WATERSHED PROJECT FINAL REPORT
SECTION 319 NONPOINT SOURCE
POLLUTION CONTROL PROGRAM

Topical Report RSI-1915

prepared for

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

January 2007

RESPEC
CONSULTING & SERVICES

WATERSHED PROJECT FINAL REPORT

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Belle Fourche River Watershed Management
Project and Implementation Plan Segment II

by

Eric L. Krantz
Jared K. Oswald

RESPEC
P.O. Box 725
Rapid City, South Dakota 57709

prepared for

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

January 2007

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 II
SECTION GRANT NUMBER(S):	999818505
PROJECT START DATE:	June 2005
PROJECT COMPLETION DATE:	December 2006
FUNDING:	
TOTAL EPA GRANT BUDGET:	$500,000
TOTAL MATCHING FUNDS BUDGET:	$424,325
TOTAL NONMATCHING FUNDS BUDGET:	$291,889
TOTAL BUDGET:	$1,216,214
BUDGET REVISIONS: Total 319 Funds did not change	
TOTAL EXPENDITURES OF EPA FUNDS:	$500,000
TOTAL 319 MATCHING FUNDS ACCRUED:	$444,593
TOTAL NONMATCHING FUNDS ACCRUED:	$263,100
TOTAL EXPENDITURES:	$1,207,693

The Belle Fourche River Watershed Management Project Segment II was sponsored by the Belle Fourche River Watershed Partnership (BFRWP) with support from agricultural organizations, federal and state agencies, local governments, 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) (15.2 mg/L reduction below the Belle Fourche Reservoir (14 percent of goal), 1.3 mg/L reduction above the Belle Fourche Reservoir).
- Conduct public education and outreach to stakeholders within the Belle Fourche River Watershed.
- Track the progress made toward reaching the goals of the TMDL to help ensure that the BMPs are being implemented.

Several activities completed resulted in a reduction of the nonused irrigation water discharged into surrounding water by 2,050 acre-feet per year (17.1 percent of the 10-year goal). Sixteen flow automation 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. Nine real-time stage/flow measuring devices were installed at key locations in the Belle Fourche Irrigation District (BFID). The data from these real-time structures are viewable from the district office in Newell, South Dakota, which allows the district manager to track water delivery and aids in decision making. Phase I of the water card and water order system was developed for the BFID to help check for mathematical errors associated with hand calculations. Data from permanent stage/flow measuring devices, flow automation units, and six portable stage measuring units were used to calibrate and validate a canal operational model. The BFID installed 2,850 feet of pipeline that delivers water from the District to the producers.

Several activities were completed to improve irrigation efficiencies after water was delivered to producers. Pipelines were installed by nine producers to deliver water to crops and four center pivot sprinkler systems were installed to replace existing surface irrigation.

Grazing/riparian areas were improved significantly within the watershed. Approximately 150 miles of pipeline was installed to provide off-stream livestock with water. Conservation plans were written for over 91,789 acres of grazing lands. An additional 77,860 acres had conservation practices installed which resulted in 1,510 acres of riparian vegetation improvements.

Several public education and outreach activities were completed during this project segment in addition to those described earlier. The Butte County, Lawrence County, and Elk Creek Conservation Districts each sent out newsletters which included project updates. The BFRWP had six general meetings to provide project updates on project work and progress being made. The BFID sends out a newsletter called the *Ditch Writer* informing producers of the activities throughout the district.

Reductions in TSS associated with BMP implementation cannot yet be evaluated statistically because of insufficient number of samples. Preliminary estimates based on BMP installation indicate that TSS load was reduced by 14,318 tons/year, which is 550 tons/year greater than what was estimated to be accomplished in this project segment.

ACKNOWLEDGEMENTS

The BFRWP would like to thank all those involved with this segment of the implementation of practices recommended from the Belle Fourche River Watershed TMDL. 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, Producers, 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. 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, eleven percent is managed by the U.S. Forest Service and four percent by the Bureau of Land Management.

The Belle Fourche River has five assigned beneficial uses: (1) fish and wildlife propagation, recreation, and stock watering; (2) warm-water permanent fish life propagation; (3) limited contact recreation; (4) immersion recreation; and (5) irrigation.

The Belle Fourche River was identified in the 1998 and 2002 South Dakota 303(d) Waterbody Lists and the 2004 and 2006 *Integrated Report for Surface Water Quality Assessment* as impaired because of elevated total suspended solids (TSS) and fecal coliform levels. 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.

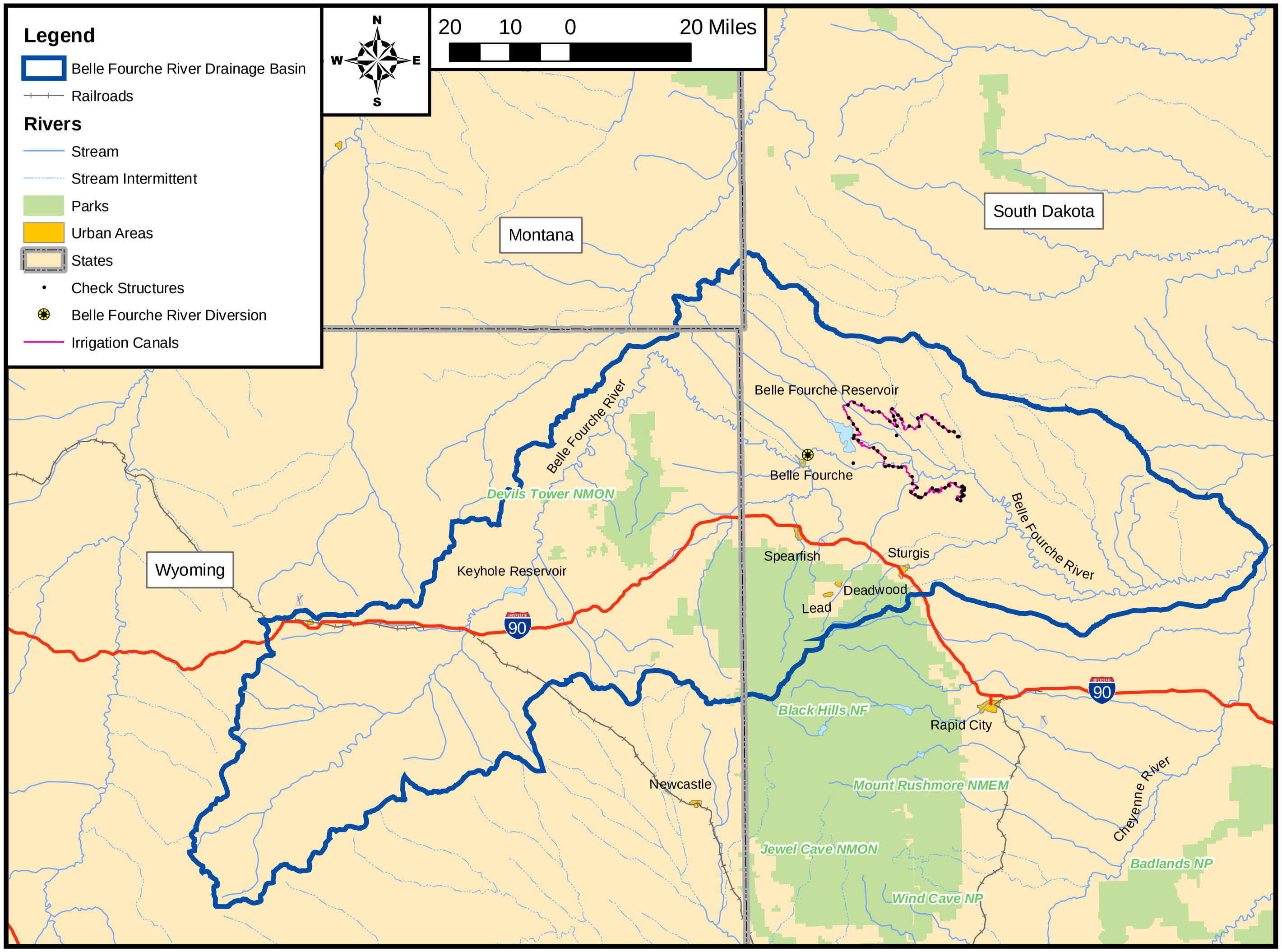

Figure 1-1. Belle Fourche River Watershed.

During the winter 2004, the BFRWP applied for and received a Clean Water Act Section 319 Grant to begin implementation of the BMPs recommended in the TMDLs for the Belle Fourche River. 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).

This project segment included funding 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. Two 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 completed in this project segment and those that will be completed in future project segments. The associated TSS and nonused water savings are presented for each action planned. Some of the BMPs recommended by the TMDLs and 10-year plan installed during this project segment include sixteen flow automation units, nine real-time stage/flow measuring devices, 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 with the irrigation BMPs being installed in the BFID (Figure 1-2).

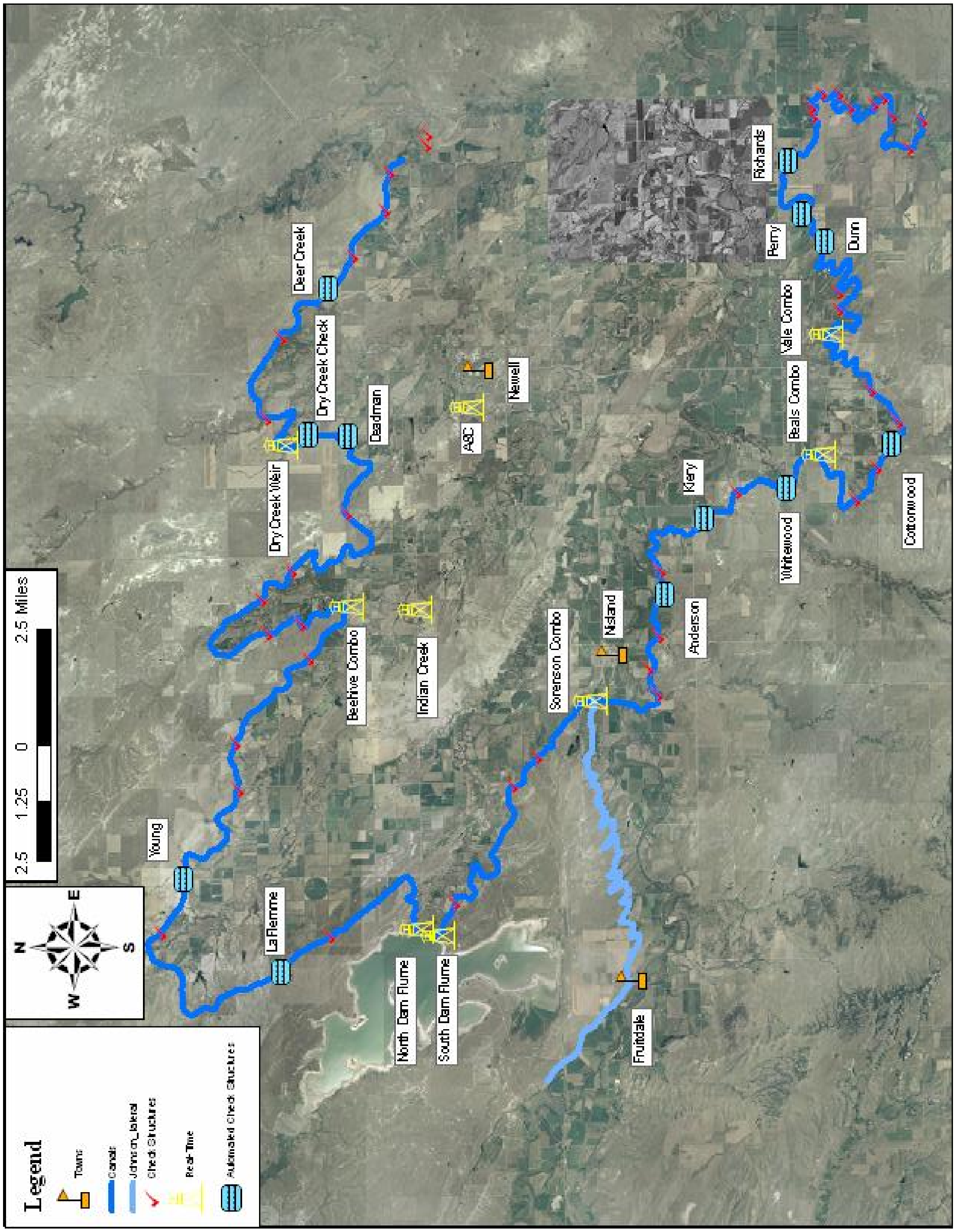

Figure 1-2. Segment II Automated Structures in the Belle Fourche Irrigation District.

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 55 percent reduction (289,910 tons/year) in TSS is required. A 41 percent reduction (2,033 tons/year) in TSS is required for Horse Creek.

In this project segment the load reduction goal is 13,768 tons/year. To accomplish this goal, this project segment had three objectives:

1. Continue implementation of BMPs in the watershed to reduce TSS [15.2 mg/L reduction (14 percent of goal) below the Belle Fourche Reservoir, 1.3 mg/L reduction 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 in the Belle Fourche River Watershed TMDL included two tasks: improving irrigation water management and implementing riparian vegetation improvements. The products of this objective included 16 flow automation units; a water card/water ordering system; six portable and nine real-time stage/flow measuring devices; a canal operational model for the South Canal; replacement of canals, laterals, and/or ditches with pipelines; installation of pipelines to deliver water from the BFID irrigation system to fields; installation of ten sprinkler irrigation systems; and implementation of riparian vegetation improvements. Implementation of the BMPs is discussed further in Chapter 3.0.

Objective 2. Conduct public education and outreach to stakeholders within the Belle Fourche River Watershed. To accomplish this objective, at least ten major information activities were to take place. There were nearly 30 outreach activities that are further discussed in Chapter 5.0 of this report.

Objective 3. Track progress toward meeting TMDL goals. Water-quality samples were collected by USGS at real-time stream gaging sites and DENR at several water-quality monitoring (WQM) sites in the watershed. It is not yet possible to statistically determine reductions in TSS because of the small number of samples collected and the short time between implementation of BMPs and collection of TSS. However, it is expected that implementation of

BMPs will reduce TSS exceedances as shown by statistical analysis after sufficient samples have been collected. 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. A comparison of the planned versus actual milestones for the quantity of each BMP installed is shown in Table 3-1.

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	May 2006	June 2006
Product 2. Improve Irrigation Application	December 2005	December 2005
Product 3. Complete and Install Riparian Vegetation Improvements	May 2006	May 2006
Objective 2. Conduct Public Education and Outreach		
Product 4. Supplement Existing Outreach Programs	May 2006	May 2006
Objective 3. Tracking Progress Toward Meeting Goals		
Product 5. GRTS and Final Reports	December 2006	December 2006

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 14,318 tons per year which is 550 tons/year greater than the goal for the project.

- Completion of nearly 30 successful education and outreach activities which lead to greater public participation in the project.

- Completion of midyear and annual GRTS reports along with this final report.

Project activities that were not completed during this segment included lining of the Belle Fourche Reservoir inlet canal and statistical analysis of reductions of solids in the Belle Fourche River and Horse Creek. Lining of the inlet canal was not completed because of water shortages in the Reservoir, and is scheduled to be completed during September 2007. Statistical analysis of reductions of solids in the Belle Fourche River will be completed after sufficient time has elapsed for BMPs to take effect and sufficient samples have been collected.

This project was successful. The project goal was attained. BMPs were implemented that are estimated to reduce total suspended solids in the Belle Fourche River by 13,768 tons/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, Belle Fourche Irrigation District (BFID), Bureau of Reclamation (BOR), United States Fish and Wildlife Service (USFWS), and Natural Resources Conservation Service (NRCS) as well as financial assistance from the project.

The BMPs installed included 16 flow automation units, nine real-time flow measuring devices, replacement of open irrigation ditch with pipeline, nine producer pipeline projects that deliver water from the BFID system to the fields, installation of four irrigation sprinkler systems, 150 miles of off-stream livestock water supply piping, 1,800 feet of lined canal, and 91,789 acres of managed grazing planning along with 77,869 acres of managed grazing implementation. The managed grazing systems implemented resulted in 1,510 acres of riparian improvements. 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	Milestone This Segment	Installed This Segment	Milestone to Date	Installed to Date
Flow Automation Units	42	16	16	17	17
Portable Stage/Flow Measuring Devices	15	6	6	6	6
Real-time Stage/Flow Measuring Devices	15	9	9	9	9
Line Open Canals and Laterals (Feet of Lining)	26,560	1,600	1,800	3,200	3,400
Replace Open Canals and Laterals With Pipeline (Feet of Pipeline)	25,000	2,000	2,850	4,000	6,850
Sprinkler Irrigation Systems	36	2	4	4	6
Install Pipeline Projects Delivering Water From BFID to Fields (No. of Projects)	40	1	9	12	19
Managed Riparian Grazing (Acres)	34,000	1,500	1,510	3,000	4,510

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 the 16 flow automation units (Figure 3-1) coupled with the nine 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. This allows for continual oversight of canal water levels and the ability to immediately adjust levels when necessary, thereby reducing waste and improving efficiency. Water-level data at each site is 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.

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Figure 3-1. Gate Automation Unit Installed in the Belle Fourche Irrigation District.

Six portable stage-measuring devices were installed at different locations throughout the BFID to monitor the flow of irrigation water. This information is used in the canal operational model developed for the BFID South Canal. Currently, the entire south canal is set up in Storm Water Management Model (SWMM), an EPA model capable of simulating all the conditions within the South Canal (Appendix E). The model was calibrated and validated using data collected with the portable measuring devices during summer 2005 and 2006, and will be used during the 2007 irrigation season to assist with irrigation delivery system settings and improve irrigation efficiency. The Beals check structure was calibrated and validated as a flow measuring device and is being used to more accurately determine irrigation settings and the amount of water being delivered on the South Canal (Appendix F).

Nine segments of open canals were replaced with pipeline, improving irrigation efficiencies after the water was delivered to producers. Four center pivot sprinkler systems (Figure 3-2) that replaced existing flood irrigation applications were installed.

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Figure 3-2. Center Pivot Sprinkler Installed in the Belle Fourche Irrigation District.

Over 1,800 feet of canal lining was completed by the BFID on the Wilson Lateral. The lining of the inlet canal, scheduled for October 2005, was postponed because of drought causing low levels in the Belle Fourche Reservoir. First-phase materials have been purchased and are scheduled to be installed beginning September 2007.

A total of 2,850 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.

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. Approximately 150 miles of pipeline, along with 300 livestock tanks, were installed for off-stream livestock water supply. Conservation plans were written for over 91,789 acres of grazing systems. Conservation practices such as cross-fencing and hayland seeding were implemented on an additional 77,860 acres. Of the grazing system conservation practices implemented, 1,510 acres were riparian vegetation improvements.

4.0 MONITORING RESULTS

The scheduled water-quality monitoring throughout the Watershed necessary to perform statistical analysis of BMP effectiveness and evaluate progress toward meeting TMDL goals is complete. Water-quality and flow monitoring data were collected at USGS gaging stations and at three DENR water-quality monitoring sites. USGS began monitoring Horse Creek Above Vale (USGS 06436760) for TSS in May 2004. To date, not enough samples have been collected to determine trends and calculate TSS reductions associated with BMP implementation through rigorous statistical analysis. Preliminary estimates indicate a load reduction of 14,318 tons/year for the River as a result of BMP installation. Based on calculations, TSS has been reduced 18 mg/L in this project and 27 mg/L in the Watershed since BMP implementation has began.

BMPs implemented in a watershed affect water quality. BMPs such as grazing management improvements, off-stream watering, and riparian area exclusion typically require several years before reductions in total suspended solids can be noticed and measured. Vegetation in pastures and along riparian corridors must be reestablished after years of overgrazing and high-density impact. Furthermore, excess loose sediments in streams and rivers that are the result of limited vegetation trapping are still present after BMPs are implemented. Time is necessary for this sediment to be flushed from the system or trapped in new vegetation. Although the short time period between BMP implementation and sample collection does not permit a rigorous statistical analysis of new data, an attempt was made to determine if trends or reductions were evident. The sections that follow describe the data collected and report statistics calculated using historic and new data.

4.1 TOTAL SUSPENDED SOLIDS (TSS) DATA

New TSS data collected by the DENR include six samples from Belle Fourche River at Belle Fourche (Appendix B), three samples from Belle Fourche River at Vale (Appendix C), and three samples from Belle Fourche River at Highway 79 (Appendix D). Of these samples, two at each site are categorized as “preliminary data,” meaning that values have yet to be finalized by laboratory staff and are yet unpublished. For each site, statistics were calculated for three data sets: data collected before the inception of BMPs (before June 2005), data collected during this project, and the entire dataset available. Statistics calculated were mean, median, standard deviation, maximum value, and number of samples. Tables 4-1 through 4-3 show descriptive statistics for the three DENR sites on the Belle Fourche River.

Reduction in mean TSS at all three sites is evident when new data is included with pre-BMP data versus pre-BMP data alone. Mean TSS was 223 mg/l for Belle Fourche River samples collected at Belle Fourche; inclusion of post-BMP samples reduces mean TSS to 203 mg/l at this site. The other two Belle Fourche River sites also show reduction in mean TSS values when new data is included with pre-BMP data. No samples collected at the three sites exceeded the South Dakota State criterion for TSS. Reductions in TSS are likely the result of grazing management improvements, limiting livestock access to streams using fencing and off-stream watering, and reductions in sediment-laden return flows from irrigation canals. It is expected that reductions in sediment concentrations will continue as vegetation has time to reestablish itself and irrigation personnel become better acquainted with the irrigation control technology.

Table 4-1. TSS Statistics (mg/l) for Belle Fourche River at Belle Fourche

Statistic	Pre-BMP Data (April 1999–May 2005)	All Data (April 1999–August 2006)	June 2005–August 2006
Mean	223	203	63.4
Median	8.00	8.00	50.5
Standard Deviation	745	697	63.1
Maximum	4520	4520	140
Number of Samples	41	47	6

Table 4-2. TSS Statistics (mg/l) for Belle Fourche River at Vale

Statistic	Pre-BMP Data (June 1977–April 2005)	All Data (June 1977–July 2006)	June 2005–August 2006
Mean	76.8	75.2	18.3
Median	34.5	32.0	15.0
Standard Deviation	153	151	12.3
Maximum	885	885	32
Number of Samples	106	109	3

Table 4-3. TSS Statistics (mg/l) for Belle Fourche River at Highway 79

Statistic	Pre-BMP Data (June 1977–April 2005)	All Data (June 1977–July 2006)	July 2005–July 2006
Mean	191	186	31.3
Median	18.0	18.0	32.0
Standard Deviation	887	875	17.0
Maximum	6,885	6,885	48
Number of Samples	106	109	3

TSS data collected at Horse Creek Above Vale (USGS 06436760) was analyzed using the Mann-Whitney U Test (also known as the “rank-sum test” or “Wilcoxon Rank-Sum test”). This is a nonparametric test that evaluates whether the median values of two groups of data are the same (the null hypothesis) and returns a confidence level for accepting or rejecting the null hypothesis. The confidence level is equivalent to the probability that choosing the alternate hypothesis is incorrect. Only data from Horse Creek Above Vale were analyzed with the Mann-Whitney U Test; not enough new samples are available at the DENR sites to develop a valid comparison.

TSS data collected at Horse Creek was separated into samples collected before June 2005 (group A) and samples collected after June 2005 (group B). The data were converted into total load per day by multiplying concentration by the discharge measurement made at the time of the sample. Samples collected during flood flows were excluded because TSS values are expected to be high during floods. The null hypothesis (H_0) in the Mann-Whitney U Test was that no significant difference existed in the medians of the two sample sets (i.e., the median SSC load of group A was equal to the median TSS load of group B). The alternate hypothesis (H_1) was that there was a significant difference in medians, and that a probability greater than one-half was associated with a higher median TSS load in group A (pre-BMP) than in group B (post-BMP). This is expressed mathematically as follows:

$$\begin{aligned} H_o&: \text{Prob}\,[A > B] = 0.5 \\ H_1&: \text{Prob}\,[A > B] > 0.5. \end{aligned} \tag{4-1}$$

The result of the Mann-Whitney U Test indicates rejection of the null hypothesis at a confidence level of 0.261. This suggests that there is a probability of 26.1 percent of being incorrect (Type I error) when assuming the median value of TSS load before June 2005, is greater than median value of TSS load after June 2005. Although this indicates that TSS loadings were less after June 2005, the test has little power due to the relatively high probability of making a Type I error. Because the lag time between implementation and measurable improvements is usually several seasons as vegetation works to reestablish itself along impacted streams and overgrazed fields, these results were expected.

4.2 REDUCTION IN IRRIGATION RETURN FLOWS AND ASSOCIATED TOTAL SUSPENDED SOLIDS CONCENTRATIONS

One goal of the 10-year plan is to reduce irrigation return flows originating from the BFID, thereby reducing TSS loading to the Belle Fourche River. This process has begun with the implementation of BMPs along the BFID canals (Segment I). Horse Creek receives much of the irrigation return flows originating in the BFID. Monthly TSS data, for the period May 2004 to August 2006, is available from the USGS stream gaging site Horse Creek Above Vale, South Dakota (USGS 06436760, see Appendix A). Instantaneous discharge measurements taken during TSS sampling at this site are also available. Figure 4-1 shows TSS samples at Horse Creek converted to units of tons per day. High flows causing high TSS are shown circled on the graph. These flows were due to a heavy snowfall event that occurred during late winter 2006 and the subsequent snowmelt. High flows caused by quick snowmelt or intense rainfall are expected to naturally contribute large loadings of TSS.

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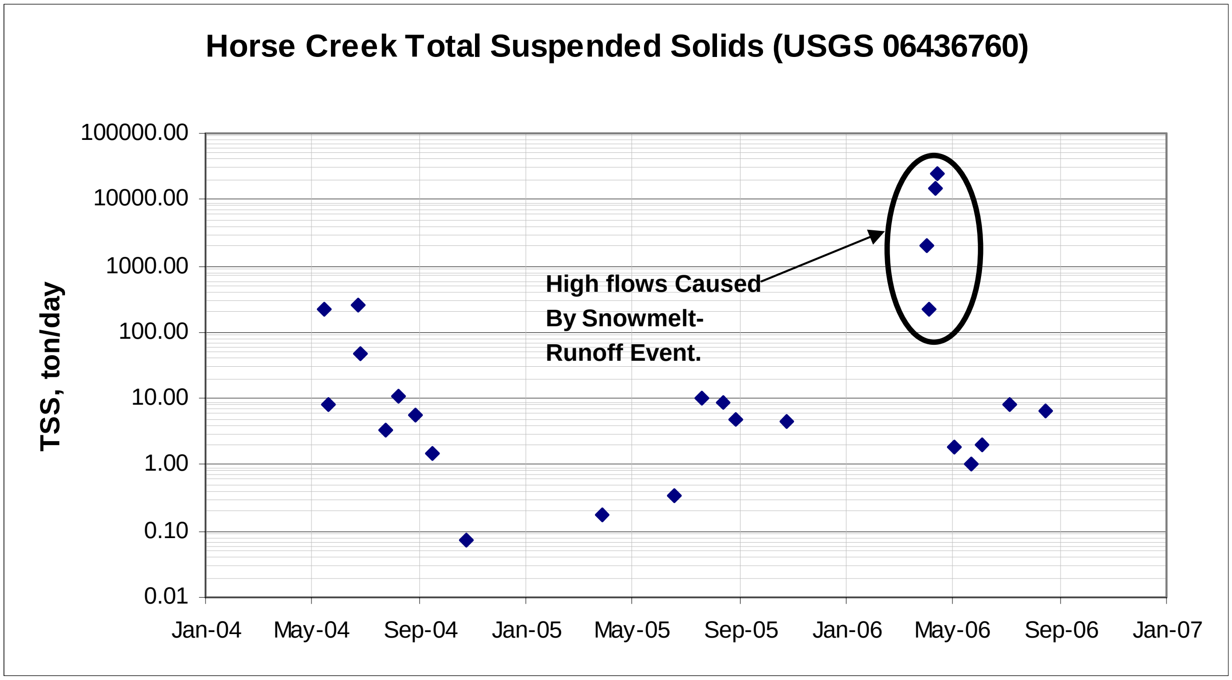

Figure 4-1. Suspended Sediment Loads at Horse Creek.

Although insufficient samples have been collected to determine TSS trends using rigorous statistical analysis techniques, the graph shows that TSS loadings are being maintained during 2005 and 2006, and suggests a downward trend from 2004 to present. TSS loading during the summer of 2006 varied from about one to ten tons TSS per day. Trend analysis techniques are typically used on long-term (i.e., 10+ years) time-series data. Long term TSS data collection began in Horse Creek in 2004, thus data is insufficient for analysis. Implementation of BMPs in the BFID and in the Belle Fourche River Watershed during 2005 and 2006 resulted in an estimated load reduction of 14,318 tons/year, and continued implementation in future years is expected to reduce TSS concentrations in the Belle Fourche River by reducing irrigation return flows and the high TSS associated with return flows.

Real-time discharge data at Horse Creek was collected by USGS from October 1980 to December 2006. As stated previously, implementation of BMPs in the BFID canals are expected to reduce return flows impacting Horse Creek. Figure 4-2 shows a box plot of USGS average daily discharge data for two time periods, pre-BMP (1980–May 2005) and post-BMP

(June 2005–November 2006). The box plot 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 is marked with a plus sign, 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. Traditional box plot whiskers extend to 1.5 times the inner-quartile range; in this case the lower whiskers would extend into negative values, hence the use of whiskers to mark 95 percent of data.

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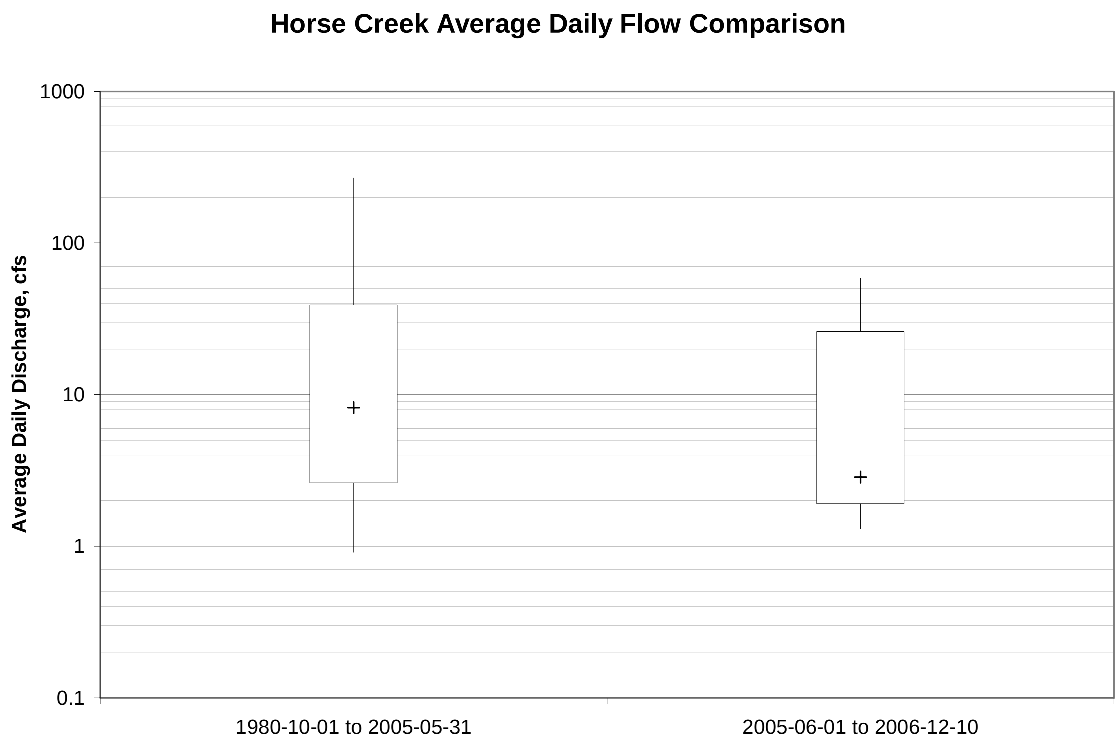

Figure 4-2. Box plot of Pre-and Post-BMP Average Daily Discharge Data at Horse Creek.

Box plots are effective and convenient tools for visualizing relationships between datasets that are too limited to analyze using other statistical methods. The box plot in Figure 4-2 implies that flow reductions are occurring in Horse Creek. Values for medians and quartiles are less in post-BMP data than in pre-BMP data. Long-term data is expected to show reductions in flows in Horse Creek associated with irrigation return flows, and consequential reductions in TSS in Horse Creek and the Belle Fourche River.

The Mann-Whitney U test was used to compare pre-BMP flows to post-BMP flows in Horse Creek. USGS average-monthly-flow data were used, with data sets separated into before June 2005 (set A) and after June 2005 (set B). The null hypothesis was that the median monthly flow after June 2005 was equal to the median monthly flow before June 2005. The alternate hypothesis was that the median monthly flow after June 2005 was less than the median monthly flow before June 2005. This is expressed mathematically as follows:

$$\begin{aligned} &H_0 : \text{Prob}\,[B > A] = 0.5 \\ &H_1 : \text{Prob}\,[B > A] < 0.5 \end{aligned} \tag{4-2}$$

The result of the Mann-Whitney U Test indicates rejection of the null hypothesis at a confidence level greater than 0.99. This suggests that there is a probability less than 1 percent of being incorrect (Type I error) when rejecting the null hypothesis and assuming the median value of monthly flow after June 2005 is greater than median value of monthly flow before June 2005. Although this test indicates high confidence that irrigation return flows are being reduced in Horse Creek, it must be remembered that flows in any stream display long-term variations caused by many factors, including drought and above-average precipitation that typically extend over periods of years. Long-term monitoring at Horse Creek and Belle Fourche River sites downstream of BMPs is necessary to separate out effects from precipitation and BMP improvements and evaluate associated trends in sediment loadings and flows.

Automatic gate controls are included in BMPs recommended to improve irrigation efficiency and reduce return flows. Water levels upstream of gate check structures must remain at set levels to provide control of the amount of water delivered to irrigators via laterals or turnouts. When levels fluctuate, irrigators and ditch-riders must continually adjust lateral and check boards, resulting in deliveries of too much or not enough water. This compounds the problem of estimating total water deliveries and reservoir releases required to supply orders. Maintaining constant upstream water levels by manually adjusting gates is difficult due to changes caused by releases at laterals and turnouts, and surges caused by releases from the reservoir. Automated gates can continually adjust water levels at check structures, letting surges through to supply downstream irrigators with ordered water while maintaining a set water level that allows upstream irrigators to receive ordered water. Figure 4-3 shows a comparison of water levels behind the Beals check structure before and after an automatic gate was installed.

Water levels before gate installation (2005) fluctuate, are lower, and do not display the control required to deliver precise quantities of water to upstream irrigators. The lower water levels during 2005 indicate that too much water was being released at the gate to downstream canal sections, possibly leaving upstream irrigators without the head required to supply water at turnouts. Peaks are caused by surges from reservoir releases or gate/weir board adjustments. Fluctuations are caused by combinations of surges along with adjustments at gates and upstream turnouts and laterals. When canal water-level fluctuation occurs, gates, turnouts and laterals must be adjusted, which causes further fluctuation (the result is level control is difficult and time consuming).

Water levels after gate installation (2006) remained at set target levels as shown in Figure 4-3. When a surge arrives at a canal check structure, the gate automatically adjusts to let the surge pass to downstream irrigators while maintaining the upstream water level. Rises and drops in the canal water level are caused by manual adjustment of the “target” water level that the on-site control unit uses to maintain a constant level. The target level can be adjusted on site by the ditch rider, or adjusted from the office for those sites which are equipped with real-time technology. The graph shows how adjusting the target water level in the controller causes immediate response in the canal, which reduces unused high-TSS irrigation water returning to Horse Creek and the Belle Fourche River, improves delivery estimates, and reduces the time that ditch riders and irrigators must spend monitoring and manually adjusting gates and turnouts.

RSI-1540-06-012

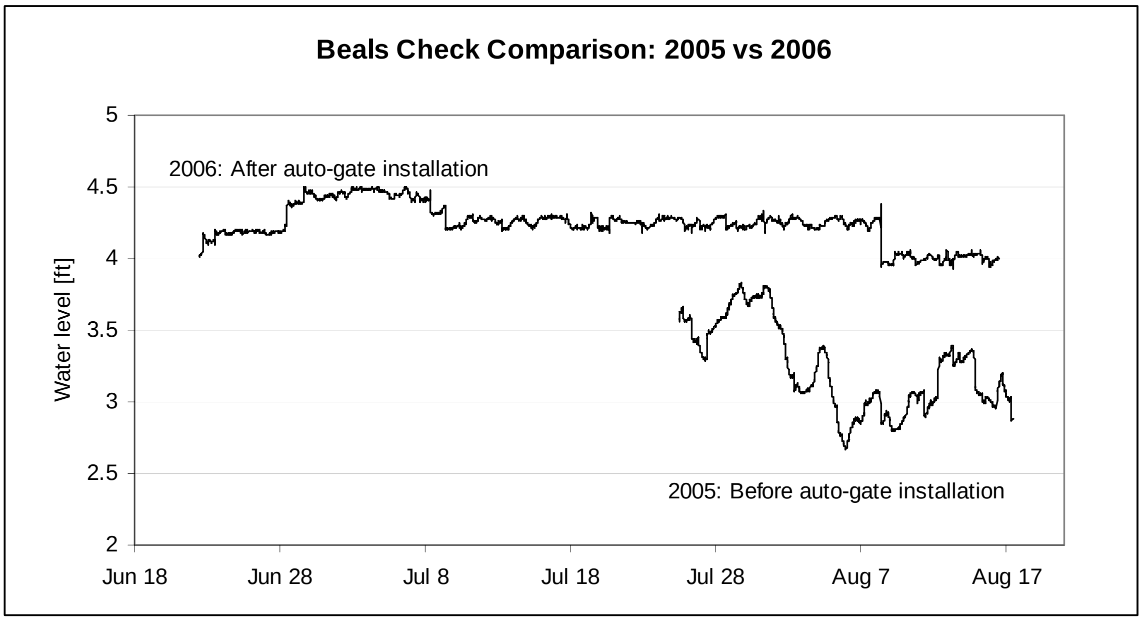

Figure 4-3. Comparison of Water Levels at Beals Check Structure Before and After Automatic Gate Installation.

Another example of the effectiveness of automated checks in reducing irrigation return flows was demonstrated at the Vale Lateral, located near the bottom of the South Canal. The Vale Check, which is approximately 50 feet downstream of the Vale Lateral, is used to control the pool level above the lateral. Figure 4-4 displays the actual water level maintained at the Vale Check from August 1–14, 2006. During this period, the target level of 4 feet plus or minus 0.05 feet was maintained over 98 percent of the time. The program moved the automated gate 55 times to maintain the target water level during this 14 day period, an average of about four changes per day. In personal conversations with the BFID manager, a ditch rider has the time to visit a site to adjust gates once each day, which makes this level of control essentially impossible without the aid of automation.

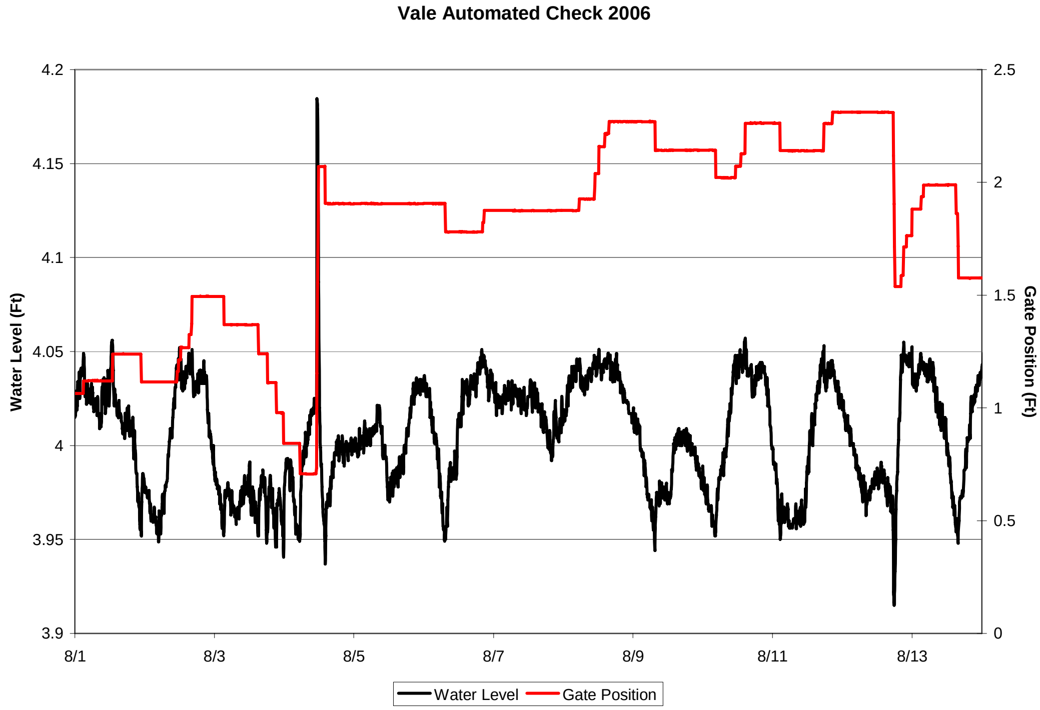

Figure 4-4. Automated Gate Position Adjustments Required to Control Target Water Level at Vale Check.

Pool level data from 2005 (preautomation) and 2006 (postautomation) were used to determine the change in delivery realized from the automation. Water orders (shown in Table 4-4) vary greatly from day to day and year to year. The total water ordered from August 1st to the 3rd was nearly identical from 2005 to 2006 (43.5 and 43 cubic feet per second (cfs)) and therefore chosen for analysis.

The flow down the Vale Lateral was calculated by inputting the recorded pool level and area of the lateral opening into Equation 4-3, where Q is flow in cfs, C is a coefficient, A is area of the opening in square feet, g is the gravitational constant, and H is the pool level in feet over the opening.

$$Q = CA\sqrt{2gH}. \tag{4-3}$$

While the actual area of the lateral opening was not recorded for either year, it was assumed that the ditch rider set the level at 9 a.m. each morning based on the daily water orders for the lateral and the water height over the gate at that time. Because the water that flows through the lateral head gate empties into an open canal, a free flow condition was assumed (i.e., no effects from backwater).

Table 4-4. Water Orders for the Vale Lateral in cfs from August 1–14, 2005 and 2006

Date	2005 Preautomation Orders (cfs)	2006 Postautomation Orders (cfs)
August 1	17	13
August 2	18	13
August 3	8.5	17
August 4	8	16
August 5	4	11
August 6	0	10.5
August 7	0	12.5
August 8	5	10.5
August 9	1.5	8.5
August 10	0	9
August 11	7	9
August 12	6	6.5
August 13	4	7.5
August 14	8.5	7.5
Total	**87.5**	**151.5**

Figure 4-5 shows the difference between the water ordered for the Vale Lateral and the water actually delivered before automation (2005) and after automation (2006). A negative difference indicates that water intended to be delivered bypassed the lateral; a positive difference indicates that more water was delivered than was ordered. During the 3-day period during 2005, a total of 0.66 acre-feet of ordered water bypassed the Lateral turnout. Assuming an average 90-day irrigation season, this translates into a seasonal loss of 19 acre-feet. Comparatively, in the same 3-day time period during 2006, 0.11 acre-feet bypassed the Lateral, which translates into a seasonal loss of only 3 acre feet and an 84 percent improvement in delivery efficiency over the preautomated check gate. Reductions in water bypassing the Lateral will reduce high TSS waters being returned to Horse Creek and the Belle Fourche River.

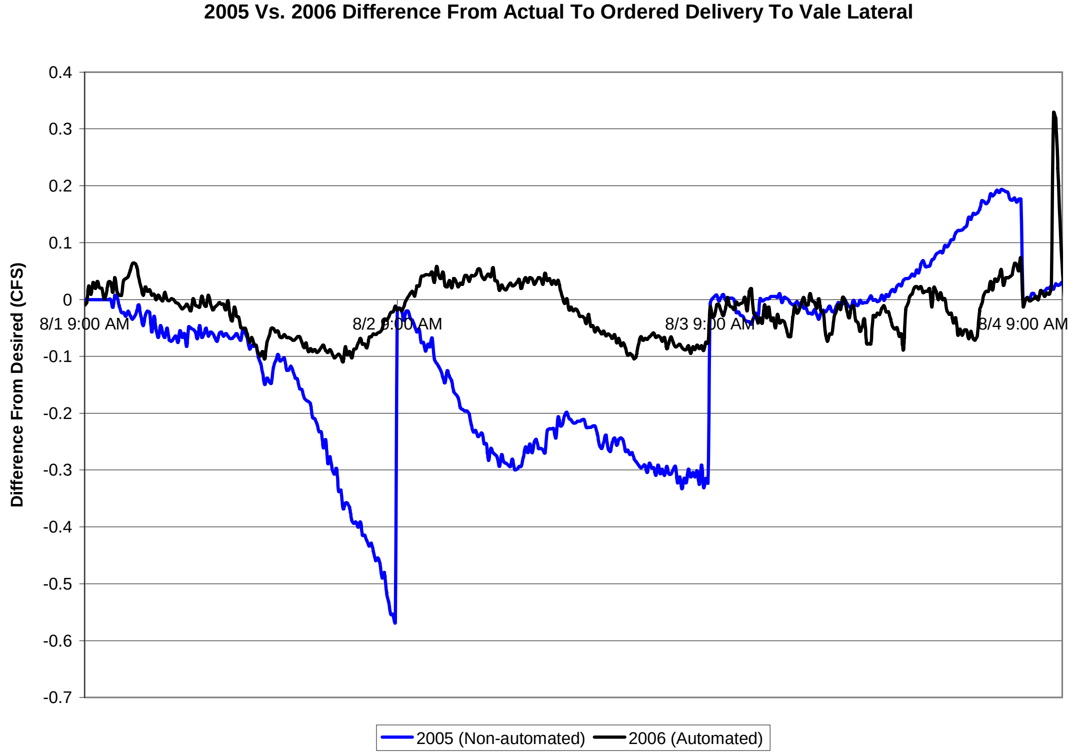

Figure 4-5. Comparison of Water Delivered Down the Vale Lateral Compared to Water Ordered in Years 2005 (Preautomation) and 2006 (Postautomation).

4.3 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 Phase I Watershed Assessment Final Report and TMDL [Hoyer and Larson, 2004].
- Load reductions, estimated as a result of BMP installation, of 14,318 tons per year which is 550 tons/year greater than the goal for the project.
- Completion of nearly 30 successful education and outreach activities which lead to greater public participation in the project.
- Completion of midyear and annual GRTS reports along with this final report.

Project activities that were not completed during this segment included lining of the Belle Fourche Reservoir inlet canal and statistical analysis of reductions of solids in the Belle Fourche River and Horse Creek. Lining of the inlet canal was not completed due to water shortages in the Reservoir, and is scheduled to be completed during September 2007. Statistical analysis of reductions of solids in the Belle Fourche River will be completed after sufficient time has elapsed for BMPs to take effect and sufficient samples have been collected.

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.

5.0 SUMMARY OF PUBLIC PARTICIPATION AND OUTREACH

The BFRWP partnered with DENR, USFWS, BOR, BFID, Elk Creek Conservation District, Lawrence County Conservation District, Butte County Conservation District, South Dakota Association of Conservation Districts, National Association of Conservation Districts, USGS, and local producers and interested parties to set up a Belle Fourche Watershed tour for a congressional staff delegation for over 50 people. The tour included stops at a flow automation and real-time monitoring site within the BFID, a sprinkler irrigation system used to improve irrigation efficiency, a ranch where new and improved grazing management systems were being implemented, and ponds developed for conserving water and improving water quality.

Several public education and outreach events were completed (Table 5-1). The Butte County, Lawrence County, and Elk Creek Conservation Districts each sent out newsletters which included project updates. The BFRWP hosted six meetings to provide updates on project work and progress being made. The BFID sent out a newsletter called the *Ditch Writer* to approximately 490 producers in the District informing them of the status of the projects going on throughout the District. Past and current project details are posted to the District's Web site at <*http://bfidsite.respec.com/*>. Consultants also presented the progress of the automation, updated water card system, and the modeling of the South Canal to approximately 45 producers at the BFID annual meeting.

The BFRWP Web site continues to be updated with happenings and project status, and is located at <*www.bellefourchewatershed.org*>. Table 5-1 shows all outreach and education activities that took place during this project segment.

The outreach activities described above helped increase participation and support in the Partnership and also gave the Partnership several contacts for BMP installation.

Table 5-1. Summary of Public Outreach and Education during Segment II

Outreach	Type of Education and Outreach	Date	Number of Participants
1	Belle Fourche River Watershed Partnership Meeting	June 2005	13
2	Belle Fourche River Watershed Partnership Meeting	August 2005	14
3	Congressional Tour	August 2005	50
4	Pierre Convention Meeting	September 2005	150
5	Belle Fourche River Watershed Partnership Meeting	October 2005	10
6	Ditch Writer Publication	December 2005	490
7	Public Information Meeting	January 2006	50
8	Belle Fourche River Watershed Partnership Meeting	January 2006	16
9	Newell Field and Home Show	February 2006	500
10	Belle Fourche Irrigation District Annual Meeting	February 2006	45
11	Belle Fourche River Watershed Partnership Meeting	June 2006	16
12	Area Meeting in Rapid City	July 2006	25
13	Belle Fourche River Watershed Partnership Meeting	August 2006	14
14	Butte-Lawrence County Fair	August 2006	2,000
15	Crooke County Conservation District Meeting	September 2006	8
16	Cheyenne River Watershed Meeting (Rapid City)	September 2006	30
17–20	Happenings Newsletter	Quarterly	750
21–26	Black Hills Multiple Use Coalition (Six Meetings)		33
27	Nonpoint Source Task Force Meeting (Pierre)	October 2005	45

6.0 ASPECTS OF THE PROJECT THAT DID NOT WORK WELL

The biggest challenge encountered during this project segment was implementing the irrigation efficiency improvements within the BFID. The contractor hired to install the automation systems did not complete installation within the time constraints of the contract. This necessitated the consultant and BFID personnel taking over installation so it could be completed before the start of the irrigation season. Once the automation systems were installed, there was a steep learning curve for the District personnel.

Another challenge was that several of the installed water-level measuring sensors malfunctioned. These sensors have been returned to the manufacturer and will be replaced with more durable units prior to the 2007 irrigation season. The sensor malfunctions resulted in some water losses, and, coupled with the learning curve and procedural changes the automation brought, caused reluctance by some of the BFID personnel to trust and use the automation systems. Training sessions are being held during the off-season to inform personnel of the reasons for sensor failures, demonstrate system effectiveness when operating correctly, and train personnel to correctly use the systems.

7. 0 PROJECT BUDGET/EXPENDITURES

The BFRWP received a $500,000 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]. All of the scheduled products were installed with no more than $36,542 (7.3 percent of the grant total) being transferred from any one product. This amount was moved from the riparian vegetation improvements budget to the operational model budget to facilitate completion of model during this project segment after it was determined several planned riparian vegetated improvements would not be installed until summer 2007. A similar adjustment was made to the Project Segment III budget. The planned and actual budgets are shown in Tables 7-1 and 7-2, respectively.

Table 7-1. Planned Belle Fourche River Watershed Implementation Budget by Funding Source

Total Budget	EPA 319	Producer	BFRWP	SD DENR Water Rights	Lawrence County	BFID	WY DEQ	USFWS	NRCS EQIP	COE	BOR	USGS	Line Item Total
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL													
Task 1. Reduce Nonused Water													
Product 1. Improved Irrigation Water Delivery 2,050 Ac ft Reduction of Nonused Water													
1a. 16 Flow Automation Projects	$ 164,000					$ 48,000							$ 212,000
1b. Water Card and Water Order System						$ 60,000							$ 60,000
1c. 6 portable and 9 real-time stage/flow measuring devices	$ 176,000												$ 176,000
1d. Operational Model	$ 57,000												$ 57,000
1e. Line open canals and laterals						$ 40,000					$ 40,000		$ 80,000
1f. Replace open canals and laterals with pipelines						$ 20,000					$ 20,000		$ 40,000
Product 2. Improved Irrigation Application 150 Ac-ft Reduction of Nonused Water													
2a. Pipeline projects delivering water from BFID to fields		$ 90,000							$ 30,000				$ 120,000
2b. Install Two Irrigation Sprinkler Systems		$ 71,400							$ 30,600				$ 102,000
Subtotal for Task 1.	$ 397,000	$ 161,400				$ 168,000			$ 60,600		$ 60,000		**$ 847,000**
Objective 2. Conduct Education and Outreach													
Task 2. Complete and Install Riparian Vegetation Improvements													
Product 3. Riparian Vegetation Improvements													
Riparian Vegetation Improvements	$ 41,000	$ 34,000						$ 12,500	$ 65,000				$ 152,500
Subtotal for Task 2.	$ 41,000	$ 34,000						$ 12,500	$ 65,000				**$ 152,500**
Objective 3. Tracking Progress Towards Meeting TMDL Goals													
Task 3. Conduct Public Outreach Program													
Product 4. Supplement Existing Outreach Programs													
4a. Public Meetings	$ 10,000		$ 5,000										$ 15,000
4b. BFRWP Meetings			$ 8,000										$ 8,000
Subtotal for Task 3.	$ 10,000		$ 13,000										**$ 23,000**
Task 4. Reports													
Product 5. Reports	$ 34,000												$ 34,000
Monitoring Progress Against Plan	$ 18,000			$ 30,750	$ 6,150	$ 4,875	$ 6,150			$ 6,150	$ 3,076	$ 84,563	$ 159,714
Subtotal for Task 4.	$ 52,000			$ 30,750	$ 6,150	$ 4,875	$ 6,150			$ 6,150	$ 3,076	$ 84,563	**$ 193,714**
Product Total By Source	$ 500,000	$ 195,400	$ 13,000	$ 30,750	$ 6,150	$ 172,875	$ 6,150	$ 12,500	$ 125,600	$ 6,150	$ 63,076	$ 84,563	$ 1,216,214

Table 7-2. Actual Expenditures of Belle Fourche River Watershed Implementation by Funding Source

Total Budget	EPA 319	Producer	BFRWP	SD DENR Water Rights	Lawrence County	BFID	WY DEQ	USFWS	NRCS EQIP	COE	BOR	USGS	Line Item Total
Objective 1. Implement BMPs Recommended in the Belle Fourche River Watershed TMDL													
Task 1. Reduce Nonused Water													
Product 1. Improved Irrigation Water Delivery 2,050 Ac ft Reduction of Nonused Water													
1a. 16 Flow Automation Projects	$ 164,000					$ 52,875							$ 216,875
1b. Water Card and Water Order System						$ 60,000							$ 60,000
1c. 6 portable and 9 real-time stage/flow measuring devices	$ 176,000												$ 176,000
1d. Operational Model	$ 93,542												$ 93,542
1e. Line open canals and laterals						$ 40,000					$ 83,000		$ 123,000
1f. Replace open canals and laterals with pipelines						$ 20,000					$ 42,000		$ 62,000
Product 2. Improved Irrigation Application 150 Ac-ft Reduction of Nonused Water													
2a. Pipeline projects delivering water from BFID to fields		$ 38,809							$ 35,000				$ 73,809
2b. Install Two Irrigation Sprinkler Systems		$ 175,208							$ 38,694				$ 213,902
Subtotal for Task 1.	$ 433,542	$ 214,017				$ 172,875			$ 73,694		$ 125,000		**$ 1,019,128**
Objective 2. Conduct Education and Outreach													
Task 2. Complete and Install Riparian Vegetation Improvements													
Product 3. Riparian Vegetation Improvements													
Riparian Vegetation Improvements	$ 4,458	$ 44,701						$ 12,500	$ 51,906				$ 113,565
Subtotal for Task 2.	$ 4,458	$ 44,701						$ 12,500	$ 51,906				**$ 113,565**
Objective 3. Tracking Progress Towards Meeting TMDL Goals													
Task 3. Conduct Public Outreach Program													
Product 4. Supplement Existing Outreach Programs													
4a. Public Meetings	$ 10,000		$ 5,000										$ 15,000
4b. BFRWP Meetings			$ 8,000										$ 8,000
Subtotal for Task 3.	$ 10,000		$ 13,000										**$ 23,000**
Task 4. Reports													
Product 5. Reports	$ 34,000												$ 34,000
Monitoring Progress Against Plan	$ 18,000			$ -	$ -	$ -	$ -			$ -	$ -	$ -	$ 18,000
Subtotal for Task 4.	$ 52,000			$ -	$ -	$ -	$ -			$ -	$ -	$ -	**$ 52,000**
Product Total By Source	$ 500,000	$ 258,718	$ 13,000	$ -	$ -	$ 172,875	$ -	$ 12,500	$ 125,600	$ -	$ 125,000	$ -	$ 1,207,693

8.0 FUTURE ACTIVITY RECOMMENDATIONS

The second segment of the Belle Fourche River Watershed Management Project was a success. All of the BMPs planned for this project segment were installed on time and within the proposed budget. Also several public outreach activities were completed and were used to inform producers and stakeholders of the progress of the current projects and also to promote the benefits of implementing recommended BMPs.

During the next 8 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 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

Hoyer, D. P. and A. Larson, 2004. *Phase I Watershed Assessment Final Report and TMDL*, prepared for the state of South Dakota, Pierre, SD.

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

UNITED STATES GEOLOGICAL SURVEY
TOTAL SUSPENDED SOLIDS DATA
AT HORSE CREEK

Table A-1. United States Geological Survey Total Suspended Solids Data at Horse Creek

Date[a]	Time	TSS, mg/l	Discharge, cfs
5/27/2004	11:10	1280	64
6/2/2004	11:12	203	15
7/6/2004	13:05	860	115
7/8/2004	9:54	320	56
8/6/2004	12:45	58	22
8/20/2004	9:30	81	51
9/7/2004	11:35	41	52
9/28/2004	10:45	96	5.8
11/5/2004	9:52	15	1.8
4/8/2005	12:00	52	1.3
6/28/2005	9:05	67	1.9
7/29/2005	11:30	79	48
8/22/2005	11:25	71	44
9/7/2005	9:20	55	32
11/02/2005	11:00	61	28
04/10/2006	12:40	5830	129
04/12/2006	14:30	3020	28
04/21/2006	16:30	6930	811
04/23/2006	14:55	5300	1680
05/12/2006	09:55	88	7.7
06/01/2006	11:15	126	3
06/13/2006	11:25	88	8.5
07/14/2006	12:00	83	37
08/23/2006		99	24

(a) At the time of report publication, data collected on and after November 2, 2005, is preliminary and subject to revision pending analysis by USGS.

APPENDIX B

SOUTH DAKOTA DEPARTMENT OF ENVIRONMENT AND NATURAL RESOURCES TOTAL SUSPENDED SOLIDS AT BELLE FOURCHE RIVER AT BELLE FOURCHE

Table B-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Belle Fourche (Page 1 of 2)

Date	Parameter	Value (mg/l)
1999/04/29	TSS	518
1999/07/21	TSS	964
1999/10/26	TSS	7
2000/01/19	TSS	6
2000/04/26	TSS	4520
2000/07/10	TSS	8
2000/10/17	TSS	11
2001/01/31	TSS	5
2001/04/16	TSS	110
2001/07/17	TSS	330
2001/10/30	TSS	2.5
2002/01/24	TSS	2.5
2002/04/16	TSS	10
2002/07/16	TSS	12
2002/10/31	TSS	2.5
2003/01/07	TSS	7
2003/04/17	TSS	10
2003/05/08	TSS	49
2003/06/04	TSS	15
2003/07/09	TSS	690
2003/08/21	TSS	120
2003/09/16	TSS	130
2003/10/16	TSS	2.5
2003/11/18	TSS	8

Table B-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Belle Fourche (Page 2 of 2)

Date	Parameter	Value (mg/l)
2003/12/09	TSS	2.5
2004/01/14	TSS	2.5
2004/02/18	TSS	6
2004/03/11	TSS	1400
2004/04/05	TSS	2.5
2004/05/12	TSS	2.5
2004/06/08	TSS	21
2004/07/14	TSS	64
2004/08/23	TSS	2.5
2004/09/14	TSS	10
2004/10/12	TSS	2.5
2004/11/16	TSS	2.5
2004/12/08	TSS	5
2005/02/17	TSS	5
2005/03/17	TSS	2.5
2005/04/18	TSS	2.5
2005/05/24	TSS	64
2005/06/21	TSS	2.5
2005/07/13	TSS	16
2005/08/24	TSS	140
2005/09/21	TSS	7
2006/07/26	TSS	85[a]
2006/08/23	TSS	130[a]

(a) Data is preliminary and subject to change pending review by South Dakota Department of Environment and Natural Resources.

APPENDIX C

SOUTH DAKOTA DEPARTMENT OF ENVIRONMENT AND NATURAL RESOURCES TOTAL SUSPENDED SOLIDS AT BELLE FOURCHE RIVER AT VALE

Table C-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Vale (Page 1 of 5)

Date	Parameter	Value, mg/l
1977/06/27	TSS	150
1977/10/26	TSS	48
1978/01/12	TSS	885
1978/03/29	TSS	221
1978/05/02	TSS	254
1978/07/11	TSS	54
1978/10/18	TSS	19
1979/01/10	TSS	77
1979/04/17	TSS	31
1979/11/05	TSS	2
1980/02/04	TSS	3
1980/08/06	TSS	61
1981/05/14	TSS	37
1981/11/02	TSS	22
1982/05/06	TSS	24
1982/08/19	TSS	35
1982/11/02	TSS	24
1983/02/03	TSS	9
1983/05/12	TSS	173
1983/08/04	TSS	45
1984/02/08	TSS	13
1984/05/09	TSS	177
1984/08/08	TSS	69
1984/11/14	TSS	17

Table C-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Vale (Page 2 of 5)

Date	Parameter	Value, mg/l
1985/02/07	TSS	3
1985/05/08	TSS	64
1985/08/22	TSS	68
1985/11/06	TSS	49
1986/02/21	TSS	2
1986/05/06	TSS	112
1986/08/13	TSS	12
1986/11/05	TSS	34
1987/02/24	TSS	6
1987/05/13	TSS	57
1987/08/19	TSS	39
1987/11/03	TSS	22
1988/02/17	TSS	3
1988/05/25	TSS	63
1988/08/25	TSS	60
1988/11/09	TSS	5
1989/02/14	TSS	3
1989/05/10	TSS	98
1989/08/09	TSS	69
1989/11/08	TSS	14
1990/02/12	TSS	4
1990/05/01	TSS	20
1990/08/14	TSS	104
1990/11/07	TSS	4

Table C-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Vale (Page 3 of 5)

Date	Parameter	Value, mg/l
1991/02/11	TSS	4
1991/05/06	TSS	32
1991/08/12	TSS	93
1991/11/12	TSS	11
1992/02/03	TSS	4
1992/05/18	TSS	78
1992/08/10	TSS	105
1992/11/09	TSS	5
1993/02/08	TSS	2.5
1993/05/12	TSS	85
1993/08/16	TSS	49
1993/11/10	TSS	11
1994/01/12	TSS	1
1994/04/28	TSS	460
1994/07/19	TSS	88
1994/10/04	TSS	28
1995/01/04	TSS	4
1995/04/13	TSS	29
1995/07/19	TSS	83
1995/10/17	TSS	19
1996/01/10	TSS	10
1996/04/29	TSS	66
1996/07/24	TSS	46
1996/10/15	TSS	22

Table C-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Vale (Page 4 of 5)

Date	Parameter	Value, mg/l
1997/01/22	TSS	39
1997/04/23	TSS	595
1997/07/22	TSS	43
1997/10/28	TSS	7
1998/01/14	TSS	1
1998/04/20	TSS	140
1998/07/15	TSS	52
1998/10/14	TSS	22
1999/01/20	TSS	44
1999/04/29	TSS	766
1999/07/21	TSS	6
1999/10/26	TSS	9
2000/01/19	TSS	14
2000/04/26	TSS	828
2000/07/10	TSS	81
2000/10/17	TSS	18
2001/01/04	TSS	2.5
2001/04/16	TSS	240
2001/07/17	TSS	50
2001/10/30	TSS	15
2002/01/24	TSS	2.5
2002/04/16	TSS	79
2002/07/16	TSS	58
2002/10/31	TSS	2.5

Table C-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Vale (Page 5 of 5)

Date	Parameter	Value, mg/l
2003/01/07	TSS	7
2003/04/17	TSS	65
2003/07/09	TSS	57
2003/10/16	TSS	7
2004/01/14	TSS	18
2004/04/07	TSS	50
2004/07/14	TSS	52
2004/10/12	TSS	10
2005/01/19	TSS	2.5
2005/04/18	TSS	53
2005/07/13	TSS	32
2005/10/27	TSS	8[a]
2006/07/26	TSS	15[a]

(a) Data is preliminary and subject to change pending review by South Dakota Department of Environment and Natural Resources.

APPENDIX D

SOUTH DAKOTA DEPARTMENT OF ENVIRONMENT AND NATURAL RESOURCES TOTAL SUSPENDED SOLIDS AT BELLE FOURCHE RIVER AT HIGHWAY 79

Table D-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Highway 79 (Page 1 of 5)

Date	Parameter	Value
1977/06/27	TSS	6005
1977/10/26	TSS	6885
1978/01/12	TSS	200
1978/03/29	TSS	739
1978/07/11	TSS	40
1978/10/18	TSS	7
1979/01/10	TSS	4.5
1979/04/17	TSS	88
1979/11/05	TSS	3
1980/02/04	TSS	9
1980/08/06	TSS	31
1981/02/03	TSS	4
1981/05/14	TSS	46
1981/11/02	TSS	18
1982/05/06	TSS	10
1982/08/19	TSS	18
1982/11/02	TSS	22
1983/02/03	TSS	5
1983/05/12	TSS	245
1983/08/04	TSS	40
1984/02/08	TSS	12
1984/05/09	TSS	348
1984/08/08	TSS	31
1984/11/14	TSS	10

Table D-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Highway 79 (Page 2 of 5)

Date	Parameter	Value
1985/02/07	TSS	18
1985/05/08	TSS	44
1985/08/22	TSS	36
1985/11/06	TSS	7
1986/02/21	TSS	2
1986/05/06	TSS	39
1986/08/13	TSS	10
1986/11/05	TSS	6
1987/02/24	TSS	11
1987/05/13	TSS	21
1987/08/19	TSS	32
1987/11/03	TSS	7
1988/02/17	TSS	7
1988/05/25	TSS	58
1988/08/25	TSS	24
1988/11/09	TSS	7
1989/02/14	TSS	3
1989/05/10	TSS	740
1989/08/09	TSS	39
1989/11/08	TSS	17
1990/02/12	TSS	17
1990/05/01	TSS	18
1990/08/14	TSS	96
1990/11/07	TSS	11

Table D-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Highway 79 (Page 3 of 5)

Date	Parameter	Value
1991/02/11	TSS	8
1991/05/06	TSS	12
1991/08/12	TSS	37
1991/11/12	TSS	2.5
1992/02/03	TSS	6
1992/05/18	TSS	23
1992/08/10	TSS	41
1992/11/09	TSS	5
1993/02/08	TSS	4
1993/05/12	TSS	114
1993/08/16	TSS	36
1993/11/10	TSS	5
1994/01/12	TSS	1
1994/04/28	TSS	453
1994/07/19	TSS	42
1994/10/04	TSS	17
1995/01/04	TSS	6
1995/04/13	TSS	21
1995/07/19	TSS	79
1995/10/17	TSS	7
1996/01/10	TSS	12
1996/04/29	TSS	69
1996/07/24	TSS	13
1996/10/15	TSS	5

Table D-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Highway 79 (Page 4 of 5)

Date	Parameter	Value
1997/01/22	TSS	19
1997/04/23	TSS	553
1997/07/22	TSS	15
1997/10/28	TSS	3
1998/01/14	TSS	3
1998/04/20	TSS	150
1998/07/15	TSS	52
1998/10/14	TSS	55
1999/01/20	TSS	44
1999/04/29	TSS	692
1999/07/21	TSS	129
1999/10/26	TSS	11
2000/01/19	TSS	13
2000/04/26	TSS	662
2000/07/10	TSS	74
2000/10/17	TSS	10
2001/01/04	TSS	6
2001/04/16	TSS	230
2001/07/17	TSS	36
2001/10/30	TSS	6
2002/01/24	TSS	2.5
2002/04/16	TSS	35
2002/07/16	TSS	24
2002/10/31	TSS	8

Table D-1. South Dakota Department of Environment and Natural Resources Total Suspended Solids at Belle Fourche River at Highway 79 (Page 5 of 5)

Date	Parameter	Value
2003/01/07	TSS	10
2003/04/17	TSS	16
2003/07/09	TSS	28
2003/10/16	TSS	6
2004/01/14	TSS	2.5
2004/04/07	TSS	30
2004/07/14	TSS	100
2004/10/12	TSS	8
2005/01/19	TSS	5
2005/04/18	TSS	49
2005/07/13	TSS	32
2005/10/27	TSS	14[a]
2006/07/26	TSS	48[a]

(a) Data is preliminary and subject to change pending review by South Dakota Department of Environment and Natural Resources.

APPENDIX E

HYDRAULIC MODEL OF THE BELLE FOURCHE IRRIGATION DISTRICT SOUTH CANAL USING EPA SWMM VERSION 5.0

Hydraulic Model of the Belle Fourche Irrigation District South Canal Using EPA SWMM Version 5.0

by

Curtis David Schoenfelder

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

2006

Prepared by:

____

Curtis D. Schoenfelder, Degree Candidate

Approved by:

____

Dr. Scott J. Kenner, Major Professor

____

Dr. Scott J. Kenner, Chair, Department of Civil and Environmental Engineering

____

Dr. Kenneth N. Han, Dean, Graduate Division

ABSTRACT

The Belle Fourche River is located in northeastern Wyoming, west-central South Dakota, and the corner of southeastern Montana, and is identified in the South Dakota 2004 Integrated Report for Surface Water Quality Assessment 303(d) Waterbody List as impaired because of elevated total suspended solids (TSS) concentrations (Belle Fourche River Watershed Partnership, 2005). The Belle Fourche River Watershed Partnership is sponsoring a project with the overall goal of bringing the Belle Fourche River into compliance for TSS within 10 years through the implementation of several best management practices (BMPs). One of the recommended BMPs is the development and implementation of an operational model for the Belle Fourche Irrigation District (BFID). The model will help to improve the operational efficiency in the BFID, which will in turn reduce the amount of nonused irrigation return flows.

A hydraulic computer model of the BFID South Canal was developed using U.S. Environmental Protection Agency's Storm Water Management Model (SWMM) Version 5.0, which is a key component of the operational model, to provide the BFID with a useful tool and resource. The model was developed with the use of U.S. Bureau of Reclamation survey data and contract drawings as well as field measurements. Stage, flow, and structure setting data at various locations throughout the BFID were collected during the 2005 irrigation season. This set of data and measurements was used to calibrate/validate the model through the first 26.4 miles, from the Dam to the Vale Flume. Data also were collected during the 2006 irrigation season to validate the model using fewer assumptions and more exact system operational changes. The simulated depths during 2005 calibration/validation were ± 10% of the observed depths 94% of the simulations and ± 5% for 77% of the simulations. The simulated depths during 2006 validation were ± 10% of the observed depths 94% of the simulations and ± 5% for 58% of the simulations. A sensitivity analysis was conducted on the model to assess effects of calibrated parameters on model results and identify important trends in parameter adjustment. The model is fully capable of simulating the irrigation system, including automated gates, which are being installed at various locations throughout the BFID to improve operational efficiency. Real-time data are used as direct time series flow inputs and control rules are used to simulate the structure settings. The model can simulate the many possible combinations of flows and structure settings and will assist the BFID personnel in making adjustments to the system, which will aid in improving operational efficiency.

ACKNOWLEDGEMENTS

I am especially thankful to Dr. Scott Kenner for giving me the opportunity to come back to wonderful South Dakota and be involved in such a great applied research project. His knowledge and support have been tremendous. My experiences at South Dakota School of Mines and Technology were positively different from those at the University of Minnesota – Twin Cities and have provided me with a well rounded and very solid education that will carry over into my professional career. I would also like to thank Dr. Dan Hoyer of RESPEC for the continued support in my research efforts, and also my other committee members, Dr. Arden Davis and Dr. Thomas Fontaine.

I also owe great appreciation and thanks to my project partners, Ms. Cheryl Rolland and Mr. Tim Olson, who provided me with the background knowledge that I needed to come into the project and provide a useful product. Tim proved to be a great mentor in learning the complex Belle Fourche Irrigation District system. Thanks are extended to Mrs. Lacy Pomarleau for help during the 2006 irrigation season and for being such a quick learner and self-starter. Thanks also are extended to Mr. Patrick Schwickerath and Mr. Jared Oswald of RESPEC who provided guidance and support in all aspects of the project, and to Ms. Christina Omdahl for her excellent peer reviews and feedback. Thanks to the Belle Fourche River Watershed Partnership and all its entities for providing the funding and cooperation for such a great project to work on. And last, but definitely not least, thanks to Mr. Clint Pitts and the entire Belle Fourche Irrigation District staff for guidance, cooperation, and implementation of all cumulative research efforts conducted by my previous, present, and future peers and me.

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INTRODUCTION

Project Background

The Belle Fourche River watershed encompasses nearly 5 million acres in northeastern Wyoming, west-central South Dakota, and a small part of southeastern Montana, with just over 2 million of those acres being in South Dakota (Figure 1). Land use within the watershed consists primarily of livestock grazing, some cropland, and a few urban and suburban areas (Belle Fourche River Watershed Partnership, 2005). Approximately 65 percent of crop production in the BFID is alfalfa and hay. Small grains and corn account for the remaining crops. Also, some livestock and dairy production exists. The soils range from heavy clays with some silts and gravels in the North Canal area to clay/sand soils in the South Canal area (Rolland, 2005).

The Belle Fourche Irrigation District (BFID) maintains and operates irrigation facilities in the South Dakota portion of the watershed (Figure 1) for the U.S. Bureau of Reclamation (BOR). The irrigation facilities consist of the North Canal and the South Canal, both of which are earth canals. The North Canal is 43 miles long and has a design capacity of 600 cfs at the dam while the South Canal is 44 miles long with a design capacity of 400 cfs at the dam. Each canal feeds a network of approximately 450 miles of laterals that deliver water to fields (Hoyer, 2003). The BFID services over 57,000 acres of irrigable land and historically has an active water conservation program that includes lining of canals, piping, and operational and maintenance procedure improvements. The irrigation water allotted to the BFID comes from the Belle Fourche Reservoir (a.k.a. Orman Dam), which receives most of its inflow from water diverted from the Belle

Fourche River. The average water allocation to the BFID is approximately 15 inches, almost doubling the amount of water per acre within the irrigated acres that would be received from the watershed's average annual precipitation (Belle Fourche River Watershed Partnership, 2005).

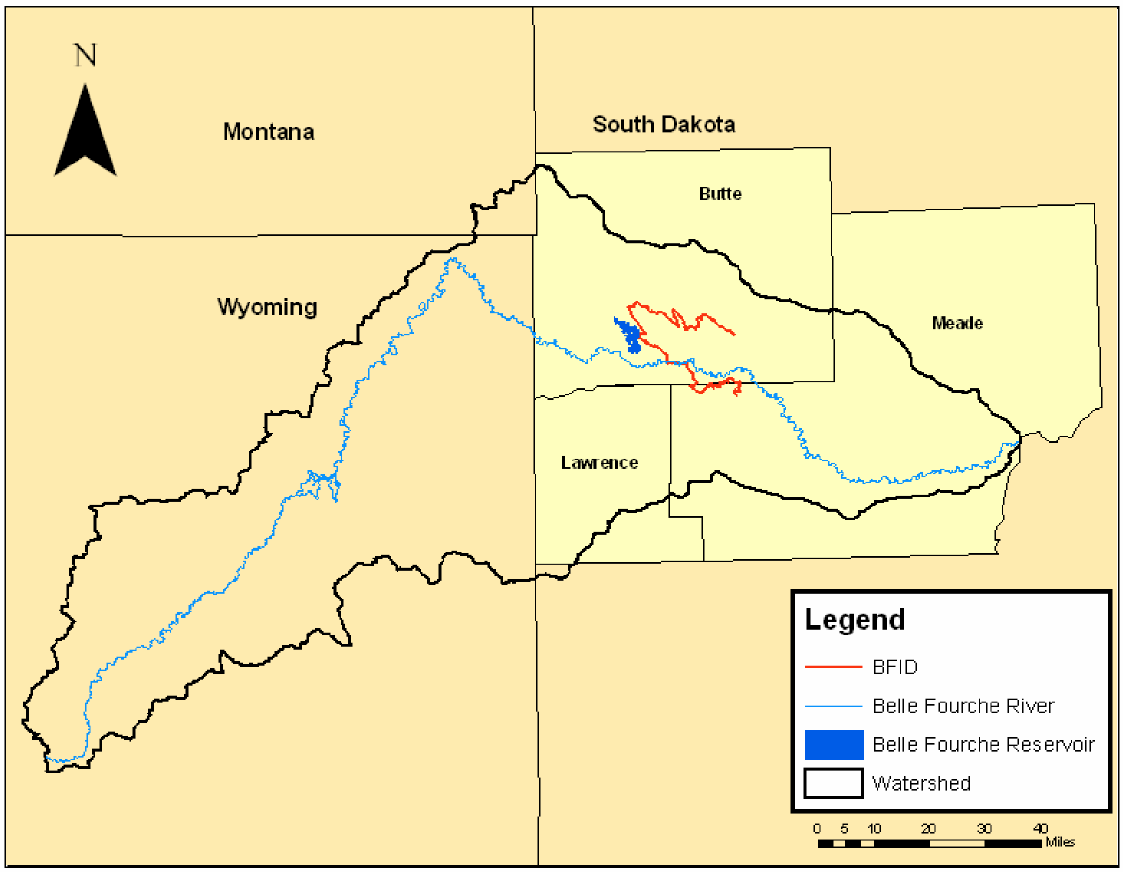

Figure 1. Belle Fourche watershed and Belle Fourche Irrigation District (Olson, 2006).

Irrigation has a significant impact on the Belle Fourche River and other streams within the watershed. The Belle Fourche River was listed as impaired on the *1998 South Dakota 303(d) Waterbody List* (Myers, 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) due to elevated total suspended solids (TSS) concentrations (Belle Fourche River Watershed Partnership, 2005). A total maximum daily load (TMDL) study (Hoyer and Larson, 2004) on the watershed identified the primary cause of TSS impairment to be natural bank sloughing, quantity of nonused irrigation water discharged to the natural waterways, and riparian habitat impairment. Stream entrenchment and bank failure are responsible for approximately 75 percent of the TSS in the Belle Fourche River system. Stream energy causes natural bank failure, particularly in the eastern portion of the watershed. These areas are dominated by high banks composed primarily of clay soils that, when eroded, supply suspended solids to the channel. Increased quantities of water resulting from the nonused irrigation return flows are the major driver causing the channel to incise, and result in additional bank failures and resultant suspended solids.

Approximately 64 percent of the water released into the BFID from the Belle Fourche Reservoir is delivered to the field and the remaining 36 percent is lost in return flows due to transportation and operational losses (Belle Fourche River Watershed Partnership, 2005). Transportation losses include seepage and evaporation. Operational losses include overflow from the canals, laterals, and gates/valves into adjacent waterways. The crops use approximately 32 percent of the water delivered to the field and the rest is lost through evaporation and nonused water discharged to adjacent waterways. Much of the irrigation in the watershed is flood-type. This type of irrigation results in sediments being mobilized by three processes: (1) as the 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. Irrigation and return flow waste are responsible for approximately 20 percent of the TSS in the Belle Fourche River system.

As a result of the Belle Fourche River Watershed TMDL report (Hoyer and Larson, 2004), several best management practices (BMPs) were recommended by the *Ten-Year Implementation Plan* (Hoyer and Schwickerath, 2005) with the goal of bringing the Belle Fourche River into compliance for TSS. The recommended BMPs included improved efficiency for delivery and application of irrigation waters and riparian vegetation improvements, and the BMPs were further defined in *Segment III of the Belle Fourche River Watershed Management and Project Implementation Plan* (Belle Fourche River Watershed Partnership, 2005). One of the recommended BMPs is the development and implementation of an operational model for the BFID. This paper focuses on the hydraulic model component of the operational model (Olson, 2006) and its development and application towards improving the operational efficiency in the BFID. Operational efficiency improvements will reduce the amount of nonused irrigation flows and in turn reduce the elevated TSS concentrations in the Belle Fourche River.

System Characteristics and Operation

The BFID structural characteristics are well defined by Olson (2006). Each of the two major canals, the North and South Canals, are controlled by a series of level pool check structures. The checks were designed to control the water surface elevation upstream to produce the necessary delivery head at each delivery structure. Water is delivered from the two major canals into laterals and farmer turnouts. Laterals are complex minor canal systems off the major canal that distribute water to multiple farmers. Farmer turnouts generally distribute water to a single farmer off the main canal and feed a field directly. Each ditch rider (system operator) is responsible for all checks, laterals, and farmer turnouts within a certain section of the major canal, which are called rides. The BFID monitors discharge with Parshall flumes and sharp-crested weirs located throughout the district on major canals, and some laterals and farmer turnouts. Discharge within a lateral system is monitored by flumes and division box weirs, which also exist just downstream of some farmer turnouts. Laterals and farmer turnouts without a weir box just downstream of the head gate on the main canal are monitored by ditch rider interpretation and experience to obtain correct discharges.

As described by Olson (2006), the operation of the BFID is governed by a series of dependent components, including both human and nonhuman. The three components to the demand/delivery system are: water call cards, Water Master sheets, and billing cards. The water call cards are the link between the farmers and ditch riders in which water orders are compiled by lateral or farmer turnout, including any additional water needed for proper delivery and system operation. The Water Master sheets include total orders according to the water call cards and are used to determine daily changes at the Dam. The billing cards document the amount of water allocated and the total water delivered to each farmer in the system.

There are several processes in the BFID that occur daily to operate the system smoothly. The daily process, beginning with water orders and ending with delivery of water to farmers, involves the following interactions and transfers of information (Olson, 2006):

1. Farmer/Ditch Rider: Farmers order water from the ditch rider taking into consideration water travel time as a result of their distance from the Dam.

2. Ditch Rider/Data Entry: The ditch rider presents the water call card to the data entry person, who enters the water orders into the database and calculates a summary of demands and the Water Master sheet.

3. Data Entry/Water Master: The data entry person presents the Water Master sheet to the Water Master, who then makes the necessary changes at the dam.

4. Data Entry/Ditch Rider: A check structure demand schedule is produced using the water call cards that can be used to make decisions about system operation.

5. Ditch Rider/Farmer: The ditch rider releases water into farmer turnout and lateral systems when available, changes are made if necessary, and the process repeats.

Previous Modeling Efforts

Previous research efforts included hydraulic model investigation, comparison, and simple trials of the BFID South Canal (Rolland, 2005). Two models were originally considered, U.S. Environmental Protection Agency's Storm Water Management Model Version 5.0 (EPA SWMM 5.0) and RootCanal (Utah State University). Issues discussed and compared between the two models 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 several overall advantages and disadvantages for the various issues.

Rolland (2005) did some simple modeling trials on the South Canal using SWMM and RootCanal. The models were used to simulate simplified typical conditions on the first eight miles (Dam to Belle Fourche River Siphon Flume) of the South Canal in an effort to compare Dam release travel times. Both models produced fairly similar results and conclusions. Rolland (2005) concluded from the trials that the time required for a change in discharge at the Dam to reach the BFRS Flume is much longer according to the models than the time used by the BFID, although the trial results were not validated with actual data. These results indicated that the BFID's problem of "missing water" could actually be that the water had not arrived yet. Work was also done using various sources, field measurements, and modeling trials to determine weir and gate discharge coefficients at the Vale Check, a typical South Canal check structure. The gate and weir discharge coefficients were determined to be 0.65 and 3.0, respectively.

Rolland (2005) found that despite RootCanal being developed specifically for irrigation applications, SWMM would be the better choice for modeling the BFID. SWMM seemed the better choice primarily because of its greater capabilities for unsteady flow computations and its general reliability, and because RootCanal was still in the development phase (Rolland, 2005).

Objectives

The overall goal of the *Belle Fourche River Watershed Management and Project Implementation Plan* is to bring the Belle Fourche River into compliance for TSS through the implementation of recommended BMPs (Belle Fourche River Watershed Partnership, 2005). One segment of BMPs includes reducing nonused irrigation water discharged to local waterways from the delivery and application system of the BFID where approximately 37 percent of the overall TSS reduction will be achieved. The objective of this research is to produce a hydraulic computer model that will provide a useful tool and reference that will aid the BFID in making changes and adjustments to the system.

Scope and Approach

The research presented in this report focuses on the South Canal of the BFID. A hydraulic computer model of the BFID South Canal was developed in an effort to improve the operational efficiency in the BFID, which will in turn reduce the amount of nonused irrigation flows. The details of the report include model development, model calibration/validation using two irrigation seasons of data, model sensitivity analysis, model application, conclusions, and recommendations for future modeling efforts. The hydraulic model was developed for the entire 44 miles of the South Canal, from the Dam to the Wasteway. The calibration/validation of the model was focused on the first 26.4 miles of the South Canal, from the Dam to the Vale Flume. The hydraulic model component of the operational model will help improve the delivery and application efficiency of the BFID. The objective of the hydraulic model will be achieved through its ability to simulate the many possible flow scenarios and structure setting combinations seen throughout the BFID on a day-to-day basis and throughout the irrigation season. Model simulations will provide a tool to analyze, predict, and assess operational changes and their effects throughout the system.

MODEL DEVELOPMENT

EPA SWMM 5.0

EPA SWMM 5.0 was chosen to develop the hydraulic model of the BFID South Canal for reasons determined by Rolland (2005) and also because it is widely used and accepted, well documented and proven, employs powerful hydraulic computational methods, and has an easy to use graphical user interface (GUI) and user's manual. EPA SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas (Environmental Protection Agency, 2005). SWMM has two major components that track the quantity and quality of runoff and flow though a simulated system: the runoff component and the routing component. SWMM was first developed in 1971 and has undergone several major upgrades to its current edition, Version 5. SWMM 5 provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations, and viewing the results in a variety of formats (Environmental Protection Agency, 2005). Only the hydraulic component of SWMM is used in the BFID South Canal model.

Modeling of the BFID Irrigation System

Input and Development Data

BOR survey data and contract drawings were collected and used to develop the SWMM model of the BFID South Canal. The survey data collected included: stationing of all structures (checks, turnouts, bridges/box culverts, siphons, and flumes), turnout pipe diameters, canal centerline invert elevations, turnout pipe invert elevations, top of structure elevations, and left and right toe and top of bank elevations. The survey data was mostly complete; where data was not found it was either interpolated or measured in the field if possible. BOR contract drawings provided all dimensions of most check structures on the canal, including gate/weir chamber dimensions and invert elevations. All check structure dimensions were verified with field measurements. Most siphon and flume data were also available.

The horizontal alignment of the South Canal model was laid out using a GIS map converted to a BMP file for the background in SWMM. The BFID irrigable acreage land maps were then used to place the structures and canal components accordingly. Although horizontal alignment is not important in SWMM it provides a realistic picture of the canal and serves as a visualization tool. The vertical profile of the South Canal was laid out using the BOR stationing and elevations. Figure 2 shows the plan view with all check structures and Figure 3 shows the vertical profile of the BFID South Canal modeled in SWMM.

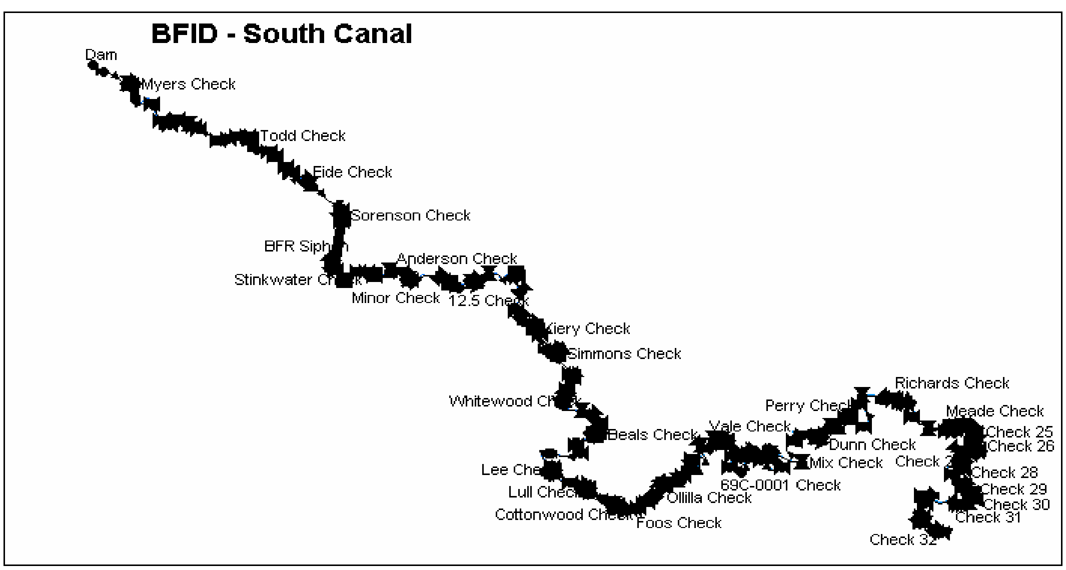

Figure 2. SWMM model's plan view.

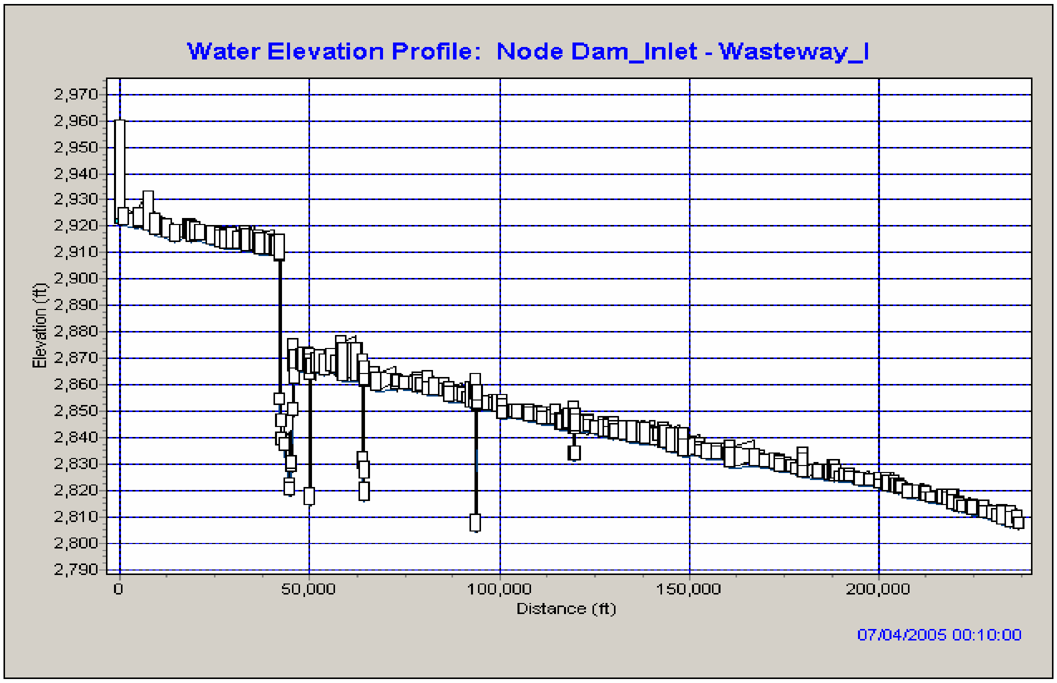

Figure 3. SWMM model's vertical profile.

Simulation of BFID Components

Modeling of Structures

A check structure holds a pool at a certain depth in order to serve upstream laterals and/or turnouts. Check structures along the South Canal vary in number and size of sluice gates and adjustable check-board weirs (Figure 4 and Figure 5). There are a total of 32 checks along the South Canal. All check structures were modeled as a combination of orifices (sluice gates) and weirs (check-board weirs) in SWMM by appropriately varying the number of chambers and dimensions. Figure 6 shows the Vale Check modeled in SWMM with an inlet junction node (Vale_Check_I), an outlet junction node (Vale_Check_O), and the gate and weir chambers that convey the water through the simulated structure. Sluice gates were modeled initially using the default discharge coefficient of 0.65 and the check-board weirs were modeled initially using a discharge coefficient of 2.6 for broad crested weirs. Check structures were also assigned entrance and exit loss coefficients of 0.4 and 1, respectively (Mays, 2001; Sturm, 2001).

Figure 4. A common Reach 4 check structure.

Figure 5. Meyer Check structure on Reach 1.

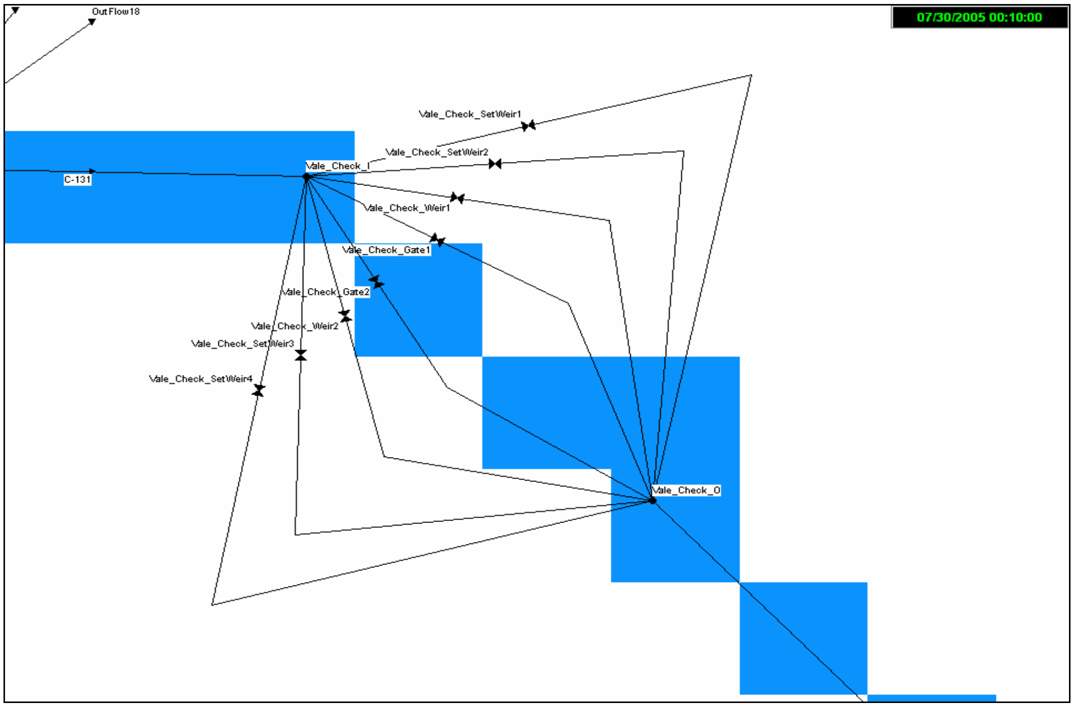

Figure 6. Vale Check structure modeled in SWMM.

Several key check structures throughout the BFID were equipped with automated gates prior to the 2006 irrigation season in order to improve operational efficiency. Automated gates control the pool level held by a check structure through the use of pressure transducer level sensors and gate actuators to maintain a constant pool level and account for fluctuations in the canal. Automated gates can be modeled in SWMM by assigning control rules that adjust the gate to hold the pool level at the specified depth. Automated gates were not simulated during the model calibration because they were not installed during the 2005 irrigation season, but they were simulated during validation with 2006 irrigation season data. An example application of the SWMM simulation of the Vale Check automated gate is presented in Figure 7, in which the automated gate is specified to hold a depth of 4 feet. The simple control rules written in SWMM to simulate the automated Vale Check gate are:

RULE close Vale automation

IF NODE Vale_Check_I DEPTH < 4

THEN ORIFICE Vale_Check_Gate2 SETTING = 0.2

PRIORITY 0.2

RULE open Vale automation

IF NODE Vale_Check_I DEPTH > 4

THEN ORIFICE Vale_Check_Gate2 SETTING = 0.8

PRIORITY 0.8

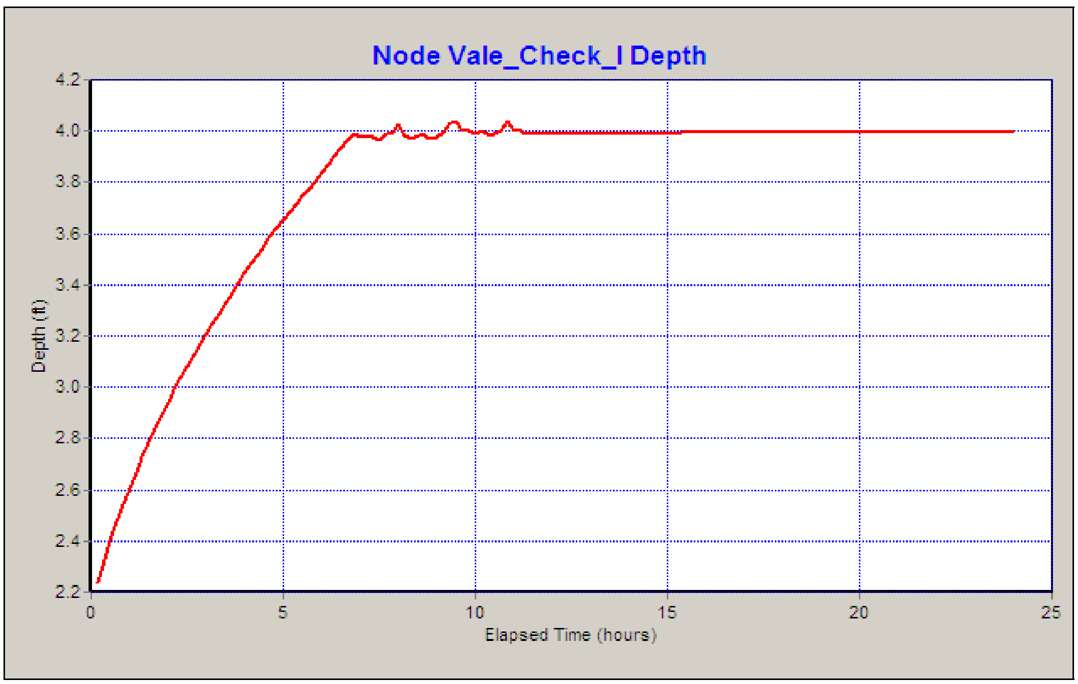

Figure 7. SWMM simulation of automated Vale Check structure.

Siphons are gravity pressurized pipe sections that serve to convey irrigation canal water below any major waterways in the canal's path. There are 5 reinforced concrete pipe (RCP) siphons along the South Canal: Belle Fourche River Siphon (BFRS), Stinkwater Siphon, Anderson Siphon, Whitewood Siphon, and Cottonwood Siphon. The siphons were modeled using collected or interpolated elevations, stationing, and appropriate pipe diameters in combination with specified surcharge depths at the pipe junctions. The surcharge depth in SWMM is the maximum depth before flooding occurs and allows the junctions to be modeled as pressurized pipe section fittings. The surcharge depth was specified so that it exceeded the maximum depth needed to overcome the head increase; the distance from the lowest part of the siphon profile to the inlet elevation. Thus, a siphon will function as a pressurized system without flooding. Siphons were also assigned entrance and exit loss coefficients of 0.4 and 1, respectively, and each section of siphon pipe conduit was also given bend loss coefficients of 0.2 and Manning's n values of 0.013 (Sturm, 2001; Mays, 2001). Figure 8 shows the SWMM water surface elevation profile of the BFRS under the typical early-season operating condition of 150 cfs.

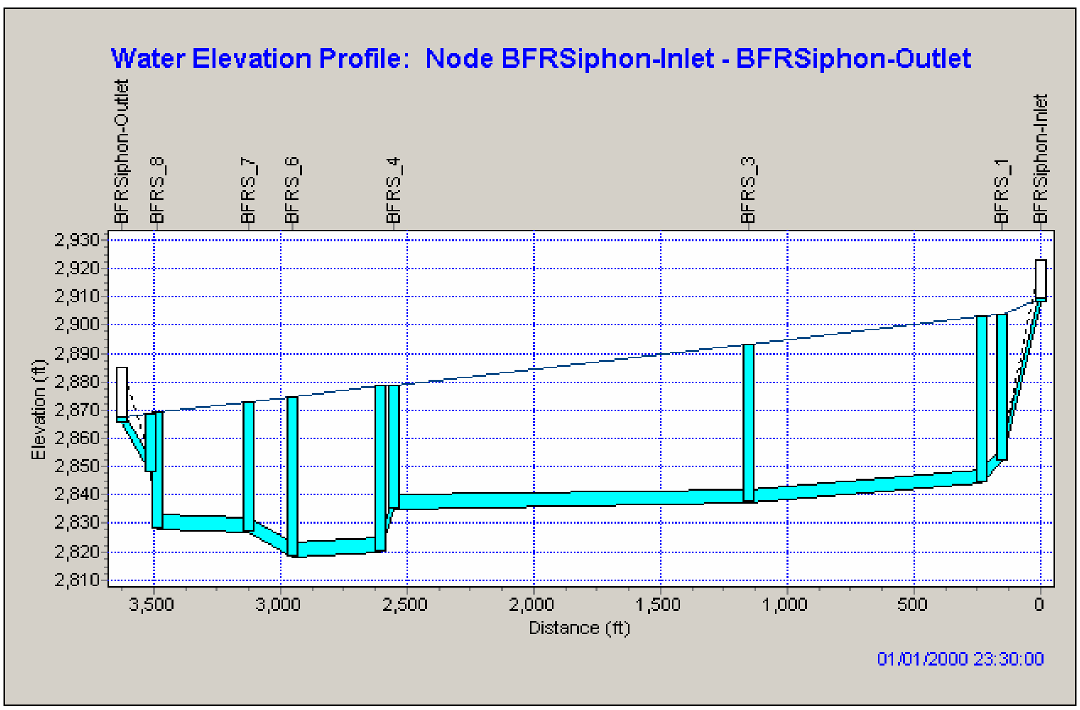

Figure 8. SWMM water elevation profile of the BFRS operating at 150 cfs.

Turnouts and laterals are set perpendicular to flow along the sides of the canal and use the head to convey water from the main canal, through the turnouts and laterals, to fields. There are approximately 130 turnout and lateral head gates on the South Canal. Turnouts/laterals were modeled as a side orifice at the junction nodes for the corresponding locations. The orifices are connected to a free falling outfall to exit the water from the main canal system. All outfall invert elevations are set to match the head gate turnout pipe invert elevations. The default discharge coefficient of 0.65 was used for each turnout/lateral. In reality most turnouts/laterals are controlled by downstream conditions along the turnout/lateral canal. Currently in the model the flow out of the turnouts/laterals is controlled artificially by using control rules where necessary. The physical modeling of the downstream control of turnouts/laterals off the main canal is recommended in the next project phase. Figure 9 shows a SWMM plan view of a set of turnout/lateral head gates as seen on the South Canal.

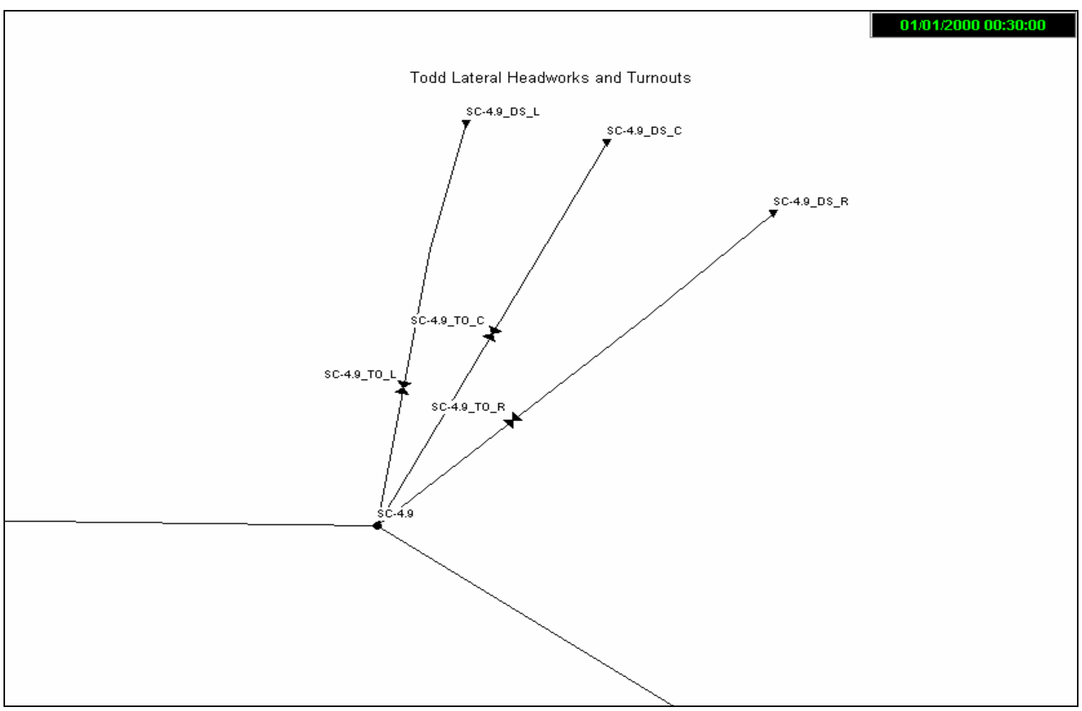

Figure 9. SWMM simulation of the Todd Lateral head gates.

Bridges and culverts are located along the South Canal to transport water beneath roads. There are 28 flow affecting bridges and/or culverts in the system, which vary in shape and size. Bridges along the South Canal were modeled as the appropriately dimensioned rectangular closed conduits with the appropriate number of barrels specified in SWMM. There are circular and arch culverts along the canal and were modeled as the appropriately sized and shaped conduits in SWMM. All RCP bridges and culverts were given entrance and exit loss coefficients of 0.5 and Manning's n values of 0.013, while corrugated metal pipe (CMP) culverts were given Manning's n values of 0.03 (Sturm, 2001; Mays, 2001).

Flumes are important structures that ditch riders use to estimate the flow going through a specific section of channel. The flumes in the BFID are Parshall flumes (Figure 10) and some may be subject to submergence problems as discussed by Olson (2006). There are 3 flumes used for flow measurement along the main canal. The flumes were modeled as rectangular open conduits in SWMM. They were given the appropriate throat width for the length of the entire flume, which lacks true flume detail. Simulations were conducted using more detailed flumes in the model, but they had no significant impacts and the simplified flumes produced similar results of flow and depth in the flume. Entrance and exit coefficients were assigned to the modeled flumes as 0.4 and 1.0, respectively (Sturm, 2001; Mays, 2001).

Figure 10. Belle Fourche River Siphon Flume outlet.

Modeling of Open Channel Canal

All open channel sections of the canal were modeled as appropriately dimensioned trapezoidal conduits. The conduits join all junction nodes in the model. BOR survey data was collected at each structure location (turnouts/laterals, checks, beginning and end of bridges/culverts, siphons, and flumes), thus all junctions entered into SWMM were given these dimensions. A particular section of open channel trapezoidal conduit was given the average depth and base width dimensions of its respective beginning and end junctions. All trapezoidal conduits were specified with 2:1 (H:V) side slopes, which is representative of the average for the South Canal. System flow stability was an issue near sections of conduit that were short (generally < 25 to 30 feet). Where stability was an issue, the problem was addressed by giving the linking conduit varying entrance and exit loss coefficients. Initial Manning's n values of 0.03 were specified for all open channel sections (Sturm, 2001; Mays, 2001).

Simulation Computational Method

During the calibration process the model simulations were run using the Dynamic Wave flow routing option in SWMM. Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results (Environmental Protection Agency, 2005). The Dynamic Wave routing option accounts for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow, essential to properly simulating the complex hydraulics of the irrigation system.

MODEL CALIBRATION

Data Collection

The data set used for model calibration was collected during the 2005 irrigation season (June 20th – September 20th) throughout the BFID South Canal. Because of its size and for purposes of organizing data collection, the South Canal was broken into four reaches. The four reaches were split at key locations and designated as follows: Reach 1) South Canal Dam Flume to BFRS Flume, Reach 2) BFRS Flume to Beals Check, Reach 3) Beals Check to Vale Flume, and Reach 4) Vale Flume to Wasteway. Figure 11 shows a map of the South Canal and locations of key structures (Olson, 2006). Each reach was monitored for two to three weeks successively beginning with Reach 1 at the start of the irrigation season. Due to time constraints caused by the late irrigation season start, data was only collected on Reaches 1 – 3 and calibration data for Reach 4 was unavailable.

During the time spent on each reach, field measurements were collected nearly every day at all laterals, turnouts, and check structures on the main canal. The measurements taken at laterals and turnouts were orifice gate stem openings and relative stage measurements of water surface to top of structure concrete. The measurements taken at each check structure included sluice gate stem openings, check-board weir openings, and upstream water surface to top of structure concrete and downstream stage measurements. Data loggers and pressure transducers were also placed throughout the South Canal. Some transducers and data loggers were left all season at key locations such as flumes, and others were moved along with reach changes. Stage measurements were also taken at all transducer locations at times of logger data collection for correction purposes. Several discharge measurements were taken at the Beals Check throughout the season. Discharge measurements were collected using a Marsh-McBirney Flo-Mate digital velocity measurement device and the 0.6-depth method.

Calibration Process

Two main steps were taken in calibrating the model of the South Canal. The first step was to conduct a water balance and correctly model the observed flows within the particular reach being calibrated. Continuous logger stage data were available at the South Canal Dam Flume, BFRS Flume, and the Vale Flume. The logger stage data at these locations was converted to flow by Olson (2006) using methods from the U.S. Bureau of Reclamation Water Measurement Manual (2001). One field measured flow was recorded per day at the Beals Check (end of Reach 2 and beginning of Reach 3) for most of the reach monitoring dates. Logger data, measured flows, and field measurements were used to calculate the water balance along the canal. Because changes to the system are made throughout the day and the field measurements were collected at one particular time of day for each structure, this task required simplifying assumptions. Assumptions were made on when major control changes occurred during the day. It was assumed that changes in Dam releases were made at 9 am each morning and that the collective structure settings were made at 9 am on the previous day. Changes were made to the SWMM control rules accordingly to calibrate to these estimated times. These estimated times were accurate within an hour or two in all cases.

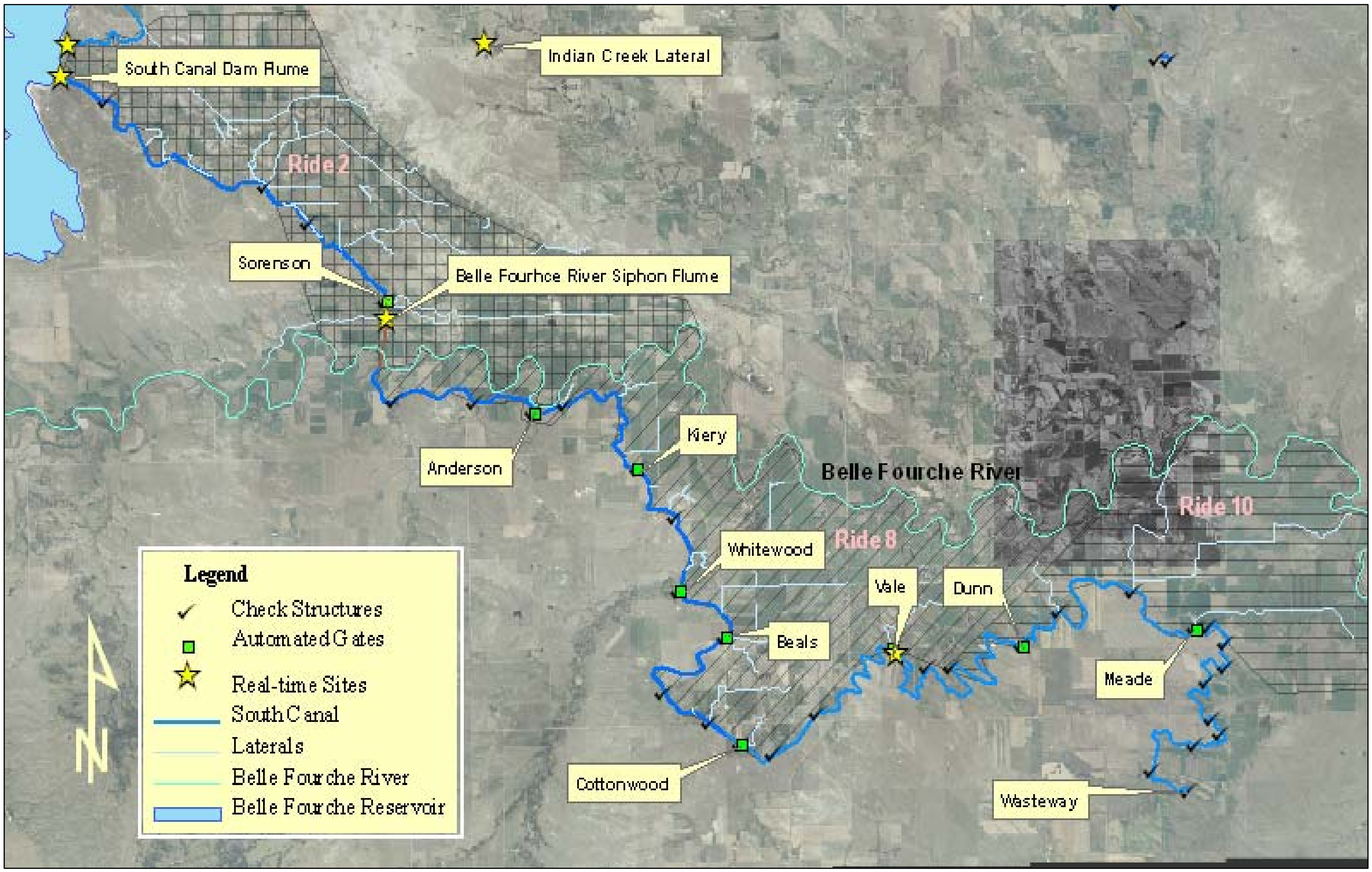

Figure 11. Map of BFID South Canal key locations (Olson, 2006).

In order to account for seepage and evaporation in the system, loss values were obtained from estimates found in 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. These values were regarded as significantly high and Reach 1 calibration indicated a lower loss rate that was used consistently throughout the calibration process. The loss rate used in the model for seepage and evaporation was 1 cfs per mile of canal. SWMM 5 does not yet have the capabilities of simulating infiltration or evaporation in conduits, thus other means of addressing these losses were employed. The assumed losses were pumped out of the system at each check structure's nearest upstream node, weighted according to its distance from the dam.

When the simulated water balance produced end of reach flows that minimized the difference from observed flows, the second step was undertaken for the same reach. The second step was to correctly model the observed stages, upstream and downstream, at each check structure within the reach. There is less uncertainty involved with this process as the downstream stage measurements were taken directly and the upstream stages were calculated using verified and accurate relative measurements. However, hand measurements are inevitably subject to error. Depths on the upstream side of the check were taken against the check structure piers (between gate/weir chambers) where water "piles up" to the magnitude of a couple tenths of feet. Also, the upstream depths were collected using a relative measurement of the water surface to the top of the structure where elevations are known. The downstream depths were measured directly, but the downstream side of check structures is quite turbulent and the measurements are only accurate to a few tenths of a foot.

Before the parameter calibration began a sensitivity analysis was conducted at the Meyer Check under typical observed structure settings in order to get a feel for how each parameter affects depth and flow (Figure 12). During the sensitivity analysis it was observed that changing Manning’s n-value had the largest effect on the depth in the system, on both sides of the check. It was also observed that the discharge coefficients for the gate orifices and weirs only affected the upstream depths of the checks; the downstream depths stayed exactly the same for all cases. For simplicity, Manning’s n-values were held consistent between check structures. Because of the parameter effects, Manning’s n-values were first changed to get the simulated downstream depths to match observed depths within $\pm$ 10% and centered about zero. The orifice and weir discharge coefficients were then adjusted in the model to match the observed upstream depths to the same standards as the downstream depths. Entrance and exit loss coefficients were held constant for similar structures over the entire South Canal and were not calibrated for individual reaches. The loss coefficients are presented in Table 1.

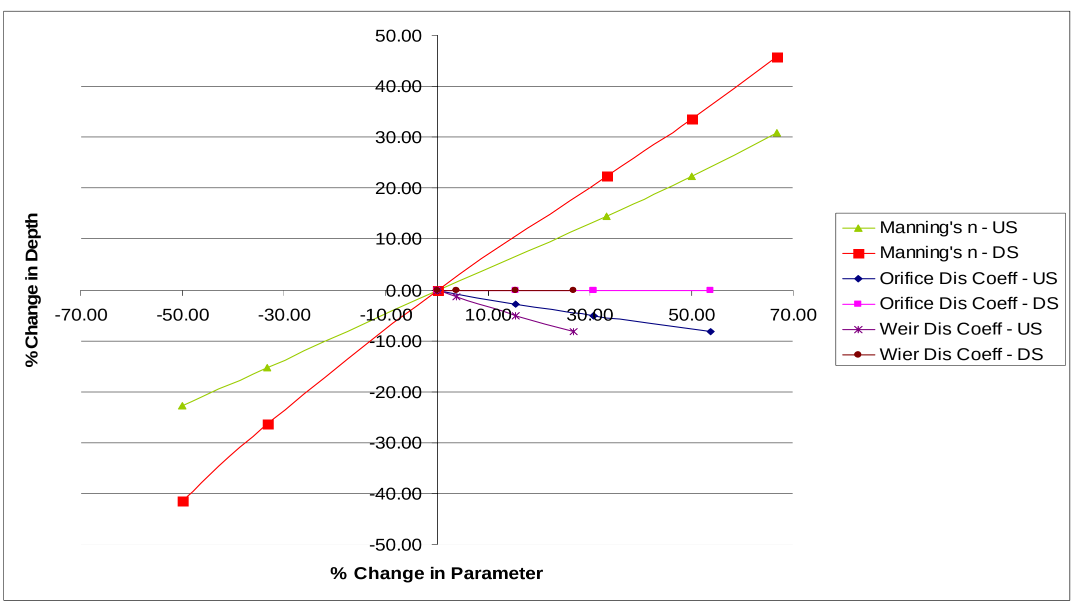

Figure 12. Pre-calibration parameter sensitivity analysis plot at Meyer Check.

Table 1. SWMM loss coefficients used in the model.

Structure	Loss Coefficients	
	Entrance	Exit
Box Culvert	0.5	0.5
Check	0.4	1
Flume	0.4	1
Pipe Bend	0.2	0.2
Siphon	0.4	1

Reach 1

Issues/Assumptions

The first major issue in matching the observed flows was simulating primed turnouts/laterals. Primed turnouts/laterals are usually opened 100% on the main canal and controlled by a downstream lateral control box. Field measurements were only collected on the main canal (the extent of this model), thus measurements on the turnout/lateral downstream control were unavailable. The primed Todd Lateral was addressed by writing SWMM control rules to artificially control the gate settings and vary them from observed settings. The lateral gate settings were adjusted to best match the reach logger data flows.

The second major issue in matching the water balance was accounting for Johnson Lateral return flows. The Johnson Lateral returns its unused excess water back into the main canal near the end of the reach just upstream of the BFRS Flume. Return flow data for the Johnson Lateral were not available during the 2005 irrigation season and the model calibration process. A direct inflow time series was entered into SWMM to add flow to the system at the Johnson Return junction node. A water balance was done to check the flows before the Johnson Lateral return flows were applied to correctly simulate the shape of the observed flow curve of the BFRS Flume logger data.

Results

Only one monitoring period was available for Reach 1 and served as the calibration period. The dates 7/5/2005 – 7/8/2005 were simulated and the model results compared to the observed data. Observed versus modeled flow through the BFRS Flume is presented in Figure 13. Observed versus modeled upstream and downstream depths at the check structures in Reach 1 are presented in Table 2. The model parameter values of Manning's n and gate and weir discharge coefficients that produced check depth results within ± 10% and balanced around zero are presented in Table 3 and Table 4, respectively.

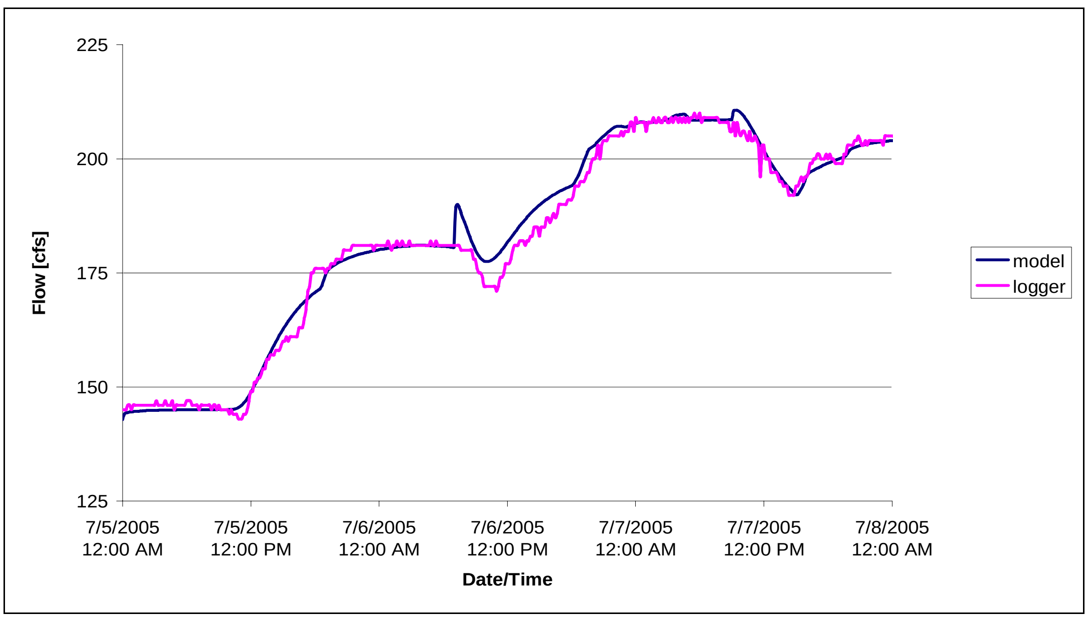

Figure 13. BFRS Flume flow, observed vs. modeled for 7/5/2005 – 7/8/2005.

Discussion

The modeled flow at the BFRS Flume varied from the observed by an average percent difference of 0.51% and a maximum of 5.7%. The modeled check depth values were acceptably close to those observed given that the observed values were hand measured in the field. Given that the accuracy of the hand measurements at check structures were approximately ± 0.2 to ± 0.3 ft, the model results matched well with a worst case difference of + 0.10 ft on the upstream side and – 0.15 ft on the downstream side. All simulated depths were within ± 5% of the observed depths.

Table 2. Reach 1 check depths, observed vs. modeled for 7/5/2005 – 7/8/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US Modeled Depth (ft)	US Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS Modeled Depth (ft)	DS Difference (ft)	Percent Error (%)
7/5/2005	Meyer	11:36 AM	2.54	2.52	-0.02	-0.79	1.75	1.82	0.07	4.00
7/7/2005	Meyer	8:55 AM	2.90	2.93	0.03	1.03	2.10	2.11	0.01	0.48
7/8/2005	Meyer	9:20 AM	3.10	3.09	-0.01	-0.32	2.20	2.19	-0.01	-0.45
7/5/2005	Todd	12:14 PM	4.60	4.66	0.06	1.30	4.20	4.20	0.00	0.00
7/7/2005	Todd	10:06 AM	5.28	5.26	-0.02	-0.38	4.90	4.87	-0.03	-0.61
7/8/2005	Todd	9:54 AM	5.10	5.18	0.08	1.57	4.70	4.77	0.07	1.49
7/5/2005	Eide	12:28 PM	4.30	4.40	0.10	2.33	3.20	3.30	0.10	3.12
7/7/2005	Eide	11:02 AM	4.90	4.83	-0.07	-1.43	3.90	3.95	0.05	1.28
7/8/2005	Eide	10:27 AM	4.65	4.75	0.10	2.15	4.00	3.85	-0.15	-3.75
7/5/2005	Sorenson	1:00 PM	4.30	4.21	-0.09	-2.09	3.20	3.09	-0.11	-3.44
7/7/2005	Sorenson	11:25 AM	4.75	4.70	-0.05	-1.05	3.60	3.69	0.09	2.50
7/8/2005	Sorenson	10:45 AM	4.50	4.57	0.07	1.56	3.60	3.58	-0.02	-0.56

Table 3. Reach 1 Manning's n values.

Canal Section	Canal Mile	Manning's n
Dam to Meyer	0 - 1	0.015
Meyer to Todd	1 - 5	0.015
Todd to Eide	5 - 6.2	0.031
Eide to Sorenson	6.2 - 7.7	0.019
Sorenson to BFRS I	7.7 - 8	0.039

Table 4. Reach 1 discharge coefficient values.

Check Structure	Discharge Coefficients: Orifice/Gate	Discharge Coefficients: Weir
Meyer	0.67	3.0
Todd	0.67	3.3
Eide	0.60	3.1
Sorenson	0.30	1.5

The Manning's n-values fit into ranges that would be expected within this reach of the South Canal (straight, uniform earth to winding and rocky); however, 0.015 for the section from the Dam outlet to the Todd Check seems a little low. Possible reasoning for the extreme low is that Manning's n may be accounting for changes in the channel cross section due to sedimentation and general morphology since the survey data was obtained in 1986. Also, channel cross section modeling generalizations may be a significant factor. The gate discharge coefficients were kept similar for the Meyer, Todd, and Eide because the gates are essentially the same, structurally and dimensionally. The Sorenson gate discharge coefficient seems a little low, but it is a much larger gate than those in the other checks. The weir discharge coefficients vary a little more. The weir boards used in the irrigation district vary and are not the same from check to check, or even within a single check. The values were all in the range commonly used for broad crested weirs, 2.6 to 3.3, except for those at the Sorenson. A discharge coefficient value of 3.3 is generally defined for a sharp crested weir. It seems that the boards are neither broad crested nor sharp crested, so the range of values is reasonable.

The Sorenson Check required unusually low discharge coefficients to match observed depth measurements as discussed previously. This may be because the Sorenson has three larger gate and weir chambers as opposed to the four or five smaller chambers more commonly seen in the district, or some possible errors in field measurements. At any rate, the low Sorenson discharge coefficients are the cause of the steep peak in flow through the BFRS Flume modeled on 7/6/2005 and the smaller peak on 7/7/2005. The peak correlated with a change made in the gate at the Sorenson; the source was confirmed in a plot of flow through the check. The weir discharge coefficient was less susceptible to producing peaks in flow, but was less effective at matching upstream depths at lower flows. Since the calibration focus was on check structures, depth accuracy and consistency had priority over any discrepancy in flow through the reach's flume.

Reach 2

Issues/Assumptions

Flow data through the BFRS Flume at the beginning of the reach were continuous and sufficient and were modeled as observed. However, flow data at the end of the reach, at the Beals Check, were not continuous and deemed to be insufficient. Even though flows were collected with the Flo-Mate on nearly all the days of Reach 2 monitoring, the one flow per day does not provide a good picture of what is happening in detail enough needed for the complete and accurate reach water balance. Even a few measurements during the day would not make much of a difference; continuous flow data would be the best and is needed for a complete picture of what is happening on the reach.

Downstream control was more prominent on this reach. When the turnout/lateral settings were modeled as observed, even with the known primed laterals (e.g., Butte/Hall) taken into consideration, much more flow was being taken out of the system than what observations indicated. To address this issue water orders were used as a system outflow guide. Olson's (2006) water demand/order macro was used to compile the orders during the period. The water orders were then compared to the turnout/lateral settings observed. There were many gates that were observed as open, but had no water orders for the period; these gates were modeled as closed (fully primed). There were also many gates that were observed with settings that produced much more outflow from the model system than what was ordered. The differences between excess model outflow and water orders at these gates were minimized in the model (partially primed). The water orders were matched in the model by adjusting the turnout gates until they produced similar flows to the water cards (+ 0.5 to 1 cfs depending on order magnitude) including operation and transportation losses.

Lastly, Reach 2 is the longest and has the most structures of the first three reaches. The BOR survey data was fairly incomplete for the reach. Most check structure (five of eight) invert elevations could not be found, specifically: Stinkwater (and siphon), Minor, Anderson (and siphon), 12.5, and Whitewood. These were linearly interpolated initially, but adjusted accordingly to produce optimal calibration results (Table 5). The adjusted invert elevations were limited and still fit into an acceptable and probable range.

Table 5. Pre- and post-calibration Reach 2 check invert elevations.

Check Structure	Interpolated Elevation (ft)	Calibrated Elevation (ft)
Stinkwater	2864.6	2865
Minor	2864	2863.5 I, 2863 O
Anderson	2861.2	2862.5 I, 2858.6 O
12.5	2860	2859
Whitewood	2851	2851

Results

Three monitoring periods were available for the calibration and validation of Reach 2. Validation in this context represents running the model for a different period where the inflows and outflows within the reach are changed but the hydraulic characteristics of the model are not adjusted. The additional validation monitoring periods were conducted using the same assumptions as the calibration periods. Reach 2 was calibrated using the monitoring period dates 7/11/2005 – 7/14/2005. For the calibration period flows were collected at the Beals Check on 7/12 and 7/13. The Beals flows were used as the end-of-reach check on water balance. Observed versus modeled

Beals flows are presented in Table 6. Observed versus modeled upstream and downstream depths at the check structures in Reach 2 are presented in Table 7. The resulting Manning's n and gate and weir discharge coefficient parameters for Reach 2 are presented in Table 8 and Table 9, respectively.

Table 6. Beals Check flow, observed vs. modeled for 7/11/2005 – 7/14/2005.

Date/Time	Flow (FloMate) [cfs]	Water Card Expected Flow [cfs]	Model Flow [cfs]	Percent Error (%)
7/12/05 11:05 AM	149	151.5	150	-1.0%
7/13/05 2:30 PM	129	149	151	1.3%

Table 7. Reach 2 check depths, observed vs. modeled for 7/11/2005 – 7/14/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
7/11/2005	Stinkwater	9:51 AM	7.01	7.04	0.03	0.43	6.35	6.38	0.03	0.47
7/12/2005	Stinkwater	8:51 AM	6.80	6.86	0.06	0.88	6.20	6.22	0.02	0.32
7/13/2005	Stinkwater	12:53 PM	6.65	6.57	-0.08	-1.20	n/a	5.97	n/a	n/a
7/14/2005	Stinkwater	9:14 AM	6.90	7.10	0.20	2.90	6.30	6.39	0.09	1.43
7/11/2005	Minor	10:30 AM	5.80	5.72	-0.08	-1.38	6.20	6.15	-0.05	-0.81
7/12/2005	Minor	9:10 AM	5.40	5.54	0.14	2.59	5.80	5.97	0.17	2.93
7/13/2005	Minor	1:12 PM	5.40	5.27	-0.13	-2.41	5.60	5.70	0.10	1.79
7/14/2005	Minor	9:31 AM	5.50	5.53	0.03	0.55	5.60	5.96	0.36	6.43
7/11/2005	Anderson	10:48 AM	6.30	6.20	-0.10	-1.59	10.00	9.93	-0.07	-0.70
7/12/2005	Anderson	9:18 AM	5.90	5.99	0.09	1.53	9.60	9.72	0.12	1.25
7/13/2005	Anderson	1:20 PM	5.75	5.65	-0.10	-1.74	9.40	9.37	-0.03	-0.32
7/14/2005	Anderson	9:37 AM	6.10	5.98	-0.12	-1.97	9.80	9.70	-0.10	-1.02
7/11/2005	12.5	11:20 AM	6.85	6.98	0.13	1.90	6.58	6.78	0.20	3.04
7/12/2005	12.5	9:24 AM	6.70	6.86	0.16	2.39	6.60	6.67	0.07	1.06
7/13/2005	12.5	1:25 PM	6.60	6.68	0.08	1.21	6.54	6.51	-0.03	-0.46
7/14/2005	12.5	9:42 AM	6.80	6.85	0.05	0.74	6.63	6.67	0.04	0.60
7/11/2005	Kiery	12:10 PM	5.10	5.02	-0.08	-1.57	4.10	4.07	-0.03	-0.73
7/12/2005	Kiery	9:44 AM	5.00	4.95	-0.05	-1.00	4.00	4.02	0.02	0.50
7/13/2005	Kiery	1:44 PM	4.90	4.97	0.07	1.43	4.10	4.04	-0.06	-1.46
7/14/2005	Kiery	9:57 AM	5.20	5.06	-0.14	-2.69	4.20	4.11	-0.09	-2.14
7/11/2005	Simmons	1:10 PM	4.40	4.29	-0.11	-2.50	3.20	3.22	0.02	0.63
7/12/2005	Simmons	10:01 AM	4.20	4.24	0.04	0.95	3.10	3.13	0.03	0.97
7/13/2005	Simmons	2:00 PM	4.15	4.29	0.14	3.37	3.10	3.22	0.12	3.87
7/14/2005	Simmons	10:09 AM	4.40	4.38	-0.02	-0.45	3.20	3.30	0.10	3.12
7/11/2005	Whitewood	1:55 PM	6.40	6.25	-0.15	-2.34	6.00	5.90	-0.10	-1.67
7/12/2005	Whitewood	10:13 AM	6.00	6.09	0.09	1.50	5.80	5.75	-0.05	-0.86
7/13/2005	Whitewood	2:15 PM	n/a	6.11	n/a	n/a	n/a	5.75	n/a	n/a
7/14/2005	Whitewood	10:25 AM	6.30	6.36	0.06	0.95	6.00	6.01	0.01	0.17
7/11/2005	Beals	3:00 PM	n/a	3.90	n/a	n/a	n/a	3.24	n/a	n/a
7/12/2005	Beals	10:50 AM	3.90	3.80	-0.10	-2.56	3.20	3.16	-0.04	-1.25
7/13/2005	Beals	2:30 PM	3.60	3.80	0.20	5.56	3.10	3.16	0.06	1.94
7/14/2005	Beals	10:55 AM	3.80	3.68	-0.12	-3.16	3.10	3.09	-0.01	-0.32

Table 8. Reach 2 Manning's n values.

Canal Section	Canal Mile	Manning's n
BFRS O to Stinkwater	8 - 9.4	0.023
Stinkwater to Minor	9.4 - 11	0.022
Minor to 12.5	11 - 12.5	0.027
12.5 to Kiery	12.5 - 15.1	0.032
Kiery to Simmons	15.1 - 16.2	0.022
Simmons to Whitewood	16.2 - 17.4	0.024
Whitewood to Beals	17.4 - 18.6	0.028

Table 9. Reach 2 discharge coefficient values.

Check	Discharge Coefficients	
Structure	Orifice/Gate	Weir
Stinkwater	0.40	2.6
Minor	0.70	3.0
Anderson	1.00	3.3
12.5	0.65	2.8
Kiery	0.60	2.6
Simmons	0.55	2.6
Whitewood	1.00	3.3
Beals	0.70	2.8

The Reach 2 calibration period was validated using data for monitoring period dates 7/18/2005 – 7/22/2005. For the validation period flows were collected at the Beals Check on 7/19 and 7/22. Observed versus modeled Beals flows are presented in Table 10 and observed versus modeled upstream and downstream depths at the check structures in are presented in Table 11.

Table 10. Beals Check flow, observed vs. modeled for 7/18/2005 – 7/22/2005.

Date/Time	Flow (FloMate) [cfs]	Model Flow [cfs]	Percent Error (%)
7/19/05 1:55 PM	152.1	150.1	-1.3
7/22/05 11:35 AM	143.4	147.1	2.6

Table 11. Reach 2 check depths, observed vs. modeled for 7/18/2005 – 7/22/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
7/18/2005	Stinkwater	10:59 AM	6.65	6.88	0.23	3.46	6.20	6.21	0.01	0.16
7/19/2005	Stinkwater	12:42 PM	6.70	6.62	-0.08	-1.19	6.00	6.01	0.01	0.17
7/20/2005	Stinkwater	10:55 AM	6.60	6.86	0.26	3.94	6.00	6.21	0.21	3.50
7/22/2005	Stinkwater	9:28 AM	6.45	6.92	0.47	7.29	6.00	6.21	0.21	3.50
7/18/2005	Minor	11:21 AM	5.65	5.29	-0.36	-6.37	5.70	5.72	0.02	0.35
7/19/2005	Minor	12:56 PM	5.30	5.20	-0.10	-1.89	5.70	5.63	-0.07	-1.23
7/20/2005	Minor	11:32 AM	5.38	5.32	-0.06	-1.12	5.80	5.74	-0.06	-1.03
7/22/2005	Minor	9:46 AM	5.10	5.04	-0.06	-1.18	5.60	5.46	-0.14	-2.50
7/18/2005	Anderson	11:28 AM	5.85	5.67	-0.18	-3.08	9.60	9.38	-0.22	-2.29
7/19/2005	Anderson	1:01 PM	5.80	5.55	-0.25	-4.31	9.50	9.28	-0.22	-2.32
7/20/2005	Anderson	11:40 AM	5.98	5.69	-0.29	-4.85	9.70	9.41	-0.29	-2.99
7/22/2005	Anderson	9:54 AM	5.60	5.33	-0.27	-4.82	9.00	9.06	0.06	0.67
7/18/2005	12.5	11:35 AM	6.70	6.63	-0.07	-1.04	6.50	6.45	-0.05	-0.77
7/19/2005	12.5	1:06 PM	6.60	6.57	-0.03	-0.45	6.33	6.41	0.08	1.21
7/20/2005	12.5	12:00 PM	6.70	6.63	-0.07	-1.04	6.58	6.46	-0.12	-1.87
7/22/2005	12.5	10:00 AM	6.55	6.45	-0.10	-1.53	6.38	6.29	-0.09	-1.33
7/18/2005	Kiery	12:05 PM	5.00	4.73	-0.27	-5.40	4.00	3.99	-0.01	-0.25
7/19/2005	Kiery	1:18 PM	4.80	4.73	-0.07	-1.46	4.20	3.92	-0.28	-6.67
7/20/2005	Kiery	12:26 PM	4.85	4.74	-0.11	-2.27	4.20	3.92	-0.28	-6.67
7/22/2005	Kiery	10:22 AM	4.60	4.59	-0.01	-0.22	4.20	3.79	-0.41	-9.76
7/18/2005	Simmons	12:41 PM	4.30	4.09	-0.21	-4.88	3.30	3.19	-0.11	-3.33
7/19/2005	Simmons	1:29 PM	4.00	3.91	-0.09	-2.25	3.30	3.12	-0.18	-5.45
7/20/2005	Simmons	12:51 PM	4.00	3.90	-0.10	-2.50	3.45	3.10	-0.35	-10.14
7/22/2005	Simmons	10:49 AM	3.70	3.62	-0.08	-2.16	3.20	2.89	-0.31	-9.69
7/18/2005	Whitewood	12:54 PM	6.08	6.15	0.07	1.15	5.80	5.82	0.02	0.34
7/19/2005	Whitewood	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7/20/2005	Whitewood	1:07 PM	6.15	5.97	-0.18	-2.93	6.00	5.66	-0.34	-5.67
7/22/2005	Whitewood	11:01 AM	5.70	5.61	-0.09	-1.58	5.70	5.37	-0.33	-5.79
7/18/2005	Beals	1:35 PM	3.85	3.74	-0.11	-2.86	3.10	3.13	0.03	0.97
7/19/2005	Beals	1:46 PM	3.85	3.62	-0.23	-5.97	3.10	3.16	0.06	1.94
7/20/2005	Beals	1:41 PM	3.85	3.60	-0.25	-6.49	3.25	3.14	-0.11	-3.38
7/22/2005	Beals	11:21 AM	3.75	3.58	-0.17	-4.53	3.10	3.12	0.02	0.65

The Reach 2 calibration period was also validated using data for monitoring period dates 7/25/2005 – 7/26/2005. For the validation period flows were collected at the Beals Check on 7/25 and 7/26. For the validation period 7/25/2005 – 7/26/2005, observed versus modeled Beals flows are presented in Table 12 and observed versus modeled upstream and downstream depths at the check structures in are presented in Table 13.

Table 12. Beals Check flow, observed vs. modeled for 7/25/2005 – 7/26/2005.

Date/Time	Flow (FloMate) [cfs]	Model Flow [cfs]	Percent Error (%)
7/25/05 11:35 AM	169.2	170.8	0.9
7/26/05 10:30 AM	154.9	178.6	15.3

Table 13. Reach 2 check depths, observed vs. modeled for 7/25/2005 – 7/26/2005

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
7/25/2005	Stinkwater	1:58 PM	6.70	7.32	0.62	9.25	6.10	6.59	0.49	8.03
7/26/2005	Stinkwater	8:56 AM	6.60	7.36	0.76	11.52	6.20	6.65	0.45	7.26
7/25/2005	Minor	2:14 PM	5.40	5.75	0.35	6.48	5.80	6.18	0.38	6.55
7/26/2005	Minor	9:14 AM	5.40	5.92	0.52	9.63	5.80	6.35	0.55	9.48
7/25/2005	Anderson	2:19 PM	6.00	6.22	0.22	3.67	9.50	9.93	0.43	4.53
7/26/2005	Anderson	9:20 AM	6.00	6.42	0.42	7.00	9.50	10.14	0.64	6.74
7/25/2005	12.5	2:25 PM	6.70	6.90	0.20	2.99	6.50	6.70	0.20	3.08
7/26/2005	12.5	9:27 AM	6.70	7.03	0.33	4.93	6.60	6.82	0.22	3.33
7/25/2005	Kiery	2:52 PM	4.75	4.73	-0.02	-0.42	4.20	3.89	-0.31	-7.38
7/26/2005	Kiery	9:43 AM	4.85	4.84	-0.01	-0.21	4.20	3.98	-0.22	-5.24
7/25/2005	Simmons	3:07 PM	4.10	3.82	-0.28	-6.83	3.40	3.07	-0.33	-9.71
7/26/2005	Simmons	9:58 AM	4.10	3.98	-0.12	-2.93	3.40	3.21	-0.19	-5.59
7/25/2005	Whitewood	3:18 PM	6.10	5.99	-0.11	-1.80	6.00	5.75	-0.25	-4.17
7/26/2005	Whitewood	10:09 AM	6.10	6.24	0.14	2.30	6.00	6.00	0.00	0.00
7/25/2005	Beals	3:37 PM	4.05	3.97	-0.08	-1.98	3.70	3.41	-0.29	-7.84
7/26/2005	Beals	10:32 AM	3.90	4.05	0.15	3.85	3.40	3.49	0.09	2.65

Discussion

The simulated Beals flows matched the observed measured flows within $\pm$ 3% with the exception of flows on 7/13 and 7/26. Early in the calibration process during the first period it was observed that there was a clear trend of the observed depths on 7/13 and 7/14 being considerably lower than 7/11 and 7/12. It seemed that the observed flow of 129 cfs at the Beals on 7/13 is low and may be in error. It did not fit the trend of Beals flows, which are very near 150 cfs for all the monitored dates. Also, the water orders on 7/13 (or the 7/12 if considering a lag in delivery) indicated that the expected flow at the Beals would be around 150 cfs. Thus, it was assumed that the Beals Check flow on 7/13 was approximately 150 cfs rather than the measured 129 cfs. The Beals flow on 7/26 was modeled larger than observed (+ 15.3 %) and produced check depth results that matched up with observed depths quite well. The observed measured flow was significantly lower than the expected flow from a water balance of incoming flows at the BFRS Flume, water orders in the reach, and assumed losses throughout the reach. The modeled flow better reflects the expected flow at the Beals for 7/26.

The check depths matched quite well considering observed depths were measured by hand as stated in the discussion of Reach 1 results. For the calibration period the simulated depths were all within $\pm$ 7% of the observed depths. The validation period produced simulated depths all within $\pm$ 12% of the observed, with the majority being $\pm$ 8% of the observed depths.

The resulting Manning's n values for Reach 2 vary much less than on Reach 1 and seem to be more consistent and fit into an expected range. However, the check gate and weir discharge coefficients vary more than on Reach 1. This may be due to more physical variance between the actual check structures themselves, both in overall dimension and number of gate and weir chambers. The Stinkwater Check produced discharge coefficients on the lower end of the range similar to the Sorenson Check. The Stinkwater Check is similar to the Sorenson Check in that it has fewer and larger gate and weir chambers. Also, the Anderson and Whitewood Checks both produced values on the upper end of the discharge coefficient range, which may be explained by their both being just upstream of siphons, which may cause some unusual hydraulics and backwater effects.

Reach 3

Issues/Assumptions

As described for Reach 2, flows at the Beals Check (beginning of Reach 3) were not continuous and for this reach were not used as the inflow point during the calibration process. Simulated flows were entered at the BFRS Flume and exited at the Wasteway. Because flows were entered at the BFRS Flume and not at the Beals Check two extra days were simulated at the beginning of the period to allow the system to fill prior to evaluation. Flows were taken out at three arbitrary turnouts (12.5, Butte/Hall, and Shaw/Baldwin) in Reach 2 to match the measured flows at the Beals Check. To create realistic backwater effects downstream of Reach 3 all Reach 4 gates and weirs were modeled as half open and flows were uniformly taken out of arbitrary turnout/laterals to achieve realistic Reach 4 water deliveries. Turnout/lateral priming was also an issue on this reach and was addressed as described for Reach 2.

There was a significant flow problem during the calibration and validation periods for Reach 3. The water balance between the BFRS Flume and the Beals Check presented no problems and matched up well. However, between the Beals Check and the Vale Flume there was a water balance issue. When Reach 3 was simulated by taking the approximate water orders out of the system, the modeled Vale Flume flow was significantly lower than the logger data flow as presented in Figure 14, Figure 15, and Figure 16. The simulated flow curve shape generally matched well, but was approximately 10 – 30 cfs lower than the observed logger data flows. The Beals Check flows measured during the monitoring period as well as calculated flows using measurements and equations are also shown in Figure 15 and Figure 16. The logger data for the Vale Flume was higher than what was measured or calculated at the Beals Check for some measurements, which did not make sense. It seemed that the logger data at the Vale Flume might be subject to errors when converting from depth to flow. There are a few possible sources of error in collective flows leading up to the Vale Flume. First, the measured flows at the Beals are subject to hand and device measurement errors. Second, the Vale Flume was submerged over 80% nearly the entire season (Olson, 2006), which leads to flow correction errors of ± 5 – 10% and ± 10 – 25% for over 90% submergence (U.S. Bureau of Reclamation, 2001).

Simulations were also run assuming the possibility that there were no Reach 3 outflows during the simulation periods. This made the model match the logger more closely but still simulated approximately 10 cfs lower than observed (worst case), which is about the amount of assumed flow lost through seepage and evaporation. If the assumptions of no Reach 3 outflows and no Reach 3 evaporation and seepage losses were true, then the modeled Vale would match very well. Both of these assumptions are most unlikely. The calibration was continued using the lower simulated flows assuming water order outflows, evaporation and seepage losses, and errors in the Vale Flume logger data flow calculations/corrections.

Results

Three monitoring periods were available for calibration and validation of Reach 3. Reach 3 was calibrated using the monitoring period dates 8/1/2005 – 8/4/2005. For the calibration period, flows were collected at the Beals Check on 8/3 and 8/4. Observed versus modeled Beals flows are presented in Table 14 and observed versus modeled flows at the Vale Flume for the period are presented in Figure 14. Observed versus modeled upstream and downstream depths at the check structures are presented in Table 15. The resulting Manning's n and gate and weir discharge coefficient parameters for Reach 3 are presented in Table 16 and Table 17, respectively.

Table 14. Beals Check flow, observed vs. modeled for 8/1/2005 – 8/4/2005.

Date/Time	Flow (FloMate) [cfs]	Model Flow [cfs]	Percent Error [%]
8/3/05 9:05 AM	142.2	142.3	0.06
8/4/05 10:40 AM	156.3	157.3	0.66

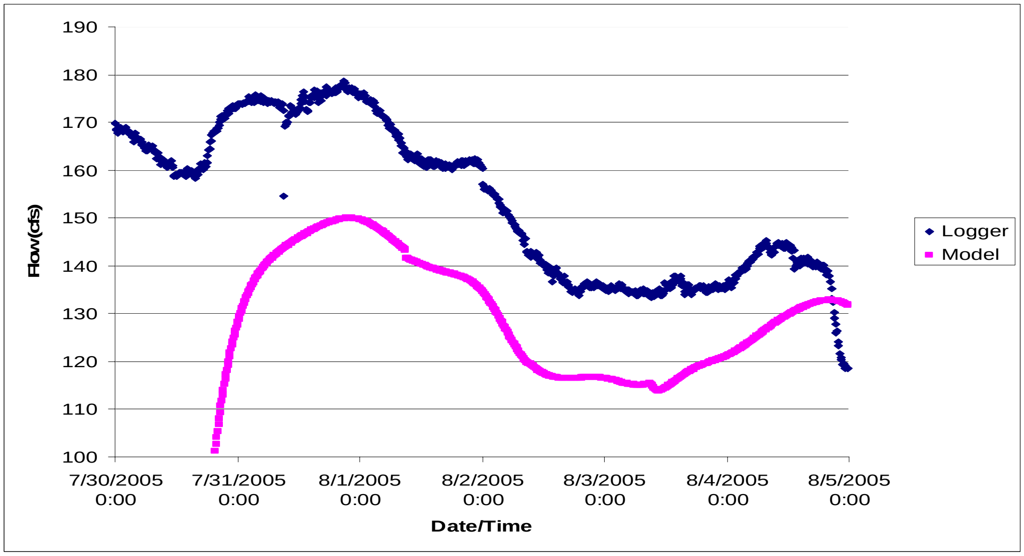

Figure 14. Vale Flume flow, observed vs. modeled for 8/1/2005 – 8/4/2005.

Table 15. Reach 3 check depths, observed vs. modeled for 8/1/2005 – 8/4/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
8/1/2005	Lee	1:52 PM	4.00	4.11	0.11	2.75	2.75	2.98	0.23	8.36
8/2/2005	Lee	12:07 PM	3.55	3.45	-0.10	-2.82	2.50	2.43	-0.07	-2.80
8/3/2005	Lee	9:34 AM	3.55	3.51	-0.04	-1.13	2.70	2.48	-0.22	-8.15
8/4/2005	Lee	11:04 AM	3.75	3.93	0.18	4.80	2.60	2.84	0.24	9.23
8/1/2005	Lull	2:15 PM	5.05	5.30	0.25	4.95	4.80	5.11	0.31	6.46
8/2/2005	Lull	12:22 PM	4.65	4.55	-0.10	-2.15	4.50	4.42	-0.08	-1.78
8/3/2005	Lull	9:55 AM	4.80	4.63	-0.17	-3.54	4.45	4.45	0.00	0.00
8/4/2005	Lull	11:15 AM	4.90	5.13	0.23	4.69	4.60	4.94	0.34	7.39
8/1/2005	Cottonwood	2:34 PM	3.50	3.53	0.03	0.86	2.40	2.54	0.14	5.83
8/2/2005	Cottonwood	12:33 PM	3.10	2.80	-0.30	-9.68	2.10	2.07	-0.03	-1.43
8/3/2005	Cottonwood	10:06 AM	3.15	2.85	-0.30	-9.52	2.10	2.11	0.01	0.48
8/4/2005	Cottonwood	11:35 AM	3.20	3.36	0.16	5.00	2.30	2.56	0.26	11.30
8/1/2005	Foos	2:43 PM	3.80	3.90	0.10	2.63	2.85	2.83	-0.02	-0.70
8/2/2005	Foos	12:40 PM	3.50	3.31	-0.19	-5.43	2.50	2.40	-0.10	-4.00
8/3/2005	Foos	10:13 AM	3.50	3.45	-0.05	-1.43	2.40	2.46	0.06	2.50
8/4/2005	Foos	11:45 AM	3.60	3.78	0.18	5.00	2.80	2.78	-0.02	-0.71
8/1/2005	Ollilla	2:58 PM	4.40	4.33	-0.07	-1.59	3.00	3.09	0.09	3.00
8/2/2005	Ollilla	12:52 PM	3.95	3.79	-0.16	-4.05	2.70	2.72	0.02	0.74
8/3/2005	Ollilla	10:21 AM	3.95	3.93	-0.02	-0.51	2.80	2.70	-0.10	-3.57
8/4/2005	Ollilla	11:55 AM	4.10	4.32	0.22	5.37	2.80	2.96	0.16	5.71
8/1/2005	Vale	10:45 AM	4.55	4.65	0.10	2.20	2.70	2.88	0.18	6.67
8/2/2005	Vale	1:26 PM	4.30	4.07	-0.23	-5.35	2.60	2.48	-0.12	-4.62
8/3/2005	Vale	10:44 AM	4.30	3.99	-0.31	-7.21	2.50	2.42	-0.08	-3.20
8/4/2005	Vale	12:15 PM	4.30	4.39	0.09	2.09	2.70	2.69	-0.01	-0.37

Table 16. Reach 3 Manning's n values.

Canal Section	Canal Mile	Manning's n
Beals to Lee	18.6 - 20.6	0.010
Lee to Lull	20.6 - 21.5	0.009
Lull to Cottonwood	21.5 - 22.1	0.010
Cottonwood to Foos	22.1 - 22.7	0.022
Foos to Ollilla	22.7 - 23.7	0.017
Ollilla to Vale	23.7 - 26.4	0.017
Vale to 69C-0001	26.4 - 28.1	0.030
69C-0001 to Wasteway	28.1 - 43.8	0.025

Table 17. Reach 3 discharge coefficient values.

Check	Discharge Coefficients	
Structure	Orifice/Gate	Weir
Lee	0.50	2.6
Lull	0.50	2.6
Cottonwood	0.60	2.0
Foos	0.45	2.6
Ollilla	0.65	2.6
Vale	0.50	2.6

The Reach 3 calibration was validated using data for monitoring period dates 8/8/2005 – 8/11/2005. For the validation period flows were collected at the Beals Check on 8/8, 8/9, and 8/11. Observed versus modeled Beals flows are presented in Table 18 and observed versus modeled flows at the Vale Flume for the period are presented in Figure 15. Also presented in Figure 15 are the measured flow at the Beals Check and the calculated flow at the Beals Check using measurements and equations for each check chamber (gate and weir) (Olson, 2006). Observed versus modeled upstream and downstream depths at the check structures are presented in Table 19.

Table 18. Beals Check flow, observed vs. modeled for 8/8/2005 – 8/11/2005.

Date/Time	Flow (FloMate) [cfs]	Model Flow [cfs]	Percent Error [%]
8/8/05 8:40 AM	119.6	122.4	2.34
8/9/05 11:40 AM	119.5	117.4	-1.77
8/11/05 9:50 AM	107.5	106.4	-1.02

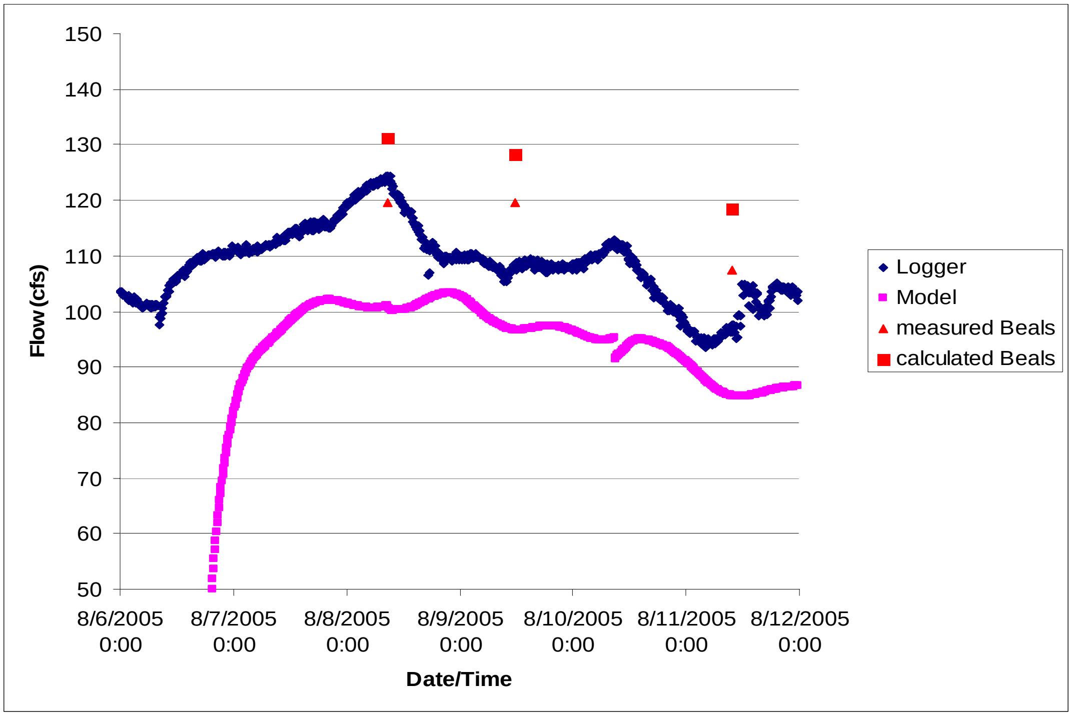

Figure 15. Vale Flume flow, observed vs. modeled for 8/8/2005 – 8/11/2005.

Table 19. Reach 3 check depths, observed vs. modeled for 8/8/2005 – 8/11/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
8/8/2005	Lee	10:31 AM	3.70	3.51	-0.19	-5.14	2.70	2.78	0.08	2.96
8/9/2005	Lee	12:15 PM	3.50	3.42	-0.08	-2.29	2.50	2.71	0.21	8.40
8/11/2005	Lee	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8/8/2005	Lull	10:41 AM	5.10	5.29	0.19	3.73	4.80	5.19	0.39	8.13
8/9/2005	Lull	12:25 PM	4.90	5.23	0.33	6.73	4.65	5.14	0.49	10.54
8/11/2005	Lull	10:11 AM	4.85	4.98	0.13	2.68	4.60	4.90	0.30	6.52
8/8/2005	Cottonwood	10:50 AM	3.75	3.77	0.02	0.53	3.50	3.39	-0.11	-3.14
8/9/2005	Cottonwood	12:36 PM	3.50	3.71	0.21	6.00	3.20	3.34	0.14	4.38
8/11/2005	Cottonwood	10:18 AM	3.45	3.48	0.03	0.87	3.20	3.17	-0.03	-0.94
8/8/2005	Foos	10:55 AM	4.90	4.95	0.05	1.02	2.30	2.32	0.02	0.87
8/9/2005	Foos	12:43 PM	4.80	4.92	0.12	2.50	2.10	2.26	0.16	7.62
8/11/2005	Foos	10:24 AM	4.80	4.79	-0.01	-0.21	2.30	2.05	-0.25	-10.87
8/8/2005	Ollilla	11:02 AM	4.10	3.87	-0.23	-5.61	2.60	2.47	-0.13	-5.00
8/9/2005	Ollilla	12:53 PM	3.75	3.80	0.05	1.33	2.35	2.41	0.06	2.55
8/11/2005	Ollilla	10:33 AM	3.70	3.55	-0.15	-4.05	2.30	2.24	-0.06	-2.61
8/8/2005	Vale	11:25 AM	4.20	3.88	-0.32	-7.62	2.50	2.24	-0.26	-10.40
8/9/2005	Vale	1:16 PM	3.85	3.77	-0.08	-2.08	2.40	2.17	-0.23	-9.58
8/11/2005	Vale	10:54 AM	3.75	3.41	-0.34	-9.07	2.20	1.93	-0.27	-12.27

The Reach 3 calibration period was also validated using data for monitoring period dates 8/15/2005 – 8/17/2005. The validation period flows were collected at the Beals Check on 8/15, 8/16, and 8/17. Observed versus modeled Beals flows are presented in Table 20. The difference between Flo-Mate and equation calculations for Beals flows during this period was significant so an average value of the two was used because it was unclear as to which was more accurate. Observed versus modeled flows at the Vale Flume for the period are presented in Figure 16. Also presented in Figure 16 are the measured flow at the Beals Check and the calculated flow at the Beals Check using measurements and equations for each check chamber (gate and weir) (Olson, 2006). Observed versus modeled upstream and downstream depths at the check structures are presented in Table 21.

Table 20. Beals Check flow, observed vs. modeled for 8/15/2005 – 8/17/2005.

Date/Time	Calculated Flow [cfs]	Flow (FloMate) [cfs]	Model Flow [cfs]	% Error Calculated	% Error FloMate
8/15/05 11:40 AM	101.0	87.8	94.2	-6.75	7.29
8/16/05 10:15 AM	92.8	76.9	84.4	-9.08	9.75
8/17/05 10:55 AM	83.0	64.9	72.4	-12.82	11.56

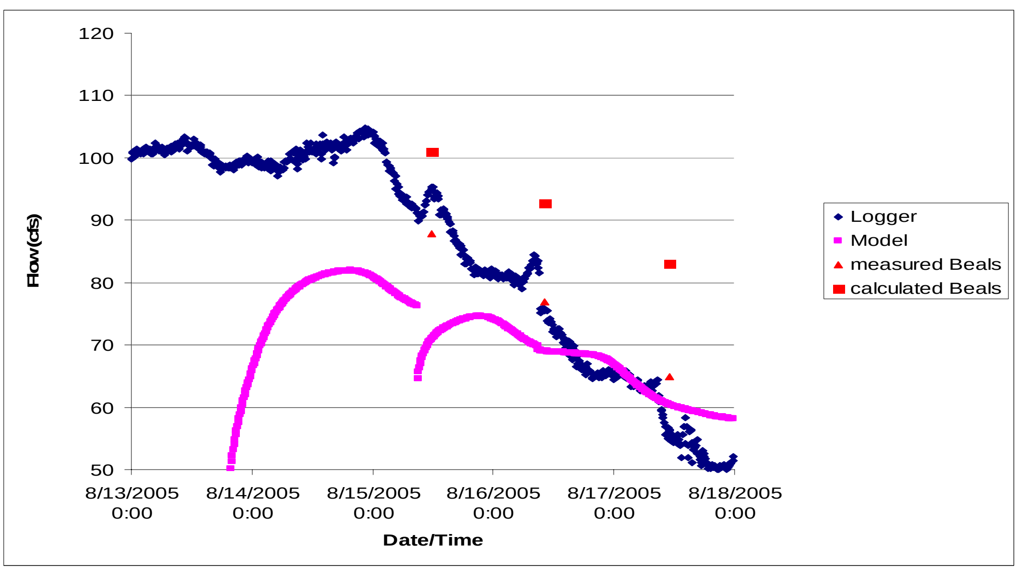

Figure 16. Vale Flume flow, observed vs. modeled for 8/15/2005 – 8/17/2005.

Table 21. Reach 3 check depths, observed vs. modeled for 8/15/2005 – 8/17/2005.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US (I Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS (O Node) Modeled Depth (ft)	Difference (ft)	Percent Error (%)
8/15/2005	Lee	12:03 PM	2.95	2.71	-0.24	-8.14	2.00	2.15	0.15	7.50
8/16/2005	Lee	10:41 AM	2.75	2.39	-0.36	-13.09	1.85	1.95	0.10	5.41
8/17/2005	Lee	11:12 AM	2.50	1.89	-0.61	-24.40	1.65	1.55	-0.10	-6.06
8/15/2005	Lull	12:13 PM	4.20	4.60	0.40	9.52	4.00	4.53	0.53	13.25
8/16/2005	Lull	10:51 AM	4.10	4.35	0.25	6.10	3.85	4.29	0.44	11.43
8/17/2005	Lull	11:20 AM	3.90	3.72	-0.18	-4.62	3.65	3.66	0.01	0.27
8/15/2005	Cottonwood	12:22 PM	2.80	3.12	0.32	11.43	2.20	2.56	0.36	16.36
8/16/2005	Cottonwood	11:02 AM	2.60	2.87	0.27	10.38	1.75	2.35	0.60	34.29
8/17/2005	Cottonwood	11:31 AM	2.40	2.22	-0.18	-7.50	1.40	1.72	0.32	22.86
8/15/2005	Foos	12:28 PM	3.70	4.21	0.51	13.78	1.80	1.70	-0.10	-5.56
8/16/2005	Foos	11:10 AM	3.40	4.00	0.60	17.65	1.70	1.61	-0.09	-5.29
8/17/2005	Foos	11:47 AM	2.90	3.38	0.48	16.55	1.60	1.44	-0.16	-10.00
8/15/2005	Ollilla	12:37 PM	3.20	2.90	-0.30	-9.38	2.30	2.05	-0.25	-10.87
8/16/2005	Ollilla	11:20 AM	2.90	2.75	-0.15	-5.17	2.00	1.96	-0.04	-2.00
8/17/2005	Ollilla	11:55 AM	2.70	2.41	-0.29	-10.74	2.00	1.77	-0.23	-11.50
8/15/2005	Vale	1:00 PM	3.50	3.34	-0.16	-4.57	2.00	1.66	-0.34	-17.00
8/16/2005	Vale	11:43 AM	3.10	3.23	0.13	4.19	1.85	1.60	-0.25	-13.51
8/17/2005	Vale	12:02 PM	2.40	2.71	0.31	12.92	1.90	1.41	-0.49	-25.79

Discussion

During the calibration period the modeled flow produced nearly correct depth results at the Vale Flume when the water orders were added up for Reach 3 (approximately 8 – 14 cfs over the period) and combined with the seepage and evaporation losses (8 cfs). During the validation periods the outflows between the Beals Check and the Vale Flume included assumed evaporation and seepage losses (8 cfs) and water orders/deliveries (~ 8-10 cfs) and produced fair results within the ± 13% range.

For the calibration period 8/1/2005 – 8/4/2005 the simulated depths were all within ± 12% of the observed with the majority being ± 8% of observed depths. The simulated depths for the first validation period 8/8/2005 – 8/11/2005 were all within ± 13% of the observed with the majority being ± 8% of observed. The check depth results for second validation period 8/15/2005 – 8/17/2005 certainly varied more than the calibration period and first validation period with a simulated range of ± 35%. When percent differences were compared, the validation period did not seem to match observed that well; however, the actual depth differences were similar when compared to the calibration and first validation periods. The flows were lower during 8/15/2005 – 8/17/2005, thus lower depths resulting in higher percentage errors for depths.

The resulting Manning's n parameters from the Beals Check outlet to the Cottonwood Check and Siphon are unusually low. It would seem that this may be due to the siphon pipe size being too small in the model, restricting flow, and increasing the upstream depths, but BOR drawings are available for this check and siphon and are modeled according to the drawings. Another likely reason for the low Manning's n parameters is that it is accounting for general cross sectional changes or morphology and sedimentation since the survey data was collected in 1986. The discharge coefficients did not vary as much as other reaches. The Foos Check fell into a discharge coefficient pattern observed in the first two reaches; the Foos Check has one larger gate that led to a lower discharge coefficient as did the Sorenson and the Stinkwater Checks. The Vale Check gate and weir discharge coefficients differed from those that Rolland (2005) found in her investigations; this study: 0.5 and 2.6, Rolland: 0.65 and 3.0. The differences may be due to more recently discovered submergence issues at the Vale Flume. Rolland (2005) was unclear whether or not submergence corrections at the Vale Flume were taken into account in the check calibration investigation.

Results Summary

The compiled calibration results for check structure depth differences and percent differences on the upstream and downstream sides are presented in Table 22 and histograms in Figure 17 and Figure 18, respectively. There were a total of 131 depth measurements simulated during calibration. The compiled validation results are presented in Table 23 and histograms in Figure 19 and Figure 20, respectively. There were 164 total depth measurements simulated during the validation periods. Notice that the histograms show an approximately normal distribution about zero for the simulated depths meaning that the model did not simulate with trends of over or under predicting. The calibration simulated depths were $\pm$ 10% of the observed depths 99% of the simulations and $\pm$ 5% for 89% of the simulations. The validation simulated depths were $\pm$ 10% of the observed depths 89% of the simulations and $\pm$ 5% for 66% of the simulations.

The resulting Manning's n values and orifice and weir discharge coefficients for the entire South Canal are presented in Table 24 and Table 25.

Table 22. Upstream and downstream calibration results statistics.

	US Difference (ft)	US % Difference	DS Difference (ft)	DS % Difference
maximum	0.25	5.56	0.36	11.30
minimum	-0.31	-9.68	-0.22	-8.15
average	0.00	-0.12	0.03	0.91
std dev	0.13	3.23	0.12	3.47

Table 23. Upstream and downstream validation results statistics.

	US Difference (ft)	US % Difference	DS Difference (ft)	DS % Difference
maximum	0.76	17.65	0.64	34.29
minimum	-0.61	-24.40	-0.49	-25.79
average	0.01	-0.13	-0.01	-0.70
std dev	0.27	6.81	0.27	8.60

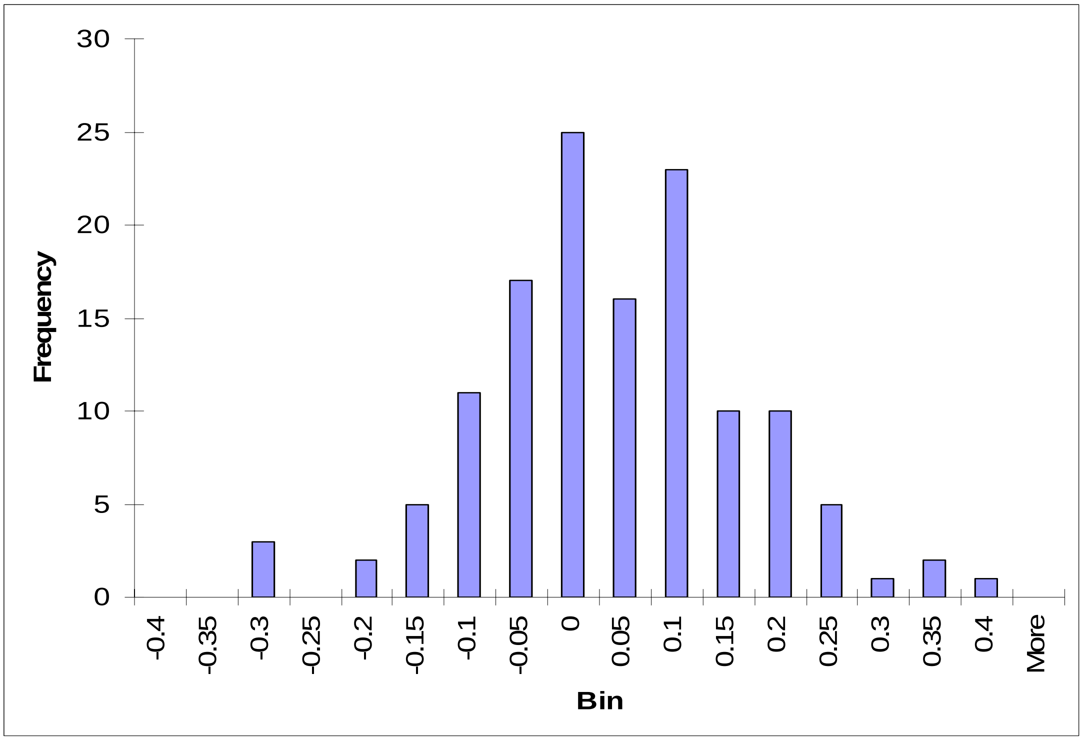

Figure 17. Upstream and downstream calibration results difference (ft) histogram.

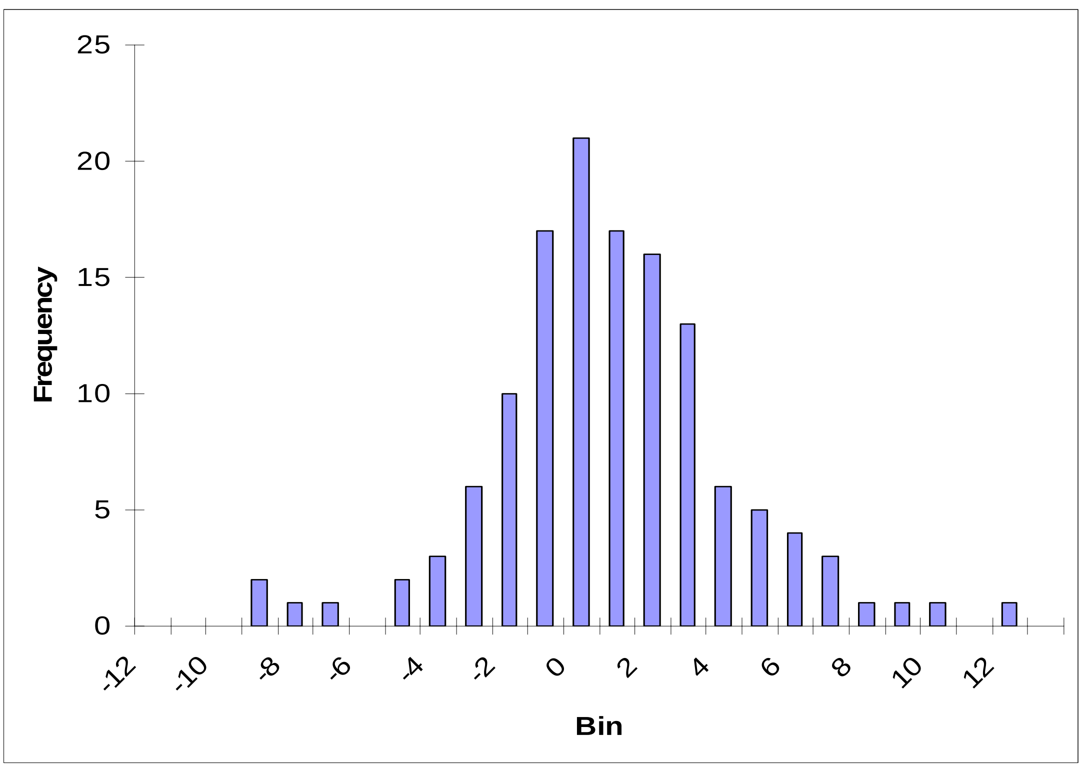

Figure 18. Upstream and downstream calibration results % difference histogram.

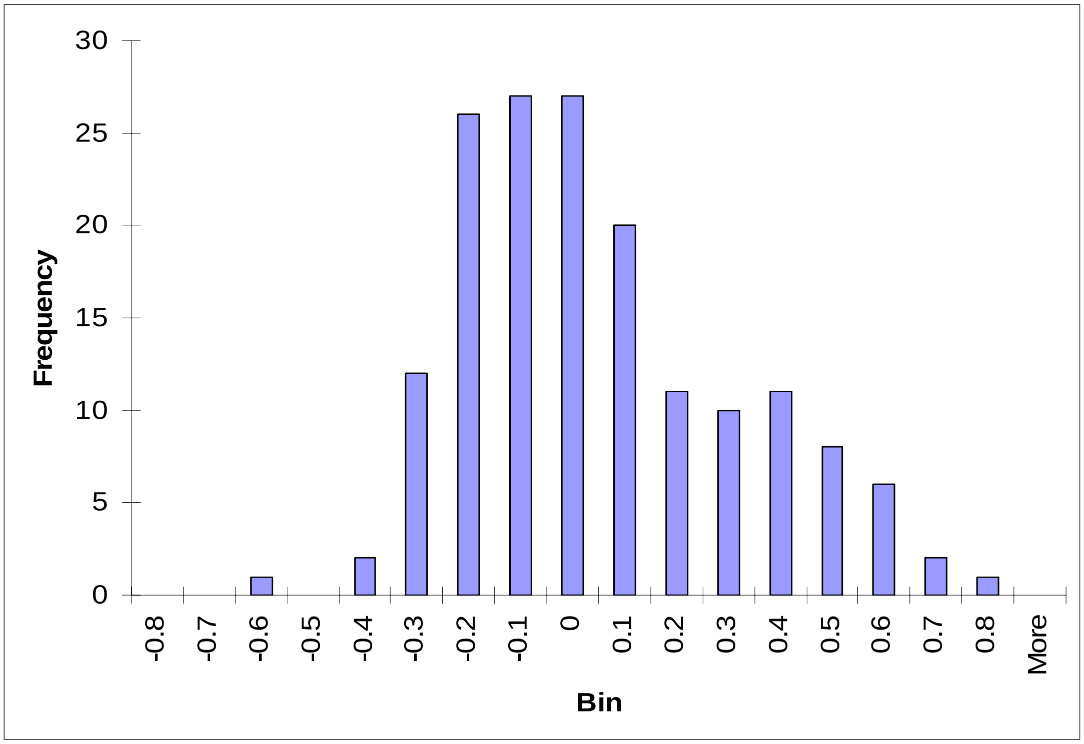

Figure 19. Upstream and downstream validation results difference (ft) histogram.

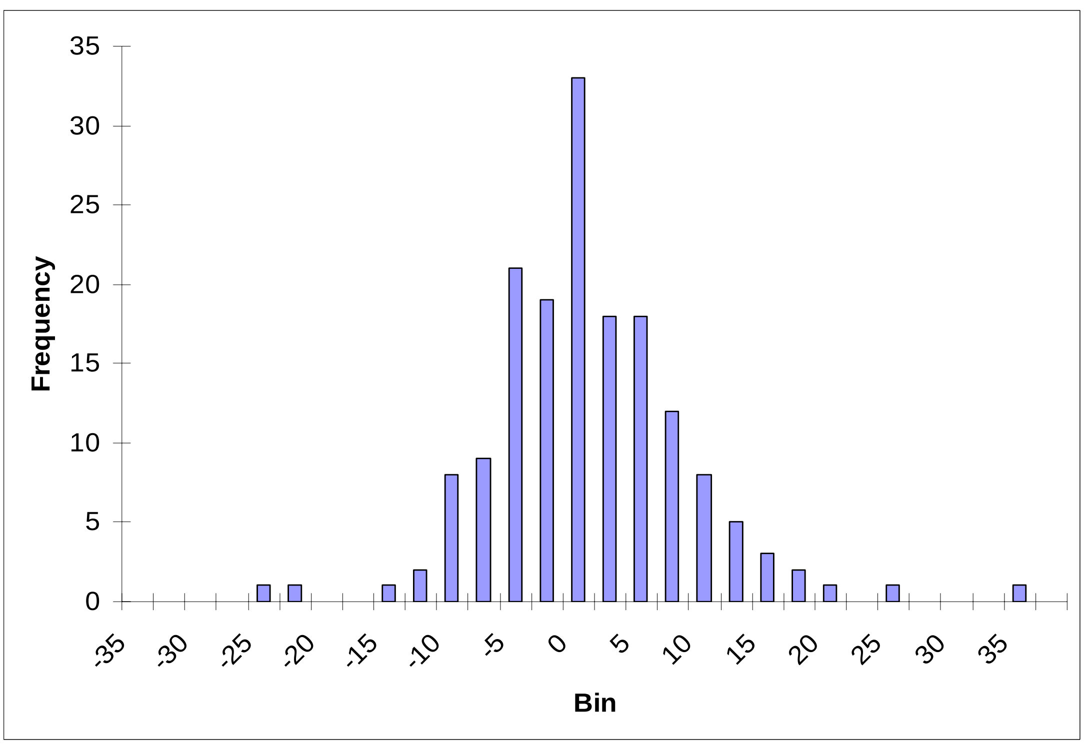

Figure 20. Upstream and downstream validation results % difference histogram.

Table 24. South Canal Manning's n values.

Canal Section	Canal Mile	Manning's n
Dam to Meyer	0 - 1	0.015
Meyer to Todd	1 - 5	0.015
Todd to Eide	5 - 6.2	0.031
Eide to Sorenson	6.2 - 7.7	0.019
Sorenson to BFRS I	7.7 - 8	0.039
BFRS O to Stinkwater	8 - 9.4	0.023
Stinkwater to Minor	9.4 - 11	0.022
Minor to 12.5	11 - 12.5	0.027
12.5 to Kiery	12.5 - 15.1	0.032
Kiery to Simmons	15.1 - 16.2	0.022
Simmons to Whitewood	16.2 - 17.4	0.024
Whitewood to Beals	17.4 - 18.6	0.028
Beals to Lee	18.6 - 20.6	0.010
Lee to Lull	20.6 - 21.5	0.009
Lull to Cottonwood	21.5 - 22.1	0.010
Cottonwood to Foos	22.1 - 22.7	0.022
Foos to Ollilla	22.7 - 23.7	0.017
Ollilla to Vale	23.7 - 26.4	0.017
Vale to 69C-0001	26.4 - 28.1	0.030
69C-0001 to Wasteway	28.1 - 43.8	0.025

Table 25. South Canal check structure discharge coefficient values.

Check Structure	Discharge Coefficients	
	Orifice/Gate	Weir
Meyer	0.67	3.0
Todd	0.67	3.3
Eide	0.60	3.1
Sorenson	0.30	1.5
Stinkwater	0.40	2.6
Minor	0.70	3.0
Anderson	1.00	3.3
12.5	0.65	2.8
Kiery	0.60	2.6
Simmons	0.55	2.6
Whitewood	1.00	3.3
Beals	0.70	2.8
Lee	0.50	2.6
Lull	0.50	2.6
Cottonwood	0.60	2.0
Foos	0.45	2.6
Ollilla	0.65	2.6
Vale	0.50	2.6

SENSITIVITY ANALYSIS

A sensitivity analysis was conducted on the South Canal SWMM model after the calibration/validation of Reaches 1 through 3 was completed. The check structures at the three most important locations along the South Canal were used to assess the sensitivity of the model; the Sorenson (mile 8), Beals (mile 19), and Vale (mile 26) Checks. The model parameters adjusted were main canal Manning's n roughness coefficients and check structure orifice and weir discharge coefficients. Check structure depths and their sensitivity to the parameters were the focus of the sensitivity analysis.

The model was run using a typical mid-to-upper range dam release of 250 cfs. The check structure settings were as normally seen during each respective reach (1 – 3) monitoring period and Reach 4 checks were all set at fifty percent open. In order to observe only the effect of the parameters being adjusted no turnouts or laterals were used to take flow out of the system. If turnouts/laterals were used the outflows would fluctuate with canal depth and therefore produce indeterminate depth changes during the analysis. In order to simulate controlled outflows, which were necessary to prevent system flooding, pumps were used to discharge amounts that were consistent and independent of canal depth. The pumping rates were determined for each reach by evaluating the normal mid-to-upper range of flows at the key flow measuring structures.

Manning's n was adjusted $\pm$ 10% and $\pm$ 25% for each of the three checks, which were analyzed separately. Manning's n was adjusted using two different methods. First, it was adjusted over the entire length of the South Canal. Second, it was adjusted to the nearest upstream and downstream checks of the check being analyzed. The results for the first method are presented in Table 26, which contains Manning's n, upstream (US) and downstream (DS) depth, and discharge percent changes. Also shown in the table are results of relative percent sensitivity, S, for upstream and downstream depths and discharge. Relative percent sensitivity is calculated by dividing the percent change in the model result by the percent change in the parameter, and a larger S value indicates higher sensitivity. The results for the second method of Manning's n adjustment are presented in Table 27, which contains the same information as Table 26 except the discharge results because they were found to be insignificant during the first method analysis.

There were two observed patterns that developed from the sensitivity analysis of Manning's n roughness coefficient. The first observed pattern was that Manning's n has a greater effect on the downstream side of check structures. The downstream depth was affected more for all three checks and for both methods of parameter adjustment. Possible explanation is that the downstream depths are most often significantly less than the upstream depths with the exception of siphon check structures. Depths of lesser value become more sensitive in terms of percent changes than greater depths.

The second observed pattern was that check structure depths become increasingly sensitive to changes in Manning's n farther downstream, which was true for both methods of parameter adjustment. A possible explanation for this pattern is that when Manning's n is adjusted over the entire length of canal it will have a cumulative effect on the flow as it travels down the canal, thus affecting downstream checks more than those located close to the Dam.

Table 26. Sensitivity analysis results for Manning's n method 1.

Check Structure	Manning's n % Change	US Depth % Change	DS Depth % Change	Discharge % Change	US Depth S	DS Depth S	Discharge S
Sorenson	-25	-4.22	-7.19	0.19	0.17	0.29	-0.007
	-10	-1.74	-3.13	0.06	0.17	0.31	-0.006
	10	1.74	3.12	-0.19	0.17	0.31	-0.019
	25	4.47	7.81	-0.25	0.18	0.31	-0.010
Beals	-25	-6.59	-13.47	0.35	0.26	0.54	-0.014
	-10	-2.33	-5.18	0.09	0.23	0.52	-0.009
	10	2.33	4.66	-0.09	0.23	0.47	-0.009
	25	5.43	11.92	-0.26	0.22	0.48	-0.011
Vale	-25	-11.08	-20.69	0.44	0.44	0.83	-0.018
	-10	-2.77	-5.91	0.11	0.28	0.59	-0.011
	10	2.77	5.91	-0.11	0.28	0.59	-0.011
	25	6.77	14.29	-0.33	0.27	0.57	-0.013

Table 27. Sensitivity analysis results for Manning's n method 2.

Check Structure	Manning's n % Change	US Depth % Change	DS Depth % Change	US Depth S	DS Depth S
Sorenson	-25	-3.97	-6.88	0.16	0.28
	-10	-1.74	-2.81	0.17	0.28
	10	1.74	3.12	0.17	0.31
	25	4.47	7.81	0.18	0.31
Beals	-25	-6.95	-12.44	0.28	0.50
	-10	-2.70	-5.18	0.27	0.52
	10	1.93	4.66	0.19	0.47
	25	5.02	11.40	0.20	0.46
Vale	-25	-7.38	-13.79	0.30	0.55
	-10	-2.77	-5.91	0.28	0.59
	10	2.77	5.42	0.28	0.54
	25	6.77	13.79	0.27	0.55

Orifice and weir discharge coefficients were adjusted $\pm$ 25% using three different methods. First, both orifice and weir discharge coefficients were all adjusted at the same time for each particular check structure. Second, only the orifice discharge coefficients were adjusted and third, only the weir discharge coefficients were adjusted. The results for all three situations are presented in Table 28, which contains discharge coefficient and upstream (US) depth percent changes, and relative percent sensitivity, S. The downstream depth results are not presented because they were not affected by the discharge coefficients.

The discharge coefficient effects on check structure depths seemed to be variable and depend on the particular check structure. The Beals has the larger discharge coefficient values for both orifice and weir of the three checks analyzed; thus it makes sense that it was the most sensitive to parameter adjustment. Following the same reasoning it makes sense that the Vale was moderately sensitive and the Sorenson was the least sensitive. Note that the same pattern of check sensitivity order was followed for the orifice only adjustment. However, in the weir only adjustments the Sorenson was the most sensitive, which may be explained by the relative size of the weirs present at the Sorenson. Although all three checks have two adjustable weirs apiece, the Sorenson has two weirs that are larger by 3 to 4 feet than those of the Beals and the Vale. It is also important to note that the orifice discharge coefficient had a significantly larger effect than the weir discharge coefficient, which may be explained by the way that flow is conveyed through each respective control structure. The sluice gate orifices convey water underneath and the weirs convey water over the top, thus the sluice gates have larger head pressure that control flow conveyance.

Table 28. Sensitivity analysis results for orifice & weir discharge coefficients.

Check Structure	Discharge Coefficient % Change	Orifice & Weir US Depth % Change	Orifice US Depth % Change	Weir US Depth % Change	Orifice & Weir US Depth S	Orifice US Depth S	Weir US Depth S
Sorenson	-25	12.16	5.21	4.71	-0.49	-0.21	-0.19
	25	-6.70	-4.22	-3.72	-0.27	-0.17	-0.15
Beals	-25	30.23	25.97	0.78	-1.21	-1.04	-0.03
	25	-28.29	-28.29	-0.39	-1.13	-1.13	-0.02
Vale	-25	18.77	16.92	0.31	-0.75	-0.68	-0.01
	25	-12.62	-12.62	-0.31	-0.50	-0.50	-0.01

In summary, check structure depth sensitivity to Manning's n adjustments depended on the depths usually occurring at the check, and the checks distance downstream 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. Sensitivity plots of the Sorenson, Beals, and Vale Checks under the conditions described above are shown below in Figure 21, Figure 22, and Figure 23.

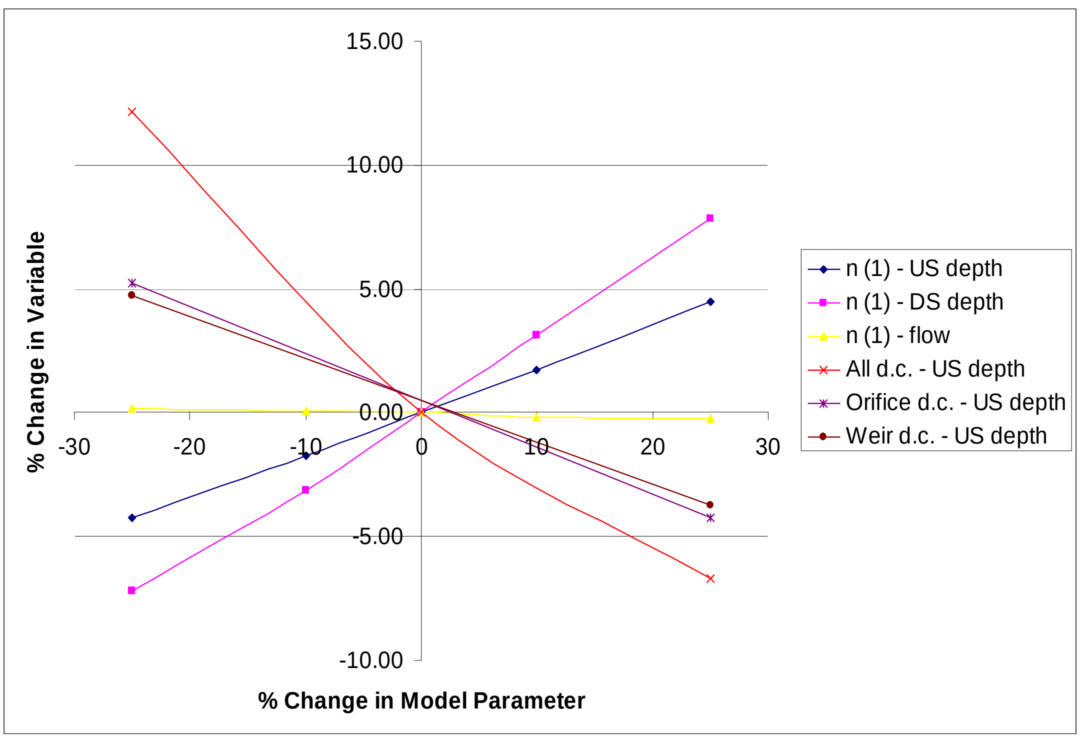

Figure 21. Sorenson Check sensitivity analysis plot.

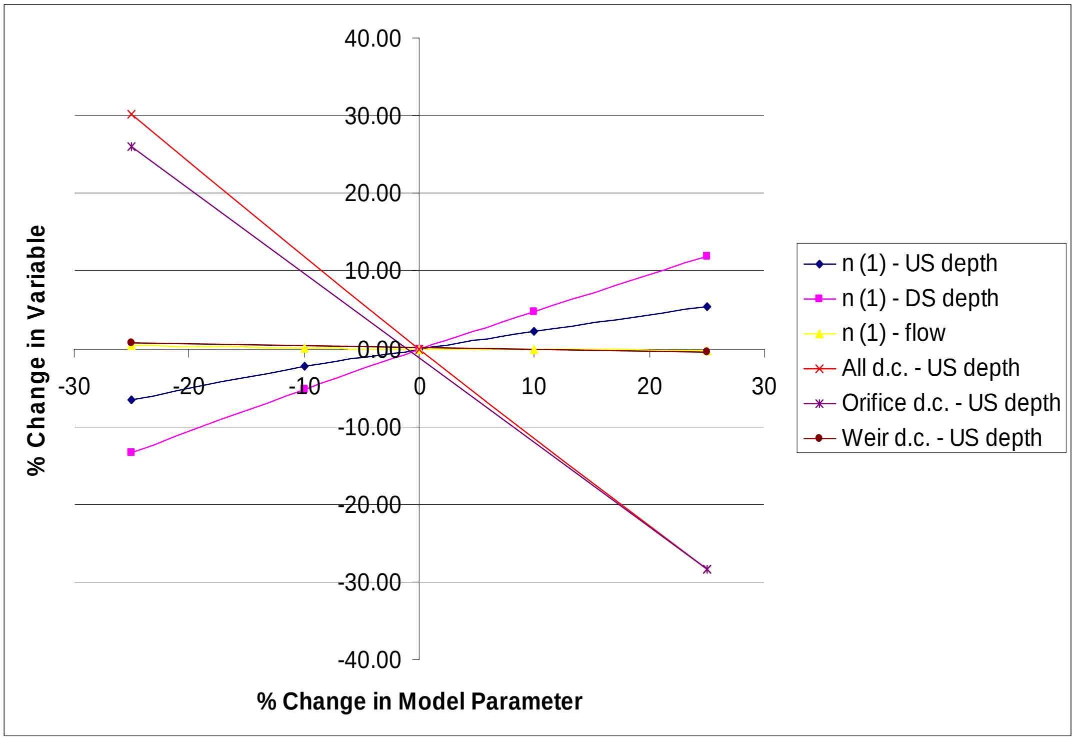

Figure 22. Beals Check sensitivity analysis plot.

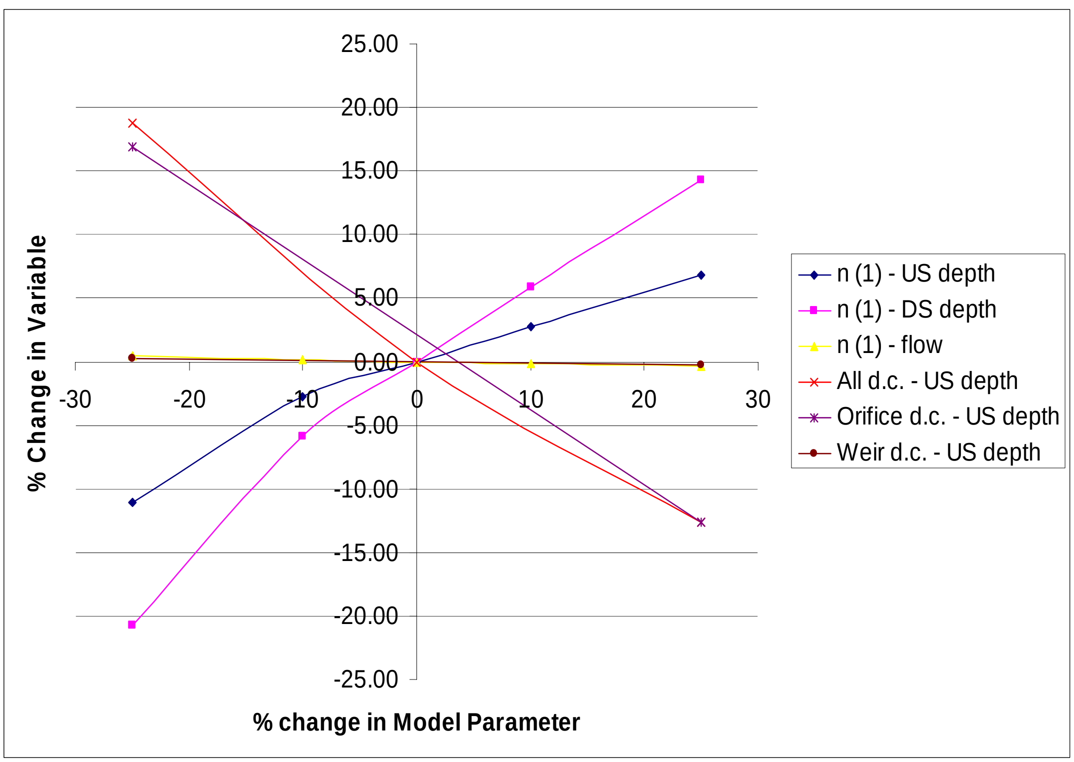

Figure 23. Vale Check sensitivity analysis plot.

MODEL VALIDATION 2006

Data for the model validation was collected during the 2006 irrigation season, which started on June 7th. Data was only collected for a few weeks due to time and effort being focused on unforeseen problems with new equipment and technology installed in the BFID. The 2006 model validation was done using fewer assumptions than the calibration/validation using 2005 data. First, continuous Johnson Lateral return flows on Reach 1 were collected using a data logger and pressure transducer at the return box weir and calculated flows were put into the model as time series data. Second, more ditch rider communication was conducted in order to correctly model the time of structure setting changes at check structures and turnouts/laterals. The ditch riders filled out a structure setting change sheet daily that was used to address the changes in the model. Third, newly available real-time data were used to enter observed Dam releases into the model as time series data.

Flush Wave

Travel times of the leading edge of water were collected from personal observations by the author or ditch riders during the initial season flush. The flush is done every season at the start up to clean debris out of the canal prior to deliveries. A comparison of the observed and model predicted arrival times of the front edge of the South Canal flush wave was conducted. The flush Dam release was approximately 77 cfs released at 6:30 am on Wednesday, June 7. The flush was modeled with all check structure gates and weirs 100% open, all turnouts/laterals closed, and all seepage and evaporation losses in effect (1 cfs/mile). During the flush the water was checked up and released out of wasteways at the Anderson, Whitewood, and Perry Checks for 1 hour, 1 hour 45 minutes, and 4 hours, respectively. These checked up wasting times were modeled accordingly and the nearby upstream head gate was opened 100% to serve as the wasteway.

The observed versus modeled arrival times are presented in Table 29. The observed travel times to the BFRS Flume and Vale Flume were 8 hours 40 minutes and 37 hours 30 minutes, respectively. The model predicted all flush wave arrival times within 2 hours. Most of the modeled arrival times were earlier than observed, which may be explained by additional losses or slower travel rates due to the wetting front and dry canal conditions at start up.

Table 29. June 7, 2006 flush wave arrival time model comparison.

South Canal Location	Date	Observed Arrival Times	Checked Up Waste Released	Modeled Arrival Times	Modeled Difference
Sorenson Check	6/7	3:10 PM	-----	2:30 PM	-40 min
BFRS Flume	6/7	3:30 PM	-----	2:50 PM	-40 min
Anderson Check	6/7	8:00 PM	9:00 PM	8:30 PM	+30 min
Whitewood Check	6/8	6:30 AM	8:15 AM	5:10 AM	-1 hr 20 min
Cottonwood Check	6/8	2:00 PM	-----	12:30 PM	-1 hr 30 min
Culvert DS S.C. 23.5	6/8	4:10 PM	-----	2:40 PM	-1 hr 30 min
Vale Check	6/8	8:00 PM	-----	6:00 PM	-2 hr
Perry Check	6/9	5:00 AM	9:00 AM	3:30 AM	-1 hr 30 min
Meade Check	6/9	10:30 AM	-----	11:40 AM	+1 hr 10 min

Reach 1: 6/19/2006 – 6/22/2006

Check structure and turnout/lateral data was collected on Reach 1 during the dates 6/19/2006 – 6/22/2006. Dam releases for the period ranged from 190 cfs to 305 cfs. Continuous real-time flow data at the Dam Flume was available and used in the model. Continuous flow data at the BFRS Flume was unavailable over the period due to newly installed real-time data equipment issues, thus daily staff gage measurements were used. Downstream data at turnouts/laterals with a weir box or flume were collected and used to assess the model outflows. Where downstream flow data did not exist on the turnouts/laterals the water cards and ditch rider input were used to assess the model outflows where water was actually being delivered.

The modeled flows at the BFRS Flume versus the observed daily measurement flows are presented in Figure 24. The modeled BFRS Flume flows matched well towards the beginning of the simulation. However, modeled flows were approximately 25 cfs (10%) and 17 cfs (7%) lower than the observed towards the end of the simulation. It seems that largest difference on 6/21 may be a timing issue. The resulting modeled check structure depths versus observed are presented in Table 30. The model predicted all of the check structure depths within 10%, except for two extreme low values of – 11% and – 16%.

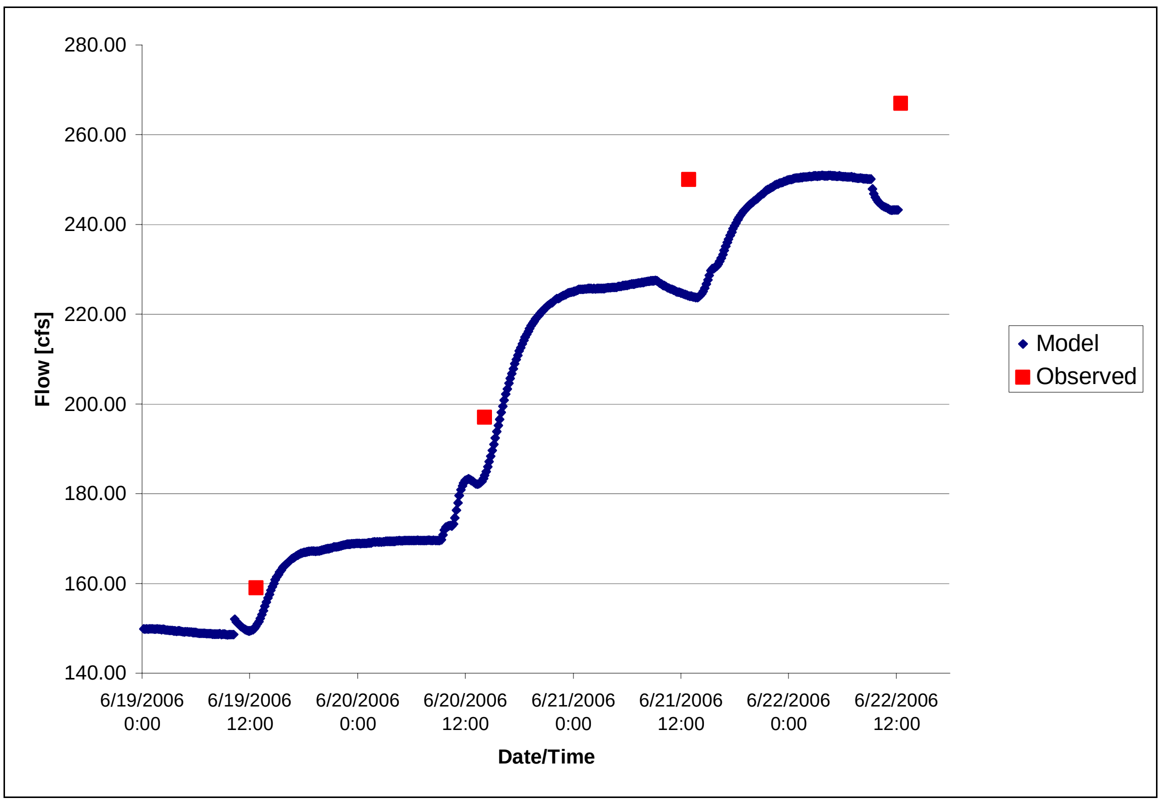

Figure 24. BFRS Flume flow, observed vs. modeled for 6/19/2006 – 6/22/2006.

Table 30. Reach 1 check depths, observed vs. modeled for 6/19/2006 – 6/22/2006.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US Modeled Depth (ft)	US Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS Modeled Depth (ft)	DS Difference (ft)	Percent Error (%)
6/19/2006	Meyer	10:30 AM	2.45	2.43	-0.02	-0.82	1.80	1.78	-0.02	-1.11
6/21/2006	Meyer	11:40 AM	3.00	3.24	0.24	8.00	2.40	2.37	-0.03	-1.25
6/22/2006	Meyer	11:18 AM	3.20	3.52	0.32	10.00	2.40	2.59	0.19	7.92
6/19/2006	Todd	11:18 AM	5.25	5.24	-0.01	-0.19	3.90	4.17	0.27	6.92
6/21/2006	Todd	11:17 AM	5.50	5.80	0.30	5.45	5.00	5.22	0.22	4.40
6/22/2006	Todd	11:49 AM	5.80	6.36	0.56	9.66	5.20	5.55	0.35	6.73
6/19/2006	Eide	11:51 AM	4.15	4.14	-0.01	-0.24	3.60	3.18	-0.42	-11.67
6/21/2006	Eide	11:50 AM	5.20	5.20	0.00	0.00	4.30	4.06	-0.24	-5.58
6/22/2006	Eide	12:03 PM	5.35	5.52	0.17	3.18	4.50	4.31	-0.19	-4.22
6/19/2006	Sorenson	12:17 PM	4.10	3.41	-0.69	-16.83	2.85	3.03	0.18	6.32
6/20/2006	Sorenson	12:10 PM	4.30	3.91	-0.39	-9.07	3.20	3.44	0.24	7.50
6/21/2006	Sorenson	12:08 PM	4.70	4.47	-0.23	-4.89	3.60	3.92	0.32	8.89
6/22/2006	Sorenson	12:18 PM	4.90	4.71	-0.19	-3.88	3.80	4.12	0.32	8.42

Reaches 1 – 3: 7/11/2006 – 7/14/2006

Check structure and turnout/lateral data was collected on Reaches 1 through 3 during the dates 7/11/2006 – 7/14/2006. Continuous data was available real-time at the South Canal Dam Flume, BFRS Flume, Beals Check, and Vale Flume. Problems with the downstream pressure transducer in the Vale Flume caused submergence to be unavailable over the validation period. A daily submergence calculation was done each day from staff gage readings at the flume and submergence values were 81%, 81%, and 82%. The submergence correction was approximately 2 cfs for each single measurement. Because submergence was not available from the real time data, the 2 cfs was assumed constant and taken off the logger recorded flow values. Downstream flow measuring devices, water cards, and ditch rider structure setting change sheets were used to assess the system outflow changes. Automated check structures functioning during the period were the Beals, Cottonwood, and Vale Checks and were simulated in the model.

For the validation period 7/11/2006 – 7/13/2006 the South Canal Dam releases ranged from 315 cfs to 345 cfs. The observed versus modeled flows at each of the major flow measuring structures on the South Canal are presented in Figure 25, Figure 26, and Figure 27. The worst case modeled flows differed from observed flows by approximately – 5 cfs (2%) at the BFRS Flume, and – 5 cfs (3%) at the Beals Check. The modeled versus observed flows at the Vale Flume produced more variability and differed by as much as – 12 cfs (8%). As discussed for the model calibration of Reach 3, accuracy at the Vale Flume is somewhat limited due to submergence and the accuracy has an attenuating negative effect by the time it reaches the end of Reach 3. The resulting observed versus modeled check structure depths are presented in Table 31. The check structure depths simulated by the model had results similar to the 2005 calibration/validation periods with most modeled values being ± 10% and a few values exceeding that range, which were consistently at or near automated check structures where depths are relatively cyclic and unstable.

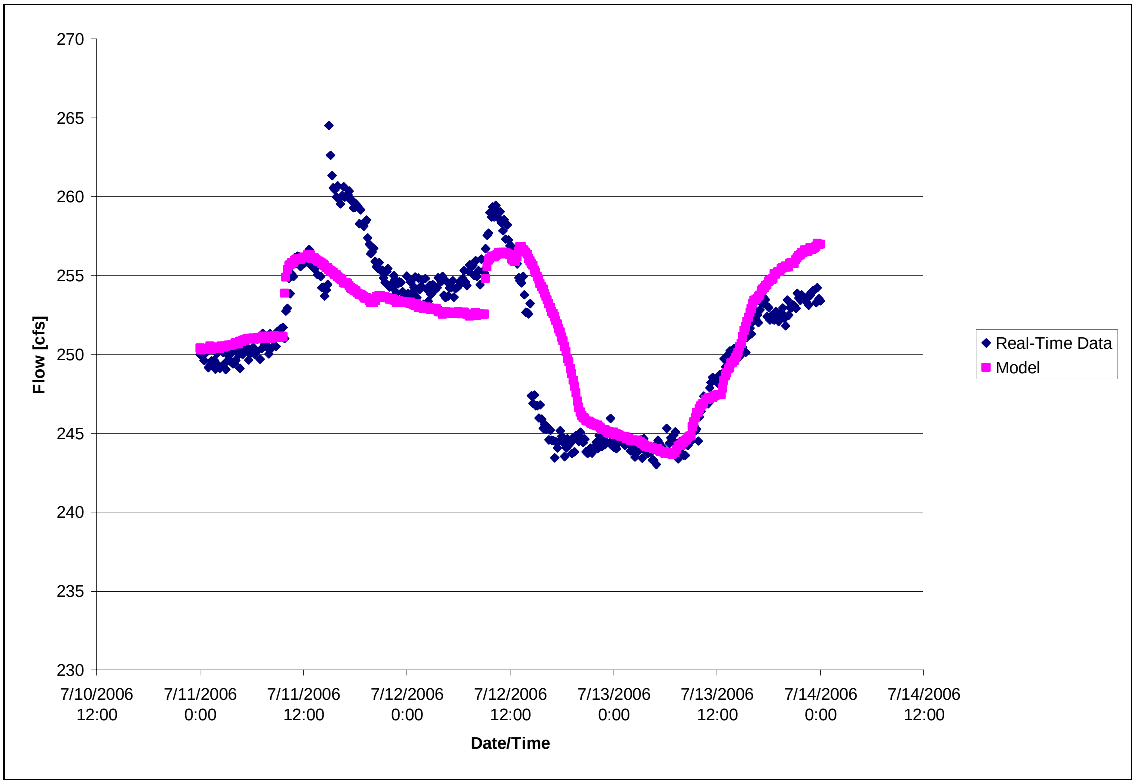

Figure 25. BFRS Flume flow, observed vs. modeled for 7/11/2006 – 7/14/2006.

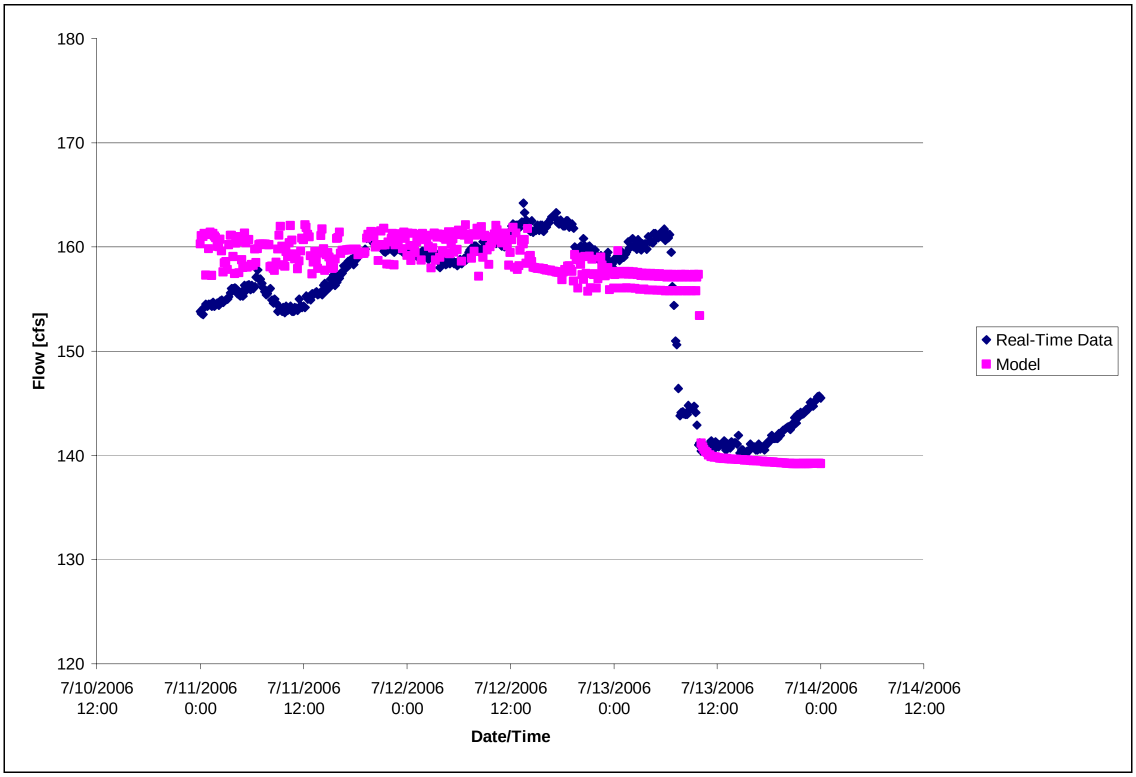

Figure 26. Beals Check flow, observed vs. modeled for 7/11/2006 – 7/14/2006.

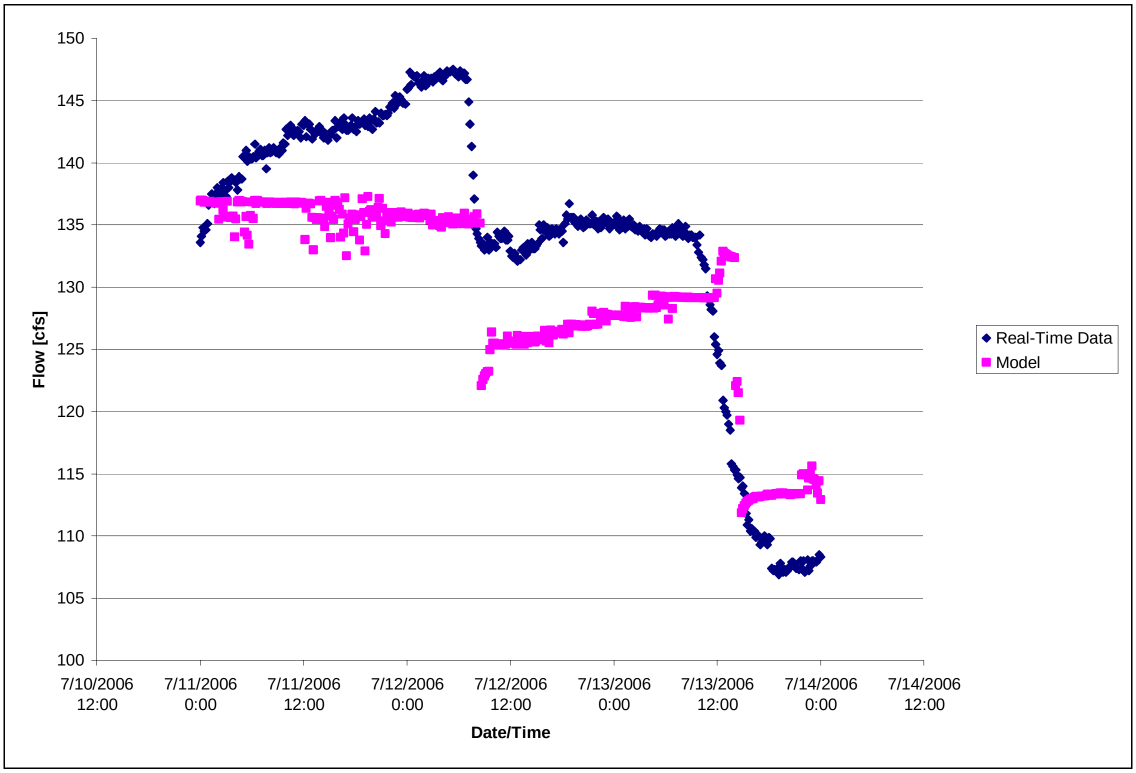

Figure 27. Vale Flume flow, observed vs. modeled for 7/11/2006 – 7/14/2006.

Table 31. Check structure depths, observed vs. modeled for 7/11/2006 – 7/14/2006.

Date	Check Structure	Observed Time	US Observed Depth (ft)	US Modeled Depth (ft)	US Difference (ft)	Percent Error (%)	DS Observed Depth (ft)	DS Modeled Depth (ft)	DS Difference (ft)	Percent Error (%)
7/11/2006	Meyer	8:08 AM	3.45	3.79	0.34	9.86	2.95	2.78	-0.17	-5.76
7/11/2006	Todd	8:55 AM	5.60	6.17	0.57	10.18	5.30	5.70	0.40	7.55
7/11/2006	Eide	9:11 AM	5.30	5.47	0.17	3.21	4.60	4.51	-0.09	-1.96
7/11/2006	Sorenson	9:33 AM	4.95	4.91	-0.04	-0.81	4.50	4.19	-0.31	-6.89
7/11/2006	Stinkwater	10:21 AM	6.30	6.41	0.11	1.75	5.80	5.79	-0.01	-0.17
7/11/2006	Minor	12:30 PM	5.00	4.96	-0.04	-0.80	5.20	5.38	0.18	3.46
7/11/2006	Anderson	12:47 PM	5.40	5.16	-0.24	-4.44	8.65	8.64	-0.01	-0.12
7/11/2006	12.5	1:00 PM	6.35	6.20	-0.15	-2.36	6.25	6.03	-0.22	-3.52
7/11/2006	Kiery	1:30 PM	4.80	5.26	0.46	9.58	4.00	4.10	0.10	2.50
7/11/2006	Simmons	1:46 PM	4.40	4.53	0.13	2.95	3.00	2.91	-0.09	-3.00
7/11/2006	Whitewood	1:58 PM	5.90	5.42	-0.48	-8.14	4.80	5.15	0.35	7.29
7/11/2006	Beals	2:22 PM	4.35	4.24	-0.11	-2.53	3.25	3.15	-0.10	-3.08
7/11/2006	Lee	2:45 PM	4.00	4.28	0.28	7.00	2.60	2.93	0.33	12.69
7/11/2006	Lull	2:54 PM	4.95	5.24	0.29	5.86	4.80	5.10	0.30	6.25
7/11/2006	Cottonwood	3:00 PM	3.40	3.54	0.14	4.12	2.60	3.01	0.41	15.77
7/11/2006	Foos	3:11 PM	3.95	4.34	0.39	9.87	2.95	2.77	-0.18	-6.10
7/11/2006	Ollilla	3:20 PM	4.20	4.27	0.07	1.67	2.75	2.90	0.15	5.45
7/11/2006	Vale	3:37 PM	4.00	4.00	0.00	0.00	2.65	2.88	0.23	8.68
7/12/2006	Meyer	10:25 AM	3.30	3.63	0.33	10.00	2.50	2.67	0.17	6.80
7/12/2006	Todd	10:40 AM	5.50	6.05	0.55	10.00	5.10	5.60	0.50	9.80
7/12/2006	Eide	10:52 AM	5.10	5.41	0.31	6.08	4.50	4.50	0.00	0.00
7/12/2006	Sorenson	11:05 AM	4.90	5.06	0.16	3.27	4.70	4.36	-0.34	-7.23
7/12/2006	Stinkwater	11:30 AM	6.25	6.34	0.09	1.44	5.80	5.73	-0.07	-1.21
7/12/2006	Minor	11:39 AM	5.10	4.93	-0.17	-3.33	5.25	5.34	0.09	1.71
7/12/2006	Anderson	11:45 AM	5.40	5.11	-0.29	-5.37	8.60	8.59	-0.01	-0.12
7/12/2006	12.5	12:02 PM	6.35	6.18	-0.17	-2.68	6.25	6.01	-0.24	-3.84
7/12/2006	Kiery	12:10 PM	4.80	5.25	0.45	9.38	4.00	4.10	0.10	2.50
7/12/2006	Simmons	12:16 PM	4.50	4.53	0.03	0.67	3.20	2.90	-0.30	-9.38
7/12/2006	Whitewood	12:23 PM	6.00	5.41	-0.59	-9.83	5.10	5.15	0.05	0.98
7/12/2006	Beals	12:30 PM	4.30	4.24	-0.06	-1.40	3.40	3.16	-0.24	-7.06
7/12/2006	Lee	12:53 PM	4.15	4.28	0.13	3.13	2.95	2.94	-0.01	-0.34
7/12/2006	Lull	1:00 PM	5.00	5.24	0.24	4.80	4.80	5.11	0.31	6.46
7/12/2006	Cottonwood	1:04 PM	3.40	3.55	0.15	4.41	2.70	3.02	0.32	11.85
7/12/2006	Foos	1:10 PM	4.10	4.34	0.24	5.85	3.20	2.78	-0.42	-13.13
7/12/2006	Ollilla	1:14 PM	4.40	4.28	-0.12	-2.73	2.90	2.90	0.00	0.00
7/12/2006	Vale	1:21 PM	4.00	4.07	0.07	1.75	2.50	2.71	0.21	8.40
7/13/2006	Meyer	10:48 AM	3.25	3.56	0.31	9.54	2.55	2.61	0.06	2.35
7/13/2006	Todd	11:00 AM	5.40	5.76	0.36	6.67	5.10	5.44	0.34	6.67
7/13/2006	Eide	11:10 AM	5.10	5.24	0.14	2.75	4.30	4.36	0.06	1.40
7/13/2006	Sorenson	11:20 AM	4.80	4.88	0.08	1.67	4.50	4.17	-0.33	-7.33
7/13/2006	Stinkwater	11:48 AM	6.10	6.30	0.20	3.28	5.50	5.70	0.20	3.64
7/13/2006	Minor	11:58 AM	4.90	4.87	-0.03	-0.61	5.00	5.29	0.29	5.80
7/13/2006	Anderson	12:02 PM	5.10	5.03	-0.07	-1.37	8.40	8.52	0.12	1.43
7/13/2006	12.5	12:09 PM	6.20	6.13	-0.07	-1.13	6.05	5.97	-0.08	-1.32
7/13/2006	Kiery	12:20 PM	4.75	5.21	0.46	9.68	3.95	4.06	0.11	2.78
7/13/2006	Simmons	12:26 PM	4.25	4.49	0.24	5.65	2.95	2.87	-0.08	-2.71
7/13/2006	Whitewood	12:35 PM	5.75	5.37	-0.38	-6.61	4.70	5.11	0.41	8.72
7/13/2006	Beals	1:01 PM	4.15	4.20	0.05	1.20	3.25	2.89	-0.36	-11.08
7/13/2006	Lee	12:53 PM	3.70	4.00	0.30	8.11	2.50	2.75	0.25	10.00
7/13/2006	Lull	1:07 PM	4.75	5.09	0.34	7.16	4.65	4.97	0.32	6.88
7/13/2006	Cottonwood	1:10 PM	3.40	3.46	0.06	1.76	2.30	2.66	0.36	15.65
7/13/2006	Foos	1:14 PM	3.70	4.06	0.36	9.73	2.70	2.60	-0.10	-3.70
7/13/2006	Ollilla	1:18 PM	4.00	4.09	0.09	2.25	2.70	2.79	0.09	3.33
7/13/2006	Vale	1:29 PM	4.00	4.00	0.00	0.00	2.40	2.82	0.42	17.50

Discussion

A few changes were made to the model either prior to or during the validation runs. First, the 12.5 Check culvert diameters were unknown at the time of model calibration with 2005 data. The model was developed with an assumed 4.5 feet diameter that was used during the calibration process. Prior to the 2006 irrigation season the culvert diameters were measured to be 5.5 feet and the model was changed accordingly. The adjustment of the 12.5 Check culverts produced minimal effects on nearby upstream and downstream reaches, which may be explained by the culverts not being at full capacity even at high flows. Second, locations of main canal lining were determined from the BFID manager and Water Master to be most of the canal between the Todd Check and the Beals Check. During the validation simulations the lined stretch of main canal was assumed to have no seepage or evaporation losses and the loss pumps were turned off in the model. Third, errors in depth measurements at the Sorenson Check collected during the 2005 irrigation season were observed during the 2006 irrigation season monitoring. The Sorenson Check discharge coefficients were re-calibrated to the correct depth measurements during the validation period simulations with minimal effects on nearby reaches. The resulting orifice and weir discharge coefficients at the Sorenson Check were 0.5 and 2.6, respectively. The updated discharge coefficient values fit more into an expected range and better match other calibrated values.

During the validation runs there was a water balance problem observed on Reach 1, which was also being observed by the BFID manager and ditch rider in the field. It seemed that the canal was gaining water somewhere between the Dam Flume and the BFRS Flume. The ditch rider was certain he was delivering the correct amount of water, but the difference in flow between the Dam Flume and the BFRS Flume indicated that he was delivering 6 – 8 cfs less than he should have been. The model simulations were producing similar results; the observed BFRS Flume flows were approximately 6 – 8 cfs higher than the model was predicting. As verification, both outside staff gages that the ditch rider reads were checked against the calibrated staff gages inside the stilling wells at the flumes. The BFRS Flume staff gage was calibrated correctly; however, the Dam Flume staff gage was reading 0.04 feet less than the stilling well gage. Thus, there was approximately 6 – 8 cfs (U.S. Bureau of Reclamation, 2001) more being released from the Dam than expected. With the Dam staff gage calibrated correctly, the water balances better matched up in reality and in the model simulations.

Results Summary

During the 2006 validation periods with upstream and downstream depths compiled (134 measurements), 94% of the simulated depths were within ± 10% of observed, 58% within ± 5%, and 40% within ± 2.5%. The model validation of 2006 produced results similar to the model calibration/validation of 2005. However, the validation results were slightly skewed to the positive, which may be explained by variability in the water balances.

MODEL APPLICATION

In order for the model to fit into the BFID daily routine a structured system must be developed. There are three necessary updating steps required in order to run the South Canal model each morning: update simulation dates and times, update Dam release and Johnson Lateral return time series data, and update structure setting control rules. Since the model will be used as a predictive tool, best guess predictions of Dam releases, Johnson Lateral returns, check structure adjustments, and turnout/lateral adjustments must also be assessed. A possible structured scenario of preparing the model simulation for the day might be as follows:

1. Modeler updates the dates and times for the day’s simulation.

2. Water cards are entered into database, summarized, and used to determine the day’s or period of days projected Dam release. Modeler updates Dam release time series data.

3. Some means of predicting Johnson Lateral return flows are assessed, perhaps an average of the last day or few days. Modeler updates Johnson Lateral return flows time series data.

4. Ditch riders give modeler a sheet of structure setting changes made during the previous day with times and adjusted measurements. Modeler updates control rules to simulate current structure settings.

5. Modeler uses water cards and ditch rider input to assess any farmer turnout shut-offs or turn-ons, and also any major lateral increases or

decreases expected during the simulated day or period of days. Modeler updates control rules to simulate expected water delivery changes.

6. The model is run simulating up-to-date BFID system characteristics to predict possible scenarios for the day or period of days.

There are a couple of options concerning how the model can be used, which will depend on what exactly the modeling objective is. First, the model could be run as a whole, from the Dam to the Wasteway, with the only real-time flow inputs being Dam releases and Johnson Lateral returns. Running the model as a whole may be beneficial in assessing effects on the lower reaches of the South Canal from upstream changes or Dam releases, and the travel times that may be expected for these effects to reach specific locations. It is important to keep in mind the complexity and attenuating variability further downstream. Second, the model could be broken down into individual ditch rider rides where real-time data is available at the beginning and end of the ride, such as the South Canal Dam Flume, BFRS Flume, Beals Check, and Vale Flume. The model could be broken down even further as more real-time flow sites become available in the BFID. A possible benefit of breaking the model into rides is that the model can be easily corrected at each real-time flow site with actual data. Inputting correct real-time flow data would lead to less variability on each ride, which is especially important for the lower reaches of the canal. The model being run for an individual ride would also make it easier to analyze and predict possible scenarios and situations specific to the ride. Each modeling situation may have its own beneficial uses and applications that would be determined by the objective of the situation.

The model could also be used to obtain information on check structures throughout the South Canal. The model could be used to run different flow scenarios at check structures to obtain a range of possible settings for optimal operation. There are two or three common flow regimes that should be looked at with the model, each varying with the flow demand for each particular reach (Olson, 2006). The model calibration was completed with a priority on check structures and the goal of producing useful outputs that will aid the ditch riders in making decisions on setting their check structure gates and weirs. The district could also use the model for timing issues. The model will allow the district to predict when changes at the Dam will affect the flow and water stages at various locations along the South Canal. These predictions will better allow the ditch riders to get to their structures at the optimal time and adjust them to account for fluctuations in the canal. The ditch riders accounting for canal fluctuations will lead to more accurate deliveries and reduce the amount of nonused water applied to fields.

RECOMMENDATIONS AND CONCLUSIONS

Recommendations for Future Modeling Efforts

The following recommendations will help the BFID improve operational efficiency in an overall effort to reduce nonused irrigation return flows to the Belle Fourche River system by 12,000 acre-feet by the year 2015. Recommendations are provided for BFID hydraulic computer modeling efforts (in no particular order):

1. <u>Complete Development of Physical Data</u>: It is recommended to have the model development data as complete as possible. All means of data collection should be exhausted prior to building the North Canal model, including possible surveying efforts. It would be best to not have to interpolate or make educated guesses on the input data as was done on some sections of the South Canal. Calibrating a model to sections of unknown elevation or geometry data proved to be frustrating and less satisfying. Additional effort could also be put into completing the missing data of the South Canal, such as the check structure elevations in Table 5. This additional effort may improve the South Canal model.
2. <u>Collecting More Accurate Check Depth Data</u>: The methods of taking check structure upstream and downstream depths used during the 2005 irrigation season monitoring can be improved upon. Rather than taking the relative upstream measurement from water surface to top of structure concrete at a chamber divider where backwater effects are present, it would be better to take the measurement at the very edge of the canal, perhaps in front of the automatic weir. On the edge of the

canal there are no visible backwater effects and the water is quite tranquil allowing for a more accurate relative measurement. The same measurement should be employed on the downstream side of the check structure. In 2005 a direct downstream depth measurement was taken in the middle of the canal where it is most turbulent. A more accurate measurement could be obtained by taking a relative water surface to top of concrete measurement at the edge of the canal. The water at the edge of the canal on the downstream side is more tranquil than the middle of the canal, but still more turbulent than the upstream side. Note that taking relative measurements on the downstream side may require different depth conversions at certain check structures that have drops, such as the Beals Check.

3. <u>Collecting More Accurate Flow Measurements:</u> During the 2005 monitoring season the 0.6-depth method was employed at the Beals Check flow measurements with the Flo-Mate device. This method is only suggested and accurate for depths less than two feet. Depths at the Beals Check measurement location were consistently greater than two feet and the two-point method should have been employed. The two-point method is more time consuming, but leads to more accurate results. It is recommended that future hand measured flows at all sites be conducted using the two-point method.

4. <u>Modeling Primed Laterals/Turnouts</u>: As discussed previously, primed laterals/turnouts are important in assessing system outflows. It is time-

consuming to balance observed lateral/turnout head gate structure settings with actual measured outflows or water orders by artificially controlling and adjusting the modeled head gate settings. Additional effort should be made in modeling the lateral and turnout systems off the main canal, especially those that are used the majority of the irrigation season and those that convey significant amounts of water from the main canal. These additional modeling efforts are recommended for the South Canal in the next phase of the project. A temporary solution for addressing primed laterals/turnouts may be to calibrate each model head gate by adjusting its discharge coefficient to obtain a more accurate match between stem height and discharge.

5. <u>Collection of Johnson Lateral Return Flows</u>: The Johnson Lateral return flows are a significant issue in calculating the South Canal Reach 1 water balance as discussed previously. 2006 monitoring data shows that the returns are as high as 12 cfs with an average of 5 cfs. Continuous data should be collected at the Johnson Lateral return box just before it enters the South Canal. The return box has a weir in it that can be used to calculate flow from continuous stage data. This continuous flow data could then become a time series input into the hydraulic model and will provide for a more complete water balance assessment. Similar data collection efforts should be made high priority on the North Canal if necessary.

6. Continuous Data at Each Check Structure: Especially in efforts towards calibration and validation of a hydraulic model, it would be beneficial to have continuous data collected at each check structure along the reach being assessed. One advantage to having continuous data at each check is that it would provide a graphical indication of the time any structure settings were changed either at the check structure or at the nearby upstream head gates. Also, the continuous stage data could be converted to approximate continuous flow data through the use of standard orifice and weir equations and calibrated discharge coefficients. This would provide a more detailed assessment of the water balance between points on the canal, which would be useful in quantifying and correctly placing system outflows. Also, having continuous data at the beginning and end of a reach is a must for calibration purposes, as discussed for the Beals Check structure.

7. South Canal Vale Flume to Wasteway Model Calibration: If found to be valuable by the BFID manager or personnel, effort should be put into extensive monitoring and data collection to complete the calibration of the South Canal model from the Vale Flume to the Wasteway, approximately miles 26.4 to 44. As discussed previously, the South Canal model was developed for the entire length of the canal, but was only calibrated and validated from the Dam to the Vale Flume. A complete calibration will improve the model's usefulness as a tool.

8. Ditch Rider Communication: A model cannot be properly calibrated, validated, or implemented if the system operation dynamics cannot be assessed. It is of utmost importance to be on the same page with the ditch riders, the operators who are constantly making changes to the system. For the South Canal model to be implemented and used in the future it will be important to have some form of documented system structure conditions. The information needed from the ditch riders is when a check or head gate structure was adjusted and how much it was adjusted. This will be necessary for keeping continuous model simulations up to date and useful. The same ditch rider interaction and information is essential in providing the district with a well calibrated and validated North Canal model.

Conclusions

This research focused on improving the operational efficiency of the BFID through the development and implementation of a hydraulic computer model for the South Canal using EPA SWMM 5.0. The model was developed over the entire 44 miles of the South Canal from the Dam to the Wasteway. Field monitoring was conducted during the 2005 irrigation season and the collected data was used to calibrate the first 26.4 miles of the South Canal model, from the Dam to the Vale Flume. The model was also validated using data from monitoring periods in addition to the calibration monitoring periods. A sensitivity analysis was conducted on the model to assess its sensitivity to the calibrated parameters and identify the crucial parameters and important trends in parameter adjustments effects on model results. Additional data were collected during the 2006 irrigation season and used to validate the model. The 2006 validation employed fewer assumptions, mainly Johnson Lateral return flows were collected, and times of Dam releases and structure setting changes were known from real-time data and thorough ditch rider interaction. The model produces simulation results that do not over or under predict and present no skew to the positive or negative. The simulated depths during 2005 calibration/validation were $\pm$ 10% of the observed depths 94% of the simulations and $\pm$ 5% for 77% of the simulations. The simulated depths during 2006 validation were $\pm$ 10% of the observed depths 94% of the simulations and $\pm$ 5% for 58% of the simulations. The model is fully capable of simulating the entire BFID irrigation system and all of its structural components, including automated check structure gates. The model will provide the district with a useful tool and reference that will aid them in making decisions concerning system operation and structure adjustment. The improved operational efficiency of the BFID will reduce nonused irrigation return flows and reduce the TSS loads entering the Belle Fourche River system.

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VITA

Curtis David Schoenfelder grew up in Piedmont, South Dakota until age 13, then moved to Huron, South Dakota. He graduated from Huron High School and attended the University of Minnesota in the Twin Cities where he competed for the Golden Gopher Track and Field team. He obtained his Bachelor's of Civil Engineering degree with a focus on water resources in May, 2005 and became a registered Engineer In Training (EIT). Curt's work experience as an undergrad includes four summers with South Dakota Department of Transportation and semesters working in the University of Minnesota Structures Laboratory and St. Anthony Falls Laboratory. Curt then attended South Dakota School of Mines and Technology in Rapid City, South Dakota and obtained a Master's of Science in Civil Engineering degree with focus on water resources in August, 2006. Curt then moved to Bellingham, Washington where he worked for a private civil engineering consulting firm.

APPENDIX F

OPERATIONAL MODEL OF THE BELLE FOURCHE IRRIGATION DISTRICT, SOUTH DAKOTA

Operational Model of the Belle Fourche Irrigation District, South Dakota

by

Timothy Jon Olson

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

2006

Prepared by:

____

Timothy J. Olson, Degree Candidate

Approved by:

____

Dr. Scott Kenner, Major Professor

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Dr. Scott Kenner, Chair, Department of Civil Engineering

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Dr. Kenneth N. Han, Dean, Graduate Division

ABSTRACT

This research examines the operational characteristics of the Belle Fourche Irrigation District (BFID) and recommends future practices to improve both the delivery and application efficiency of the irrigation system. The operational model is developed to work directly with a hydraulic model of the irrigation system.

The Belle Fourche River supplies the Belle Fourche Reservoir, which in turn, provides water to the 54,500 irrigable acres within the District. The Belle Fourche River currently does not meet the total suspend solids (TSS) water-quality criteria for warm-water permanent fish life propagation due to natural bank sloughing, riparian habitat impairment, and nonused irrigation water discharged into natural waterways. One segment of the scheduled best management practices (BMPs) would improve the delivery and application systems and reduce the TSS concentration by 37 percent (of the total concentration) by reducing return flows into the Belle Fourche River by 12,000 acre-feet per year by 2015.

Several goals for water conservation were identified during 2004 and spring 2005. The goals included: (1) increase of flow-measurement capabilities on canals and laterals through the utilization of data loggers and monitoring data, (2) reduction of the variability of stage and discharge in canals by installation of automated gate operators at check structures, and (3) automation of the daily water-ordering process and simulate the flow conditions with hydraulic and operational computer models. During summer 2005, logger and monitoring data were collected to calibrate a hydraulic model. The operational model was created to define the daily operating process in the BFID and create a set of operational guidelines in order to improve the efficiency.

This report contains several elements which work in conjunction to create an operational model. Upgrades to the water ordering system were made to convert water-ordering information to a digital format in an operational database. Major flow-measuring structures in particular Parshall flumes, were examined for submergence. Delivery lag time was defined and the methods for calculating were described. A check structure rating curve was created for the Beals check structure and recommendations for data collection in 2006 were made to complete rating curves for additional check structures on the South Canal. Recommendations were made for check structure operation and locations of future check structure automation, further upgrades to the water card operational database, continued hydraulic modeling of the North and South Canals, and incorporation of personnel support.

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After receiving my bachelor of science degree in civil engineering at the School of Mines and Technology, I was presented with an opportunity to continue my education and receive a master of science degree. I would like to thank Dr. Dan Hoyer and Dr. Scott Kenner for presenting the project to me and encouraging me to stay in school and get my master's degree. I would also like to thank my other committee members, Dr. Thomas Fontaine and Dr. Arden Davis, for continued support.

I am pleased with my education at the School of Mines, especially in regard to the faculty members listed above. My knowledge in water resources is a function of their expertise in the field and ability to communicate the subject matter. All have been an inspiration over the past 6 years and I thank each for his motivation.

I had the pleasure of working on this project for two years with the help of two field partners. Thank you to Ms. Cheryl Rolland and Mr. Curt Schoenfelder for assistance in field work that undoubtedly supported the completion of this project. I would like to thank the Belle Fourche River Watershed Partnership for giving us the opportunity to work on this project because without their funding and cooperation, the project would not exist. I cannot thank the Belle Fourche Irrigation District enough, especially Mr. Clint Pitts, for supporting our research and turning our ideas into reality.

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1.0 INTRODUCTION

1.1 PROJECT BACKGROUND

The Belle Fourche River extends across west-central South Dakota upstream into the northeastern corner of Wyoming, with a small part in the extreme southeastern corner of Montana. The total area of the watershed is roughly 5 million acres; 2.1 million acres are located in South Dakota. Land use in South Dakota consists primarily of livestock grazing, some cropland, and few urban and suburban areas (Belle Fourche River Watershed Partnership, 2005). Figure 1-1 displays the geographic location of the Belle Fourche Watershed and the location of the Belle Fourche Irrigation District (BFID) within the Watershed.

The Belle Fourche Irrigation District maintains and operates irrigation facilities for the U.S. Bureau of Reclamation (BR) and contains roughly 54,500 irrigable acres. The North and South Canals are supplied by the Belle Fourche Reservoir, which is off-stream storage of diverted Belle Fourche River waters. Water is diverted from the Belle Fourche River by the Inlet Canal, which is also used for irrigation operations. The 94 miles of major canals distribute water to 450 miles of lateral systems (Hoyer, 2003). The District is drained by 255 miles of open drains and 7 miles of piped drains. The Johnson Lateral discharges water from the Inlet Canal to 2,900 additional irrigable acres. The design capacity for the North Canal is 600 cubic feet per second (cfs) and 300 cfs for the South Canal at the dam.

Approximately 65 percent of crop production in the BFID is alfalfa and hay. Small grains and corn account for the remaining crops. Also, some livestock and dairy production exists. The type of crop varies with soil type across the District, with better

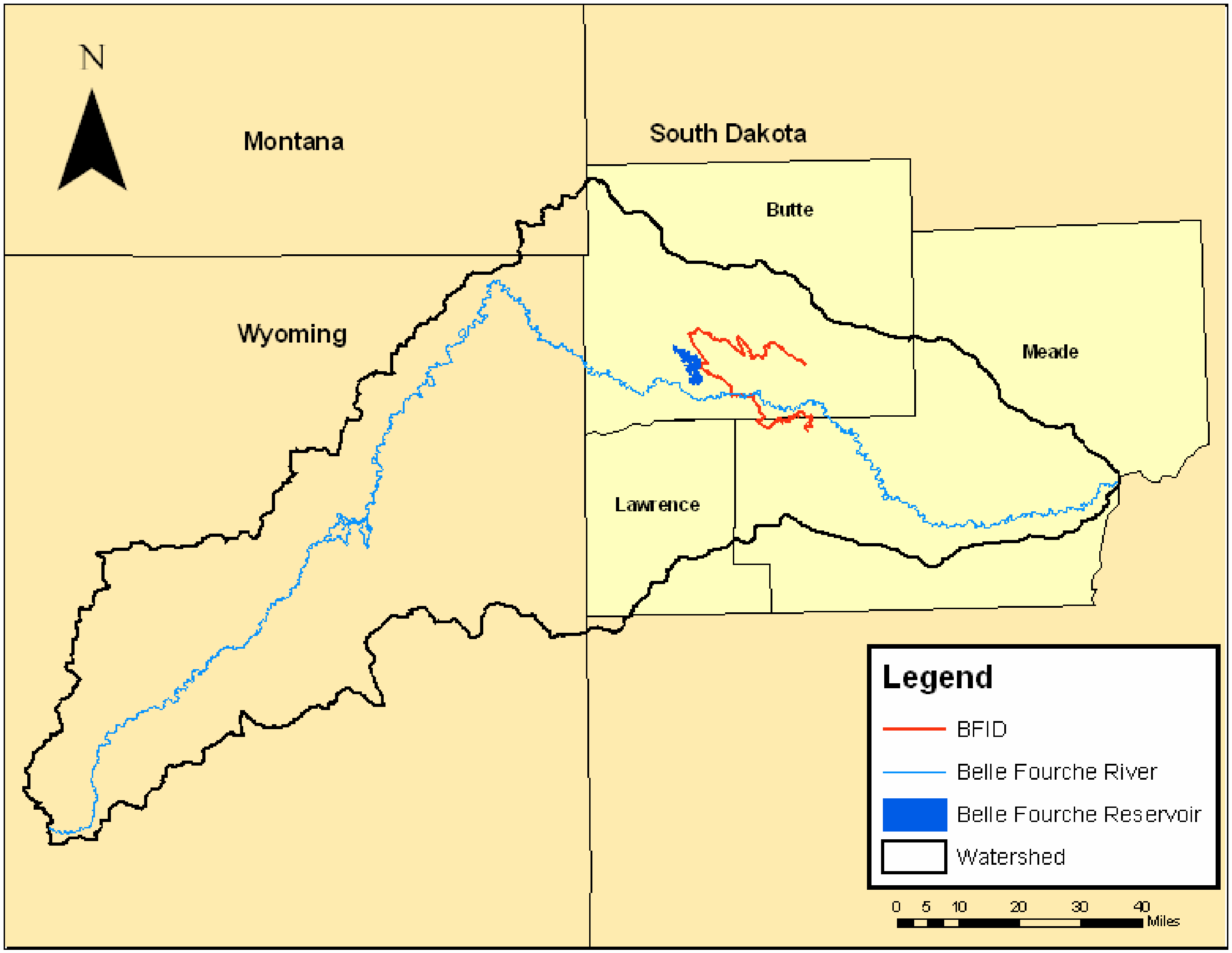


Figure 1-1. Belle Fourche River Watershed and Location of Belle Fourche Irrigation District.

production trends in the South Canal area. The soils range from heavy clays with some silts and gravels in the North Canal area and clay/sand soils in the South Canal area. Table 1-1 (Rolland, 2005) is a summary of the STATSGO map unit identifications in the BFID and the total composition of each component in the soil. Also shown is a rough estimation of the percent of each map unit within the BFID.

Table 1-1. Major STATSGO Map Units and Component Percentage (Rolland, 2005)

Map Unit	% in BFID	Clay	Silt	Sand	Gravel
SD019	29%	67%	19%		14%
SD021	34%	100%			
SD062	37%	50%	12%	38%	

1.2 WATER QUALITY

The Belle Fourche River was listed as impaired on the *1998 South Dakota 303(d) Waterbody List* (Myers, 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) due to elevated total suspended solids (TSS) concentrations (Belle Fourche River Watershed Partnership, 2005). The total maximum daily load (TMDL) report (Hoyer, 2003) for TSS identified natural bank sloughing, riparian habitat impairment, and nonused irrigation water discharged into natural waterways to be the primary contributors of TSS.

There are five water-quality monitoring reaches on the Belle Fourche River in South Dakota, one of which is listed on the 2006 303(d) list. The reach listed is from the Wyoming state line to near Fruitdale, South Dakota, and is in violation of fecal coliform and TSS criteria. Four other sites on the Belle Fourche River are listed on the

Department of Environment and Natural Resources (DENR) list as nonsupporting of TSS and are:

1. Near Fruitdale to Whitewood Creek
2. Whitewood Creek to Willow Creek
3. Willow Creek to Alkali Creek
4. Alkali Creek to mouth.

The major sources of the fecal coliform violation include livestock grazing, especially in riparian zones. The major sources of the TSS violation are rangeland grazing in riparian zones and managed pastures, crop production, and natural sources.

The remainder of this paper focuses on the BFID as a major contributor of TSS to the Belle Fourche River. Several irrigation best management practices (BMP) were recommended by the Ten-Year Implementation Plan (Hoyer and Schwickerath, 2005), including improved efficiency for delivery and application of irrigation waters and riparian vegetation improvements, and the BMPs were further defined in Segment III of the Belle Fourche River Watershed Management and Project Implementation Plan (Belle Fourche River Watershed Partnership, 2005). Segment III of the Project Implementation Plan described the current work implemented as of September 2005. The recommended BMPs for upstream and downstream of the Belle Fourche Reservoir are shown in Table 1-2 and Table 1-3, respectively. The tables display the BMP, total number of applications to be implemented, cost per BMP, total cost, and savings in both water and TSS. Table 1-4 was taken from the Implementation Plan and describes the current amount of the listed BMP implemented and the amount scheduled for the remainder of the 10-year plan. Improvements in the operational efficiency of the BFID are necessary to reduce the TSS concentration in the Belle Fourche River.

Table 1-2. Belle Fourche River Watershed Best Management Practices Upstream of the Belle Fourche Reservoir (Hoyer and Schwickerath, 2005)

TSS BMPs	Number	Cost/# ($)	Total ($)	Savings	
				Ac-ft	TSS mg/L
Delivery					
Flow Automation Projects (Gates)	2	10,000	20,000	70	2.9
Upgraded Water Card and Water Order System (System)	1	120,000	120,000	50	2.1
Portable Stage/Flow-Measuring Devices (Portable Devices)	3	3,750	11,250	45	1.9
Real-Time Stage/Flow-Measuring Devices Installed (Real-Time Devices)	3	15,000	45,000	195	8.1
Digital Map (Map, for below Diversion as well)	1	165,700	165,700	50	2.1
Alternative Keyhole Water Delivery Study (Delivery Study)	1	50,400	50,400	0	0.0
Alternative Keyhole Water Supply Method (Holding Pond)	1	2,000,000	2,000,000	2,000	82.7
Nonused Water Storage Pond (Ponds)	1	250,000	250,000	1,000	41.4
Inlet Canal Lining (Feet of Inlet Lining)	10,560	50–150	525,000 – 1,600,000	1,500	62.0
Application					
Pipeline Projects Delivering Water From BFID to Fields (Pipelines for Sprinklers)	500	200	100,000	50	2.1
Irrigation Sprinkler Systems (Sprinkler Systems)	2	60,000	120,000	150	6.2
Scheduling of Irrigation Water (Scheduling Plan)				20	0.8
Reuse of Tail Water (ac-ft of Water Reused)				30	12
Riparian Vegetation Improvements					
Grazing Management System (Grazing Management Acres) Including:	14,750	45	120,000		36
Cross Fencing (Feet of Fence)	237,500				
Off-Stream Water (Off-Stream Water Systems)	45				
Pipelines (Miles of Pipelines to Off-Stream Water Systems)	12				

Table 1-3. Belle Fourche River Watershed Best Management Practices Downstream of the Belle Fourche Reservoir (Hoyer and Schwickerath, 2005)

TSS BMPs	Number	Cost/# ($)	Total ($)	Savings	
				Ac-ft	TSS mg/L
Delivery					
Flow Automation Projects (Gates)	2	10,000	20,000	70	2.9
Upgraded Water Card and Water Order System (System)	1	120,000	120,000	50	2.1
Portable Stage/Flow-Measuring Devices (Portable Devices)	3	3,750	11,250	45	1.9
Real-Time Stage/Flow-Measuring Devices Installed (Real-Time Devices)	3	15,000	45,000	195	8.1
Digital Map (Map, for Below Diversion as Well)	1	165,700	165,700	50	2.1
Alternative Keyhole Water Delivery Study (Delivery Study)	1	50,400	50,400	0	0.0
Alternative Keyhole Water Supply Method (Holding Pond)	1	2,000,000	2,000,000	2,000	82.7
Nonused Water Storage Pond (Ponds)	1	250,000	250,000	1,000	41.4
Inlet Canal Lining (Feet of Inlet Lining)	10,560	50–150	525,000 – 1,600,000	1,500	62.0
Application					
Pipeline Projects Delivering Water From BFID to Fields (Pipelines for Sprinklers)	500	200	100,000	50	2.1
Irrigation Sprinkler Systems (Sprinkler Systems)	2	60,000	120,000	150	6.2
Scheduling of Irrigation Water (Scheduling Plan)				20	0.8
Reuse of Tail Water (ac-ft of Water Reused)				30	12
Riparian Vegetation Improvements					
Grazing Management System (Grazing Management Acres) Including:	14,750	45	120,000		36
Cross Fencing (Feet of Fence)	237,500				
Off-Stream Water (Off-Stream Water Systems)	45				
Pipelines (Miles of Pipelines to Off-Stream Water Systems)	12				

Table 1-4. Best Management Practices Installed and Scheduled as of September 2005 (Belle Fourche River Watershed Partnership, 2005)

Best Management Plan	Amount Implemented	Amount Scheduled From 10-Year Plan
Flow Automation Units	17	42
Upgraded Water Card and Water Order System	Phase I	Three Phases
Portable Stage/Flow-Measuring Devices	6	15
Real-Time Stage Flow-Measuring Devices	9	15
Canal and Lateral Operational Models	1	5
Line Open Canals and Laterals (feet of lining)	3,200	26,560
Replace Open Canals and Laterals With Pipeline (feet of pipeline)	4,000	25,000
Nonused Water Storage Ponds	0	2
Alternative Irrigation Water System for Johnson Lateral	0	1
Sprinkler Irrigation Systems	4	36
Managed Riparian Grazing	15,000	34,000
Public Meetings	12	40
Project Tours	2	8

1.3 STRUCTURAL CHARACTERISTICS

Two major canal systems, the North and South Canals, and several hundred lateral and farmer turnout systems distribute the water from the Reservoir to fields (Figure 1-2). Each canal is controlled by a series of level pool check (control) structures. There are 26 check structures on the North Canal and 33 check structures on the South Canal. The checks were designed to control the water surface elevation upstream to produce the necessary delivery head at each delivery structure. Water is distributed

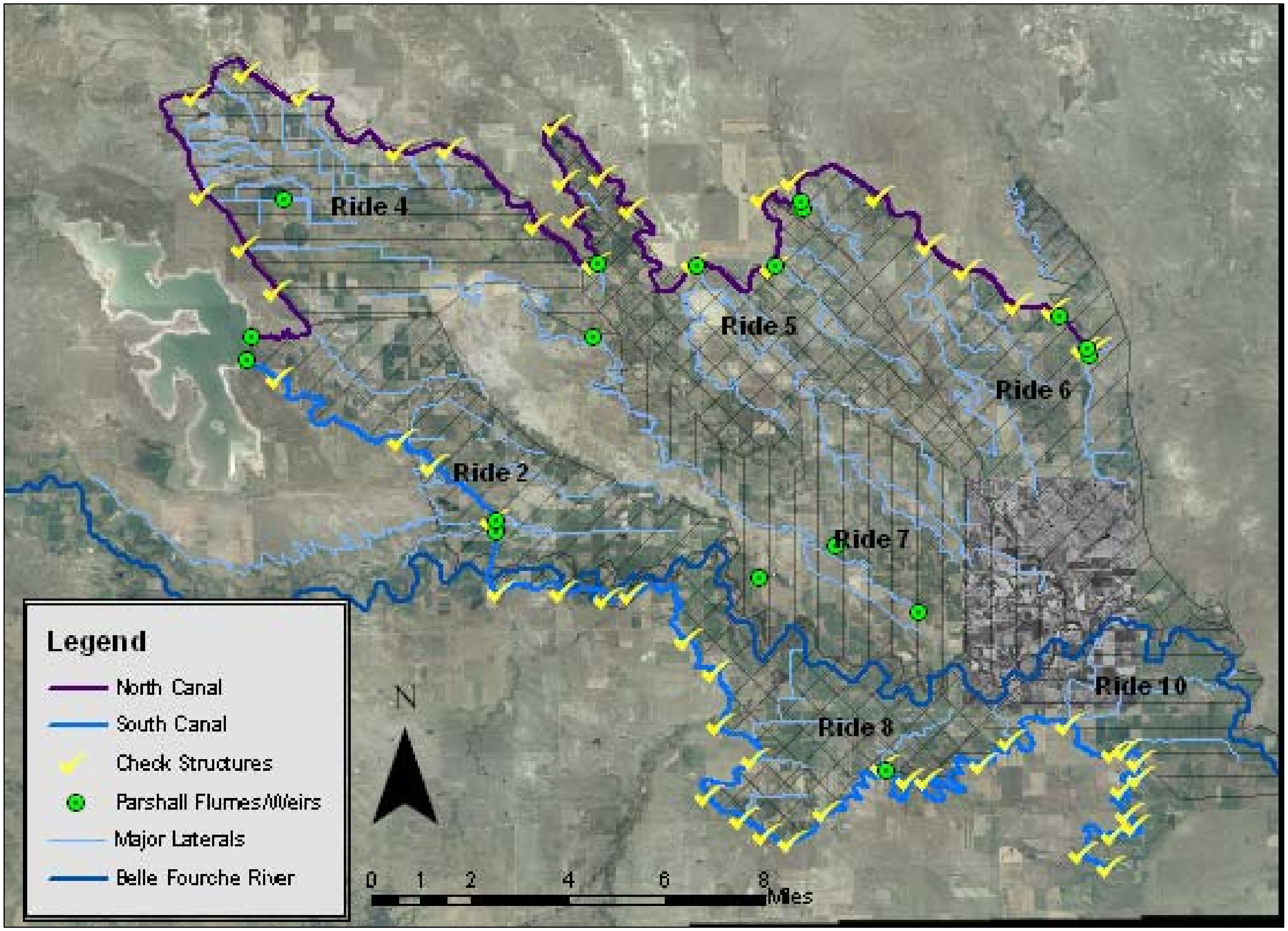


Figure 1-2. Map of North and South Canals, Belle Fourche Irrigation District.

from the North and South Canals into farmer turnouts and laterals. Farmer turnouts have head gates directly on the canal or within a lateral system and generally distribute water to a single farmer. Lateral systems are more complex and distribute water to multiple farmers. The BFID is further broken into operating sections, or rides, that are a responsibility of the ditch rider.

Figure 1-2 is a map of the BFID including all of the check structures, major laterals, and major Parshall flumes and weirs. The BFID monitors discharge with Parshall flumes and sharp-crested weirs. The canal discharge is monitored at the dam just downstream of the outlet with Parshall flumes as well as throughout the canal systems. Other flumes exist to monitor the flow entering a lateral, generally located just downstream of the head gate. Sharp-crested weirs are also used downstream of head gates to monitor the flow entering the lateral or farmer turnout. Discharge within a lateral system is monitored by flumes and weirs or division boxes, where piped water enters a concrete box, which increases the delivery head to additional segments of the lateral and spills over a weir for flow measurement. Some pipeline systems have current meters installed in the pipe and a gage displays the discharge. In the case of farmer turnouts or laterals without a weir box just downstream of the head gate or a current meter in a pipeline system, ditch riders depend on feel and experience to produce the correct discharge. Ditch riders measure the gate stem height and relate it to a flow rate based on a simple understanding of the hydraulics and comparison to similar head gate structures with downstream control.

1.4 SYSTEM OPERATION

The operation of the BFID is governed by a series of dependent components, including both human and nonhuman. The human components interact in a specific order and the nonhuman component links them. There are three components to the demand/delivery system: water call cards, Water Master sheets, and billing cards. The demand/delivery system was converted to a digital format to eliminate mathematical mistakes, expedite the ordering/billing process, and create an electronic database. The water call cards are the link between the farmers and ditch riders. Water orders are completed by lateral or farmer turnout, including any additional water needed for proper delivery and system operation. The Water Master sheets include the total orders according to the water call cards and are used to determine daily changes at the dam. The billing cards document the amount of water allocated and the total water delivered to each farmer in the system. The annual allotment is the total available water distributed to each farmer in acre-inches based on the usable capacity of the Belle Fourche Reservoir. Often, the billing card is less than the water call card due to operational losses, precipitation, canal fluctuations, or reservoir capacity.

There are several processes in the BFID that occur daily to operate the system smoothly, including a combination of human interaction and information transfer. The daily process, beginning with water orders and ending with delivery of water to farmers, is defined by the transfer of information among the components. The interactions and daily processes are:

1. <u>Farmer/Ditch Rider:</u> The process of ordering water begins with the farmer. If water is needed for irrigation, he or she must contact the ditch rider with ample time for the water to arrive. In other words, a farmer 6 miles away from the Reservoir can expect water sooner than a farmer 30 miles away. Travel time is a major component of the water-ordering system. Once orders are given by farmers, the ditch rider fills out the water call card by hand and presents it to the District office.

2. Ditch Rider/Data Entry: Each morning, the ditch rider presents the water call card to the data entry person, who is then responsible for entering the water orders into the database. The data spreadsheet automatically calculates the demand per ride and the Water Master sheet is created. Any differences between the ditch rider total orders and the data spreadsheet are corrected.

3. Data Entry/Water Master: The data entry person presents the Water Master with the total orders, which have been automatically converted to the Water Master sheet. The Water Master is responsible for making the necessary changes at the dam to satisfy the orders.

4. Data Entry/Ditch Rider: A simple water budget is created using the water call cards to produce a check structure demand schedule. The irrigation demands at each check can be used to make decisions about system operation.

5. Ditch Rider/Farmer: The ditch rider releases water into farmer turnout and lateral systems when available. The farmer makes changes to water orders, if necessary, and the process repeats.

1.5 EFFICIENCY IN THE BELLE FOURCHE IRRIGATION DISTRICT

Nonused water is related to inefficient delivery and application of irrigation water in the Belle Fourche Irrigation District. Hoyer (2003) identified return flows in the BFID as a major contributor to TSS in Horse Creek and the Belle Fourche River.

According to Rolland (2005), efficiency in the BFID is calculated by dividing the total volume of water billed to farmers by the total volume of water released from the dam over the course of the season. Rolland concluded that the average efficiency over the last 5 years was 49 percent with an average loss of 63,200 acre-feet per year. Water billed to farmers often does not equal the water ordered from the dam. In fact, the 1998 Water Management Study prepared by the Bureau of Reclamation estimated that farmers receive 15 percent more water than billed (Belle Fourche Irrigation District, 1998). The study concluded that removing inaccuracies in billing would increase the 1998 BFID efficiency from 55 to 63 percent. The extra 15 percent is accounted for by increases in head pressure at laterals or farmer turnouts due to unaccounted for waves of water, illegal irrigation, or inaccuracies in flow measurement structures (Rolland, 2005).

In the BFID, approximately 64 percent of the water released from the Reservoir is delivered to the field and 32 percent is used by crops. The remaining water is lost to evaporation and nonused water discharged into adjacent waterways (Belle Fourche River Watershed Partnership, 2005). Losses can be further broken down into two categories: operational and transportation losses. Operational loss is the additional water needed to keep a pipeline primed, create enough head pressure to deliver a quantity of water, or any extra water needed to operate a structure correctly. Transportation loss is the additional water needed to account for seepage or evaporation in an open channel section. These losses are recorded on the water call card and are included in the total water ordered from the dam for the North and South Canals.

To improve the delivery and application efficiency of the BFID, automated stage control devices were installed on check (control) structures, canal and lateral systems were lined and piped, pivot sprinkler systems were installed, and hydraulic and operational models are being produced.

Parshall flumes on the South Canal are located at the South Canal Dam (SCD) Flume, Belle Fourche River Siphon (BFRS) Flume, and Vale Flume and are used to monitor the inflows and outflows to each ride. Each flume has an upstream and downstream stilling well, and a staff gage is located in the upstream well to convert the stage into discharge using the Bureau of Reclamation *Water Measurement Manual* (U.S. Bureau of Reclamation, 2001). It was hypothesized that the flumes on the South Canal (and other areas of the BFID) were operating under submerged conditions and, therefore, producing errors in the discharge readings provided by the upstream staff gage. A hydraulic analysis of the flumes was performed during the 2005 irrigation season. Stage-measuring devices were installed in the upstream and downstream stilling wells to calculate submergence, and it was found that several of the flumes on the project operate under submerged conditions for more than half of the irrigation season.

1.6 OBJECTIVES

The overall goal of the *Belle Fourche River Watershed Management and Project Implementation Plan* (Belle Fourche River Watershed Partnership, 2005) is to bring the river into compliance for TSS through the implementation of recommended BMPs. One segment of BMPs includes reducing nonused water discharged to local waterways from the delivery and application systems of the BFID where approximately 37 percent of the overall TSS reduction will be achieved. The objective of this study is to evaluate the structural and operational characteristics of the system and to develop and recommend system improvements that will result in increased system efficiency.

1.7 SCOPE AND APPROACH

This research focuses on the South Canal of the BFID. Reach 1 (Ride 2: SCD Flume to BFRS Flume) of the South Canal is the simplest reach to operate because it receives water directly from the dam with no upstream demand. Reach 2 (Ride 8: BFRS Flume to Beals Check) and Reach 3 (Ride 8: Beals Check to Vale Flume) have an upstream and downstream demand (Figure 1-2). Many of the discrepancies in the water budget come from Ride 8 because it is the most complex to operate. If anomalies are seen in Ride 8, then Reach 4 (Ride 10: Vale Flume to end of the South Canal) will also have problems.

This research focuses on the operational model produced to improve the operational efficiency of the BFID. The details include water-ordering system upgrades, data entry into a spreadsheet to produce a demand-delivery product, and an operational model making use of two flow scenarios to produce a set of check structure settings. The operational model focuses on Reaches 2 and 3 with monitoring of the flow coming into the reach at the BFRS Flume, check structure operation in each reach, and a final check of the flows entering Reach 4 at the Vale Flume. The operational model will improve the delivery and application efficiency, and the proposed reduction in TSS of 1 milligram per liter (mg/L) will be achieved (Hoyer and Schwickerath, 2005).

2.0 CONTROL STRUCTURES AND FIELD MONITORING

2.1 TYPES AND DESCRIPTION OF AUTOMATED EQUIPMENT

According to Segment III of the Watershed Implementation Plan, 25 flow automation units were scheduled for installation during the 2005 and 2006 irrigation seasons to improve the irrigation delivery system (Belle Fourche River Watershed Partnership, 2005). During 2005, 16 check structures were automated with 9 real-time sites installed. Automated gates were equipped with data loggers, submersible pressure transducers, gate actuators, solar panels with solar regulators, and other miscellaneous equipment that works together to control the level pool upstream of the structure. In addition, the real-time sites were equipped with 30-foot towers, 900 MHz spread spectrum radios, and 6db-gain omni antennas, which together allow the District to access stage data at the station remotely. Figure 2-1 is a picture of the inside of a control box with the components labeled. The District office acts as the base station with a 60-foot tower, allowing personnel to make changes in gate position, detect mechanical problems, and check discharges at the major flow structures from a computer. A mobile base station also exists which allows the Water Master or District Manager to access the real-time sites from a vehicle.

The purpose of the automated gate is to maintain a constant pool level and to allow for consistent flow out of lateral gates. This helps control the variability of the head pressure around farmer turnouts and lateral head gates. Without constant control of the pool, a downstream water order can be consumed by open turnouts and head gates because an increase in head pressure results in an increase in flow to the lateral. The actuators allow for changes in discharge of about 50 cfs. Because the check structures see more than a 50-cfs change over the course of the season, human interaction is necessary to properly operate a check.

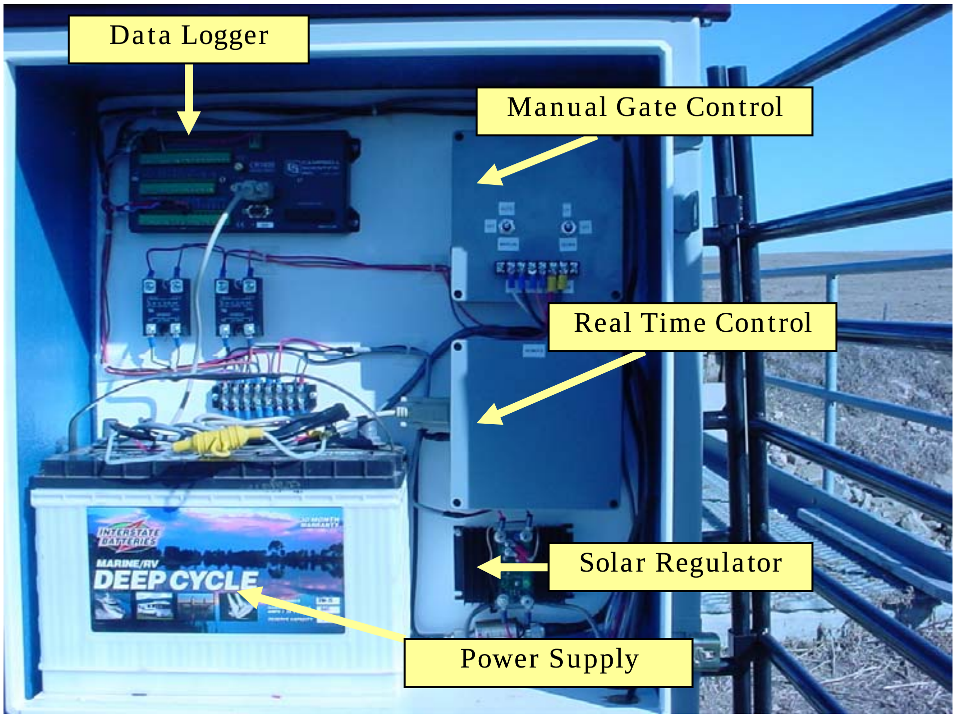


Figure 2-1. Inside of the Vale Check Control Box Including all Components of a Real-Time Site.

Each automated site falls into one of three categories: automated check structure, real-time site, or combination site. A single gate fluctuates based on changing discharge and, in turn, water level and the stage are recorded continuously at 10-minute intervals in the data logger. The real-time sites include both automated check structures and flumes. The setup for a real-time check structure is identical to an automated check structure but is equipped with a radio to access stage data remotely. Real-time flumes are equipped with level sensors in the upstream and downstream stilling wells and the stage data are available remotely. The combination sites are automated check structures with a flume directly downstream. Level sensors are located at the check structure and in both upstream and downstream stilling wells of the flume, and are connected to the same logger. Stage data are also available real-time at the check structure and flume.

A ditch rider is often responsible for eight or more check structures, in addition to complex lateral systems, on his ride. The manual check structures on the South Canal must be monitored at least daily to adjust the pool level for incoming orders. Farmers also expect the ditch riders to deliver water when needed or make sure a lateral is hydraulically prepared for delivery. With check structure automation, the ditch riders will have fewer structures to attend to and efficiency efforts can be focused elsewhere on the canal, at manual check structures, or on the lateral systems.

2.2 LOCATIONS OF AUTOMATION

Figure 2-2 is a map of the check structures and other upgraded sites including the type and name. A total of 22 automated and/or real-time sites were installed over the course of the 2005 irrigation season. One real-time site is on the A&C Lateral Flume, which is the top of Ride 7. This is a unique ride because the water is not ordered directly off the North Canal; instead, it receives water through another lateral (Townsite Lateral) with head works on the North Canal. The North Canal has nine total sites and the South Canal has eleven sites. One additional site was installed on the Johnson Lateral head works. The automated/real-time sites were positioned along the canals to minimize the distance between sites and to maximize the time-series data collected per mile of canal.

The automated check structures on the South Canal include the Sorenson, Anderson, Kiery, Whitewood Siphon, Cottonwood Siphon, Dunn, Perry, and Richards. The real-time sites include the SCD Flume and Beals Check. The Beals Check is unique in that

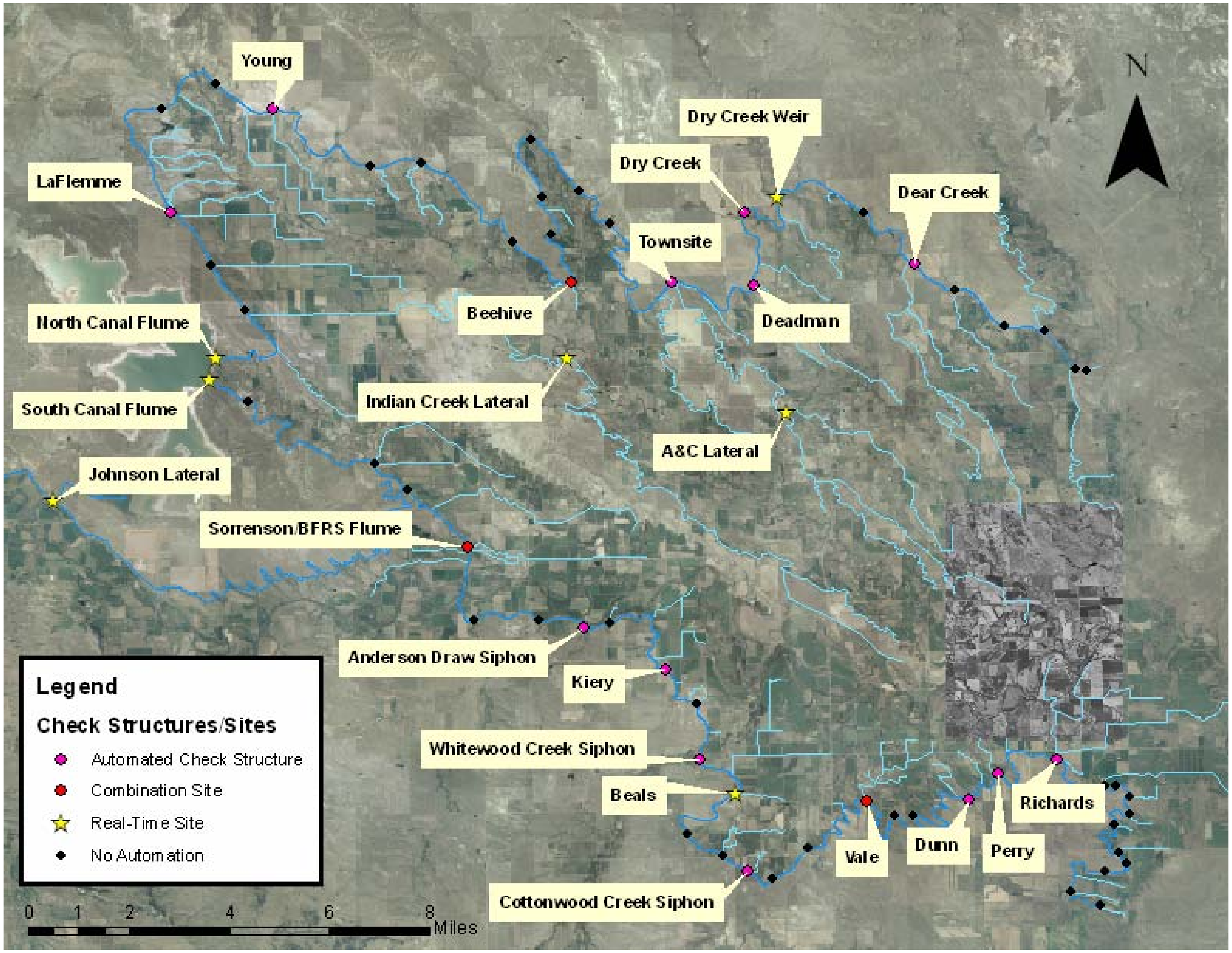


Figure 2-2. Check Structures and Other Sites With Upgrades in the Belle Fourche Irrigation District.

gate position on both the automated and manual gates, as well as upstream and downstream depths, are available real time. The combination site includes the Sorenson Check and BFRS Flume and Vale Check and Flume (Figure 2-3).

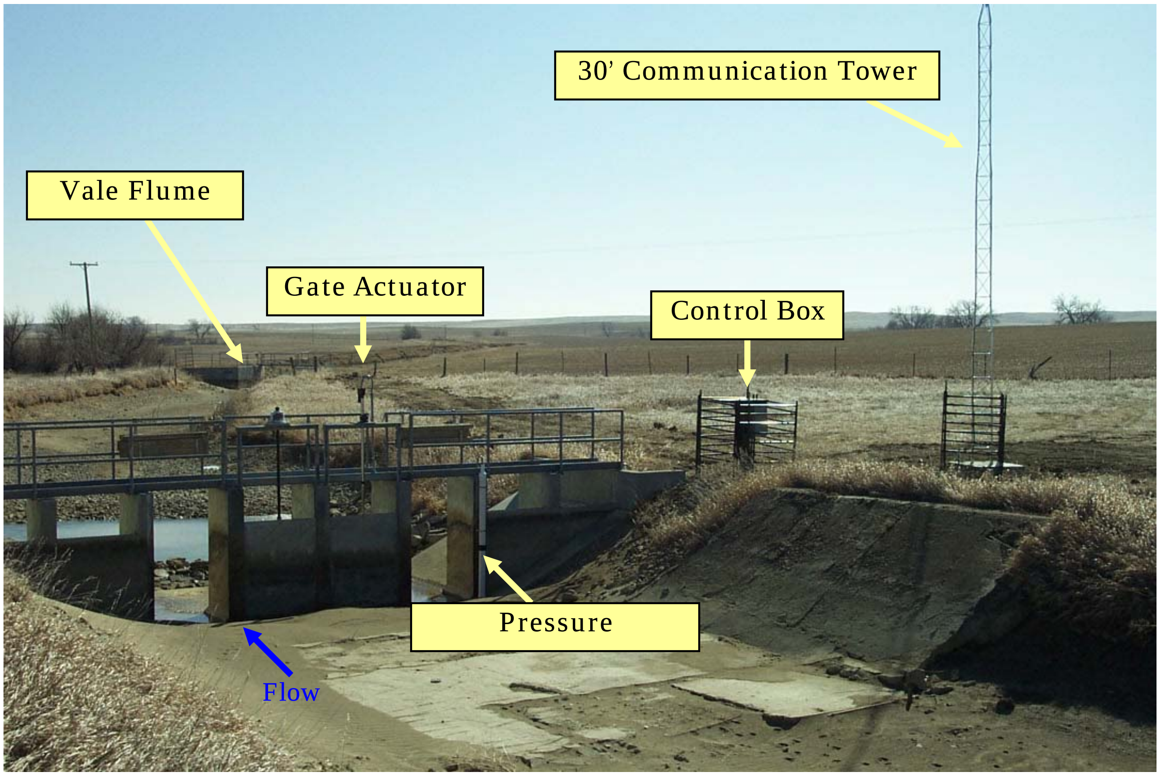


Figure 2-3. Vale Check and Flume Combination Site.

2.3 MOBILE LOGGERS

Nine pressure transducers were used as mobile units. The portable units are equipped with data loggers and controllers and submersible pressure transducers. The units were installed on each reach for an approximate period of 2 weeks starting with Reach 1. Subsequently, Reaches 2 and 3 were monitored for periods of 2 weeks at several locations along the monitoring reach to develop as much additional time-series data on the reach as possible. Once the monitoring of the reach was complete, the units were moved to the next reach and the process was repeated. The time-series data supported the additional field monitoring described below. Figure 2-4 is a picture of a portable unit with the field laptop connected at the 12.5 Check. The logger was placed at the 12.5 Lateral head gate 20 feet upstream of the check structure. The water is much less turbulent at the head gate, and if water surface measurements are collected to a set point on both the head gate and check, the relative depth can be determined at the check. This method was performed at several other locations along the canal. The data were downloaded from each logger at least once a week to avoid overwritten data.

Figure 2-4. Portable Data Logger and Laptop Installed at the 12.5 Lateral Head Gate.

Four mobile pressure transducers and loggers were installed in the downstream stilling wells of the North and South Canal Dam, BFRS Flume, and Vale Flume to collect information about flume submergence and to correct the data if needed. The transducers were installed in the downstream stilling wells roughly 1 month into the 2005 season and remained through the end of the season. The data showed that the BFRS and Vale Flumes on the South Canal submerge at high flows; at times, the submergence exceeds 90 percent and the standard correction factor for submergence is inaccurate, according to the Bureau of Reclamation *Water Measurement Manual* (U.S. Bureau of Reclamation, 2001). The flows measured at the South Canal Flumes during the 2005 irrigation season were corrected to ± 10 percent during unsubmerged, or 65 percent to 80 percent submerged conditions; ± 10 percent when the correction factor is used; and ± 20 percent for conditions beyond 90-percent submergence. Table 2-1 is a summary of the submergence seen in the 2005 irrigation season. The SCD Flume was omitted because it was not submerged during the 2005 season. Figure 2-5 is the continuous submergence data for the Vale Flume as an example of the distribution of values over the course of the season. The red line represents the 80-percent limit when the submergence correction factor must be used. Submergence in the BFID Parshall Flumes tends to be less during high flows.

Table 2-1. Submergence Summary for the Belle Fourche River Siphon and Vale Flumes for the 2005 Irrigation Season

Location	>80% Submerged	>90% Submerged
BFRS Flume	43.79%	14.60%
Vale Flume	99.72%	5.08%

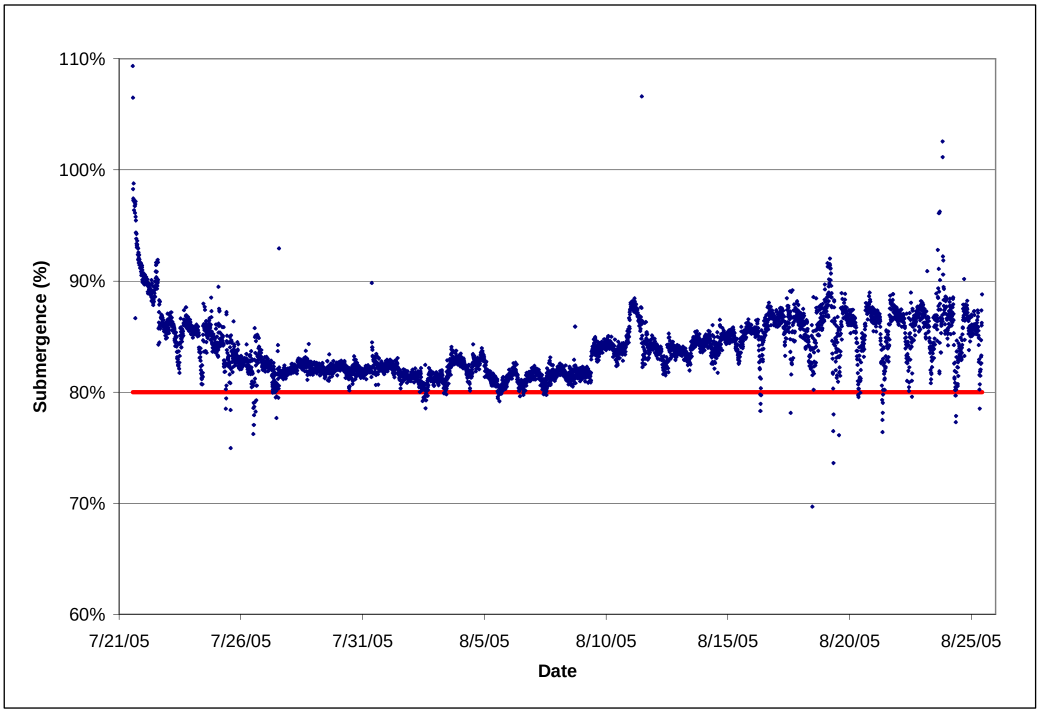


Figure 2-5. Vale Flume Submergence for the 2005 Irrigation Season.

2.4 FIELD MONITORING

In addition to the mobile transducers, Reaches 1, 2, and 3 of the South Canal were individually monitored over the course of the 2005 irrigation season. Gate position and water surface to a set point on the concrete structure were measured at each lateral and farmer turnout head gate as well as water surface measurements at each culvert and bridge along the reach. The check structures along the reach were measured, including upstream and downstream depths, gate and weir positions, and head over the automatic weirs (when available). Figure 2-6 is an example of a water surface to the top-of-concrete measurement that was later used to reference the depth recorded by the pressure transducer installed at the site. The field monitoring data were used to develop stage discharge rating equations, converting stage measurements to discharge.

The flow and stage data were also used to calibrate the hydraulic model by running the model with the observed field conditions and checking flow rates at the flumes along the canal. All field monitoring data from the 2005 season is found in Appendix A.

Figure 2-6. Field Monitoring Measurement at a Lateral Head Gate.

The Beals Check is an important site because the automation is complex. Its location along the South Canal is also important because it is located roughly half way between the BFRS Flume and the Vale Flume and was chosen as an additional discharge measuring site. Discharge measurements were taken using a Marsh-MacBirney FloMate roughly 200 feet downstream of the structure 16 times during the season. A rating curve was prepared relating the upstream head and discharge to the gate and weir positioning.

2.5 DATA COMPILATION AND ANALYSIS

A database of field monitoring and mobile logger data was created for the 2005 irrigation season and is located in Appendix A. An initial survey of the system was performed to collect information about the head gates, including pipe invert to a reference point, closed gate stem height, and pipe diameter. Check structure information was also collected, including the number of gates and weirs, upstream invert to a reference point on the structure, size of gate inlets, and closed gate stem height. Field monitoring data, including all information described above, was entered into spreadsheet format by reach and date. The data from the initial survey were used in conjunction with field monitoring data to produce percent openness of head and check gates, head above pipe invert and check weirs, height of check weirs, and upstream and downstream head at check structures. The data were essential to the calibration of the hydraulic model.

The second component of the data compilation was the mobile logger data. Many times, the pressure transducer was not set to a zero datum. The stage data were corrected using hand measurements of the water surface relative to a set point on the structure. A correction factor was applied to the logger data to produce the actual depth at each location. The sensitivity of the correction factor was especially significant at the Parshall flumes, where submergence factors affect the actual discharge. The corrected data were also used to calibrate the hydraulic model.

In addition to model calibration, the data collected in the field were used to calculate the delivery lag time between the Parshall flumes on the South Canal. The travel time between rides significantly affects the water-ordering and delivery process.

2.6 DELIVERY LAG TIME

Travel time, or delivery lag time, in the BFID is defined as the time it takes to deliver 90 percent of a water order to a head gate structure upon release from the dam. Understanding delivery lag time in the BFID is important for several reasons. The water-ordering system and expectation of delivery must take into consideration the delivery lag time. Farmers on Ride 2 can order water and expect it the same day it is released from the dam. Ordering from Ride 8 must be done earlier because the delivery lag time is 24 hours longer. Near the end of Ride 10, farmers must order water 4 days in advance to apply it to a field because of a 3-day lag time.

Proper check structure operation also depends on the delivery lag time. Several problems arise if a check structure is not set correctly to receive a new order. If the wave arrives at the check before it is properly adjusted, the pool elevation will increase and the head at the upstream laterals and farmer turnouts will increase. The farmer receives more water than ordered and the downstream ditch riders have less water available to deliver. If this happens at several locations along the South Canal, the orders for Ride 10 can be completely consumed upstream. If a check structure is adjusted before an expected increase in demand, the pool elevation can drop dramatically, causing the deliveries upstream to decrease. A significant amount of water and time is needed to reestablish the pool behind the check which, in turn, delays downstream deliveries.

Discharge measurements were collected at the SCD Flume, BFRS Flume, and Vale Flume and used to calculate an average delivery lag time over the course of the 2005 irrigation season. The average release from the dam into the South Canal from July 21, 2005, to September 8, 2005, was roughly 215 cfs. The average discharge at the BFRS Flume and Vale Flume was 167 cfs and 92 cfs, respectively, for the same time period.

The delivery lag time was further broken down into ranges of discharge because higher flows will travel faster than lower flows. Water card data, continuous stage measurements, and the hydraulic model were used to calculate lag time using three different methods.

2.6.1 Water Master Sheets

Water Master record sheets and water card data were used to make an initial prediction of travel time. Daily discharge data from July 2, 2005, to July 14, 2005, were gathered for the SCD Flume, BFRS Flume, and Vale Flume. The difference in discharge from the previous day was calculated so that an increase or decrease in the dam release could be correlated to an increase or decrease at the other structures. The peaks and valleys were plotted and analyzed and a rough estimate of travel time was estimated rounded to the nearest whole day. Figure 2-7 is a plot of the estimated delivery lag time from the SCD Flume to the BFRS Flume, and Figure 2-8 is the same plot from the SCD Flume to the Vale Flume. This method is limited because the discharge data are daily (i.e., one value per day), and travel time can only be estimated to the whole day or, at best, half day.

According to the Water Master sheets, the estimated time for a large change in discharge (greater than ± 30 cfs) to travel from the SCD Flume to the BFRS Flume is 1 day, while a smaller change (less than ± 30 cfs) requires between 1 and 2 days. The travel time from the SCD Flume to the Vale Flume is between 2 and 4 days, with a higher likelihood of 3 to 4 days. Another finding from this method was that the low flow scenarios consistently produced a longer delivery lag time than higher flows because lower flows were generally in the tail of the hydrograph where the discharge is lagged.

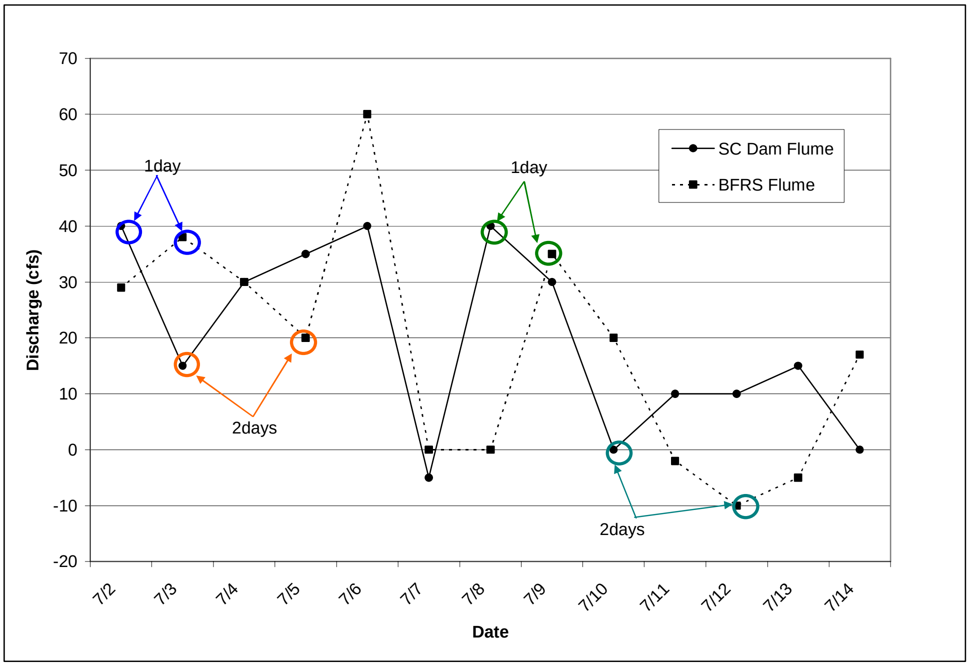


Figure 2-7. Changes in Discharge From the Previous Day Taken From the Water Master Sheets to Calculate Delivery Lag Time From the South Canal Dam Flume to the Belle Fourche River Siphon Flume.

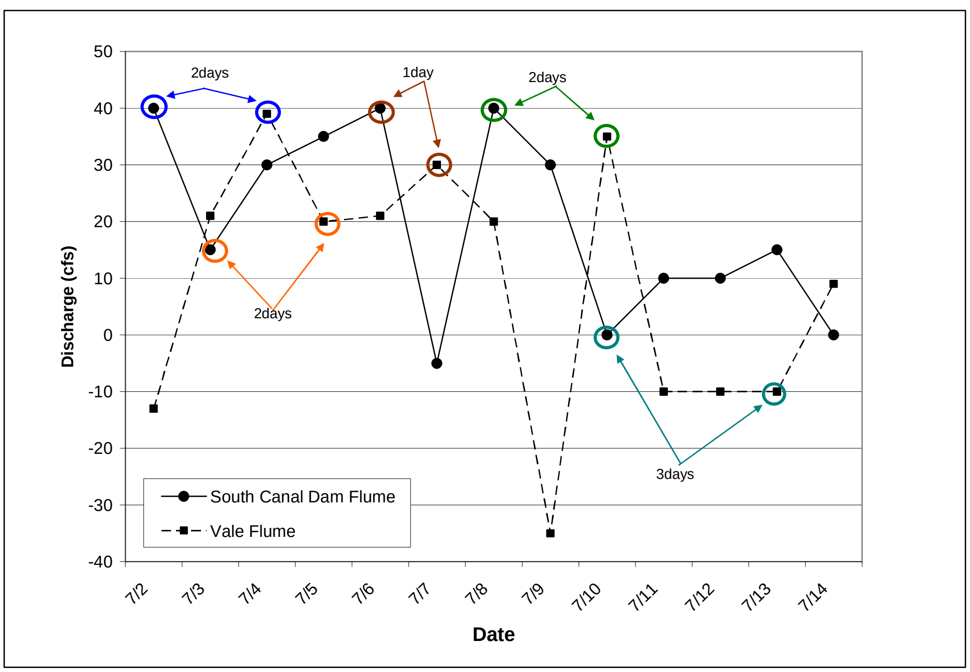


Figure 2-8. Changes in Discharge From the Previous Day Taken From the Water Master Sheets to Calculate Delivery Lag Time From the South Canal Dam Flume to the Vale Flume.

2.6.2 Continuous Discharge Data and Regression

Time-series stage data were collected at the SCD Flume, BFRS Flume, and Vale Flume from July 21, 2005, to September 8, 2005, and converted to discharge using the U.S. Bureau of Reclamation *Water Measurement Manual* (U.S. Bureau of Reclamation, 2001). The SCD Flume data were lagged at half-hour increments from 4 to 8 hours and compared to the discharge recorded at the BFRS Flume. The objective was to correlate the BFRS Flume discharge to the lagged SCD Flume discharge. The SCD Flume was plotted against the BFRS Flume and regression techniques were used to determine which lag had the largest R-square value. The lag associated with the largest R-square had the best correlation and was assumed to approximate mean lag time. This process was repeated for the Vale Flume at quarter-day increments from 3.5 to 5.0 days. The difference was the estimated delivery lag from the BFRS Flume to the Vale Flume, and lag time per linear mile of canal was estimated. Three regression techniques were used: (1) linear with an intercept, (2) linear with no intercept, and (3) second-order polynomial.

The first regression of the time-series data was linear, including an intercept. This technique produced a 5 ± 0.5-hour delivery lag time from the SC Dam Flume to the BFRS Flume with a 0.65-percent improvement in R-square over the no-lag R-square value. A 4.125 ± 0.125-day lag is estimated between the SCD Flume to the Vale Flume with a 27-percent improvement in R-square over the no-lag R-square.

A second linear technique was performed by forcing the regression through the origin. This technique produced a 5 ± 0.5-hour delivery lag time from the SCD Flume to the BFRS Flume with a 0.63-percent improvement in R-square over the no-lag R-square value. A 3.5 ± 0.25-day lag is estimated between the SCD Flume to the Vale Flume with an 11-percent improvement in R-square over the no-lag R-square value.

A third technique was performed using a second-order polynomial fit with the intercept set to zero. This technique produced a 5 ± 0.5-hour delivery lag time from the SCD Flume to the BFRS Flume with a 0.71-percent improvement in R-square over the no-lag R-square value. A 4.5 ± 0.25-day lag is estimated between the SCD Flume to the Vale Flume with a 35-percent improvement in R-square over the no-lag R-square value. The technique was stopped at a second-order polynomial because the x^2 coefficient was on the order of 10^{-4} and 10^{-3} for the BFRS and Vale Flumes, respectively, and decreased with an increasing order of polynomial. This indicates the x^2 and higher terms are insignificant and a linear model is sufficient.

Table 2-2 is a summary of the SCD Flume to BFRS Flume delivery lag times estimated and the corresponding R-square values, and Table 2-3 is the same for the Vale Flume. The delivery lag times displayed are the estimated values used in each trial and represent the cumulative lag time from the SCD Flume to the given structure. The results of the second-order polynomial test were not included because it did not explain any further error than the linear tests. Figure 2-9 displays discharges at the SCD Flume versus the BFRS Flume and Vale Flume with no lag including the R-square value. Figure 2-10 displays discharges at the SCD Flume at the corresponding delivery lag time for the BFRS Flume and Vale Flume. According to this figure, the delivery lag time from the SCD Flume to the BFRS Flume is 5 ± 0.5 hours and the delivery lag time from the SCD Flume to the Vale Flume is 3.5 ± 0.25 days. The delivery lag between the BFRS and Vale Flumes is then approximately 3.3 ± 0.25 days, or 79 ± 6 hours.

2.6.3 Hydraulic Model

A third method was used to determine the delivery lag time using the hydraulic model. Discharge was modeled from the SCD Flume to the BFRS and Vale Flumes

Table 2-2. South Canal Dam Flume to Belle Fourche River Siphon Flume Delivery Lag Times Tested and the Corresponding R-Square Values for Each Regression Technique

Delivery Lag (hrs)	Linear, Intercept R^2	Linear, Origin R^2
0	0.96467	0.96219
4	0.97075	0.96877
4.5	0.97092	0.96897
5	0.97097	0.96906
5.5	0.97094	0.96906
6	0.97078	0.96892
6.5	0.97057	0.96873
7	0.97032	0.96850
8	0.96971	0.96792

Table 2-3. South Canal Dam Flume to Vale Flume Delivery Lag Times Tested and the Corresponding R-Square Values for Each Regression Technique

Delivery Lag (Days)	Linear, Intercept R^2	Linear, Origin R^2
0	0.68339	0.63405
3.25	0.84973	0.70564
3.5	0.85797	0.70621
3.75	0.86361	0.70507
4	0.86700	0.70263
4.125	0.86738	0.70075
4.5	0.86699	0.69878
5	0.85852	0.69432

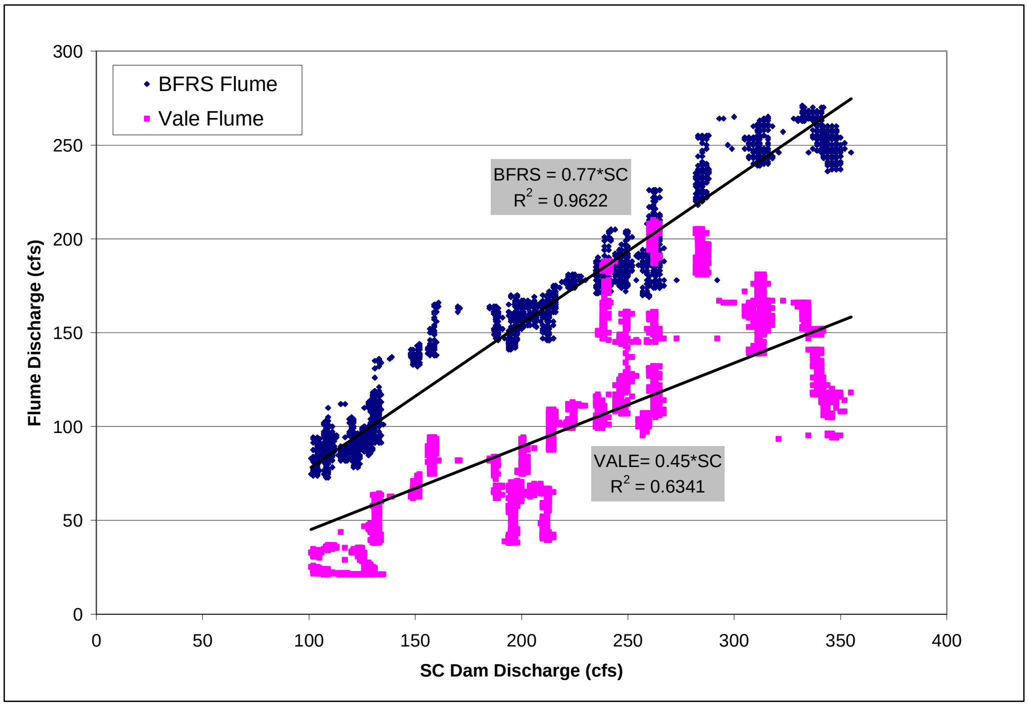


Figure 2-9. South Canal Dam Flume Recorded Discharge Versus the Belle Fourche River Siphon Flumes With No Delivery Lag Applied and Corresponding R-Square Value.

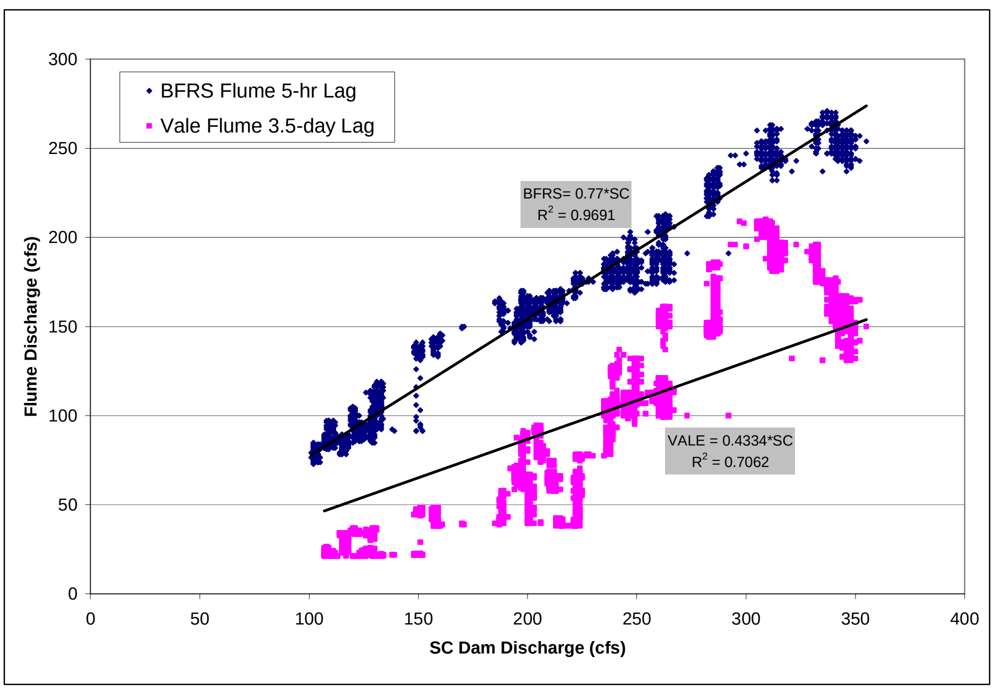


Figure 2-10. South Canal Dam Flume Recorded Discharge With Corresponding Delivery Lag Applied Versus the Belle Fourche River Siphon and Vale Flumes, Equations of the Regression Line, and R-Square Value.

using low- and high-flow scenarios. Daily water orders and dam discharges were used to determine an average South Canal Dam release and average water orders upstream of each check structure to set the hydraulic model as close to an observed condition as possible in the 2005 irrigation season. The hydraulic model accounted for the typical channel geometry and lateral discharges seen in the BFID and the model was calibrated using field monitoring data.

The discharge values used during each simulation period were obtained from the Water Master sheets for the South Canal and are displayed in Table 2-4. The total simulation time for both low- and high-flow scenarios was 6 days, including 1 additional day to allow for equilibrium at the Beals Check and Vale Flume. The time for equilibrium, or 90 percent of the total mass of flow released from the dam, at the BFRS Flume, Beals Check, and Vale Flume was the delivery lag time.

Table 2-4. South Canal Dam Release Schedule for Calculating Delivery Lag Time Using the Hydraulic Model

Elapsed Time (hrs)	Low-Flow Dam Release (cfs)	High-Flow Dam Release (cfs)
0–24	75	225
24–48	125	275
48–72	150	325
72–96	125	275
96–120	75	225

Figure 2-11 is the low-flow scenarios modeled for the SCD Flume to the BFRS and Vale Flumes. The red dots indicate the points at which 90 percent of the equilibrium is achieved, or when 90 percent of the downstream water orders have arrived at the structure. Table 2-5 is a summary of the modeled delivery lag times under low-flow conditions for the major South Canal structures. The average delivery lag time from the South Canal Dam outlet to the BFRS Flume is 7.6 hours. The average delivery lag time to the Beals Check is 19.2 hours. The average delivery lag time to the Vale Flume is 28.3 hours.

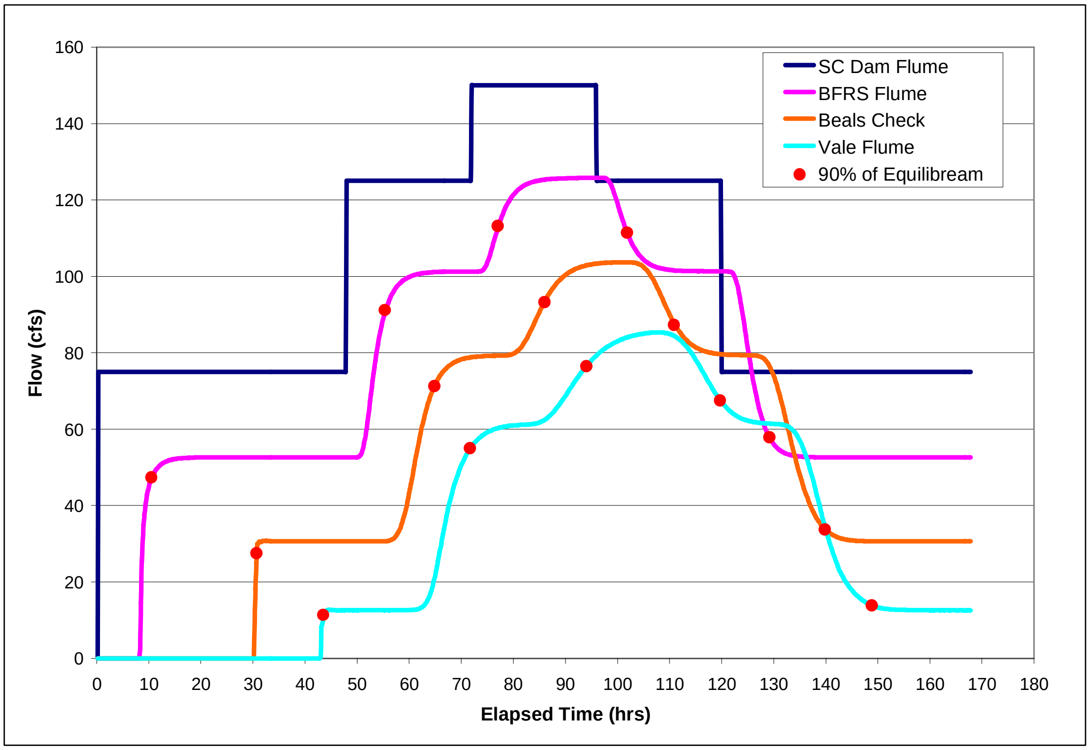


Figure 2-11. South Canal Dam Flume to the Belle Fourche River Siphon Flume Using Hydraulic Model for Low-Flow Conditions.

Figure 2-12 is the high-flow scenarios for the same reaches. Table 2-6 is a summary of the delivery lag times produced in the model. The delivery lag times for high flow were, on average, 1 hour less than low-flow conditions. This is true because the velocity wave is momentum driven and a higher flow rate will push the water faster than under low flows. The average delivery lag time from the South Canal Dam outlet to the BFRS Flume is 6.7 hours. The average delivery lag time to the Beals Check is 18.3 hours. The average delivery lag time to the Vale Flume is 27.7 hours.

Table 2-5. South Canal Modeled Delivery Lag Time Under Low-Flow Conditions

Elapsed Time (hrs)	Low-Flow South Canal Dam Change (cfs)	Time to 90% Equilibrium, BFRS Flume (hrs)	BFRS Delivery Lag Time (hrs)	Time to 90% Equilibrium, Beals Check (hrs)	Beals Delivery Lag Time (hrs)	Time to 90% Equilibrium, Vale Flume (hrs)	Vale Delivery Lag Time (hrs)
0	75	10.5	*10.5*	30.7	*30.7*	43.5	*43.5*
48	125	55.3	*7.3*	64.8	*16.8*	71.7	*23.7*
72	150	77.0	*5.0*	86.0	*14.0*	94.0	*22.0*
96	125	101.8	*5.8*	110.8	*14.8*	119.7	*23.7*
120	75	129.2	*9.2*	139.8	*19.8*	148.8	*28.8*

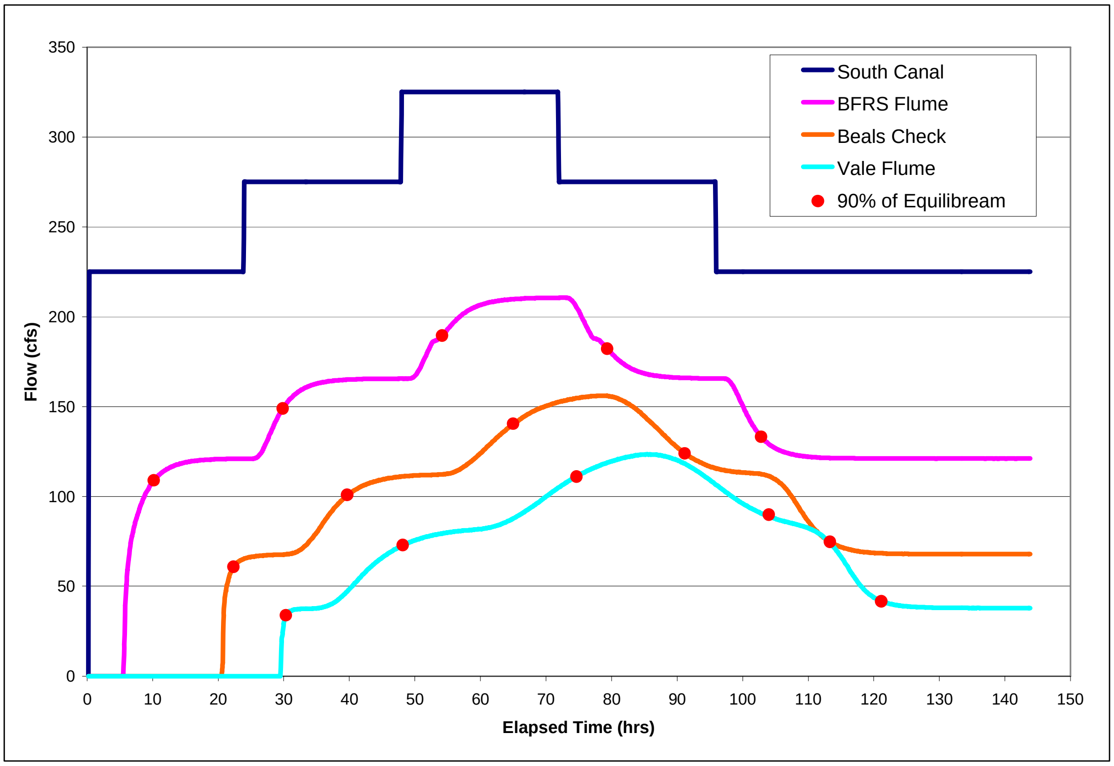


Figure 2-12. South Canal Dam Flume to the Belle Fourche River Siphon Flume Using Hydraulic Model for High-Flow Conditions.

2.6.4 Summary and Discussion of Delivery Lag Time

The three techniques used above produced different results for delivery lag time in the BFID. The estimated delivery lag times were chosen according to the three techniques described previously and advice from the District Manager. The delivery lag time is defined as the time for 90 percent of a water order to reach its destination. The regression technique was stopped when the lagged South Canal data were closest to the corresponding flume, indicating the flow is at a maximum (or equilibrium). The delivery lag time produced by the model was chosen at 90 percent of the equilibrium point. According to the District Manager, the regression technique estimates the lag time closely at the major structures. The model, however, tends to underestimate the lag

Table 2-6. South Canal Modeled Delivery Lag Time Under High-Flow Conditions

Elapsed Time (hrs)	High-Flow South Canal Dam Change (cfs)	Time to 90% Equilibrium, BFRS Flume (hrs)	BFRS Delivery Lag Time (hrs)	Time to 90% Equilibrium, Beals Check (hrs)	Beals Delivery Lag Time (hrs)	Time to 90% Equilibrium, Vale Flume (hrs)	Vale Delivery Lag Time (hrs)
0	225	10.2	*10.2*	22.3	*22.3*	30.3	*30.3*
24	250	29.8	*5.8*	39.7	*15.7*	48.2	*24.2*
48	300	54.2	*6.2*	65.0	*17.0*	74.7	*26.7*
72	250	76.3	*4.3*	91.2	*19.2*	104.0	*32.0*
96	225	102.8	*6.8*	113.3	*17.3*	121.2	*25.2*

time to the Beals Check and Vale Flume according to experience. For this reason, the delivery lag time from the SCD Flume to the BFRS Flume is best described by the hydraulic model while the lag from the SCD Flume to the Beals Check and Vale Flume is best described by the regression techniques. This reasoning is also based on a conservative estimate of the delivery lag time where a longer estimate is most conservative.

Table 2-7 is a summary of the delivery lag times for the South Canal. The values are rounded up to the nearest whole hour to be conservative. Because the Beals Check did not have logger data to perform the regression technique, the fraction of time between the BFRS and Vale Flumes was calculated according to the hydraulic model. On average, the delivery lag time to the Beals Check is 55 percent of the total time between the BFRS and Vale Flumes.

Table 2-7. Summary of Delivery Lag Times on the South Canal

South Canal Dam Flume to	Delivery Lag Time (hrs)
BFRS Flume	8
Beals Check	47
Vale Flume	84

2.6.5 Application

Understanding the delivery lag time in the BFID is essential to understanding system operation. Before the hydraulic model is validated, water call card information will be used to describe the total demand at each check structure. Orders for Rides 8 and 10 occur daily, but the actual demand at the check structures do not directly correspond to same-day orders. Delivery lag time must be applied to the water card data in order to get an accurate demand-delivery schedule at each check structure. A linear relationship was used to distribute the delivery lag time between the BFRS Flume and the Vale Flume. Approximately 4.3 hours of delivery lag per linear mile of canal exist between the structures. Water call card organization and data collection, as well as check structure operation curves.

3.0 OPERATIONAL MODEL

3.1 WATER CARD OPERATIONAL DATABASE

There are three components to the water demand/delivery system in the BFID. The first component, and the most important, is the *water call cards*. The water call cards are completed each day by the ditch riders describing the current orders by farmer and farmer turnout or lateral in units of cubic feet per second (cfs). Travel time becomes very important because Ride 2 orders and uses water on the same day while Ride 8 and Ride 10 must order water 2 or 3 days in advance. However, the call cards represent the total water released from the dam the day they are written, including transportation and operation losses, to satisfy the demand of the farmers.

The second component is the *water billing cards* organized by the farmer and the total amount of water used by the farmer in acre-feet per day of the month. This sheet is generally written at the end of the month and is used to track the amount of water deducted from the annual allotment for each farmer.

Finally, the *Water Master sheets* calculate the total dam release for each canal per day and record the daily activity on a per-ride basis. The sheets are organized by day and total release, demand in each ride, and discharge at each major flow-measuring structure (manually recorded by the Water Master). Upon release of water at the dam through large inlet structures controlled by the Water Master, the Water Master travels to each of the major flow structures in the system and records the discharge. The Water Master sheets are also the tool used to calculate year-end efficiency. Historically, the Water Master sheets were filled out manually and later converted to a digital format. With the addition of the water card database, all necessary discharge data are available in a digital format. Also, a flow calculator was created to calculate the discharge at the major flow-measuring structures and to account for submergence.

3.2 DESCRIPTION OF THE DATABASE

The water call cards have been converted to an electronic format and data are entered into a data spreadsheet each morning by District office personnel. The ditch riders manually fill out the paper water cards and present them to District personnel each morning. The spreadsheet was formatted to be identical to the paper water cards. Simple addition is performed to obtain total demand for each lateral and farmer turnout on each ride of the system. The digital format allows for math errors to be detected and corrected immediately when the ditch riders are present in the office each morning. The database construction, including the 2005 water call cards and blank water call cards for 2006, is located in Appendix A.

Figure 3-1 is a screen image of an electronic water call card for Ride 8 on the South Canal. Each month of the irrigation season has a workbook with one spreadsheet for each day of the month and a summary spreadsheet. Each worksheet is organized by farmer lateral or farmer turnout. Also included in the worksheet is a cell for operational and transportation waste entered as numerical values by the ditch riders which is included in the total. The totals represent the individual demand per lateral or farmer turnout and the sum is the total demand for the ride. Figure 3-2 is an example of the electronic database organizational format used to store the water card data on the disc. Under the main directory, each ride has a separate subfolder where the water card spreadsheets are organized by month.

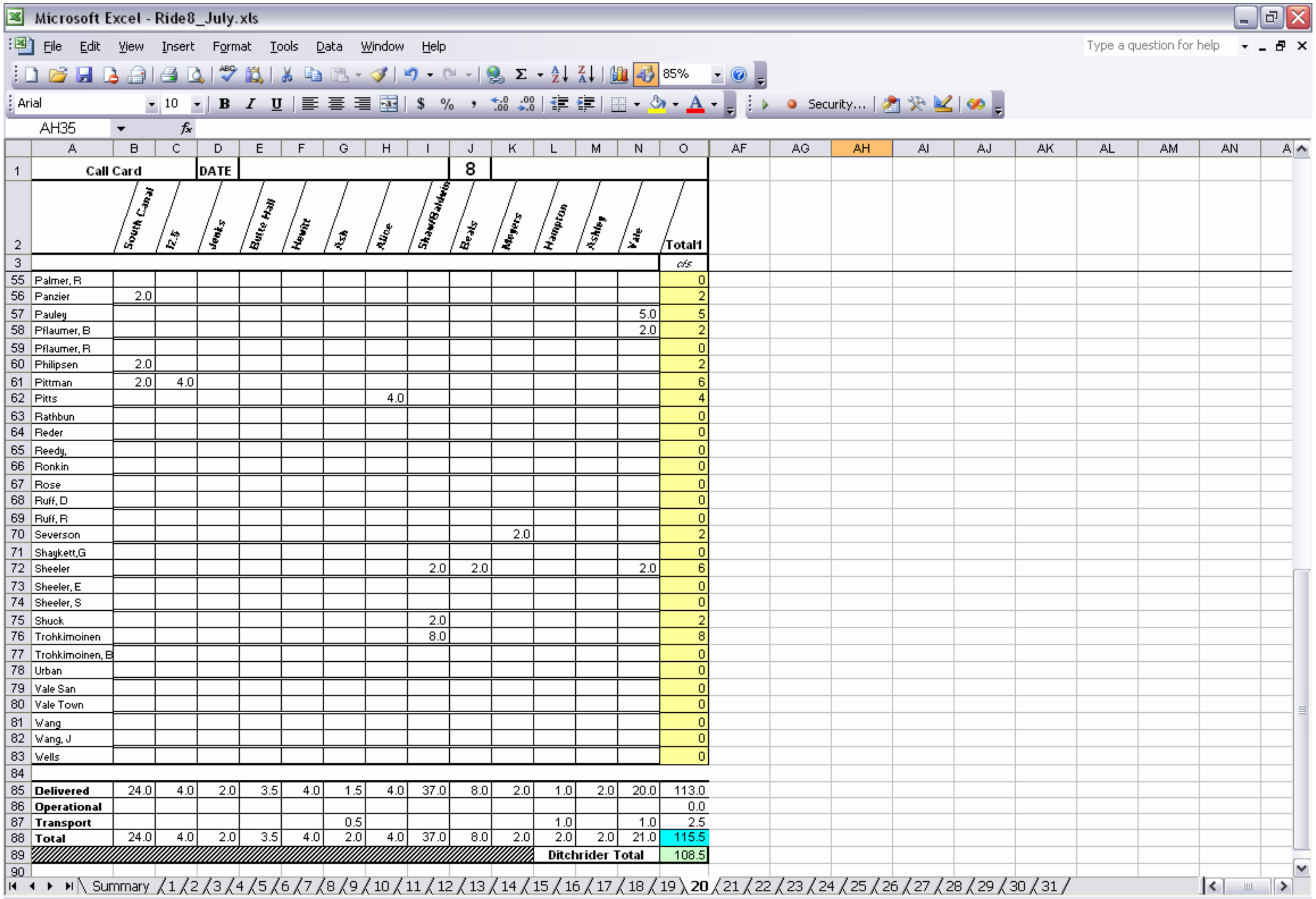


Figure 3-1. Example of an Electronic Water Call Card for Ride 8 of the South Canal.

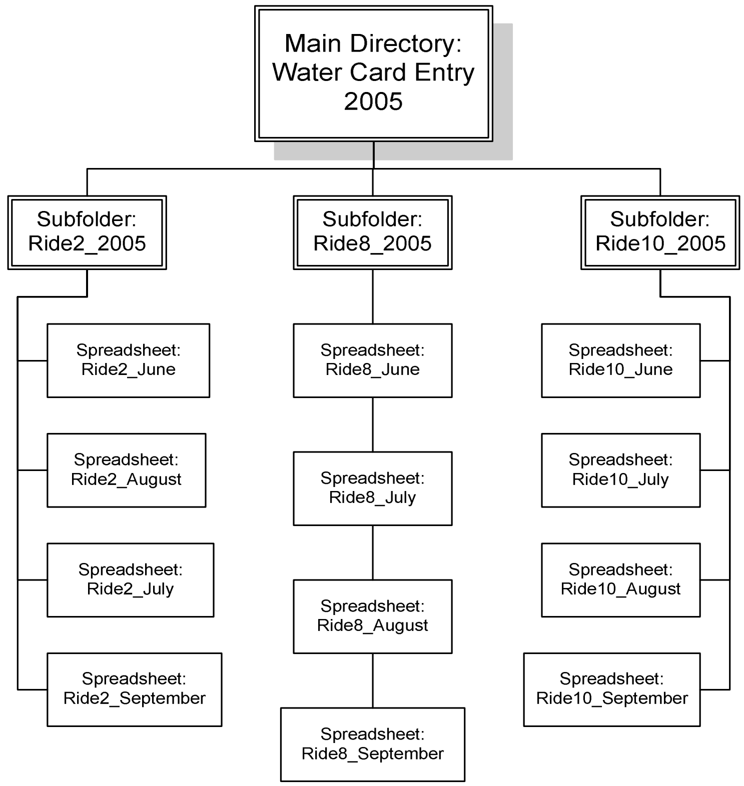


Figure 3-2. Water Card Data Organization Tree for the South Canal Rides.

3.3 DATA APPLICATION

Improvements in the water-ordering and data storage systems will support other improvements in the system for the 2006 irrigation season. Water orders are used to create a demand-delivery summary for each ride on the South Canal. A data collection and organization Excel macro was written to search the disc for the correct water orders and to calculate a water budget for the South Canal. The hydraulic model will be validated using data collected from the 2006 irrigation season, including real-time and automated check structure and field monitoring information. Water card data will be a necessary component of the hydraulic model to match the model to field conditions. A flow calculator was created to utilize the real-time stage data available at the flumes along the South Canal. Finally, the water billing cards were upgraded. With improvements to the water ordering system, the billing system is more accurate.

3.3.1 Demand/Delivery Schedule

Finding the total demand per lateral for each ride on the South Canal would be time consuming because each daily water order worksheet is contained within several layers. A macro was written to organize the necessary data into a format that is useful and easy to read. The macro uses three inputs: (1) the irrigation season selected for a dropdown list, (2) the month of the season also obtained from a dropdown list, and (3) the day of the month entered manually. When the macro is executed, it searches the directory for the correct season and produces the total water order (including nonused water from operational and transportation inefficiencies) for each farmer turnout and lateral on the South Canal. In a second worksheet, a summary of the nonused water is produced. Nonused water includes both operational and transportation losses needed to operate the system correctly. The total demand per ride and nonused water are calculated for the current day only and does not take into account the delivery lag time. Figure 3-3 is a screen image of the data spreadsheet and user form created to compile daily data.

Due to delivery lag time, water orders for a given day do not directly correspond to the demand at a check structure for the same day. In order to calculate the correct demand at each check structure, a lag must be applied to the water card orders that best represents field conditions. The delivery lag time from the SCD Flume to the BFRS Flume is approximately 8 hours and no delivery lag is applied because a water order, or demand, will travel from the dam to the BFRS Flume the same day. A delivery lag time of 3.5 days is expected from the SCD Flume to the Vale Flume. Here, the delivery lag must be applied and was accounted for in the data collection macro. A linear relationship was used to distribute the delivery lag time between the BFRS Flume and the Vale Flume. Approximately 4.3 hours of delivery lag per linear mile of canal exists between the structures, and the distribution of delivery lag from structure to structure can be seen in Table 3-1.

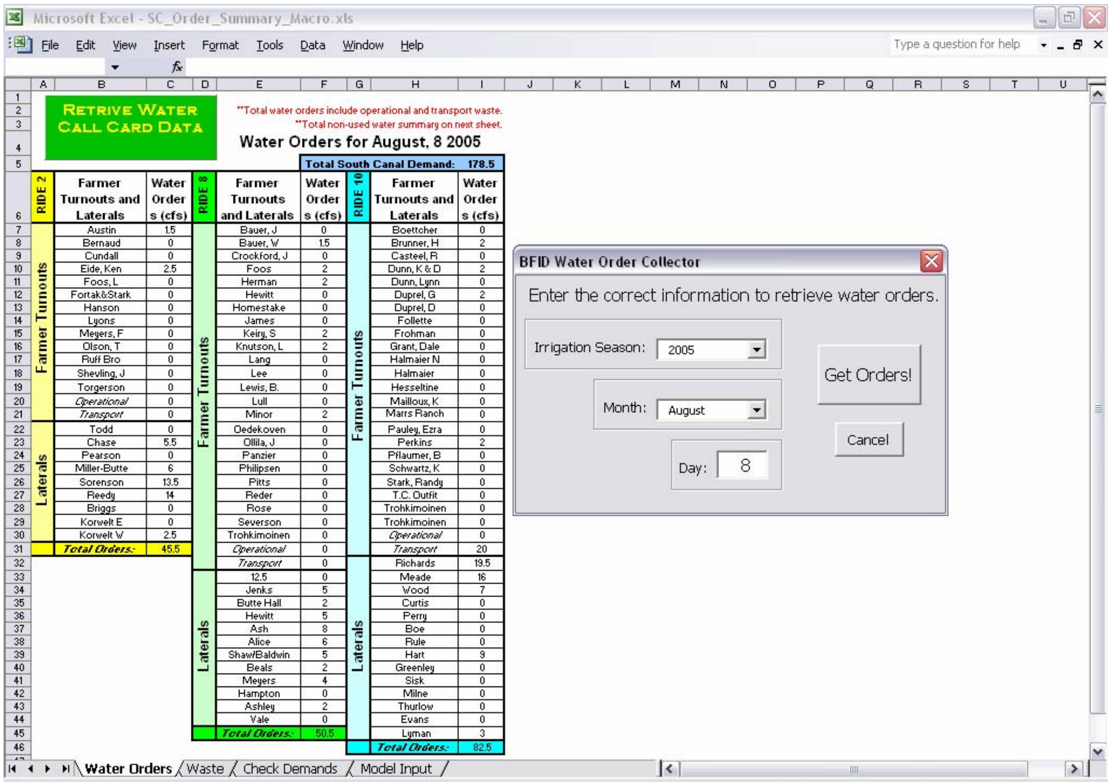


Figure 3-3. Screen Image of the Water Call Card Organization Spreadsheet for the South Canal.

In addition to the delivery lag time, another correction must be made to produce accurate demand-delivery schedules at check structures. The water cards are organized by farmer and the lateral and/or farmer turnout from which he/she orders. Many farmers order from multiple laterals, and farmer turnouts are often both upstream and downstream of a check structure. A decision was made based on the location of each farmer's irrigated land relative to the check structure to condense orders to the closest single farmer turnout. For example, a small system contains Check Structures A and B, with Farmers X and Y. Farmer X orders from four locations above Check A and one location above Check B while Farmer Y orders from one location above Check A and four locations above Check B. For this case, all of Farmer X's orders were placed as a single demand upstream of Check A and all of Farmer Y's orders were placed as a single demand upstream of Check B. Farmer turnouts generally have much smaller demands than lateral systems, so the check structure demands remain accurate with these assumptions.

Table 3-1. Summary of Delivery Lag Times Applied to the South Canal in the Data Collection Macro

Location	Distance (miles)	Lag from Dam (days)
SC Flume to BFRS Flume	7.9	0
BFRS Flume to 12.5	3	1
Kierry Check to Beals Check	6.1	2
Beals Check to Vale Flume	5.9	3

Another spreadsheet in the macro workbook, titled Check Demands, uses a macro that applies the delivery lag to the check structure demand, or the sum of water orders downstream of the check structure. It also simplifies the check structure demands by grouping farmer water orders as described above. The information is essential for proper use of a check structure rating curve.

3.3.2 Hydraulic Model

A hydraulic model of the BFID was created using U.S. Environmental Protection Agency's Storm Water Management Model (EPA SWMM) Version 5.0 (U.S. Environmental Protection Agency, 2005; Schoenfelder, 2006). Rolland (2005) evaluated SWMM and Root Canal (Utah State University), which is a model specifically designed for irrigation purposes. Root Canal was eliminated because the model is still in the trial stages and is not well documented. While SWMM was not specifically designed for irrigation applications, it applies the full Saint Venant equations, is widely trusted and accepted, and is well documented and proven. Figure 3-4 is a screen image of the complex hydraulic system represented in SWMM. The model contains every farmer turnout and lateral head gate structure, check structure, flume, siphon, bridge, and culvert on the South Canal, resulting in roughly 200 hydraulically unique points. Survey data from the Bureau of Reclamation were used to determine map the distances between structures, invert elevations, and channel cross sections.

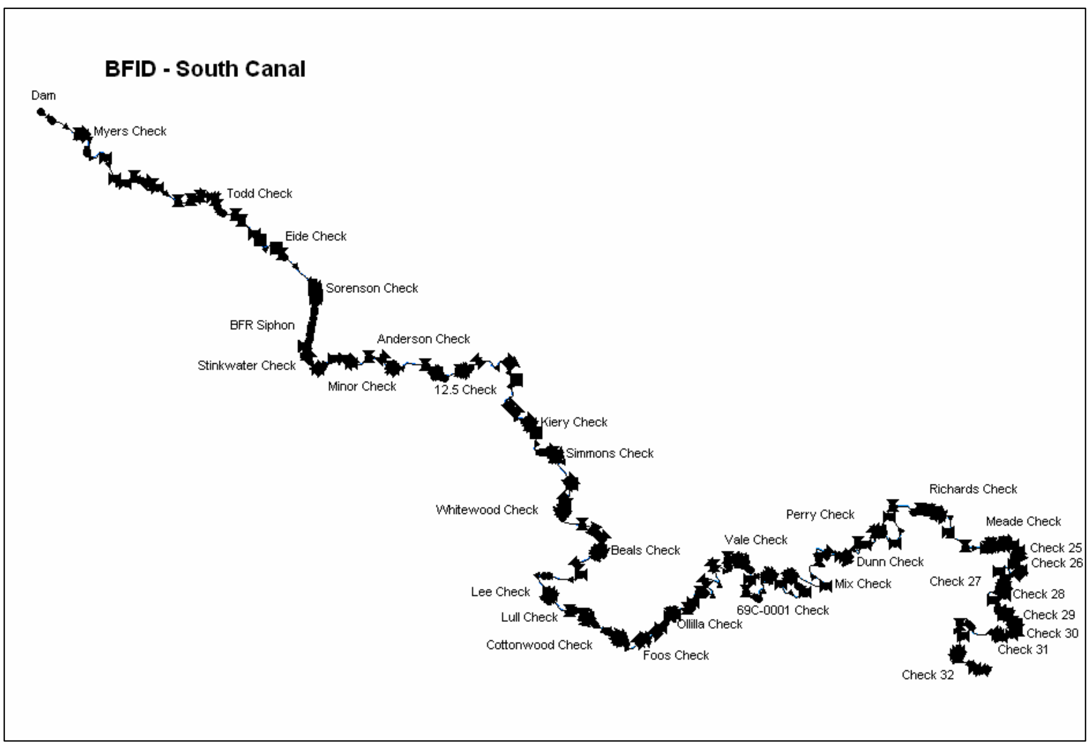


Figure 3-4. South Canal Hydraulic Components in the Hydraulic Model, EPA SWMM 5.0. The Check Structures Are Labeled and all Other Components Are Displayed.

A major challenge faced with the hydraulic model was calibration and validation. There are no laboratory conditions to test the model against; the field essentially becomes the laboratory to test the model. Monitoring data were collected over the 2005 irrigation season, including lateral and farmer turnout characteristics and settings, check structure characteristics and settings, and time-series stage and discharge data using pressure transducers and data loggers. These data were used to calibrate the hydraulic model. The first irrigation season after completion of the hydraulic and operational models will be a validation season. In order to make full use of the hydraulic model, the model must match field conditions in order to decide where variations in system operation occur. Real-time flow and stage data at the flumes on the South Canal are used to check the model on a daily basis. The operation procedures will be applied to adjust water levels at control structures and farmer turnouts. Discharge at real-time equipped flow structures will provide feedback to validate the SWMM output. In an ideal situation, the hydraulic model will predict the flow situation exactly. However, a significant difference in real-time data in a reach or at a flow structure occurs under one of two conditions: (1) the system structures are set correctly and the hydraulic model is not simulating the distribution correctly or (2) the system indicates a lateral gate or check structure is not set correctly and an adjustment needs to be made in the field. During the validation process, the output file should be formatted in such a way that the current real-time flows can be used to make corrections and transition it to an accurate input file. Once the hydraulic model is validated over the range of possible flows in the BFID, the second condition described above can be used to improve the operational efficiencies of the delivery system.

Completion of the hydraulic model is dependent on two separate periods: calibration and validation. During the calibration process, water card orders were used to force the model to discharge the correct amount of water. Primed laterals exist in the District, where the pipeline system is completely full of water and has a 100-percent open head gate but is not necessarily discharging water downstream. Instead, Schoenfelder (2005) adjusted the lateral head gate until the model matched the water card orders. This process must be repeated during the validation process to ensure the correct amount of water is exiting the system.

An assumption must be made during the validation process: Ditch Riders adjust the farmer turnout and lateral head gates to correctly discharge the orders described on the water call card. In other words, it must be assumed during the validation season that field conditions are correct and the model must be adjusted to match it. The farmer turnout and lateral head gates should be set according to the water call cards when field conditions are unknown or primed pipeline conditions exist. The engineer responsible for running the model on a daily basis must be in close contact with the water call cards and the data must be readily available. An additional spreadsheet is included in the data organization macro workbook described above which contains the total water orders for each node in the hydraulic model with correct delivery lag times applied.

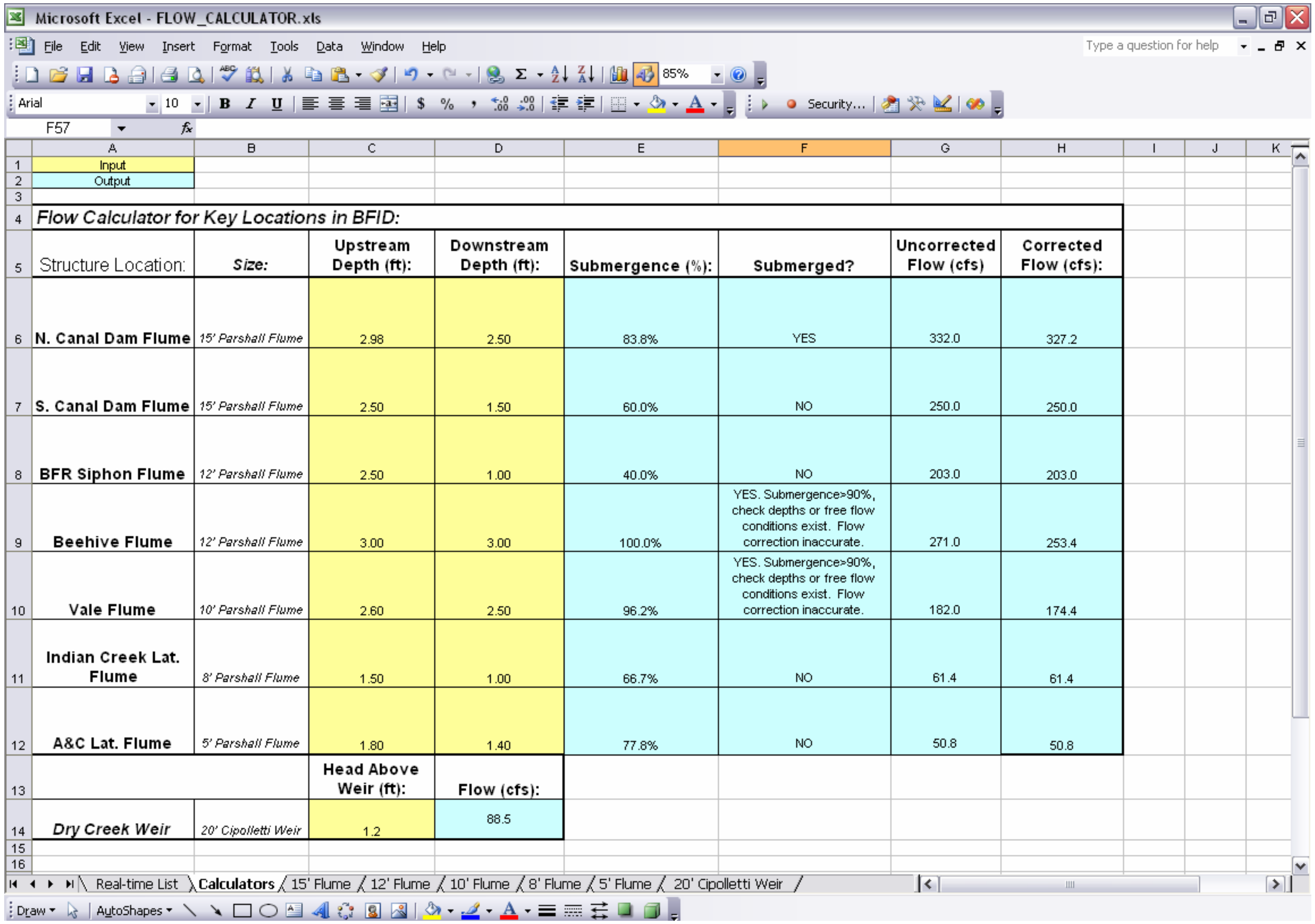
3.3.3 Flow Calculator

Historically, a Water Master sheet was produced manually, including the total release from the dam into the North and South Canals; total demand per ride on the canal; and additional nonused water, or "spill," required for proper operation for each day of the season. Also included were the discharge readings at the upstream stilling well at each flume (uncorrected for submergence). This sheet was essentially used to record the daily activity in the BFID and to calculate the overall efficiency for the season. With advances in technology, each flume is equipped with stage-recording devices in both the upstream and downstream stilling wells and is available real time.

A flow calculation spreadsheet was produced that allows District personnel to enter the upstream and downstream stage and calculate the total submergence and corrected discharge at all flumes and weirs on the North and South Canals. The Bureau of Reclamation rating tables in the *Water Measurement Manual* (U.S. Bureau of Reclamation, 2001) were used to relate the stage to a discharge in each size flume. The discharge is corrected using the techniques described in the *Water Measurement Manual* for all levels of submergence, and a warning appears if the submergence is beyond the 90 percent limit for accurate submergence correction. The Water Master can now make adjustments at the dam using accurate calculated demands from the water cards and use the flow calculation spreadsheet from the office or vehicle to monitor the discharge throughout the day. The flow calculator is displayed in Figure 3-5 and in Appendix A.

3.3.4 Billing Cards

The water billing cards were also upgraded. The water call cards represent the total water delivered to the farmer and the billing cards represent the total water used by the farmer. Often the water billed is less than ordered due to operational losses, precipitation, canal fluctuations, or reservoir capacity. In the past, ditch riders used the paper water call card data and memory to calculate the total water billed to each farmer on a daily basis at the end of the month. If a discrepancy was seen at the beginning of the month, it was up to the ditch rider to remember the event and correct the billed total. With a digital format, a running total of water ordered and billed for each farmer is recorded and the billing system is mathematically more accurate.



Microsoft Excel - FLOW_CALCULATOR.xls

Structure Location:	Size:	Upstream Depth (ft):	Downstream Depth (ft):	Submergence (%):	Submerged?	Uncorrected Flow (cfs)	Corrected Flow (cfs):
N. Canal Dam Flume	15' Parshall Flume	2.98	2.50	83.8%	YES	332.0	327.2
S. Canal Dam Flume	15' Parshall Flume	2.50	1.50	60.0%	NO	250.0	250.0
BFR Siphon Flume	12' Parshall Flume	2.50	1.00	40.0%	NO	203.0	203.0
Beehive Flume	12' Parshall Flume	3.00	3.00	100.0%	YES. Submergence>90%, check depths or free flow conditions exist. Flow correction inaccurate.	271.0	253.4
Vale Flume	10' Parshall Flume	2.60	2.50	96.2%	YES. Submergence>90%, check depths or free flow conditions exist. Flow correction inaccurate.	182.0	174.4
Indian Creek Lat. Flume	8' Parshall Flume	1.50	1.00	66.7%	NO	61.4	61.4
A&C Lat. Flume	5' Parshall Flume	1.80	1.40	77.8%	NO	50.8	50.8
		Head Above Weir (ft):	Flow (cfs):				
Dry Creek Weir	20' Cipolletti Weir	1.2	88.5				

Figure 3-5. Flow Calculator Used to Calculate Discharge at the Major Measuring Structures on the North and South Canals.

Figure 3-6 is a process diagram which describes the development of submerged and unsubmerged discharge in the flow calculator.

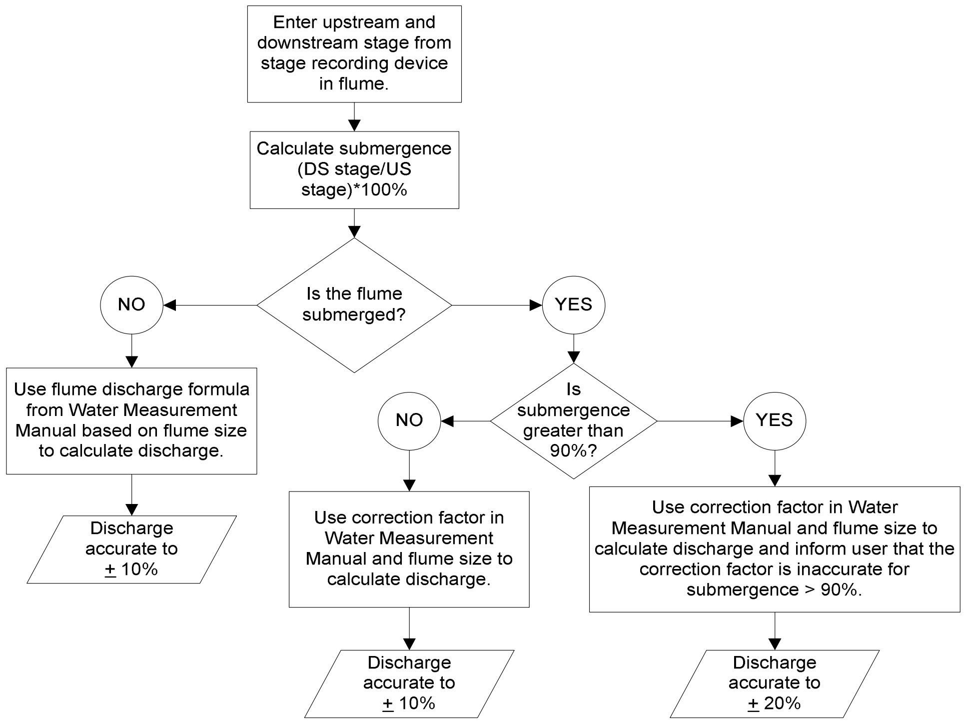


Figure 3-6. Process Flow Diagram for Development of Submerged and Unsubmerged Discharge Calculations.

3.4 CHECK STRUCTURE OPERATION

Each check structure in the BFID is hydraulically unique and has an optimum range of settings for low- and high-flow situations. The ranges for low and high flow are defined in the *Delivery Lag Time* section of this report (Table 2-4). Each check contains three main variables: the head or pool elevation behind the check, the weir board settings, and the sluice gate settings. Another component of each check are the emergency weirs which are permanent concrete weirs designed to allow water to continue downstream in the case of an intense rainstorm, a large wave of water, or any other dramatic increase in the pool elevation. Each check has slots in which weir boards can be added or removed to increase or decrease the head behind the check. The dimensions of the weir boards are generally 6 × 8 inch pieces cut to fit the weir opening. The gate structures are typical sluice gates which are adjustable up or down to vary the discharge under the gate. The main purpose of the check is to create a level pool upstream which produces the necessary elevation head at the lateral/farmer turnout head gates. Several check structures are displayed in Figure 3-7.

Figure 3-7. Four Examples of Check Structures Along the South Canal, Belle Fourche Irrigation District. Each has a Different Combination of Weir Boards and Sluice Gates With the Exception of the Bottom Left Which has no Sluice Gates. Each Structure has, in Turn, a Unique Rating Curve.

3.4.1 <u>Check Structure Operation Curves</u>

Proper check structure operation in the BFID is essential to deliver the correct amount of water to a lateral or farmer turnout and to control downstream fluctuations in the canal. Also, timing of adjustment must be well defined. Two situations arise if a check structure is not adjusted properly or at the correct time. If an increase in discharge occurs in the South Canal and a check structure is adjusted late, the head behind the check structure will increase and the flow to open lateral and farmer turnout head gates increases. If a check structure is adjusted too soon, especially near the end Of Ride 8, the existing pool elevation can dramatically decrease. When the pool is forming, water must be checked, the wave is attenuated downstream, and downstream water orders take longer to arrive.

Check structures are generally adjusted to the demand in the canal and only adjusted around a range of discharges. In other words, check structures are set to one combination of settings and left until the flow increases or decreases beyond the capacity of the setting. This is especially important in regard to automated check structures because the automated gate only has a capacity to control flow over a range of about 50 cfs. Adjustment of the manual gates will be necessary to ensure the automation is operating under the full range of conditions.

Check structure operation curves are a tool that the BFID can use to compare the upstream water surface elevation to discharge through the check based on a typical setting. The upstream stage is often a controlling force in operation because each check structure has a design water surface elevation, generally 1 inch below the automatic weirs. However, exceptions arise when the design water surface elevation does not produce the necessary delivery head at the upstream lateral and farmer turnout head gates and the automatic weirs must be used. Now the ditch rider has a choice as to how he wants the check structure to operate and some guidelines for what structure settings will work.

The Beals Check was used to develop example check structure operation curves because field monitoring efforts were focused at this location during the 2005 irrigation season. The Beals Check has two sluice gates and two weir board slots. It was monitored from July 11, 2005, to August 17, 2005, where 16 discharge measurements were taken using a flow meter roughly 200 feet downstream of the check. Gate and weir settings and water surface elevations were also measured. Typical sluice gate and weir equations were used to calculate the discharge through the check. The sluice equation used was:

$$Q = C_d ab \times \sqrt{2 \times g \times h_0} \tag{3-1}$$

where:

Q = discharge in cfs
C_d = discharge coefficient
a = sluice gate opening in feet
b = sluice gate width in feet
g = gravitational acceleration in feet per second squared
h_0 = upstream water depth above the invert in feet.

The weir equation used was:

$$Q = C \times L \times h^{15} \tag{3-2}$$

where:

Q = discharge in cfs
C = discharge coefficient
L = weir length in feet
h = head above the weir in feet (Gupta, 2001).

The discharge coefficient used for the sluice gate was 0.42. The discharge coefficient for the weir equation was 2.8 for the weir boards and automatic weirs as calibrated by the hydraulic model (Schoenfelder, 2005). The equations produced discharge within ± 15 percent (maximum) of observed. On average, the calculated discharge was within

± 7 percent of observed. Figure 3-8 is a plot of calculated to observed discharges at the Beals Check.

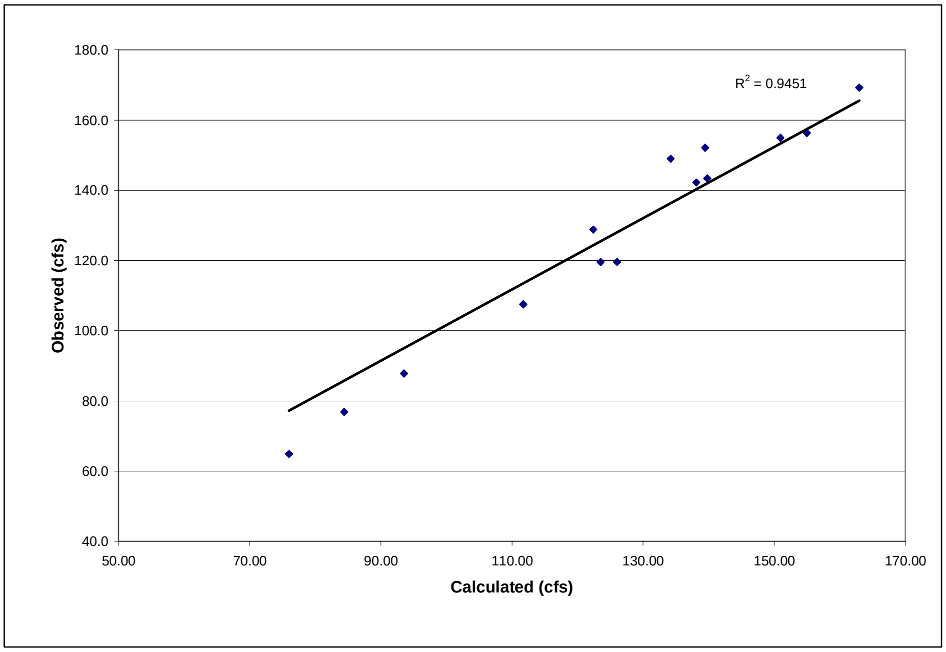


Figure 3-8. Comparison of Calculated Versus Observed Discharges at the Beals Creek.

Upstream water depths ranged from 3.29 to 4.05 feet during the monitoring period and five different combinations of structure settings were observed. The equations described above were used to calculate the discharge over the range of depths observed in the 2005 irrigation season and plotted. Figure 3-9 is a plot of the possible range of discharges versus upstream depth for the given structure settings. Left gate opening, right gate opening, left weir height, and right weir height are labeled as LG, RG, LW, and RW, respectively. Weir height can be easily converted into the number of weir boards by dividing the height by the size of the board. See Appendix A for all data collected at the Beals Check during the 2005 season and development of the operation curves.

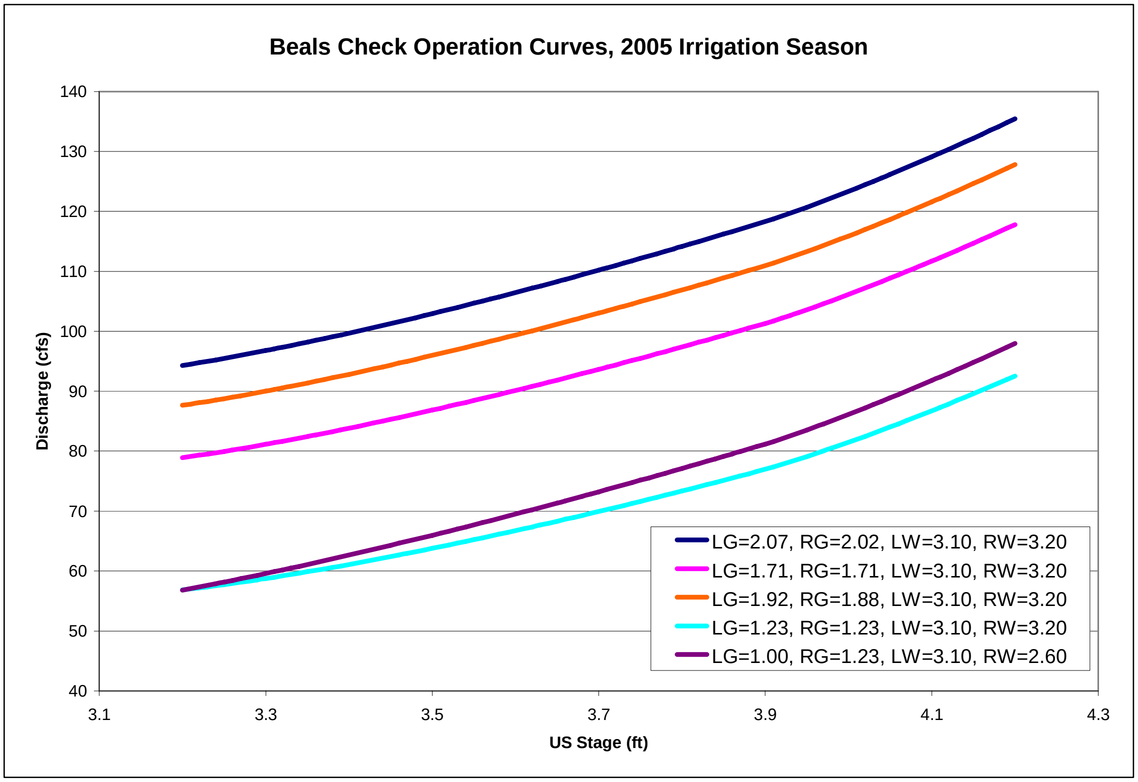


Figure 3-9. Beals Check Structure Discharge Versus Upstream Depth for Typical Structure Settings Observed During the 2005 Irrigation Season.

3.4.2 Application of Check Structure Operation Curves

The three major components required for proper check structure operation are:

1. Timing parameters: Understanding how long it takes for a water order to arrive at a check structure upon release from the dam is the most important component to check structure operation. Delivery lag time was calculated for the South Canal.

2. Check structure demands: Delivery lag times can be used in conjunction with water card information to estimate the time it takes for a certain demand to reach a check structure.

3. Proper check structure settings: Check structures must be set correctly so the capacity of the check under the setting is not exceeded. Also, the structure must be set correctly to pool enough water so proper delivery head is achieved at the upstream head gates.

The combination of these items will guide the ditch riders in proper check structure operation and a process must be fit into the daily routine. Each morning, the ditch riders will be presented a demand schedule based on the current water card conditions with delivery lag times applied. A set of timing parameters is produced by the hydraulic model. With check structure operation curves in hand, the ditch riders can make adjustments when applicable.

4.0 RECOMMENDATIONS AND CONCLUSIONS

4.1 RECOMMENDED DAILY OPERATIONAL PLAN FOR 2006 SEASON

The daily operation should be well defined for the 2006 irrigation season because of the advances in efficiency efforts. The water-ordering system and Water Master interactions have changed with the installation of the Water Card Operational Database. Check structure automation and real-time flow data must be considered in regard to manual system operation. The hydraulic model will perform simulations every day and, due to the time it takes for the model to run, coordination must be established between the ditch riders, office personnel, Water Master, and engineer.

It was expressed by the District Manager that changes to normal daily operation be altered as little as possible. The goal was to fit the upgrades to the water-ordering system, including water call cards and Water Master sheets, and running the hydraulic model into the process and still keep the same schedule as in previous years. Historically, ditch riders meet at the District Office between 7 a.m. and 7:30 a.m. to give the Water Master the daily water orders so adjustments at the dam can be made by 9 a.m. The new interactions that exist in the District office will occur between these times and include digital water card data entry, running the hydraulic model, and producing a daily activity report.

The operational process can be considered using two separate operational models. The first process (Figure 4-1) utilizes the Water Card Operational Database and data collection macros to produce a water balance of the South Canal and to estimate the discharge at check structures and lateral and farmer turnout head gates. The second process (Figure 4-2) utilizes the hydraulic model to predict discharge in the South Canal

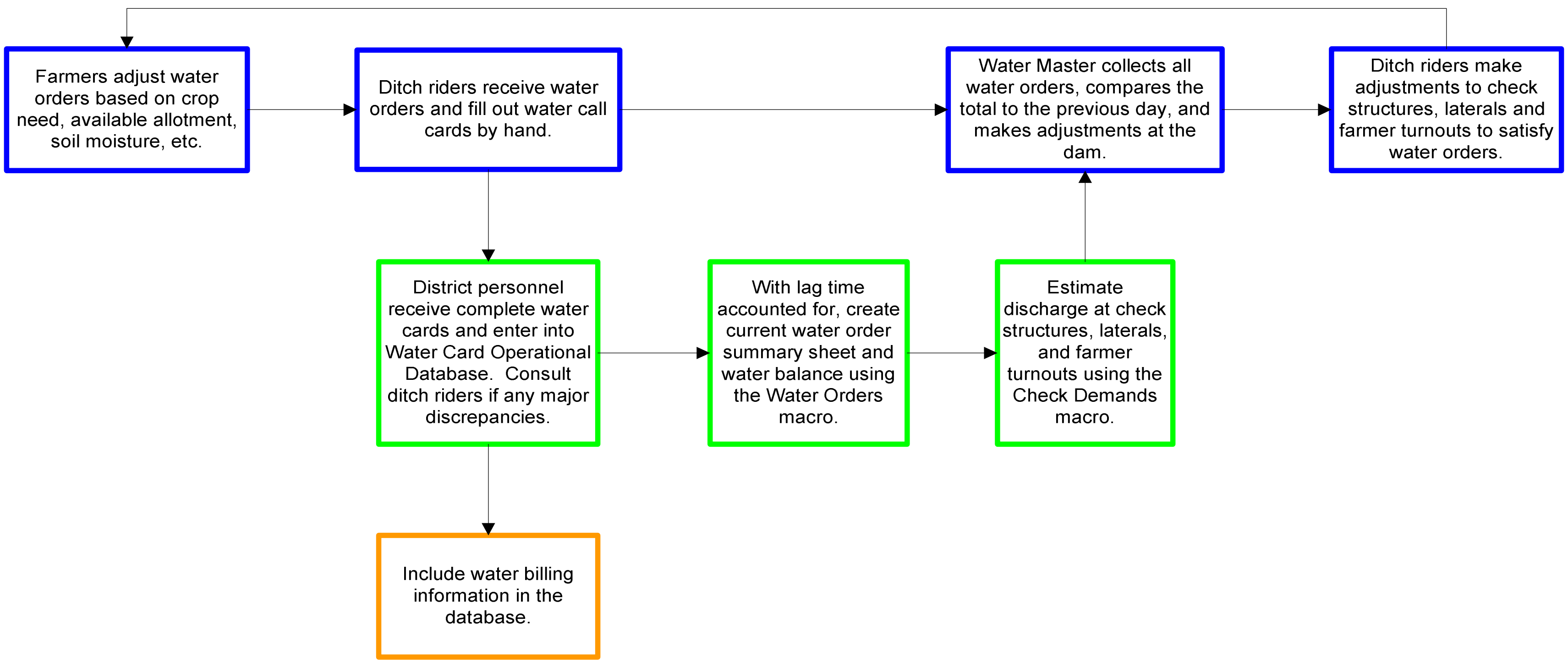


Figure 4-1. Belle Fourche Irrigation District Daily Process Diagram Incorporating the Water Card Operational Database.

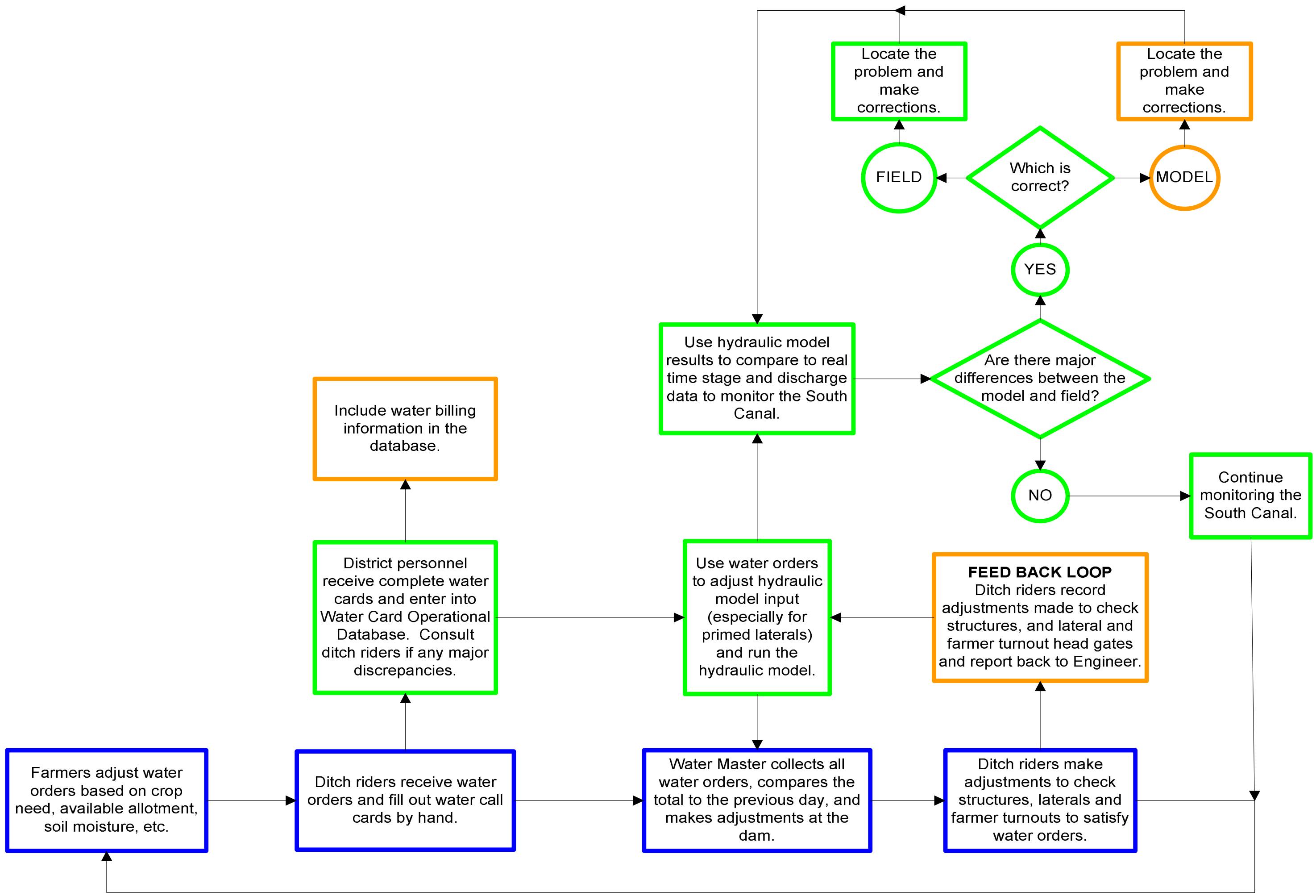


Figure 4-2. Belle Fourche Irrigation District Process Diagram Incorporating the Hydraulic Model.

and to provide a link between the model and real-time stage and discharge data collected at automated sites. Also, a feedback loop is recommended which provides the engineer with structure settings recorded by ditch riders. In both models, blue figures represent the current practices, green figures represent future practices (summer 2006), and orange figures represent recommended practices.

4.2 RECOMMENDATIONS FOR 2006 AND 2007

It is recommended that efficiency efforts be focused on improving the operational efficiency efforts through structural improvements. A reduction of 12,000 acre-feet of return flows to the Belle Fourche River by the Year 2015 would be achieved. The following recommendations will assist the BFID in reaching this reduction in operational losses:

1. Structure Automation: Segment III of the Project Implementation Plan (Belle Fourche River Watershed Partnership, 2005) calls for 25 additional units to be installed from June 2006 to October 2007. During the 2006 irrigation season, 12 units will be installed. The locations of the additional units must be placed to minimize the distance between new and existing units and to maximize the information obtained per ride along the North and South Canals. Focus should also be placed on the Parshall flumes along the canals and laterals to include recorded stage in both the upstream and downstream stilling wells for submergence correction. Recommendations in regard to locations of new automation from the District Manager and Water Master were taken into consideration. Table 4-1 lists the new structure automation locations and corresponding ride, the type of structure, and estimated cost associated with the

new automation. The total cost associated with each type of automation was taken from a budget sheet provided by the BFID.

Table 4-1. Additional Automated Sites to be Installed During the 2006 Irrigation Season

Location	Ride	Type	Estimated Cost ($)
Indian Creek Check/Lateral Flume (Head Works)	4	Combination Site	18,000
Capp Check	4	Automated Check	6,500
Stumpf Check	4	Automated Check	6,500
Horse Creek Siphon Check	5	Automated Check	6,500
Deyo Check	6	Automated Check	6,500
Deer Creek Lateral Flume	6	Real-Time Site	12,000
North Lateral Check	6	Automated Check	6,500
Willow Creek Lateral Flume	6	Real-Time Site	12,000
Indian Creek Lateral Flume	7	Combination Site	18,000
Wilson Lateral Flume	7	Combination Site	18,000
Shaw Baldwin Lateral	8	Real-Time Site	12,000
Nine Mile Creek	10	Real-Time Site	12,000
TOTAL COST			**134,500**

2. Parshall Flume Submergence: The submergence issues in the BFID must be resolved because the Parshall flumes are linked between accurate discharge measurement and the hydraulic model. The operational efficiency is limited by the accuracy of discharge measuring structures, and improvements in the

accuracy will, in itself, account for a portion of the overall water savings. The hydraulic environment; i.e., slope, channel geometry, and depth measurement, must be analyzed. Suggestions are:

a. Explore the conversion of the Parshall flume to a more fitting device such as a long-throated or ramp flume.

b. Survey the channel upstream and downstream of the flume and compare to surveys and construction specification drawings to determine if the channel has been affected by siltation or erosion. If the channel is reshaped to an original specification, the slope of the channel will increase and allow for an increase in head at the flume.

c. Continue to measure and monitor the submergence at the Parshall flumes to further define the submergence issues.

3. Portable Continuous Stage Measuring Devices: Additional monitoring information will be a necessary component to hydraulic model validation and efficient system operation. Several pressure transducers from the portable units were removed and installed in the downstream stilling wells of flumes. However, six data loggers are still available and the pressure transducers should be replaced. Nine additional mobile units should be purchased at an estimated total cost of $10,000 to be used for canal or lateral monitoring. A plan similar to the 2005 irrigation season should be developed so the mobile units are not along a single reach for more than 3 weeks. Also, the units could be placed along the entire canal to utilize stage measurements logged at automated gates. This would allow for a large section of canal to have continuous stage data at each check structure.

The pressure transducer should be protected by a 2-or 3-inch plastic (polyvinyl chloride (PVC)) pipe cut to an appropriate length determined by the structure. Small holes should be drilled into the pipe using a 1/16-inch drill bit at no more than five locations along the submerged portion of the pipe. The fewer holes, the better, as this will minimize the turbulence inside the pipe. Also, the pressure transducer should be secured to the inside of the pipe using zip ties to eliminate the opportunity for the transducer to float, or move, inside the pipe. The pipe should be well secured to the head gate structure using zip ties, and the data logger should be as far out of visible range as possible.

Setting the reference elevation of the pressure transducer is important. Ideally, the pressure transducer should be set to “zero” elevation, or the elevation at which the pressure transducer is measuring the actual depth of water and no depth adjustment is needed. However, if this is not possible, a reference elevation must be clearly defined so a depth adjustment can be applied. The reference point should be permanent; the top of concrete or point on the gate structure tends to work well. The depth is corrected according to:

$$h_a = h_t + z_a \tag{4-1}$$

where:

h_a = actual depth of water to the reference datum
h_t = depth of water measured by the transducer according to its position
z_a = offset.

The invert elevation measured relatively to the top of concrete (or clearly defined reference point) should be defined. A measurement of the water surface elevation to the top reference point can then be easily converted into actual water depth.

This measurement should be taken at least five times while the pressure transducer is in place so an average adjustment is produced.

From experience obtained during the 2005 irrigation season, upstream stage at a given check structure is best collected at the nearest upstream head gate. The water entering the weir and gate chambers at a check structure is at a very high velocity. For this reason, a pipe protecting a pressure transducer will not be secure unless physically attached to the check structure. However, the water is tranquil 50 feet upstream. Reference elevations must be collected to correct the recorded pressure transducer depth to the observed depth at both the head gate structure and check structure. If the difference in elevation between the head gate reference point and the check structure reference point is known, the water depth collected by the pressure transducer can be converted to the actual depth observed at the check structure.

4. The Johnson Lateral Return Flows: Return flows from the Johnson Lateral are a significant addition to the discharge in the South Canal just upstream of the BFRS Flume. If the discharge is not monitored, an assumption must be made in the hydraulic model to accurately predict the sudden increase in flow at this location. A data logger and pressure transducer should be installed at the diversion box at the end of the Johnson Lateral. The box contains a weir and is used to measure the Johnson Lateral return flows. The transducer depth should be set and an adjustment made for the depth of the transducer relative to the crest of the weir.

5. Check Structure Operation Curves: Additional monitoring is needed to fully develop the check structure operation curves. Discharge measurements are

necessary downstream of the check structure to support the water card balance and hydraulic model simulations. With a fully validated model, simulations can be run over a variety of flow conditions and structure settings to support the operational curves. Focus should be placed on the South Canal because a hydraulic model has been developed and can be used to support the development of the operation curves. Operation curves should be developed for the Sorenson and Vale Checks because continuous discharge data are available at the flumes located just downstream and both are automated.

a. Monitoring of five check structures (including the Sorenson and Vale Checks) is recommended along the South Canal. The check structures should be chosen based on hydraulic properties so as to group several checks into one category. Table 4-2 lists the recommended check structures to monitor during the 2006 irrigation season.

Table 4-2. Recommended Check Structures to Monitor and Develop Check Structure Rating Curves During the 2006 Irrigation Season

Check Structure Name	Reach	# of Gates	# of Weirs
Sorenson Check	1	1	1
12.5 Check	2	2	3
Lull Check	3	3	2
Vale Check	3	2	2
Dam Check	4	2	2

b. Check structures must be monitored at least once daily to verify check structure settings. Settings to monitor include gate positions, weir board positions, upstream water surface elevations for pressure transducer datum corrections and relative head above the automatic and board weirs, and downstream water surface elevations for pressure transducer datum corrections and submergence calculations.

c. Discharge should be measured 200 feet downstream of each check structure using a flow meter. Daily monitoring of the check structure is required to observe any changes to the settings. At least 10 discharge measurements for each check structure setting should be taken during the monitoring period. At the time of the discharge measurement, all check structure settings listed above must be recorded. Certain locations along the South Canal will be dangerous for a person to measure discharge. Check structures were chosen based on safety factors as well.

d. The data collected should be simulated using the hydraulic model to validate the model and support rating curve development.

6. Water Card Operational Database Automation: When a farm changes owners, the water cards must be updated to fit the changes. If done manually, the process takes hours because the names must be changed in every spreadsheet for each month of the ride. An automated process should be developed so District personnel have the ability to make the changes in a simple and efficient manner. A macro that allows the user to enter the previous land owner and new land owner and make the necessary change is recommended.

7. Water Billing Information: The digital water cards are set up to include the billing information. It would be advantageous for District personnel to record the billing information in a digital format immediately when data are available. This will eliminate math errors and keep the database updated so information does not depend on the memory of the ditch riders.

8. Ditch Rider Feedback: The input file for the hydraulic model requires current structure settings, including farmer turnout and lateral head gate openings and check structure settings. During model calibration, the model was set according to previously observed field conditions. During validation, however, structure settings must be gathered from the field for the current model run. An interaction between the ditch riders and the engineer is necessary to communicate any changes to the structure settings so adjustments can be made in the model. A feedback loop in the daily operational process flow diagram is shown in Figure 4-2. Ditch riders are provided with a worksheet to record the date and time of the change, the location, and the new setting (i.e., stem height to be converted to percent open or weir board changes) for farmer turnout and lateral head gates and check structures. The worksheet should be presented to the Water Master or engineer each morning with the water call cards. Changes should also be communicated to the engineer by radio during daily operation if rerunning the hydraulic model is required to match model conditions to field conditions.

9. Hydraulic Model of the North Canal: Recommended reaches for analysis of the North Canal are: Reach 1: North Canal Dam Flume to Beehive Flume, Reach 2: Beehive Flume to Dry Creek Weir, and Reach 3: Dry Creek Weir to North Canal Wasteway. At this time, the South Canal model validation and check structure operation curve development is priority. However, monitoring of Reach 1 of the North Canal should begin by repeating similar monitoring efforts performed on the South Canal during the 2005 irrigation season. In order to fully develop the North Canal model according to the process used during development of the South Canal model, two students are required to monitor the entire system.

10. Personnel Support: Currently, three students from the South Dakota School of Mines and Technology are scheduled to participate in the project. Mr. Curt Schoenfelder, who worked on the project in 2005 and wrote the South Canal hydraulic model, is the lead engineer. He will run the model daily, perform Water Master duties, and monitor sites along the South Canal for model validation. Two additional students are responsible for further South Canal monitoring, check structure operation curve development, automating the water card operational database, and initiating modeling work on the North Canal.

4.3 CONCLUSIONS

This research focuses on improving the application and delivery efficiency of the BFID to reduce the total suspended solids (TSS) load entering the Belle Fourche River. Hoyer (2003) identified natural bank sloughing, riparian habitat impairment, and nonused irrigation water discharged into natural waterways as the primary contributors of TSS. Extensive field monitoring of the South Canal was performed during the 2005 irrigation season to provide calibration data for the hydraulic model (Schoenfelder, 2006). Several system improvements took place during 2005 to improve the operational efficiency, including:

1. Check structure automation and real-time site installation: Structure automation can be broken into three types, including automated check structure, real-time site, and combination site, and 22 sites were installed during 2005. Automating check structures allow for constant pool level to be held upstream of the check, which allows for constant head pressure at each farmer turnout or lateral head gate. Also, ditch riders are able to focus efficiency efforts on lateral systems or nonautomated check structures.

2. Water card operational database: The water-ordering system was upgraded to a digital format, which allows for simple demand-delivery scheduling, mathematical error checking, and proper storage of water ordering information. Due to the complexity of the storage system, an Excel macro was written to search for workbooks of a given irrigation season, month, and day to retrieve the proper daily ordering information, operational and transportation nonused water, develop check structure demands based on delivery lag time, and produce the total demand at each node in the hydraulic model. Improvements to the water-ordering system included upgrades to the Water Master sheets and billing sheets.

3. Hydraulic model: Schoenfelder (2006) created a hydraulic model of the South Canal using the U.S. Environmental Protection Agency's Storm Water Management Model Version 5.0 (EPA SWMM 5.0), which is capable of modeling the full range of flows through each hydraulic element on the South Canal. The model will be validated during the 2006 irrigation season and used as a tool to locate problems in the field and to make decisions on how to fix the problems.

4. Operational model and plan: In conjunction with hydraulic model, an operational model and plan were created for the 2006 irrigation season. The model defines the daily process and provides a daily check list, or decision matrix, that must be followed for proper integration of the hydraulic model and other improvements to the system. With each component of the operational model working together, the operational system will undoubtedly improve, water conservation goals will be achieved, and the TSS levels of the Belle Fourche River will be reduced.

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VITA

Timothy Olson was born in St. Paul, Minnesota, on October 7, 1981. He grew up in Inver Grove Heights, Minnesota, and attended Simley Senior High School where he graduated in June 2000. He continued his education at the South Dakota School of Mines and Technology in Rapid City, South Dakota, in the fall of 2000 and graduated with a bachelor's of science degree in civil engineering in December 2004. He then went on to graduate school at the School of Mines and Technology where he worked as a research assistant for two summers plus three semesters of course work and research under Dr. Scott Kenner. He received his master's degree in civil engineering in May 2006. Tim was married in November 2005 to his beautiful wife, Melissa. They moved to Mankato, Minnesota, where Tim accepted a position in water resources with a private engineering consultant.

APPENDIX A
OPTICAL DISK