weirs, which typically does not happen. The sensitivity due to the Horse Creek Check/Siphon made calibration efforts much more difficult as one minor adjustment could affect the entire upstream reach. For this reason, calibration was divided into two phases. Phase 1 included the canal from the Beehive Check/Flume combo site downstream to the Horse Creek Check and Siphon and Phase 2 included the rest of Reach 3, downstream of the Horse Creek Siphon ending at the Dry Creek Weir.

Real-time stage recorded by dataloggers was available at the Beehive Check, Williamson Check, Townsite Check and Deadman Check. The Dry Creek Check was not simulated as an automated check since the automated gate was out of the water and therefore not holding the target level. Real-time flow data was available at the Beehive check/flume combo site and the Dry Creek Weir, both sites corresponding to the beginning and the end of Reach 3, respectively. The operation curve developed for the Beehive Check was not used as a comparison because the discharge coefficients were developed for the 2007 irrigation season when the check was operating under submerged conditions which is unrepresentative of the 2008 irrigation season. A new operational curve developed from data collected in the 2008 irrigation season was still in the development stage at the time of calibration. The final curves should be available for the 2009 irrigation season. The Townsite Check operational curve was also not used for flow comparison due to differences seen in the early validation efforts. The number of measurements taken during the 2007 irrigation season was limited; therefore, the discharge coefficients will probably change after the 2008 data is incorporated. Another issue that could affect the calibration results is the fact that during field measurements, water was flowing over some of the check structure gates. There is no way to model this in SWMM which could increase the amount of uncertainty to a small degree.

4.5.c.ii Results and Discussion

The results for Reach 3 are very good considering the size of the reach and the variability at hand. The water balance of the entire reach was simulated very well. At the start of the reach, the Beehive Combo site had an average relative percent difference of -

0.0008 percent (Figure 53). The average difference was 0.15 cfs with a maximum difference 8.95 cfs and a minimum of -4.09 cfs. At the end of the Reach, the Dry Creek Weir had an average relative percent difference of -1.74 (Figure 54). The average difference was -1.59 cfs with a maximum difference of 8.61 cfs and a minimum of -8.98 cfs.

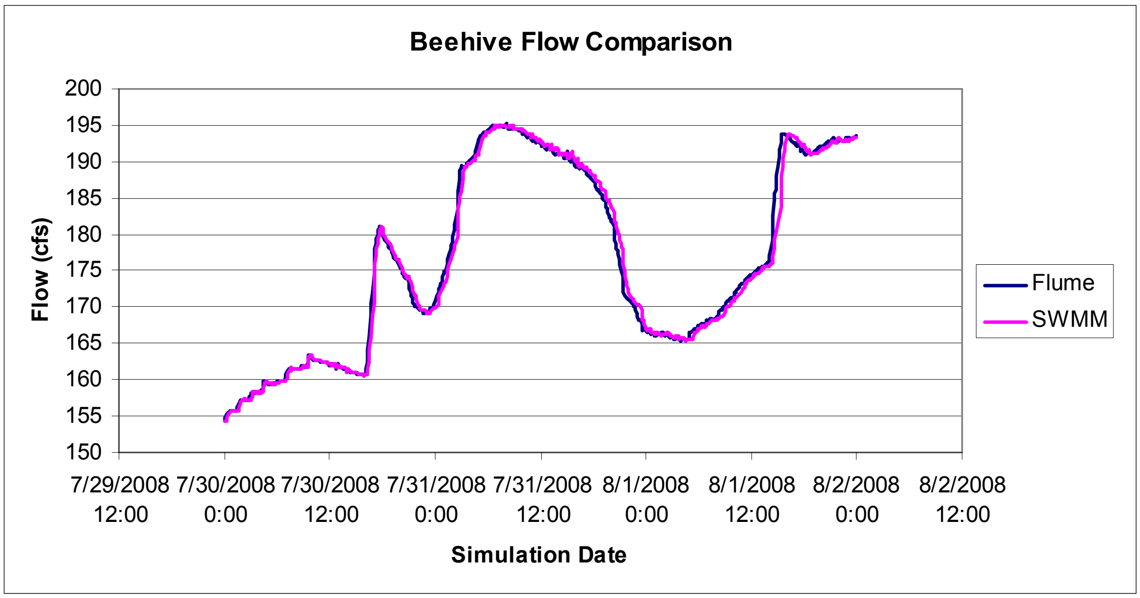

Figure 53. Comparison of the input flow recorded by the Beehive Flume and the flow simulated by SWMM at the Beehive Check Structure.

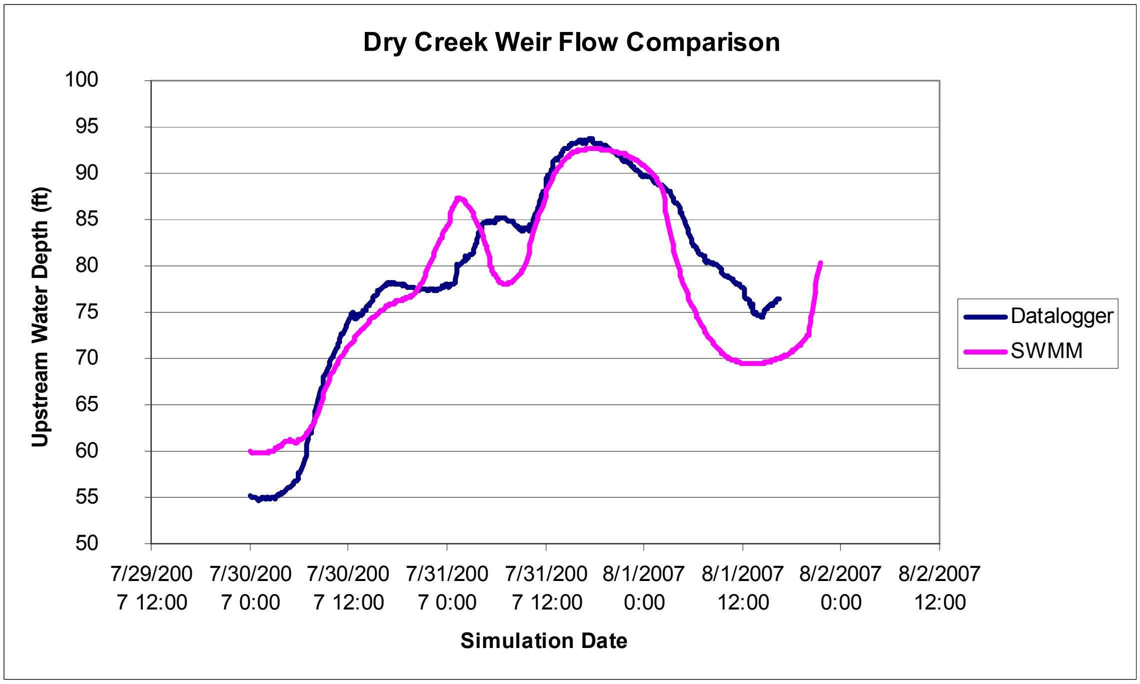

Figure 54. Comparison of the recorded flow and the flow simulated by SWMM at the Dry Creek Weir.

All automated gates were capable of holding the specified upstream target level including the Williamson check which had real-time datalogger measurements but was not holding a target level during the calibration period (Figure 55 to Figure 58). All of the upstream water levels were within the acceptable range of ±10 percent or within 0.3 feet of the field measurement (Table 25). Five of the 27 downstream measurements did not fall within the acceptable range, of which, only 2 measurements were unexplainable. At the Horse Creek Siphon Check, one of the downstream depths was -17 percent lower than the field measurement but was still only 0.1 ft less, which is easily explained by the turbulent downstream conditions that occur at check structures. The downstream water depth at the Beehive Check ranged from -12 to -28 percent less than the observed values. This is because the Beehive Flume was not modeled due to stability issues. When the flume was placed in the model for trial purposes, it caused significant backwater at the

Beehive Check and increased the downstream depth. Dredging the canal could also have caused differences at the Beehive site and the Williamson Check. Since the difference was explainable it was not considered a major issue. The two unexplainable downstream exceedances of 20 percent and -13 percent occurred at the Berg and Williamson Checks, respectively.

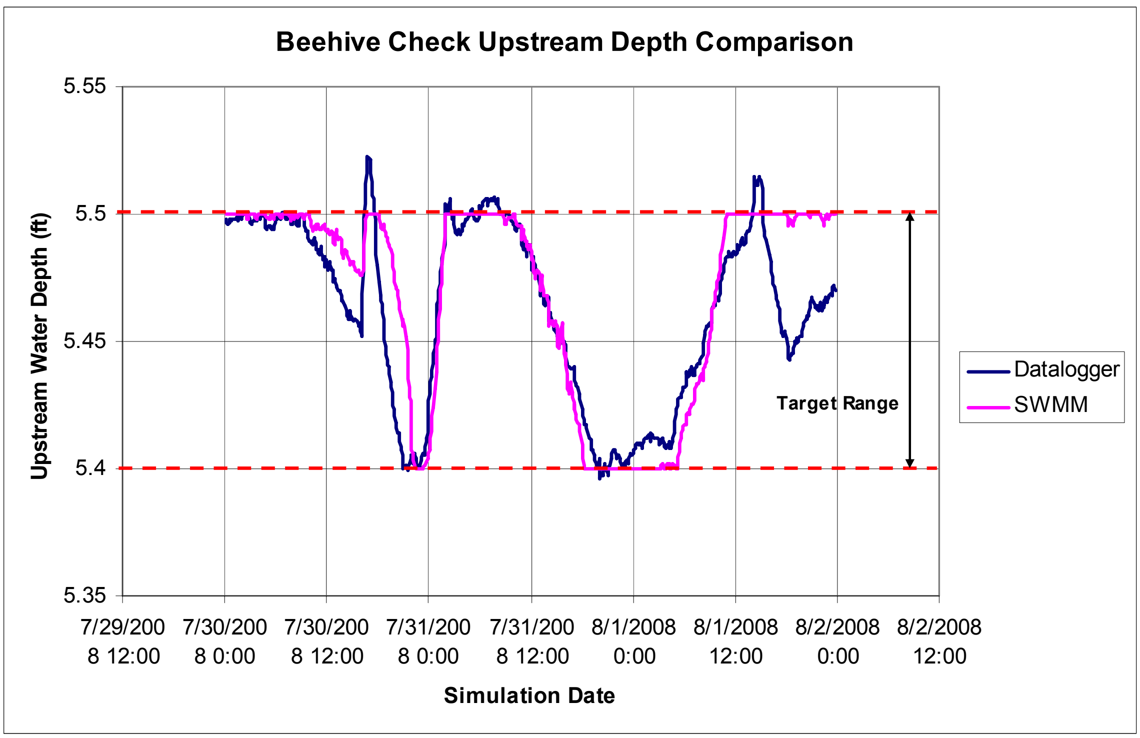

Figure 55. Comparison of water levels recorded in the field by a datalogger and the water levels modeled by SWMM at the automated Beehive Check.

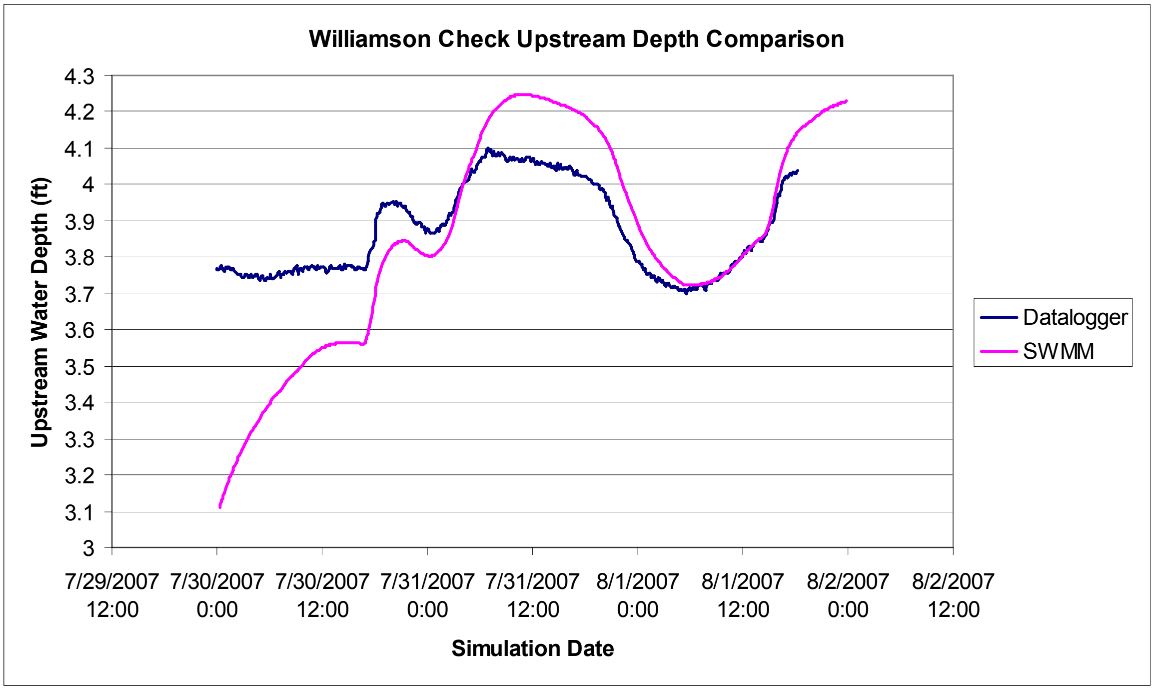

Figure 56. Comparison of Simulated upstream water level to measurements recorded in the field at the Williamson Check.

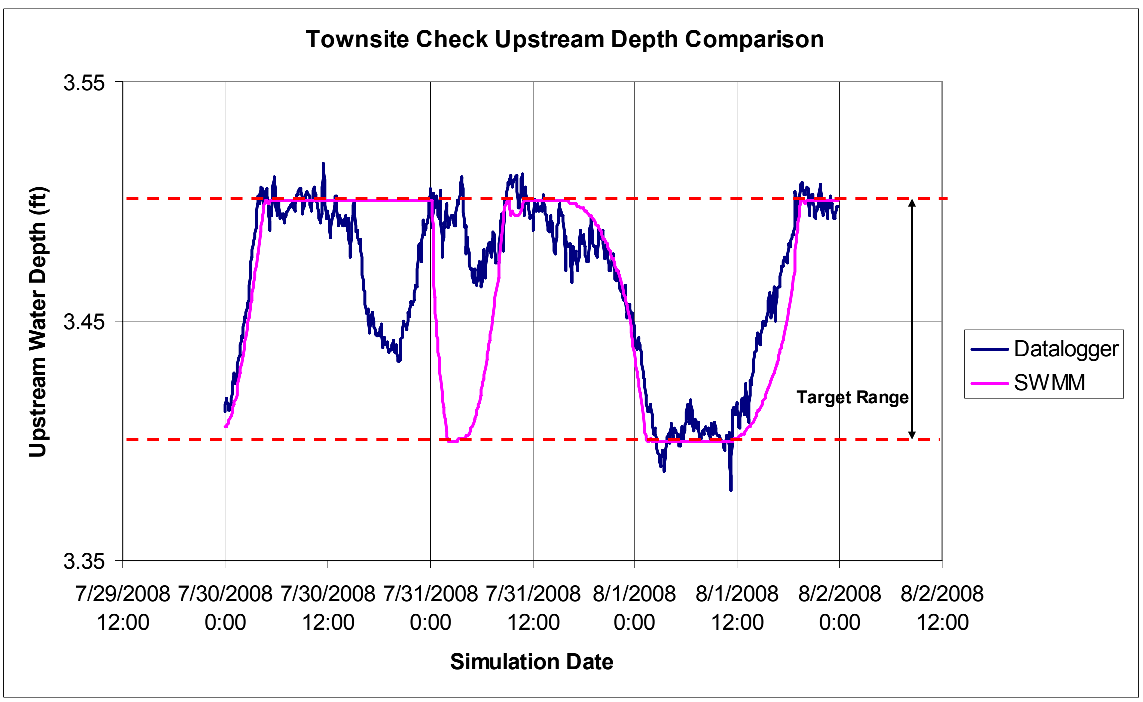

Figure 57. Comparison of water levels recorded in the field by a datalogger and the water levels modeled by SWMM at the automated Townsite Check.

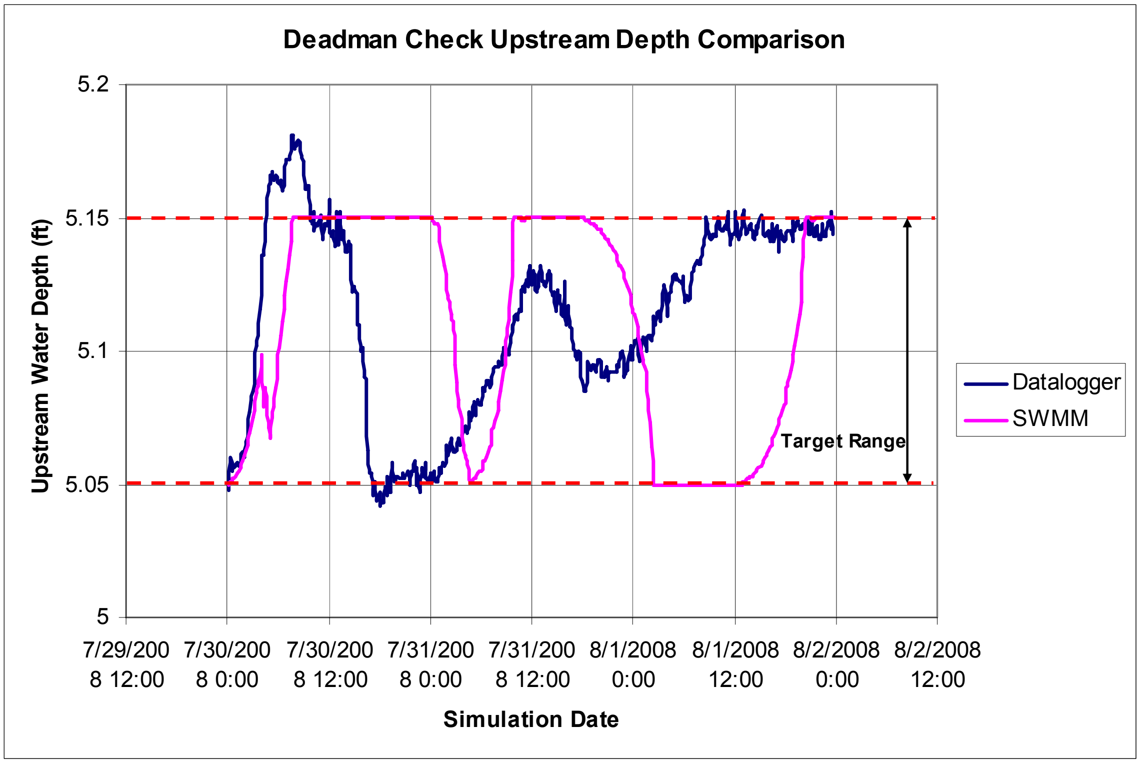

Figure 58. Comparison of water levels recorded in the field by a datalogger and the water levels modeled by SWMM at the automated Deadman Check.

Table 25. Comparison of field observations and SWMM modeled water depths at the check structures within Reach 3 during the calibration period.

Date	Check Structure	Upstream Depth				Downstream Depth			
		SWMM	*Observed*	*Difference*	*% Difference*	*SWMM*	*Observed*	*Difference*	*% Difference*
7/30/2008	Beehive	5.50	5.49	0.01	0%	3.33	4.65	-1.32	-28%
7/31/2008	Beehive	5.50	5.48	0.02	0%	4.03	4.59	-0.56	-12%
8/1/2008	Beehive	5.45	5.30	0.15	3%	3.54	4.40	-0.86	-19%
7/30/2008	Williamson	3.50	3.60	-0.10	-3%	2.80	3.20	-0.40	-13%
7/31/2008	Williamson	4.24	4.04	0.20	5%	3.52	3.60	-0.08	-2%
8/1/2008	Williamson	3.74	3.70	0.04	1%	3.04	3.22	-0.18	-6%
7/30/2008	Berg	4.06	4.52	-0.46	-10%	2.73	2.60	0.13	5%
7/31/2008	Berg	4.81	4.70	0.11	2%	3.49	2.90	0.59	20%
8/1/2008	Berg	4.31	4.38	-0.07	-2%	3.00	2.88	0.12	4%
7/30/2008	Horse Creek Siphon	3.85	4.16	-0.31	-7%	0.44	0.45	-0.01	-2%
7/31/2008	Horse Creek Siphon	4.67	4.45	0.22	5%	0.74	0.70	0.04	6%
8/1/2008	Horse Creek Siphon	4.16	4.07	0.09	2%	0.50	0.60	-0.10	-17%
7/30/2008	Haffle	5.00	5.40	-0.40	-7%	4.04	4.00	0.04	1%
7/31/2008	Haffle	5.77	5.80	-0.04	-1%	4.48	4.40	0.08	2%
8/1/2008	Haffle	5.24	5.50	-0.27	-5%	4.18	4.20	-0.03	-1%
7/30/2008	Boylan	3.84	3.80	0.04	1%	2.62	2.80	-0.18	-6%
7/31/2008	Boylan	4.27	4.21	0.06	1%	2.88	2.95	-0.07	-2%
8/1/2008	Boylan	3.97	3.92	0.05	1%	2.68	2.90	-0.22	-8%
7/30/2008	Townsite	3.50	3.27	0.23	7%	1.88	1.80	0.08	4%
7/31/2008	Townsite	3.50	3.25	0.25	8%	1.95	1.90	0.05	3%
8/1/2008	Townsite	3.40	3.10	0.30	10%	1.78	1.64	0.14	8%
7/30/2008	Deadman	5.15	4.88	0.27	6%	3.48	3.60	-0.12	-3%
7/31/2008	Deadman	5.15	4.88	0.27	6%	3.76	3.90	-0.14	-4%
8/1/2008	Deadman	4.51	4.78	-0.27	-6%	3.41	3.70	-0.29	-8%
7/30/2008	Dry Creek	4.55	4.80	-0.25	-5%	3.44	3.25	0.19	6%
7/31/2008	Dry Creek	4.81	5.05	-0.24	-5%	3.68	3.45	0.23	7%
8/1/2008	Dry Creek	4.51	4.80	-0.29	-6%	3.41	3.50	-0.09	-3%

The Manning's n values for roughness were slightly lower than expected in some areas (Table 26). Some of the weir discharge coefficients are also outside of the expected range of 2.6 to 3.4 (Table 27). These factors may be outside of the expected range because the assumptions and water order errors as well as changes in the channel geometry are absorbed by these coefficients.

Table 26. Manning's n values assigned to conduits representing canal regions in Reach 3

Canal Section	Manning's n
Beehive to Williamson	0.01
Williamson to Berg	0.011
Berg to Horse Creek	0.015
Horse Creek to Haffle	0.01
Haffle to Boylan	0.01
Boylan to Townsite	0.01-0.015
Townsite to Deadman	0.01
Deadman to Dry Creek	0.01
Dry Creek to Dry Creek Weir	0.0145

Table 27. Discharge coefficients assigned to checks in Reach 1.

Structure	Discharge Coefficients	
	Weir	Orifice/Gate
Beehive	3.7	0.9
Williamson	4	1
Berg	3.2	0.7
Horse Creek Siphon	2.8	0.4
Haffle	3.2	0.7
Boylan	3.2	0.7
Townsite	4.5	1
Deadman	4.4	1
Dry Creek	2.8	0.6
Dry Creek Weir	3.2	-

4.6 Summary of Calibration Results

Overall, the calibration results are very good considering the assumptions made and the amount of variability in the system. When looking at the entire model, only one out of 49 upstream depths and only 9 out of 49 downstream depths exceeded the acceptable range set at ± 10 percent, or less than 0.3 feet (Table 28). The total percent exceedence for upstream depths was two percent and 18 percent for downstream depths. All of the automated check structures were capable of holding the target level set in the PID controller. These statistics prove that the North Canal SWMM model is fully capable of modeling irrigation operations within the BFID. The issue now becomes the collection and recording of manual check adjustments and the amount and timing of water deliveries which would need to be entered into the SWMM model in order to use the tool to the best of its abilities.

Table 28. Overall exceedances of the acceptable range set at ± 10 percent, or less than 0.3 feet.

	Number of Calibration Data	Upstream Exceedances	Upstream Percent Exceedance	Downstream Exceedances	Downstream Percent Exceedance
Reach 1	12	1	8%	4	33%
Reach 2	10	0	0%	0	0%
Reach 3	27	0	0%	5	19%
Total	49	1	2%	9	18%

5.0 Sensitivity Analysis

A sensitivity analysis was completed by Schoenfelder (2006) for important check structures on the South Canal. The check structure depths and their sensitivity to

Manning's n and gate and discharge weir coefficients were the focus of the analysis. The sensitivity results show that check structure depth has an inverse relationship with Manning's n and a direct relationship with orifice and weir discharge coefficients. Schoenfelder (2006) found that discharge coefficient effects on check structure depths seemed to vary depending on the particular check structure. The check structure depth sensitivity to Manning's n adjustments depended on the depths occurring at the check and the structures distance from the dam. Check structure depth sensitivity to discharge coefficient adjustments depended on the number of orifices (gates) and weirs, and the magnitude of the calibrated discharge coefficients. Schoenfelder's sensitivity results are shown in Figure 59, Figure 60, and Figure 61. Schoenfelder's results were used as a guide in the North Canal calibration process because of the similarity between canal structures.

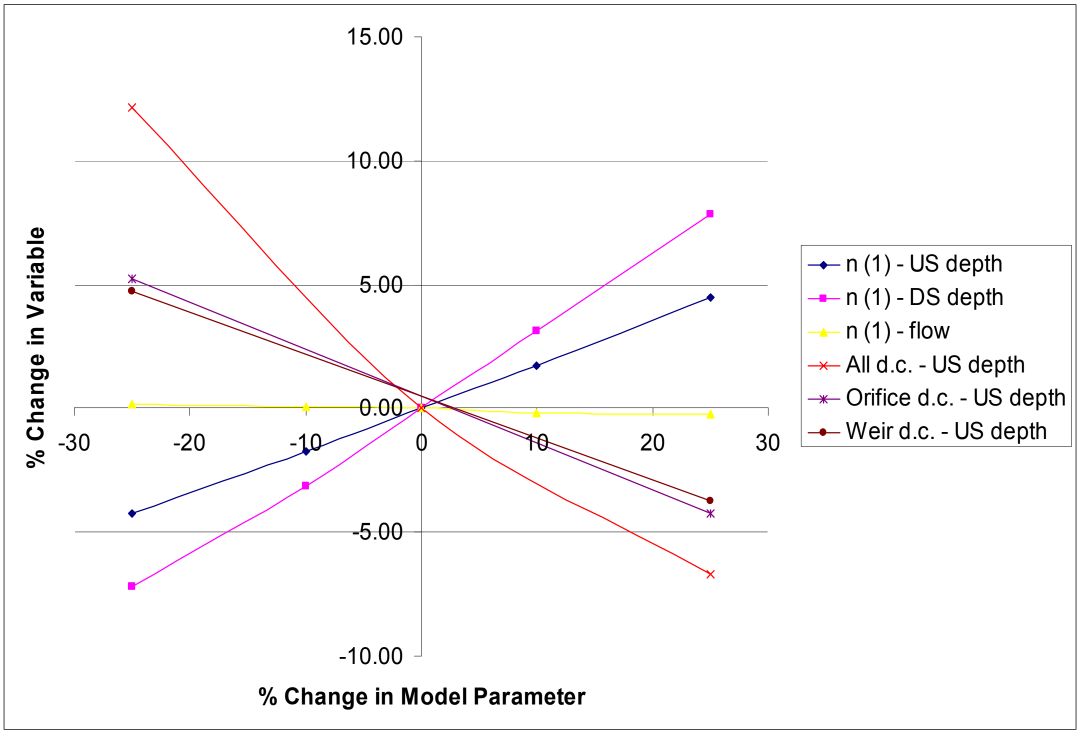

Figure 59. Sorenson Check sensitivity analysis plot (Schoenfelder, 2006).

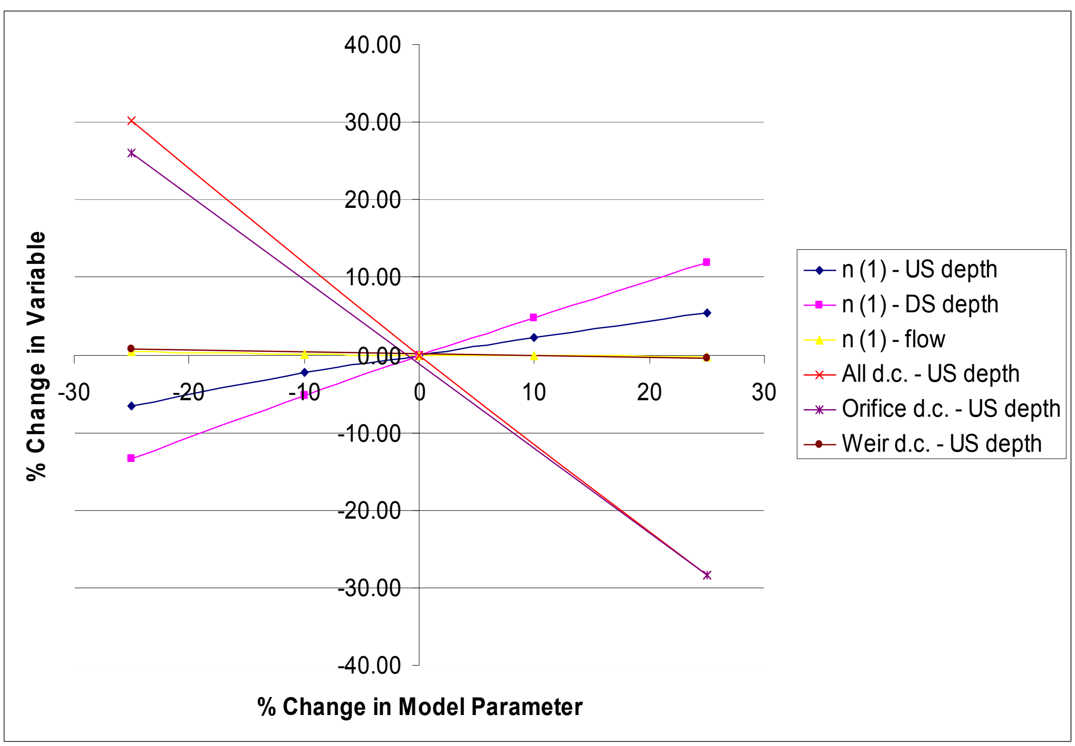

Figure 60. Beals Check sensitivity analysis plot (Schoenfelder, 2006).

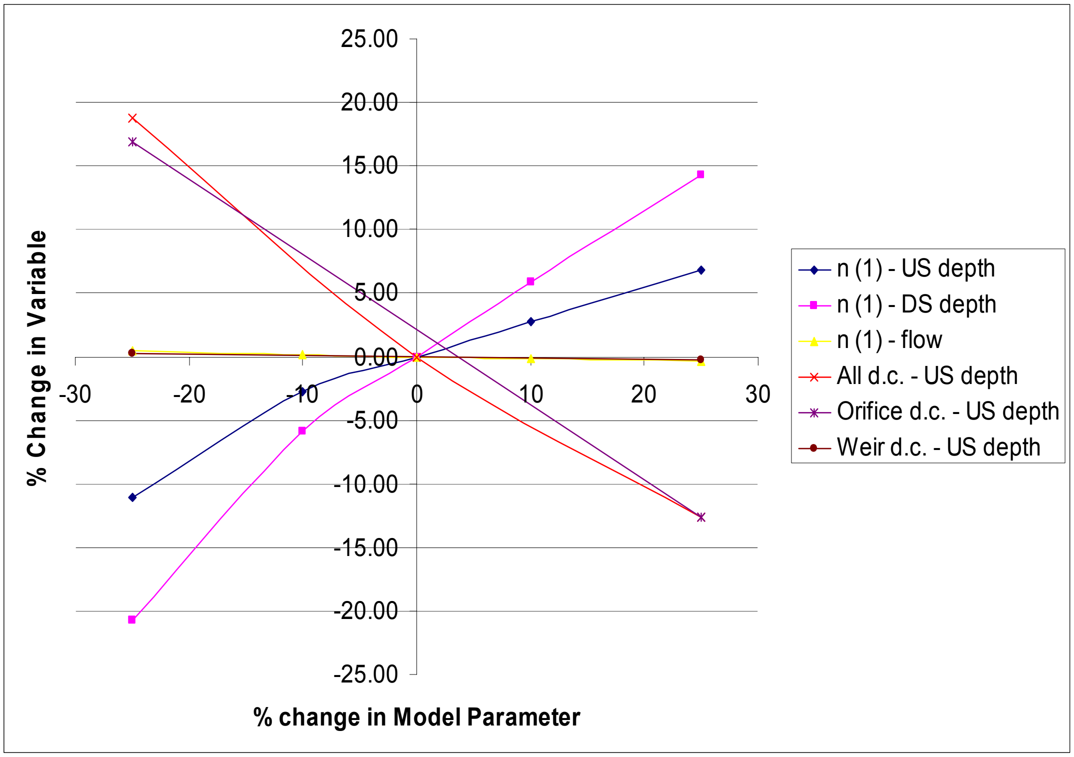

Figure 61. Vale Check sensitivity analysis plot (Schoenfelder, 2006).

6.0 Recommendations

The following recommendations (listed in no particular order) describe ways to improve the developed North Canal SWMM model. The recommendations should be considered before implementing the SWMM model as a decision making tool.

1. Inaccurate and missing physical properties proved to be most frustrating while calibrating the North Canal Model. A detailed survey of the North Canal including structures along the canal should be conducted. This should include: accurate canal dimensions, siphon, turnout, and check invert elevations and stationing especially for a few of the checks that were unavailable in BOR survey data.

2. Re-calibrate the model for automated checks that were inoperable during the calibration period. For example, the automation at the Williamson Check was not operating during the calibration period therefore the Williamson check was not calibrated as an automated check. For future use, a PID control rule would need to be added for the Williamson check in order to simulate the newly installed automated gate. The new automation could alter the discharge coefficients and Manning's n values of adjacent canal sections and check structures that were found during the 2008 calibration. If automation is installed at any of the current unautomated checks the same process should be followed. This issue should be addressed before integrating the model into the BFID operations.
3. The operational chart/curves should be validated based on the field measurements collected in 2008. The validated operational curves/charts should then be used to validate the North Canal SWMM model. Field measurements from 2008 are available to validate Reach 1 and Reach 2 of the North Canal SWMM model. New data would need to be collected to validate Reach 3 since the data collected in 2008 for Reach 3 was used for calibration because of insufficient data during previous years which was discussed in Section 3.5.c.i.
4. Calibrate and validate Reach 4 of the North Canal SWMM model by collecting more field measurements. This would provide a complete understanding of the entire North Canal system.
5. A user-friendly interface file should be developed which would provide all the input data SWMM needs to run a simulation. This would allow BFID personnel to run the model and produce results without ever opening SWMM. This would

make the model more user-friendly to people who are unfamiliar with the SWMM program as it can be overwhelming to those who may not know how to use it. It would also reduce the time and personnel needed to obtain results.

6. The proportional coefficients for each check should be fine tuned to include different Kp values based on ranges of flow. Additional effort could be done to improve the integral and differentiation terms within the PID controller to further fine-tune the model.
7. As newer versions of SWMM are released, the ability to model bridges and culverts as well as other relatively short conduits should be studied. Modeling short conduits would especially improve the model because it would provide a better representation of the actual channel dimensions and locations of turnouts and laterals.
8. Limited data pertaining to check adjustments and water orders provided the highest degree of uncertainty while calibrating the North Canal Model. Optimally, it would be best to have a log for each check containing adjustments made to checks and turnouts and the timing of these changes which would reduce the assumptions made during calibration. The variability in water orders and check adjustments are absorbed by changing the discharge coefficients of the gates and weirs of the check structures. If the amount of water and the timing of the water deliveries is fine-tuned (more accurately known), the discharge coefficients could be calibrated more precisely. The calibration of the model is ultimately only as good as the data collected and used for calibration.

7.0 Model Applications

The North Canal SWMM model could be used as a whole or broken down into the designated BFID rides. The model could be broken down further wherever real-time flow data is available. Real-time flow data could be produced using operational spreadsheets where they are available. By inputting actual flow data into the model, variability produced from the model would be minimized.

7.1 Decision making and Predictor tool

The North Canal SWMM model could be a valuable decision making tool. The model would be able to predict when water orders will be available for deliveries at specific locations along the canal. It would also be able to predict the “lag time” for any location along the canal. The lag time is the length of time it will take water released from the dam to reach any specific delivery location on the main canal. Once this is known, ditch-riders and farmers will know minimum advancement time needed in order to receive water on a particular day.

The SWMM model could also be used to foresee whether or not check structures will need to be adjusted to handle the expected flow. In particular, the SWMM model would be able predict if an automated gate at a check structure will become maxed out due to the expected flow and the current manual gate settings. If so, new manual check settings could be input into the model to tell the ditchrider approximately where to set the manual gates and weirs. On the other hand, available operational charts/curves and/or spreadsheets could be used to find the appropriate manual gate settings to give the automated gate its maximum range of motion for the expected flow. The SWMM model would also be able to give BFID personnel the information to assess how lower reaches of the canal will be affected by changes or adjustments occurring upstream.

The SWMM model, check structure automation and the operational curves/charts and spreadsheets are all valuable but limited tools by themselves. By using the SWMM model, the automation and the operational curves/charts and spreadsheets in tandem, the BFID could maximize their understanding of the irrigation system and would be able to utilize the full potential of each of these unique tools.

7.2 Ditch-rider/Employee Training

Currently, one of the most realistic applications for this model would be to use it as a training tool for new BFID employees who are unfamiliar with the system. If a new ditch-rider is hired prior to the irrigation season, the SWMM model could give them an overview of the complexity of the BFID and show them how adjustments can affect the entire upstream and downstream portions of the canal. It could also illustrate how automated gates work and explain the correct operation of an automated check structure or unautomated check structures. Ditch-riders could become familiar with the approximate locations of structures along the canal. Although using SWMM as a training tool could never replace the importance of in-field training provided by the BFID, it could reduce the stress on new employees and reduce the amount of common mistakes due to inexperience.

8.0 Conclusions

The overall goal of this research was to increase the efficiency of the BFID by creating a hydraulic model of the North Canal supplemented by operational curves, charts and spreadsheets developed for key check structures. Operational curves, charts, and spreadsheets were developed for key automated checks based on field measurements which included gate openings, manual weir settings, upstream and downstream water levels and discharge, which was measured using the ADCP. Based on these field measurements, discharge was calculated using common weir and orifice discharge equations. The orifice and weir discharge coefficients were adjusted in the weir and orifice equations until the sum of the calculated discharge matched the discharge measured in the field. The coefficients that minimized the error were used to develop charts, curves and spreadsheets that can be used to improve the operation of automated check structures and turn automated checks into flow measuring devices.

A North Canal model was developed using EPA SWMM for the entire North Canal. The North Canal was divided into four reaches; of these, the first three were calibrated based on recorded water orders, field measurements collected during the 2007 and 2008 irrigation seasons, and rating curves developed from the field measurements. Various different methods of simulating automated gates at check structures were investigated. Due to limited field data and time constraints, the North Canal model and the operational curves/charts were not validated. Differences between the observed data and the simulated results are believed to be primarily due to uncertainty in water orders and water deliveries, insufficient physical characteristics of the canal, and assumptions pertaining to manual structure adjustments.

The North Canal SWMM model is fully capable of simulating the entire BFID irrigation system if the appropriate amount of data is collected. The issue now becomes the collection and recording of manual check adjustments and the amount and timing of water deliveries that would need to be entered into the SWMM model in order to use the tool to the best of its abilities. The BFID can use this model as a tool for ditch-rider training, and for understanding the complexities of the North Canal, and as a decision-making tool concerning system operation and structure adjustments. By using the developed SWMM model and the operational curves, charts, and spreadsheets, the BFID could reduce non-used irrigation return flows, which would in turn reduce the TSS in the Belle Fourche River.

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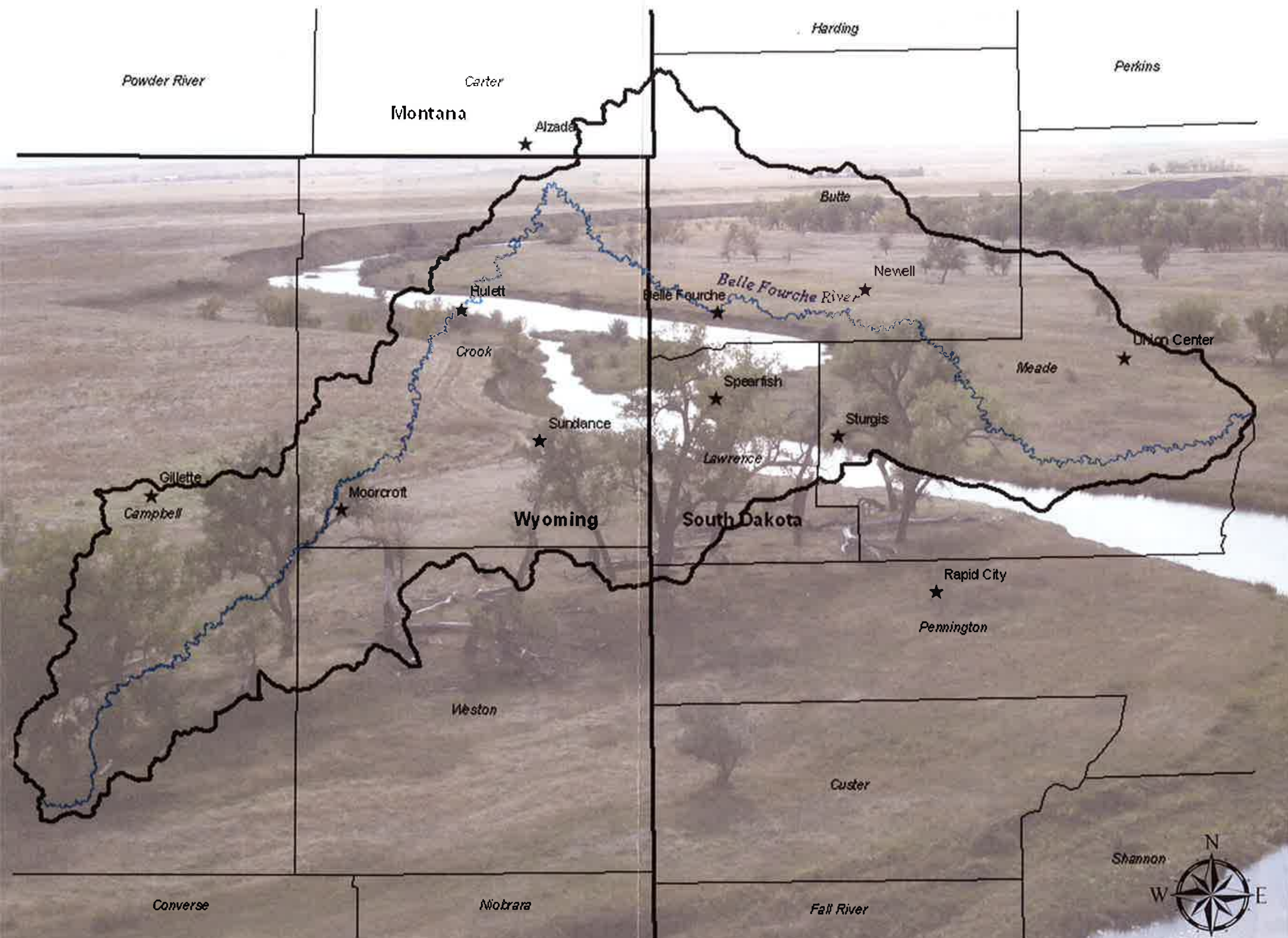
VITA

Lacy Pomarleau was born on November 26, 1981 in Dickinson, North Dakota, where she was raised by her parents, Tony and Margie Metz. She graduated from Trinity High School in 2000 and continued to live in Dickinson while she attended Dickinson State University for two years with an emphasis in Business Administration. In January 2003, she decided to pursue a pre-engineering degree at Bismarck State College in Bismarck, North Dakota. After receiving her Associate of Science degree in May 2004, she moved to Rapid City, South Dakota. Lacy was married to her best friend, Matthew Pomarleau, in July 2006 and completed a Bachelor of Science degree in Civil Engineering in December 2006 at the School of Mines and Technology. She immediately continued on for a Master of Science degree in Civil Engineering with a water resource emphasis at the South Dakota School of Mines and Technology under Dr. Scott Kenner's advising. She completed her course work in May 2008 and finished her research during the summer of 2008. Lacy accepted a position at RESPEC in Rapid City, South Dakota, as a Staff Engineer in the Water and Natural Resource Department. Lacy and Matthew recently moved near Hermosa, South Dakota, and plan to make it their permanent residence.

APPENDIX B

BELLE FOURCHE RIVER WATERSHED BROCHURE

Belle Fourche River Watershed



Belle Fourche River Watershed Partnership

Mission Statement

Coordinate available resources to address concerns associated with the Belle Fourche River Watershed and the riparian areas within.

The Partnership's Goal

We are a volunteer group of local people and organizations dedicated to the enhancement of the Belle Fourche River Watershed. The Partnership's goal is to provide a voluntary management approach to the Belle Fourche River Watershed to conserve its natural resources, foster the long term economic stability of its communities, maintain the social and cultural values of those communities, and ensure the sustainability of the primary aquifer basin's safe yield.



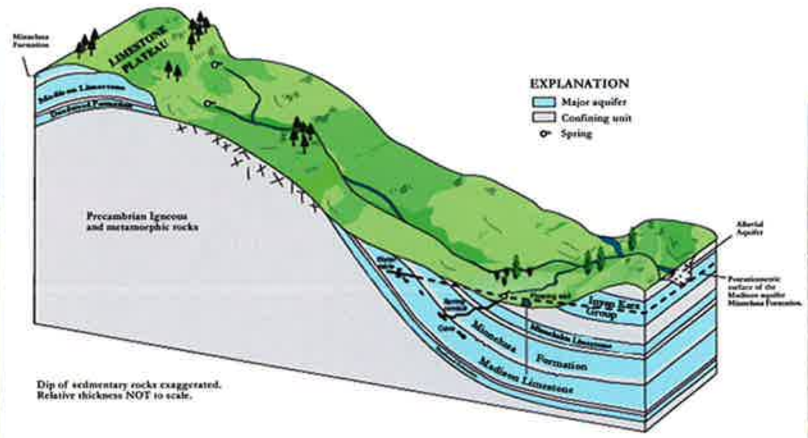
Diverse land use involves ranching, farming, logging, mining, recreation, tourist economies, urban sprawl, and development activities.


For more information, visit our website at
www.bellefourchewatershed.org

We are continually seeking new and innovative ways to improve conservation in the watershed while promoting economic sustainability and growth.

Future goals regarding groundwater inventory:

- Determining the basin's safe yield
- Determining sustainability of the aquifer
- Preventing aquifer mining



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We support conservation out-reach programs for producers, students, civic and government leaders and others, including pasture walks, riparian work-shops and range camp.



Bimonthly meetings are open to the public to encourage community involvement. Watershed tours demonstrate/educate local, state and federal governing groups on Partnership activities.

We continue to successfully build relationships with state and federal agencies and local agriculture producers to promote conservation on the ground.

Conservation planning includes proper grazing, which improves surface water quality and overall range health. Healthy range lands are profitable and sustainable.



Examples of Partnership assistance include:

- Finding funding for improved conservation practices
- Improving irrigation delivery and application efficiency
- Automation, lining and piping projects in the Belle Fourche Irrigation District.


- Funding support for improved irrigation application with center pivots.