Emerging Challenges and Opportunities for Irrigation Managers

Energy, Efficiency and Infrastructure

A USCID Water Management Conference

Albuquerque, New Mexico
April 26-29, 2011

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Preface

The papers included in these Proceedings were presented during the **USCID Water Management Conference**, held April 26-29, 2011, in Albuquerque, New Mexico.. The Theme of the Conference was ***Emerging Challenges and Opportunities for Irrigation Managers — Energy, Efficiency and Infrastructure.*** An accompanying book presents abstracts of each paper.

Irrigation managers must continuously seek to improve efficiencies as well as identify and exploit sources of water supply, water conservation and district revenues. **Technology** at all irrigation water management levels is changing, and **accountability** for water resource use is improving, in response to increasing demand and competition. The potential for water districts to generate **hydro power**, the need to upgrade **infrastructure** for existing and emerging multiple uses, and **urbanization** of water districts present both challenges and opportunities for irrigation managers. The Conference was designed to provide information on a wide variety of topics of critical interest to irrigation and water resource managers, researchers, and both technology users and developers.

The authors of papers presented in these Proceedings are professionals from academia; federal, state and local government agencies; water and irrigation districts; and the private sector.

USCID and the Conference Chairman express gratitude to the authors, session moderators and participants for their contributions.

Rhonda Skaggs
New Mexico State University
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Conference Chairman

Contents

Flow Measurement

Crop Issues and Water Banking

Poster Session

VERIFYING CONSERVATION ESTIMATES FOR ON-FARM AGRICULTURAL WATER CONSERVATION PROGRAMS

John S. McLeod[1]
Stacy Pandey[2]
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ABSTRACT

This paper presents a statistical analysis of water use practices for precision leveled rice fields irrigated by the Lower Colorado River Authority (LCRA) Lakeside irrigation division. Results from this analysis indicate there is a statistically significant difference in water use between leveled and non-leveled fields. The study also evaluated the effects of other water use factors such as other on-farm conservation measures, farmer management practices, and environmental factors. The analysis used a Hierarchical Linear Model (HLM) technique to statistically model water use and farm practice data over a 4-year period. This study is a conservation verification component of LCRA's HB 1437 Agriculture Water Conservation Program.

The House Bill 1437 (HB 1437) Agriculture Water Conservation Program is an innovative way to meet rising municipal demands in Williamson County (located in the Colorado River Basin of Texas), conserve river water used for irrigation, and maintain agriculture productivity. The grant program began in 2006, and from 2006-2009 has funded up to a 30% cost share to precision level 18,869 acres of farm land irrigated with surface water from LCRA. To date an estimated 5,567 acre-feet of water has been conserved as a result of these precision land leveling grants.

LCRA partnered with the LBJ School of Public Affairs at the University of Texas to develop the statistical model and analysis presented in this paper.

INTRODUCTION

The HB 1437 Agricultural Water Conservation Program began in 2006 and has funded up to a 30% cost share to precision level 18,869 acres of farm land irrigated with surface water from the Lower Colorado River Authority (LCRA). To date an estimated 5,567 acre-feet of water has been conserved as a result of these precision land leveling projects. The purpose of this paper is to report the results of a statistical evaluation of water conservation estimates between precision leveled and non-leveled rice fields in the LCRA's Lakeside irrigation division (Figure 1).

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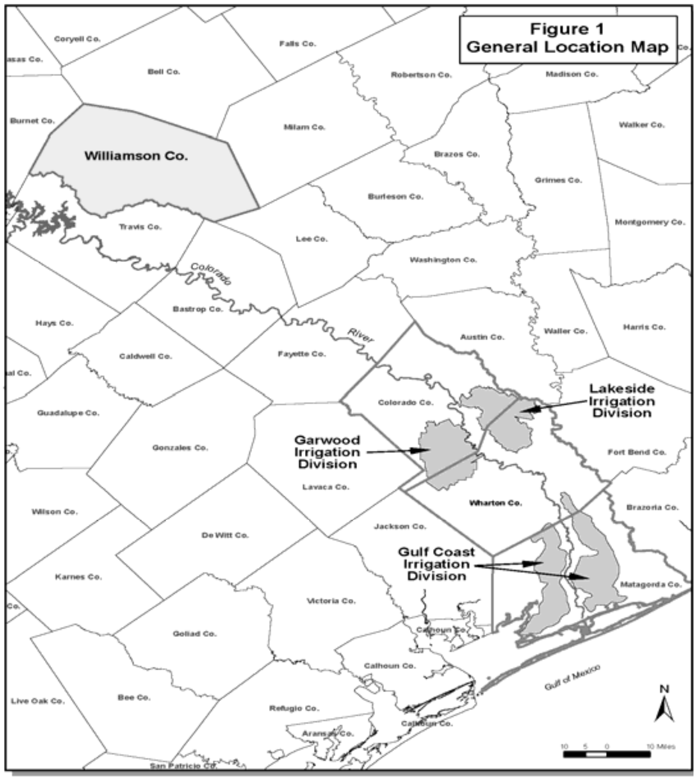


Figure 1. General Location Map

The LCRA is a conservation and reclamation district created by the Texas Legislature in 1934. LCRA supplies electricity for Central Texas, manages water supplies and floods in the lower Colorado River basin through the operation of six dams, manages three irrigation divisions (Lakeside, Garwood, and Gulf Coast), develops water and wastewater utilities, provides public parks, and supports community and economic development in 58 Texas counties.

PROGRAM OVERVIEW

The House Bill (HB) 1437 Agriculture Water Conservation Program is an innovative way to conserve agricultural water, meet rising municipal demands, and maintain agricultural productivity. A bill, HB1437, passed by the Texas Legislature in 1999, authorized the LCRA to transfer up to 25,000 acre-feet of water annually to Williamson County, if the transfer results in “no net loss” of water to the lower Colorado River basin. "No Net Loss” is generally defined as the hydrologic condition where the volume of water transferred is equivalent to the volume of water conserved within the LCRA irrigation divisions.

The bill also established a conservation surcharge on the transferred water to fund on-farm and in division agricultural conservation projects within the LCRA irrigation divisions. Additional details of the program history and legislation are available at www.hb1437.com

This program is a major part of the LCRA's water conservation program for agricultural uses. The program joins individual producers, local soil and water conservation districts, and the NRCS in a collaborative effort to conserve water. The goals of the HB 1437 program are to: 1) Reduce agricultural use of surface water; 2) Plan and implement conservation projects to fulfill obligations of the HB 1437 water sales contract and interbasin transfer permit; 3) Provide grants from the Agricultural Water Conservation Fund to implement water conservation projects; and 4) Provide program performance and conservation metrics to the LCRA Board, water customers, and the public.

Demand Projections for HB 1437 Water

The water demand projections were developed by the Brazos River Authority (BRA) and its customers and are reviewed and updated annually. Figure 2 compares the HB 1437 water demands used to develop the current HB 1437 implementation plan with the updated demand projections recently provided by BRA and their customers. The updated projections indicate an initial delay in demand, relative to the previous projections, followed by a more uniform growth.

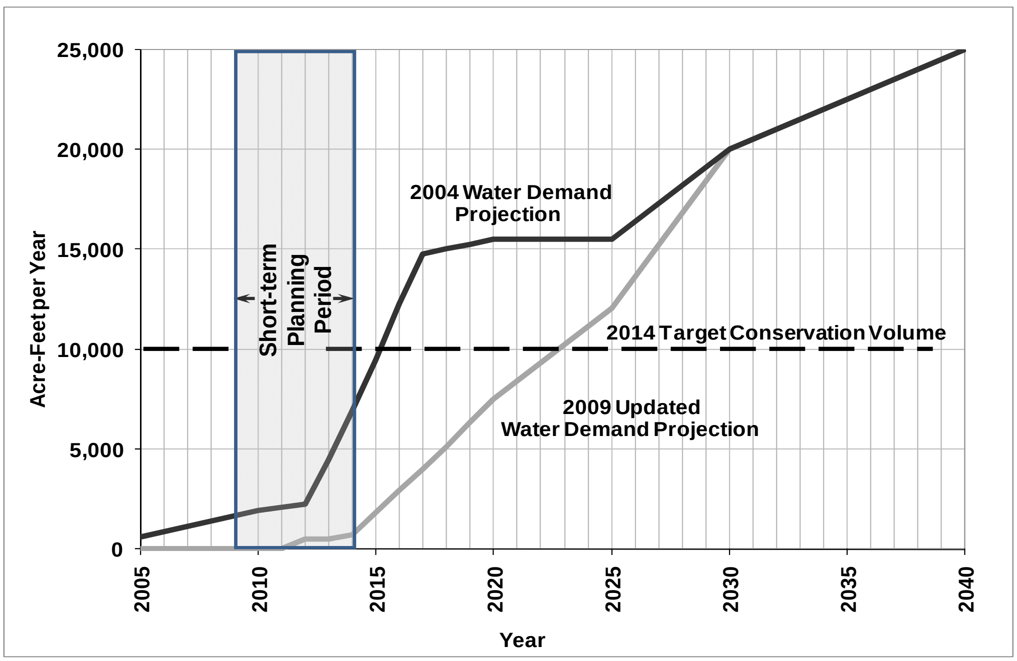


Figure 2. Water Demand Projections for HB 1437 Water

Program Plan

The current program plan includes a series of on-farm and in division conservation projects and studies to be completed during the period 2009 to 2014. The goal of this short-term plan is to conserve 10,000 acre-feet of HB 1437 water per year for transfer to Williamson County by 2014. This target provides for development of conservation improvements 4 to 6 years ahead of their need while accounting for other uncertainties, such as reliability of conservation during drought. A summary of the HB 1437 program plan is presented in Table 1.

Table 1. 2010-2014 Conservation Projects and Program Costs

On-Farm Projects	In-Division Projects	Studies and Management
Precision level 12,500 acres of farmland (2,500 acres per year)	Implement volumetric measurement in the Garwood Irrigation division Retrofit to automate eleven canal check structures with centralized control in the Gulf Coast Irrigation Division	Conservation verification and monitoring
Construction Cost - $1.2 million	Construction Cost - $1.6 million	Oversight and customer communication Program administration
Total cost: $8.0 million **Funding sources:** Ag Fund - $3.1 million, EQIP, USBR Grant, and TWDB Grant - $3.1 million, Farmer - $2.1 million **HB 1437 Water Available for Transfer**: 10,000 acre-feet per year		

Program Funding

The program is funded through the income stream generated from a conservation surcharge applied to the HB 1437 water sales contract. The conservation surcharge is applied to both reserved water and transferred water. Income to the Ag Fund is based on the following rates:

- Conservation Surcharge 25%
- Normal Raw Water Cost: $151/ac-ft
- Max Available Water: 25,000 ac-ft/yr
- Reserved Water Cost: $75/ac-ft

CONSERVATION VERIFICATION STUDY

Verification of the water savings from the HB 1437 program is essential to comply with the "no net loss" provision of the law, and to accurately judge the cost effectiveness of water conservation projects.

Estimating Volume of Conserved Water

Water conserved through precision leveling is estimated by multiplying the number of acres leveled times the Conservation Factor (C_f) for precision leveling.

- For example: In 2009, approximately 10,652 acres were in production saving an estimated 7,989 acre-feet of water – (10,652 acres * 0.75 acre-ft/acre leveled).
- The 0.75 acre-ft/acre conservation factor was developed based on results from field studies at the Texas A&M's Texas Agricultural Experiment Station (TAES) in Eagle Lake, Texas.

Previous Work

Studies by others have examined the role of precision leveled fields in agricultural water conservation (Goel et al. 1981, Anderson et al. 1999, Bjornlund et al. 2009, Smith et al. 2007) and have identified several factors affecting conservation estimates including: farmer's age and education, dependence on off-farm work, acres farmed, a field's ownership, the quality of land leveling work and water costs.

Current Work

In August 2009, LCRA partnered with the University of Texas at Austin LBJ School of Public Affairs to conduct a statistical analysis of water use factors for the HB 1437 water conservation program. The study evaluated four years (2006 - 2009) of water use data and other farming practices in the LCRA's Lakeside irrigation division. The goals of this study included:

- Determine the extent to which precision land leveling explains on-farm water use;
- Identify other factors that affect water use such as temperature, rainfall, duration of crop season, and other water conservation measures; and
- Examine how these water use factors operate at the field level as well as among groups of fields managed by the same farmer.

Initial Analysis. An initial look at comparing water use between leveled and non-leveled fields within one crop season indicated that the data is normally distributed, and that there is a statistically significant difference in water use between leveled versus non-leveled fields using Student t-tests statistics. Findings from this initial analysis also identified the need to:

- Consider multiple years in the analysis;
- Incorporate other variables to extend the statistical analysis to a complete model, reducing or eliminating the effects of confounding factors (other conservation or management practices) measured along with the variable of interest (precision land leveling); and
- Account for the lack of independence between observations, which is an assumption required when using Student t-test statistics, by specifying a model that incorporates clusters of fields at the farmer/ownership level.

HLM Model. The LBJ School developed a series of Hierarchical Linear Models (HLM) to sort out the effects of factors contributing to water use. HLM analysis allows for both correlation between observations and correlation through time.

HLM models have several advantages: They allow comparisons across multiple years, incorporate all field data even when a rice field is not in production every year, and provides a robust data structure suitable for small sample sizes. Additional details of the model development are presented in the final report: "Statistical Testing for Precision Graded Verification" (http://www.lcra.org/library/media/public/docs/water/hb1437/LBJ_Final_Interim_Report_12-2010.pdf)

A graphical representation of the model is presented in Figure3. The initial model consisted of three levels and 17 factors: Level 1 – The Crop Season (TIME) to test the predictive relationship between year-to-year variation and field water use; Level 2 - FIELD tests the predictive relationship between specific field characteristics and water use; and finally, Level 3 - FARMER which tests the predictive relationship between farmer characteristics and water use. Table 2 presents the general form of the regression equations used in the analysis.

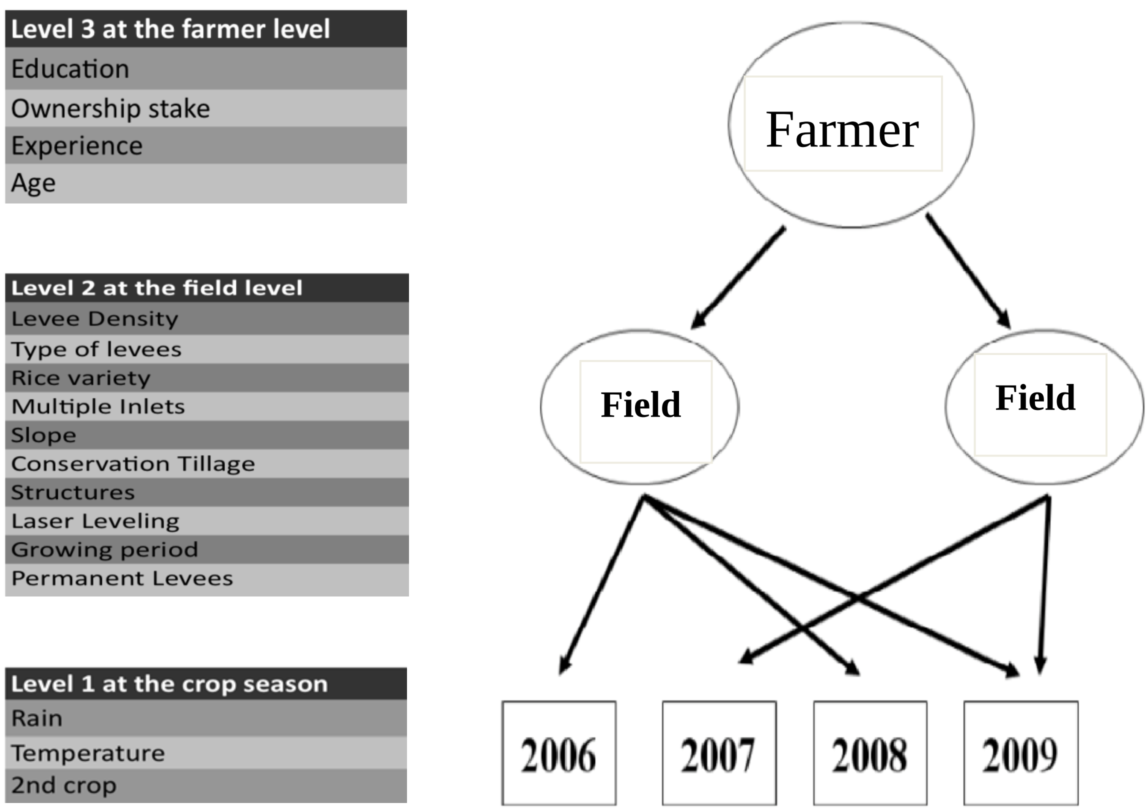


Figure 3. Graphical Depiction of the HLM Analytical Approach

Table 2. General HLM Linear Regression Equations [1]

Level 1:

$$WATER\_DEMAND_{tik} = \pi_{0ik} + \pi_1 RAIN_t + \pi_2 TEMP_t + \pi_3 SCROP_t + \varepsilon_{tik}$$

Level 2:

$$\pi_{1t} = \beta_{10t}$$
$$\pi_{2t} = \beta_{20t}$$
$$\pi_{3t} = \beta_{30t}$$

Level 3:

$$\beta_{04k} = \gamma_{010} + u_{01k}$$

$$\beta_{06k} = \gamma_{020} + u_{02k}$$

[1] (http://www.lcra.org/library/media/public/docs/water/hb1437/LBJ_Final_Interim_Report_12-2010.pdf)

Model Hypothesis

Table 3 summarized the research questions and model hypotheses for this statistical study to explore the effect of precision leveling on field water use, and the complex interaction between the contributing factors including weather conditions, fields characteristics and farming practices.

Table 3. Hypotheses: What Factors Affect On-farm Water Use?

Research Questions	Factors	Hypothesis
How do annual characteristics affect on-farm water use?	**Rain**	A relatively distinct wet crop season will reduce the water usage of fields.
	Temperature	A relatively hot crop season will increase the water usage of fields.
	Second Crop	During the second crop, fields have lower water usage than during the first crop.
How do the characteristics of fields affect on-farm water use?	**Precision Leveling**	Precision-leveled fields have lower water usage than non-precision leveled fields.
		The effect of precision leveling differs according to the levee system present in a field.
	Levee-System	When fields have a straight-levee system, the water usage of fields decrease.
	Multiple Inlet	The effect of a straight-levee system on the water use of fields differs according to the levee density in each field.
		The effect of a straight-levee system on the water use of fields differs according to the number of multiple inlets present in a field.
	Structures	Fields with four or more multiple inlets have lower water usage than fields with three or less multiple inlets.

Research Questions	Factors	Hypothesis
		As the number of measured structures in a field increases the water usage of that field decrease.
How do the characteristics of farmers affect on-farm water use?	**Growing Period**	An extended growing season leads to higher levels of water use while a shorter growing season results in lower on-farm water use.
	Ownership	The water usage of contract holders who farm their land is lower than the water usage of contract holders who rent their land.
	Rice Variety	The water usage of farmers cultivating hybrid rice is higher than those planting conventional cultivars.

Data Sources

This study uses three data sources: 1) LCRA contract and billing data from LCRA's WAMS (Water Application Management System), 2) Farmer Survey Data - information collected from the farmer survey developed for this study; and 3) Weather data. A description of each is presented below.

Water Application Management System (WAMS) Database. LCRA staff collects information about field characteristics through its annual water contracting process. Data collected in this system include: information for first and second crop, contract name, field name, year the field was in production, whether the field was in production during the 2nd crop, field acreage, field water use (ac-ft) and delivery structure information.

Table 4 presents a summary of the fields included in the study and includes approximately 195 fields each year over four years of data. The number of precision-leveled fields funded through the HB1437 program increased from 6 (2006), to 13 (2007), to 32 (2008), to 28 (2009).

Table 4. Total Fields in Production 2006-2009 during the First Crop

Year	Total fields	Non-Leveled fields		Leveled fields	
		Fields	Percentage	Fields	Percentage
2006	178	135	76%	43	24%
2007	174	120	69%	54	31%
2008	201	122	61%	79	39%
2009	227	143	63%	84	37%

Source: Survey and WAMS database 2010

Farmer Survey Data. A farmer survey instrument was developed and mailed to existing irrigation customers in the Lakeside irrigation division to collect information about conservation measures in place, water usage, and management decisions that affect water use. It focused on fields in production from 2006 to 2009.

The survey was divided into three main sections. Part 1, General Information, elicited information about the respondent including years of farming, age and education. Part 2, Farming Practices, asked for information about the entire farming operation including off-farm work, upgrades on irrigation equipment and farmers rationale for investing on water-conserving technology. In Part 3, Field Characteristics, detailed questions were asked on farming practices and upgrades implemented by field and year.

The surveys were mailed in mid-February 2010. Follow-up phone calls were made to all non-respondents about a month after the initial mailing and an additional mail survey was sent again as needed. Reminder post-cards were sent the third week of May 2010.

Over a period of seven months, 36 surveys were completed and returned, which accounted for 59 percent of the surveys mailed, 61 percent of rice fields in production and 62 percent of the annual planted acreage. Table 5 compares field information from contract holders and survey respondents and indicates that the field survey data are representative of most rice fields when considering field size and water use.

Table 5. Representative Sample: Field Size and Water Use

WAMS DATA					SURVEY DATA				
	ACRES		WATER USE			ACRES		WATER USE	
YEAR	MEAN (ac-ft/ac)	STD DEV	MEAN (ac-ft/ac)	STD DEV	YEAR	MEAN (ac-ft/ac)	STD DEV	MEAN (ac-ft/ac)	STD DEV
2006	118.960	78.754	2.494	0.666	2006	119.239	79.378	2.472	0.581
2007	131.564	97.233	1.492	0.593	2007	134.334	91.998	1.510	0.525
2008	143.006	115.190	2.956	0.885	2008	146.049	120.685	2.912	0.806
2009	127.527	82.763	3.007	1.051	2009	126.413	80.044	2.954	0.990

Some data collected in the survey was not sufficiently complete to be used in the HLN analysis of water use characteristics. Some data on conservation measures was available from a previous study, but it was necessary to expand and validate this data due to substantial changes in field characteristics. A summary of the Field Characteristics factors included in the analysis is presented in Table 6.

Table 6. Survey Information: Field Characteristics by Year

Part of HLM analysis		Not part of HLM analysis
EXPAND & VALIDATE	NEW INFORMATION	UNRELIABLE INFORMATION
Multiple inlets	Type of levees	Failed 2nd crop
Conservation tillage	Rice variety	Row crop
Historical leveled fields	Slope	Number of flushes
	Ownership	
	Permanent perimeter levees	

Weather data. Weather data were collected from 3 stations: Eagle Lake 7 NE station, Colorado River at Altair, and Wharton station from the LCRA's Hydromet System. Weather data were averaged during the average growing season for each station. Growing season refers to the average time between the first and last water delivery of the set of fields within each weather station polygon.

RESULTS AND CONCLUSIONS

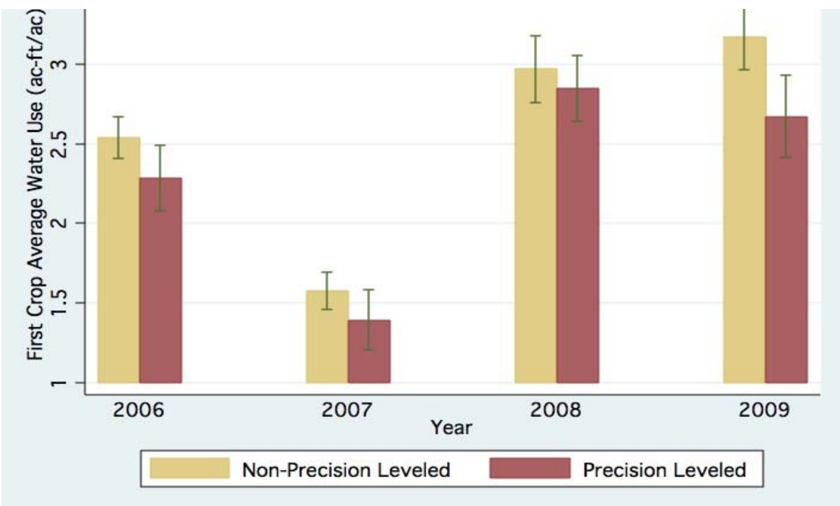
Due to the relative small sample size, some factors and hypotheses could not be tested with the HLM model. Table 7 summarizes those factors included in the HLM analysis as well as those factors excluded. A complete discussion of the results and conclusions are presented in the final report from the LBJ School and can be viewed at http://www.lcra.org/library/media/public/docs/water/hb1437/LBJ_Final_Interim_Report_12-2010.pdf

Table 7. Factors in the HLM Analysis

Part of HLM analysis		Not part of HLM analysis
FACTORS		FACTORS
Average Temperature	Multiple inlets	Permanent perimeter levees
Average Rain	Straight-levee system	Conservation Tillage
Second Crop	Levee Density	Age
Growing period	Rice Variety	Education
Precision leveling	Ownership	Experience
Structures		

The statistical analysis of the HLM modeling results show that precision leveling has both a direct and indirect effect on field water use. Figure 5 compares 1st crop water use between precision and non-precision leveled fields.

Figure 5. Comparison of Water Use in Precision and Non-Precision Leveled Fields



The results show that within a 95% confidence interval, precision leveling directly accounts for a 0.31 ac-ft/ac reduction in on-farm water use for the first crop compared to unleveled fields. The upper and lower bounds on the water saving suggest precision leveling reduces the water usage of fields by no less than 0.16 ac-ft/ac and no more than 0.46 ac-ft/ac.

The results also show that straight levees have both a direct and indirect effect on water use. The indirect effect is through the variable "precision leveling" - primarily due to the fact that fields with straight levees are more likely to be precision leveled. Results also showed that fields with straight levees exhibit lower overall water usage than fields with contour levees or a mixed-levee type system.

The results also indicate precision-leveled fields in combination with straight levees can save approximately 0.606 ac-ft/ac of water during the first crop. Using a 95% confidence interval, the upper and lower bounds of these results suggest that during the first crop water savings range from, 0.20 to 1 acre-feet less water, on precision-leveled fields with a straight levees system.

Recommendations for Future Work

The HLM statistical analysis of water use data from the Lakeside irrigation division has demonstrated it to be a suitable tool for estimating the conservation factor for precision leveling, as well as predicting the interaction with other variable contributing to water use. While much of the data is available from the LCRA contracting process, additional process refinements will be necessary to collect the necessary data to build upon the data set developed in this study.

This analysis also found that refinements to the model are necessary to improve the accuracy of these water savings. Recommendations include:

- Expand the model to include information from a fifth year of data (2010) which will allow 1st crop data to be evaluated separately from 2nd crop data,
- Include a evapotranspiration factor in the model,
- Evaluate the need for additional rain gauges, and
- Revaluate those factors considered to have unreliable data, including multiple inlets and levee density.

This research may be used to develop future guidelines for evaluating water conservation policies for the HB 1437 program and may influence the direction of implementing water-conserving technology. Additionally, water use data from the other districts will be evaluated to determine if a similar methodology can be used for LCRA's other divisions, Gulf Coast and Garwood.

REFERENCES

Agarwal, MC, and Goel AC. (1981). "Effect of field leveling quality on irrigation efficiency." Agricultural Water Management 4(4): 457-464.

Anderson, David P., and Thompson, Gary D. (1999). "Adoption and Diffusion of Level Fields and Basins." Journal of Agricultural and Resource Economics 24(1): 186-203.

Bjornlund H, Nicol L, and Klein KK. (2009). "The adoption of improved irrigation technology and management practices-A study of two irrigation districts in Alberta, Canada." Agricultural Water Management 96(1): 121-131.

Smith, M.C.; Massey, J.H.; Branson, J.; Epting, J. W.; Penington, D.; Tacker, P.L.; Thomas, J.; Vories, E.D.; and C. Wilson. (2007). "Water use estimates for various rice production systems in Mississippi and Arkansas." Irrigation Science. 25: 141-147.

"Statistical Testing for Precision Graded Verification", Final Interim Report. University of Texas at Austin. December, 2010. http://www.lcra.org/library/media/public/docs/water/hb1437/LBJ_Final_Interim_Report_12-2010.p df

IMPROVING IRRIGATION SYSTEM PERFORMANCE IN THE MIDDLE RIO GRANDE THROUGH SCHEDULED WATER DELIVERY

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Nabil Shafike[5]

ABSTRACT

Scheduled water delivery (SWD) provides the opportunity to increase overall irrigation system performance and define legitimate water use in regions without adjudication. A well-managed program of scheduled water delivery is able to fulfill seasonal crop water requirements in a timely manner, but requires less water than on-demand water delivery. In order to successfully realize SWD in an irrigation district, several components need to be addressed and developed simultaneously.

This paper will present results of on-going research in the Middle Rio Grande Conservancy District (MRGCD) related to implementation of scheduled water delivery supported by a decision-support system (DSS) and modernization of irrigation infrastructure. A DSS developed over the last four years uses linear programming to find an optimum water delivery schedule for all canal service areas in the MRGCD irrigation system. The DSS has been developed for the entire MRGCD and a significant validation effort of input parameters and model logic has been completed.

The second component for implementing scheduled water delivery is a program of irrigation infrastructure modernization with Supervisory Control and Data Acquisition (SCADA) system. Over the past six years, the MRGCD has modernized canal infrastructure and developed a SCADA system with the focus being to improve water use efficiency.

The third component in implementing scheduled water delivery is its acceptance by all water users as a matter of district policy and practice. To gain acceptance and disseminate information regarding SWD, a public outreach program was formulated that includes providing water users information through newsletters, websites, and public meetings. It also included training related MRGCD staff in the concepts and practice of scheduled water delivery and the use of related decision-support systems.

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INTRODUCTION

Irrigated agriculture in the Western United States has traditionally been the backbone of the rural economy. The climate in the American West, with low annual rainfall of 8-14 inches is not conducive to dry land farming. Topography in the West is characterized by multiple high mountain ranges which accumulate significant snowfall, interspersed with relatively dry valleys. These valleys are well suited for agriculture, with fine soils and moderate climate, but typically receive only scant rainfall. Early settlers in the region rapidly learned to farm the valleys using snowmelt runoff imported from the high mountains. In general, the peaks of the snowmelt hydrograph are stored in reservoirs, then delivered through complex canal networks as needed, allowing for irrigation throughout the summer crop growing season. Irrigated agriculture in general is a large water user that consumes roughly 80% of freshwater supplies worldwide and in the Western United States (Oad et al. 2009; Oad and Kullman, 2006). The combined demands of agriculture, urban, and industrial sectors in the past have left little water for fish and wildlife. Since irrigated agriculture uses a large and visible portion of surface water in the West, it is often targeted for increased efficiency to free water for other uses. Due to fish and wildlife concerns, and demands from a growing urban population, the pressure to reduce consumption by irrigated agriculture increases every year. In order to sustain itself and deal with external pressure for reduced water usage, irrigated agriculture has to become more efficient in its water delivery. This paper focuses on research regarding improving water delivery operations, specifically scheduled water delivery, in the Middle Rio Grande irrigation system through the use of a decision support system and SCADA technology.

Middle Rio Grande Valley

The Middle Rio Grande (MRG) Valley runs north to south through central New Mexico from Cochiti Reservoir to the headwaters of Elephant Butte Reservoir, a distance of approximately 175 miles. The valley is narrow, with the majority of water use occurring within five miles on either side of the river. The bosque, or riverside forest of cottonwood and salt cedar, is supported by waters of the Rio Grande and is surrounded by widespread irrigated farming. The Cities of Albuquerque, Rio Rancho, Belen and several smaller communities are located in and adjacent to the MRG Valley. Although the valley receives less than 10 inches of rainfall annually, in addition to a strong agricultural economy, it supports a rich and diverse ecosystem of fish and wildlife and is a common outdoor resource for communities in the region. Water supply available for use in the MRG Valley includes: native flow of the Rio Grande and its tributaries, allocated according to the Rio Grande Compact of 1938; San Juan-Chama (SJC) project water, obtained via a trans-mountain diversion from the Colorado River system; and groundwater. Water is fully appropriated in the MRG Valley and its utilization is limited by the Rio Grande Compact, which sets forth a schedule of deliveries of native Rio Grande water from Colorado to New Mexico and from New Mexico to Texas (Rio Grande Compact Commission, 1997), and between the United States and the Republic of Mexico. Water demand in the MRG Valley includes irrigated agriculture in the Middle Rio Grande Conservancy District (MRGCD), Pueblo prior and paramount and other currently un-adjudicated rights, and municipal and industrial consumption. The right to use water is goverened by a complex system of rights dating back over 400 years. Pueblo Indians in the valley have a uniqie category of water rights superceeding all others termed “prior and paramount”. In addition to these human demands, there are significant consumptive uses associated with the riparian vegetation, and reservoir evaporation. There are also river flow targets associated with two federally-listed endangered species, the Rio Grande silvery minnow (*Hybognathus amarus*), and the southwestern willow fly catcher (*Empidonax traillii extimus)* (USFWS, 2003).

Middle Rio Grande Conservancy District.

The MRGCD was formed in 1925 in response to flooding and the deterioration of irrigation works (Shah, 2001). Water diverted by the MRGCD originates as native flow of the Rio Grande and its tributaries, including the Rio Chama. The MRGCD primarily stores water in El Vado reservoir and maintains a small regulation pool in Abiquiu reservoir. A large flood control dam which forms Cochiti Reservoir sits at the head of the MRGCD service area but no irrigation water is stored there. Moving water from El Vado reservoir to the first MRGCD diversion point requires two days travel time, to the furthest downstream user requires nearly a week. The MRGCD services irrigators from Cochiti Reservoir to the northern boundary of the Bosque del Apache National Wildlife Refuge. Irrigation facilities managed by the MRGCD divert water from the river to service agricultural lands, from small urban parcels to large commercial tracts that produce alfalfa, pasture, corn, and vegetable crops. One unusual crop is green chile, which is famous throughout the United States. The MRGCD supplies water to its four divisions -- Cochiti, Albuquerque, Belen and Socorro -- through Cochiti Dam and Angostura, Isleta and San Acacia diversion weirs, respectively. Water is conveyed in the MRGCD by gravity flow through primarily earthen ditches. On-farm water management is entirely the responsibility of water users and water application is typically surface flood irrigation, either basin or furrow. The MRGCD does not meter individual farm turnouts, and ditch-riders estimate water delivery on the basis of time required for irrigation. Therefore, the quantity of water applied to fields is not measured. The total irrigated land within the MRGCD is approximately 70,000 acres. Figure 1 displays the location of the MRGCD.

During the recent drought years the MRGCD has experienced somewhat lower than normal natural flows, and reduced snowpacks have resulted in decreased reservoir storage. At the same time, increased demands have been placed on the region by flow requirements for the endangered Rio Grande silvery minnow and a rapidly expanding urban population. In order to deal with reduced water availability, the MRGCD has taken a proactive approach to be a more efficient water user and service its irrigators while simultaneously reducing river diversions. Towards this end, the division managers and ditch-riders are increasingly practicing scheduled water delivery, which is an effective way to fulfill demand with reduced available water.

Scheduled Water Delivery (SWD) is used in irrigation systems worldwide to improve water delivery and to support water conservation, and in fact was once a part of regular MRGCD practices. In SWD, lateral canals receive water from the main canal by turns, allowing water use in some laterals while others are closed. In addition to this water scheduling among laterals, there can be scheduling within laterals whereby water use is distributed in turns among farm turnouts along a lateral. By distributing water among users in a systematic scheduled fashion, an irrigation district can decrease water diversions and still meet crop water use requirements.

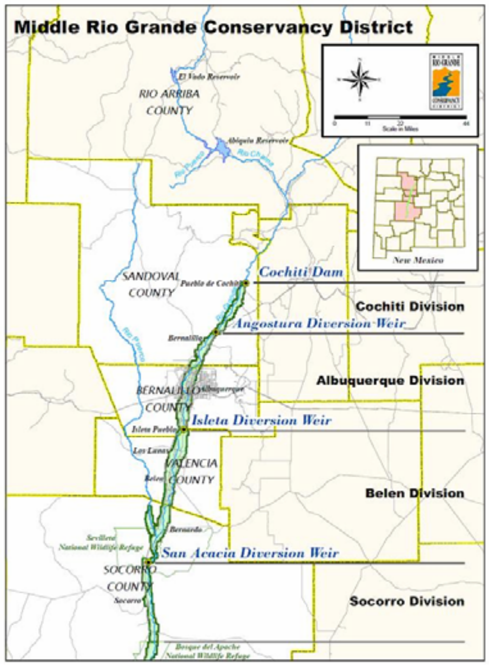

Figure 1. Middle Rio Grande Conservancy District (MRGCD)

Decision Support Modelling of Irrigation Systems

The New Mexico Interstate Stream Commission (NMISC) and the MRGCD sponsored a research project with Colorado State University to develop a decision support system (DSS) to model and assist implementation of scheduled water delivery in the MRGCD's service area. A DSS is a logical arrangement of information including engineering models, field data, Geographic Information System (GIS) and graphical user interfaces, and is used by managers to make informed decisions. In irrigation systems, a DSS can organize information about water demand in the service area and then schedule available water supplies to efficiently fulfill the demand. Figure 2 displays a conceptual view of how a DSS can be used to develop scheduled water delivery.

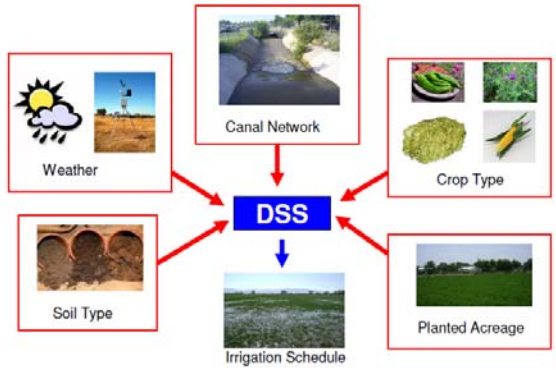

Figure 2. Conceptual View of a Generic SWD DSS

The conceptual problem addressed by a DSS for an irrigation system is how to first determine a water demand that may be expected, then how to best to route water supply in a main canal to its laterals so that the required river water diversion is minimized. The desirable solution to this problem must be "demand-driven", in the sense that it is based on a realistic estimation of water demand. But how can this demand be estimated in system such as the MRGCD, where on farm measurement is largely absent, and delivery needs are rarely communicated to water managers? For the purposes of this model, it has been found that the water demand in a lateral canal service area, or for an irrigated parcel, can be predicted throughout the season through analysis of information on the irrigated area, crop type, weather data, and soil characteristics. The important demand concepts are: 1) When is water supply needed to meet crop demand (Irrigation Timing), 2) How long is the water supply needed during an irrigation event (Irrigation Duration), and 3) How often must irrigation events occur for a given service area (Frequency of Irrigation).

Decision support systems have found implementation throughout the American West and are mostly used to regulate river flow. Decision support systems on the river level are linked to gauging stations and are used to administer water rights at diversions points. Although decision support systems have proved their worth in river management, few have been implemented for modeling irrigation canals and laterals (NMISC, 2006).

DECISION SUPPORT SYSTEM FOR THE MIDDLE RIO GRANDE

The first component in achieving scheduled water delivery in the MRGCD is the DSS. The DSS was formulated using linear programming with the use of an objective function. A detailed description of model programming can be found in Oad et al. 2009. Overall model structure consists of three modules that function in concert to calculate the most efficient irrigation water delivery.

Model Structure

The DSS consists of three model elements or modules: a water demand module, a supply network module, and a scheduling module. A Graphical User Interface (GUI) provides a means for linking the three elements of the DSS. This GUI is an interactive means for the user to access data and output for the system. The project GIS and databases are used to develop input for both the water demand and the supply network modules. Some of the input is directly linked through the GUI and some is handled externally in the DSS. Figure 3 displays the structure of the MRGCD DSS.

Water Demand Module: The water demand module of the MRGCD DSS is implemented either through the ET TOOLBOX for the Middle Rio Grande or the Integrated Decision Support Consumptive Use, or IDSCU model, a model developed over a period of years at the Colorado State University. The ET Toolbox is a web application developed by the Bureau of Reclamation that estimates real-time evapotranspiration from distributed climate stations, NexRAD precipitation data, and remotely sensed cropping patterns. Crop consumptive use is calculated using the Penman-Montieth method. The reference ET (ET_o) is calculated using weather data from the MRGCD. Crop coefficients using growing degree days are applied to the Penman-based ET_o to obtain a consumptive use for each crop type throughout the growing season. The water demand module performs these calculations to obtain a spatially-averaged consumptive use at the lateral service area level, using the distribution of crop types within each service area.

The crop irrigation requirement (CIR) is calculated by accounting for the effective precipitation using the Soil Conservation Service Method. The crop irrigation requirement is calculated on a daily basis, corresponding to the water needed to directly satisfy crop needs for all acres in the service area. The crop irrigation requirement for the service area is subsequently passed to the supply network module, where it is divided by an efficiency factor to obtain a lateral service area delivery requirement (LDR).

Based on acreages, crop types and soil types within each lateral service area, a Readily Available Moisture (RAM) is calculated. The RAM calculated in this context represents a soil water storage capacity to be filled and depleted over several irrigation cycles during the course of the irrigation season. During irrigation, it is expected that an amount of water equal to the RAM will be stored in soils, which is then depleted, due to crop water use.

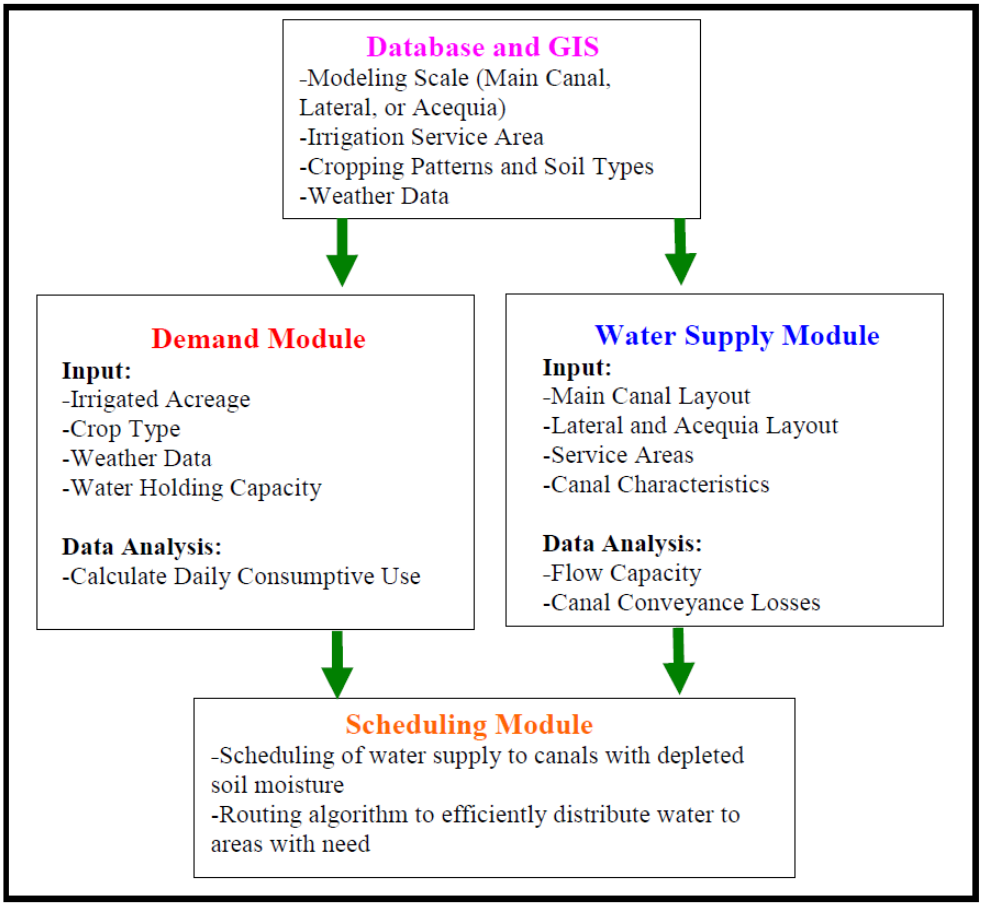

Figure 3. DSS Structure Displaying the Three Modules and Database

Supply Network Module: The supply network module represents the layout of the canal conveyance system, its physical properties, supply to the conveyance network, and the relative location of diversions from the network to the lateral service area. The layout of the conveyance system is specified through a user-designed link-node network. Through the DSS GUI, a user can create different types of nodes such as inflows, demands and return flow nodes. The link-node network represents the connections between canals or laterals and demands for water at each service area. Figure 4 displays the supply network of the Peralta Main Canal service area.

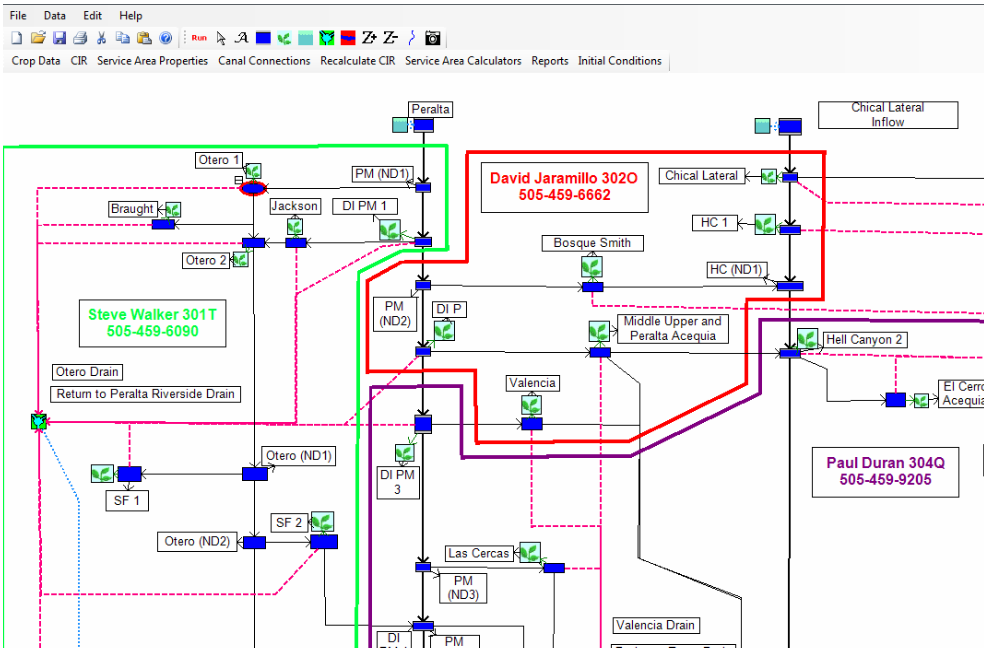

Figure 4. Representation of DSS Supply Network

<u>Irrigation Scheduling Module:</u> The irrigation scheduling module can be used to plan water deliveries to meet crop demand at the lateral and at the main canal level. The module calculates and displays a schedule for the laterals on a given main canal. This schedule indicates how many laterals can be run at a time, how long each lateral should run and how often. The module is currently set up to run on a daily time step. This module calculates the daily irrigation schedule using mass balance equations and a linear programming solver. The approach is based on the consideration that the farm soil root-zone is a reservoir for water storage, for which irrigation applications are inflows and CIR is an outflow. Figure 5 displays a calendar developed by the irrigation scheduling module. The DSS has undergone extensive calibration and validation and has proved to be reliable and able to create irrigation schedules based on crop demand (Kinzli et al. 2010; Kinzli 2010; Oad et al. 2009).

May

Sun	Mon	Tues	Wed	Thur	Fri	Sat
					1 35 cfs	2 35 cfs
3 35 cfs	4 35 cfs	5 35 cfs	6 35 cfs	7 35 cfs	8 35 cfs	9 35 cfs
10 35 cfs	11 35 cfs	12 35 cfs	13 20 cfs	14 20 cfs	15 10 cfs	16 10 cfs
17 10 cfs	18 10 cfs	19 10 cfs	20 10 cfs	21	22	23
24	25	26 35 cfs	27 35 cfs	28 35 cfs	29 35 cfs	30 35 cfs
31 35 cfs						

Figure 5. Irrigation Calendar Developed Using the Scheduling Module

SUPERVISORY CONTROL AND DATA ACQUISITION (SCADA) SYSTEM

Along with the development of the DSS to aid in scheduled water delivery the MGRCD has been proactive in updating aging infrastructure as well as incorporating advanced technology such as SCADA (Supervisory Control and Data Acquisition) for more precise and controlled water delivery. This updated technology will allow for the control that is necessary for implementing the irrigation schedules recommended by the DSS and represents the second component which is a prerequisite for achieving scheduled water delivery.

Over the past 12 years, the MRGCD has developed a SCADA system with the focus being to improve water use efficiency throughout the Middle Rio Grande Valley (Gensler et al. 2009). The MRGCD program of measurement and automation was built entirely in-house using inexpensive components due to budget constraints. Using traditional SCADA components as well as adaptations of technology from other industries makes the MRGCD SCADA setup unique. The developed SCADA system consists of five main components:

- Water Measurement Structures
- Automated Control Structures
- Instrumentation
- Telemetry
- Control Software

Water Measurement Structures

Water measurement is the single most important component of the MRGCD's efficiency improvement program, since all operational decisions require sound knowledge of available water supplies and the demand throughout the system. When the MRGCD was initially constructed in the 1930's, considerable thought to water measurement was given but over the years, gauging stations equipped with measurement instrumentation gradually deteriorated and quality of flow records declined.

In 1996, the MRGCD was operating only 15 gauges on 1,200 miles of canals. The following year, the MRGCD officially embarked upon its modernization program. The construction of new flow gauges was the first step in this program. New gauges were constructed at key points in the canal system, notably at diversion structures, primary canals, and at return flow points. Over time, the measurement program was expanded to second and even third tier canals. Efforts were also made to improve the quality of measurements. Open channel gauging sites with no control structures gave way to site specific measuring structures. A variety of flow measurement structures were built in the MRGCD and include sharp crested weirs, broad crested weirs, adjustable weirs and Parshall flumes. Current MRGCD design standards specify that new gauges are constructed with broad-crested weirs using WINFLUME for design and calibration. Currently, MRGCD is operating over 100 gauges.

Automated Control Structures

With the advent of better data collection, it became apparent to the MRGCD that automated control was necessary. Data from gauges revealed that many operational problems occurred because canal operators could not be physically present at all times. Automation began with an experimental effort at a wasteway that had been fitted with a Langemann gate (Figure 6) for water measurement. The MRGCD built the prototype electronic controller and created the control software for this first automated gate, borrowing heavily from Bureau of Reclamation experience in Utah. Success and invaluable experience from the first automated structure led to installation of over 40 additional automated structures using commercial control products.

Most of the MRGCD's recent automation efforts have involved the installation of Langemann overshot gates (Aqua Systems, 2006). The majority of these can be easily retrofitted to existing structures, though some involve the construction of new check or heading structures. The Langemann Gate has the capability to maintain a constant upstream water level as a check structure or it can provide a constant flow rate to downstream users (Figure 6). The Langemann gate is equipped with solar panels to power both gate operation and telemetry units. The gates employ integrated electronic controllers built around Control Design Units (RTU's) and Aqua Systems 2000 software. Langemann gates in the MRGCD are used as checks, turnouts, spillways, and diversion structures.

6. Langemann Gate

Some existing undershot radial gates have also been automated. Conversion involves selection of a gearbox, motor, and controller. Some in-house fabrication is involved to adapt the drive unit to the existing gate hoist shaft. Early conversion attempts used an AMI controller supplied by Aqua Systems 2000, but recently the MRGCD has used the Control Design RTU, which can be programmed to calculate flow through automated radial gates. Though not as simple as overshot gates, this is useful for setting target bypass flows at diversion structures for endangered species flow requirements.

Instrumentation

Flow measurement and automated control must include some level of instrumentation. In the 1930's, a float in a stilling well driving a pen across a revolving strip of paper was adequate. In fact, at the beginning of modernization efforts, the MRGCD was still using 15 Stevens A-71 stage recorders. Diversions into the canal system were only known after the strip charts were collected and processed at the end of the irrigation season.

Modernization meant a device was needed to generate an electronic output that could be digitally stored or transmitted. This provided instantaneous real time data so that efficient real time water management decisions could be made. Initially, floats and shaft encoders were used for this purpose, providing input for electronic data loggers. Experimentation with submersible pressure sensors soon followed, and these have been generally adopted, although a number of shaft encoders are still in use. Recently, sonar sensors have been used satisfactorily at a number of sites. The MRGCD has learned that different situations call for specific sensor types and sensors are selected for applications where they are most appropriate. It has also been learned that the sensor is usually the weakest point at any automation site and should be considered carefully.

Telemetry

Data from electronic data-loggers was initially downloaded manually and proved to be only a minimal improvement over strip chart recording, though processing was much faster. To address data downloading concerns, telemetry was adopted to bring the recorded data back to MRGCD headquarters at regular intervals. The MRGCD's initial exposure to telemetry was through the addition of GOES (Geo-stationary Orbiting Earth Satelite) transmitters to existing electronic data loggers. This method worked reliably, but presented limitations. Data could only be transmitted at regularly scheduled intervals. Of greater consequence was that the GOES system, at least as used by the MRGCD, was a one-way link. Data could be received from gauging stations, but not sent back to them.

A second approach was the use of cellular telephone service, what was commonly called "CDPD" technology at the time. This solved the problem of the one-way data link, as a site could be contacted at anytime, or as often as desired. Unfortunately this instant communication was accompanied with a recurring monthly fee on a per site basis. When this involved only a couple of sites, this was manageable, but as the MRGCD contemplated having hundreds of sites, each incurring monthly charges into the indefinite future, it was obvious that this approach had inherent and significant financial disadvantages. Also, power consumption with this technology was surprisingly high, requiring considerable investment in solar panels.

To address the rising cost of telemetry using cell phone service, experiments with FM radio telemetry were conducted. These began as a way to bring multiple stream gage sites to a central data logger, which would then be relayed via GOES to MRGCD. First attempts with FM radio were not encouraging. This technology proved to have a steep learning curve, and the MRGCD was committed to doing an “in-house” installation. However a successful system was eventually developed, and recurring costs quickly dwindled to near zero. Today, the installation cost has been reduced to approximately $2500 US per site, and operation is essentially free. Installations are expected to have a practical life of about 10 years.

As this use of FM radio telemetry (licensed 450 mHz) expanded, and knowledge of radio telemetry grew, it was soon realized that data could be directly transmitted to MRGCD headquarters without using the GOES system. This led to what is one of the more unique features of the MRGCD telemetry system. The data link proved so reliable, that there was no longer a need to store data on site, and the use of data loggers was mostly discontinued, the exception being weather stations where considerable on-site processing of data is performed. In effect, a single desktop computer at the MRGCD headquarters has become the data-logger for the entire stream gauge and gate system, being connected to sensors in the field through the FM radio link. Three repeater sites are used to relay data up and down the length of the valley, with transmission up to 75 miles. Also, this has the benefit of being a 2-way link, so various setup and control parameters can be transmitted to devices along the canals.

The MRGCD telemetry network consists exclusively of Control Design RTU's. Several different types of these units are used, depending on the application. The simplest units contain only a modem and radio, and transmit collected and processed weather station data from Campbell Scientific CR10X dataloggers.
The majority of the RTU's contain a modem, radio, and an input/output (I/O) board packaged into a single unit. Sensors can be connected directly to these and read remotely over the radio link. A variety of analog (4-20ma, 0-20ma, 0-5v) and digital (SDI-12, RS-485) output devices can be accommodated this way. Another type includes a programmable (RP-52 BASIC) controller. This type is used for all automatic control sites and places where unusual processing of sensor outputs such as averaging values, combining values, or timed functions, are required. At the present time, the MRGCD telemetry network gathers data from 40 stream flow gages and 18 ag-met stations, and controls 70 automated gates, which also produce flow measurements. Figure 7 represents the early MRGCD telemetry network and Figure 8 the newest iteration of the telemetry network for one of the four MRGCD divisions.

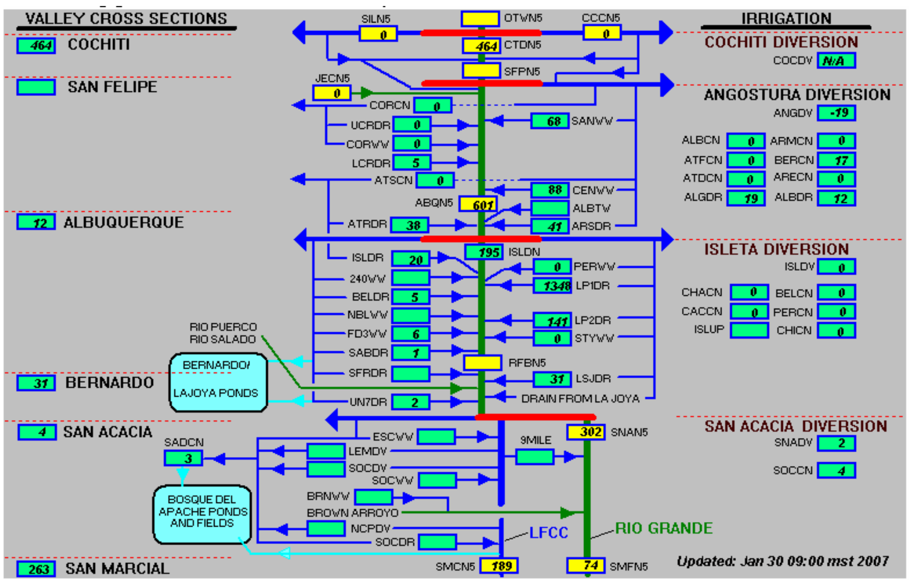

Figure 7. Early MRGCD Telemetry Network Representing Entire System

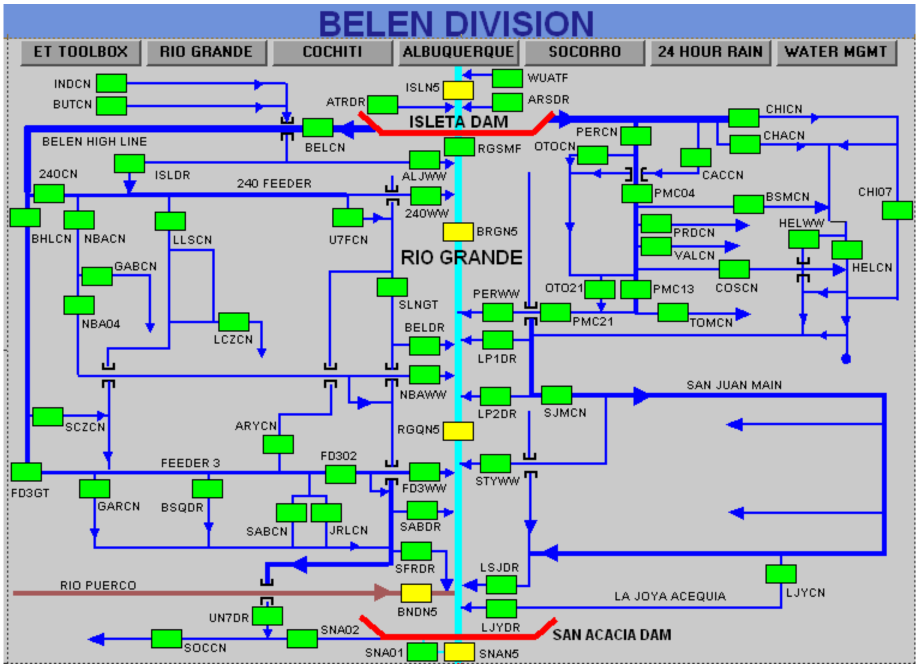

Figure 8. Newest Iteration of the MRGCD Telemetry Network Representing 1 of 4 Divisions

Control Software

Measurement, automation, and telemetry components were developed simultaneously, but largely independent of one another. While each component functioned as expected, components did not exist as a harmonious whole, or what could truly be called a SCADA system. The missing component was software to tie all the processes together. There are a variety of commercially available software packages for such use and the MRGCD experimented with several. Ultimately, the MRGCD chose to purchase the commercial software package Vsystem and to employ the vendor Vista Controls to develop new features specific to the control of a canal network. Installation and setup was done by the MRGCD.

This system, known affectionately as the Supervisory Hydro-data Acquisition and Handling System (SHAHS, named after Mr. Subhas. K. Shah), gathers data from RTU's on a regular basis. With the capability to define both timed and event driven poll routines, and specify a virtually unlimited number of RTU’s and MODBUS register locations, virtually any piece of information can be collected at any desired time. The Vsystem software can process data through a myriad of mathematical functions, and combine outputs from multiple stations. Vsystem also incorporates the ability to permanently store data in its own internal database, Microsoft® Structured Query Language (SQL) databases, or export data in other formats. Data can be displayed in a user-created graphical user interface (GUI) which MRGCD water operations personnel use to monitor water movement. The screens can also execute scripts to generate data, control parameters, control gate set points, and monitor alarm conditions for automated control structures. Finally, the GUI's can be used to control automated structures by transmitting new parameters, setpoints, and flowrates. With the simultaneous development of the MRGCD DSS and SCADA system, the implementation of scheduled water delivery based on crop demand could be realized.

Linking DSS and SCADA

Implementation of SWD was made easier by incorporating the DSS into the MRGCD SCADA System. This involved converting the DSS output into a data stream format that was compatible with the MRGCD Vsystem software. The DSS gives MRGCD operators a required irrigation delivery on a lateral level based on crop demand, as well as the timing of that irrigation. The required delivery and timing is imported into the graphical user interface (GUI) of the MRGCD SCADA system daily, so that actual deliveries along the canal system can be compared to the required deliveries. The GUI allows water managers to remotely change automated gate settings so that actual diversions closely represent water requirements. This provides better water management within the MRGCD and allows for a minimized river diversion as the required and actual diversion values converge. Figure 9 displays the MRGCD SCADA screen with actual deliveries and DSS recommendations.

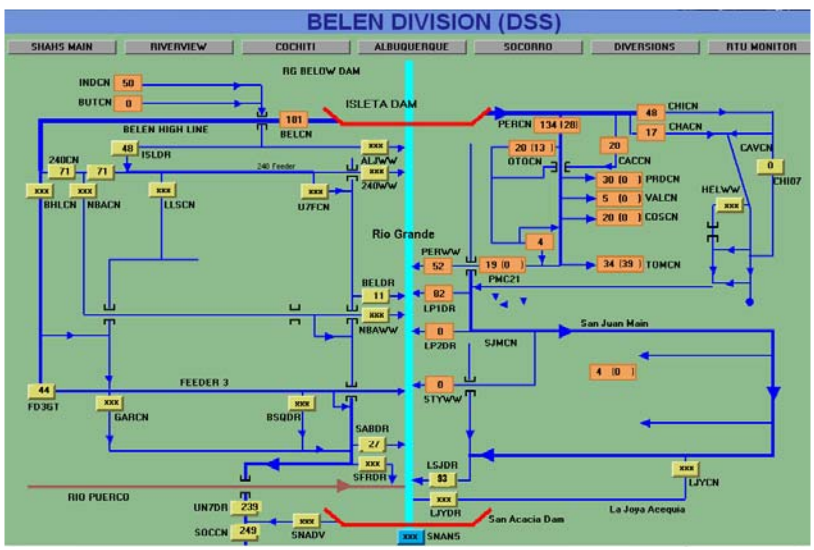

Figure 9. MRGCD SCADA Screen with Actual Deliveries and DSS Recommendations

IMPLEMENTATON OF DSS

The final component of achieving scheduled water delivery in the MRGCD was an in depth public outreach campaign. The adoption and acceptance of scheduled water delivery by the MRGCD and its water users is closely tied to understanding the principles and the benefits that this more intensive management provides. Public outreach is a timely and effective strategy for disseminating information and a necessity if water users are to accept the policy of scheduled water delivery. The program was designed to provide education and information to MRGCD water users. The information included the need to practice scheduled water delivery, that schedules are based on crop water requirements, how it will be implemented, and that it leads to fair and efficient water distribution for all concerned. Additionally, a major goal of the public outreach program was to get feedback and comments from water users and address concerns that they might have with scheduled water delivery.

There were two broad categories of information that needed to be conveyed and discussed with the MRGCD water users. The first was information related to the science, policy, and practice of scheduled water delivery as compared to the historic practice of continuous canal water delivery. The second category was the explanation of the tools, such as the DSS and SCADA, available to the MRGCD to effectively facilitate and implement scheduled water delivery.

The first step in public outreach was providing information on scheduled water delivery and the associated technology on the MRGCD website. The information provided explains the DSS and the practice of scheduled water delivery under a section of the MRGCD website that is devoted solely to the DSS and water scheduling. Figure 10 displays the links on the MRGCD homepage www.mrgcd.com and an article about the DSS.

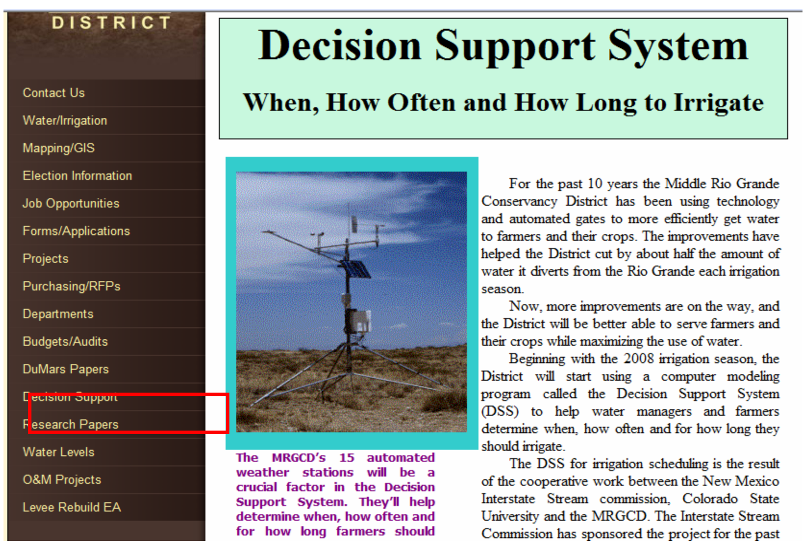

Figure 10. Article Explaining DSS on MRGCD Website

The second step of the public outreach program was including an article about scheduled water delivery in the MRGCD newsletter that gets delivered every two months. The article in the newsletter was entitled, "Computer Irrigation Scheduling Software to Remove Guesswork for Irrigators," and was delivered to over 50,000 water users, property owners, and other stakeholders in the Middle Rio Grande Valley. The article was also posted on the MRGCD website and linked to the Decision Support Section of the website. Developments regarding scheduled water delivery are periodically inserted into the newsletter to inform farmers about any changes or progress.

The third and key component of the public outreach program has been to conduct outreach meetings with water users throughout the MRGCD. Large scale public outreach meetings have been held in the Belen, Socorro, and Albuquerque Divisions. Small scale neighborhood meetings were held in the more urbanized sections of the MRGCD to deal with the higher population density. These meetings were advertised in the MRGCD newsletter and personal invitations were sent to water users resulting in excellent turnout. Smaller meetings and presentations have also been held at local farms, Workshops, and at various times and places as requested by irrigators throughout the MRGCD divisions. These meetings provided a productive venue to educate farmers about scheduled water delivery, modernization efforts, and the DSS. The meetings also provided the opportunity to inform water users about future plans in the MRGCD. Additionally, water users were able to ask questions, voice concerns, offer valuable suggestions, and provide information critical to successfully implementing scheduled water delivery. One unexpected benefit of the outreach meetings has been that reporters have been present at several of the meetings which in articles published in the local Newspapers. Three articles have now been published describing scheduled water delivery, its benefits, and the technology being used to implement scheduling.

The fourth aspect of the public outreach campaign has been to gain the support of the MRGCD Board of Directors. Presentations of scheduled water delivery and the DSS have been made to the MRGCD Board on four occasions and have been received well. The MRGCD CEO/Chief Engineer provided valuable political and practical insight for gaining support from the public, as well as elected officials, for the implementation of scheduled water delivery. The MRGCD Board understands the need for scheduled water delivery and supported the use of the DSS to develop water delivery schedules beginning in 2008. At a recent meeting the board re-emphasized their complete support of scheduled water delivery practice utilizing the DSS as an advisory tool. In tandem, the MRGCD water policy has been placed on the website in order to clarify any confusion. The policy states that water for irrigation must be scheduled with the ditch-rider and that rotational scheduling will be implemented during times of water shortage. Such political support has been invaluable in gaining water user acceptance of scheduled water delivery.

The fifth aspect of implementing scheduled water delivery and the DSS has been the training of ditch-riders and water management personnel. For the DSS to be accepted by the MRGCD, it was necessary to have the water operations personnel running the DSS and creating water delivery schedules. The training of the ditch-riders consisted of education in regards to the scientific principles used in the DSS, a tutorial on how to develop schedules with the DSS, and training on the use of soil moisture sensors. For the 2009 irrigation season ditch-riders were given portable Aquaterr™ soil moisture meters to ensure that water delivery schedules were not adversely affecting crop growth in their service areas.

The five steps of the public outreach campaign have resulted in positive progress towards district wide scheduled water delivery. First, MRGCD water users can easily access information about relevant issues such as irrigation water delivery and scheduling of their water supply. The public outreach program also provided a much needed opportunity for water users and managers to meet and discuss issues related to an extremely precious resource – irrigation water. Before this program, there was no structured process whereby the water users could meet as a group and discuss their concerns and questions with their water provider.

Second, the public outreach program has resulted in the limited implementation of the DSS. The DSS is currently being used to develop irrigation schedules in the form of a calendar which determines when certain lateral canals need to be running to meet crop demand. The area over which the implementation is occurring represents roughly 14% of the total irrigated acreage in the MRGCD. The calendars are allowing irrigators to plan their water use and provide for a more reliable water delivery method. Without calendars or scheduling, water deliveries were often unreliable and unpredictable. Creating schedules that address water deliveries in advance allows managers to adjust deliveries upstream accordingly.

Overall, scheduling has been successful in several aspects. The schedules have resulted in increased head in the irrigation ditches, increased reliability in water delivery, and efficiency improvements. From a management standpoint, the DSS has resulted in a much more organized protocol for delivering water by determining water delivery targets in advance, which allows managers to adjust deliveries upstream accordingly. Over time, scheduled water delivery and the MRGCD DSS could be used throughout the entire district.

RESULTS

Using scheduled water delivery and infrastructural improvements, the MRGCD has been able to significantly reduce river diversions. Historically, the MRGCD diverted as much as 600,000 AF/year from the Rio Grande. Over the last 6 years, diversions have averaged less than 350,000 AF/year. This is a significant accomplishment as the MRGCD has been able to reduce diversion to meet fish and wildlife concerns, while still providing the needed water to irrigators. Figure 11 displays the decreasing trend in total MRGCD river diversions. These obvious changes are attributable to the SWD, supported by the measurement, automation, and management program undertaken by the MRGCD. What is not as well illustrated is the role that the DSS plays in this. To a very large degree, SWD has been involuntary, forced upon irrigators by circumstances, and of course has met with considerable resistance. In the absence of a tool such as the DSS, SWD may or may not be meeting crop demands properly. The recent incorporation of the DSS into the program will allow optimization, not just from the standpoint of water efficiency, but also in the farm productivity area ensuring acceptance by irrigators. Its expansion is really not expected to result in significant additional water savings, but instead is expected to increase efficiency by maximizing crop yield and productivity.

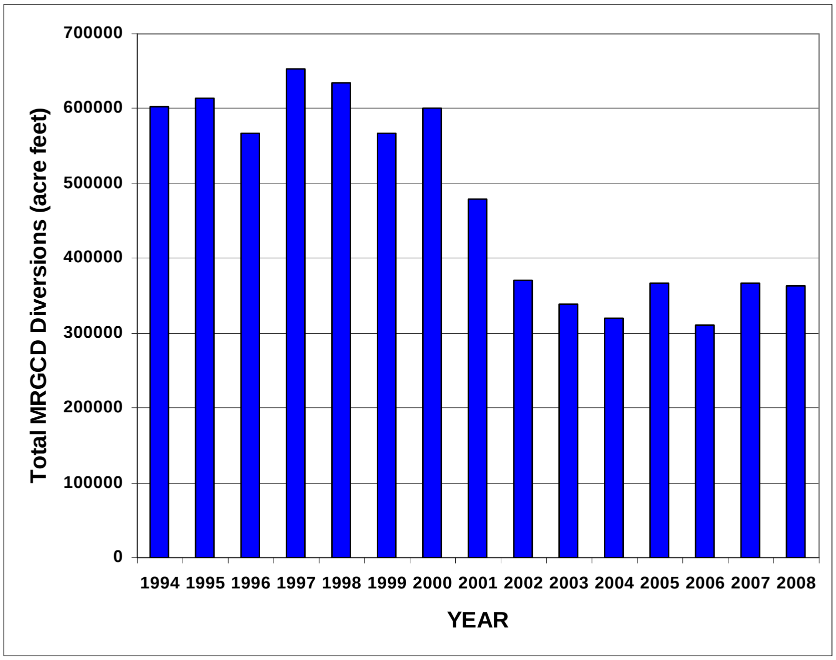

Figure 11. MRGCD River Diversions by Year

CONCLUSIONS AND FURTHER RESEARCH

An integrated decision support system and SCADA system for the Middle Rio Grande Conservancy District has been developed that models the canal network and can compute water delivery options for optimum water delivery scheduling. The system additionally allows for local and automated controls which can be actuated at a central office. The linking of the MRGCD SCADA and the DSS provides operators with a required irrigation delivery on a lateral level based on crop demand as well as the timing of that irrigation. This provides better water management within the MRGCD and allows for a minimized river diversion, while eliminating potential demand/supply mismatches. The system has also resulted in increased head in the irrigation ditches, increased reliability in water delivery, efficiency improvements, and improved protocol for anticipating future water demands. The public outreach campaign has been successful in educating water users on the principles of scheduled water delivery as well as providing much needed opportunities for water users and water managers to discuss water delivery issues.

Future plans for scheduled water delivery in the MRGCD include expanding the use of the DSS and scheduled water delivery. Plans also include further modernization efforts and continued public outreach and training programs to facilitate scheduled water delivery. Through expanded implementation of scheduled water delivery and the DSS the MRCGD will further reduce river diversions, while continuing to sustain irrigated agriculture in the Middle Rio Grande Valley.

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The authors would like to thank Subhas Shah, the MRGCD Board of Directors, the staff of the MRGCD, the New Mexico Interstate Stream Commission, the New Mexico Office of the State Engineer, the Middle Rio Grande Endangered Species Act Collaborative Program, the National Science Foundation, the United States Army Corp of Engineers, and the United States Bureau of Reclamation. Also, the exceptional support of Jim Conley at IC Tech, Gerald Robinson and Lee Allen at Aqua Systems 2000, and Cathy Laughlin, Virginia King, and Peter Clout of Vista Control Systems is graciously recognized.

REFERENCES

Aqua Systems 2000 Inc (2006). www.aquasystems2000.com Accessed 2/28/2006.

Barta, R. 2003. Improving Irrigation System Performance in the Middle Rio Grande Conservancy District. Masters Thesis. Department of Civil Engineering, Colorado State University. Fort Collins, Colorado.

DuMars, C.T. and S.C. Nunn (eds.). 1993. Working Document: Middle Rio Grande Conservancy District Water Policies Plan. Middle Rio Grande Conservancy District. Albuquerque, New Mexico.

Gensler, D. Oad, R. and Kinzli, K. (2009). "Irrigation System Modernization: A Case Study of the Middle Rio Grande Valley." ASCE Journal of Irrigation and Drainage. 135(2): 169-176.

King, J.P., A.S. Bawazir, and T.W. Sammis. 2000. Evapotranspiration Crop Coefficients as a Function of Heat Units for Some Agricultural Crops in New Mexico. New Mexico Water Resources Research Institute. Technical Completion Report. Project No. 01-4-23955.

Kinzli, K-D., Martinez, M., Oad, R., Prior, A., and D. Gensler. (2010). Using an ADCP to Determine Canal Seepage Loss in an Irrigation District. Agricultural Water Management. 97(6): 801-810.

Kinzli, K-D. (2010).Improving Irrigation System Performance through Scheduled Water Delivery in the Middle Rio Grande Conservancy District. Dissertation. Colorado State University. May.

New Mexico Interstate Stream Commission (NMISC); 2006 (authors Ramchand Oad, Luis Garcia, Dave Patterson and Kristoph-Dietrich Kinzli of Colorado State University, and Deborah Hathaway, Dagmar Llewellyn and Rick Young of S. S. Papadopulos & Assoc.); *Water Management Decision-Support System for Irrigation in the Middle Rio Grande Conservancy District*; March, 2006.

Oad, R. Garcia, L. Kinzli, K, Patterson, D and N. Shafike. (2009). “Decision Support Systems for Efficient Irrigation in the Middle Rio Grande Valley.” ASCE Journal of Irrigation and Drainage. 135(2): 177-185

Oad, Ramchand and R. Kullman (2006). Managing Irrigated Agriculture for Better River Ecosystems—A Case study of the Middle Rio Grande. Journal of Irrigation and Drainage Engineering, Vol. 132, No. 6: 579-586. American Society of Civil Engineers, Reston, VA.

Oad, R. Garcia, L. Kinzli, K-D. Patterson, D. (2006) Decision Support Systems for Efficient Irrigated Agriculture. (2006). Sustainable Irrigation Management, Technologies and Policies. WIT Transactions on Ecology and Environment. Wessex Institute of Technology, Southampton UK.

Oad, Ramchand and K. Kinzli (2006). SCADA Employed in Middle Rio Grande Valley to Help Deliver Water Efficiently. Colorado Water – Newsletter of the Water Center of Colorado State University. April 2006,

Rio Grande Compact Commission. 1997. Report of the Rio Grande Compact Commission, 1997.

Shah, S.K. 2001. The Middle Rio Grande Conservancy District. Water, Watersheds, and Land use in New Mexico: Impacts of Population Growth on Natural Resources, New Mexico Bureau of Mines and Mineral Resources. Socorro, New Mexico. 123-125.

United States Fish and Wildlife Service (USFWS). 2003. Federal Register 50 CFR Part 17. Endangered and Threatened Wildlife and Plants; Designation of Critical Habitat for the Rio Grande Silvery Minnow; Final Rule. Vol. 68, No. 33. U.S. Department of Interior.

United States Department of Agriculture (USDA). 1967. Irrigation Water Requirements. Technical Release No. 21.

U.S. Department of Interior (2003). Water 2025: Preventing Crisis and Conflict in the West. U.S Bureau of Reclamation.

URBANIZATION OF IRRIGATION DISTRICTS IN THE TEXAS RIO GRANDE RIVER BASIN

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ABSTRACT

The region of Texas along the Mexican border has been experiencing rapid urban growth. This has caused fragmentation of many irrigation districts who are struggling to address the challenges resulting from urbanization. This paper presents an analysis of the growth of urban area in five Texas border counties with irrigation districts. Over the ten year period, 1996 to 2006, urban area within these counties increased at a regional average of 21%. The urban area within districts increased an average of 44% based on total district service area. The paper also presents a density analysis of urbanized area and analysis of the impacts on water distribution networks. Urbanization issues related to the operation, management, and planning within districts are also discussed.

INTRODUCTION

Industrial, commercial and retirement community development are resulting in rapid urban growth within portions of the Texas Rio Grande River Basin. The fastest growing areas are Hidalgo and Cameron Counties. The four largest cities of Alamo, McAllen, Brownsville and Harlingen are among the fastest growing cities in the USA (Stubbs et al., 2003; City of McAllen, 2010).

Irrigation districts hold the vast majority of the agricultural water rights (i.e., Texas Class A or similar allocations) in the border region which accounts for about 70% of the total available surface water in the seven counties of El Paso, Hudspeth, Maverick, Kinney, Hidalgo, Willacy, and Cameron (TCEQ, 2010). As districts urbanize, Texas water laws and regulations require that the associated water rights are transferred from agricultural to municipal water use. Thus, not only does urbanization reduce the size of their service areas, but the amount of water the districts have access to and which flows through their canals and pipelines.

Most districts in the region do very little analysis of the effects of urbanization on their operation and management procedures, or incorporate urbanization trends into planning for future infrastructure improvements.

This paper discusses the potential impacts of urbanization and identifies methodologies that can help to interpret the urban growth dynamics and effects.

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MATERIALS AND METHODS

Study area

Results are presented on five of the six counties along the Texas-Mexico border, which contain irrigation districts with Texas Class A water rights, and the El Paso Water Improvement District, which has a water allocation based on the Rio Grande River Compact (Fig. 1). Presidio Water Improvement District No.1 does not contain any urbanized areas so this district and county are not included in the results presented here.

El Paso and Maverick Districts have a total service area of 279,713 acres and 606 miles of main canals. The Lower Rio Grande Valley contains 29 irrigation districts with a total service area of 759,481 acres, and a canal system 3,174 miles long.

The authorized Class A water rights of the irrigation districts in the Lower Rio Grande Region are listed in Table 1 along with the reported water allotment for the El Paso district under the Rio Grande Compact. Based on water rights, the districts vary greatly in size. In the Lower Rio Grande Basin, the smallest active district has 1,120 ac-ft of Class A Water Right, while the largest district has 177,151 ac-ft. Actual water allocations in any given year depend on the amount of water stored in Amistad and Falcon Reservoirs for region B and C.

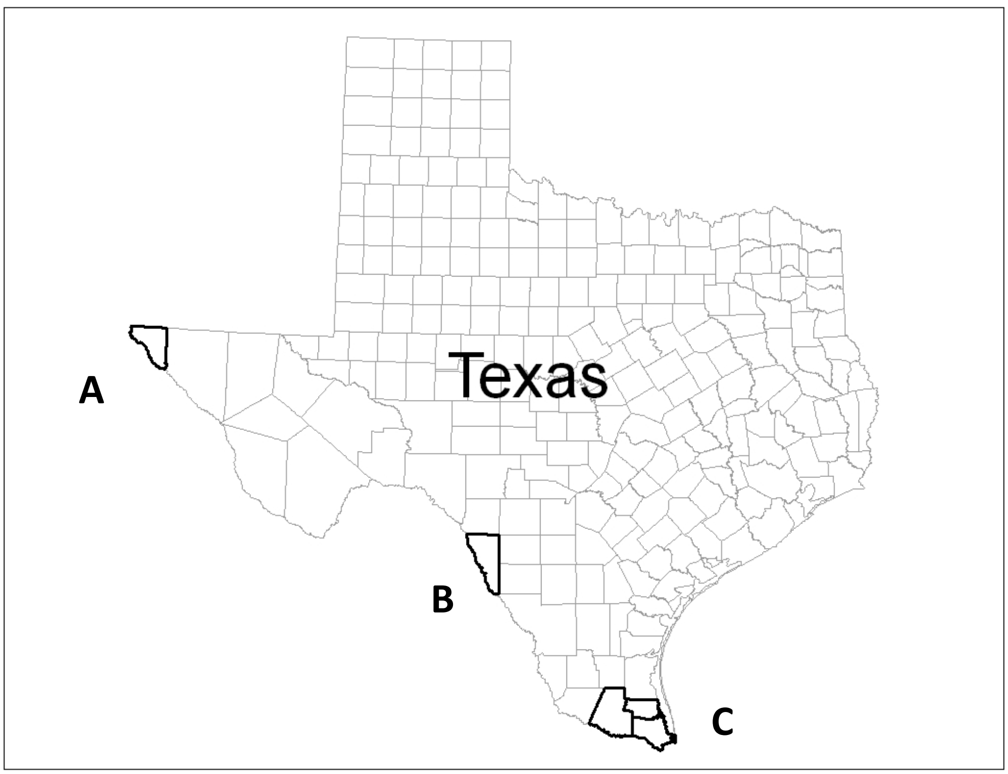

Figure 1. Area of study. A: El Paso County; B: Maverick County; C: Hidalgo, Willacy, and Cameron Counties

Table 1. Class A Water Rights of districts in the Lower Rio Grande Basin and Water Allocation for El Paso under the Rio Grande Compact

District	Class A Water Right (Acre-Feet)
Adams Garden Irrigation District No.19 (Adams Garden)	18,738
Bayview Irrigation District No.11 (Bayview)	16,978
Brownsville Irrigation District (Brownsville)	33,949
Cameron County Water Improvement District No.16 (CCWID16)	3,713
Cameron County Irrigation District No.2 (CCID2)	147,824
Cameron County Irrigation District No.6 (CCID6)	52,142
Cameron County Water Improvement District No.10 (CCWID10)	8,488
Delta Lake Irrigation District (Delta Lake)	174,776
Donna Irrigation District-Hidalgo County No.1 (Donna)	94,064
El Paso County Water Improvement District No.1 (El Paso)	388,000*
Engelman Irrigation District (Engelman)	20,044
Harlingen Irrigation District-Cameron County No.1 (Harlingen)	98,233
Hidalgo and Cameron County Irrigation District No.9 (HCCID9)	177,152
Hidalgo County Irrigation District No.1 (HCID1)	85,615
Hidalgo County Irrigation District No.13 (HCID13)	4,857
Hidalgo County Irrigation District No.16 (HCID16)	30,749
Hidalgo County Irrigation District No.19 (HCID19)	9,048
Hidalgo County Water Control and Improvement District No.18 (HCWCID18)	5,318
Hidalgo County Irrigation District No.2 (HCID2)	137,675
Hidalgo County Water Improvement District No.5 (HCWID5)	14,235
Hidalgo County Irrigation District No.6 (HCID6)	34,913
Hidalgo County Municipal Utility District No.1 (HCMUD1)	1,120
Hidalgo County Water Improvement District No.3 (HCWID3)	9,753
La Feria Irrigation District-Cameron County No.3 (La Feria)	75,626
Maverick County Water Control & Improvement District No.1 (Maverick)	134,900
Presidio County Water Improvement District No.1 (Presidio)	2,780
Santa Cruz Irrigation District No.15 (Santa Cruz)	75,080
Santa Maria Irrigation District-Cameron County No.4 (Santa Maria)	10,183
United Irrigation District of Hidalgo County (United)	57,374
Valley Acres Water District (Valley Acres)	16,124
Valley Municipal Utility District No.2 (VMUD2)	5,511

* Water allocation under the Rio Grande Compact

El Paso County Water Improvement District No.1 is allocated water according to the Rio Grande Compact. The District receives 388,000 ac-ft ("full allocation") or 43% of the available water supply in Elephant Butt and Caballo Reservoirs, whichever is less. Hudspeth County Conservation & Reclamation District No.1 has Texas Class B water rights and is not included in this analysis.

Urbanization Analysis

Urban area expansion. The maps and calculations of urban area were done using the Geographic Information System (GIS) software ArcView 9.3 and are based on aerial photography taken in 1996 and 2006. This aerial photography or Digital Orthophoto Quadrangle Imagery (DOQs) was obtained from the Texas Natural Resources Information System (http://www.tnris.state.tx.us). The 1996 DOQs have a resolution of 1 meter, while the 2006 DOQs have a 2 meter resolution.

For this paper, “urban area” is loosely defined as a continuous developed and/or developing area that is no longer in agricultural use. We included all residential communities and subdivisions (with or without homes) that are clearly identifiable from aerial photographs. We also included properties with more than one dwelling or other structure on a single piece of property. Single dwellings on large properties outside the city limits were excluded (Leigh et al., 2009).

The results may be viewed as a density analysis. A similar density analytic approached was used by Ritters (2000) in determining fragmentation of forests through an automatic pixel analysis of aerial photography. Ritters’ analysis was used to determine the progressive intrusion of urbanization classified into the categories: edge, perforated, transition and patched.

Overlap of urban area with water distribution networks. A further analysis was done to determine the overlap of urbanization with the water distribution network. We used the option of the Kernel density to count the times in a given area that the canals were overlapped by urbanization. This method is a data smoothing technique that gives more weight to points near the center of each search area and allows for creating a more continuous surface that is easier to interpret (Kloog et al., 2009). To facilitate comparison among the different study areas, we normalized the Kernel density based on the highest observed value. We obtained a scale that ranges from 0 to 1, and we called it Network Fragmentation Index (NFI).

For each district, we calculated the ratio between the times that the canals were overlapped by urbanization and the total length of canals. This computation has the advantage of giving one number for each irrigation district. We called this ratio District Fragmentation Index (DFI).

RESULTS

Urbanization Analysis

Table 2 lists the increase in total urban area between 1996 and 2006 by county. The highest increase in both area and as a percentage of total area was in Hidalgo County. Table 3 lists the percentage of urban area within 30 irrigation districts in 1996 and 2006. As a percentage of the district, the most urbanized district is HCMUD1 at 89.5%, while Valley Acres and Bayview are the least urbanized. Table 4 lists our estimate of the total urban area within each district in terms of both acres and percentage increase from 1996 and 2006. HCID2 has the largest number of urban acres, while the largest increases in urban area as a percentage of the district were in HCID16, HCWCID18 and HCID19. There were no increases in VMUD2 and Valley Acres.

Table 2. Urban area within Counties in 1996 and 2006

County	Total Area (Acres)	Urban Area 1996 (Acres)	Urban Area 2006 (Acres)	Increase (%)
Cameron	613,036	66,189	81,635	23
El Paso	656,492	208,180	234,155	12
Hidalgo	1,012,982	118,466	160,095	35
Maverick	826,915	9,816	12,019	22
Willacy	393,819	3,084	3,509	14

Table 3. Urban area within districts as a percentage of total district service area in 1996 and 2006

District	Approx. District Area (Acres)	Percentage of District Area	
		Urban Area 1996	Urban Area 2006
Adams Garden	9,600	5.5 %	14.4 %
Bayview	10,700	0.2 %	1.1 %
Brownsville	22,000	40.0 %	45.3 %
CCWID16	2,200	12.0 %	19.2 %
CCID2	79,000	10.6 %	13.8 %
CCID6	33,000	13.3 %	23.8 %
CCWID10	4,700	3.0 %	4.8 %
Delta Lake	85,600	1.3 %	2.2 %
Donna	47,000	9.3 %	15.5 %
El Paso	92,800	35.5 %	38.2 %
Engelman	11,200	1.3 %	2.9 %
Harlingen	56,500	26.0 %	30.0 %
HCCID9	87,900	19.0 %	26.0 %
HCID1	38,600	58.7 %	66.0 %
HCID13	2,200	5.4 %	21.5 %
HCID16	13,600	0.6 %	7.4 %
HCID19	4,800	0.0 %	40.0 %
HCWCID18	2,400	0.6 %	12.6 %
HCID2	72,600	45.5 %	54.0 %
HCWID5	8,100	14.1 %	17.6 %
HCID6	22,900	24.8 %	42.0 %
HCMUD1	2,000	50.3 %	89.5 %
HCWID3	9,100	72.4 %	76.0 %
La Feria	36,200	7.3 %	10.5 %
Maverick	148,700	0.1 %	8.1 %
Santa Cruz	39,500	7.3 %	9.4 %
Santa Maria	4,000	6.0 %	9.1 %
United	37,800	40.6 %	47.1 %
Valley Acres	11,200	1.4 %	1.4 %
VMUD2	4,800	23.8 %	23.8 %

Table 4. Urban acreage within districts in 1996 and 2006

District	Urban Area 1996 (Acres)	Urban Area 2006 (Acres)	Percent Increase
Adams Garden	532	1,380	160 %
Bayview	24	120	392 %
Brownsville	8,724	9,915	14 %
CCWID16	260	415	60 %
CCID2	8,384	10,925	30 %
CCID6	4,439	7,948	79 %
CCWID10	135	224	66 %
Delta Lake	1,127	1,841	63 %
Donna	4,357	7,310	68 %
El Paso	32,967	35,443	8 %
Engelman	144	331	130 %
Harlingen	14,662	16,955	16 %
HCCID9	16,721	22,716	36 %
HCID1	22,633	25,327	12 %
HCID13	117	469	302 %
HCID16	83	1,005	1109 %
HCID19	0	1,908	–
HCWCID18	15	300	1924 %
HCID2	33,006	39,107	19 %
HCWID5	1,142	1,424	25 %
HCID6	5,677	9,595	69 %
HCMUD1	1,016	1,811	78 %
HCWID3	6,618	6,936	5 %
La Feria	2,626	3,809	45 %
Maverick	9,794	11,972	22 %
Santa Cruz	2,889	3,715	29 %
Santa Maria	242	365	51 %
United	15,336	17,794	16 %
Valley Acres	162	162	0 %
VMUD2	1,142	1,142	0 %

Table 5. Percent (%) increase in the length of canals and pipelines overlapped by urbanization from 1996 to 2006

	Category		Material			Type		
Irrigation District	Secondary	Main	Concrete	Earth	PVC	Canal	Pipeline	Total
Adams Garden	53	163	62	588	33	210	51	66
Bayview	432	39		130		225	279	255
Brownsville	28	8	21		44		22	21
CCWID16		5		5		5		5
CCID2	69	37	42	50	163	52	51	52
CCID6	58	21	49	40		48	35	45
CCWID10		168		72		72		182
Delta Lake	104	107	111			94	110	104
Donna	41	74	49	14		70	18	46
Engelman	62	148	76			129	70	76
Harlingen	37	9	35	7		9	37	28
HCCID9	22	12	20	9		12	22	20
HCID1	11	12	12	6	22	8	13	11
HCID13	0	93	0		161		93	84
HCID16	780	294	752		262	387	808	648
HCID2	12	20	12	55	3	27	13	15
HCWID5		1	1				1	1
HCID6	28	38	37			32	27	29
HCWID3		22		81		31		21
La Feria	32	31	37	4		24	35	32
Santa Cruz	16	29	19			17	19	19
Santa Maria	103		103				103	58
United	9	18	10		41	14	9	10
Total	29	24	27	30	36	34	24	27

Effects on the water distribution network

The distribution networks are also increasingly engrossed by urban areas (Table 5). During this ten year period, about eight hundred more acres (28% increase) of storage facilities (reservoirs and *resacas*[3]) became a part of urban areas and an additional 27% of canals (360 miles) flow through urban areas. Figure 2 shows the urban areas in Hidalgo County in 1996 and 2006, along with the service area boundaries of the irrigation districts. Figure 3 shows the network overlapped by urbanization and the NFI (Network Fragmentation Index), where an index of 1 represents the greatest fragmentation of canals.

Figures 4 and 5 show the DFI (District Fragmentation Index) for 1996 and 2006, respectively, as a single number for each district. Also shown are the NFI. We found that the two indexes are consistent.

[3] An area of river bed that is flooded in periods of high water; an artificial reservoir (Dictionary of American Regional English, 2011)

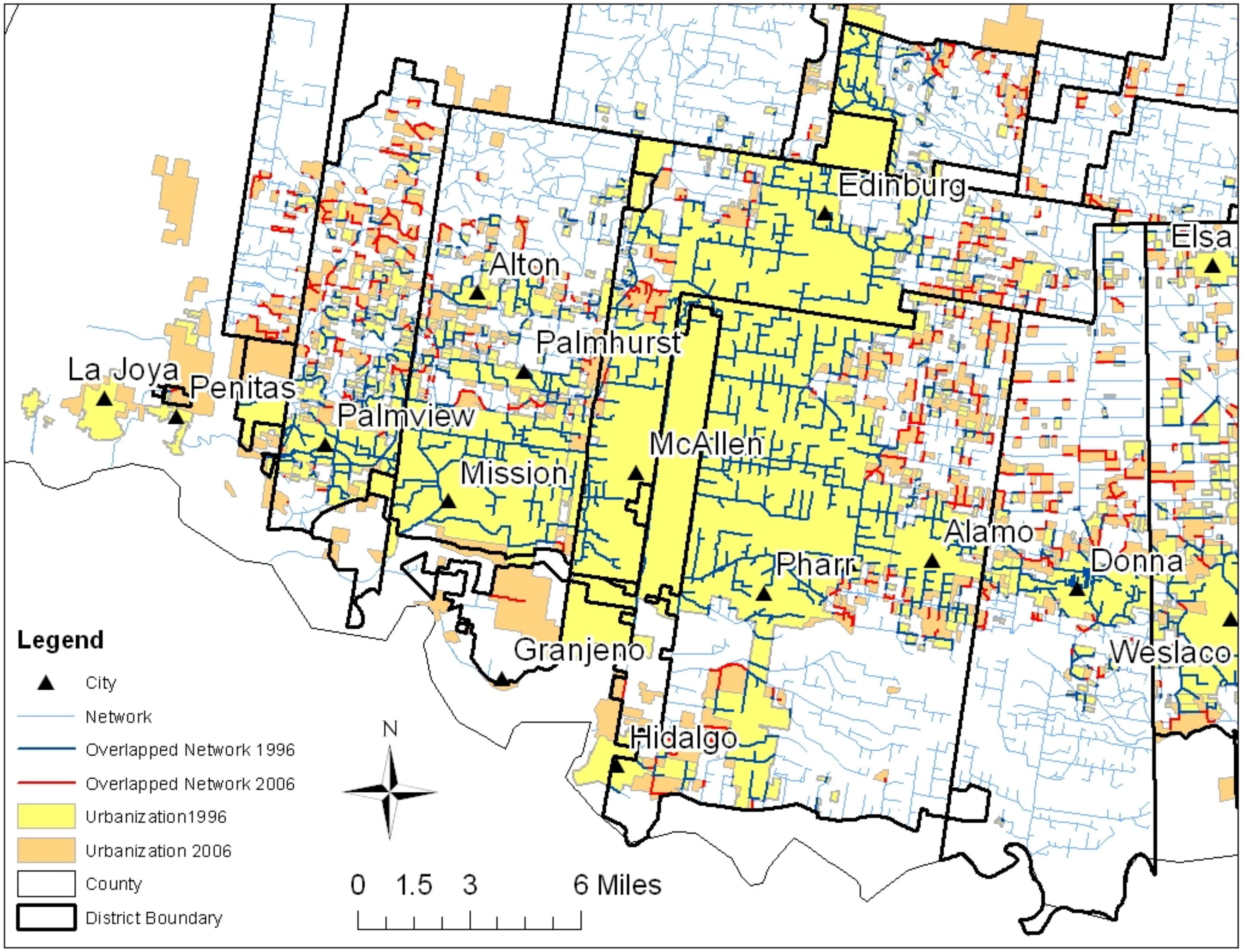

Figure 2. Urbanization in the McAllen area of the Hidalgo County in 1996 and 2006

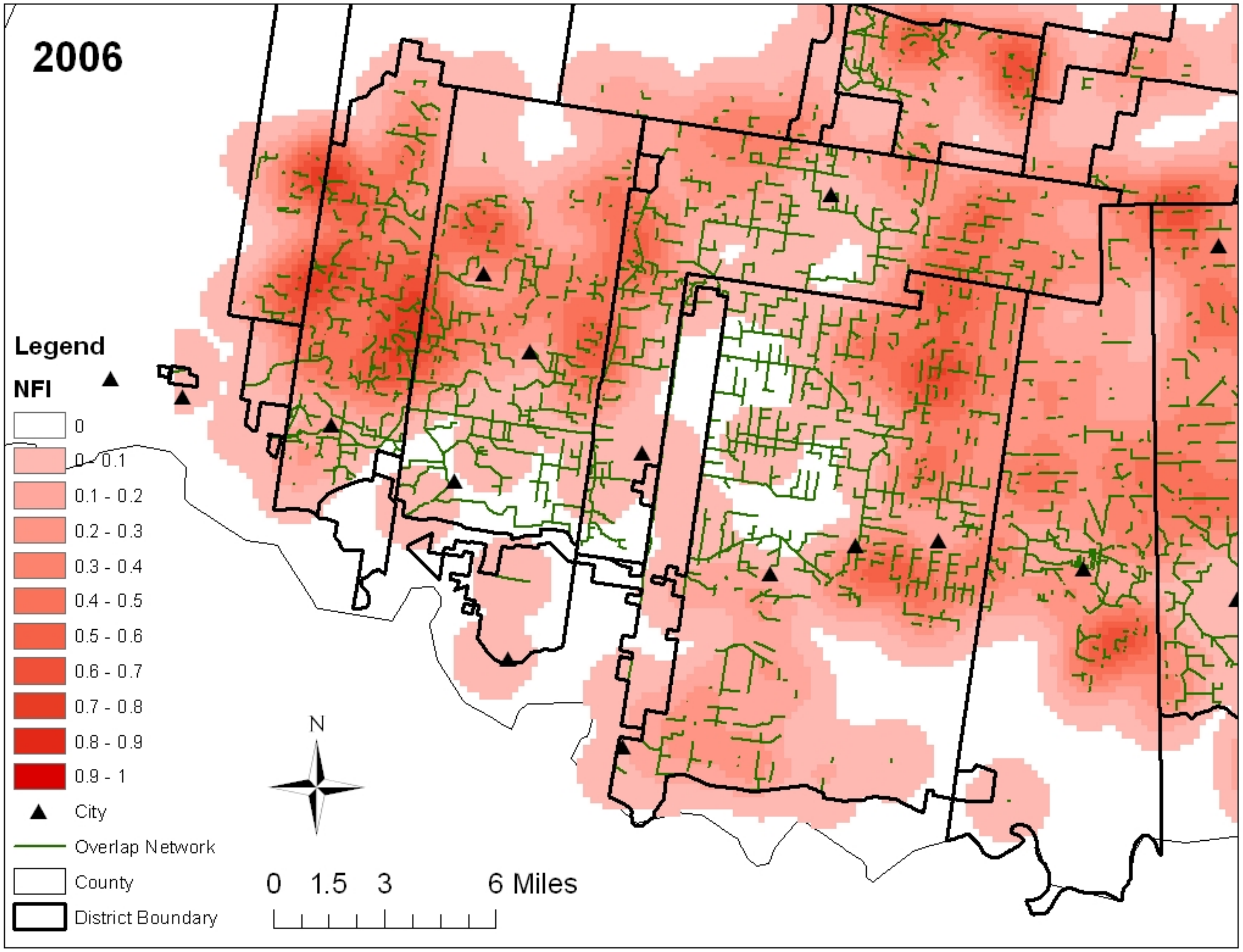

Figure 3. Overlapped network by urbanization (green lines) and Network Fragmentation Index (red areas) in the McAllen area of the Hidalgo County, in the year 2006

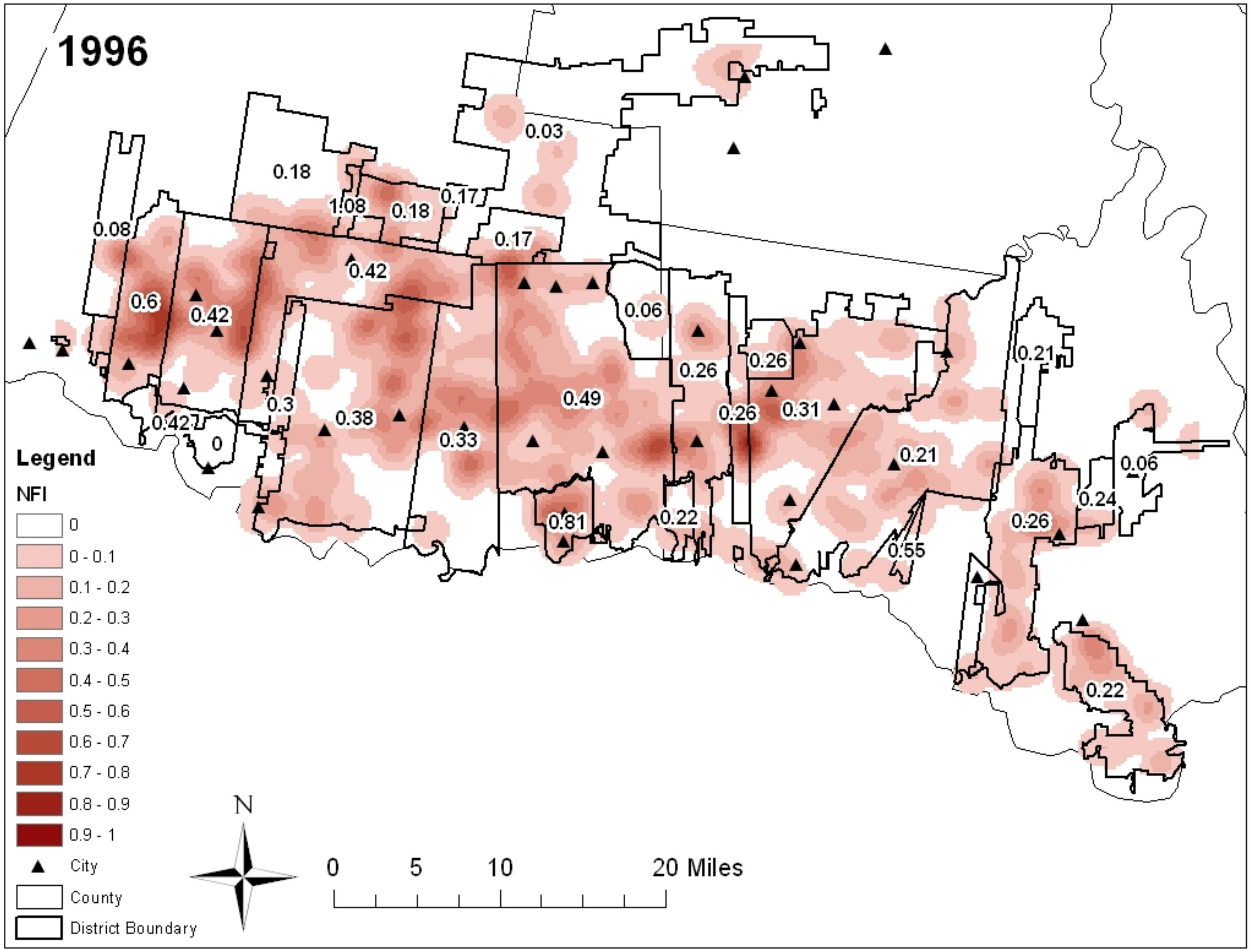

Figure 4. District Fragmentation Index (DFI) for each district along with the NFI (Network Fragmentation Index), shown as a density map, in the year 1996

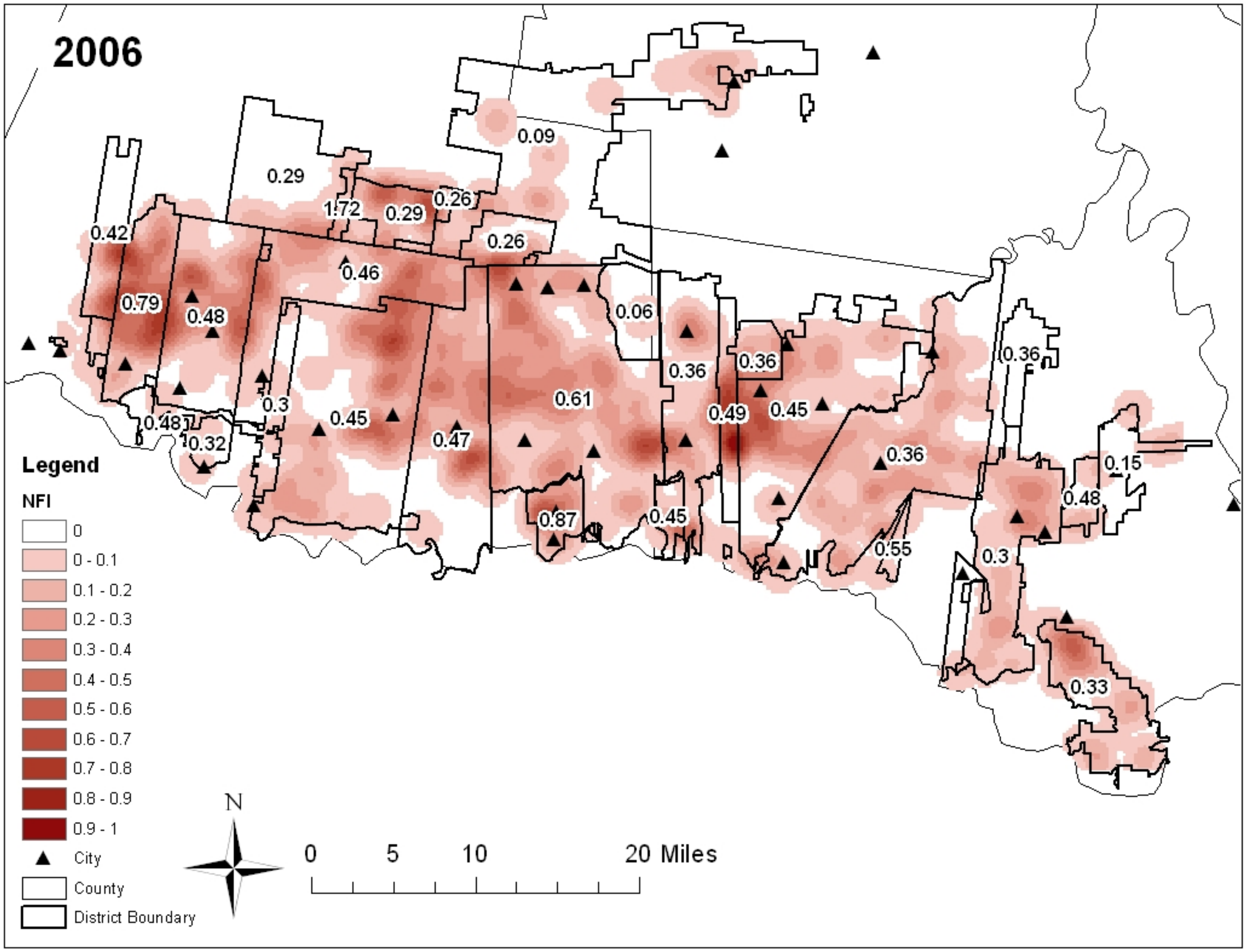

Figure 5. District Fragmentation Index (DFI) for each district along with the NFI (Network Fragmentation Index), shown as a density map, in the year 2006

DISCUSSION

Potential Impacts

Impacts of urbanization can affect Districts in several different ways.

Access to network and structures. Districts in this region primarily operate their systems manually, with a canal rider personally moving from site to site. An increasing presence of subdivisions or isolated houses can create access to and maintenance of facilities difficult or more time consuming. As a consequence, operations might take more time due to discontinuous access to structures or requiring the district to construct new facilities to operate the network correctly.

Transfer of water rights from agricultural to other uses. Transfer of water rights from agricultural to other uses reduces the total amount of water flowing through the water distribution networks, which typically decreases conveyance efficiency and increases losses.

Increasing liability for canal breaks and flooding. The increasing presence of subdivisions and industrial areas in the vicinity of the delivery network increases liability for canal breaks and flooding. Such areas may suffer significant damages from minor flooding events. This is not a new phenomenon in most districts, but such situations are rapidly increasing, requiring investments in studies and structural changes. Subbasins must be identified, and flood management plans put in place to clearly define risks, potential impacts, emergency action, and short and long term measures and investments.

Fragmentation and shrinking of irrigation area. Urbanization is causing the fragmentation and loss of agricultural land. Districts eventually will have to abandon structures that are no longer needed and invest in new ones to ensure good operations. Urbanization causes canals to become oversized, thereby affecting: how the system operates, operational efficiency, and the ability to deliver increasing smaller volumes of water. In addition, revenues from water sales decrease, requiring districts to increase rates.

CONCLUSIONS

Methodologies were presented to interpret the fast urban growth dynamics in the region of Texas along the Mexican border. They show promise in helping irrigation districts identify the impact of urban growth. The density analysis produces maps that clearly identify and quantify urbanization and that are easy to use and interpret. Two new indexes, the Network Fragmentation Index (NFI) and District Fragmentation Index (DFI) are used to describe the impact of urban growth on water distribution networks. These values are consistent with the density analysis. The NFI has the advantage of identifying detailed locations of impact, while the DFI is able to synthesize such information in one value per district.

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REFERENCES

City of McAllen website, 2011.Chamber of Commerce web page. [online] URL: http://www.mcallen.org/Business-Community/McAllen-Overview

Dictionary of American Regional English, 2011. [online] URL: http://dare.wisc.edu/?q=node/144

Kloog, I., Haim, A., Portnov, B.A., 2009. Using kernel density function as an urban analysis tool: Investigating the association between nightlight exposure and the incidence of breast cancer in Haifa, Israel. Computers, Environment and Urban Systems 33, 55–6.

Leigh, E., Barroso, M., Fipps, G., 2009. Expansion of Urban Area in Irrigation Districts of the Rio Grande River Basin, 1996 - 2006: A Map Series. Texas Water Resources Institute Technical Report EM-105.

Ritters, K., J. Wickham, R. O'Neill, B. Jones, and E. Smith, 2000. Global-scale patterns of forest fragmentation. Conservation Ecology 4(2): 3. [online] URL: http://www.consecol.org/vol4/iss2/art3/

Stubbs, M.J., Rister, M.E., Lacewell, R.D., Ellis, J.R., Sturdivant, A.W., Robinson, J.R.C., Fernandez, L., 2003. Evolution of Irrigation Districts and Operating Institutions: Texas, Lower Rio Grande Valley. Texas Water Resources Institute, Technical Report No. TR-228.

TCEQ, 2011. "Water Rights Database and Related Files" web page, Texas Commission on Environmental Quality website. [online] URL: http://www.tceq.state.tx.us/permitting/water_supply/water_rights/wr_databases.html

URBANIZATION ISSUES IN THE MIDDLE RIO GRANDE CONSERVANCY DISTRICT

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Tyler Holmes
Zohrab Samani
A. Salim Bawazir

ABSTRACT

Numerous rapidly growing urban areas in the western United States are located in irrigated river valleys. Agricultural irrigation in these communities is being affected by urbanization, and the characteristics and objectives of the irrigator population are also changing. New Mexico's Middle Rio Grande Conservancy District (MRGCD) encompasses the rapidly growing Albuquerque metropolitan area. The South Valley is one of the state's oldest traditional agricultural communities, and is located on the southern fringe of Albuquerque within the MRGCD. South Valley irrigated agriculture is in a state of transition, and many lands that were previously used to produce a diverse mix of fruits, vegetables, grains and forages have been converted into commercial and residential development. A few relatively large farms continue to operate in the area; however, hundreds of small or micro-scale irrigated properties are located in the South Valley. These small rural-residential properties predominately apply MRGCD irrigation water to hay and pasture. Little is known about agronomic, irrigation, or marketing practices on South Valley micro-farms, as well as the economic outcomes or impacts of irrigation water use. A team of New Mexico State University researchers is currently surveying MRGCD irrigators, measuring alfalfa consumptive use, and attempting to quantify the economic impact of South Valley irrigated agriculture.

INTRODUCTION

Population and economic growth throughout the United States have accelerated the conversion of agricultural land to non-agricultural uses over the last several decades, with approximately 20% of all U.S. cropland now subject to some degree of development pressure (Heimlich and Barnard 2003). Many rapidly growing urban areas in the western United States are located in irrigated river valleys, with the future of agricultural land closely linked to the future of water resources (and vice versa). Advocates of traditional irrigated agricultural communities thus struggle to justify economic, historic, cultural, and lifestyle-based claims on combined land and water resources in the face of growing competition for both assets from non-agricultural users. The changing character of the

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agricultural irrigator population also challenges conventional notions of successful agricultural irrigation and beneficial use of water resources.

This situation is particularly acute in the Albuquerque, New Mexico metropolitan area, located in the arid southwest where both arable land and water are scarce. One of New Mexico's oldest, traditional agricultural communities is known as the South Valley. This unincorporated community is located on the southern fringe of the Albuquerque metro area, has been home to irrigated agriculture for many centuries, and is in the process of dramatic transformation. South Valley agricultural lands, agricultural irrigation water, agrarian values and traditions are being supplanted by suburban and urban land and water uses, with values and traditions also shifting from rural to urban. Lands that were once home to small, medium and large farms producing a diverse mix of fruits, vegetables, grains, forages, irrigated pasture, and small-scale mixed livestock species have been converted into commercial and residential development. A few large farms continue to operate in the area, although the majority of farms in the South Valley are small to very small, and while their numbers are large, these farms contribute marginally to the value of total agricultural output and economic activity in the state. The majority of large-lot rural residences that continue to use irrigation water do so primarily for lifestyle and landscape purposes, often for the production of pasture or relatively low-valued hay.

THE MIDDLE RIO GRANDE CONSERVANCY DISTRICT

The South Valley is located within the Middle Rio Grande Conservancy District (MRGCD) in Bernalillo County, New Mexico. The MRGCD was officially founded in the 1920s, but may be the oldest operating irrigation system in North America (Gensler, Oad and Kinzli 2009). The MRGCD delivers irrigation water to approximately 22,300 ha, and is important for flood protection and soil drainage in the region along the Rio Grande from Cochiti Dam in the north to the Bosque del Apache Wildlife Refuge in the south. The MRGCD covers numerous jurisdictions including federal, city, county, and tribal (Figure 1).

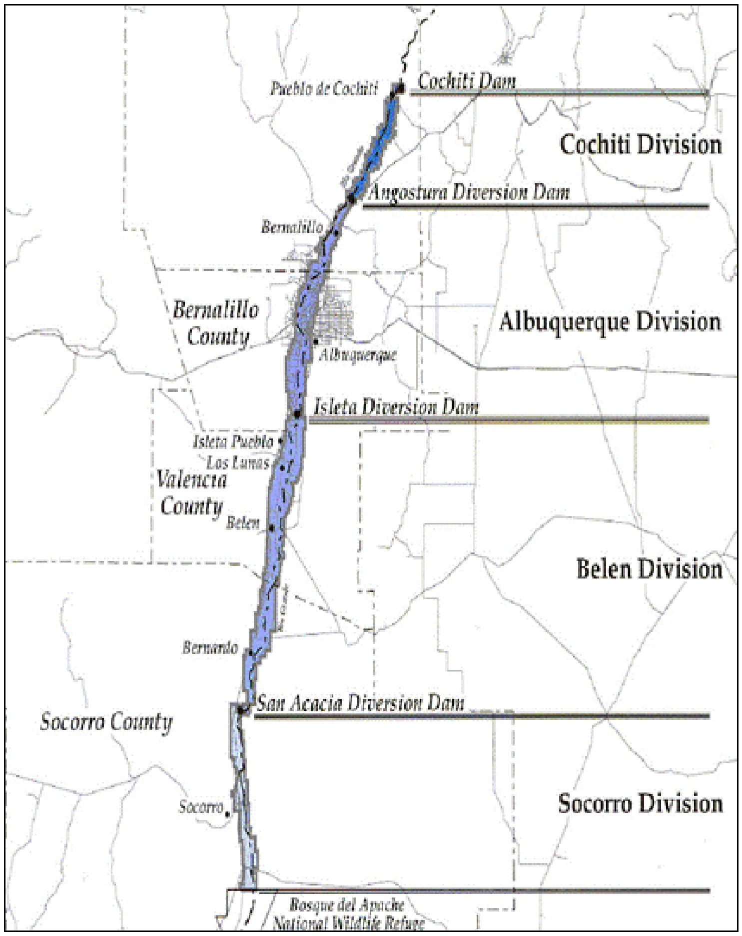

Figure 1. Central New Mexico's Middle Rio Grande Conservancy District[2].

RURAL-TO-URBAN LAND USE TRANSITION

Land use and land cover changes in Albuquerque's South Valley (bordered on the north by Rio Grande Boulevard, and on the south by Interstate-25) from 1985 to 2009 are illustrated in Table 1 and Figure 2.[3] During the past 25 years, the region experienced a 33% increase in urban land use along with a 20% decrease in agricultural land use and an

[2] Source of map: Middle Rio Grande Conservancy District.

[3] The land classification map in Figure 2 was created using remote sensing analysis of Landsat 5 Thematic Mapper satellite images. The Landsat 5 images were acquired for 13 September 13 1985 and 1 October 1 2009 to illustrate change in land cover types over a 24-year period. Both images were taken during vegetation leaf-on periods and have homogeneous reflectance values. A maximum likelihood land classification technique within EVNI 4.7 software was used to classify five land cover types: Agriculture, urban, water, desert and barren. Desert and barren land covers were combined for ease of processing the two images. The 1985 image has an overall classification accuracy of 87% and the 2009 image has an accuracy of 85%. The accuracy was assessed by creating regions of interest (ROI) that depict the actual land cover type of the image (e.g., ground truthing). Classification data within the ROIs are collected for actual sites, with the area in those sites compared to the output classification image for the entire image.

8% decrease in desert or barren area.[4] The increase in urbanization in the South Valley is a result of both commercial and residential development, as well as increases in surface area devoted to new roads and the driveways which access or are part of subdivisions and fragmented rural-residential home sites.

Table 1. Percentage and area changes of land cover types, 1985-2009, South Valley

Land Cover Type	% Change 1985-2009	Area Change in Km^2 1985-2009
Urban	33.715	123.21
Agriculture	-20.954	-22.32
Water	-48.507	-4.58
Desert/Barren	-8.970	-96.32

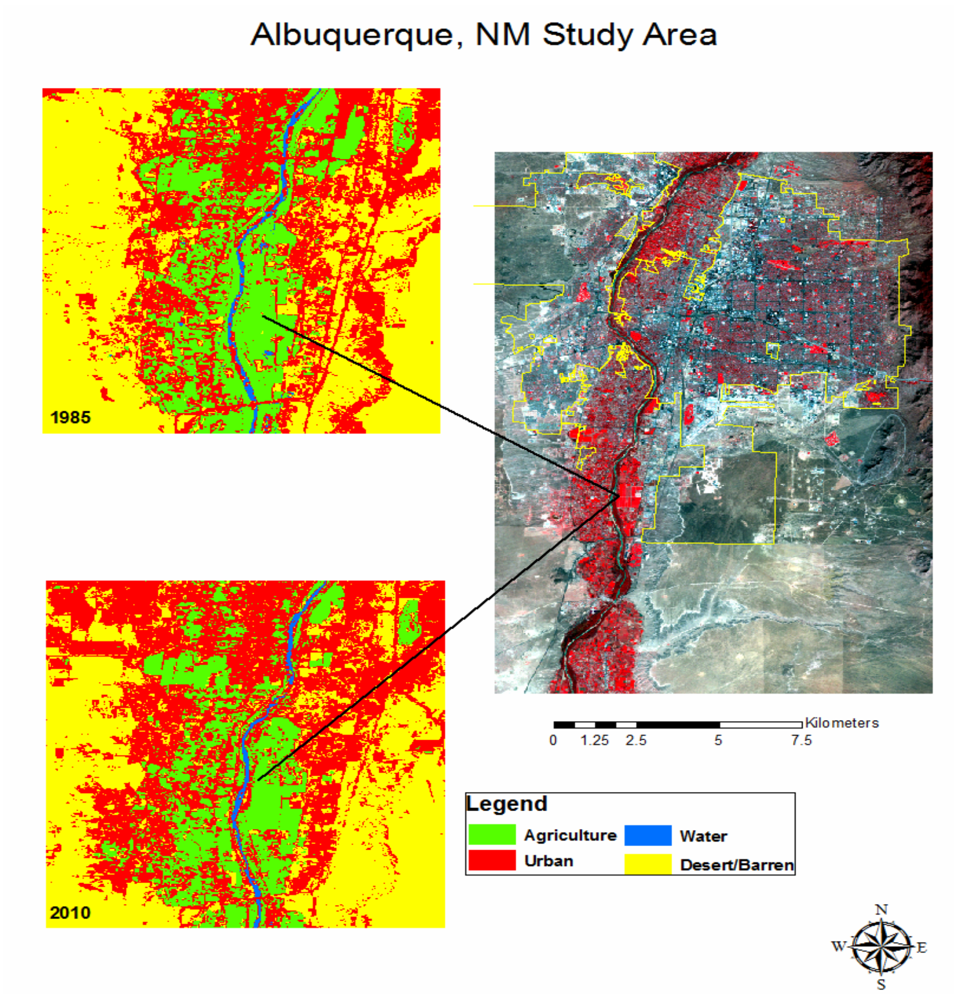

Figure 2. Land use and land cover change in the southern Albuquerque, NM metropolitan area (South Valley community).

[4] The region's small amount of land surface covered by water decreased during the period 1985-2009; however, the region's small water surface area fluctuates annually due to variable river flows in the Middle Rio Grande Basin and decreased water area is not related to urbanization trends.

AGRICULTURE IN THE SOUTH VALLEY

Land cover change data for the South Valley illustrate the shift in land use in the southern Albuquerque metro area, with the urbanized landscape increasing at the expense of agricultural and desert land area over the last few decades. Historically, many South Valley producers grew high-value crops such as apples, grapes, chile peppers, and other orchard and vegetable crops for human consumption. Today, most South Valley farms produce lower-valued and less management intensive pasture grass, grass hay, and alfalfa. The majority of the community's households are dependent on off-farm income, both earned and unearned, and there is relatively little local food production.

A previous study which examined attitudes held by residents of the South Valley found they believe agriculture and the ecosystem are inextricably linked, that small-scale agricultural producers are an integral part of the ecosystem and serve as producers of ecosystem goods and services that range from the provisioning of food, fiber, and fresh water to the regulation of processes that affect air quality, climate, erosion control and human diseases (Wang 2007). Moreover, many South Valley citizens strongly recognize the landscape, open space, cultural, and social contributions of their small-scale farms and want to keep local agriculture sustainable in their community (Wang 2007). These people believe that small-scale irrigated agriculture is an essential component of their culture and heritage, is the foundation of their identity as land-based people, and is something to which they are deeply connected (Wang 2007).

Relative to other irrigated areas in New Mexico and the western United States, research and data for South Valley crop water use, farm management practices, irrigation efficiency, and the hydrologic impacts of agriculture are limited. Little information is available regarding the economic and environmental impacts of small-scale agricultural production and agricultural water use in this region. No data have been developed for the South Valley on relationships between crops, water application, water management practices, and surface and ground water interactions. Furthermore, no research has examined the South Valley's agricultural structure, agricultural incomes, agricultural households and their attitudes, motivations and objectives, and these households' use of and relationship to natural resources (e.g., land and water).

This paper briefly describes a current research project which is developing baseline information and data about irrigated agriculture in New Mexico's South Valley. The project is funded by a three-year grant from the United States Department of Agriculture.[5] Project personnel include agricultural economists, water resource engineers, and strategic planning specialist from New Mexico State University. The project's study area includes that portion of the MRGCD located between Rio Grande Boulevard in the north and Interstate-25 in the south, is located in the southern part of the Albuquerque Division of the MRGCD (as illustrated in Figure 1), and does not include any tribal lands. The project began in 2009 and since that time, the research team has

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[5] USDA-CSREES Small and Medium Size Farm Prosperity Agreement No. 2009-55618-05096, "Improving Economic Returns & Long-Run Sustainability in a Rapidly Growing Peri-Urban, Multicultural, Traditional Farming Community".

been documenting hydrologic, agronomic, environmental, and economic variables in South Valley irrigated agriculture. One of the objectives of the project is to generate better information about the small-scale irrigator population and their agricultural activities. A telephone survey of these irrigators began in 2010 and is providing new insight into the nature of local irrigated agriculture as well as the impacts of urbanization on local agriculture. Specifically, the research team is learning about the characteristics of the small-farm irrigator population's agricultural activities and their perceived obstacles to and opportunities for increased agricultural production in the South Valley. As noted above, there is currently little intensive agricultural production in the region and most irrigation water is used in forage production. Some community residents and local activists strongly believe that local food production has the potential to contribute significantly to the region's economic development and quality of life (including landscape, environmental, and human health improvements) (Wang 2007). However, very few data are available which can be used to evaluate the potential for local food production in the South Valley.

In order to assess a region's agricultural output or potential for output, the U.S. Census of Agriculture is usually the first place to look for data on producers and production as it provides a county-level snapshot every five years. Since the 1970s, a "farm" has been defined by the U.S. Census of Agriculture as any place from which $1,000 or more of agricultural products were produced and sold, or normally would have been sold, in a given year.[6] Thus, small irrigated residential/lifestyle farms selling very small amounts of agricultural products or producing only for home consumption or barter are not enumerated in the Census of Agriculture and very little is known about the nature and character of these farms. As of early 2011, the current research project has telephone interviewed ~50 small-scale irrigated agricultural producers in the South Valley in an effort to develop a better understanding of this population. Surveying continues in early 2011, and statistical analysis of the results will be conducted when a larger number of responses have been obtained. Preliminary results of the survey are discussed below.

SURVEY OF SOUTH VALLEY IRRIGATORS

A list of approximately 1,200 South Valley irrigators was obtained from the MRGCD. A random sample of irrigators was drawn from this list, and telephone numbers for these people were located using telephone listing services for the region. Telephone surveying has been conducted from the NMSU campus in Las Cruces, and was preceded by a mailed letter to the irrigator. The letter explained the objectives of the survey, and indicated that the recipient would soon be receiving a phone call from NMSU researchers. The letter also provided information about NMSU administration contacts and contact information for the research team. The letter noted that participation in the survey was entirely voluntary, and that the survey was part of a larger research project

[6] United States Department of Agriculture National Agricultural Statistics Service, 2007 Census of Agriculture United States Summary and State Data, Volume 1, Geographic Area Series, Part 51 (February 2009), available at http://www.agcensus.usda.gov/Publications/2007/Full_Report/usv1.pdf.

seeking to develop baseline data and information about South Valley agriculture and agricultural irrigation.

Of the approximately 40 small-scale farmers interviewed thus far, the majority indicated that they use their water to irrigate grass pasture or to produce alfalfa or grass hay. A small number of respondents reported irrigating a few fruit trees (peach, pear, and apples) and one respondent reported an irrigated garden. The irrigated acreages reported ranged from 0.5 to 25, although the majority of respondents reported less than four acres. Hay yields were reported in numbers of small bales per acre, although bale weights were not reported. The respondents did not report yield estimates for their pasture, grass or alfalfa hay that harvested through grazing.

The most frequent response for irrigation frequency among the respondents is once every week, and flood irrigation is the only technology reported. No respondents indicated that they use any technology for deciding when and how much to irrigate their fields, while seven respondents reported laser-leveled their fields. Two respondents reported active insect pest management using insecticides, three reported active weed management through cultivation, herbicides, or burning. Fertilization also was reported by two respondents, and two individuals reported having their soil tested or receiving technical soil fertility advice. Advice on seeding rates for alfalfa or grass or herbicide usage was reportedly obtained from local merchants.

The presence of livestock (horses, goats, chickens, cows, sheep, and pigs) was reported by a majority of respondents. When asked why they keep livestock, respondents indicated it was for personal use and consumption, a hobby, to sell eggs, for weed and grass control, rodeo, and to earn a little cash. Then asked why they are engaged in agricultural production, only one respondent said that it was a primary source of household income. Several people reported that agriculture is a secondary source of household income, while a few noted that it is a hobby, a retirement activity, a way to produce feed for their horses, or a fun pastime. When asked where they sell their agricultural products, the most frequent response was to neighbors, usually in return for cash payment. Only three respondents reported hiring any employees to help with their agricultural production or marketing activities.

The survey included a question which asked respondents to assess the importance of different objectives for their farming operations. The list of objectives they were given to choose from was: minimize production cost, maximize income from sales of agricultural products, ensure farm survival, hold on to land until it can be developed, increase farm size, increase crop quality and reputation, increase leisure time, decrease financial risk, and preserve agricultural lifestyle. For the ~40 respondents who have been surveyed at the time of this writing, “preserve agricultural lifestyle” received the highest rankings.

As part of the survey, respondents were asked their opinion of local agriculture and its future. The comments received from the respondents covered a range of reasons or topics which have been condensed into three broad categories. Several respondents noted that many irrigators in the region lack knowledge about their soils and the relationships between weather and crop production, and also have little understanding of good irrigation and agronomic practices. Some respondents stated that there is a widespread lack of capital to invest in new agricultural (including irrigation) technologies on small-scale farms in the region, and a few stated that local culture, traditions, and attitudes were obstacles to increasing agricultural production. The term "apathy" was used by some respondents, who indicated that many South Valley residents of small-scale farms are too old to significantly change their agricultural practices, and that agriculture is more of a lifestyle or a hobby, or a garden – rather than an income generating enterprise. Some respondents stated that it was difficult to make a profit from their small farms, that farmers in the region were unorganized, lacked information, were not willing to take risks, and that crime in the region was an impediment to agricultural growth and development.

SOUTH VALLEY IRRIGATION AND ECONOMIC IMPACT RESEARCH

The preliminary survey results for small-scale irrigators in the South Valley tend to confirm our hypotheses that much agricultural production in the region can be characterized as low-input, low-management intensity, and low-output. Irrigator surveying continues, and it remains to be seen how, or if, future responses are different from the responses reported above. The engineering component of this project is measuring crop water use by alfalfa in the South Valley in order to assess the hydrologic balance within the region. A flux tower has been installed on one ~10 ha parcel of alfalfa, and two weather stations have been installed at two other locations. Point measurements of crop water use will be combined with remotely sensed data to develop region-wide estimates of agricultural evapotranspiration (ET). Existing yield-ET models for alfalfa will be used to generate estimates of the region's hay production, although the small size of many of the fields will be a challenge to the use of the remotely sensed data. Installing weather stations in the study area has also been challenged by the need to place the stations on larger fields.

Estimates of crop yields derived from the yield-ET models will provide an additional dimension to the project's attempts to estimate the economic value and economic impact of South Valley agriculture (and by extension, impute a value to the use of water in small-scale agricultural irrigation in the region). An existing input-output model (IMPLAN, Impact Analysis for Planning™) is being used to analyze the economic impact of local agriculture in MRGCD counties using data reported by county in the U.S. Census of Agriculture. Our survey of small-scale South Valley irrigators and our estimates of crop evapotranspiration and yields are providing previously unavailable insight into the household objectives, crop production and irrigation practices, consumptive use outcomes and economic impacts of small-scale agricultural production.

SUMMARY

Irrigated agriculture in the South Valley has been impacted dramatically by urbanization in the last few decades. The number of Bernalillo County farms enumerated by the Census of Agriculture[7] has increased, the numbers of very small irrigated properties has increased dramatically, the amount of irrigated land has decreased, there is little local food production, and smallholder irrigation is strongly motivated by lifestyle rather than income objectives. The majority of irrigated small farms in Bernalillo County and in the South Valley are so small that they are not enumerated by the Census of Agriculture, thus their numbers are not reflected in the urbanization trends reported within Census data.

As a result of urbanization, widespread on-farm irrigation investments, improvements, and increased water use efficiencies are unlikely to occur in the South Valley. Preliminary results from our survey of small-scale irrigators indicate that the majority of them are not motivated to increase the intensity of their agronomic practices, marketing, or irrigation management. Management intensive production of alfalfa hay continues to take place on the few large irrigated parcels that remain in the area. Management intensive or commercially-oriented production of higher-valued crops for direct human consumption (e.g., fruits, vegetables) in the region is unlikely to increase significantly in the future. Improvements in irrigation practices and increased management intensity in grass and alfalfa hay production are possible and would contribute to increased water use efficiency in the region; however, many small-scale irrigators in the South Valley are unlikely to be motivated to change their practices.

ACKNOWLEDGEMENTS

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[7] United States Census of Agriculture data for New Mexico counties are available at: http://aces.nmsu.edu/academics/aeab/trends-in-new-mexico-agr.html#anchor_73028.

REFERENCES

Gensler, D., R. Oad, and K.D. Kinzli. (March-April 2009). Irrigation System Modernization: Case Study of the Middle Rio Grande Valley. *Journal of Irrigation and Drainage Engineering*, Vol. 1335, No. 2, pages 169-176.

Heimlich, R.E. and W.D. Anderson. (2001). *Development at the Urban Fringe and Beyond: Impacts on Agriculture and Rural Land*. USDA-Economic Research Service. Agricultural Economic Report 803. June, 2001.

Wang, Y. (2007). *Agriculture in Peri-Urban Regions: An Action Research Model for a New Economic Development Strategy*. Unpublished master's thesis. New Mexico State University, Las Cruces, NM.

FROM CANYONS TO CANALS: APPLYING REGULATED RIVER RESEARCH TO CANAL BANK ANALYSIS

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Brian Wahlin, Ph.D., P.E., D.WRE[2]

ABSTRACT

Numerous studies have analyzed river bank dynamic porewater responses to regulated flows. This research has been found to be critical to understanding not only river inflows and outflows from groundwater sources, but also bank failures as a result of flow scheduling. Although the success of these models comes largely from further developing advancements in other related fields, likewise transfer of the research to other related fields has been slow. In response, this paper extends a recently developed analytical porewater pressure response model, utilized to advise flow scheduling in the Grand Canyon, to analyze irrigation canal leakage and resulting large scale groundwater reactions. The new model directly accounts for canal bank geometry, driving upstream and / or downstream water tables, and time varied irrigation flow schedules given by any piecewise continuous function. This model can be used to analyze both near and far hydraulic effects, executes quickly, and is easy to implement on any spreadsheet program. The model showed good agreement between predicted and measured canal leakage and resulting downstream water table changes for the Interstate Canal in Nebraska. Recommendations are made for further uses of the model.

INTRODUCTION

Regulating rivers through controlled dam flows can cause tremendous geomorphological effects, often expressed through numerous streambank failures. These failures can cause unchecked lateral bank migration, thalweg reorienting and even avulsions, resulting in an unintended, unnatural, and uncontrolled restructuring of the entire riparian area. The adverse geomorphologic consequences of river regulation have been well documented at the Glen Canyon Dam, located on the Colorado River within the Grand Canyon. In particular, the riverbank stability has been found to be particularly sensitive to loading conditions such as the river stage fluctuations and the resulting porewater pressure changes.

In response, an analytical model of saturated flow in a deep streambank was derived by Travis (2010). This solution is capable of analyzing any one of numerous periodic river stage conditions, such as those expected downstream from a hydroelectric dam or due to natural hydrologic events.

Like riverbanks, unlined canals are both significantly affected by and significantly contribute to groundwater conditions. Leakage results in lost revenue, unregulated

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groundwater contributions, and is driven by both canal flow schedules and existing water table elements. Seepage into the canal can adversely affect water quality and complicate canal design (Swamee, Mishra, and Chahar, 2004). Both leakage and seepage can cause loss of bank stability (Lorenzo et al., 2003; Thomas, Iverson, and Burkart, 2009). Recent modeling efforts include Lal et al. (2010) who successfully applied an analytical solution to sinusoidal canal flows to improve aquifer property measurements; and Li, Boufadel, and Weaver (2008) who utilized a numerical solution to account for unsaturated flow in canal banks.

In this paper, the Grand Canyon porewater pressure model is extended to account for groundwater effects of canal leakage. This new model collapses to the well known one dimensional solution utilized by Lal et al. (2010) model for sinusoidal canal flows, horizontal water table conditions, and vertical banks. Verification is obtained using the detailed field measurements of the Interstate Canal in Nebraska reported by Harvey and Sibray (2001).

POREWATER RESPONSE MODEL

Porewater response modeling is an established application of the basic laws of saturated groundwater flow. Indeed, from the well accepted observation of Henry Darcy in 1856 that groundwater flow is proportional to hydraulic head (Darcy, 1856), the complete governing equations of dynamic seepage flow can be immediately derived (see Mays and Todd, 2005). And while groundwater flow remains a highly active area of research, current efforts on the subject tend to focus on application specific finite difference / element algorithms, rather than pursuing analytical solutions to the governing equations (e.g. Boutt, 2010; Haitjema et al., 2010; others).

Unfortunately, numerical flow solutions become difficult to achieve for periodic, tidal type, loading conditions, since porewater pressure distributions are dependent on their history, and it is not clear what constitutes reasonable initial conditions of periodic fluctuations. One approach to resolving the initial condition problem is to run the finite element model through sufficient cycles that risk response also becomes periodic. The alternative approach is to iteratively adjust the initial conditions until they are in agreement with those at the end of the period. Either method would be expected to significantly increase computing time.

A resolution to this challenge is to derive an analytical model general of the porewater response to periodic adjacent water stages. Figure 1 shows the simplified bank geometry, defining x (m) the horizontal coordinate, y (m) the vertical coordinate, w the bank width, b (m) the bank height, and z(t) the adjacent stage (in meters) as a function of time t (sec). The definition and units of the soil variables are shown in Table 1, which also includes the specific values for the example application shown subsequently.

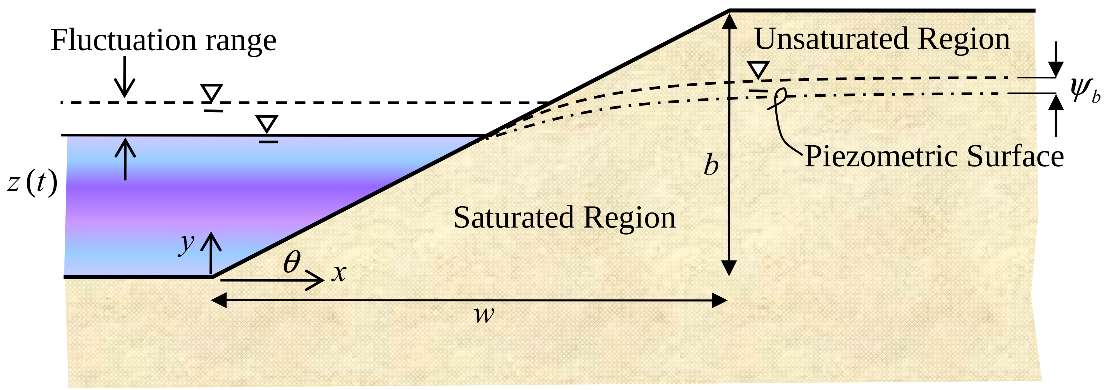

Figure 1. Canal Bank Model

Saturated Flow

The saturated region in the riverbank is described by the two-dimensional Richards equation for saturated flow (Fredlund and Rahardjo, 1993):

$$\frac{d^2h_s}{dx^2}+\frac{d^2h_s}{dy^2}=\frac{s_s}{k_s}\frac{dh_s}{dt} \tag{1}$$

where $h_s(x,y,t)$ (m) is the hydraulic head in the saturated region, s_s (m^{-1}) is the specific storativity, and k_s (m/sec) is the saturated hydraulic conductivity. The bank is assumed to be homogeneous and all of the soil properties are assumed to be constant. The origin is located at the interface of the sandbar with the river base.

Equation (1) is simplified by introducing the composite variable u, where

$$u = x - y/\tan\theta \tag{2}$$

resulting in

$$\frac{d^2h_s}{du^2}=\frac{s_s\sin^2\theta}{k_s}\frac{dh_s}{dt} \tag{3}$$

Limiting the solution to periodic functions, the time constraint is the periodic condition:

$$h_s(u,t)=h_s(u,t+p) \tag{4}$$

The boundary conditions for h_s are

$$\left.\frac{dh_s}{du}\right|_{u\to\infty} = h_\infty^{/} \tag{5}$$

$$h_s\left(0,t\right) = z\left(t\right) \tag{6}$$

The first boundary condition requires that the hydraulic head equation converge to a known water table gradient $h_\infty^{/}$ (m/m) as u increases, whereas the second accounts for periodic river stage changes over time.

The solution to Equation (3) is

$$h_s\left(u,t\right) = h_0 + h_\infty^{/} u + \sum_{n=1}^{\infty} e^{-\varepsilon_n u}\left[S_n \sin\left(\eta_n t - \varepsilon_n u\right) + C_n \cos\left(\eta_n t - \varepsilon_n u\right)\right] \tag{7}$$

where h_0 (m) is the average of the $z(t)$ function over the time period; S_n (m) and C_n (m) are constants; n is an integer; and η_n (sec^{-1}) and ε_n (m^{-1}) are

$$\eta_n = \frac{2n\pi}{p} \tag{8}$$

$$\varepsilon_n = \sin\theta \sqrt{\frac{2s_s n\pi}{k_s p}} \tag{9}$$

Through standard methods for Fourier application, Equation (7) can be applied to any periodic stage function at $u = 1$. In particular, for sinusoidal $z(t)$ and $\theta = 90^o$, Equation (7) becomes one dimensional and collapses to the well known solution for tidal driven groundwater fluctuations (e.g. Furbish, 1997); and successfully applied by Lal et al. (2010) to describe canal effects. Expansions of the tidal solution by Fourier series were also developed by Nielsen (1990) for sloping beaches, wherein he utilized perturbation to derive similar equations but expressed in terms of x only and did not account for the long range water table gradient.

See Travis (2010) for examples and Fourier series application.

Unsaturated Flow

The saturated solution h_s is valid only when $h_s \geq y + \psi_b$, where ψ_b (m) is the pressure head at the air entry value (a negative value). When this condition is violated (e.g. $h_s < y + \psi_b$), flow is governed by the unsaturated head.

Travis (2010) analyzed the unsaturated head for short time periodic loading by simplifying the governing equation through a scale comparison of the unsaturated hydraulic conductivities and specific storativities as presented in several key studies. In general, however, this approach is not valid for applications to canals, where the timeframes of interest are much longer – on the order of months rather than hours. It is suspected that the long timeframes can be utilized in a similar matter to simplify the unsaturated analysis.

The present work follows simply ignores unsaturated flow, and thereby introduces some degree of error. This approach is weakly defensible by noting that it is consistent with numerous other studies. Future work will account for unsaturated flow by utilizing potential simplifications to the governing equations and insights provided by Li, Boufadel, and Weaver (2008).

Leakage

Leakage at the canal is governed by Darcy's law, $q = k_s i a$, where q (m^3/sec) is the leakage (negative for seepage), i is the hydraulic head gradient (m/m) and a (m^2) the flux area. Specific to the defined canal geometry, the leakage is given by

$$q = -2k_s \ell b^2 \tan\theta \left.\frac{dh_s}{du}\right|_{u=0} \tag{10}$$

where ℓ is the length of the canal. Note that Equation (10) accounts bank flows on both sides.

The fluctuating component of the leakage may be obtained by utilizing Equation (10) with the derivative (with respect to u) of Equation (7), and applying the derived formula at the bank. The result is

$$q = \left\{ -h_\infty^/ + \sum_{n=1}^{\infty} \varepsilon_n \left[\left(S_n + C_n\right)\cos\left(\eta_n t\right) + \left(S_n - C_n\right)\sin\left(\eta_n t\right) \right] \right\} 2k_s \ell b^2 \tan\theta \tag{11}$$

The gradient term in Equation (11) must be carefully considered. Because the leakage effect is local, assuming a large scale gradient would be incorrect. Instead, since it is expected that leakage will descend freely through the aquifer until encountering the water table, the long range hydraulic gradient is simply $h_\infty^/ = -1$ (m/m). Thus, the correct leakage formula is

$$q = 2k_s \ell b^2 \tan\theta \left\{ -1 + \sum_{n=1}^{\infty} \varepsilon_n \left[\left(S_n + C_n\right)\cos\left(\eta_n t\right) + \left(S_n - C_n\right)\sin\left(\eta_n t\right) \right] \right\} \tag{12}$$

On average, the trigonometric terms will average to zero, resulting in the simple formula

$$q = -2k_s \ell b^2 \tan\theta \tag{13}$$

CASE STUDY: INTERSTATE CANAL

Parameters

Harvey and Sibray (2001) describe a detailed, long term field study of leakage from Nebraska's Interstate Canal. The data from the Interstate Canal is particularly applicable given the potential that the leakage from the base of the canal may be limited, because of sand intrusion into the highly fractured aquifer (Harvey and Sibray, 2001). Thus, bank leakage may account for a large portion of the measured effects.

Downstream groundwater conditions appear to be governed by University Lake, located approximately 900 m downstream at an elevation approximately 14 m below the nearest point on the canal (see Figure 2).

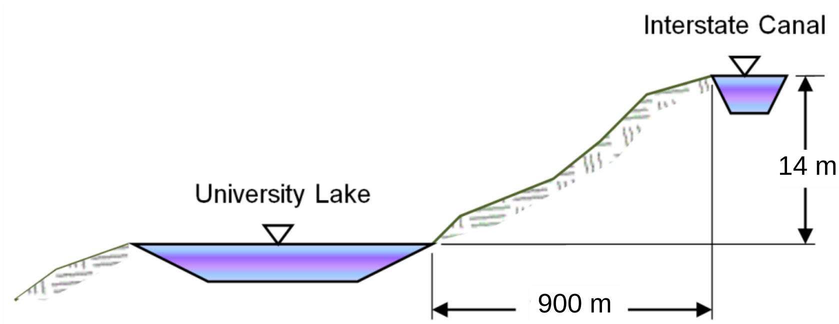

Figure 2. Interstate Canal Schematic

As shown in Table 1, Harvey and Sibray (2001) report hydraulic conductivities from a number of sources. Both horizontal (k_h) and vertical (k_v) values are shown, along with the median and mean values. Anisotropy is high, a complication resolved by determining an effective hydraulic conductivity.

Following Todd and Mays (2005) the effective hydraulic conductivity in the direction of University Lake is given in terms of the vertical and horizontal values by

$$k_s = \left[k_h^{-1} \cos^2 \beta + k_v^{-1} \sin^2 \beta \right]^{-1} \tag{14}$$

where β is the angle from the horizontal between the canal and the lake (0.6°). From Equation (14), the k_s based on both the mean and median values is approximately 10^{-2} m/sec, wherein rounding to the order of magnitude was applied to reflect the significant uncertainty of this approximation.

Likewise for the leakage calculation, where the gradient is governed by local conditions, applying Equation (14) results in estimates for k_s of 1 m / day for the median conditions and 10 m / day based on average values.

Table 1. Reported Hydraulic Conductivities in the Study Region

	k_h (m/day)	k_v (m/day)	Source	Comment
	6912	0.86	Sibray and Zhang (1994)	Reported average
	864	0.86	Barash and Ralston, (1991)	Reported lower bound
	3456	17.30	Barash and Ralston, (1991)	Reported upper bound
Mean:	**3744**	**6.34**		
Median:	**3456**	**0.86**		

Canal water surface elevations (WSE) were not reported by Harvey and Sibray (2001), but flows were. Assuming normal flow conditions, the WSE for the periods of record were converted from the reported flows, and are shown in Figure 3.

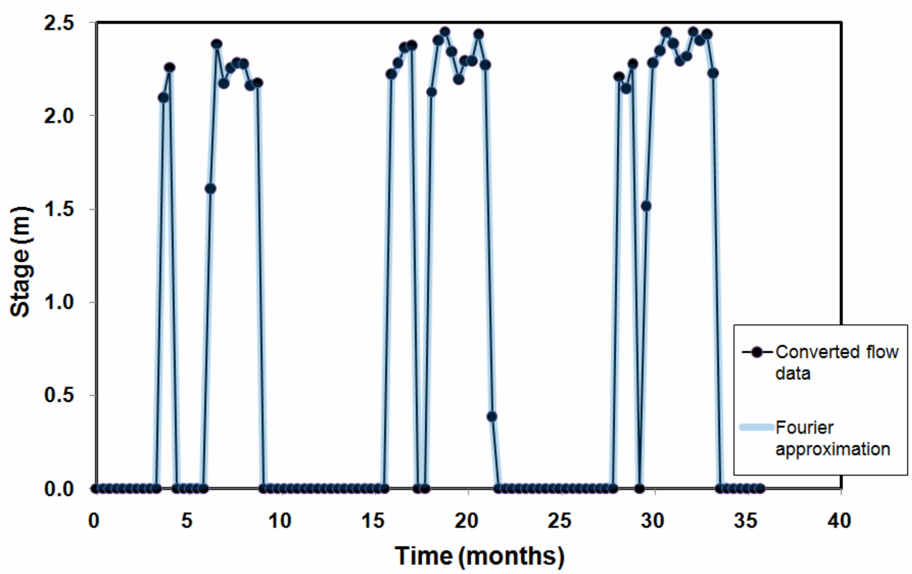

Figure 3. Converted Flow Data and Corresponding Fourier Approximation

For an unconfined aquifer, the specific storativity may be approximated as the porosity divided by the aquifer depth. Thus, assuming a porosity and depth of the shallow aquifer of 40% and 4 m respectively, s_s was estimated to be 10^{-1} m^{-1}.

The coefficients of the Fourier series were evaluated based on the converted flow data. The series was evaluated out to 500 terms which resulted in excellent convergence (Figure 3).

Model execution took about 10 seconds on an Excel spreadsheet.

Fluctuation Predictions

Figure 4 compares the predicted and recorded water table elevations at monitoring well 12, located just upstream of the University Lake (approximately 900 meters from the nearest point on the canal). The predicted elevations were based on hydraulic head estimates projected from the average water surface elevation at the canal (elevation 1277.7).

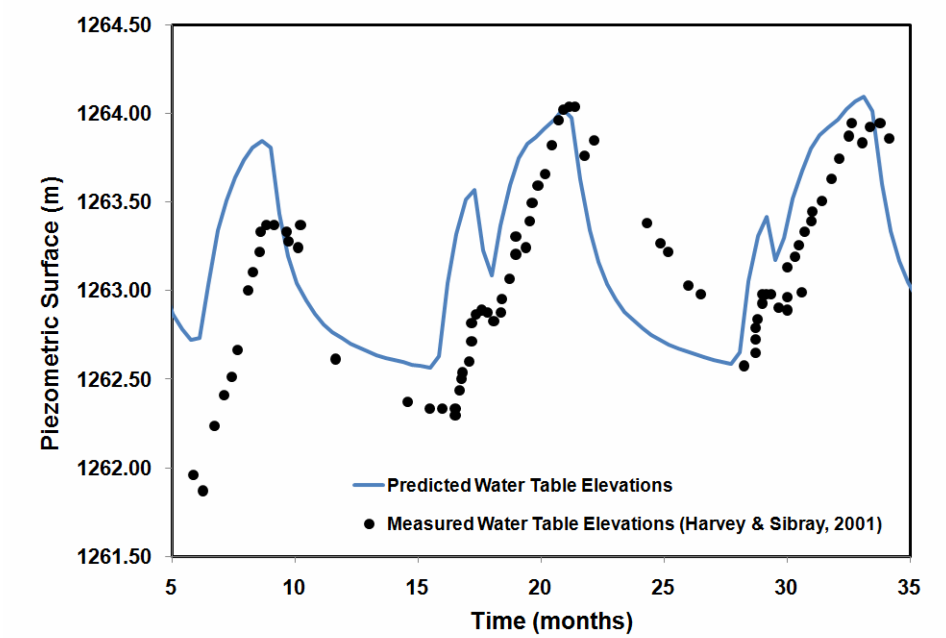

Figure 4. Predicted Versus Measured WSE at Monitoring Well 12

The predicted water table approximates the recorded data with notable similarities and differences. The similarities are:

1. The peak elevations for the second and third seasons are in good agreement;
2. A smaller peak at each season is predicted, consistent with the data;
3. The recorded concavity of the water table recession is predicted;
4. The predicted low water table elevation at the last recorded season is in good agreement with the recorded data.

The differences between the predicted and measured water table elevations are:

1. Although the initial, smaller water table peak at the beginning of each season is predicted, the predicted magnitude is much higher than recorded;
2. The predicted peak elevation at the first season is much higher than recorded, as is the low elevation;
3. The predicted recession limb of the second season is much lower than measured.

Some of the observed differences between the predicted and measured water table elevations result from the assumption of periodic conditions. The result of the periodic assumption is that the best fit is expected near the middle of the time range considered. It is also likely that the predictions would be improved through a more rigorous accounting of the hydraulic conductivity anisotropy and a field measurement of the specific storativity.

Leakage

From Equation (13), the estimated leakage per unit length of the canal when flowing full is about 7 m^3/day/m and 70 m^3/day/m for the median and mean hydraulic conductivities respectively. This compares reasonably with field measurements indicating that $6 \le q \le 35$ m^3/day/m (Harvey and Sibray, 2001; attributed to Ann Bleed).

CONCLUSIONS

By applying the methodology developed for describing riverbank porewater effects of regulated flows, several analytical solutions have been presented to describe leakage and seepage effects in canals. These solutions account for canal bank geometry, driving upstream and / or downstream water table gradients, and time varied irrigation flow schedules given by any piecewise continuous function.

While several measurements of the Interstate Canal verify the derived porewater response model, there are other, potentially more important applications that should be considered. Several examples include:

1. **Canal bank stability**. Bank stability is often critically dependent on the internal seepage processes, particularly for applications such as canals where the adjacent flows are mild. By coupling the derived model with a slope stability program, a powerful analytical tool would be developed.
2. **Canal design**. With a simple analytical method of estimating leakage and seepage, canals could be located and designed to minimize these effects and potentially avoid the necessity of lining part or all of the canal.
3. **Aquifer property measurements.** A useful application of the model presented here is to utilize the Lal et al. (2010) field measurement techniques without the need to change the scheduled flows. Since the model can incorporate any existing flow schedule, the measured data can be compared with predicted results and the aquifer properties thereby calibrated.

While useful, the derived model needs to be expanded in order to adequately account for matric suction effects, which is likely to affect the porewater responses as well as the water retained in the banks, both of which need to be considered for bank stability.

REFERENCES

Barrash, W., and Ralston, D.R. (1991). “Analytical modeling of a fracture zone in the Brule Formation as an aquifer receiving leakage from watertable and elastic aquitards”, *Journal of Hydrology* 125, 1-24.

Boutt, D. F. (2010). “Poroelastic Loading of an Aquifer Due to Upstream Dam Releases” *Ground Water*, 48: 580–592.

Darcy, H. (1856). *Les fontaines publiques de la ville de Dijon*, V. Dalmont, Paris, 647 pp.

Fredlund, D.G. and Rahardjo, H. (1993). *Soil Mechanics for Unsaturated Soils*, John Wiley and Sons, Hoboken, N.J.

Furbish, D.J. (1997). *Fluid Physics in Geology*, Oxford University Press.

Haitjema, H., Feinstein, D., Hunt, R. and Gusyev, M. (2010). "A Hybrid Finite-Difference and Analytic Element Groundwater Model", *Ground Water*, 48: 538–548.

Harvey, F.E., and Sibray, S.S. (2001) "Delineating ground water recharge from leaking irrigation canals using water chemistry and isotopes", *Ground Water*, 39(3), 408 – 421.

Lal, A.M.W., Dhanapala, N., Gunadasa, W.K., Jayasingha, J.A.S.A., Liyanagama, B.S., Laltiha, S., and Lal, W. (2010). "Use of sinusoidal discharge pulses to determine canal and aquifer parameters in the Jaya Ganga canal of the Kala-Oya basin of Sri Lanka", *International Perspective on Current and Future State of Water Resources*, Chennai, India, January 5-7,

Li, H., Boufadel, M.C., and Weaver, J.W. (2008). "Quantifying bank storage of variably saturated aquifers", *Ground Water*, 46(6), 841 – 850.

Lorenzo, G.A., Bergado, D.T., Bunthai, W., Hormdee, D., and Phothiraksanon, P. (2004) "Innovations and performances of PVD and dual function geosynthetic applications", *Geotextiles and Geomembranes*, 22, 75 – 99.

Mays, L.W., and Todd, D.K. (2005). *Groundwater Hydrology*. John Wiley and Sons, Inc. McGraw-Hill, New York.

Nielsen, P. (1990). "Tidal Dynamics of the Water Table in Beaches", Water Resources Research, 26(9), 2127 – 2134.

Sibray, S.S., and Zhang, Y.K. (1994). "Three-dimensional modeling of hydraulic behavior of a highly conductive secondary permeability zone in the Brule Formation". In *Proceedings of the* 1994 *Groundwater Modeling Conference,* ed. J.W. Warner and P. van der Heijde, 445 – 452. Colorado State University, Fort Collins, Colorado.

Swamee, P.K., Mishra, G.C., and Chahar, B.R. (2000). "Minimum cost design of lined canal sections", *Water Resources Management,* 14, 1 – 12.

Thomas, J.T., Iverson, N.J., and Burkart, M.R. (2009). Bank-collapse processes in a valley-bottom gully, western Iowa", *Earth Surface Processes and Landforms*, 34(1), 109 – 122.

Travis, Q.B. (2010). *Ebb and Flow: Preserving Regulated Rivers through Strategic Dam Operations*, Doctoral Dissertation, Arizona State University, Tempe.

INTEGRATED WATER SUPPLY PLANNING FOR SUSTAINABILITY — A CASE STUDY OF THE ALBUQUERQUE BERNALILLO COUNTY WATER UTILITY AUTHORITY

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Greg Gates, P.E.[2]

ABSTRACT

In the mid-1990s the Albuquerque Bernalillo County Water Utility Authority (Authority) embarked on developing and implementing a Water Resources Management Strategy (Strategy). The Strategy considered multiple water supply alternatives in a triple bottom line framework through a structured multi-attribute decision process. This process allowed for a development of a logical defensible strategy that could meet the long-term needs of water users. This process also resulted in a framework to evaluate more than 32 supply alternatives, realized in a suite of supply projects creating a diverse water supply portfolio. Chosen projects included, wastewater reuse, surface water supply, groundwater production, and ASR.

In the past decade the Authority has implemented a number of these projects-diversifying its water supply portfolio. Implemented projects include the Nonpotable project; the Bear Canyon Arroyo Pilot ASR Project; the San Juan-Chama Drinking Water Project; and the Southside Reuse Project. The Authority has also demonstrated national leadership through successful implementation of its water conservation program.

Today, the Authority is completing design of the Large-Scale ASR project.

The Strategy has been continuously updated and refined as new information becomes available or as lessons learned are realized. The Authority is in the process of evaluating additional reuse demand as well as water availability of municipal wastewater for future reuse and ASR projects and is assessing the potential effects of climate change on available supply. In addition, the Authority is developing a dynamic simulation screening tool that will allow for rapid analysis of multiple alternatives for future long-term sustainable supply.

INTRODUCTION AND BACKGROUND

In 1962 Albuquerque purchased rights to San Juan-Chama (SJC) water in order to meet its future demands. The San Juan-Chama project imports a portion of New Mexico's share of water from the Colorado River Compact to the Rio Grande basin. Planning from the 1960s suggested that this water supply source could be used over time to offset groundwater depletions of surface water allowing for continued aquifer pumping over time. It was an elegant and simple way to provide Albuquerque's water supply and meet New Mexico Office of State Engineer requirements: the aquifer would supply the community, the river would re-supply the aquifer, and San Juan-Chama water would re-

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supply the river. Plans suggested that it would be many decades before the full San Juan-Chama supply would be needed. In the mean time, this water was stored or leased to others for beneficial use.

In the late 1980s and early 1990s hydrologists working in the Middle Rio Grande Basin began to note that aquifer drawdown was occurring at a faster rate than anticipated based on the current understanding of the aquifer system. A plan of study was developed to better characterize the hydrogeology resulting in conclusions that: 1) the aquifer was not as connected to the river as previously thought, 2) the aquifer was not as large as previously thought, 3) long-term aquifer drawdown would likely be problematic – potentially resulting in land-surface subsidence, and 4) naturally occurring As contaminated a number of areas of the aquifer. Each of these conclusions suggested that the assumptions that led to the 1960s plan were invalid and that a new Water Resource Management Strategy (WRM Strategy) was needed.

Community leaders at this time chose to embark on a technically based, comprehensive, transparent, and defensible process to develop a sustainable water supply. The process was a true *integrated water resource planning* effort in that it considered an array of potential water supply options and combinations of options and their common interrelation to the overall hydrogeologic system, water rights, and the community.

The following paragraphs describe strategy development and implementation.

STRATEGY DEVELOPMENT

Strategy development by design included extensive stakeholder involvement. A multi-tiered series of teams interacted to guide the process at governmental, community, public, and project levels. To facilitate this involvement, a step-wise scalable multi-attribute decision process model was used. This model allowed for a clearly documented decision process with multiple stakeholders throughout the process and across teams. Likewise, because strategy decisions were made on issues ranging from selection of the best component parts of future water supply, to site selection for the water treatment plant, to process train selection, this model was applied in a consistent manner with related criteria across multiple levels of resolution. This approach allowed for consideration of the triple bottom line throughout the decision process and ensured consistency across decisions with the overarching goals set forth in the strategy.

Stakeholder Involvement

To facilitate a transparent community process, extensive public involvement was sought early in the process. A Steering Committee was developed from City personnel and community leaders to provide valuable review and oversight to the planning process. This Steering Committee guided the project team in decision making. The Project Team consisted of key Authority staff and outside consultants.

The Steering Committee helped the project team formulate a set of 32 alternatives for use of existing water resources. The public and stakeholders (agencies and groups interested in the outcome) suggested many aspects of these alternatives. Each alternative included a specific and practical means to meet future water demands using existing water supplies.

The alternatives addressed management, conservation, water and infrastructure development, and water quality management. The project team assessed the hydrologic effects of each alternative on the aquifer and river system; developed conceptual cost estimates; and considered social, political, and institutional issues. To evaluate the alternatives, the team developed system-level evaluation criteria based on community values and technical reality. Evaluation of the 32 alternatives considered:

- Long-term reliability and sustainability, including degree of beneficial use of existing renewable surface water sources, reduction in aquifer mining, and preservation of a ground-water reserve for use in times of drought.
- Protection of valued environmental resources, including physical, hydrological, and environmental impacts.
- Project implementability, including technical and political feasibility, the ability to obtain necessary permits, implementation schedule, and public support.
- Ability to support the quality of life in the region, including socioeconomic benefits, basic water and sewer services, public amenities such as parks and greenbelts; and the equitable sharing of costs and benefits in terms of social, cultural, and generational considerations.
  Financial feasibility, including life-cycle costs of facilities and potential costs related to other water quality issues and responses to drought.

Decision Process

For each decision, the decision making process was completed in a similar manner.

Steps 1 and 2 – Kickoff meeting with development of goals, objectives, problem definition

Step 3 - Workshop to develop evaluation criteria, weight criteria, and develop alternatives

Step 4 - Data collection and preliminary scoring of alternatives

Step 5 - Workshop to confirm scoring and complete preliminary evaluation, identify data gaps, and plan subsequent data collection / testing

Step 6 - Complete evaluation and identify preferred alternative(s). Develop concept report based on preferred alternative(s)

Each step is scalable based on the complexity of the decision. For example for a large-scale water resources problem, Step 3 may take place over a series of workshops. Whereas for a better defined or less complex problem, this step may take place in a single meeting. In addition, this process can be construed as a feedback loop – if circumstances change or assumptions prove to be false, the process can be revisited with new information. Figure 1 shows a flow diagram of this process.

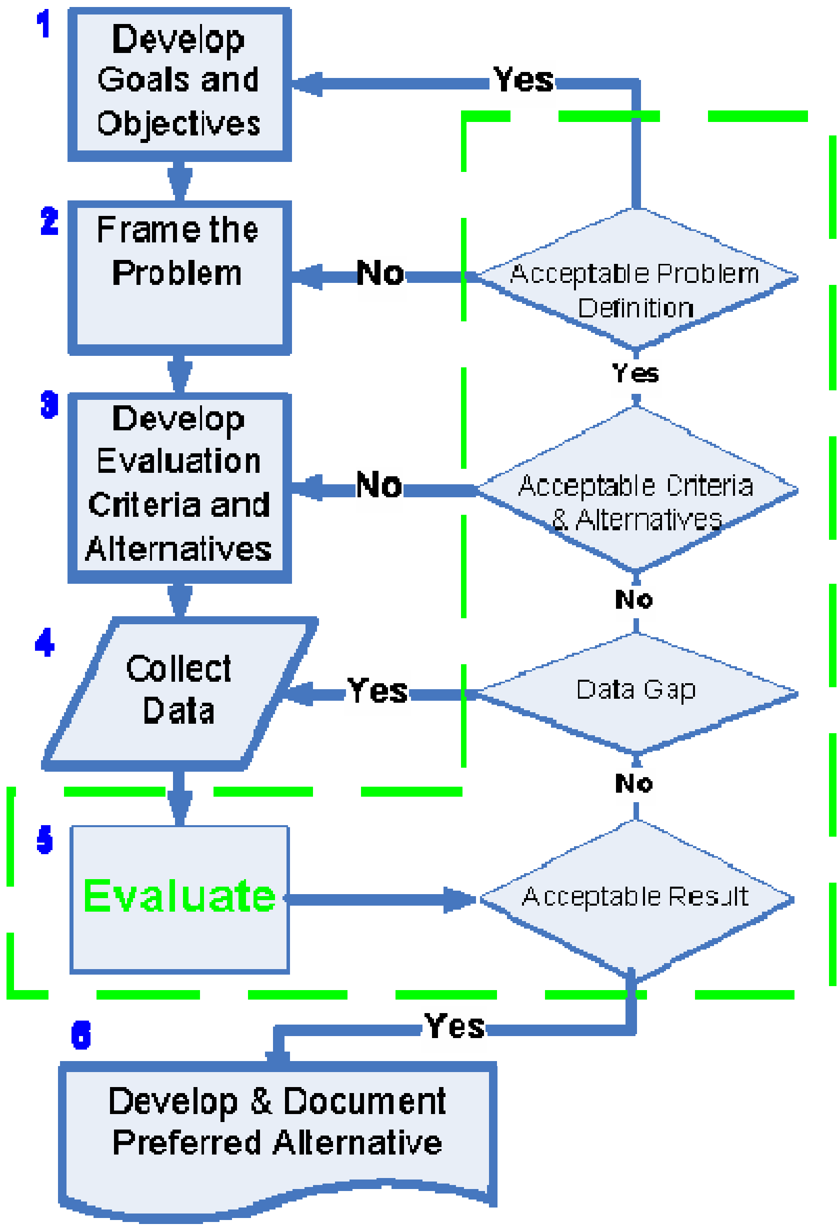


Figure 1. Generic Decision Flow Diagram

To aid in clarity, decision criteria should be independent (avoids double counting or over-representation), differentiate the alternatives, and objectively quantifiable. In this way the decision process is repeatable and not subject to interpretation. Likewise, every component of a problem need not be represented, only the criteria needed to make a decision.

This general framework was used for decision making throughout strategy development.

<u>Snapshot No. 1 - Water Supply Alternatives</u> As noted previously, the project team with input from the Steering Committee developed 32 alternatives for evaluation. These alternatives included combinations of water supply and demand reduction alternatives such as conservation, ASR, reuse, direct diversion, etc.

To select from these 32 alternatives, decision criteria were developed to examine the water rights balance over time and the alternatives effects on local resources. Because the supply and to some extent demand sources were intertwined, a water balance model was developed that examined the interrelationship of the various supply sources and balanced these effects with available water rights. This model allowed for equal comparison among alternatives.

This analysis was completed through the use of a groundwater flow model to examine aquifer pumping effects on streamflow coupled with the water balance model. These tools were used to look at future drawdown magnitude and rate and ability to meet demands over the long term while keeping the river whole and meeting water rights requirements. Figure 2 shows model output of drawdown from a consumptive use scenario that was used in the decision process.

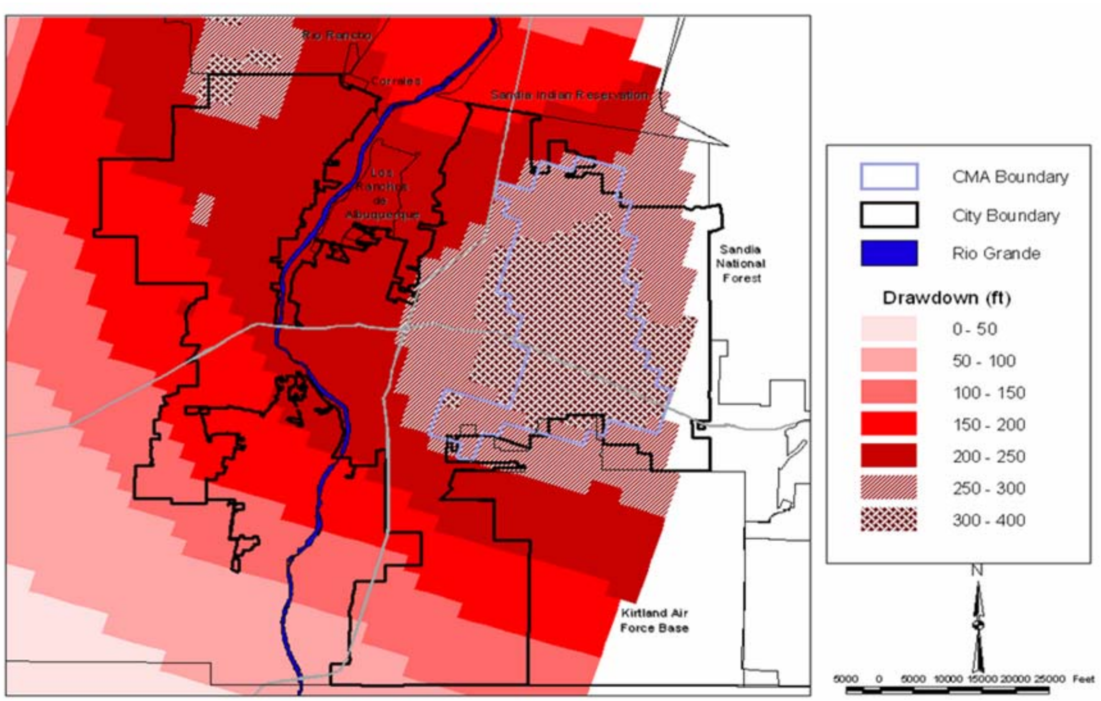


Figure 2. Decision Criteria - Modeled Projections of Drawdown in 2060

Once these effects were quantified and evaluated for fitness, alternative costs were applied for further differentiation.

The alternative found to have the greatest non-monetary benefit for the least costs included as a key feature direct diversion and full consumption of the Authority's San Juan-Chama supply along with a number of reuse projects, conservation and drought management, and aquifer storage and recovery.

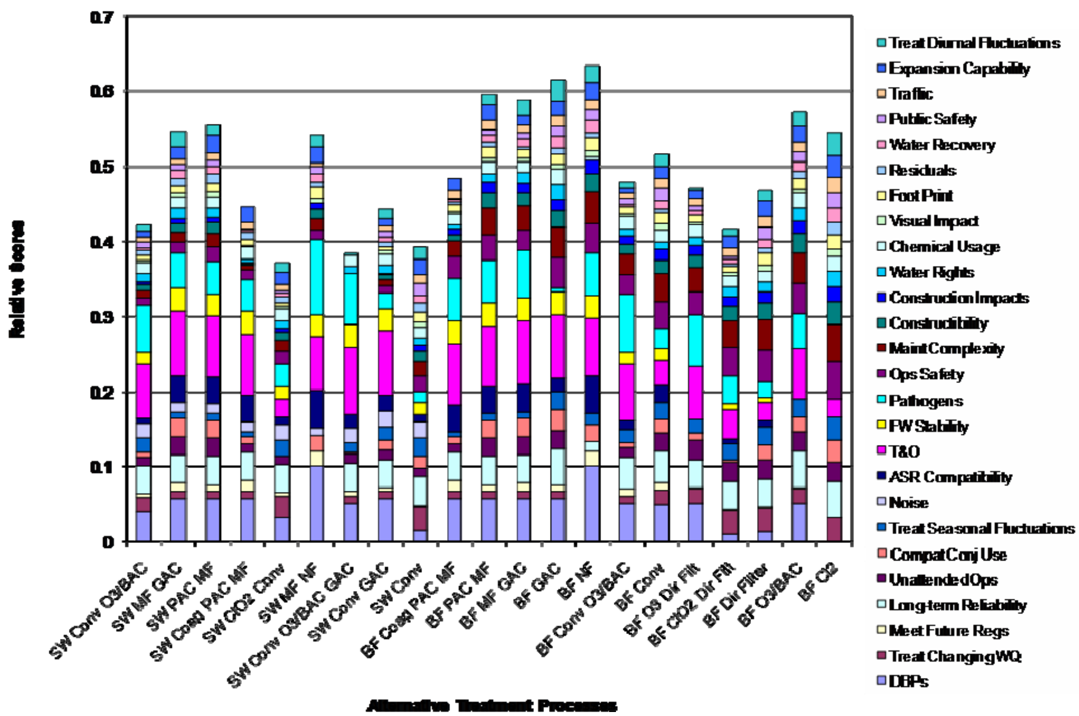
Snapshot No. 2 – Treatment Process Once the overall water supply strategy was selected, a number of other decision processes were used to examine individual elements within the strategy. One of these elements was treatment train process for the surface water treatment plant.

Twenty one treatment alternatives were developed from a combination of technologies with the following major elements:

- Major alternatives: Surface water diversion, River bank filtration
- Unit processes: Coagulation, enhanced coagulation, Direct filtration, conventional treatment w/ clarification, Ozone, chlorine dioxide, chlorine, GAC, PAC, biofiltration, Particle membranes, granular media filtration

Figure 3 shows the benefit scores associated with the non-monetary benefits for the 21 alternatives. Figure 4 shows the final alternatives after ranking and costs.

Figure 3. Decision Process - Non-Monetary Benefits



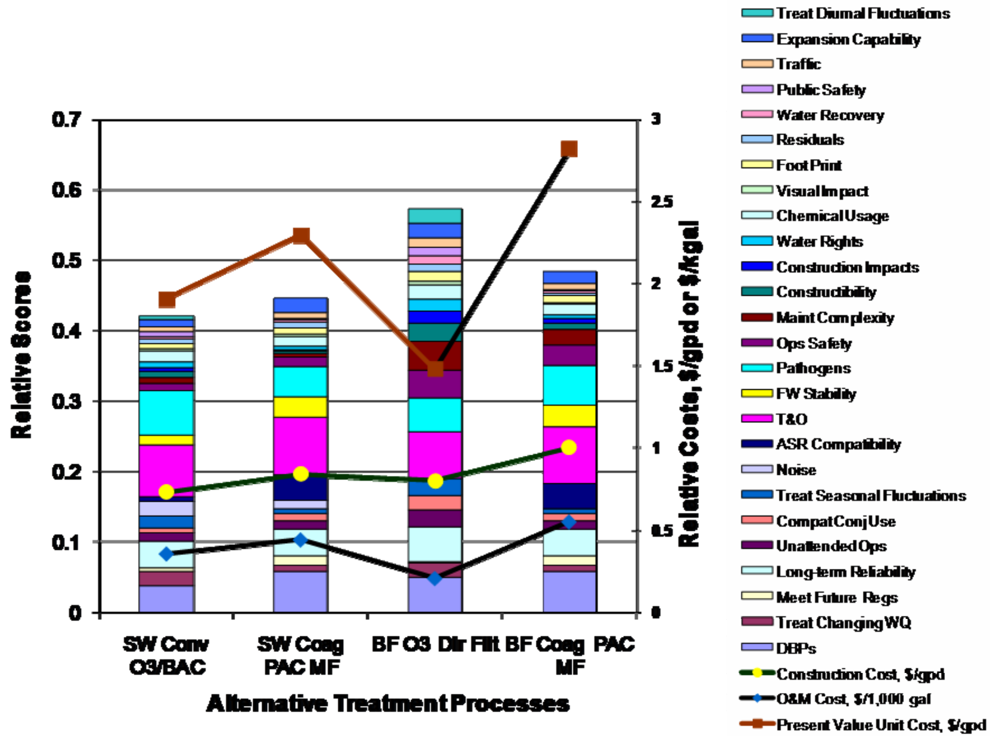


Figure 4. Decision Process - Rank with Costs

When costs were applied and infeasible options ruled out, enhanced conventional ozone with biological activated carbon was chosen as the most sustainable.

Having a consistent process with overarching goals helped to produce a purposely integrated strategy that on the whole and throughout its elements meets the values and objectives of the community and provides for the water supply needs of the community.

IMPLEMENTATION

Conservation

Recognizing the immediate value of permanently reducing water demand, the water conservation program was started in 1994 – prior to completion and implementation of the WRM Strategy. A 30 percent reduction in gallons per capita day (gpcd) use was targeted by 2006. This goal was further revised in 2000 to reflect an additional 10 percentage point reduction to approximately 150 gpcd by 2014. In addition, the Water Authority developed a drought management plan to preserve groundwater. The plan provides for short-term demand reductions to offset increased groundwater production. The Authority remains ahead of schedule at about 163 gpcd in 2009 for meeting its conservation goal as shown in Figure 5.

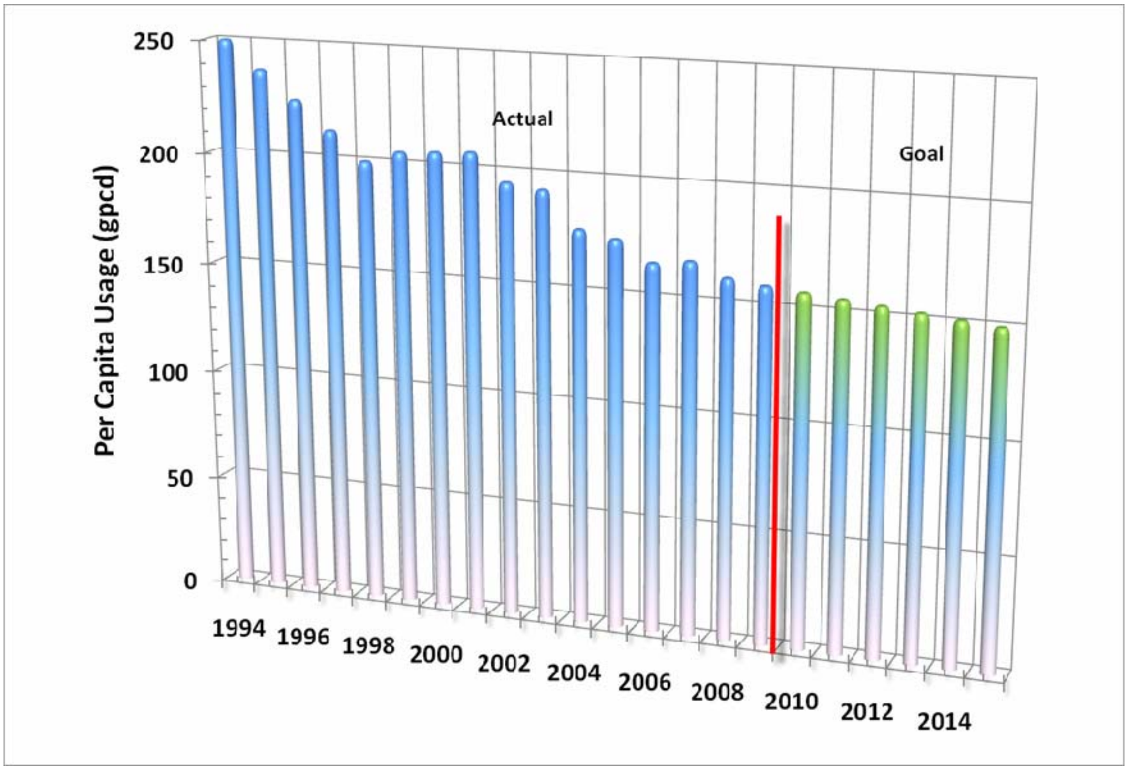


Figure 5. Authority Conservation Progress and Future Goal

Non-Potable Project

A non-potable supply project was implemented in the Northeast quadrant of the City to irrigate large turf areas with industrial wastewater and untreated surface water. This project provides about 3,000 ac-ft per year of supply and contributes in the non-irrigation season to the Authority's aquifer storage and recovery program. Surface water is diverted through a subsurface collector that removes sediment and provides treatment as water percolates through the alluvial aquifer. The collector was originally envisioned as a pilot diversion for the drinking water project and includes both radial collector wells at depth and more shallow infiltration galleries directly beneath the Rio Grande. Water is supplied to Albuquerque's Balloon Fiesta Park, Arroyo del Oso and Tanoan Golf Courses, and numerous parks and athletic fields. Figure 6 shows the project site as represented in a finite element model.

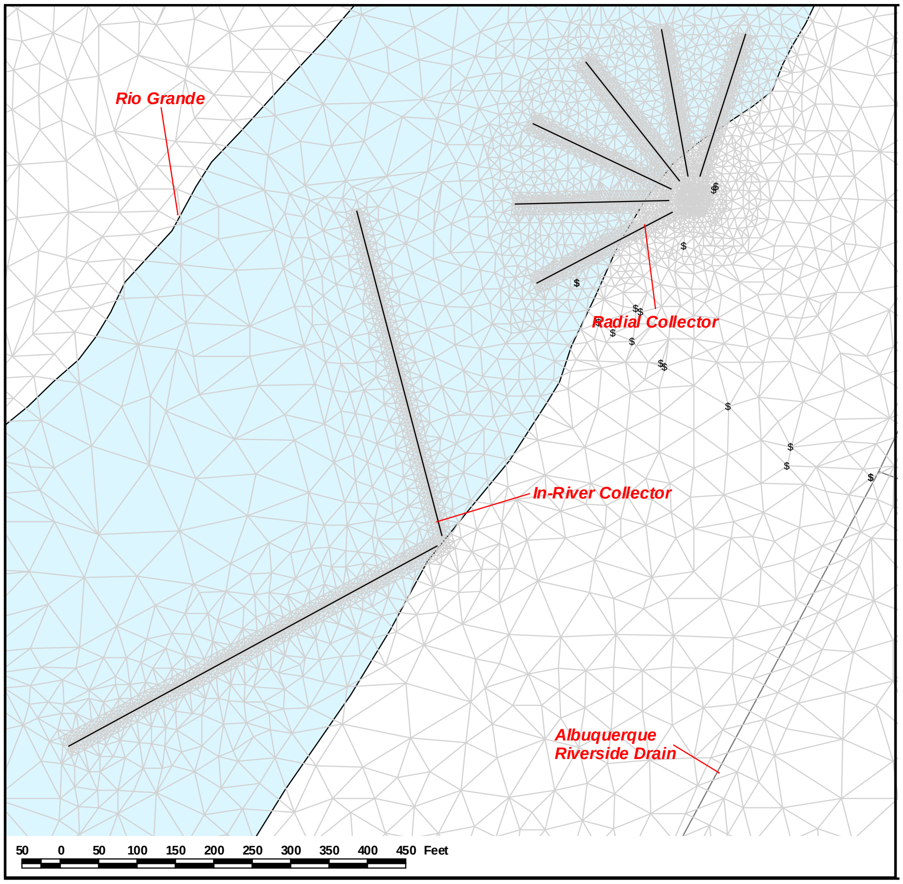


Figure 6. MicroFEM Model Grid of the Non-Potable Diversion System

Drinking Water Project

The San Juan-Chama Drinking Water Treatment Plant is the centerpiece of the Water Authority's 15-year transition to a renewable surface water supply. The 92-million-gallon-per-day (mgd) expandable to 120 mgd, state-of-the-art water treatment plant (WTP) is Albuquerque's first facility for treating surface water and the largest surface water treatment facility in New Mexico. By diverting and treating surface water rather than groundwater, the plant provides a sustainable water supply for future generations.

The water treatment system includes grit and settling basins and provides for ozonation and chlorination disinfection to address current and pending safe drinking water standards. The WTP also includes a patented Actiflo system for colloidal material removal and deep bed granular activated carbon filtration for final water polishing. All plant flows are re-circulated to the head of the treatment process, thus eliminating off-site process waste streams. The Authority began diverting and treating surface water in 2008 and anticipates that the facility will reach full capacity in 2011. Figure 7 presents the WTP

Figure 7. The Water Treatment Plant Administration Building

Southside Reuse

The Southside Reuse project was developed to provide treated municipal and industrial wastewater for non-potable irrigation of large turf areas in southeast Albuquerque. The project also provides supply for a large County sports complex at Mesa del Sol. This project provides a little less than 3,000 ac-ft/yr of supply. Figure 8 presents a map of the Southside Reuse Project.

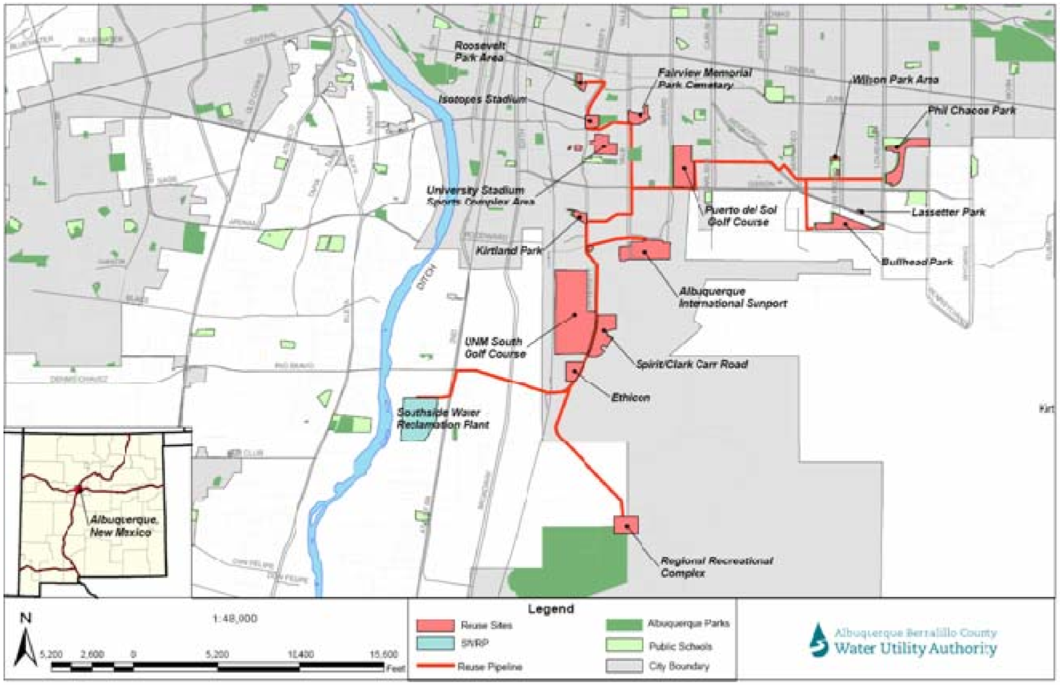


Figure 8. The Southside Reuse Project

Aquifer Storage and Recovery

The Bear Canyon Arroyo Pilot project was operated over 2007 through 2009 to demonstrate the viability of surface infiltration of non-potable water to the aquifer. Water for this project was derived from excess capacity in winter months of the previously described Non-Potable project. Water quality sampling, temperature logs, and modeling demonstrated that water allowed to slowly flow over the surface in an existing native channel would infiltrate to the groundwater aquifer. A little over 1,000 ac-ft was stored through this pilot project.

NEXT STEPS

The Water Authority recognizes that providing a sustainable supply is not a static process and that planning must be continuously revisited to ensure that long-term goals are met. The Water Authority continues to make strides in conserving water and is in the process of advancing the Large Scale ASR project. This project will allow for the use of excess water treatment plant capacity in the winter months to provide aquifer storage for future years and potentially provide a vehicle for other as yet unidentified ASR projects.

The Water Authority has examined both potential reuse demand and available wastewater supply to estimate the timing and magnitude of future reuse opportunities. This analysis is coupled with examination of potential new wastewater “scalping” plants that could expand the geographic reach of the reuse system.

The Water Authority is also developing an Integrated Water Balance Model at this time to help facilitate planning for a variety of future conditions and water supply options including the full range of historic hydroclimatic variability as well as the potential for climate change. Figure 9 shows a typical layout in a water supply dynamic simulation model.

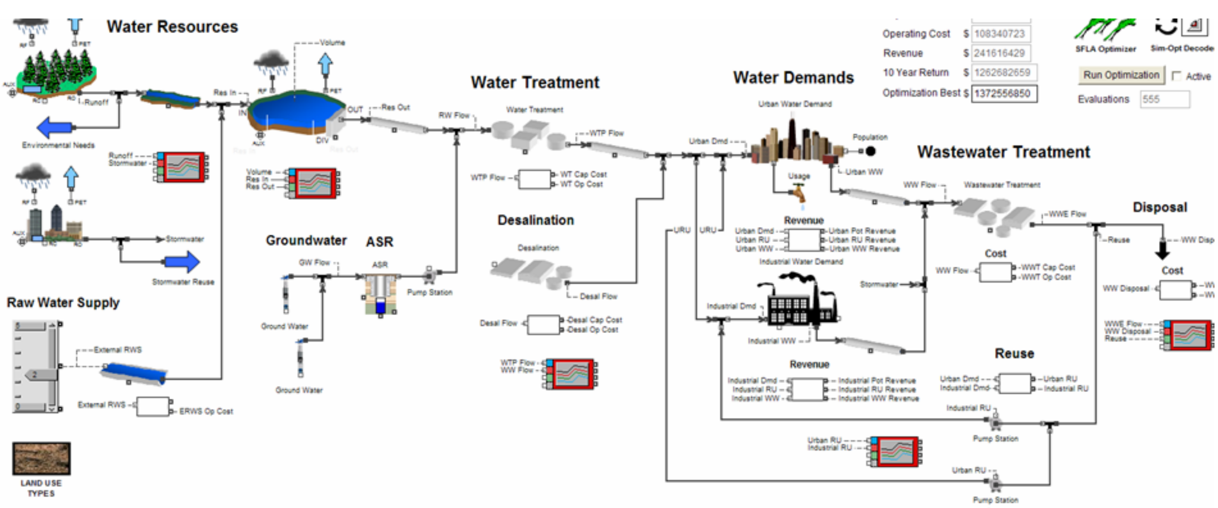


Figure 9. Representation of a Water Supply System in a Dynamic Simulation Model

REFERENCES

CH2M HILL, 1995. Albuquerque Water Resources Management Strategy, San Juan-Chama Diversion Project Options, Appendices A, B and C. Prepared for the City of Albuquerque. July.

CH2M HILL, 1997. City of Albuquerque Water Resources Management Strategy. Prepared for the City of Albuquerque, New Mexico. May.

CH2M HILL, 2002. Hydrologic Effects of the Proposed City of Albuquerque Drinking Water Project on the Rio Grande and Rio Chama Systems: Final Report for OSE Hearing. Prepared for City of Albuquerque Department of Public Works. Authors: Walter G.Hines, P.E. and Gregory B. Gates, P.E.

WATER FOR IRRIGATION, STREAMS AND ECONOMY: EVALUATING PAST AND FUTURE CLIMATE CHANGE TO SECURE A RELIABLE WATER SUPPLY FOR MULTIPLE NEEDS

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Steve Mason[2]
Ronan Igloria[3]
Anders Rasmussen[4]

ABSTRACT

In the Little Butte Creek and Bear Creek watersheds in southern Oregon a regional, cooperative effort among water users and stakeholders is working to improve water quality and quantity for irrigation, aquatic habitat, and municipal/domestic and other uses in an economically and environmentally feasible manner. The project is called Water for Irrigation, Streams and Economy (WISE). WISE has six primary partners which includes municipalities and irrigation districts. Additionally, a Project Advisory Committee (PAC) also includes U.S. Bureau of Reclamation (”Reclamation”), Oregon Water Resources Department (OWRD) and Oregon Department of Environmental Quality (DEQ). Initial technical screening of conceptual projects that could address the WISE goals includes piping irrigation canals, limited reservoir expansion, and water reuse projects. An operational model was developed using the MODified SIMyld (MODSIM) software. Assessments using the model included evaluation of water reclamation, groundwater-surface water impacts, past climate, and future climate change. The later coupled several global circulation models from the International Panel on Climate Change (IPCC) with snow accumulation/melt and crop irrigation requirement models to estimate potential changes in agricultural water needs as well as changes in the magnitude and occurrence of stream flows. The result of the modeling effort contributed to quantified recommendations regarding projects and phasing which will be further developed and evaluated in a subsequent feasibility study/environmental impact statement.

INTRODUCTION AND SETTING

Jackson County in southern Oregon was one of the first European settled areas of Oregon. The County is located in the Rogue River basin, which includes the Bear Creek and Little Butte Creek watersheds. The Bear Creek Watershed includes six municipalities within its boundaries, including the Cities of Medford, Ashland, Talent, Central Point, Phoenix, Jacksonville, and White City. The City of Eagle Point is the only municipality within the Little Butte Creek watershed boundaries. The majority of the land use in the

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two basins is agricultural with irrigation primarily served by the U.S. Bureau of Reclamation's Rogue River Basin Project ("Project") and the Talent, Medford, and Rogue River Valley Irrigation Districts.

The majority of water used in the WISE study area is surface water from Bear and Little Butte Creeks and their tributaries. A significant feature of the Rogue River Basin Project are multiple interbasin and interwatershed transfers used in providing water to the irrigation districts, which are primarily located in Bear Creek. Figure 1 illustrates the operational schematic of the Project (USBR, 2009). Table 1 summarizes the major reservoirs in the Project.

The highest snowpack in the basin, averaging 60 inches, occurs west of the Cascade Mountains in the Fourmile Creek watershed. Fourmile Lake and Fish Lake, originally natural lakes, store the spring snow melt. The Cascade Canal diverts flow from Fourmile Lake in the Klamath basin across the Cascade divide to Fish Lake on the North Fork of the Little Butte Creek. The Little Butte Creek watershed is bounded on the north by Big Butte Creek, on the south by Bear Creek, on the west by the Rogue River, and on the east by the Cascade Divide. Little Butte Creek flows from its headwaters in the Cascade Mountains northwest about 43 miles to its confluence with the Rogue River. Elevations in the watershed range from 7,300 feet to about 1,200 feet at the confluence with the Rogue River. A portion of river and storage flows are diverted by the Joint System Canal into the Bear Creek watershed. Agate Reservoir serves primarily as a reregulating feature along this later canal, and is typically emptied by the end of the irrigation season.

In the South Fork of the Little Butte Creek, a series of canals partially captures snow-melt flows. These flows are transferred into the Klamath River basin and stored in Howard Prairie Lake. Releases from this Lake and also Hyatt Reservoir reenter the Bear Creek watershed via the Green Springs Powerplant Tunnel. Part of the Bonneville Power Administration, the Green Springs plant has an installed capacity of 17,290 kW.

The Bear Creek watershed is flanked by the Siskiyou Mountains on the west and the Cascade Mountains on the east in the southeast corner of the Rogue River Basin. The high point in the Bear Creek watershed is Mt. Ashland at approximately 7,500 feet, and the lowest elevation is at Bear Creek's confluence with the Rogue River at an elevation of 1,160 feet. Bear Creek Valley is approximately 25 miles long and ranges from 2 to 6 miles wide. The Bear Creek watershed has a drainage area of about 383 square miles–about 8 percent of the Rogue River Watershed. The watershed is characterized by steep gradients, shallow soils, and limited groundwater availability. The mainstem of Bear Creek, formed by Emigrant and Neil Creeks, flows approximately 27 miles northwest to its confluence with the Rogue River.

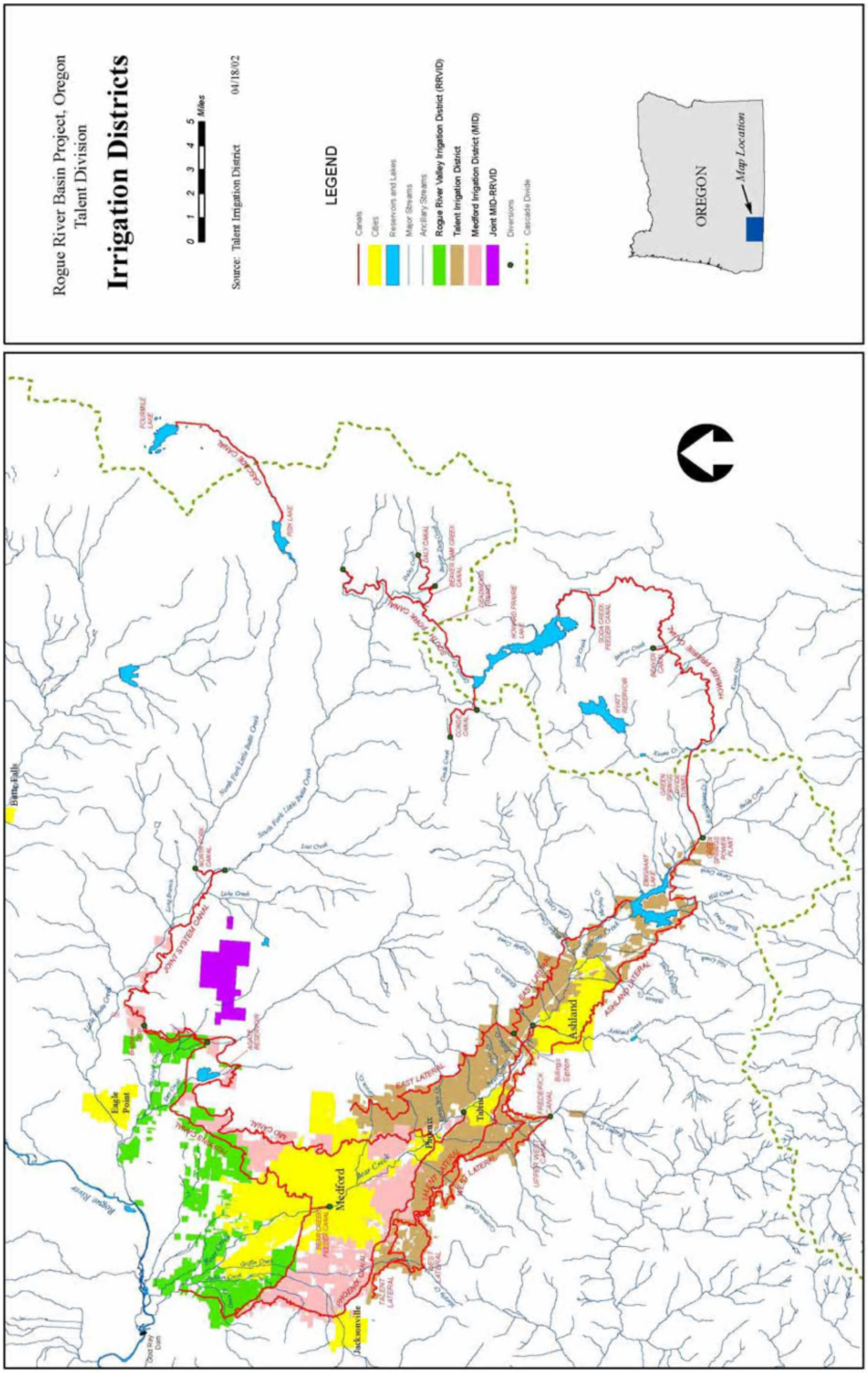

Figure 1. Operations Schematic of the Rogue River Basin Project (USBR, 2009)

Table 1. Rogue River Project Reservoirs

Reservoir	Location		Storage [ac-ft]
	River Basin	Watershed	
Fourmile Lake	Klamath River	Fourmile Creek	15,600
Howard Prairie Lake	Klamath River	Klamath River mainstem	62,100
Hyatt Lake	Klamath River	Klamath River mainstem	16,200
Little Hyatt Reservoir	Klamath River	Klamath River mainstem	370
Emigrant Lake	Rogue River	Bear Creek	40,500
Fish Lake	Rogue River	Little Butte	7,900
Agate Reservoir	Rogue River	Little Butte	4,800
Total Capacity			147,470

The Talent Irrigation District (TID) is the southernmost district in the Bear Creek watershed extending from the lower eastern slope of the Cascades to the southern end of the city of Phoenix. One arm of the irrigation district extends around the southwest of Phoenix, skirts the southwest edge of Medford, and terminates about one mile from Jacksonville. The other arm of TID skirts the northeast side of Phoenix and abuts a portion of the lower southeast edge of Medford. The cities of Ashland and Talent are within the boundaries of the Talent Irrigation District.

The Medford Irrigation District (MID) in the Bear Creek watershed abuts the northwest boundary of Talent Irrigation District and extends both to the northwest and northeast around Medford. The northern boundary of Medford Irrigation District abuts the southern edge of the Rogue River Valley Irrigation District (RRVID). Rogue River Valley Irrigation District bisects the city of Medford continues northwest in the Bear Creek watershed to the Rogue River and extends northeast into the Little Butte Creek watershed, coming within about one mile of the southern edge of Eagle Point.

TID and MID were organized in 1916 and 1917, respectively. The service areas are similar, with TID containing approximately 16,000 acres and MID 12,000 acres. RRVID was organized in 1929 and services 9,000 acres. Table 2 provides typical cropping patterns for each irrigation district. Orchards, including pears, are a significant part of TID and MID.

For an extended description of the Rogue River Valley Project Features, see Vinsonhaler, 2002.

Table 2. Irrigation District Cropping Patterns

Crop	Irrigated Area [acres]			Total	
	TID	MID	RRVID	Acres	%
Cereals	50	610	90	750	2.0%
Forage	9,950	2,850	8,050	20,850	55.8%
Orchards	3,320	3,490	450	7,260	19.4%
Grapes and Berries	660	240	1	901	2.4%
Legumes	0	4	0	4	< 1%
Roots, Tubers	25	460	0	485	1.3%
Vegetables	63	128	0	191	< 1 %
Other	2,180	4,270	450	6,900	18.5%
TOTAL	16,248	12,052	9,041	37,341	100.0%

Common Water Problems and Common Solutions

The Bear Creek and Little Butte watersheds have a relative abundance of water during the winter but little precipitation during the growing season. Issues facing the watersheds include:

- Water Losses: Irrigation districts and farmers are experiencing increasingly high water losses due to inefficient and aging agricultural infrastructure. In part this is also a feature of the natural environment. The volcanic origins and basalt geology of the area leads to high rates of seepage. For example, the USBR estimates a loss rate of up to 10 cfs through the natural rock embankment of Fish Lake. The study authors, after examining winter flow records, found that this rate may be as high as three times this at full pool.

- Water Scarcity: Both Bear and Little Butte Creek are over-appropriated. A 1993 study concluded that the Bear Creek basin needed an additional 50,000 acre-feet of water to meet agricultural water rights and demands in a drought year (Dittmer, 1993). This over-appropriation continues to threaten the reliability of the irrigation water supply for the Medford, Talent, and Rogue River Valley Irrigation Districts. The nearby Klamath River basin is a reminder of how over-appropriation amongst competing uses can severely affect irrigators, municipal, industrial, and environmental water uses.

- Aquatic Habitat: Degraded water quality and water quantity conditions are not ideal for anadromous salmonids. Further, the use of Bear Creek for irrigation conveyance and canal-stream interactions with valley tributaries alter the natural hydrologic flows in ways contrary to salmon life cycle needs.

- Water Quality: With the exception of the City of Ashland, other municipalities share common water and wastewater treatment facilities. Temperature discharges from Medford's Regional Wastewater Reclamation Facility (RWRF) exceed the proposed state temperature standard and the Clean Water Act for the Rogue River. Degraded water quality on the Rogue River during the summer months at the Robert Duff Water Treatment Facility threatens the quality and reliability for Medford Water Commission municipal water customers.

Solutions to any one of these problems have the potential for creating conflicts between different water interest groups. Further, funding to solve any one of these problems could exceed the capacity for any single interest group. In the late 1990s, local leaders representing local, state, and federal government, utility and regulatory agencies, agriculture, business, and environmental interests developed a framework to address these water issues using a basin-wide and multistakeholder process. Initially, they conceived of a creative plan to move the points of diversion for the Rogue River Valley Irrigation District and Medford Irrigation District from South Fork Little Butte Creek to the Rogue River. Since then, this project has evolved into a visionary and multi-faceted water management program known as WISE: Water for Irrigation, Streams and Economy. The WISE project not only lays the groundwork for implementation of comprehensive watershed improvements, but also fosters ownership among the agricultural, agency, regulatory, and public communities for a holistic approach to improve resource management. Finally, the WISE project adds to the viability of agriculture in a quickly urbanizing community.

The current members or advisory agencies of the WISE project include:

- City of Medford
- Medford Water Commission (MWC)
- Jackson County
- Talent Irrigation District (TID)
- Medford Irrigation District (MID)
- Rogue River Valley Irrigation District (RRVID)
- U.S. Bureau of Reclamation (”Reclamation”)
- Oregon Water Resources Department (OWRD)
- Oregon Department of Environmental Quality (DEQ)
- Several other environmental, land owner and local stakeholders.

Stakeholders developed conceptual alternatives in the Preliminary Feasibility Study to achieve these goals, which included water reuse, irrigation system improvements, and reservoir expansion and reoperation. Table 3 summaries the alternatives considered.

Conveyance alternatives focused on piping of various canal segments to reduce conveyance losses. The option variations ranged from strategic piping of specific canals to development of a fully pressurized system. The more extensive piping options would remove interactions between the irrigation systems and the tributaries. Many of the valley tributaries to Bear Creek are currently intercepted by the main irrigation canals where the irrigation districts have water rights. Piping or siphoning past these tributaries would promote the natural flow regime. A fully pressurized piping system would also move the points of diversion from the mainstem Bear Creek to the reservoirs, further promoting the natural flow. The later also provides for energy conservation and promotes on-farm water conservation through full conversion to sprinkler systems.

The storage options examined expansion of key reservoirs and reoperation of others. Agate Reservoir currently functions as a reregulating feature for RRVID. The reservoir is typically fully drawn down by the end of the irrigation season. This reservoir might be expanded along with Howard Prairie Lake. Reoperation of the flood control rules for Fish, Fourmile, and Emigrant lakes were also examined as alternatives.

Table 3. WISE Conceptual Options for Alternatives

Option	Description
Conveyance Options	
C1. Limited piping	Piping of key irrigation canals, particularly those with high losses or interbasin canals
C2. Pipe main canals	Piping of all main irrigation canals, while maintaining existing points of diversions. Canal and tributary interactions would be removed.
C3. Fully pressurized system	Full separation of natural and irrigation conveyances achieved by moving points of diversion to reservoirs
Storage Options	
S1. Expand Agate Reservoir	Adding additional storage to Agate Reservoir
S2. Reoperation of Fish and Fourmile lakes	Adjusting flood control procedures to increase water supply carryover storage
S3. Reoperation of Emigrant Lake	Adjusting flood control procedures to increase water supply carryover storage
S4. Expand Howard Prairie Lake	Expand Howard Prairie Lake, also allowing for additional transfer of water from Klamath into the Rogue basin
Demand Option	
D1. On farm conservation	A range of measures to promote on-farm water conservation
Water Reuse Option	
RW1. WWTP reuse	Application of reclaimed water within RRVID within State of Oregon’s nonpotable use guidelines

Two additional demand and supply options were considered. Separate on-farm water conservation was included in addition to that which might occur under the conveyance options. At this conceptual level, no specific conservation programs were detailed. Instead what was felt was an achievable improvement in efficiency was considered. Water reuse from the RWRF applied to RRVID lands in the lower portion of the watershed was also an option. This would provide RRVID with an additional firm water supply, offset water quality issues with direct discharges to the Rogue River, and provide for a municipal revenue source.

OPERATIONAL MODEL

Operational modeling was conducted to evaluate the project alternatives. The MODified SIMyld model (MODSIM) software was selected as the basis of operational modeling. MODSIM is a joint project of Colorado State University and the U.S. Bureau of Reclamation, Pacific Northwest Division (Labadie and Larson, 2007). MODSIM has previously been applied to the WISE area as part of the 2003 Biological Assessment (USBR, 2003). MODSIM uses an optimization technique to allocate water considering hydrology, water rights, and reservoir operations.

The model simulates water use and flows on a monthly time step. Based on available climate data, a model period of record from 1928 to 2007 was selected. Over the model period of record there are several phases of both dry or drought conditions and wet conditions. Droughts dominated the early portion of the model period of record while wet periods generally dominate the later portion. The drought of record, in terms of duration and severity, was in the 1930s while significant floods were recorded in Bear Creek in 1955, 1964 and 1997.

The availability of water during the model period of record was compiled or reconstructed using various techniques. Stream gauge information was used when available, supplemented by climate data, snow melt, and information from other watersheds. Historical diversion or water use data is not typically available. The historic stream gauge data will, in most cases, contain the effects of past water uses. To determine the potential impacts of future system changes it is necessary to understand the impacts of this historic water use. Natural flows, flows that could have occurred at a given location if all human-related water use had not taken place, were calculated for various locations within the WISE area. By estimating the potential available water without human uses, various current and proposed uses can be modeled and compared.

Irrigation water use was estimated from evapotranspiration from crops and system efficiencies. A weighted net evapotranspiration based on a crop mix was calculated for each irrigation area. Cropping data were determined from available irrigation district information and aerial imagery. An estimate of the irrigated area associated with each canal was also identified from the above sources. The consumptive water use was calculated using the Hargreaves-Samai method (Allen et al., 2006).

The Bureau of Reclamation estimated seepage for the study area canals through calibration of the water supply accounting model (USBR, 2003). The estimate provided an average annual seepage loss for each major canal. These losses were distributed along canal reaches using geologic information (Golder, 2005). While this was a conceptual estimate of seepage, this method is considered adequate for the purpose of comparing the variable project elements on a relative basis. Several interbasin canals were estimated to have relatively high seepage loss rates (greater than 30% of the flow) while one main canal was assumed to lose over 50% of the flow.

Canal seepage creates a recharge mound in the local ground water aquifer. While this recharge may support phreatophytes and be a source of subsequent groundwater pumping, it is assumed here to eventually return as base flow to Bear Creek. As a result, canal seepage may support instream flow needs. Also, from an irrigation perspective, canal seepage may not be entirely “lost” from the irrigation system as there is some potential for recapture of seepage return flows in downstream diversions on Bear Creek. There is a time delay between when the flow is lost from the canal and when it may return to the creek. The Glover-Balmer method was used to estimate the pattern of the return flow.

The overall efficiency of the irrigation system was also assessed as possible using Reclamation and irrigation district flow records. Main canal diversions were assumed to be able to divert a maximum of 80% of river flows. The combined seepage losses from the main canals ranged from 15% to 34%. On-farm irrigation efficiency was estimated based on irrigation system types.

Alternative Evaluation

The operational model was used to evaluate 19 alternatives formed from the conveyance, storage, demand, and reuse options. The model estimated the reduction of irrigation shortages during drought years, improved reservoir carryover storage, and development of a favorable hydrologic regime for salmon lifecycle needs. Additional analysis outside of the model examined environmental and water supply goals for each alternative, which included:

- Water Supply Reliability: Improve water supply reliability for the irrigation districts and for native anadromous salmonids.
- Irrigation System Efficiency: Improve efficiency of irrigation deliveries and estimate possible pressures in a piped system.
- Effluent Reuse: Minimize cost and maximize reliability of the reuse of the RWRF effluent for agricultural irrigation.
- Environmental: Minimize negative environmental impacts.
- Water Quality: Improve water quality for native anadromous salmonids at the Robert Duff Water Treatment Facility intake and irrigation districts' diversion points.
- Cost Allocation: Allow a fair distribution of cost (capital, operational, and maintenance) among water users such that no stakeholder shoulders an unfair financial burden.
- Aesthetics: Improve aesthetic value of the reservoirs, streams, and rivers.
- Institutional: Minimize the magnitude and difficulty of required institutional changes such as local/regional governmental and stakeholder reorganization, transfer of authority, or creation of new institutional entities.

- Legal/Regulatory: Minimize legal and regulatory obstacles while maximizing the ability to meet local and regional goals.
- Recreation: Improve recreational values of the reservoirs, streams, and rivers.
- Financial: Minimize cumulative construction, operation and maintenance cost, and maximize the economic benefits of the water.
- Technical: Must be technically implementable.

Figure 2 shows an example of one aspect of the operational model output. In this Figure, alternative comparisons for reduction in irrigation system shortages relative to a no action condition for a severe drought year are shown. Figure 3 illustrates several alternative flow traces for a river location.

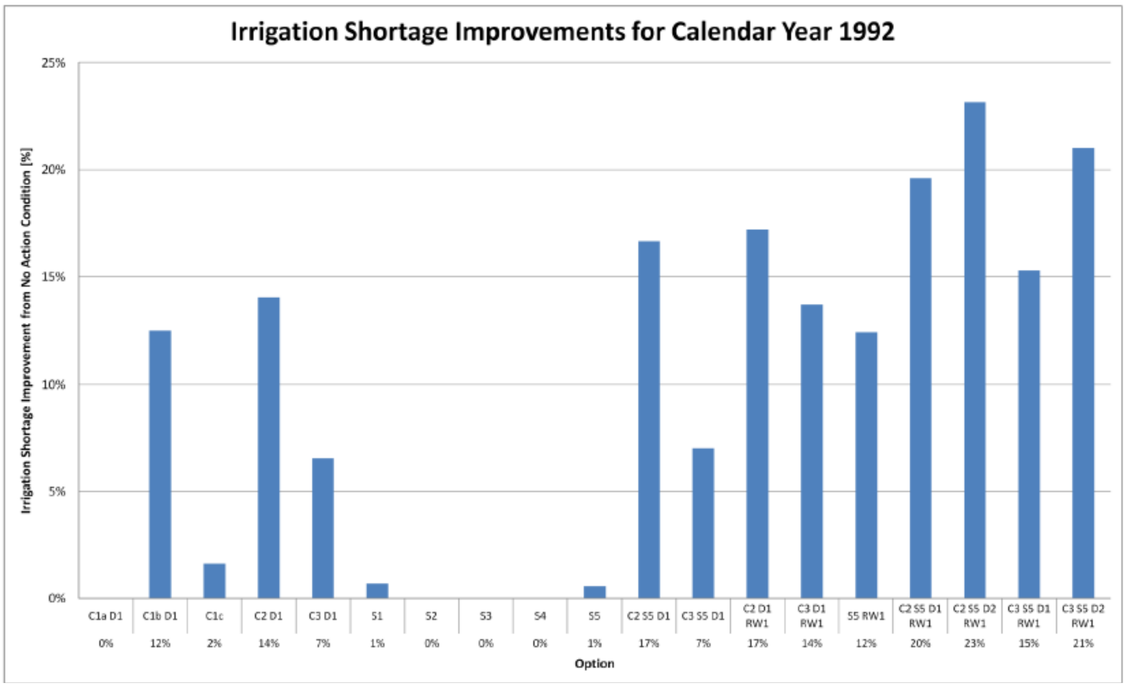

Figure 2. Modeled Irrigation Shortage Improvements in a Severe Drought Year

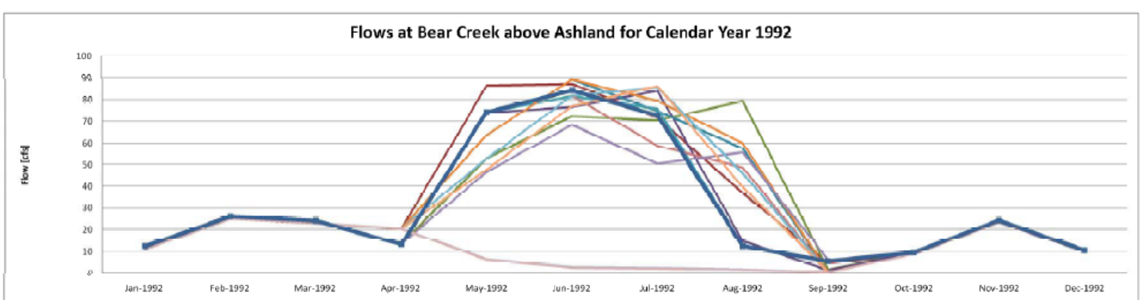

Figure 3. Envelope of Modeled Flows at Bear Creek above Ashland in a Severe Drought Year for Conceptual Scenarios

Possible Climate Change Impacts

For the purposes of the Preliminary Feasibility Study, the following question forms the basis of the operational model: If the same hydrology that historically occurred was to reoccur under current conditions of water use, how would a change to the existing irrigation system affect irrigation deliveries, instream flows, and reservoir storages? Future changes in climate may be important to water resources conditions in the basin, if the changes alter the volume or timing of available streamflow or consumptive uses.

Three possible climate change scenarios were obtained from the University of Washington's Climate Impacts Group. These scenarios are downscaled estimates of future temperature and precipitation based on global circulation models (GCM) compiled by the Intergovernmental Panel on Climate Change (IPCC). These scenarios are:

- GISS_ER B1, a low adverse climate scenario (on the basis of changes in temperature and precipitation)
- ECHAM5 SRES A2, a moderately adverse climate scenario
- IPSL_CM4 A2, a highly adverse climate scenario

These models predict an increase in summer temperatures, ranging from an increase of 1% near Fish Lake to 9% for the Medford area by the year 2100. At some locations, winter temperatures are forecast to be lower than the historic record. Winter precipitation was forecasted to be higher in the three climate change scenarios examined, whereas summer precipitation is similar to historic conditions.

The forecasted temperature and precipitation were used as inputs to estimate future water supply and demands. For water supply, a temperature-index snow accumulation and melt model was developed. This model, using the three climate change scenarios, indicates a higher snowpack than historic conditions. Higher spring and summer temperatures in the climate change scenarios may cause faster snowpack melt, although the increased snowpack is projected to persist one month longer than it has historically. The evapotranspiration and cropping model showed an average irrigation requirement increasing from 56,900 acft/year to 84,300 acft/year (ECHAM5 SRES A2 scenario).

Not all GCMs, however, reach this same conclusion. A report produced by the Climate Leadership Initiative (CLI) for the Rogue River Basin (University of Oregon, et al., 2008), examined GCMs that predict near normal precipitation coupled with higher temperatures. The CLI models have generally lower or no increases in water supply.

CONCLUSIONS AND NEXT STEPS

The Preliminary Feasibility Study focused on water supply reliability, environmental impacts and cost to screen the project elements. The alternatives and options were evaluated to carry forward into a future detailed studies and Environmental Impact Statements. Based on modeling and other analyses, further studies are recommended to:

- Retain Conveyance Option C2 (main canal piping) for alternatives development. In terms of phasing, piping the areas where there is limited potential for recapture of seepage downstream is most effective (in this case, the RRVID, MID, and transbasin canals).
- Retain Conveyance Option C3 (fully pressurized system) for alternatives development. The Option C3 irrigation benefits are less than those from C2; however, there are other considerations such as desirability of maintaining a pressurized supply that C3 provides. From a water supply perspective, the difference between options C2 and C3 is that C3 has one less source of supply. By removing connections to Bear Creek the potential to capture tributary flows and return flows upstream is removed.
- Retain Storage Option S1 (Agate Reservoir storage increase) for alternatives development. The estimated hydrology on Dry and Antelope Creek supports expanding Agate storage. As this reservoir storage is typically exhausted at the end of each season, an expanded storage would have use in meeting irrigation needs. This appears to be one of the more cost-effective options (at $33.7 million), despite having less absolute benefits to improving water supply reliability.
- Eliminate Storage Option S2, S3, and S4 operational changes (flood control operations) to reservoirs from further consideration. The options are cost-effective and likely have the least environmental issues. However, these options appear to have limited benefit for water supply reliability while increasing “risk/liability” during floods. Removing surcharge limits only has benefits in a small number of years when the reservoir did not fill to capacity but could have if the limits were reduced or removed.
- Eliminate Storage Option S5 (expand Howard Prairie Lake) due to insufficient water rights to fully fill the lake.
- Retain Option RW1 (water reuse and evaluated in 7 of the 19 alternatives) to include reclaimed water for alternatives development. From the perspective of reduced overall shortages, the reclaimed component has merit. By introducing this source to senior natural flow right holders on the Hopkins canal this provides greater opportunity for junior right holders in TID. This also encourages carry over storage capacity in Emigrant Lake. This option also appears to be one of the more cost-effective (at $71 million), despite facing more substantial technical and regulatory issues than piping.
- Microhydropower opportunities exist. There appears to be some microhydropower potential in Option C2 (partially piped system) at Cascade below Fourmile Reservoir, below Howard Prairie Reservoir, Bradshaw Drop, and below Emigrant Reservoir. Additionally, for option C3 (a fully closed, pressurized system below Agate and Emigrant reservoirs) pressure in the main delivery pipelines was estimated from 50 psi to 100 psi using the InfoWater software. These opportunities were only partially explored in the Preliminary Feasibility Study.

Evaluation of the recommended alternatives will require additional engineering feasibility based on a more developed engineering pre-design. In addition, the water quality benefits and impacts need to be evaluated, as well as specific water rights planning for each alternative (in particular how conserved water will be allocated for instream or other environmental benefit). Finally, climate change impacts need to be evaluated in detail for each alternative. The operational model developed for this preliminary feasibility study can be modified to evaluate more specific water rights, climate change and water quality issues for each alternative.

REFERENCES

Allen, Richard, et. al. 2006. *FAO Irrigation and Drainage Paper No. 56: Crop Evapotranspiration,* Food and Agricultural Organization: Rome, Italy.

Dittmer, Eric. 1993. Agricultural Water Use Inventory for the Bear Creek Basin. In *2050 Regional Water Resources Plan*.

Doppelt, Bob, Roger Hamilton,Cindy Deacon Williams, and Marni Koopman. 2008. *Preparing for Climate Change in the Rogue River Basin of Southwest Oregon*. Institute for Sustainable Environment University of Oregon: Eugene, OR.

Golder Associates. 2005. *Report on WISE Project Subtask 4.6 Groundwater Investigations*.

Labadie, John and Roger Larson. 2007. *MODSIM 8.1: River Basin Management Decision Support System User Manual and Documentation*. Colorado State University: Fort Collins, CO.

U.S. Bureau of Reclamation. 2003. "Biological Assessment of Continued Operation and Maintenance of the Rogue River Basin Project and Effects on Essential Fish Habitat under the Magnuson-Stevents Act". Pacific Northwest Regional Office: Boise, Idaho.

U.S. Bureau of Reclamation. 2009. *Biological Assessment on the Future Operation and Maintenance of the Rogue River Basin Project*. Pacific Northwest Regional Office: Boise, Idaho,

Vinsonhaler, Larry. 2002. *Rogue River Basin Project Talent Division - Oregon: Facilities and Operations*. U.S. Bureau of Reclamation: Boise Idaho.

AN ASSESSMENT OF PECOS RIVER FLOW AND WATER QUALITY BETWEEN SANTA ROSA AND PUERTO DE LUNA, NEW MEXICO

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ABSTRACT

In 2007 and 2008, the New Mexico Interstate Stream Commission performed a flow and water quality study on the Pecos River between the City of Santa Rosa and the U.S. Geological Survey's near Puerto de Luna gage. This study was one component of a larger, multi-agency effort to investigate the feasibility of reintroducing the federally threatened Pecos bluntnose shiner to the studied reach. The primary purpose of this study was to determine the extent and rate of perennial flow in the reach, and a secondary purpose was to evaluate whether the reach's water quality is suitable for shiner reintroduction. River flows were uninterrupted during the study period, fed by one or more perennial tributaries and numerous continuously flowing springs in the reach. Additionally, water quality parameters were within acceptable ranges for the fish. On the basis of water availability and water quality, the Pecos River reach between the City of Santa Rosa and Puerto de Luna may contain habitat suitable for Pecos bluntnose shiner reintroduction. These results are an important component of evaluating potential management alternatives for the Pecos bluntnose shiner.

INTRODUCTION

The New Mexico Interstate Stream Commission (NMISC) is a statutory agency of the State of New Mexico charged with protecting, conserving, and developing the waters and stream systems of the state. The NMISC Pecos River Bureau assists with Pecos River water management in accordance with the Pecos River Compact (1948), the U.S. Supreme Court Amended Decree (1988), and the Pecos River Settlement Agreement (2003). The NMISC is involved in this study because the agency is committed to investigating a variety of potential management alternatives to provide water for New Mexico irrigators and threatened fish species and to comply with Compact, Amended Decree, and Settlement requirements. One such management alternative may be to have an additional population of Pecos bluntnose shiner in the study reach. Prior to this study, however, the flow and water quality conditions in the reach were not well known to the NMISC. Therefore, the acceptability of the reach for possible shiner re-introduction was not known.

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BACKGROUND

In 1987, the U.S. Fish & Wildlife Service (Service) listed the Pecos bluntnose shiner (*Notropis simus pecosensis*) as threatened under the Endangered Species Act (1973). Being listed as "threatened" indicates that the fish is likely to become "endangered" in the future throughout all or a significant portion of its range. When a species is listed as "endangered", it means the species is in danger of extinction in the future throughout all or a significant portion of its range. The Service designated two reaches of the Pecos River as critical habitat for the fish. The Service defines "critical habitat" as a specific geographic area that contains features essential for the conservation of a threatened or endangered species and that may require special management and protection. The upper critical habitat reach comprises 64 river miles from approximately the Pecos River's confluence with Taiban Creek south of Fort Sumner to Crockett Draw north of Roswell. The lower critical habitat reach encompasses 37 river miles between the towns of Hagerman and Artesia. Whereas the upper critical habitat is prone to drying because of a wide sandy channel, the lower critical habitat receives groundwater discharge from the Roswell Artesian Basin aquifer and flows perennially. Both of the reaches designated as critical habitat for the threatened Pecos bluntnose shiner are located downstream of the study area.

The Service issued a "jeopardy opinion" regarding the Pecos bluntnose shiner, which means that a Federal action is likely to jeopardize the continued existence of a listed species or result in the destruction or adverse modification of critical habitat. As a result of the jeopardy opinion, the U.S. Bureau of Reclamation (Bureau) and the NMISC jointly completed the Carlsbad Project Water Operations and Water Supply Conservation Environmental Impact Statement (Carlsbad Ops EIS) in 2006 (U.S. Bureau of Reclamation and New Mexico Interstate Stream Commission, 2006). The Carlsbad Ops EIS investigated a series of alternatives for re-operating Sumner Dam near Ft. Sumner, New Mexico, for the benefit of the threatened fish. The Service issued a Biological Opinion covering the period from 2006 through 2016 requiring the entire length of the upper critical habitat remain wet at all times (U.S. Fish and Wildlife Service, 2006). More specifically, the Bureau must provide a flow of no less than 35 cubic feet per second (cfs) at all times at the U.S. Geological Survey (USGS) Pecos River below Taiban Creek gage, and the USGS Pecos River near Acme gage may not go dry at any time.

The motivation for this study was to investigate the possibility of establishing a redundant population as a potential step towards eventual recovery and delisting of the species. According to the Service (U.S. Fish and Wildlife Service, 2006), the shiner once inhabited the study reach, but are no longer present there. The shiner may have been removed from the study reach when they were considered an undesirable species.

From a hydrologic perspective, the primary criteria for reintroduction of the Pecos bluntnose shiner are: 1) adequate perennial flow in the river and 2) acceptable water quality for the fish. Other important factors include: 3) sufficient quality habitat and 4) acceptable levels of predation. These other factors were not investigated in this portion of the study, but instead were handled by other investigators.

METHODS

Over the approximately 32-mile reach of the Pecos River between the City of Santa Rosa and Puerto de Luna, the NMISC selected ten locations for flow measurement and water quality measurements. Because much of the Pecos River in this reach flows over private property, access to measurement sites was limited. Therefore, the NMISC selected measurement sites accessible to the public and where private landowners granted access. The NMISC's goal of having the measurement locations spaced evenly throughout the study reach was not always achieved. The measurement locations are listed in Table 1 and shown on Figure 1. The approximate river mile of each measurement location is given in Table 1.

Table 1. Pecos River Flow and Water Quality Measurement Locations[1]

Measurement Location	Location Description	River Mile	Distance (miles)
NA	USGS Pecos River Below Santa Rosa Dam Gage (not measured in this study)	62	9.92
1	Pecos River at Santa Rosa Gage	52	1.65
2	El Rito Creek Above Confluence with Pecos River	50	4.06
3	City of Santa Rosa Waste Water Treatment Plant (The amount of water discharged from the City of Santa Rosa Waste Water Treatment Plant was not measured directly. Instead, the discharge amount was obtained via several communications with plant staff).	NA	NA
4	Pecos River Below Acequia Diversion Dam	46	4.80
5	Irrigation Ditch on Left Bank at Acequia Diversion Dam	NA	NA
6	Pecos River at Lopez Ranch Near Puerto de Luna Village	41	2.51
7	Pecos River at New Mexico Highway 91 Bridge at Puerto de Luna Village	39	2.70
8	Pecos River Below Puerto de Luna Village at New Mexico Highway 91	36	15.95
9	Pecos River Above Puerto de Luna Gage at Chavez Ranch	20	1.26
10	USGS Pecos River near Puerto de Luna Gage	19	18.99
NA	USGS Pecos River Below Sumner Dam Gage (not measured in this study)	0	0

[1]The starting point for the river mile measurements was the USGS Pecos River below Sumner Dam stream gage. This starting point was selected merely for convenience. River miles increase in the upstream direction. The "Distance" column gives the distance from the downstream gage to the upstream gage. Whereas distances given in the "River Miles" column are rounded to the nearest mile, distances given in the "Distance" column are given to the one-hundredth of a mile. NA = not applicable

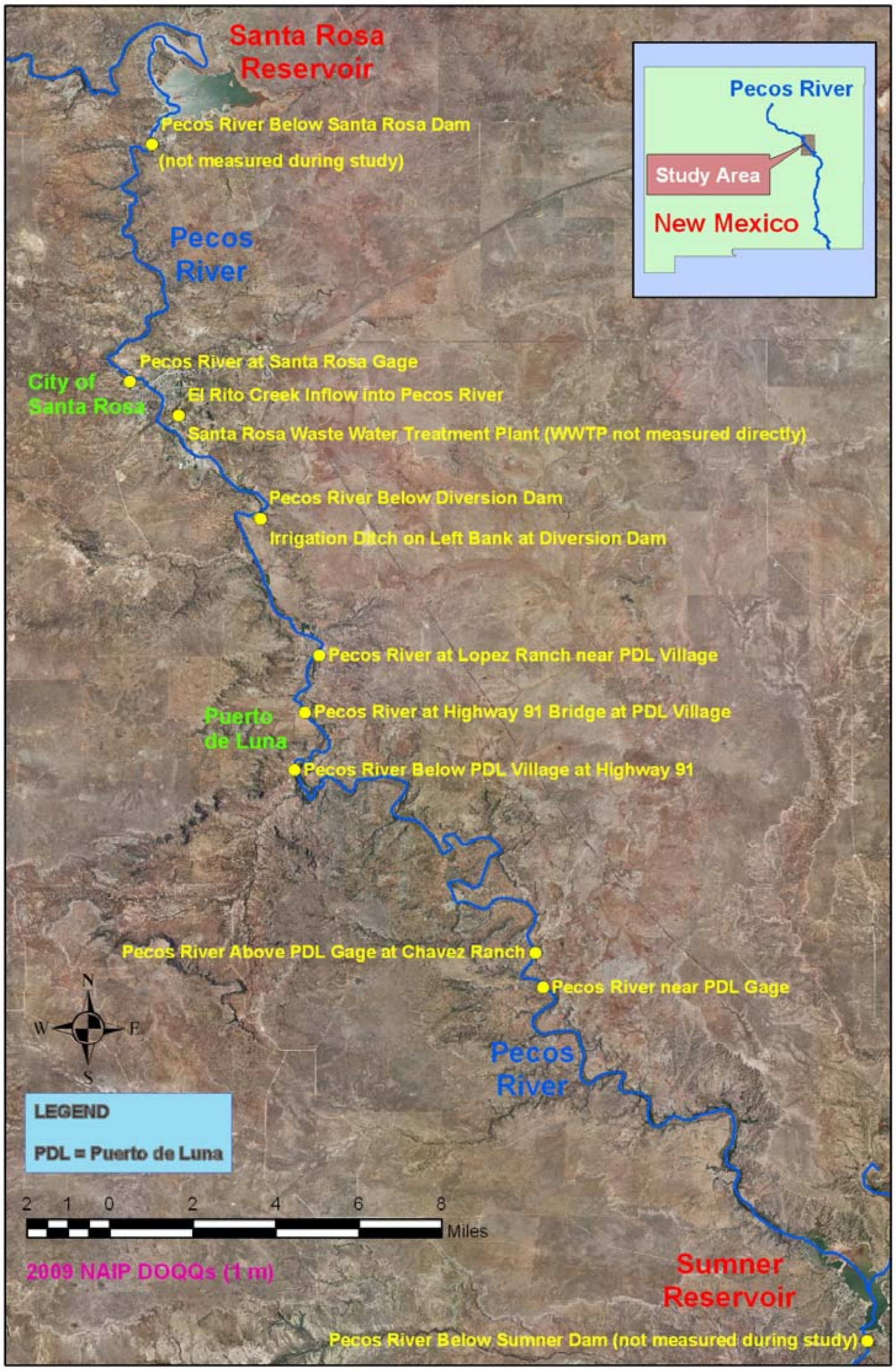

Figure 1. Location Map of the Study Area Including Measurement and Sampling Locations

In addition to the ten measurement locations selected by the NMISC, discharge information also included data from the USGS Pecos River below Santa Rosa Dam and the USGS Pecos River near Puerto de Luna stream gaging stations. To verify the USGS discharge information given at the USGS Pecos River near Puerto de Luna stream gage, the NMISC made duplicate discharge measurements at that location.

STREAM FLOW MEASUREMENT CRITERIA AND METHODOLOGY

To increase the likelihood of measuring just the river base flow, NMISC's contract stream gagers performed the water flow measurements at times when the following criteria were met: 1) at least several weeks had passed following any releases of water from Santa Rosa Reservoir for agricultural deliveries (known as "block releases") or other purposes (e.g., for dam maintenance or flood control); 2) steady flow in the river for at least several days as measured at the USGS near Puerto de Luna Gage; and 3) no precipitation had occurred for at least several days near the City of Santa Rosa or near Puerto de Luna. The stream gagers measured flow in June 2007, October 2007, January 2008, and May 2008. The stream gagers performed the discharge measurements using standard U.S. Geological Survey wading methods (Rantz et al., 1982). The stream gagers used Price AA and pygmy-type handheld meters.

WATER QUALITY MEASUREMENT METHODS

NMISC personnel performed the water quality measurements in the field using a hand-held meter. Water quality measurements were collected in January and May 2008 at the same locations and at the same times as the discharge measurements. In January 2008, NMISC used a Horiba model UX22 meter. In May 2008, NMISC used a YSI, Inc. model 556 meter. Prior to use, NMISC personnel field-calibrated each meter in accordance with the manufacturer's instructions. Measured water quality parameters included pH, conductivity, dissolved oxygen, water temperature, and salinity.

DISCHARGE RESULTS

The discharge results are given in Table 2. Whereas the June 2007 measurements were completed in one long field day, the October 2007, January 2008, and May 2008 were conducted over two shorter field days. The last measurement location completed on the first day was re-measured on the second day. The results show the river consistently gains water in the downstream direction. The source of the water appears to be El Rito Creek and numerous perennial springs in and south of the City of Santa Rosa. The results suggest there are no large seasonal differences in flow over the study's time frame. In most cases, the NMISC-measured flow values obtained in this study are in good agreement with the USGS results at the Pecos River near Puerto de Luna gage.

Ungaged inflows into the system from baseflow and side inflow can be inferred from the increase in flows in the river in reaches where no tributaries or springs have been identified explicitly.

Table 2. Pecos River Discharge Results

Measurement Location	Flow 6/13/07 (cfs)	Flow 10/11/07 (cfs)	Flow 10/12/07 (cfs)	Flow 1/23/08 (cfs)	Flow 1/24/08 (cfs)	Flow 5/6/08 (cfs)	Flow 5/7/08 (cfs)
USGS Pecos River Below Santa Rosa Dam Gage	0.00	0.00	-	0.00	-	0.00	-
Pecos River at Santa Rosa Gage	4.87	4.71	-	4.69	-	5.78	-
El Rito Creek Above Confluence with Pecos River (side inflow)	23.8	23.3	-	24.7	-	22.9	-
City of Santa Rosa Waste Water Treatment Plant [Flow reported by plant staff] (side inflow)	0.31	0.46	-	0.47	-	0.59	-
SUM of FLOW Below City of SANTA ROSA	**29.0**	**28.5**	**-**	**29.9**	**-**	**29.3**	**-**
Pecos River Below Acequia Diversion Dam	53.5	56.2	-	64.2	-	67.7	-
Irrigation Ditch on Left Bank at Acequia Diversion Dam	13.7	10.5	-	0	-	3.85	-
SUM of FLOW Below ACEQUIA DIVERSION DAM	**67.2**	**66.7**	**-**	**64.2**	**-**	**71.55**	**-**
Pecos River at Lopez Ranch near PDL Village	63.6	69.2	-	74.1	-	65.0	-
Pecos River at New Mexico Highway 91 Bridge at PDL Village	66.3	70.7	-	70.8	80.0	77.0	-
Pecos River Below PDL Village at New Mexico Highway 91	67.3	73.2	-	-	78.9	75.8	68.1
Pecos River Above PDL Gage at Chavez Ranch	63.6	71.3	68.9	-	80.0	-	70.8
NMISC Verification Measurement at Pecos River near PDL Gage	70.7	-	69.5	-	80.0	-	68.3
USGS PR near PDL Gage	71	-	70	-	79	-	77
USGS PR Below Sumner Dam Gage	-	-	-	-	-	-	-

cfs = cubic feet per second; PR = Pecos River; PDL = Puerto de Luna; “-“ = Not Measured or Not Applicable

WATER QUALITY RESULTS

January 2008 water quality results are presented in Table 3. May 2008 water quality results are reported in Table 4.

Table 3. January 2008 Water Quality Results (1/23/2008 and 1/24/2008)

Measurement Location	**pH** (SU)		**COND**[1] (S/m)		**DO** (mg/L)		**Water Temp** (deg C)		**Salinity** (%)	
	1/23	1/24	1/23	1/24	1/23	1/24	1/23	1/24	1/23	1/24
USGS Pecos River Below Santa Rosa Dam Gage	-	-	-	-	-	-	-	-	-	-
Pecos River at Santa Rosa Gage	8.24	-	0.288	-	10.65	-	9.2	-	0.1	-
El Rito Creek Above Confluence with Pecos River (side inflow)	7.98	-	0.292	-	8.65	-	15.0	-	0.1	-
City of Santa Rosa Waste Water Treatment Plant [Flow reported by plant staff] (side inflow)	-	-	-	-	-	-	-	-	-	-
Pecos River Below Acequia Diversion Dam	8.48	-	0.310	-	11.19	-	12.8	-	0.2	-
Irrigation Ditch on Left Bank at Acequia Diversion Dam	-	-	-	-	-	-	-	-	-	-
Pecos River at Lopez Ranch near PDL Village	8.31	-	0.313	-	11.08	-	11.5	-	0.2	-
Pecos River at New Mexico Highway 91 Bridge at PDL Village	8.45	8.28	0.314	0.321	10.91	10.18	10.9	6.6	0.2	0.2
Pecos River Below PDL Village at New Mexico Highway 91	-	8.42	-	0.320	-	10.56	-	6.3	-	0.2
Pecos River Above PDL Gage at Chavez Ranch	-	8.48	-	0.323	-	10.88	-	4.9	-	0.2
NMISC Verification Measurement at Pecos River near PDL Gage	-	8.51	-	0.325	-	10.97	-	4.4	-	0.2
USGS PR near PDL Gage	-	-	-	-	-	-	-	-	-	-
USGS PR Below Sumner Dam Gage	-	-	-	-	-	-	-	-	-	-

[1] Corrected to 25 degrees Celsius

PR = Pecos River; PDL = Puerto de Luna; “-“ = Not Measured or Not Applicable; SU = Standard Units

Table 4. May 2008 Water Quality Results (5/6/2008 and 5/7/2008)

Measurement Location	**pH** (SU)		**COND**[1] (S/m)		**DO** (mg/L)		**Water Temp** (deg C)		**Salinity** (%)	
	5/6	5/7	5/6	5/7	5/6	5/7	5/6	5/7	5/6	5/7
USGS Pecos River Below Santa Rosa Dam Gage	-	-	-	-	-	-	-	-	-	-
Pecos River at Santa Rosa Gage	7.58	-	0.2426	-	7.23	-	18.27	-	0.145	-
El Rito Creek Above Confluence with Pecos River (side inflow)	7.17	-	0.2425	-	3.26	-	17.76	-	0.147	-
City of Santa Rosa Waste Water Treatment Plant [Flow reported by plant staff] (side inflow)	-	-	-	-	-	-	-	-	-	-
Pecos River Below Acequia Diversion Dam	7.82	-	0.2630	-	8.33	-	18.93	-	0.156	-
Irrigation Ditch on Left Bank at Acequia Diversion Dam	7.85	-	0.2685	-	9.52	-	19.86	-	0.156	-
Pecos River at Lopez Ranch near PDL Village	7.94	-	0.2831	-	8.14	-	22.37	-	0.155	-
Pecos River at New Mexico Highway 91 Bridge at PDL Village	7.96	-	0.2882	-	7.88	-	22.92	-	0.156	-
Pecos River Below PDL Village at New Mexico Highway 91	7.97	7.89	0.2905	0.2496	7.81	7.97	23.42	16.08	0.156	0.158
Pecos River Above PDL Gage at Chavez Ranch	-	7.99	-	0.2218	-	8.39	-	16.93	-	0.136
NMISC Verification Measurement at Pecos River near PDL Gage	-	7.97	-	0.2504	-	8.27	-	18.66	-	0.149
USGS PR near PDL Gage	-	-	-	-	-	-	-	-	-	-
USGS PR Below Sumner Dam Gage	-	-	-	-	-	-	-	-	-	-

[1] Corrected to 25 degrees Celsius

PR = Pecos River; PDL = Puerto de Luna; “-“ = Not Measured or Not Applicable; SU = Standard Units

CONCLUSIONS

Study results indicated that flows in this reach of the Pecos River were continuous during the period investigated. Discharge increased consistently in the downstream direction as at least one perennial tributary and numerous springs contributed water to the river.

Measured flows ranged from about 5 cfs at the Pecos River at Santa Rosa Gage to as much as 80 cfs at the Pecos River near Puerto de Luna gage. Measured water quality parameters, which included dissolved oxygen, pH, water temperature, salinity, and conductivity, were within satisfactory ranges for the fish. Water managers will be able to use the study results to assist in evaluating scenarios involving the possible reintroduction of the Pecos bluntnose shiner to the Pecos River reach between the City of Santa Rosa and Puerto de Luna.

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REFERENCES

Endangered Species Act of 1973 (16 U.S.C. 1531-1544, 87 Stat. 884), as amended -- Public Law 93-205.

Pecos River Compact, 1948.

Pecos River Settlement Agreement, 2003.

Rantz, S.E. et al., 1982. Measurement and Computation of Streamflow, Volumes 1 and 2, US Geological Survey Water-Supply Paper 2175, Washington, D.C.

U.S. Bureau of Reclamation and New Mexico Interstate Stream Commission, 2006. Carlsbad Project Water Operations and Water Supply Conservation Final Environmental Impact Statement.

U.S. Fish and Wildlife Service, 2006. Biological Opinion for Bureau of Reclamation’s Proposed Carlsbad Project Water Operations and Water Supply Conservation 2006-2016.

U.S. Supreme Court Amended Decree, 1988. Texas v. New Mexico, 485 U.S. 388 (1988).

IRRIGATION MANAGEMENT STRATEGIES TO FOSTER CONSERVATION OF ENDANGERED NATIVE SPECIES

David E. Cowley, Ph.D.[1]

ABSTRACT

This paper describes biodiversity and habitat conditions of irrigation canals and presents ideas for ways that irrigation management might be carried out to benefit native species. Its objective is to promote discussion amongst irrigation managers and water users on the general topic of how to sustain agricultural uses of water while simultaneously sustaining native species. In the Middle Rio Grande (MRG) of New Mexico, the endangered Rio Grande silvery minnow occurs along with other native and nonnative fishes in conveyance, drainage, and return canals. Conveyance return canals have been found to have a role in supporting biodiversity of fishes in the MRG. In contrast, drainage canals are a high-risk environment for small-bodied native fishes. Invasive species in MRG drain canals including parrotfeather and virile and rusty crayfishes, appear to enable establishment of two nonnative predators, largemouth bass and channel catfish, which pose risks to native species of concern. A number of strategies might provide positive benefits to native species conservation while providing water to agriculture. These include alternatives such as development of refugial habitats that could remain wetted during periods of high demand but low water supply, use of coarse screens to control movement of large-bodied nonnative predators while allowing smaller native fishes to pass through the canal system, and using irrigation diversion dams for experimental flooding of lateral habitats upstream of the dams to encourage successful spawning of native species like the Rio Grande silvery minnow. Other alternatives could likely be identified.

INTRODUCTION

Rivers in arid regions expand with precipitation runoff and contract with drought in long-term natural cycles (Lake 2000). The fishes and other biota endemic to these rivers adapt and flourish under these predictably variable flow regimes (Lytle and Poff 2004) and they seem especially adept at seeking refuge from floods and droughts (Magoulick and Kobza 2003; Dodds et al. 2004). River systems globally have been transformed by the development of irrigation systems that couple advancement of human social and cultural institutions with decline of river ecosystems and the biota those systems support (Cowley 2006; Sallenave et al. 2010). An important challenge is to balance the use of highly variable water supplies so that competing demands from cities, farmers, and ecological systems can all be sustained (Sallenave and Cowley 2004).

The Rio Grande of New Mexico (Figure 1) is one example of a highly altered river ecosystem that has over a thousand year history of human settlements and it has had irrigation networks for nearly 500 years (reviewed by Cowley 2006). In spite of the long

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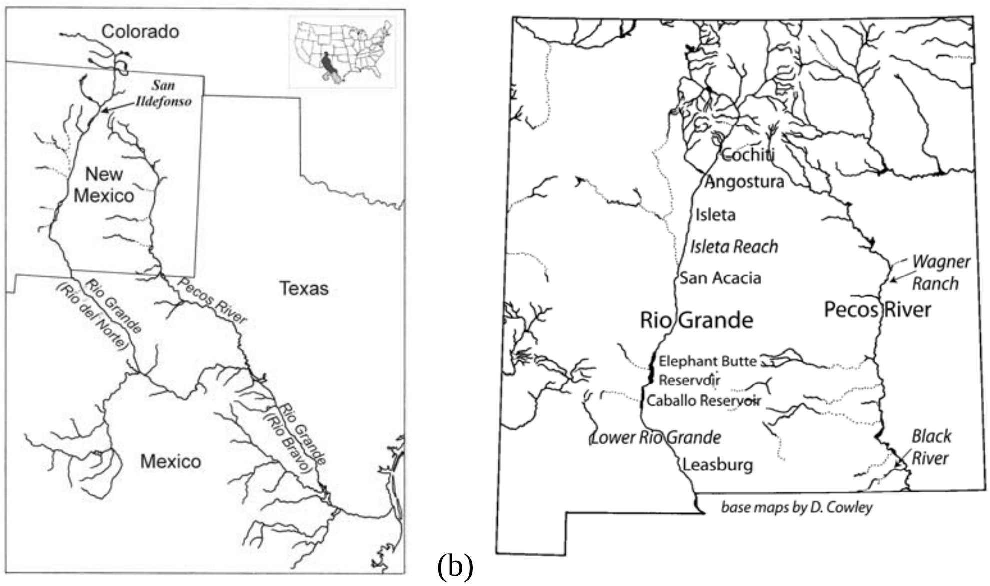
history of farming in the Rio Grande basin of New Mexico, most of the native[2] fishes were still present about 1880.



(a) (b)

Figure 1. The Rio Grande (a) gains flows from Mexico and the United States. This paper focuses on the middle Rio Grande and Pecos Rivers of New Mexico (b) and ways that irrigation might be managed to benefit native species. The middle Rio Grande of New Mexico extends from Cochiti to the headwaters of Elephant Butte Reservoir. The Isleta Reach of the middle Rio Grande lies between Isleta and San Acacia diversion dams. Not shown are the Albuquerque Reach, which is between Angostura and Isleta, and the Cochiti Reach between Cochiti and Angostura. The lower Rio Grande of New Mexico is downstream of Caballo Reservoir.

In the last quarter of the 19th century irrigation diversions in the upper basin in Colorado began to deplete flows, sometimes drying the river from Albuquerque, New Mexico Territory to beyond El Paso, Texas (U.S. Senate 1898). Actions by the U.S. and Mexican governments initiated an the era of dam-building to increase water storage and expand irrigation infrastructure.

With the accumulation of water delivery projects that disrupted natural river flows and sediment retention dams that trapped nutrients supporting the food web, the native fishes of the Rio Grande began a long decline. Today about half of the 27 native fish species are either extinct or eliminated from the river (Figure 2).

[2] A native species is one that is endemic to an ecosystem and that was present in that ecosystem long before human influences began to alter it. Nowadays native species can occur in natural and human-created habitats. An extant native species is one still occupying its historical area whereas extirpated indicates local elimination from a particular area.

Concurrent with the decline of native fishes was a steady increase of nonnative[3] fishes (Figure 2) so that today, the biodiversity of fishes in the Rio Grande of New Mexico includes 45 species of which 31 are introduced (Cowley 2006). The contemporary biodiversity of fishes in the Rio Grande is 1.7 times richer than it was historically. Some might think that the ecosystem is in better condition than ever given the abundance of fish species in the river basin. Unfortunately most of the nonnative fish species in the middle Rio Grande are predatory game fishes introduced into reservoirs for sport fishing. Spread of these species from reservoirs via canals and river channels can pose significant hazards to smaller sized native species and to smaller age classes of larger sized native species.

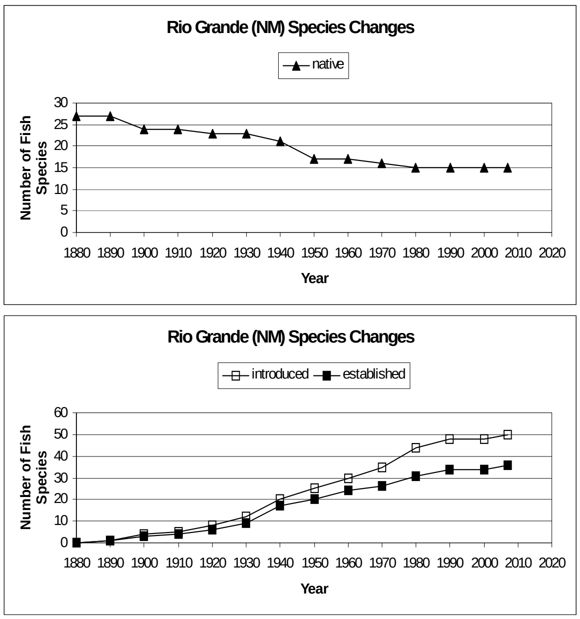


Figure 2. The number of extant native fish species (upper) has declined episodically with the accumulation of water projects on the Rio Grande. Cumulative nonnative species introductions and establishments are shown by decade (lower). At present rates, 1 or 2 new fish species could be introduced by 2020. Establishment rates over the past 3 decades have been about 70%.

Water management in the Rio Grande (Figure 1) is complicated by an endangered species, the Rio Grande silvery minnow (*Hybognathus amarus*). This species once occupied the river from near San Ildefonso, New Mexico, where it occurred alongside Rio Grande cutthroat trout (*Oncorhynchus clarkii virginalis*) (Cope and Yarrow 1875), to Brownsville, Texas (Girard 1856). Today the species is only found in the Middle Rio Grande (MRG) of New Mexico, a region between Cochiti Dam and the headwaters of Elephant Butte Reservoir (Figure 1).

The Rio Grande silvery minnow is a resilient, strong-swimming species that historically ate algae and organic matter from mud substrates in the Rio Grande that were nutrient rich and deficient in dissolved oxygen (Shirey 2004; Cowley et al. 2006). A comparison of diatoms[4] foraged by Rio Grande silvery minnows in 1874 and 1978 (Shirey et al. 2008) showed that

[3] A nonnative species is one introduced into an ecosystem by humans.
[4] Diatoms are single-celled algae in silica cases with distinct frequency of occurrence on different substrates. A narrow range of habitat conditions occupied by some diatom species make them useful in water quality monitoring. See Shirey et al. (2008) for a full description of inferred ecological conditions of the Rio Grande as evidenced by diatoms consumed by the Rio Grande silvery minnow.

the ecological conditions of the Rio Grande were nutrient-starved in 1978. Those silvery minnows, the last record of the species from above Cochiti Dam, were eating sand and attached diatoms.

In comparison, the Rio Grande was nutrient-enriched in 1874, with silvery minnows foraging on mud substrates and consuming sediment and organic matter[5]. Present day reservoirs have reduced the biological productivity of the Rio Grande because they trap sediment and nutrients that once-supported the aquatic food web of the river (Shirey et al. 2008). The biological productivity of the Rio Grande has also declined in part because of reduced peak discharge with river flow regulation. Reduction of floodplain inundation alters sediment and nutrient exchange between the river ecosystem and its floodplain (Kingsford 2000; Hauer and Lorang 2004). Water on the floodplain stimulates high rates of biological productivity that can contribute to the river ecosystem (Molles et al. 1998).

There are additional factors beyond cultural oligotrophication[6] (Stockner et al. 2000) of habitat that endanger the Rio Grande silvery minnow (Cowley 2006). This species is like other native minnows now lost from the Rio Grande of New Mexico (Cowley et al. 2008) that utilize floodplains for spawning and nursery areas (Cowley et al. 2009). Thus another factor in the endangerment of the species is the decline of seasonal floodplain inundation that causes a loss of spawning habitat for the species.

A third factor endangering the silvery minnow is the depletion of river flows by irrigation diversions during periods of drought. When substantial sections of the river are dewatered, significant numbers of fish die. A fourth source of endangerment comes via unscreened irrigation diversions that entrain fish, fish eggs, and other nuisance aquatic organisms along with the water. The eggs of the silvery minnow and other related minnows are buoyant and nonadhesive and they are easily transported downstream by water currents (Cowley et al. 2008). Although transport of fishes through the canal system appears to be involved in the rebound of the fish assemblage following river channel drying (Cowley 2006; Cowley et al. 2007), small-bodied native fishes delivered into drain canals face a high risk of predation from nonnative predators (Muldoon 2007; Cowley et al. 2007; Sallenave et al. 2010). Finally, accumulation of salts and other contaminants with irrigation drainage poses another source of endangerment (Cowley et al. 2003, 2009).

FISHES IN CANALS[7]

Only a few publications discuss fish and other aquatic species in irrigation systems (Cowley et al. 2007). In France, Poizot et al. (1999) found high biodiversity of fishes in canals adjacent to the Rhône River. In an English agricultural landscape Williams et al.

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[5] Organic matter included detritus, pine pollen, cyanobacteria, algae, and diatoms (Cowley et al. 2006).
[6] Cultural oligotrophication is human-caused removal of nutrients from aquatic ecosystems. It has two primary causes: nutrient retention by reservoirs and nutrient reduction pursuant to the Clean Water Act.
[7] The following terms are used for canals: a *conveyance canal* delivers irrigation water from the river, a *drain canal* removes excess irrigation water, and a *return canal* delivers irrigation return flows to the river. Two types of return canals are used in this paper: *conveyance return* and *drain return*.

(2004) compared biodiversity of rivers, streams, canals and ponds and found that canals supported 36% of the regional aquatic species of invertebrates. Armitage et al. (2003) attributed the high biodiversity of canals in southern England to slow currents, canal connectivity to the River Frome, and water and vegetation management practices. Irrigation canal systems in California often have a majority of the fishes of nonnative species (May and Brown 2002; Martin and Saiki 2005).

An older study (Cowley 1979; Cowley and Sublette 1987) noted the probable influence of an irrigation canals linked to the Black River in southeastern New Mexico (Figure 1) on fish biodiversity in the river. In the vicinity of the canals there were 19 species of fish and eight of them were associated with the irrigation canals (Cowley 1979). Cowley found half of the fish species in the Willow Lake Canal were nonnative species. Sampling conducted in 1978 in the Harroun Canal east of the Pecos River and a short distance north of Black River (Figure 1) found three of eight fish species were nonnative (Cowley unpublished data). Combined across locations nearly one-half of the fish species in the canals were nonnative species (5 of 12). These nonnative species included common carp (*Cyprinus carpio*), white crappie (*Pomoxis annularis*), warmouth (*Lepomis gulosus*), channel catfish (*Ictalurus punctatus*), and walleye (*Zander vitreus*).

In the middle and lower Rio Grande of New Mexico (Figure 1) relatively high biodiversity of fishes occurs in river and canal habitats (Cowley et al. 2007; Carrasco 2010; Sallenave et al. 2010). In the middle Rio Grande all of the species found in the river occurred at least occasionally in the canal system of the Middle Rio Grande Conservancy District (MRGCD). A total of 27 fish species have been collected in the MRGCD canals, 16 nonnative species and 11 native species (Sallenave et al. 2010).

The most abundant native species in the MRGCD canals have been fathead minnow (*Pimephales promelas*), red shiner (*Cyprinella lutrensis*), and Rio Grande silvery minnow. Nonnative fishes that were especially abundant in the drain canals (Cowley et al. 2007) included channel catfish and largemouth bass (*Micropterus salmoides*), but other nonnative predators were regularly found such as walleye and crappie (*P. annularis*, *P. nigromaculatus*). Some of the nonnative aquatic species in addition to channel catfish and largemouth bass are invasive in the canals such as parrotfeather (*Myriophyllum aquaticum*), virile crayfish (*Orconectes virilis*), rusty crayfish (*O. rusticus*), common carp, white sucker (*Catostomus commersoni*), yellow bullhead (*Ameiurus natalis*), Asiatic clam (*Corbicula fluminea*), and bullfrog (*Rana americana*) (Cowley 2006).

Carrasco (2010) sampled river and canal habitats in the lower Rio Grande of New Mexico downstream of Leasburg (Figure 1). He found a total of 20 fish species, 18 from river sites, 17 from drain canals, and 13 from conveyance canals. Nine of the twenty species were native to the Rio Grande and 11 were nonnative. Fishes in drain canals were usually green sunfish (*Lepomis cyanellus*), largemouth bass, longear sunfish (*Lepomis megalotis*), mosquitofish (*Gambusia affinis*), carp, and yellow bullhead (*Ameiurus natalis*). His studies also showed that canal sites more proximal to the river had more fish species.

In a 5 km section of the Peralta Canal system and the adjacent Isleta Reach of the MRG, Cowley et al. (2007) estimated the relative contributions of fish species to the river channel from upstream river habitats as opposed to lateral irrigation canal habitats. They found that fish movements down the river channel from the Albuquerque Reach were the most important and that conveyance return was secondary but far more important than drainage return. The results suggested that nonnative predators in the drain canals may rapidly deplete small native fishes from the canals at the conclusion of the irrigation season. The results further suggested that the occurrence of small native species in the drain canals during the irrigation season was attributable to their delivery from the Peralta conveyance canal.

WHY CONSERVE NATIVE FISHES?

The Endangered Species Act extends protection to listed species. Federally-mandated recovery programs for endangered fish species can result in reduced water available for agricultural uses (Adams and Cho 1998) and in extreme cases irrigation diversions have been closed to protect endangered species (Service 2003). But focusing only on an endangered species such as the Rio Grande silvery minnow is too narrow a focus. New native species are likely to be recognized with future analyses that distinguish Rio Grande fishes from those in the Mississippi River basin. Candidates likely to be examined include river carpsucker (*Carpiodes carpio*), smallmouth buffalo (*Ictiobus bubalus*), and blue sucker (*Cycleptus elongatus*). Recognition of these as unique species would further complicate Rio Grande water management to meet the needs of cities and farmers. Emphasizing strategies to conserve all extant native fishes would enable more sustainable water management into an uncertain future.

IDEAS FOR ECOLOGICAL MANAGEMENT OF CANALS

What are the ecological conditions in irrigation canals that favor native as opposed to nonnative fishes? Understanding the ecological conditions in canals and the various conditions that favor some species over others could enable one to think of creative ways to manage conditions in the canals. For example, native fishes of the Rio Grande are adapted to turbid sediment-laden water while most nonnative predators (game fishes) need clearer water to see their prey. Also, native species in the Rio Grande seem adept at finding flow refuges at flood stage and cooler irrigation return waters at low flows. Native fishes in rivers with extreme environmental conditions typically have adaptations that help them find and survive in isolated refugial habitats, such as strong swimming ability and preferences for stronger currents and cooler waters (Labbe and Fausch 2000; Ostrand and Wilde 2001; Magoulick and Kobza 2003; Dodds et al. 2004).

Manipulation of Turbidity in Canals

The conveyance canals of the MRGCD carry sediment-turbid water from the Rio Grande. At points where this turbid water is discharged in drain canals or a conveyance return canal, native species occur in moderate abundance during the irrigation season (Cowley et al. 2007). It is thought that the turbidity of the drain canal caused by conveyance discharge provides protection to small native fishes from larger nonnative predators. Native species ought to be adapted to cope with the turbid character of the Rio Grande whereas many of the nonnative species in drain canals are predators that rely on seeing and chasing prey and are disadvantaged in turbid water (Cowley et al. 2007).

Elsewhere in MRGCD drain canals the water is clear and extensive beds of parrotfeather occur. Surveys in clear-water canals found abundant virile crayfish of many sizes along with large catfish and basses but very few smaller-bodied fishes. The nonnative predators in the clear-water drain canals appear to establish longer term locations. They are frequently collected from beneath extensive patches of parrotfeather (Muldoon 2007; Sallenave et al 2010). Thus, water clarity is one factor that might be amenable to manipulation in drain canals to benefit native fishes. Manipulation of turbidity might be another way to control parrotfeather; it does not occur in the turbid waters of the Rio Grande.

Habitat Diversity

The relatively low diversity of canal habitats means there are few places where fish can escape the current. Downstream of flow regulation structures, culverts and siphons are typically sites of higher abundance of fish species in the MRGCD canals. Manipulating habitat conditions in canals could also be accomplished in other ways. For example, in strategic locations on drain canals large wood or other obstructions could be used to provide more resting places for fish transiting the irrigation system.

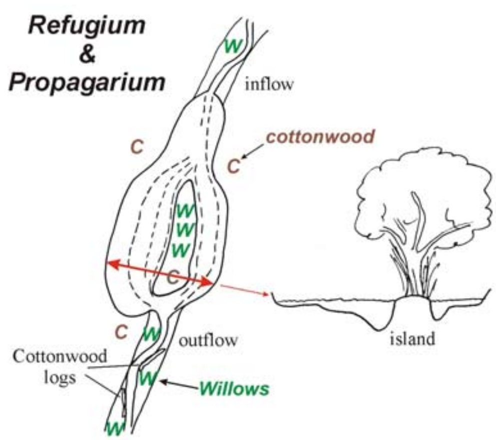

Figure 3. Conceptual drawing of a naturalized habitat on a drain canal that provides variable depths and extensive shallows for growth of algae. Such habitats might be managed to facilitate native fish conservation.

Refugial Habitats

Refugial fish habitats are one idea to promote survival of native fishes that occur in irrigation canals. Refugia on canals could address two potential problems: (1) provision of wetted habitats when the adjacent river channel is dry and (2) retention of entrained fish eggs and larvae in a favorable habitat.

Cowley (2003) proposed a beads-on-a-string concept for naturalized refugial fish habitats on drain canals (Figure 3). He envisioned a smaller-scale habitat that would mimic the character of abandoned channels of the Middle Rio Grande. Such refugia would address the two potential problems noted above. Deployment of such naturalized habitats on conveyance and drain canals could provide significant additions to native fish habitat along the Middle Rio Grande.

Developed Recreational Areas

In discussing strategies for irrigation management to benefit native species, Cowley et al. (2007) noted that coarser fish screens offer the ability to limit movement of the larger predatory fish in the irrigation system. They might be used, for instance, to concentrate game fish and make them more available to anglers. Such locations might offer opportunities to develop a fee-based recreational fishing area. There are also educational benefits that can be associated with developed nature trails that help the irrigation managers to inform the public about their ecologically-aimed management.

Floodplain Inundation

A basic conflict in management of the endangered Rio Grande silvery minnow is that the minnow is adapted to time its reproduction coincident with snow melt and its flooding of lateral habitats (Cowley et al. 2009). But nowadays in the irrigated agricultural landscape of the Middle Rio Grande flooding of lateral habitats is rare and nursery habitats for the silvery minnow are unavailable especially in drier years. Cowley et al. (2007) proposed that irrigation diversion dams might be engineered to flood lateral habitats upstream of the dams and thus mimic even in dry years a condition needed by the species for successful reproduction.

Timing of Water Deliveries

In studies of minnows on the Pecos River of New Mexico, Cowley et al. (2008) suggested that smaller scale water transfers on a frequent basis might facilitate greater recruitment of minnow species like the Rio Grande silvery minnow. These authors observed large numbers of larval minnows in shallow floodplain pools immediately following small-scale increases of discharge of about 15 cm. The apparent inducement of spawning by a slight increase in discharge suggests that numerous small releases of water might yield better recruitment than fewer large releases. With larval fishes in floodplain pools, pulses of water delivery every third or fourth day might reduce stranding of the fish in lateral nursery habitats and thus might increase recruitment.

Share Water with the Environment

A final idea is to share water with the environment during times of high water demand and low supply. Sampling in 2004 during a time much of the river channel of the Isleta Reach was dry showed a high abundance of native fishes in irrigation return canals (Cowley et al. 2007). This led to the idea of leaking water from the irrigation system to the return canal during times of river drying thus providing a refugial habitat for fishes. Endangered species recovery funding was provided to the MRGCD for a demonstration project to create "Drain Outfall Refugia." This project installed dead cottonwood tree trunks in the river banks where three return canals empty into the Rio Grande. Subsequent monitoring showed that large wood can be used to create dynamic fish habitats in association with irrigation return canals and fish sampling has demonstrated that Rio Grande silvery minnows occupy them (Wesche et al. 2010).

Screen the Irrigation Diversions

Fish surveys in the middle Rio Grande routinely show a majority of the fishes collected belong to native species (e.g. Dudley et al. 2005, 2006), which implies that the nonnative predators are disadvantaged in the turbid. The apparent disadvantage suffered by nonnative fish predators in the turbid environment of the Rio Grande suggests that simply screening the irrigation diversions would eliminate fish, fish eggs, and nuisance aquatic species from the irrigation system. Cowley et al. (2007) cautioned that adopting this strategy would also eliminate one source of contribution of fishes to the river channel after channel drying. On the other hand forcing the nonnative species to occupy less desirable habitat conditions might have long term benefits to many of the native fishes.

CLOSING REMARKS

Endangered species pit water users against environmentalists and the biologists of federal agencies mandated to preserve the species. It is unlikely in the near future that agency biologists will voluntarily seek strategies to simultaneously benefit farmers and the endangered species. In the author's opinion, strategies that minimize collateral damage to irrigated agriculture are unlikely to be implemented unless they come from irrigators offering innovative strategies to use their water in ways that provide benefits to ecological systems and to themselves. It is possible that agency biologists will oppose conservation initiatives arising from water users and patience may be required to overcome the practice that has been called "combat biology" (Service 2003). None of the ideas proposed here may be feasible, but it is hoped that by articulating them, a focused discussion might occur among water users to find ways to contribute to native species conservation while retaining use of their water.

REFERENCES

Adams, R.M. and S.H. Cho. 1998. Agriculture and endangered species: an analysis of trade-offs in the Klamath Basin. Water Resources Research 34:2741-2749.

Armitage, P.D., K. Szoszkiewicz, J.H. Blackburn, and I. Nesbitt. 2003. Ditch communities: a major contributor to floodplain biodiversity. Aquatic Conservation: Marine and Freshwater Ecosystems 13:165-185.

Carrasco, C.P. 2010. Comparisons of the fish fauna within the lower Rio Grande of New Mexico to the adjacent irrigation system and historical records of occurrence. MS Thesis, New Mexico State University, Las Cruces.

Cope, E.D. and H.C. Yarrow. 1875. Report upon the collections of fishes made in portions of Nevada, Utah, California, Colorado, New Mexico, and Arizona during the years 1871-1874, pp. 635-703 In: United States Army Engineers Department Report, in charge of George M. Wheeler, Geographer and Geologist of the Exploration and Surveys west of the 100th Meridian, 5, Chapter 6.

Cowley, D.E. 1979. Temporal and spatial distributions of fishes in the Black River, Eddy County, New Mexico. Master's Thesis, Eastern New Mexico University, Portales.

Cowley, D.E. 2003. Water requirements for endangered species - Rio Grande silvery minnow (*Hybognathus amarus*). 47th Annual New Mexico Water Conference, "There's No Doubt We're in a Drought", WRRI Report No. 326: 97-107, http://wrri.nmsu.edu/publish/watcon/proc47/cowley.pdf.

Cowley, D.E. 2006. Strategies for ecological restoration of the Middle Rio Grande in New Mexico and recovery of the endangered Rio Grande silvery minnow. Reviews in Fisheries Science 14:169-186.

Cowley, DE & JE Sublette. 1987. Distribution of fishes in the Black River, Eddy County, New Mexico. Southwestern Naturalist 32:213-221.

Cowley, D.E., P. Shirey, and C. Hohman. 2003. Agriculture Irrigation Systems and Conservation of Native Fishes: Issues in the Rio Grande Valley of New Mexico. NMSU Water Task Force Report 1:49-55, http://cahe.nmsu.edu:16080/pubs/taskforce/water/WTFRpt1.pdf.

Cowley, D.E., P.D. Shirey, and M.D. Hatch. 2006. Ecology of the Rio Grande silvery minnow (Cyprinidae: *Hybognathus amarus*) inferred from specimens collected in 1874. Reviews in Fisheries Science 14:111-125.

Cowley, D.E., R.C. Wissmar, and R. Sallenave. 2007. Fish assemblages and seasonal movements of fish in irrigation canals and river reaches of the middle Rio Grande, New Mexico (U.S.A.). Ecology of Freshwater Fish 16:548-558.

Cowley, D.E., J. Alleman, and R. Sallenave. 2008. Studies on Eggs of *Notropis simus pecosensis* (Pecos Bluntnose Shiner) and Other Minnows of Concern to Water Managers in New Mexico. Final Report, US Bureau of Reclamation Contract No. 04-CS-40-8082.

Cowley, D.E., J.C. Alleman, R. Sallenave, R.R. McShane, and P.D. Shirey. 2009. Effects of salinity on specific gravity and viability of eggs of a North American minnow (Cyprinidae). Scientia Marina 73(S1):47-58.

Dodds, W.K., K. Gido, M.R. Whiles, K.M. Fritz, and W.J. Matthews. 2004. Life on the edge: the ecology of Great Plains prairie streams. Bioscience 54:205-216.

Dudley, R.K., S.P. Platania, and S.J. Gotlieb. 2005. Summary of the Rio Grande silvery minnow population monitoring program results from 2004. U.S. Bureau of Reclamation, Final Report, Contract Number 03CR408029.

Dudley, R.K., S.P. Platania, and S.J. Gotlieb. 2006. Summary of the Rio Grande silvery minnow population monitoring program results from 2005. U.S. Bureau of Reclamation, Final Report, Contract Number 03CR408029.

Girard, C. 1856. Researches upon the cyprinid fishes inhabiting the freshwaters of the United States of America, west of the Mississippi Valley, from specimens in the museum of the Smithsonian Institution. Proceedings of the National Academy of Sciences Philadelphia 8:165-213.

Hauer, F.R. and M.S. Lorang. 2004. River regulation, decline of ecological resources, and potential for restoration in a semi-arid lands river in the western USA. Aquatic Sciences 66:388-401.

Kingsford, R.T. 2000. Ecological impacts of dams, water diversions and river management on floodplain wetlands in Australia. Austral Ecology 25:109-127.

Labbe, T.R. and K.D. Fausch. 2000. Dynamics of intermittent stream habitat structure regulate persistence of a threatened fish at multiple scales. Ecological Applications 10:1774-1791.

Lake, P.S. 2000. Disturbance, patchiness, and diversity in streams. Journal of the North American Benthological Society 19:573-592.

Lytle, D.A. and N.L. Poff. 2004. Adaptation to natural flow regimes. Trends in Ecology and Evolution 19:94-100.

Magoulick, D.D. and R.M. Kobza. 2003. The role of refugia for fishes during drought: a review and synthesis. Freshwater Biology 48:1186-1198.

Martin, B.A. and M.K. Saiki. 2005. Relation of desert pupfish abundance to selected environmental variables in natural and manmade habitats of the Salton Sea basin. Environmental Biology of Fishes 73:97-107.

May, J.T. and L.R. Brown. 2002. Fish communities of the Sacramento River Basin: implications for conservation of natives in the Central Valley, California. Environmental Biology of Fishes 63:373-388.

Molles, M.C., C.S. Crawford, L.M. Ellis, H.M. Valett and C.N. Dahm. 1998. Managed flooding for riparian ecosystem restoration. Bioscience 48:749-756.

Muldoon, A. 2007. Site fidelity and habitat use of a nonnative predatory fish in a manmade canal in New Mexico. MS Thesis, New Mexico State University, Las Cruces.

Ostrand, K.G. and G.R. Wilde. 2001. Temperature, dissolved oxygen, and salinity tolerances of five prairie stream fishes and their role in explaining fish assemblage patterns. Transactions of the American Fisheries Society 130:742-749.

Poizot, G., P. Chauvelon, E. Rosecchi and A.J. Crivelli. 1999. Passage of Rhône River fish through irrigation pumps in the Camargue: first results. Bulletin Francais de la Peche et de la Pisciculture 1999(352):31-43.

Sallenave, R. and D.E. Cowley. 2004. Aquatic resources in arid lands: issues and opportunities. Aquatic Sciences 66(4):343-345.

Sallenave, R., C.P. Carrasco, and D.E. Cowley. 2010. Fishes in the middle and lower Rio Grande irrigation systems of New Mexico. New Mexico Agricultural Extension Service, Circular 653.

Service, R.F. 2003. Combat biology on the Klamath. Science 300:36-39.

Shirey, P.D. 2004. Foraging habits and habitat utilization of Rio Grande silvery minnow (Hybognathus amarus) as inferred by diatom frustules. MS Thesis, New Mexico State University, Las Cruces.

Shirey, P.D., D.E. Cowley and R. Sallenave. 2008. Diatoms from gut contents of museum specimens of an endangered minnow suggest long-term ecological changes in the Rio Grande (USA). Journal of Paleolimnology 40:263-272.

Stockner, J.G., E. Rydin, and P. Hyenstrand. 2000. Cultural oligotrophication: causes and consequences for fisheries resources. Fisheries 25(5):7-14.

U.S. Senate. 1898. Equitable distribution of the waters of the Rio Grande. Senate Document 229, 55th Congress, 2nd Session.

Wesche, T.A., L.B. Wesche, S. Broderick, D.E. Cowley, and B. Wyman. 2010. Development of Rio Grande silvery minnow refugia at drain outfalls in the Isleta Reach of the middle Rio Grande, New Mexico. Project Completion Report No. 05-FG-40-2436, Middle Rio Grande ESA Collaborative Program, US Bureau of Reclamation, Albuquerque.

Williams, P., M. Whitfield, J. Biggs, S. Bray, G. Fox, P. Nicolet, and D. Sear. 2004. Comparative biodiversity of rivers, streams, ditches and ponds in an agricultural landscape in Southern England. Biological Conservation 115:329-341.

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LOW-HEAD HYDRO, AS EASY AS 1-2-3

Ed Gerak, P.E[1]

ABSTRACT

Hydroelectricity isn't as unattainable as it used to be. A combination of technological improvements and political openness has made low-head hydro a potential reality.

This paper will introduce the process of low-head hydro development, and provide a framework to analyze each potential location for installation.

INTRODUCTION AND BACKGROUND

Buckeye Water Conservation and Drainage District (BWCDD) is an irrigation district that operates as a municipal corporation in the state of Arizona. The District provides water and power to approximately 22,000 acres. The District's mission is to provide reasonably priced water to the agricultural and urban lands in the District.

BWCDD has an allocation of Hoover power, which aids in keeping our water and electrical costs affordable. This allocation expires in 2017, and legislation is currently ongoing to renew the contracts. However, the District initiated a pilot project in an attempt to replace the Hoover power if it did not get reallocated.

When the Buckeye Irrigation Canal was surveyed in 1909, potential locations for hydro sites were identified. Due to the drop sizes, it would not have been practical to develop a hydro unit until recently. With improvements in low-head technology, we decided to investigate if this was now feasible.

We contacted K.R. Saline, our electrical engineering contractor, to inquire if they had any experience with low-head hydro units. They connected us with Natel, the manufacturer of the SLH (Schneider Linear Hydro) engine. We were impressed by the technology and decided to partner with the inventor on the first commercial installation of the SLH.

PROCESS

The process for evaluating whether low-head hydro is right for you is a lot easier than you might think. Once you know the variables of each potential location (head, flow rate, utilization), you can determine your capacity and power generation. Once these are determined, a more thorough analysis of installation costs needs to be performed.

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Capacity

Whether you have a 5, 50, or 500 foot drop, the time might be right to evaluate the hydro potential in your system. The power capacity for hydro is a simple calculation ($P = \rho hrgk$).

- P is power in watts,
- ρ is the density of water (~1000 kg/m^3)
- h is height in meters,
- r is flow rate in cubic meters per second,
- g is acceleration due to gravity of 9.8 m/s^2,
- k is a coefficient of efficiency ranging from 0 to 1 (0.85 is a good efficiency to use)

If you know your flow and your head differential, you can get a good estimate on your true capacity.

Kilowatt Hours

Once you have your capacity, you need to consider the capacity utilization. If the location is only utilized 50% of the time, this will seriously affect your power generation. A five year flow log should be sufficient to estimate the capacity utilization for the site selected.

Once you have calculated your capacity and percent utilization, you can find your total kWhr's produced. Revenue generated by the unit will be dependent on your retail or wholesale sales of those kWhr's but also could include renewable energy credits.

Certain set-ups, like pump-storage are designed to take advantage of peaking power. Peaking power is advantageous to large utility providers because it provides the opportunity to meet increased demand without the capital costs of a large base load plant. If you are planning on selling this power to a utility, you really should discuss this issue with them prior to moving forward.

If you are going to use the energy produced from the investment, you should evaluate your current electrical rates versus the cost to operations, maintenance and power generation from the unit. An initial number of $0.02 per kWhr will be sufficient for O&M costs calculations.

Once you understand the economics of the revenue side of the calculation, it is time to work on the cost to construction.

Installation Costs

The cost for installation is a little more complicated than the revenue calculation simply because of the variables involved in each individual site. The FERC application should be fairly standardized, with a small conduit exemption as the best option if you can qualify.

Civil costs will vary from site to site. Some sites will lend themselves nicely to a hydro installation with minor modifications. This will result in reduced engineering and construction costs. Other sites may make it cost prohibitive to install a unit. It all depends on what you are trying to accomplish.

For our unit, we knew that safety/security was an issue. We chose to house the unit in a concrete vault to protect it from vandals. Our intent was to have everything housed inside the vault for security purposes. Unfortunately, we did not have sufficient clearance on the panels to meet code, so we had to move the electrical boxes to the top of the vault. Security fencing and lights were added as a secondary precaution.

Site location can have significant impact on construction costs. Remote locations will result in increased costs for staging, crew housing, construction, transportation, etc. If you have to run significant power lines to the site, this may also kill the economics, because once you are generating power, you have to send it somewhere.

Unit costs will be a function of technology and capacity, with some possible variability depending on head size.

Financing

Once you have the total capital cost for construction, you can start preparing your financial model. There is currently a 30% Investor Tax Credit, and a 5 year accelerated depreciations schedule. Since the future cost of fuel should remain at zero, excluding minor maintenance costs, you have everything you need for calculating a return on your investment.

ENERGY PRODUCTION WITHIN AN IRRIGATION SYSTEM

Joe E. Blankenship[1]

ABSTRACT

There is a science to the design of irrigation systems that permits the most efficient movement of water from its source to its use. The design of the system may also offer opportunities to recover much of the energy used to get water into the system at the check structures and drops that are used to dissipate energy from accelerating water flow or to accommodate changes in terrain level.

Most turbine technology is inefficient at recovering energy from low head drops (five feet to fifteen feet) and generally requires a substantial investment in machinery and civil structures to be installed. This has made economic consideration of energy recovery in low head, slow flow environments impractical until now.

NatEl Energy has developed and installed a technological innovation called the "SLH" in an irrigation canal in Buckeye, AZ. This engine was designed specifically for low head applications like irrigation canals. The technology has been tested in hydraulic laboratories and undergone field trials in an irrigation canal as well as stream diversions. The first commercial installation incorporates improvements in design and materials of construction gained from the hydraulic testing and field trials.

The NatEl package includes the engine, inlet throttle, penstock, drafttube, generator and PLC control system, making it as close to "plug-and-play" as practical. Incorporating the SLH into existing check structures may be possible with a minimum of civil engineering and construction activity. The levelized cost of electricity from a SLH system will typically be lower than generation from any other renewable resource.

INITIATING ENERGY PRODUCTION IN A CANAL SETTING

In early 2007 the managers of the Buckeye Water Conservation and Drainage District (BWCDD) saw that controlling electricity costs was going to become a bigger issue in the Districts operating budget. The General Manager of the District, Ed Gerak, began to research the alternatives for generation at the three existing drops on the BWCDD canals. In discussion with the District's electrical consultant, K. R. Saline and Associates, which had long been retained to advise on the District's Hoover Dam power allocation, the consultant provided several alternative methods of generation, including the SLH. After discussions with the management of NatEl, Mr. Gerak and his Board of Directors approved a joint project by BWCDD and NatEl to construct a demonstration project for a nominal 20 kW capacity SLH engine at a drop site on the South Extension of the main canal.

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Mr. Gerak crafted a partnership of site owner, machinery supplier, and civil designer and electrical consultant to design, build and operate a pilot operation on at the South Extension drop. The District would provide the site and modifications of the drop to accommodate the SLH engine: NatEl would contribute the engine as a demonstration of its low cost, low impact, low head hydro generation capability; Stantec, Inc., wanting to be part of the development of a unique green technology would contribute the civil design and K. R. Saline and Associates would contribute the permitting and electrical connection consulting, as well as the Federal Energy Regulatory Commission ("FERC") request for exemption from licensing. The pilot would allow a demonstration of technology that could be implemented at other sites in the District that may have the potential to generate between 200 – 300 kW of additional capacity for the District.

By the end January of 2009 the engine has been designed, manufactured and assembled. The site civil design has been completed and the FERC request for exemption has been filed. At each stage of the process, the partners have learned how to deal with the technology and the regulations for creating a methodology to provide a low cost option for electrical generation in low head environments.

The FERC exemption was received in November of 2009; the engine was installed in December and interconnection with the grid was finalized in April of 1010.

Beyond Buckeye, Natel is targeting applications of the SLH in irrigation canals and water supply conduits. Smaller machines may be economical in wastewater treatment plant outfalls. While this market is being cultivated, NatEl will begin to work with developers that wish to add generation to the approximately 75,000 non-powered existing dams in the U. S.

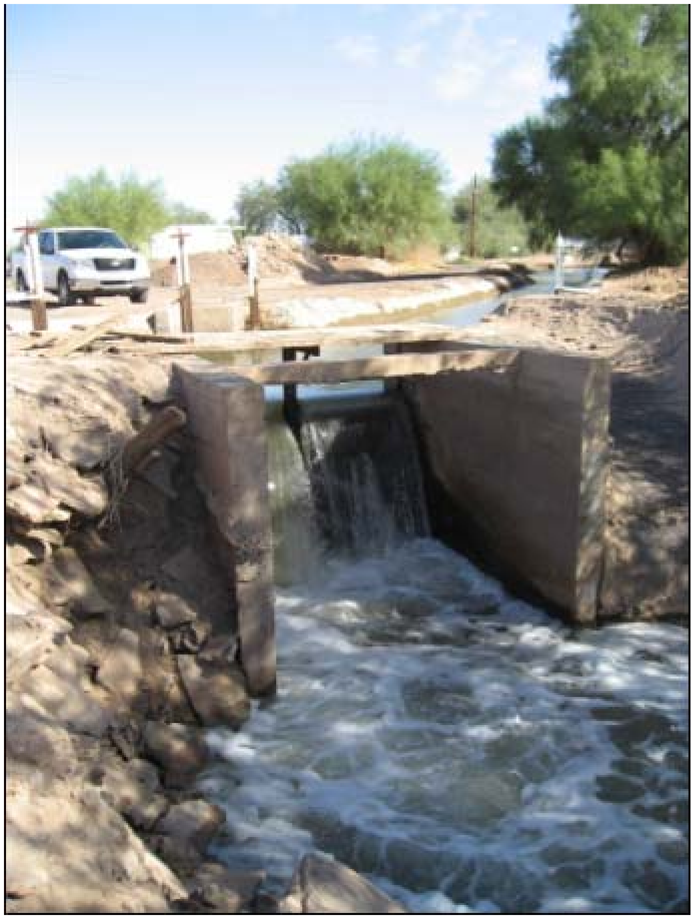

Figure 1. South Extension canal drop before the SLH

DEMONSTRATING THE HARD SCIENCE — MACHINERY DESIGN AND ELECTRICAL GENERATION

The design of the SLH engine, as well as its materials of construction, methods of manufacture and assembly are a matter of established engineering and design. What is unique is the design of the foils and proprietary direction of flow through the engine. Updated designs provide for efficiency of operation as well as durability and reliability. The design of the BWCDD demonstration unit is such that it is scalable from 20 kW to over 1,000 kW of nominal capacity. The cost estimates from the current design effort for the SLH system, which includes the engine, generator, inlet gates, penstock, draft tube and PLC (essentially a system ready for installation), are indicated at a capacity cost of between $1,000 and $1,500/kW. The generator is off the shelf, and the other parts lend themselves to stamping, bending and simple milling that does not require expensive multi-axis CNC machines. The PLC is a special design that will meet SCADA requirements and can be adopted for automated and remote operation as well as control of multiple units in series.

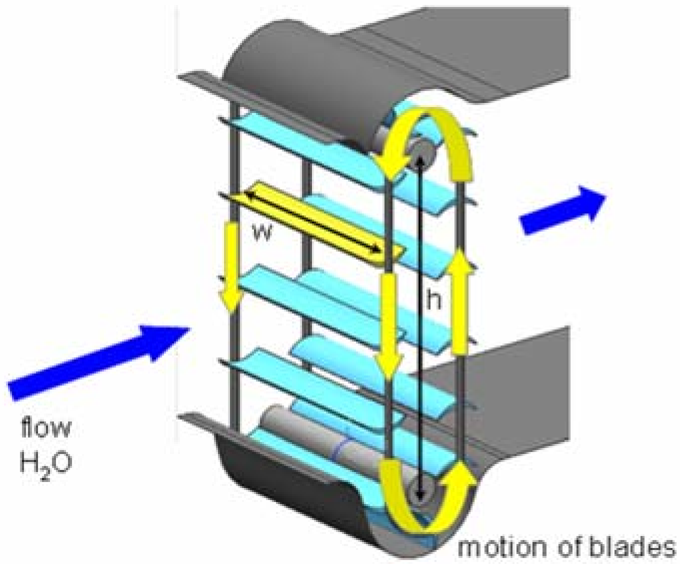

Figure 2. Working Configuration of the SLH machine

The SLH operates in a significantly different manner than a rotary turbine. Water impacts a series of foils that are linked by chain or belt. The foils travel in a linear direction up and down and over the bottom and top shafts. The upper shaft is connected to a speed increaser and generator, providing the electrical output. The significant difference between this design and a rotary turbine is that the SLH can handle large volumes of slow moving water and convert the kinetic energy to electricity with efficiencies of over 80% across a broad range of heads and flows.

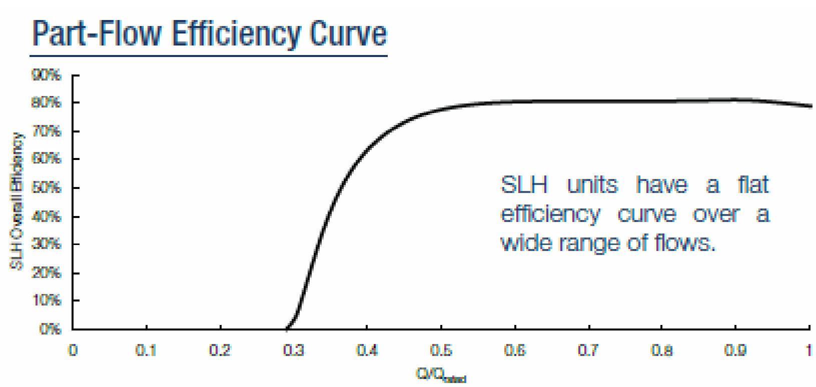

Figure 3. Efficiency curve of the SLH engine

ECONOMIC, REGULATORY AND ENVIRONMENTAL ASPECTS OF THE SLH INSTALLATION

Economics of SLH in Low Head Hydro

Favorable economic results generally are when there is a net monetary income. For the regulatory community, favorable economics means all rules are complied with. And for the environment, a project must look at life cycle effects to be sure that air, water, soil, plants and animals are not adversely impacted, in addition to considering human safety factors, productive land use and the recycling of all construction materials.

In the U. S. there has been very little development in low head hydro over the past 50 years. Some of this may be attributed to the social forces that have put hydro development in the environmentally unfriendly category, but a great deal has to do with the fact that using a standard turbine technology is too expensive to design, build and install. Generally, an installation of a turbine meant design for a specific application and then manufacturing one unit on a multi-axis CNC machine. The civil works had to be designed to carry the heavy loads of the machinery as well as the constant force of falling water.

In the initial phases of installation, the SLH will not escape some unfavorable attitudes held about hydro generation. Attitudes will change only after favorable environmental benefits are demonstrated. As to the economic feasibility, design and cost estimates have confirmed a realistic opportunity to again look at low head hydro as a means of meeting the renewable energy needs of the nation. Since the engine can be produced by standard stamping, forming and machining methods and the engine housing, penstock and draft tube are fabricated of heavy steel, the cost of capacity can be competitive with coal fired plants and nearly as competitive as combined cycle gas turbines. With low capital cost and renewable flowing water providing low or no cost fuel, the overall cost of electricity can be very competitive.

To determine the cost competitiveness for the SLH, data requirements are the system head, flow and duration of the flow. A review of the record of water flows over a drop for one or two years will provide sufficient data to calculate a duration curve. With this data, along with efficiency of conversion, the calculation of the annual amount of electricity generated can be made. Revenue is determined by the kWh production and the feed in tariff at the utility.

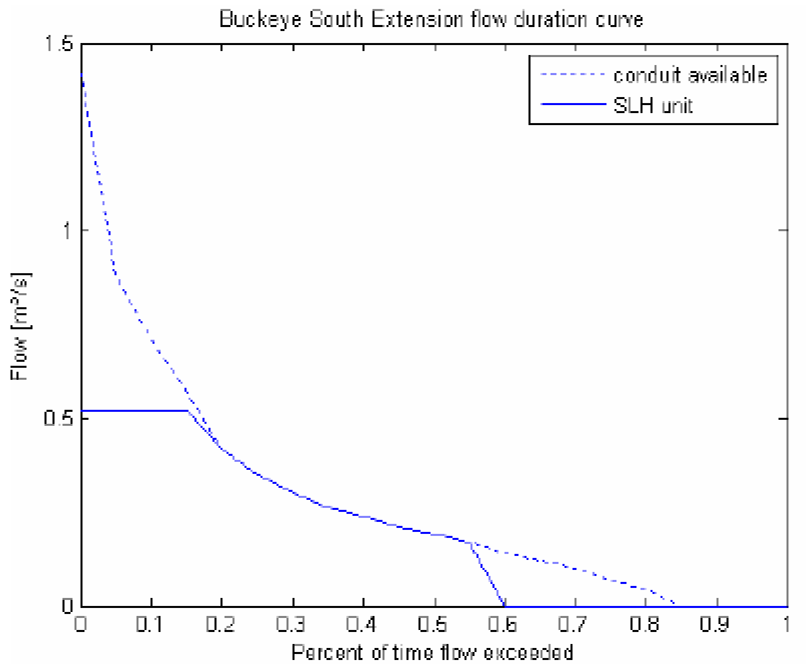

Figure 4. Flow Duration Curve for South Extension Drop

The flow duration curve (Fig. 4) provides the basis for a pro forma operating statement for the demonstration unit at BWCDD. The engine design is for 20 kW of capacity at 4 m of head and flow of 0.52 m^3/s. The actual drop is 2.74 m and average flow is 0.29 m^3/s. With the duration curve providing time and flow the calculation of capacity utilization of the Buckeye pilot is approximately 25%. Under these conditions the projected production is 38,000 kWh/yr against a design capacity of 158,000 kWh/yr based on a 90% availability.

There are several things that could change the actual economic outcome of the BWCDD installation. The District has the opportunity to lower the level of the downstream pool to make the elevation change larger. Another change would be to alter the schedule of water directed through the drop to have a longer period of flow through the SLH. Either of these would impact the actual results to make the installation more favorable than in the forecast.

Economic considerations for SLH sizes above 20 kW are more favorable. A scaling study has provided system cost estimates for all sizes up to 1000 kW. The lowest cost per kW for the machinery is estimated to be in the 500 kW – 1,000 kW range. Adding in civil design, construction and permitting the all-in estimates for a 200 kW capacity installation is likely to range from $2,500 - $3,500 per kW of capacity. Operation and maintenance cost is estimated to be approximately $0.02 kW/h. The biggest variable will be the amount of capacity utilization experienced. NatEl estimates of lifecycle cost per kWh based on a 20 year life, 8% cost of capital and $0.02 O&M is shown in Fig. 5.

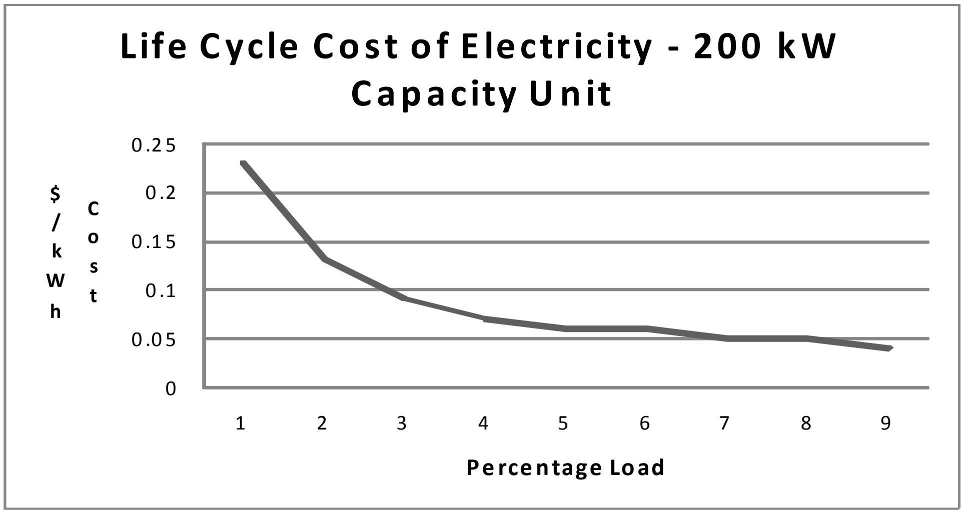

Figure 5. Estimated SLH kWh lifecycle cost based on percentage load

Beyond the price-cost relationship cost of electricity, the economic benefits are likely to be enhanced by the incentives that continue to develop around production of renewable energy. For small hydro, the Federal Tax Code allows taxable entities to take an Investment Tax Credit ("ITC") of 30%, or alternatively, an approximate $0.015 kW/hr production tax credit ("PTC") for ten years. For irrigation districts these incentives will generally not be available, but there may be ways to monetize the PTC for a portion of the cost of an installation. More readily monetized are the Renewable Energy Credits ("RECs") generally bought by utilities to meet Renewable Energy Standards ("RES"). These REC's will become more valuable if a cap-and-trade program for carbon offsets is enacted. A cap-and-trade system has been instituted in California and is indicated to be an integral part of the Western Climate Initiative of seven western states. Under the most favorable circumstances, low head hydro may provide between two and ten cents ($0.02 - $0.10) per kWh in RECs over the coming years.

Regulatory Considerations

Regulations tend to reflect the social considerations in the community in which we live. Regulation of hydro electric generation reflects society's attitudes about the environmental effects of generation using high dams that have caused river obstructions to fish passage or riparian ecological impacts. These concerns are reflected in the factors required for an application for the FERC exemption for a low head hydro exemption. An Environmental Impact Statement is not required by statute for a low head hydro exemption. However, FERC does require an Environmental Assessment and notification of potentially impacted agencies and organizations of the intent to build a facility in a waterway, even a conduit such as an irrigation canal or aqueduct.

This number of parties to be notified in a FERC exemption request illustrates the lengths to which regulations allows participation in the approval process. The process also can provide potential delays and alterations as comments and/or objections come from any of the notified parties. In addition to the U. S. Fish and Wildlife, State Game and Fish departments and state permitting agencies, archeological discoveries and historical agencies may have effect the schedule. Consideration for Native American lands must also be taken into account. A requirement by FERC is a GIS map of the site, with ownership of attached parcels identified to reflect neighborhood impacts. Under normal circumstances, the cost of the preparing the FERC exemption request as well as the time required for FERC approval and post approval conditions could make a facility with low capacity utilization such as the one at Buckeye too expensive for a reasonable economic return.

Regulation of projects to prevent environmentally damaging events is a concept we at Natel approve of. What is required is for the process to work efficiently and timely to realize the full benefits of a project's possibilities. For the small hydro construction process, obtaining the FERC exemption is **THE** critical path element in going from conception to operation.

Before January of 2009 there were only ten exemptions for low head hydro generation issued nationally over the past four years[2.] However, two Conduit exemptions were issued in January of 2009[3]. One of the 2009 issued projects took over nine months from application to granting of an exemption. The second took five and a half months which is what is expected if there are no protests or motions to intervene. In addition to the processing time, a condition of approval is filing of final construction drawings 60 days prior to beginning construction.

The preparation, processing and post approval conditions of the FERC exemption can take several times longer than the design, installation and commissioning of a project, particularly if the machine is already manufactured. In economic terms this could mean a delayed revenue by several months from a project.

Environmental Considerations

The SLH has been designed to mitigate several potentially harmful effects to the environment. For irrigation canals, since no additional dams or impoundments are to be constructed, there may be very little environmental impact from installing a SLH. There are typically no Fish and Wildlife considerations and endangered species concerns should have been cleared in the construction of the canal. The biggest environmental advantage is the positive benefit to be gained by using existing infrastructure of irrigation canals and non-generating low head dams to offset many of the negative impacts from coal and natural gas fired electrical generators. These benefits come about while recovering energy that is currently being wasted.

As legislation for national renewable energy standards are debated and regional cap-and-trade programs are enacted, the drive for carbon dioxide reduction will become more intense. The ability to accomplish a part of the CO_2 reduction objective by using existing infrastructure, and at the same time derive significant economic benefit will become more appealing. In an attempt to quantify the potential for reducing CO_2 emissions for the in-

[2] www.FERC.gov/industries/hydropower.asp

conduit market of irrigation districts and water supply we examined the carbon dioxide emissions per MWh of a local utility:

> Average CO_2 emissions from existing coal fired units - 0.98 metric tons/MWh
> Average CO_2 emissions from existing gas CC – units – 0.42 metric tons/MWh
> Average CO_2 emissions from existing gas CT units - 0.61 metric tons/MWh[3]

At NatEl, we believe the potential capacity of low head hydro installations in irrigation and aqueducts in the western states regulated by the Bureau of Reclamation to be approximately 4,000 MW. At 50% capacity utilization the annual carbon dioxide reduction may potentially be around 17,520,000 metric tons of CO_2 per year if only coal fired plants are considered. With an average CO_2 emission of Combined Cycle and Combustion Turbine units of 0.50 metric tons/MWh of carbon dioxide emissions, the potential for carbon dioxide reduction may be one-half of coal, or 8,760,000 metric tons of CO_2 per year.

The design considerations for the machinery and surrounding housings, penstock and draft tube encompass a "cradle to cradle" philosophy - make everything recyclable. Of the parts and pieces in the SLH system, we estimate that 98% of the materials of construction can be recycled. Of the cement and mechanics of water control in the surrounding housing and structures, that may be true as well.

LARGE SCALE BENEFITS, SMALL SCALE IMPACTS

One principal attraction for BWCDD in partnering with NatEl for a SLH demonstration plant installation was making available a technology that can provide economic generation in several more drops in its canal system, thus offsetting its electrical costs by as much as one third. Another attraction was the District's engrained pioneering vision for adoption of this technology worldwide in a system that could bring environmentally friendly electricity generation too many underdeveloped parts of the world. The technology provides a ready alternative to high dam construction that has so many detrimental environmental effects wherever they are installed. The litany of complaints about hydro power using high dams and impoundments are many: Flooding of human and fauna habitat; uprooting families and destroying farm land and grazing areas; impeding fish passage for spawning and migration; forever altering canyon and valley ecology and geographic attractions, as well as others. From its design inception, NatEl has incorporated physics and aquatic physiology criteria to achieve many of the power generation attributes of high dams with a minimum of environmental disturbances and impacts. Through a method called Linear Reservoir Routing ("LRR"), studies indicate that placement of strategic small dams along a long river path can provide up to 80% of the power of a high dam while flooding as little as 5% of the land required by installation of a large dam.

This conclusion has been developed after studies of a dam already installed as well as with a proposed installation. A study at the University of North Texas compared the cost

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[3] Arizona Public Service; Resource Plan Report; January 29, 2009; p.34.

and effects of a high dam built in Nepal with the estimated economic, ecological and social costs if a LRR system of stair-step dams had been constructed.[4] The study of the dam in Nepal concluded that the return on investment in economic measures could have possibly been several times that provided by the actual installed conventional high head structure, and the social, ecological and societal benefits would have been dramatically different based on lower human displacement and sustaining fishing and farming that had occurred for centuries[5].

A controversial river valley program being considered in the 1970s was in the St. John River Basin of Maine. The plan as proposed would build two high dams; Dickey Dam at about 90 meters of head and the Lincoln School Dam at about 30 m of head. From the two dams, 88,240 acres of wilderness, agricultural and habituated land would be flooded for power generation. The installed capacity of these two dams would have been 830 MW. Dr. Daniel Schneider and Emory Damstrom presented a paper at the Waterpower '79 International Conference on Small Scale Hydropower that illustrated a prospective series of eight dams each having a head of 5 to 8 m. Pumped storage reservoirs were added to provide peaking capability and control flooding. This proposal would have flooded approximately 4,500 acres, or 5% of the high dam amount and could produce 80% of the power stipulated in the high dam approach[5]. The dams were not constructed and the area was converted to a national wilderness area.
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To obtain high dam benefits with low dam designs requires a programmatic demonstration of the SLH attributes of efficiency, durability, reliability, fish passage, balance of system cost and cost of manufacture and installation. The demonstration site at BWCDD is a small step in the program of demonstration, scaling and implementation of larger size systems.

DEMONSTRATION OF THE INSTALLATION BENEFITS TO BWCDD

The data necessary to calculate SLH efficiency in the production of electricity had been gathered in laboratory and pilot plants previously installed. The objectives for the installation at BWCDD of a demonstration of the SLH technology was to provide data on reliability and durability for design components and use machine engineering data of the 20 kW engine to scale the system to larger sizes. Efficiency data is now being collected as the machine production has exceeded 1,000 kWh. By providing access to its site at the South Extension, BWCDD will end up with ownership of the generating plant as well as demonstrated capability for installation of several additional sites.

When all of the remaining installations are made at BWCDD, the District may offset up to one-third of its electrical costs into the indefinite future. This becomes a permanent

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[4] Nieswiadomy, Dr. Michael; Wang, Hana; "The Benefits of Sustainable Hydropower Using Low-Head Dams in Stair-Step Series"; University of North Texas; Department of Economics; July 17, 2008.

[5] Schneider, Daniel J,; Damstrom, Emory K.; "The Schneider Engine: Performance and Application For Hydropower"; Waterpower '79; October 1-3, 1979.

hedge of electrical costs for that portion of its operating expense. In the District's pioneering tradition it is using its own resources to provide a long term contribution to systems that support an expanded, sustainable future.

REFERENCES

Economic Development Fact Book; Economic Development Department; Town of Buckeye, AZ, 2008.

Nieswiadomy, Dr. Michael; Wang, Hana; "The Benefits of Sustainable Hydropower Using Low-Head Dams in Stair-Step Series"; University of North Texas; Department of Economics; Paper Presented at Western Economic Association International meeting; June 29-July 3, 2008. .

Parkman, I. H.; History of Buckeye Canal; 100 Year Commemorative of the Buckeye Irrigation Company; 1957

Schneider, Daniel J,; Damstrom, Emory K.; "The Schneider Engine: Performance and Application for Hydropower"; Waterpower '79; October 1-3, 1979.

EXAMINING THE FEASIBILITY OF HYDROPOWER GENERATION IN THE MIDDLE RIO GRANDE CONSERVANCY DISTRICT

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ABSTRACT

Irrigation districts in the American West and throughout the world have extensive canal networks that could be utilized to generate a significant amount of hydropower. Small hydropower systems were once commonplace and the technology was well known throughout the world. In the 20th century smaller systems tended to be abandoned in favor of fossil fuel, and hydropower development focused on large hydraulic systems featuring high head, large rate of flow, and massive purpose built dams and reservoirs. The economy of energy production favored larger scale systems, while small scale systems tended to be ignored. As the world attempts to become less dependent on carbon based energy resources, small scale hydropower is once again an attractive potential resource for new energy development.

The Middle Rio Grande Conservancy District (MRGCD), in central New Mexico, was created in its present form around 1930. During the design of the district, the engineers considered incorporating hydropower generation into the canal network. Although they concluded that hydropower was technically possible within the MRGCD, it was not economically viable as existing power supply far exceeded demand.

Today, the situation has changed. Hydropower is once again emerging as a viable, perhaps ideal, form of renewable energy. Recent technological advancements have enhanced the efficiency of low head turbines, making the possibilities for hydropower production in the MRGCD even more attractive. Currently, the MRGCD is identifying and categorizing all potential hydropower sites within its system with the goal of maximizing its contribution to the regions energy needs. This paper addresses key issues related to hydropower in irrigation districts and examines the feasibility of incorporating hydropower generation in the MRGCD canal system.

INTRODUCTION

The MRGCD has a long and interesting history, and may well be the oldest continuously functioning irrigation project in North America. Pueblo Indians were already making use of the waters of the Rio Grande for irrigation when Spanish explorers visited the area in

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the mid 16th century. The first actions of these Spanish settlers was the construction of acequias (irrigation canals), and presumably many of these early irrigation projects made use of existing Pueblo canals. The Spanish network of acequias rapidly grew in extent and complexity. A survey of ditches, made in 1706 when the city of Albuquerque was founded, documented the existence of much of what is today known as the Middle Rio Grande Conservancy District. Agriculture in the area reached its greatest extent in the late 19th century, with over 48,600 hectares in active cultivation.

By 1920, agriculture in the region was experiencing serious problems. The Rio Grande was a wild, uncontrolled, and unpredictable river. Flooding and drainage issues had resulted in damage to two thirds of the productive land in the area. What productive land remained suffered from an unreliable water supply during many years. Almost certainly, the lack of coordination of operations between approximately 80 acequia associations contributed to water supply and drainage problems. In 1917 citizens of the Middle Rio Grande (MRG) area contacted the United States Bureau of Reclamation (BOR) asking for assistance in solving these problems. The BOR declined to initiate a project, so the local population lobbied the State Legislature for assistance. The New Mexico Conservancy Act of 1923 authorized the formation of a Conservancy District for the MRG. This quickly led to initiation of study and survey work for the construction of the MRGCD. Design work did not abandon the existing elaborate hydraulic infrastructure in the MRG, but incorporated the various disparate elements into a cohesive centrally managed system. El Vado storage reservoir (242 million m^3) was constructed 160 kilometers upstream of the area; the river was channelized and flooding controlled through the construction of levees and jetties; the function of dozens of small diversion structures were consolidated into six major diversion works feeding a network of main canals; existing acequias became lateral canals; and a network of drainage canals returned excess irrigation water to the river and intercepted lateral seepage from the Rio Grande.

The plan for the MRGCD was documented in a report published August 15, 1928 (Burkholder, 1928). Included in that report is a discussion of hydropower potential in the MRGCD. The original plan included references to four types of power production potential

1. El Vado Reservoir
2. Diversion structures
3. Small drop structures in irrigation canals
4. Specific points where canals were considerably higher than the valley floor

Of these four types, the report concluded that the first three were impractical at the time. This is somewhat ironic in that one of these, El Vado, is the only site at which hydropower has actually been developed. The initial conclusion about El Vado potential was that it was not likely to operate often, or at constant rates. The report states*: It is evident that the necessities of operation as a storage reservoir conflict with those of a power reservoir and no continuous dependable flow of water for the generation of power can be counted upon*. (Burkholder, 1928).

Today, Los Alamos county operates an eight megawatt (MW) plant at El Vado, generating power from both natural flow and releases of stored water for irrigation needs in the MRGCD

The report then goes on to discuss potential at the Cochiti Diversion structure. Cochiti was a low head weir constructed for the purposes of supplying irrigation water from the Rio Grande into irrigation canals on both sides of the river. The Valley is steeper near its northern end, and power production at a diversion structure was considered most practical at that location. The plan assessed the power potential of the Cochiti diversion site as 884 kilowatts (KW) with year-round continuous operation, and noted that this estimate was based on a very conservative flow rate of 14.16 m^3/s. Even so, plans for a combined power/diversion structure at Cochiti were shelved as being cost prohibitive. In the 1960's, the United States Army Corps of Engineers (USACE) constructed a large flood control dam at the Cochiti Diversion site, inundating the original MRGCD structure and replacing its function with provisions for supplying the MRGCD canals through the outlet works. Water through the Cochiti Dam outlet works drops over 15 meters to the Rio Grande (Figure 1). Estimates of power production at this site range from 5 to 23 megawatts (MW) (Heggen, 1982), leading one to ponder why construction of a hydropower facility was not part of this recent structure.

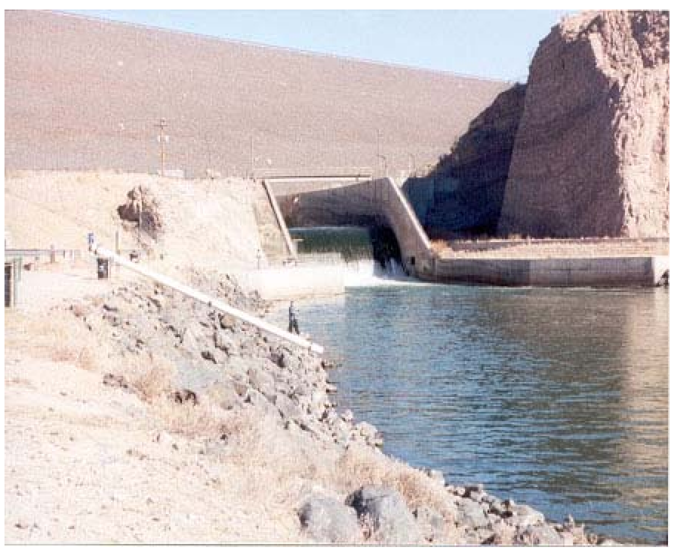

Figure 1. Cochiti Dam Outlet works, typical flow 28-42 m^3/s, max. 200 m^3/s

Small drop structures in the irrigation canals were considered next, but not explored in any great detail. However the report concludes: *it is thought that small individual installations might be of some value* (Burkholder, 1928). The report also concluded that hydropower was suitable at a few select locations where the irrigation canals were high above the valley floor and water was dropped to the valley floor either for irrigation delivery or return to the river system. Two of these sites were specifically noted and assessed for their power potential. Both of these sites were in the MRGCD Belen Division, and involved drops of 15 and 22.5 meters, with estimated power potential of 1.2 MW and 1.5 MW respectively (Burkholder, 1928) .

Unfortunately, none of these potential hydropower sites were constructed by the MRGCD. While clearly feasible, the economic setting of the MRG was not conducive to building these works. At that time, NM was rural and sparsely populated. There was minimal demand for electric power, and little money to buy it if produced, but the report did recommend that hydropower projects should be kept in mind as rather attractive future possibilities (Burkholder, 1928). The report also made indirect reference to the notion of operating MRGCD canals specifically for power production, keeping them in operation 12 months of the year, though irrigation water was only required eight of those months. The plan concludes with the following statement:

With present demand for power, such developments do not appear very attractive, but it is possible that in the future, with the country more thickly settled, a greater power demand might be created, possibly sufficient to absorb the entire output. In this case, the estimated financial return appears attractive enough to warrant construction of one or more of these power plants by the District, and this can be done at any time with very little change in the irrigation system now proposed (Burkholder, 1928).

At the time of the report, the population of the entire state of New Mexico was 423,317 (Forstall, 1995) When reading this statement, one can only wonder what those making the assessment would have thought had they foreseen the present day New Mexico population of 2,059,178 (US Census, 2010), the environmental movement, green energy credits, the potential for a market in carbon credits, and debate over causes of global climate change.

As a minor footnote, during the actual construction of MRGCD works between 1928 and 1932, one small scale hydropower (SSH) application was incorporated. This took the form of direct hydraulic drive of two turbine pumps to lift water 1.5 meters from the Albuquerque Main Canal into a lateral canal, the Bernalillo acequia. This system operated satisfactorily for over 20 years until the hydraulic drive was replaced with electric motors.

The idea of SSH in the MRGCD made a re-appearance in the form of a report (Heggen, 1982) sponsored by the New Mexico Energy Research and Development Institute. This report was oriented towards providing the MRGCD internal electrical needs at specific locations, rather than generation of power for other consumers. It did however make some interesting observations, and included references to both the environmentally sound nature of SSH; and evidence of an evolving social consciousness regarding the environment and energy production, which is still ongoing today. Another observation of note from that report is that peak power from canals is likely to be produced in summer and could be used for conjunctive groundwater and lateral lift pumping, reducing irrigation related power demands during a peak consumption period and allowing more efficient utilization of the state's other energy resources.

This report went on to examine 10 sites in the MRGCD, including the outlet works from the Cochiti Dam. Excluding the Cochiti Dam site, the remaining nine sites were estimated to have an aggregate power production potential of 2 MW. One site, known as

Feeder 3, accounted for half of that total, with the remainder being between 40 and 300 KW. The report concluded that SSH in the MRGCD was technically feasible, but that regulation, environment, and power integration make the idea a more complex issue than originally foreseen and that institutional constraints are significant, but increasing cost of purchased power, and the reduced costs of standardized SSH package units make SSH increasingly economically attractive (Heggen, 1982). Perhaps the time was just not right in the early 1980's for SSH to take hold in the MRGCD.

THE CHANGING ENERGY PICTURE

In 2008 New Mexico produced a total of 37,009,837 MWH, and consumed 24,019,000 MWH (USEIA, 2009). While New Mexico is still a net exporter of energy, local consumptive needs are expanding. New Mexico electrical consumption is growing at 3.3% per year, 50% higher than the national average of 2.2% per year. (DOE, 2008) Total New Mexico electrical consumption rose from 11,873,232 MWH in 1985, to 20,638,951 MWH in 2005 (USEIA, 2009) New Mexico energy production sources are at present heavily dependent on fossil fuel consumption at 49.8% coal, 42.5% natural gas, and 0.4% petroleum. Renewable sources make up only 6.3% and hydropower accounts for only 1% (USEIA, 2008).

After the passage of nearly three decades SSH is once again getting a fresh re-examination in the MRGCD, and the playing field has changed dramatically. Public consciousness is clearly changing. New Mexico is a state with great scenic beauty and many public lands. Tourism and outdoor recreation make up a significant segment of the economy. As a result, there has always been a strong environmental movement here, and it appears to be growing in influence. This changing attitude about energy and resource consumption resulted in the Renewable Energy Act of 2004, and passage of SB 418 by the New Mexico Legislature in March, 2007. These acts mandate Renewable Portfolio Standards (RPS) that require investor-owned utilities to generate 20% of total retail sales to New Mexico customers from renewable energy resources by 2020, with interim standards of 10% by 2011 and 15% by 2015. The RPS for rural electric cooperatives is 10% by 2020. Renewable energy is defined in the bill as electric energy generated by low- or zero-emissions generation technology with substantial long-term production potential. Specific categories include solar, wind, geothermal, and hydropower facilities brought in service after July 1, 2007. (Blankenship, 2010, letter to MRGCD) Renewable energy does not include electric energy generated from fossil fuel or nuclear facilities.

The RPS has in turn given rise to the phenomena of Renewable Energy Certificates (REC). A REC represents one kilowatt-hour (KWH) of renewable electricity. Utilities seeking to meet renewable energy standards document compliance with a Renewable Portfolio Standard (RPS) through the use of RECs. It is important to note that the REC have a potential real value, and may be separate from the actual energy produced, and an active market is emerging for the trading of REC's. RECs used for compliance on or after January 1, 2008 must be registered with the Western Renewable Energy Generation Information System (WREGIS).

In addition to the requirement that New Mexico utilities obtain power from renewable resources, there are direct benefits to agricultural water users from SSH production in MRGCD through the enhancement of beneficial use. As in many western states, water rights in New Mexico are defined in terms of beneficial use. The use of irrigation water for agricultural production is clearly a beneficial use. The production of SSH could make the act of delivering that water a beneficial use. Two possible outcomes might result. Either some component of the incidental loss associated with irrigation delivery could be attributed to power generation, increasing the beneficial use associated with existing agricultural rights. Or a supplier of irrigation water such as the MRGCD could directly hold and prove water rights by being able to demonstrate beneficial use.

Along similar lines, there is an important public relations benefit to SSH. Irrigated agriculture is often criticized by the urban population as being an inefficient user of water. This same population is so well insulated from the reality of human existence that many of its members simply fail to understand that modern society depends on agriculture. However, these same individuals, accustomed as they are to the comforts of a modern electrified world, are acutely sensitive to power. While the dollar value of electricity produced from SSH pales in comparison with the agricultural output associated with the same water use, urban dwellers might be more likely to perceive and support SSH production as a direct benefit to them.

AVAILABLE TECHNOLOGY FOR SMALL SCALE HYDROPOWER

The term SSH is used in this report to describe projects with capacity ranging between 15 and 100 KW. There are different terms in use, and no universally agreed upon standard. In many places, the projects being investigated by the MRGCD might be termed mini or micro-hydro (Paisch, 2002). The MRGCD SSH projects will involve use of water already in conduits (irrigation canals) for other (agricultural) purposes. No construction of dams or reservoirs are required, making SSH one of the most cost-effective and environmentally benign technologies available for future energy development (Paisch, 2002).

At the most basic level, hydropower involves harnessing the potential energy of water through a mechanical device. The higher the potential energy the greater the power. Power is also dependent on the mass of water (volume) and with higher volume comes increased power. Hydropower technology is very old, having been in use over 2000 years. Hydropower reached impressive levels of refinement by the 18th century when simple wood and metal waterwheels were achieving efficiencies approaching 70%. In the 19th century more sophisticated engineering and the need to generate electricity led to the development of modern day turbines (Paisch, 2002).

Hydropower turbines can generally be categorized as either impulse or reaction devices. An impulse turbine rotates freely in the air, and the energy of water striking its blades or buckets imparts motion. Impulse turbines are generally not suited to heads of less than 10 meters, so will not be considered for SSH projects in the MRGCD. Reaction turbines operate fully immersed in water. Movement of water across the blades of the runner (shaft and blades combined) creates a pressure differential (lift), imparting rotational motion to the runner. Reaction devices require more sophisticated fabrication and careful design of the blade and casing, but when properly engineered can result in high efficiency at low head (Energy Trust of Oregon, 2009). There are numerous variations on the reaction turbine, but in general all may be classed as propeller, Kaplan, or Francis style devices. The MRGCD SSH projects will feature low head/high flow and will tend to be best suited to the Kaplan style turbine (Energy Trust of Oregon, 2009). Reaction turbines may be mounted directly in the stream (canal), but access for operation and maintenance may be improved by elevating the turbine above normal water level and passing water through the device by means of a siphon tube. This tends to eliminate the construction of below grade civil works and their inherent moisture sealing difficulties.

There are also kinetic energy turbines, which operate by placing the device into flowing water. The most familiar example of this type of device would be the undershot waterwheel. Although the kinetic energy turbine seems to derive its energy from water flow, flow is created by the drop of the natural stream or channel, over some distance, and thus the energy produced still ultimately derives from potential energy between two points. Efficiency of this type of device is likely to be low, but since it may be used almost anywhere with minimal construction or modification, may prove to have some utility for the MRGCD. A modern example of such a device is the Hydrovolts flip wing which can be installed in a matter of hours. Given the potential for widespread use on irrigation canals, these open channel kinetic energy turbines probably merit further development. With increases in efficiency and debris-handling characteristics, the kinetic energy turbine could prove increasingly attractive. Since the kinetic energy turbine is placed into the flow of canal, there might also be the possibility of developing these devices as upstream level controllers, allowing canal control for irrigation purposes to also generate electrical power. The Schneider linear hydroengine (Natal Energy) appears to be an interesting hybrid between a reaction turbine and a kinetic energy device. The manufacturer claims high efficiency in low head situations at rates of flow typically found in the MRGCD system.

One very important consideration must be present during engineering design work for the MRGCD SSH projects. Due to the low head conditions of the MRGCD projects, turbines must be sized and designed to provide high efficiency. Though some turbine designs operate across a wider range of discharges than others in all cases efficiency decreases as the rate of flow through the turbine deviates from the rated specification. For SSH to be successful in the MRGCD, sites must be selected and operated to stay within a very narrow range of planned discharge. Due to the highly controlled nature of the MRGCD works, this may not be a difficult task, and will coincide with the improvement of irrigation water deliveries to agricultural users of the water supply.

SITES FOR SMALL SCALE HYDROPOWER IN THE MRGCD

Initial identification of potential sites for SSH in the MRGCD was done sequentially following the protocol presented. Discussions with vendors of SSH equipment suggested that head differential of less than one meter was technically feasible, but likely to have an unfavorable cost/benefit ratio in the range of flows typically found in MRGCD facilities

(0.6-7 m^3/s). A total of nine sites were initially listed, based on their relatively large available head. These are by no means all of the available sites in the MRGCD. There are probably in excess of 100 sites with SSH potential. These nine sites probably represent the most advantageous (Figure 2), although there are still many additional locations that merit investigation. Of the remaining unexamined sites, we estimate that 20% will likely have similar potential to those described below.

For comparative evaluation purposes, consistent evaluation parameters were used (unless otherwise noted) to estimate each site's SSH potential. Generation efficiency (GE), or how much of the available potential energy can be converted to electrical energy, was derived from literature of various equipment manufacturers. For purposes of this report, generation efficiency includes both turbine efficiency and the generator efficiency. Most of this variability appears due to turbine efficiency. Generator efficiency appears consistently higher. The overall GE range was as low as 60%, to in excess of 90%. Most vendors claimed in the 80 to 90 % range, so 80% was assumed for this report as a reasonable estimate of generation efficiency.

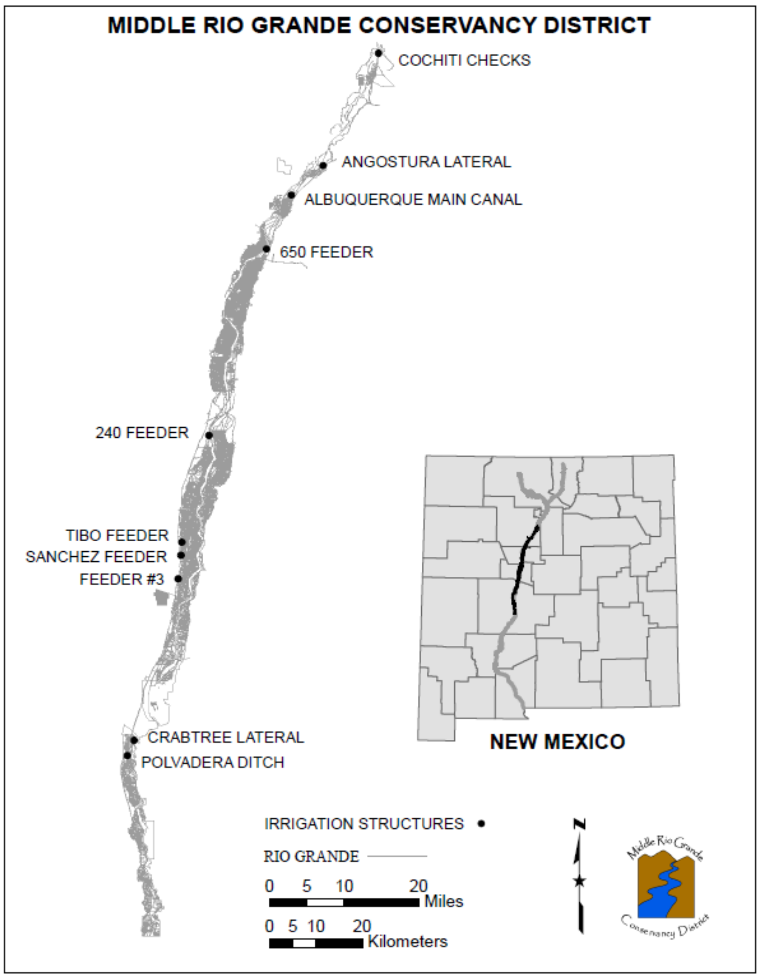

Figure 2. Potential Small Scale Hydropower sites in the MRGCD

Service factor (SF), or the percentage of a given period of time that normal generation could be expected to occur, tended to be high in vendor's literature. SF includes down time for maintenance/repairs. Most vendors quoted a higher service factor, 90 to 95%. However a more conservative 80% value was chosen.

Annual hours (AH) is the expected number of hours in each calendar year which a SSH plant could be expected to operate. In some cases this is the length of the normal irrigation season (245 days), in other cases it is the entire year. AH is multiplied by SF to determine the total annual operating hours (OH) for each site.

To provide a general idea of the economic scale of these projects, an estimated value of power was made for each. The estimated value includes a purchase price (PP) of the produced power assuming it will be sold to a local utility, and a value for the REC. The actual value of these parameters will depend on negotiations with local utilities. The average retail residential rate in New Mexico is approximately $0.10/KWH. Negotiated rates for large commercial users may be as low as $0.065/KWH. In the vicinity of MRGCD, the local electric utility has purchased renewable power at rates as high as $0.15/KWH. (personal communication 2010, Andrew Camillo, City of Belen) At sites in other states, PP ranging between $0.05 and $0.08/KWH was found (Blankenship, 2010). A reasonable compromise for evaluation purposes has been selected as $0.075/KWH. A value of $0.05/KWH was assigned to each REC (Blankenship, 2010). These evaluation factors are far from precise, and will certainly be revised over time, but at present add some economic dimension possible projects. Table 1 and Equations 1 and 2 summarize the terms used to evaluate each location in the MRGCD.

Table 1. Evaluation terms and values utilized for determining feasibility of small scale hydropower in the MRGCD

Term	ID	Value	Unit
Operating hours	OH		hour
Annual hours	AH		hour
Service factor	SF	0.8	
Generation efficiency	GE	0.8	
Available Head	H		meter
Discharge	Q		cubic meter/sec
Capacity	KW		kilowatt
Production	KWH		kilowatt-hour
Purchase price	PP	0.075	$/KWH
Renewable credit	REC	0.050	$/KWH

Generation Capacity = (H)(Q)(9.81)(GE) = KW Equation 1 (Paish, 2002)

Annual Production = (KW) (OH) = KWH Equation 2

The Belen Feeders

The first four sites described are located along the Belen High Line Canal (BHC). The BHC is the MRGCD's largest and longest canal, and it is a true highline canal, following the uppermost irrigable contour along the west side of the MRG valley. By remaining on the highest possible contour, it eventually attains considerable vertical distance above the valley floor. With this arrangement, several feeder ditches turn water off the BHC and drop it to an elevation where water can be utilized. These feeders combine significant discharge with significant drop, and so are obvious choices for SSH evaluation. The four feeders described below do not represent all possibilities. There are a number of lesser lateral headings, some of which have significant drops. There is also Feeder #2, which was examined in both the original report (Official Plan of MRGCD, 1928). and the 1982 report (Heggen, 1982).

Collectively, these four locations (some utilizing more than one turbine/generator) have the potential to produce over 1.7 million KWH annually if operated on normal irrigation flows through the eight month irrigation season. If higher rates of flows were to be diverted intentionally for SSH generation purposes, and maintained throughout the entire 12-month period, annual generation potential could potentially increase to 6.2 million KWH.

Feeder 3

Feeder 3 may be easily considered the star of the MRGCD's potential SSH sites. This site is noted both in the original report (Burkholder, 1928) and the 1982 report (Heggen, 1982) where it is identified as site #10. This is for good reason, as the BHL climbs to as much 22.5 meters above the valley floor here, and discharge can be quite high. The original report (Burkholder, 1928) contemplated an irrigation demand for Feeder 3 of approximately 7.08 m^3/s, suggesting power potential of 1.25 MW. Unfortunately, the irrigation potential of this portion of MRGCD was never fully developed, and in recent years some irrigated lands have gone out of production, so discharges are less than the level originally considered. Also, because hydropower was not made a part of the original project, the arrangement of drop structures along Feeder 3 was guided by cost and practicality, not the intent to maximize power production. The existing facility today has 3 sets of multiple weir drops, each totaling 6.1meters of drop. The remaining drop occurs through miscellaneous pipes and check structures. It would be practical to incorporate SSH on the three sets of drop structures. The most likely arrangement would be to take the flow into a pipe at the top of each set, leaving the original drops to act as overflows or bypasses should greater flow be required to the canals. Figure 3 shows the lower of these three sets of drop structures along Feeder 3.

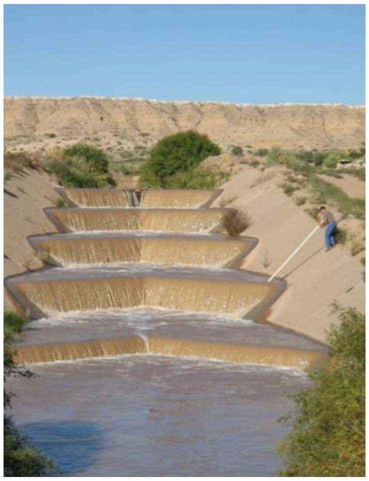

Figure 3. Lower set of drops on Feeder 3

The upper two sets of weirs can have discharges ranging from 0.85 to 4.25 m^3/s. A nearly constant irrigation demand of about 6.1 m^3/s is present below Feeder 3. It would be in the interests of both irrigation delivery and power production to stabilize discharge at 6.1 m^3/s. At that rate, power production potential for the two upper sets of drops is 95KW each.

There is a significant water take off between the 2nd and 3rd set of drops. The Garcia Lateral removes 1.42 m^3/s of the discharge from Feeder 3. So the final set of drops, while still at 6.1 meters head, will only receive a constant discharge of 0.57 m^3/s. This reduces its power potential to 27 kW. The total annual production if all 3 drops are fitted with SSH generation would be 999,000 KWH, with an annual value of about $103,000. It may however be most practical to do only the upper two sets of drops, as the output from the 3rd set is low, and discharge is likely to be most variable there. In this case, the total annual production from Feeder 3 drops to 874,000 KWH, or about $109,000 (Table 2).

There is a possibility of using the Feeder 3 site for year-round SSH generation, and perhaps even increasing discharges. The existing BHL has ample excess capacity, and the Feeder 3 canal and wasteway can readily pass 4.25 m^3/s back to the Rio Grande. Under this operating scenario, 4.25 m^3/s constant diversion would be maintained to the BHL during the non-irrigation season. An additional 2.26 m^3/s would be diverted to the BHL during the irrigation season with the intent to stabilize Feeder 3 discharge at 4.25 m^3/s. Many potential benefits could accrue to irrigation uses, but there would be challenges. Increased diversions would not always be possible during low-flow periods in the Rio Grande. Also, considerable opposition from environmental groups might occur based on the presence of an endangered species, the Rio Grande Silvery Minnow (RGSM), in the Rio Grande. Should these hurdles be overcome, power production potential of the Feeder 3 increases to 203 KW for each drop, though the 3rd drop would only achieve 135 KW during irrigation season. Annual production would rise to nearly 4 million KWH, and $500,000 (Table 2).

Tebo Feeder

At the Tebo Feeder, water is split off the BHC to supplement the New Belen Acequia (NBA), and several lesser ditches. There is an upper series of drops, where water passes from the BHC down to the NBA, then a single large drop from the NBA to the lower ditches. The upper series of drops totals 7.6 meters over a series of six structures in a distance of 60 meters. By replacing this section of canal with pipe the entire 7.6 meters of head could be harnessed for SSH generation. Flow can be variable in the upper section, as it is used to supplement the lower ditch flows. While discharge might be as high as 1.42 m^3/s, it would most typically be about 0.57 m^3/s. For the purposes of SSH, discharge could be stabilized at 0.57 m^3/s, and higher flows bypassed to the original canal and drop structures when required. The upper Tebo drop has a power production potential estimated at 34 kW. Figures 4 shows the two drops along the upper Tebo Feeder.

Figure 4. Three of six drops on the upper Tebo Feeder

The lower drop occurs on the NBA just after water from the Tebo Feeder has entered. The drop occurs over a single steeply inclined chute, and is 6.1 meters. Discharge here is typically 1.13 m^3/s. Electrical production capacity of the lower Tebo drop is 54 kw. Figure 5 shows the chute on the lower Tebo Feeder.

Figure 5. Steep chute at lower Tebo Feeder

A SSH installation at the Tebo Feeder is expected to include two generating units, one for each drop. These would be located about 40 meters apart. PNM utility lines pass directly south of the lower drop location. Both installations would operate at the same time, and throughout the irrigation season. Routing water back to the Rio Grande from this location is complicated, involving several small ditches, so operation would be limited to the irrigation season. Annual production from the Tebo Feeder site is estimated to be 406,000 KWH, with a value of $50,000 (Table 2).

Sanchez Feeder

The Sanchez Feeder supplies a single small lateral canal at a nearly constant rate of 0.42 m^3/s. This relatively small discharge is countered with a drop of 9.1 meters, and this is currently contained in a pipe. The installation of a SSH at the mouth of this pipe would be simple, and would tend to remedy erosion problems which currently exist near the exit from the pipe. This site would be operated only during the irrigation season. The Sanchez Feeder has an estimated power production potential of 30 KW. Annual production from the Sanchez Feeder site is estimated to be 138,000 kWh, with a value of $17,000 (Table 2).

240 Feeder

The 240 Feeder is the first point where water is split from the BHC and dropped to several lower laterals. Since this feeder is near the heading of the BHC from the Rio Grande (4.5 kilometers), the vertical drop is not great. The discharge at this point is potentially quite high. Also of significance is that the 240 Feeder is a short distance from the Rio Grande and ends in a wasteway from the BHC directly back to the Rio Grande. The capacity of both the BHC and the 240 Feeder are relatively large. This situation raises the possibility of operating the 240 Feeder specifically for the purposes of power generation12 months of the year. Existing PNM utility lines are within 70 meters of this structure. Figure 6 shows the 240 Feeder structure.

Figure 6. 240 Feeder structure

Normal discharge to the 240 Feeder for irrigation purposes is 2.26 m^3/s, (range:1.98-2.83 m^3/s). This discharge is relatively constant for the eight months of the irrigation season, and with little effort could be stabilized even further. The total drop from the BHL to the first irrigation lateral supplied by the 240 Feeder is 2.4 meters, and this occurs in two steps about 15 meters apart. With minor modification the entire drop could be made available for SSH generation. Power production potential of normal irrigation use of the 240 Feeder is estimated at 43 KW. If operated only during the irrigation season annual production would be 196,000 KWH, with a value of $24,000.

If the 240 Feeder were to be operated 12 months of the year, at the same discharge rate, the installation would potentially yield 294,000 kWh, with an estimated annual value of $36,000. With the ability to directly return water to the Rio Grande at the 240 Feeder, it is possible to divert larger volumes of water for SSH generation purposes in excess of what would normally be required to meet irrigation demands. If maximum flow to SSH was increased to 4.25 m^3/s, generating capacity could be increased to 81 kW. At times, there might be insufficient flow available in the RG to meet the added diversion requirement, and discharge would be reduced to that required to meet irrigation demand, causing output to drop to 43kW. If operated in this manner, total annual production could be expected to increase to 500,000 KWH, with a value of $50,000 (Table 2).

Various other types of situations and structures abound throughout the MRGCD. While few locations have drops of the magnitude found along the BHL, some are still quite significant, and often have large rates of flow. Most obvious are check structures along the MRGCD's main canals. While not all are candidates, some are worth considering.

Cochiti Checks

Like the BHL, the Cochiti Main Canal (CMC) is a highline canal following the highest possible contour along the east side of the MRG valley. While there are no large drops to laterals, the northern end of the valley where the CMC is located has a considerably steeper gradient than the southern portions. As a result, there are many check structures along the CMC. At the upper end of the Cochiti Main canal there are a series of four structures over a distance of about three kilometers. The first three of these are dimensionally identical, and result in drops of 0.9 meter each. While the drop is minimal, discharge here is unusually constant, at 2.55 m^3/s. A PNM utility line parallels the CMC right-of-way in this area. Since the three checks are identical, it may prove practical to install three SSH units. Each unit would have a potential for 17KW, with annual production estimated at 84,000 KWH each (Table 2).

The fourth check along this section of the CMC has 1.8 meters of drop and a slightly smaller discharge of 2.26 m^3/s. A SSH unit at Check Four would have a capacity of 32 KW and annual production of 150,000 KWH. Considering the close proximity of these sites to one another, and their easy access to transmission lines, it is easy to imagine these sites operating as a unit. Though individually their outputs are relatively small, collectively they have potential for 400,000 KWH annually, with an estimated value of $49,000 (Table 2). These four sites are located on the lands of the Cochiti Pueblo, raising the interesting possibility of a tribal/MRGCD partnership.

It is also worth noting that there is a drop chute on the CMC, near the Santo Domingo Pueblo. This structure was not recently evaluated, but with a potential drop of about 4.5 meters, certainly should be investigated when time allows. Figure 7 displays the fourth check on the CMC.

Figure 7. Fourth check on the Cochiti Main Canal

Angostura Lateral

The Angostura Lateral (AL) is near the southern end of the Cochiti division, and receives its flow from the CMC. For all practical purposes, it could be considered the southernmost extension of the CMC. The AL is a small canal, but there are several interesting circumstances surrounding it which might make SSH an attractive proposition.

While flows to the AL are much reduced from the CMC, there is a series of structures involving considerable vertical drop. Over a distance of about 500 meters, the AL drops a total of 12.2 meters. There are no takeoffs along the AL in this area, so presumably the AL could be placed into closed conduit and that entire energy head made available for SSH generation. Discharge to the AL is normally 0.56 m^3/s to meet irrigation demand. However the AL has a design capacity of 1.13 m^3/s. The AL serves almost entirely Indian irrigators on the San Felipe and Santa Ana Pueblos. There has been interest from Indian irrigators in the area served by this canal of bringing lands back into production and operating the canal to its original design capacity. If this were to be done, a constant discharge of 1.13 m^3/s could be maintained through the eight month irrigation season. SSH capacity of the AL would be 108 KW. It may be most practical to break the total drop up into two segments, and operate two 54 KW SSH units. Annual power production would be 498,000 KWH, with a value of $62,000 (Table 2). As an interesting aside, this location is a few hundred meters from a PNM natural gas fired generating station, which currently hosts a demonstration photovoltaic installation. Figure 8 displays the drop along the Angostura Lateral

Figure 8. Drop in elevation along the Angostura Lateral

Albuquerque Main Check #1

The Albuquerque Main Canal (AMC) is the primary canal for delivering irrigation water to about 10,000 acres in the Albuquerque Division. Water is diverted from the Rio Grande at the Angostura Diversion dam. About eight kilometers downstream from the heading the AMC encounters its first major structure. This structure drops the entire flow of the AMC 2.5 meters. This location is unique in that it was originally constructed as a hydropower facility for the MRGCD. A pair of turbines used the energy of the vertical drop to mechanically drive a pair of pumps, lifting 0.71 m^3/s five vertical meters into the heading of the Bernalillo Acequia. These turbines were supplied by the Pelton Water Wheel Company of San Francisco, California but were most likely not Pelton wheel design. Unfortunately these machines are lost to history, but a drawing of the original structure indicates a many-bladed runner on a vertical shaft with a tapered draft tube guiding flow to the bottom of the structure. These turbines were replaced in the 1950's with typical shallow draft irrigation pumps coupled with electric motors.

The AMC typically operates at discharge of 3.96 m^3/s with 2.4 meters of available drop at the Check #1. SSH capacity of this structure would be 79 KW. Operation of this location would only be practical during the eight month irrigation season. Annual power production would be 344,000 KWH with a value of $43,000 (Table 2). Figure 9 displays the drop associated with Albuquerque Main Canal Check #1

Figure 9. Drop associated with Albuquerque Main Canal Check #1

650 Feeder

The AMC joins with another canal, the Atrisco Feeder Canal, at a short cross-linking canal known as the 650 Feeder (650). At the 650, residual flows from the upper end of the AMC are combined with a fresh influx of water from the Atrisco Feeder (AF), and are then distributed either to the lower end of the AMC, or to a separate area via continuation of the AF. There is also a short wasteway from the 650 directly back to the Rio Grande, so that flows in excess of irrigation demand may be released. The 650 drops 2.4 meters into the continuation of the AF. Figure 10 displays the 650 Feeder. Discharge through this drop structure is maintained throughout the irrigation season at a near constant 2.26 m^3/s. During the winter months, drain accretions are routed through the 650 Feeder via this same drop structure at nearly the same rate. SSH capacity of the 650 would be 43kW. Annual power production would be 196,000 KWH, with a value of $24,000 (Table 2).

Figure 10. 650 Feeder

While operation of the AMC would not be practical in the off-season, and the drop structure discussed above is near capacity at 2.3 m^3/s, there are other possibilities for SSH at the 650. The wasteway to the Rio Grande has a large capacity, probably in excess of 12 m^3/s, and depending on stage of the RG, as much as 2.1 meters drop. It is possible to route water from the Angostura diversion dam to this wasteway via the AF without affecting irrigation deliveries. A diversion at Angostura Dam of 5.66 m^3/s made specifically for SSH production could result in capacity at the wasteway of 95 KW. Annual power production could be as much as 665,000 KWH, with a value of $83,000. All water used in this process would be returned to the Rio Grande after a temporary bypass of about 16 kilometers.

There is also a drop of about 1.5 meters from the upper AMC into the lower AMC at the 650. Discharge at this point is typically 1.98 to 2.83 m^3/s, depending on irrigation demand. A steady discharge of 2.26 m^3/s results in capacity of 27 kW, and production of 125,000 KWH with a value of $15,600 over the eight month irrigation season. All three of these possible SSH sites are located within 100 meters of each other, and adjacent to existing utility lines. This site is also located on lands of the Sandia Pueblo again raising the possibility of tribal/MRGCD partnership. If the entire location was exploited to its maximum potential of 165 KW, annual production could be a little more than 986,000 KWH, with a value of $123,000.

Socorro Main Canal

The Socorro Main canal (SMC) is the sole source of water for the MRGCD's southernmost division. Designed with a capacity of 7.50 m^3/s, it typically operates at 5.10-6.23 m^3/s. There are a number of major check/drop structures along the SMC. The uppermost structure is known as the Crabtree Check, and involves a drop of 1.5 meters. One lateral leaves the SMC just above this structure, so discharge is reduced from 5.66 m^3/s to 4.53 m^3/s. SSH capacity of the Crabtree check is 54kW. A short distance downstream is a second check of similar dimensions, but water is not generally checked to any useful degree at this point.

The third drop/check on the SMC is to provide operating head for the Polvadera Lateral (Figure 11). The off-take to the Polvadera Lateral is typically 1.13 m^3/s, reducing flow at the check to 3.40 m^3/s. But the drop at this structure is significant at 2.4 meters so SSH capacity is 65 KW, higher than at the upstream structure. These two structures combined have a power production potential of 550,000 KWH during the 8-month irrigation season, with a value of $69,000 (Table 2).

Figure 11. Drop at Polvadera Check

Several additional structures downstream from these two locations would also be likely candidates for SSH generation, though discharge gets successively smaller with each structure. If fully exploited, the SMC might make a notable contribution to the power supply of the Socorro Rural Electric Cooperative. While at present it would be unrealistic to operate the SMC during the winter months, at some time in the future it might be advantageous to consider. If necessary improvements were made to the canal and structures, a discharge at or near canal capacity could be maintained during the winter months for the specific purpose of generating power. This flow could be returned either to the Rio Grande, or to the Low Flow Conveyance Channel. While this would undoubtedly have a noticeable impact on river flow, and thus would raise environmental considerations, the amount of power produced could approach 3,000,000 KWH over just the short 4-month winter period.

Table 2. Summary table of nine locations, normal irrigation season only

Location	H (m)	Q (cms)	GE	Capacity kW	OH	Production KWH	PP ($)	REC ($)	Value ($)	Sum ($)
Feeder 3 #1	6.1	1.98	0.8	95	4608	436,784	0.075	0.05	54,598	
Feeder 3 #2	6.1	1.98	0.8	95	4608	436,784	0.075	0.05	54,598	
Feeder 3 #3	6.1	0.57	0.8	27	4608	125,741	0.075	0.05	15,718	124,914
Tebo Feeder upper	7.6	0.57	0.8	34	4608	156,661	0.075	0.05	19,583	
Tebo Feeder lower	6.1	1.13	0.8	54	4608	249,276	0.075	0.05	31,159	50,742
Sanchez Feeder	9.1	0.42	0.8	30	4608	138,217	0.075	0.05	17,277	17,277
240 Feeder	2.4	2.26	0.8	43	4608	196,151	0.075	0.05	24,519	24,519
CMC #1	0.9	2.55	0.8	18	4608	82,995	0.075	0.05	10,374	
CMC #2	0.9	2.55	0.8	18	4608	82,995	0.075	0.05	10,374	
CMC #3	0.9	2.55	0.8	18	4608	82,995	0.075	0.05	10,374	
CMC #4	1.8	2.26	0.8	32	4608	147,113	0.075	0.05	18,389	49,512
AL #1	6.1	1.13	0.8	54	4608	249,276	0.075	0.05	31,159	
AL #2	6.1	1.13	0.8	54	4608	249,276	0.075	0.05	31,159	62,319
AMC #1	2.4	3.96	0.8	75	4608	343,699	0.075	0.05	42,962	42,962
650	2.4	2.26	0.8	43	4608	196,151	0.075	0.05	24,519	24,519
SMC Crabtree	1.5	4.53	0.8	53	4608	245,732	0.075	0.05	30,716	
SMC Polvadera	2.4	3.4	0.8	64	4608	295,095	0.075	0.05	36,887	67,603
Nine location total:				806		3,714,940				464,368

BENEFITS

The development of SSH in the MRGCD has several important benefits. The initial set of nine locations(Table 2), if developed for normal irrigation season use, represent about 800 KW in capacity. The power produced represents an income stream, and one which could reasonably be expected to continue over a long period of time. No lifecycle cost estimates of these installations have been made, but the estimated life span of a small SSH installation should be at least 20-25 years (Blankenship, 2010), and perhaps as much as 50 years (Paish, 2002). Initial cost recovery is expected to be on the order of 4 to 12 years (Blankenship, 2010). Annual electrical production of these facilities is estimated at 3.7 million KWH, with a value of $464,000. Adding 12-month generation at two of the locations (650, 240 Feeder) would increase annual output to 3.95 million KWH, with a value of $495,000. Construction and operation of these proposed locations would have no negative impact on irrigation water delivery, and in a some cases would be beneficial to irrigation by focusing attention on maintaining stable rates of flow.

The public relations benefit of SSH is undeniable, and likely outweighs its economic value. As previously discussed, there is widespread misunderstanding of the importance of irrigated agriculture. If delivery of irrigation water becomes associated with the production of green energy, the non-agricultural sector will benefit. While the nine locations currently being considered for SSH in the MRGCD would add only about 0.01% to New Mexico’s total electrical output, this would be a useful contribution to local energy needs. This may be portrayed in a different manner. Annual average residential energy consumption in the US was 11,040kWH in 2008. New Mexico averages a somewhat lower annual residential energy consumption of 7704 KWH (2009). The estimated total annual power production potential of these 9 sites is 3,999,880 KWH, or enough electricity to power 519 homes. The fact that these 519 homes would be powered by 100% sustainable, non-carbon consuming power is noteworthy.

If all of these sites were maximized to their fullest potential, diverting year round specifically for power generation, annual power production value would be approx $1,032,000. Total SSH production potential could be 2.2MW, and annual output in excess of 10 million KWH . Under this scenario MRGCD could power 1340 homes, and there is the possibility of additional locations in the future.

Potentially interesting partnerships might develop to encourage SSH development in the MRGCD. Many potential SSH locations are on Native American lands. The works of the MRGCD pass through six of New Mexico's 19 Pueblos. While most MRGCD facilities in non-tribal areas are held fee- simple, this is not the case on tribal lands. The Pueblos granted easements to the MRGCD for its facilities, in exchange for improved water delivery. Should the MRGCD desire to develop SSH resources on tribal lands, some recognition of this unique relationship will be required. An MRGCD/Tribal partnership might make sense for both parties. The inclusion of a Tribal interest would likely open doors for federal funding, and would encourage federal agency support for any permitting or environmental issues which might arise. For their part, an involved Pueblo might be assured of a steady source of either income or electrical power, as some sort of sharing agreement would seem logical. There might also be considerable public relations benefit for a Pueblo involved in the development of a SSH project. Another likely possibility would be partnerships between the MRGCD and local municipalities. Local municipalities could assist the MRGCD with regulatory and utility interaction through their political involvement.

The MRGCD is a publicly funded political subdivision of the State of New Mexico. A question that would arise, should the MRGCD successfully implement SSH, and generate a significant annual revenue stream as a result, is how this money should be used. The simplest and most obvious answer is that this revenue would return to the MRGCD general operating fund, offsetting some of the cost of providing services to its constituents. This could then result in a reduction of the ad-valorem property tax currently assessed on MRGCD constituents. An alternate possibility would be the reduction in the water delivery charges levied on water users, though this approach would likely be met with opposition by the large and sometimes vocal non-water using constituents. Another possibility would be to dedicate the revenue from SSH to a specific purpose. In recent years there has been considerable interest from urban MRGCD constituents who do not receive water to enhance recreational opportunities on MRGCD facilities. One drawback to providing developed parks and walking, running, and riding paths on MRGCD lands is that it necessitates increased maintenance costs. These costs are resisted by agricultural water users who correctly view maintenance of canals and water delivery structures as the primary focus for limited funds. The use of SSH revenue to offset the increased costs incurred by development of recreational amenities could help to close the gap between these diverse constituent groups.

CONCLUSIONS

The development of SSH in the MRGCD was considered technically feasible over 80 years ago, and engineers at that time predicted it would likely occur as population in the region grew. Today, there is a clear need for this country to maximize any potential sustainable and green energy source. SSH is an obvious choice, and while not likely to ever constitute a major percentage of the region's energy needs, it is low hanging fruit with essentially zero environmental or regulatory drawbacks. In the case of projects such as those contemplated by the MRGCD, water has already been removed from the stream system, placed in conduits for agricultural delivery, and is already passing through structures designed to direct or dissipate its energy.

The MRGCD has numerous structures well suited to SSH today. A few of these stand out as having higher potential than others, and those should logically be the first to be exploited. As MRGCD gains more experience with SSH, and the energy market demands increasing percentages of green, locally produced power, second and third tier sites will be developed. Technological improvements to SSH may also be expected in the future, but these will likely be incremental improvements, fine tuning of efficiency and reliability numbers, and not revolutionary changes. After all, the physics of mass and gravity govern power output, and efficiencies are already relatively high.

The development of partnerships; tribal, municipal, or perhaps even commercial, will help ease political and financial challenges of SSH development, and should be explored. These partnerships may have great benefit to MRGCD in terms of building relationships and support for its mission of delivering water and sustaining agriculture in the region. Similarly, the production of a valuable consumer commodity in addition to agricultural output has important political and legal benefits for MRGCD and its constituents.

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REFERENCES

Burkholder, Joseph L. (1928). Report of the Chief Engineer. Submitting a Plan for Flood Control, Drainage, and irrigation of the Middle Rio Grande Conservancy Project
Forstall, Richard L., Population Division, US Bureau of the Census (1995). Population of Counties by Decennial Census: 1900 to 1990

US Bureau of the Census (2010). 2010 US Census data

Heggen, Richard J. (1982). Small Scale Hydropower from Irrigation Canals Near Albuquerque, New Mexico

Blankenship, Joe (2010). Manufacturers Feasibility Analysis of Prospective Projects

Paish, Oliver (2002). Small Hydro Power: Technology and Current Status. Renewable and Sustainable Energy Reviews 6: 537-556

Summit Blue Consulting, submitted to Energy Trust of Oregon (2009). Small Hydropower Technology and Market Assessment

U.S. Energy Information Administration (2009), Form EIA-923, Power Plant Operating Report

United States Department of Energy, Energy Efficiency and Renewable Energy Information Office (2008). Electric Power and Renewable Energy in New Mexico

U.S. Energy Information Administration (2009), Form EIA-860, Annual Electric Generator Report

Blankenship, Joe , Natal Energy (2010) Letter to MRGCD "NM RPS Requirements"

DEVELOPMENT OF THE CARTER LAKE HYDROPOWER PROJECT

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ABSTRACT

The Northern Colorado Water Conservancy District is developing the Carter Lake Hydropower Project. The project will connect to a recently constructed new outlet works bypass at the United States Bureau of Reclamation's Carter Lake Reservoir which is part of the Colorado-Big Thompson Project. The hydro project consists of 200 feet of new 72-inch penstock, two 1,300 kilowatt turbines, a powerhouse, switch gear, and a connection to the existing St. Vrain Supply Canal. The turbine/generator equipment has recently been ordered, and the design is underway for the remaining civil works. The project will be in operation in mid-2012. Challenges have included working through Reclamation's recently enacted Lease of Power Privilege requirements, determining the amount of risk acceptable in year-to-year flow variability as it relates to power production and loan repayment, obtaining a power purchase contract, and securing funding for the project. This paper will present information about both the technical and non-technical aspects of moving forward with the project and lessons learned along the way.

BACKGROUND

Carter Lake Reservoir is a 112,000-acre-ft impoundment located approximately 50 miles northwest of Denver. It was constructed in the early 1950's as part of the United States Bureau of Reclamation (Reclamation) Colorado-Big Thompson Project (C-BT). The reservoir is owned by Reclamation and operated by Northern Colorado Water Conservancy District (Northern Water). A map of the C-BT is shown on Figure 1.

Water from the C-BT project originates from the Colorado River, is diverted under the continental divide through the Adams Tunnel, generates hydropower through Reclamation power facilities, and then is delivered to both the Horsetooth Reservoir and Carter Lake Reservoir east slope terminal storage facilities. In total, an average of 210,000 acre-ft of C-BT is delivered per year.

Historically, the majority of C-BT water was delivered to agriculture. However, as the northern Colorado Front Range has urbanized, half of the deliveries are now made to municipal and industrial users. The year-round nature of municipal water use has caused Northern Water to look at adding redundancy to its delivery system. One such place was at Carter Lake Reservoir. The original outlet relies on two parallel sets of gates that discharge to a tunnel midway under the dam. Until 2008, this was the only outlet. Therefore, if any maintenance is required on the gates, the entire system had to be taken down, thus leaving downstream users without water. In 2008 Northern Water in cooperation with Reclamation installed a new redundant outlet. The new system consists of a multi-level outlet tower, 800 feet of 72-inch diameter tunnel, 200 feet of

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downstream piping, and an energy dissipating structure which discharges in the St. Vrain Supply Canal. The general layout of the site is shown on Figure 2.

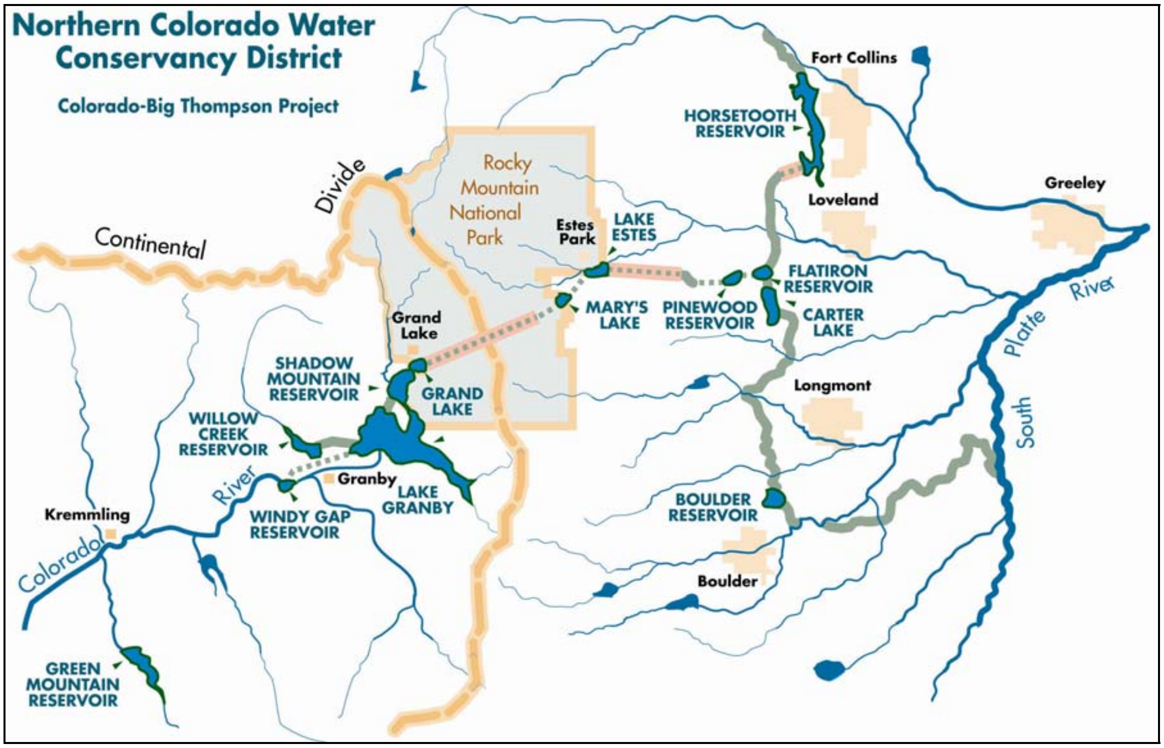

Figure 1. Colorado-Big Thompson Map

The new system has the capability of flowing at 250 cfs which can meet the entire downstream needs for all but the peak summer flow months. This new outlet enabled the potential of a new hydropower facility that could be constructed adjacent to the flow control structure.

HYDROPOWER POTENTIAL AND PROJECT SIZING

The planning of the Project involved a balancing of the project size versus the frequency in which it would be used. The Project could be sized to accommodate the largest water delivery that might be made but the conduits and turbine(s) would have to be very large, and therefore very expensive. Conversely, the Project could be sized to run at full capacity all year, but at such a low flow, very little power would be produced.

Figure 2. Layout of Project Facilities

A flow rate of 250 cfs was selected based on a flow duration analysis of computer modeled outlet flows using present-day water ownership and historic demand patterns. This flow rate provides redundancy to the existing outlet for approximately nine months out of the year. When the bypass outlet flow rate is exceeded, the remainder of the flow is delivered through the original outlet. Table 1 shows the average head and flow for the Project. Figures 3 and 4 illustrate the flow and head duration curves respectively.

Table 1. Project Flow and Head

Month	Average Carter Lake End of Month Elevation (feet)	Average Discharge (cfs)
January	5,729	32
February	5,740	40
March	5,748	39
April	5,751	87
May	5,749	151
June	5,746	245
July	5,729	381
August	5,709	400
September	5,699	242
October	5,699	123
November	5,708	33
December	5,718	30

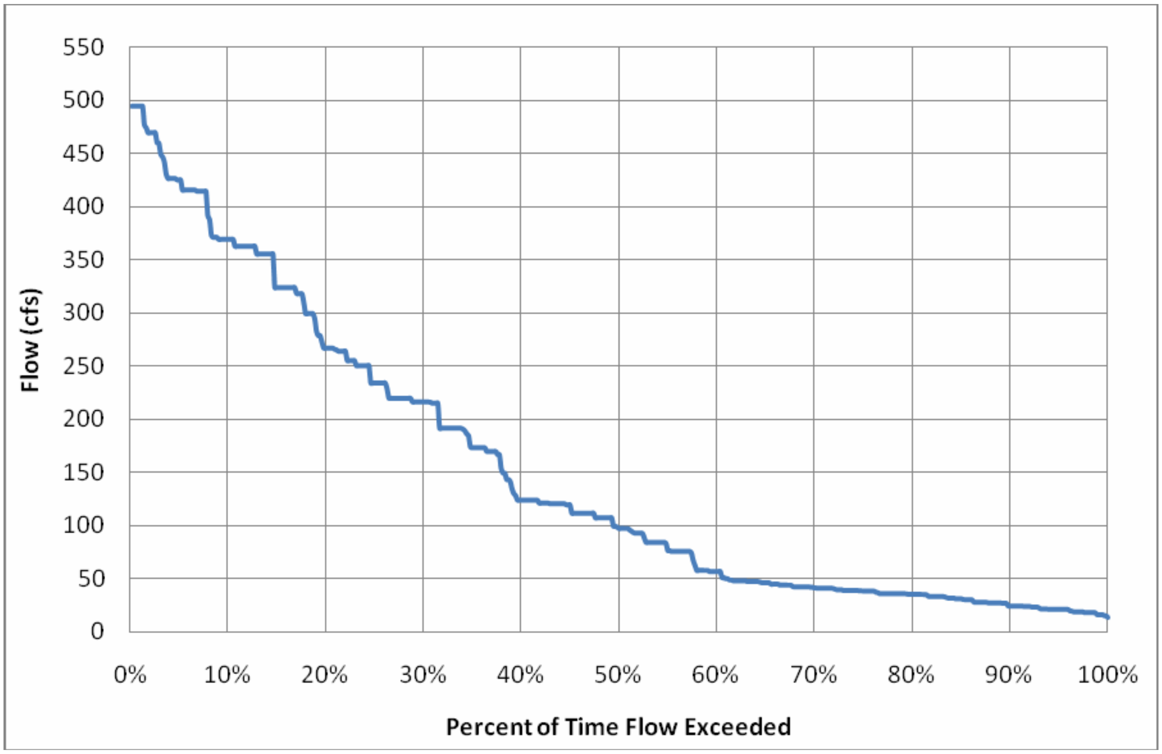

Figure 3. Flow Duration Curve

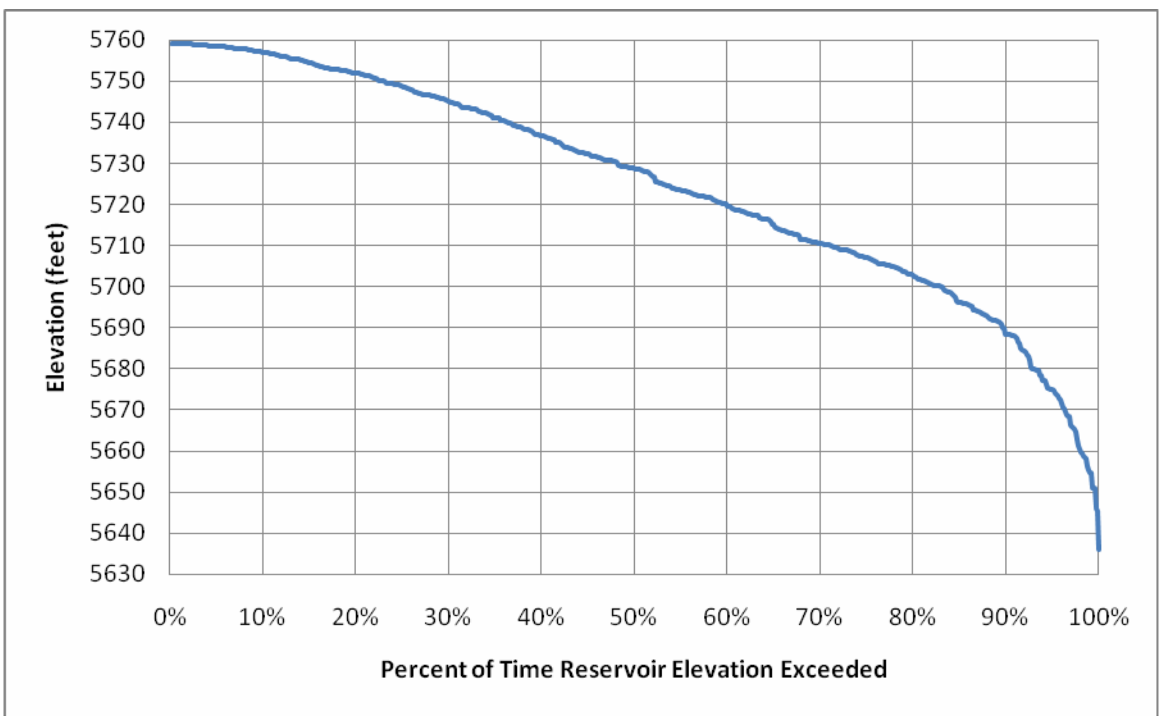

Figure 4. Reservoir Elevation Duration Curve

PROJECT FACILITIES AND LAYOUT

The project design engineer - Black & Veatch (B&V) - conducted an initial feasibility study of the Project to determine the optimum turbine type and configuration[2]. Three primary configurations were modeled: a single turbine; one smaller and one larger turbine; and two same-size turbines. A Francis runner is able to operate down to approximately 40 percent of design flow. A single turbine was the least expensive alternative, but also had the least amount of energy output since flows were only sufficient to operate the plant during the irrigation season. A two-turbine configuration with a small and large turbine covered the greatest range of flow and had the largest energy output. However, because the manufacturer would have to build two different-size turbines, the cost for this option was the greatest. The optimum balance was using two same-size turbines. There is also maintenance advantage to having two identical turbine/generator systems. Table 2 shows the results of this analysis based upon multiple turbine supplier quotations.

Table 2. Turbine Analysis Summary

Alternative	Avg. Annual Projected Generation (MWhrs)	Capital Cost
Single 250 cfs Turbine	7,000	\$5,580,000 to \$6,645,000
One 83 cfs and one 167 cfs Turbine	7,900	\$6,171,400 to \$7,726,000
Two 125 cfs Turbines	7,700	\$6,054,000 to \$6,094,000

A variety of turbine layout configurations were evaluated for the Project by B&V. The principal decision was between using a horizontal versus vertical layout. The price for either layout was nearly the same. While the vertical layout would result in a smaller building footprint, the building would need to be taller in order to accommodate removal of the vertical generator with the bridge crane. The horizontal configuration allows for greater ease of inspection and maintenance of the turbine and wicket gate assembly by being able to remove the draft tube. It also does not require an overhung generator thrust bearing which is required in the vertical configuration. For these reasons, a horizontal configuration was selected. Based on input from the turbine supplier Gilkes, a similar layout from a recent installation in Scotland was used to further refine the building footprint. A picture of this installation is shown in Figure 5. The Carter Lake layout is shown in Figure 6.

[2] Carter Lake Hydroelectric Project Alternatives Analysis Report, Prepared for Northern Colorado Water Conservancy District by Black & Veatch Corporation, 2009.

Figure 5. Similar Off-Set Installation by Gilkes

The powerhouse will consist of the following equipment:

- Two 1,300 kw turbine generators
- Two 48-inch butterfly valves with hydraulic controls and counterweight shut-off
- Two hydraulic power units to control the wicket gates and butterfly valves
- Emergency backup battery power
- Electrical controls
- Switch gear
- A 20-ton bridge crane

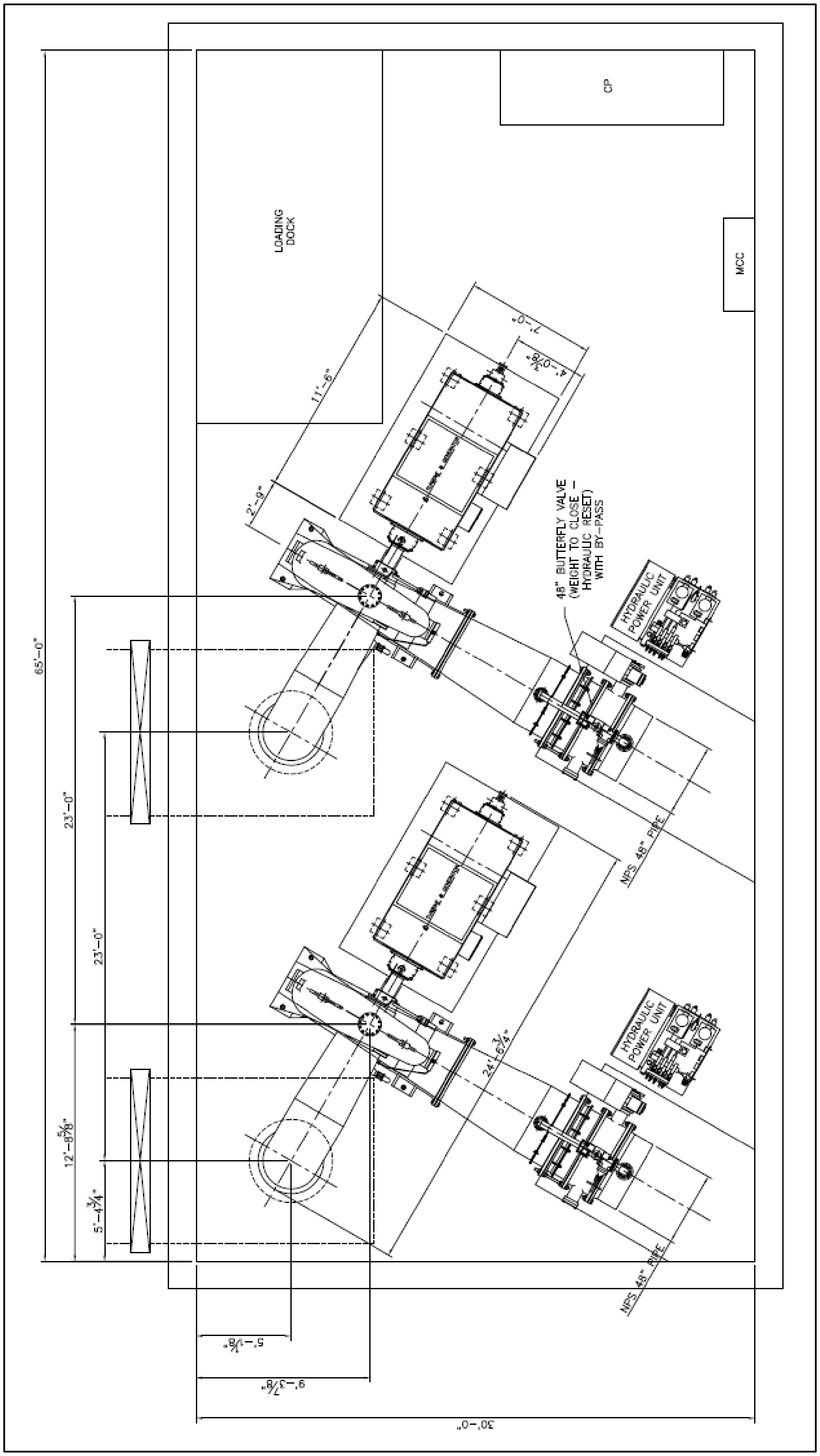

Figure 6. Proposed Powerhouse Layout

In addition to this equipment, the Project will also require approximately 200 feet of 72-inch penstock, a connection to the existing St. Vrain Supply Canal, and 600 feet of new power transmission line. Ease of maintenance was considered throughout the design process. All of the equipment can be accessed with the bridge crane. Furthermore, if in the future a piece of equipment needs to be shipped off-site, a lay-down area is provided where the equipment can be loaded from the bridge crane onto a truck.

Consideration was made for the generating voltage from the units with the choice being either 480 volts or 4,160 volts. In the case of 480 volt, the generator is smaller and less expensive. However, the amperage is considerably higher and therefore takes larger conductors to the electrical panel and switchgear. An evaluation of the total system found a slight cost advantage for the 480 volt. The primary benefit of the 480 volt was the ease of maintenance and lower cost replacement parts for the lower voltage. The adjoining transformer will increase the 480 volt power to the 12.47kV line voltage.

RECLAMATION LEASE OF POWER PRIVILEGE PROCESS

The Great Plains Region of Reclamation has entered into a Memorandum of Understanding (MOU) with the Federal Energy Regulatory Commission (FERC) to replace FERC's normal licensing procedure with a Lease of Power Privilege (LOPP) process. The LOPP process is in place for Reclamation projects such as C-BT where power was included in the authorizing legislation. In most ways LOPP is patterned after the FERC licensing process and is summarized as follows:

1. The applicant (Northern Water) requests that Reclamation initiate the LOPP process.

2. Reclamation advertises in the Federal Register for proposals to develop power at the requested facility (Carter Lake Reservoir).

3. Proposals describing the project, how the work will be performed, and a benefit/cost analysis is prepared by the applicant.

4. Reclamation reviews the proposals in light of merits of the application and the preference standing of the applicant (Federal power users are given highest preference; next are project users; and finally private developers).

5. Reclamation grants a preliminary LOPP during which time the applicant can begin the environment assessment process, design, financing, and power purchase negotiations.

6. Reclamation grants the final LOPP.

7. The applicant pays to Reclamation approximately $5/Megawatt-hour for all power based on an equation developed by Reclamation.

In the specific case of the Carter Lake Hydropower project, Northern Water submitted its proposal to Reclamation in September, 2009. Northern's was the only submittal and a preliminary LOPP was granted in November, 2009. After selection Reclamation and Northern Water entered into a Memorandum of Understanding to allow for payment for Reclamation's review of plans and specification, preparation of necessary environmental documentation, and preparation of the LOPP.

The final LOPP was signed in February, 2011. There have been few issues raised by Reclamation as part of the process. An Environmental Assessment (EA) was prepared during the summer of 2010. However, because the site was already disturbed during the construction of the outlet bypass project, and because the operations at Carter Lake Reservoir would not change, the EA was minimal, resulting in a Finding of No Significant Impact by Reclamation.

Most of technical review by Reclamation relates to the potential for hydraulic transients and the potential impact on the outlet bypass, particularly during a unit trip (power failure). Since the outlet bypass was designed anticipating a hydropower facility, this review has been straight-forward.

POWER SALES AND REVENUE PRODUCTION

Early in the Project development process, Northern Water met with the local energy supplier, Poudre Valley REA (PVREA). Presently, PVREA receives all of its power from Tri-State Generation and Transmission. However, PVREA can self produce up to five percent of their own generation of which the Carter Hydro could be part.

In Colorado, a 2004 voter-approved ballot issue requires electrical utilities to obtain certain percentage of their power from renewable energy. The REAs must obtain 10 percent of their electrical power from renewable energy by 2020.[3] Renewables include wind, solar, biomass, geothermal, and small hydro. Qualifying hydro must be 10 MW or less.

The estimated gross revenue for the site is approximately $600,000 annually. From this, annual costs must be subtracted for the operation and maintenance costs of approximately $80,000, and the payment to Reclamation of LOPP of approximately $40,000. This results in a net revenue of $480,000. Over time as revenues accumulate, a turbine/generator rehabilitation fund will be established to cover any large-scale rehabilitation of the various systems.

It should be pointed out, the energy output from the Project can vary considerably depending on the outflow from Carter Lake resulting in a variable revenue. Though the average energy production is estimated to be 7,700 MWhrs, it can vary from as little as 6,000 MWhrs to as much as10,000 MWhrs. As will be explained in the Financing section, one year's debt service was included in the loan to cover revenue deficiencies, particularly in the earlier years of the loan.

[3] Database for Renewable Energy & Efficiency, 2010. http://www.ucsusa.org/assets/documents/clean_energy/colorado.pdf

TURBINE/GENERATOR PROCUREMENT

Northern Water chose to bid the turbine/generator package ahead of moving forward with the civil works portion of the Project. The rationale for this decision was that if the equipment cost more than the project economics would allow, the system could be re-configured or delayed. The turbine/generator package consists of the following items:

- Francis turbines (two)
- Generators (two)
- Hydraulic power units (two)
- Isolation butterfly valves (two)
- Electrical controls and internal switchgear

B&V prepared contract documents include contract conditions, specifications, preliminary site layout drawings, and electrical and instrumentation drawings. Northern Water first issued a prequalification request to potentially interested suppliers. Information requested included general company information, their ability to do the work, and references. Four companies sent submittals and all four were allowed to proceed with the proposal submission.

The bid proposal required not only cost information but also specific information about the turbine and proposed delivery schedule. All of the suppliers submitted satisfactory information about their turbines and their ability to deliver the equipment in a timely manner. Therefore the selection decision was made on the price. The equipment bids ranged from $4.2M to $5.8M. The engineer’s estimate was $3.2M. Gilbert Gilkes and Gordon LTD (Gilkes) of Kendal, England was selected as the supplier. Following selection, Northern Water and B&V met with Gilkes to determine ways of reducing the cost while maintaining the function and quality of the project. By reducing the generator voltage from 4160 volt to 480 volt, using more standard electrical equipment, and changing the orientation of the draft tube the equipment price was reduced to $3.5M.

PROJECT FINANCING AND ECONOMICS

The total cost of the Project is $6,200,000. To fund the Project, Northern Water reviewed several financing methods including the following:

1. Internal funding through project reserves

2. Federal no-interest Clean Renewable Energy Bonds (CREBs)

3. State of Colorado hydropower loan interest loan program ($2M maximum) through the Colorado Water Resources and Power Development Authority (Authority)

4. Revenue Bonds

5. A loan from Northern Water reserves to a newly formed hydropower enterprise

Northern Water elected not to fund the Project through a grant from its reserves and therefore pursued debt financing. An application for CREBs was submitted in 2009 but was not approved by the Federal government. Because the newly created enterprise was to be wholly reliant upon power sales revenue, and because the enterprise had no historic track record, the interest rate for revenue bonds was found to be high and would not allow for a feasible project. Therefore, a combination of financing with an Authority 20-year low interest loan of 2-percent for $2,000,000 and the balance of $4,200,000 being a loan from Northern Water Project improvement reserves with loan similar terms as the Authority loan.

As explained previously, the energy output from the Project can vary considerably from year to year. The test of financial feasibility was that the Project would be able to cover all of the costs during the finance period – O&M, LOPP, and loan repayment. As previously mentioned, the annual power output from the project can vary greatly from year to year. Therefore, a year's worth of debt service, or approximately $400,000 was added to the loan to take care of any revenue shortfalls in the early years. Over time, this variability is averaged out as the excess revenue forms a project reserve fund.

The cash-flow during the 20-year loan repayment period results in little accumulated net revenue as shown in Figure 7. However, after the 20 year loan repayment period, the net revenue is approximately $500,000 per year.

Over the 50-year economic life of the Project, the long-term results are as follows:

Present Value of Costs: $8,300,000

Present Value of Revenue: $13,400,000

Net Present Value of Project: $5,100,000

Benefit/Cost Ratio: 1.6

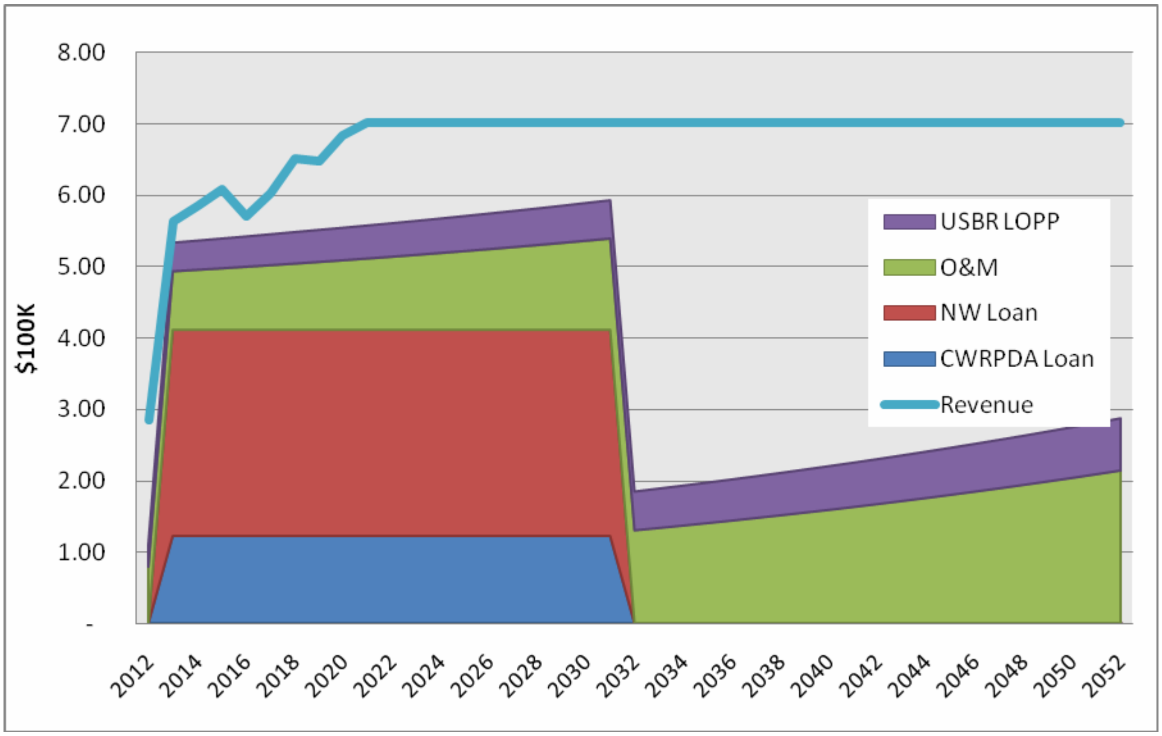

Figure 7. Projected Carter Hydro Cash-flow

In 2007-2008 Northern Water paid for the design and construction of the Carter Lake Outlet Bypass at a cost of $12 million. By adding the hydroelectric Project to this new outlet, approximately half of the cost will be recovered and returned to Northern Water's water system improvement reserve.

CIRCULAR PLANNING PROCESS

Ideally, the planning of this Project could occur in a linear process with feasibility planning occurring first, followed by permitting, design, power purchase contracting, financing, and construction. This ideal may work where the annual power sales far exceed the cost of paying for the project. However, since hydropower is very capital intensive, the linear process becomes more of an interconnected web. Ultimately, project viability depends on the project being able to produce enough power, at a certain power sale rate, to pay for the loan payment and operations and maintenance costs. In the case of this Project, the goal was to systematically lock parts of the planning web.

Flow rate, and therefore turbine size and power production, was the first item to be locked. This decision was made early on based on the variables such as the outlet capacity, projected hydrology, and economies of scale with the turbine/generator equipment. The selection of turbine size allowed for a layout to be prepared which then allowed for the permitting to take place. A LOPP was able to then be secured with Reclamation based on the proposed turbine size.

The cost of the Project next needed to be determined. While civil works can be estimated with reasonable certainty, the turbine/generator equipment cost can be highly variable. In the case of this Project, bids ranged from 30 to 80 percent higher than the initial quotes. As previously explained, discussions with the manufacturer resulted in a reduction in costs to an acceptable level.

The next item to be locked in place was the financing terms, both rate and payback period. Loan repayment became the ultimate driver in the ability of the Project to move forward. Knowing all of cost variables allowed the power purchase contract to be negotiated. However, none of the previous items could be finalized until the power purchase contract was in place.

Finally, knowing the Project cost and financing terms, a power purchase contract was able to be negotiated with the power purchaser. With this part of the puzzle in place, the loans could be finalized, the equipment ordered, and the civil design could commence.

LESSONS LEARNED

At time of this paper, the loans have been finalized, the equipment ordered, and the civil design is underway. Through this process, a number of important lessons have been learned.

1) Hydroelectric projects are expensive. The cost per KW for this Project is $2,300. That includes only a short piece of penstock and about 600 feet of new power transmission line. While the “fuel” is free, hydropower is very capital intensive.

2) Estimate conservatively the amount of power to be produced annually, particularly in evaluating the financing the project.

3) Talk to the power purchaser early and often. The project feasibility is ultimately dependent on a positive revenue stream and clearly knowing the purchaser’s expectations is important.

4) Federal regulatory processes take a considerable amount of time. It took approximately a year between being selected for Reclamation’s LOPP and obtaining the final LOPP. If dealing with Reclamation, at least six months should be allotted for the environmental process.

LOW HEAD HYDRO POTENTIAL IN COLORADO

Lindsay George, PhD, PE [1]
Daniel Zimmerle [2]

ABSTRACT

Over 3 million acres of land are irrigated in Colorado using existing pipelines and canals. This existing infrastructure holds the potential to produce low head hydroelectric power, but how much? An overall survey is currently being conducted to determine the aggregate potential for small hydroelectric generation within Colorado's irrigation systems.

As a part of this study, low head turbine technologies have been researched and individual sites studied. A summary of available generation and interconnect technologies applicable to these sites is presented. Also, a detailed investigation into eight sites on a typical irrigation system is presented to illuminate implementation potential and challenges. This system contains elements representative of typical irrigation infrastructure in Colorado, and can be used as an example for other systems in the western states.

INTRODUCTION AND BACKGROUND

A research study is currently underway to quantify the potential of Colorado's irrigation infrastructure to produce low head hydropower. This study is funded through the Colorado Department of Agriculture, Advancing Colorado's Renewable Energy program. The mission of the program is to "promote agricultural energy-related projects" and such projects must, in some way, benefit Colorado's agriculture industry. This study is aimed at identifying the potential of low head hydropower in Colorado in order to help agricultural producers understand the opportunities that exist.

This research focuses on low head turbines for two main reasons 1) the recent advances in the technology, and 2) the suitability of low head turbines for use in irrigation canals in Colorado. There is very little research available on low head hydro turbines and most of what is available focuses on very large turbines (Kpordze and Warnick, 1982, Hatch Energy, 2008), or very small turbines for developing countries (Williams, et. Al, 2000).

This research focuses on the smaller turbines that would be appropriate in Colorado's irrigation canals. Colorado has approximately 250 irrigation canals that have a decreed capacity over 100 cfs. The decreed capacity is the maximum amount of water that the canal is legally allowed to divert when available. This amount may be much higher than the amount of water diverted on a regular basis. The following chart shows the

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distribution of sizes of the larger canals in Colorado. More than half of the larger canals have a decreed capacity of less than 250 cfs (State of Colorado, 2010).

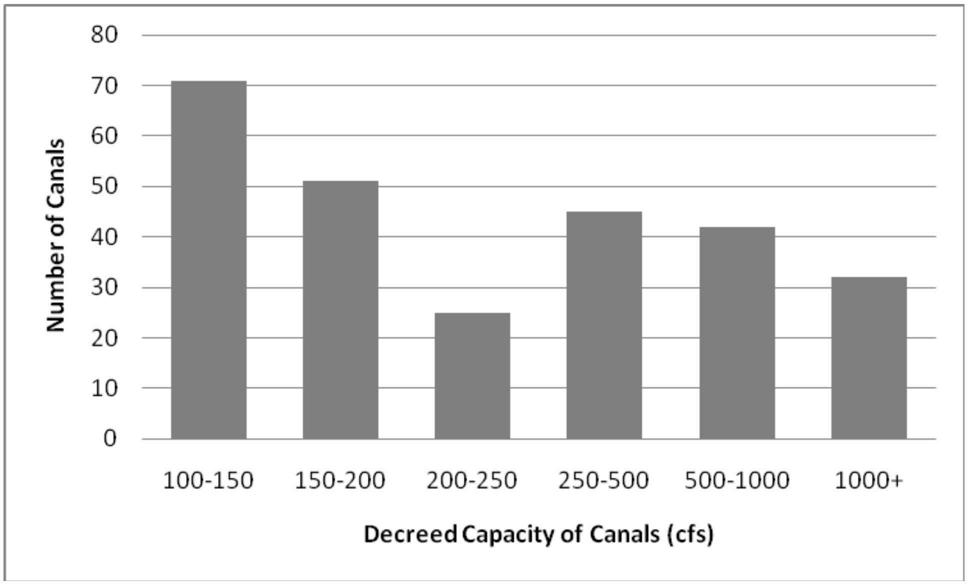

Figure 1. Canals in Colorado

There are three main objectives of this research study; (1) Identify turbine technologies, (2) Investigate two project canals to match technologies with typical site conditions, and (3) Quantify the potential of Colorado's irrigation infrastructure to produce low head hydropower. This paper presents the results of the first two objectives. Surveys were sent to 250 irrigation companies and canal owners to identify Colorado sites that have hydropower potential. The data from these surveys are currently being collected and analyzed; results are expected in the Spring of 2011.

AVAILABLE LOW HEAD TURBINES

Low head turbine technologies were researched through the internet and by contacting turbine manufacturers across the world, with a focus on North America. The list of available turbines shown in Table 1 is not exhaustive, but represents the range of low head turbines currently available. The range of operating conditions for most turbines was found and is presented in the following table. A turbine selection chart, using all of the available turbines, was published in an interim report for this study (Applegate Group, 2010). We urge the reader to visit the websites for each turbine listed below for more information. The turbine selection chart, photographs and a description of each turbine is available in the interim report (Applegate Group, 2010)

Table 1. Available Low Head Turbines

Manufacturer	Description	Head (ft)			Flow (cfs)			Website
Impulse								
Ossberger	Cross Flow	6.6	-	656	1.4	-	423	1
Gilkes	Cross Flow	23	-	197	0.7	-	28	2
Reaction								
Energy Systems and Design	Axial Flow Propeller	2	-	10	0.7	-	2.2	3
Power Pal	Propeller	4.9	-	4.9	1.25	-	4.25	4
Canyon Hydro	Kaplan	30	-	50	100	-	400	5
Toshiba International	Hydro-eKIDS Type S	10	-	50	3.5	-	10.6	6
Toshiba International	Hydro-eKIDS Type M	6.5	-	50	3.5	-	49.5	6
Toshiba International	Hydro-eKIDS Type L	6.5	-	50	35	-	124	6
Very Low Head Turbines	VLH	4.6	-	10.5	367	-	1095	7
Mavel	MT3	5	-	20	5	-	14	8
Mavel	MT5	5	-	20	25	-	50	8
Mavel	MT10	7	-	16	70	-	175	8
Ossberger	Cross Flow	3	-	40	9	-	1412	1
Gilkes	Kaplan	7	-	70				2
Natel America	SLH-10	3.3	-	19.7	15	-	37	9
Natel America	SLH-50	3.3	-	19.7	63	-	155	9
Natel America	SLH-100	3.3	-	19.7	127	-	310	9
Natel America	SLH-200	3.3	-	19.7	253	-	620	9
Natel America	SLH-500	3.3	-	19.7	633	-	1550	9
Voith Hydro	Kaplan	10	-	131.2	176	-	7063	10
Andritz	Belt Drive Bulb	6.6	-	15.6	212	-	883	11
Andritz	Bevel Gear Bulb	6.6	-	39.4	80	-	1625	11
Andritz	Axial	19.7	-	98.4	80	-	2295	11
Andritz	Kaplan	6.6	-	39.4	141	-	2119	11
Andritz	eco-bulb	6.6	-	49.2	529	-	3531	11
Screw Type								
HydroCoil Power	Screw Type	13	-	66	1.8	-	1.8	12
Ritz-Atro	Hydrodynamic Screw	3	-	30	10	-	175	13
Waterwheel								
HydroWatt	Overshot Waterwheel	8	-	32	3.5	-	88	14
HydroWatt	Breastshot Waterwheel	3	-	10	18	-	250	14
Hydrokenetic								
Alternative Hydro Solutions	Hydrokinetic	*based on velocity						15
Hydrovolts	Hydrokinetic	*based on velocity						16
Do It Yourself								
Zotloeterer	Vortex Power Plant							17
Elephant Butte Irrigation Company	Propeller Type	6	-	10	30	-	60	18

[1] www.hts-inc.com/ossbergerturbines.html
[2] http://www.gilkes.com/
[3] http://www.microhydropower.com/
[4] www.powerpal.com
[5] www.canyonhydro.com
[6] http://www.tic.toshiba.com.au/hydro-ekids__8482_/
[7] http://www.vlh-turbine.com/EN/php/Accueil.php
[8] www.mavel.cz
[9] www.natelenergy.com
[10] www.us.voithhydro.com/vh_en_pas_small_hydro.htm
[11] www.andritz.com
[12] www.hydrocoilpower.com
[13] http://www.ritz-atro.de/2006/index_neu.html
[14] http://www.hydrowatt.de/sites/english/home.html
[15] www.althydrosolutions.com
[16] www.hydrovolts.com
[17] www.zotloeterer.com
[18] http://www.ebid-nm.org/

PROJECT CANAL

The Grand Valley Irrigation was chosen as a large representative canal in Colorado. The canal was visited and locations were identified as possible low head hydro sites. The head and flow rate available at each site was measured, and the turbines that fit the available head and flow were evaluated. Pros and cons to each turbine are presented, along with necessary modifications to the existing infrastructure.

Grand Valley Irrigation Canal

The Grand Valley Irrigation Canal is located in the Grand Valley on the Western Slope of Colorado. The headgate diverts water from the Colorado River near the town of Palisade. The canal extends westward approximately 30 miles to Loma and consists of almost 100 miles of canals. The canal is owned and operated by the Grand Valley Irrigation Company and irrigates approximately 40,000 acres. Approximately 50% of the shareholders are involved in agriculture.The canal is decreed for 640 cfs and annually diverts over 250,000 ac-ft. The canal is generally in operation for over 200 days of the year.

Eight sites have been identified in this system that may hold the potential to produce low head hydropower. These eight sites were plotted on the turbine selection charts to identify which turbines are suitable for each site based on available head and flow. Each site and turbine is discussed in the following sections.

All sites are located in Xcel Energy’s service area, except for 13 Rd Loma, which is serviced by Grand Valley Power. Both utilities have programs to support small power providers, but different interconnection requirements, size cutoffs and other factors. These issues are discussed following the discussion of each site’s characteristics.

Site 1 Oldham's Check

The Oldham's Check consists of a concrete lined trapezoidal section that raises the water surface approximately 6 inches. The average width of this section is 30 feet and the depth of water through the section is approximately 3.5 feet. The three phase power line is located just adjacent to this check. Figure 2 shows this check with water out of the canal. The amount of head available at this site is not sufficient for a low head turbine. Most of the turbines on the market require at least 5 feet of head differential between the upstream and the downstream water surface.

Figure 2. Photograph of Oldham's Check

The concrete lined section does constrict the flow through the canal and increase the velocity of the water. We estimate that the velocity through this section is 6 ft/sec which may make this site appropriate for a hydrokinetic turbine. If sufficient velocity is available, the feasibility of a hydrokinetic turbine is dependent on the geometry of the section. At this site it would be possible to install multiple vertical axis turbines, like a Darrieus Water Turbine. Two 10 foot diameter turbines or three 8 foot diameter turbines with a depth of 3 feet could be accommodated at this site. This site could produce between 4 and 6 kW of electricity, resulting in approximately 20,000 kWhrs annually.

These turbines would be suspended from a bridge spanning the canal. The generator would be located on the vertical axis, above the water. This will create a location for floating debris to collect. Deflectors could be installed, and regular cleaning would need to be performed when floating debris is excessive.

A second type of hydrokinetic turbine that may be appropriate at this site is the Hydrovolts turbine. This turbine is not in commercial production yet, but the company would be willing to discuss producing a custom turbine for this site. This turbine would be anchored to each side of the canal, and would not require a bridge spanning the canal.

Site 2 Gates Check

The Gates check consists of a concrete structure that spans the width of the canal with a raised concrete floor (Figure 3). A pedestrian bridge is supported by two concrete piers located in the channel. The structure is 29.5 feet wide at the narrowest location. Water passes over the structure at a depth of approximately 2.5 feet. There is about 2 feet of fall between either side of the structure. The velocity of the water passing over the structure is estimated to be 8 ft/sec.

Figure 3. Photograph of Gates Check

This site also has too little head to make it feasible for a low head turbine. Again, vertical axis hydrokinetic turbines could be appropriate for this site. One advantage to this site is the structure above the canal that could be used to mount the turbines. If this pedestrian bridge is used for public traffic, measures would have to be taken to secure the turbine from vandalism or damage. If a turbine is installed in each bay of the structure, trash accumulation may be a problem. It would be possible to leave one bay empty and deflect floating debris away from the turbines and through the empty bay. These turbines can also be easily removed, if floating debris is present for only a short time of the year.

It would be possible to install three 8 foot diameter, 2 foot deep turbines in this structure with very little infrastructure modification. Each turbine could produce about 2.5 kW, or a total of 38,000 kWhrs of electricity annually. The velocity of this site makes it more economical to install the hydrokinetic turbines. The same turbines can produce almost twice as much electricity at this site compared to the Oldham's check.

Site 3 The Falls

The Falls is a concrete lined section of canal that drops about 3.5 feet (Figure 4). The lining is irregular, but generally trapezoidal. The section is approximately 19 feet wide, and the water travels through the section 1.75 feet deep. This results in a very high velocity of approximately 17 ft/sec. The head and flow of this site falls within the range of the Natel Energy Hydroengine, specifically the SLH-500. This is the largest standard model that Natel offers, and is required at this site because of the relatively high flow. This turbine would produce about 125 kW, or 650,000 kWhrs of electricity annually. It would also be possible to install one of the smaller Natel models that would not utilize the entire flow.

Figure 4. Photograph of the Falls

The Natel turbine is best installed at a site where the entire drop occurs over a very short distance. In this case the drop occurs over about 275 feet. To install this type of turbine, the drop would need to be consolidated at the upper end of the existing drop. A structure could be installed across the canal, and the remaining slope excavated to a lower elevation. This type of modification to the existing infrastructure would only be economical if this part of the canal was being reconstructed for other reasons. The concrete in this drop appears to be in good condition and is not in need of replacement in the near future. A hydrokinetic turbine is not considered at this site because of the shallow depth of water through the structure.

Site 4 The Dividers

The Dividers is a rectangular concrete-lined chute that discharges into a shotcrete lined stilling basin, as shown in Figure 5. There is approximately 13 feet of head and 200 cfs available. The chute is about 125 feet long and flow is controlled with two gates on the upstream end. This structure is located at a split in the canal, as shown in Figure 6. The slide gate and radial gate control the flow into the concrete chute, with the remaining water flowing down the main canal. The turbine selection chart showed that this site is suitable for five different types of low head turbines.

Figure 5. Photograph of the Dividers

The Mavel, Natel and Ossberger turbines require similar infrastructure to operate. There are several configurations that would be possible for the installation of these turbines. Each turbine could be located on the upstream end of the chute with the head obtained using a draft tube extending to the bottom of the chute. Alternatively, the turbine could be located at the lower end of the chute with the head delivered through a pressurized pipe upstream of the turbine. Either configuration would require significant alteration to the existing infrastructure. Likely the least expensive option would be to install the turbine at the top of the drop and use a draft tube, placed in the existing channel to create the head differential. This draft tube could then be buried or be left exposed. Additionally, a bypass would be required if the turbine needs to be maintained or removed for any reason. This could be accomplished using the existing canal, or a parallel pipe, to ensure continued water deliveries downstream.

Figure 6. Photograph of the gate upstream of the chute

The SLH-100 model offered by Natel Energy would be appropriate for this site, and produce about 185 kW or 960,000 kWhrs annually. Mavel’s TM10 is designed to operate at a maximum of 175 cfs, and would produce between 150 and 170 kW or 830,000 kWhrs. In this case the Natel turbine appears to be more efficient. The choice of turbine would be based on comparing the installed cost and related infrastructure improvements.

This site was historically the site of a waterwheel that was used to lift water. The passing water would power the water wheel and carry a small portion up to the top of the bank. This site does fall within the range of conditions required for a modern water wheel. Most likely an overshot water wheel would be appropriate because of the relatively high head available. Water would enter the wheel at the top and fall around the wheel. This would require the wheel to be entirely below the elevation of the incoming water. At this site the wheel would essentially need to be below the ground surface. The extensive alterations to the existing infrastructure would likely not be balanced by the relatively low efficiency water wheel. Although this site did historically support a water wheel, it may not be appropriate at this time.

Finally, this site is a candidate for a hydrodynamic screw, based on the head and flow available. Also the existing infrastructure suggests that it may be a good site for this type of turbine. A hydrodynamic screw is placed inside of a sloped concrete chute with the turbine located at the upstream end. These turbines are placed on a slope between 22 and 40 degrees. This site has a slope of only 5.4 degrees. Also, the screw for this site would have a diameter of approximately 11.3 feet, and the existing width of the chute is only 6 feet. For this turbine to be appropriate at this site, a second chute would need to be installed parallel to the existing chute. This would allow for the appropriate geometry to be constructed and a bypass would exist. The topography of the surrounding land suggests that this may be a possibility. This site could support a 170 kW hydrodynamic screw, with significant modification to the existing infrastructure.

Site 5 First Street Chute

The First Street Chute has similar infrastructure as the Dividers. This is a concrete lined, rectangular chute, with 38.1 feet of head and 167 cfs available (Figure 7). The drop occurs over about 200 feet. According to GVIC, this site was originally intended for hydroelectric development. This is the most fall seen at one structure over the entire Grand Valley Irrigation Canal. The relatively high amount of head available makes several more traditional turbines appropriate at this site. The turbine selection chart indicates that a Kaplan turbine would be appropriate.

Figure 7. Photograph of the First Street Chute

A Kaplan turbine would be installed at or near the end of this chute, with the entire length of the chute put into a pressurized pipe. The flow available at this site (167 cfs) is at the low end of the range for the larger turbines, the Andritz and Voith Kaplans. This generally means that the turbine that will fit these conditions, could also handle a lot more flow, and therefore may be “oversized” for the site. The Canyon Hydro Kaplan may be more suited for this site, as the head and flow available is near the center of the range. This site is very similar to a recent installation by Canyon Hydro near Logan, Utah. That site had 30 feet of net head and 143 cfs available. The Canyon Hydro turbine could produce approximately 450 kW, or 2,300,000 kWhrs annually.

Site 6 18.5 Road Chute

This site has a 100 foot long concrete lined chute that carries 30 cfs and falls about 11 feet (Figure 8). The turbine selection chart shows that four turbines could be appropriate for these conditions, the Mavel, Voith Ecoflow, Natel’s Hydroengine, or a Hydrowatt waterwheel. This site is similar to the other chutes presented here, and the 100 feet of length would need to be piped to install the Mavel, Ecoflow or Hydroengine. The waterwheel would require significant modification to this site, and may not be appropriate for the conditions. There is a three phase power line that follows the road and is adjacent to the turbine location.

Figure 8. Photograph of the 18.5 Road Chute

Any of these three turbines would produce about 25 kW or 130,000 kWhrs annually. This site could easily and inexpensively be developed if the chute was enclosed in a pressurized pipe. If this chute was slated for replacement, then would be a good time to consider adding hydropower.

Site 7 13 Road Drop - Loma

This drop occurs at the very western end of the GVIC system. At this point in the canal there is 25 cfs left flowing. This site is a 360 foot long concrete lined drop, that falls about 30 feet. This site is located in a rural area of Loma, near Interstate 70, see Figure 9. There is a single phase power line near the turbine location to serve the lighting at the exit.

Figure 9. Photograph of the 13 Road Drop

The conditions at this site fall within the range of three turbines, the Toshiba eKids series, the Ossberger cross flow and the Ritz-Atro hydrodynamic screw turbine. The site conditions are not conducive to the hydrodynamic screw option, because of the long, low angled slope. The Toshiba eKids and the Ossberger cross flow turbine would both require that the entire length of the drop be piped and the turbine located at the base of the drop. The Toshiba e-Kids (Type M) turbine or the Ossberger turbine could produce about 50 kW, or 260,000 kWhrs annually. A comparison of turbine cost and related infrastructure would determine which turbine is more economical for this site. This site would become more favorable if the open ditch was converted into a pipeline for another reason.

Electrical Interconnect

With the exception of the 13 Rd Drop in Loma, all sites are located within the Xcel Energy service area. Therefore, emphasis will be placed upon interconnection to the Xcel system. Since the investigation is ongoing, preliminary information is provided here.

Xcel divides power small generators into three size classes:

- **0-10KW –** Power purchase is structured as a net-metering agreement, which nets out generation with other customer loads. In the event that production exceeds consumption in a given month, no payment is issued but power production is credited against consumption in following months. This class would apply to the Oldham and Gate's check structures, and could be serviced by single- or three-phase power.

- **10-100KW –** Xcel provides a standardized tariff with rates that include a capacity payment (currently $7.06/KW/month) and an energy payment (currently $0.023/KWh). For the estimated annual operational period of approximately 6 months and 5,000 hours, combined capacity and energy revenue nets to $0.031/KWh. Capacity payments required uninterrupted operation during a billing month. Projects in this size range require access to three-phase power.

- **> 100KW –** Power producers must bid into the Xcel's *Integrated Resource Planning* process and acquire a specific power purchase agreement. Typical rates for such agreements are under investigation. Projects in this size range require three-phase power and may also require electrical studies to determine impact on the local power distribution.

Approximate revenue for each site is included in the Table 2, below. For sites larger than 100 KW, revenue was approximated using the small provider (10-100KW) tariff structure. The value of renewable energy credits, if any, could not be estimated at this time and is not included in estimated revenue. Since sites below 10KW in size operate under "net metering" rules, revenue depends upon the project owner's load profile and load location.

In all cases, the project developer must pay for infrastructure upgrades required to connect the generating facility to the utility network. A key cost for such upgrades includes extension of the distribution lines to the generation location. Fortunately, many of the sites are in built-up areas near Grand Junction, and thus are close to distribution lines. In addition, larger sites must provide utility-approved protection equipment and a lockable disconnect accessible by the utility. Total installation costs will be estimated for selected sites during the remainder of this project.

Summary of Sites

Basically two types of sites were looked at in the Grand Valley Irrigation Canal; a relatively short check type structure that consists of a raised floor or narrowed width, and a concrete lined chute. While only two types of structures were investigated, it was shown that different turbines were appropriate for each site. Factors that contribute to a turbine's applicability are shown to be head, flow, geometry of the existing infrastructure, water velocity, water depth, and age of the infrastructure. The feasibility of these turbines was not analyzed from an economic standpoint. Although, general observations on the amount of modifications required were used to recommend the most appropriate turbine.

Table 2. Summary of GVIC sites

Name	Type	Possible Turbines	Power	Est. Gross Annual Revenue (K$)
Oldham's Check	Short Check	Vertical Axis Hydrokinetic	4 kW	Net Meter
Gate's Check	Short Check	Vertical Axis Hydrokinetic	7.5 kW	Net Meter
The Falls	Short Check	Natel Hydroengine	125 kW	16.0
The Dividers	Concrete Chute	Natel Hydroengine, Mavel Turbine, Ossberger Cross Flow turbine	185 kW	~23.7 PPA Req.
First Street Chute	Concrete Chute	Kaplan Turbine	450 kW	~57.6 PPA Req.
18.5 Road Chute	Concrete Chute	Mavel Turbine, Voith Ecoflow, Natel Hydroengine	25 kW	3.2
13 Road Drop	Concrete Chute	Toshiba e-Kids Turbine, Ossberger Cross Flow Turbine	50 kW	TBD

Notes:
- Net Meter – site size dictates that implementation use net metering
- PPA Req – site size requires a negotiated power purchase agreement
- Estimated revenue is based upon preliminary tariff information and should not be utilized for planning or decision purposes.

From this summary the recommendations to the irrigation company from a development standpoint would be to investigate the First Street Chute in more detail, as it appears to require little modification to existing infrastructure and has the potential to produce the most amount of revenue. Also, it is recommended to consider the other hydropower options if and when any of these structures require maintenance or replacement.

CONCLUSIONS

This project canal illustrates that a turbine selection chart can aid in the initial selection of turbines that are appropriate for the site conditions available, but the site geometry and existing infrastructure will determine which turbine can be installed effectively. Potential revenue is highly dependent upon local site conditions, which will be explored more fully during the balance of the project. In addition, non-traditional revenue sources will also be investigated, including renewable energy credits and participating in seasonal or ancillary power markets.

REFERENCES

Applegate Group, Inc and Colorado State University, “Interim Report - Summary of Available Low Head Turbine Technologies”, Colorado Department of Agriculture, ACRE Program, September 2010.

Hatch Energy, “Low Head Hydro Market Assessment”, National Resources Canada, Hydralic Energy Group, Ottawa, Ontario, March 2008.

Kpordze, C. S. K. and C. C. Warnick “Experience Curves for Feasibility Studies and Planning of Modern Low-Head Hydro Turbines”, Research Technical Completion Report A-077-IDA, Idaho Water and Energy Resources Institute, University of Idaho, Moscow, Idaho, December 1982.

State of Colorado, *Colorado Decision Support System*, December 2010 <http://cdss.state.co.us/>

Williams, A. A., Upadhyay, D. R., Demetriades, G. M. and N. P. A. Smith, “Low Head Pico Hydropower: A Review of Available Turbine Technologies”, World Renewable Energy Congress VI, Renewables: The Energy for the 21st Century World Renewable Energy Congress VI 1–7 July 2000 Brighton, UK 2000, Pages 1475-1480.

Public Service Company of Colorado, “Small Power Production and Cogeneration Facility Policy,” Colorado PUC Filings, C96-901, September 23, 1996, C05-0597, May 23, 2006, and C84-635, December 1, 2009.

OPTIMAL PLACEMENT OF HYDROELECTRIC DAMS IN CANALS

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ABSTRACT

As worldwide hydroelectric demand increases, the need for sustainable power sources increases as well. Small hydroelectric dams (or micro-dams) have become recognized as a means of achieving sustainable power with minimum disruption to the environment. Moreover, installation of these dams on existing canals provides sustainable power with little to no adverse environmental effects. Because of the costs associated with installing the micro-dams, a methodology to determine the best placement of the dams is desired. Optimal placement of these micro-dams can result in higher power output, lower construction costs, and better control of the water distribution system. This work utilizes the dynamic programming method previously developed for retention and detention basin location optimization to identify ideal locations for new dam installation on proposed or existing canals. The algorithm considers a number of constraints, including the costs of obtaining land, available hydraulic head, and installation costs. The algorithm also determines the optimal location of micro-dams as a function of return period on the investment. An example application of the model to a hypothetical canal is included.

INTRODUCTION

Worldwide demand for electrical power continues to rise. Indeed, there are currently more than 800,000 dams in operation, generating enough hydroelectricity to supply nearly one-fifth of the world's energy. Unfortunately, although hydroelectric dams are sustainable sources of energy, they can damage riparian systems. Jacquot (2009) reports that hydroelectric dams are currently adversely affecting more than half of the world's large water systems. These adverse effects include ecological damage, environmental changes, and water quality reduction (Goodwin et al., 2000).

An alternative to the large scale hydroelectric dams placed on natural rivers is to install smaller, "micro-hydroelectric dams" on engineered waterways such as irrigation canals. Typically less than 30 feet in height, these micro-dams provide unique benefits, not only supplying a renewable source of power, but doing so without endangering a fragile ecosystem, preventing natural fish migration, or causing severe geomorphological effects on downstream river conditions (Travis, 2010).

Many irrigation canals can utilize these micro-dams as a cost effective alternate form of energy dissipation. Since canals are typically constructed at relatively mild slopes, steep grades are often accommodated by installing drop structures just downstream of checks.

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Instead of dissipating this energy, however, a micro-dam can be utilized instead, removing and recycling the energy rather than losing it to imposed turbulence.

Placing micro-dams at every grade drop may be inefficient, however. Particularly for new canals, optimal placement of the micro-dams can result in higher power output, lower construction costs, and better control of the water distribution system. Determining optimal micro-dam placement is a non-trivial problem, however. From the planning standpoint, the type, number, elevations, and vertical drop of each micro-dam must be determined. Moreover, each micro-dam site carries with it specific design considerations, such as installation cost, earthwork, property purchase cost, zoning issues, and geotechnical considerations.

While optimal sizing and placement of micro-dams on canals appears to be new, the optimization literature on similar topics provides guidance. Mays and Bedient (1982) recognized the need for optimality in detention basin networks and developed a model utilizing the dynamic programming method. Travis and Mays (2008) optimized retention basin networks by dynamic programming. Bakhtyar, Mousavi, and Afshar (2007) optimized drop height and length of a preselected number of stilling basin cascades by dynamic programming.

Specific to micro-dam design, Özbay and Gençoğlu (2010) optimized control systems of micro-hydroelectric dams, and the benefits and key considerations for micro-hydroelectric power in riparian systems was explored by Uitto (2008). Cobaner, Haktanir, and Kisi (2008) developed a cost-benefit model for hydroelectric dam retrofitting of existing irrigation dams utilizing artificial neural networks.

Effective planning for micro-dams on canals requires an optimization approach. This paper optimizes micro-dam placement, number, elevations, and sizing through discrete dynamic programming. Grade elevations are utilized as independent variables, positioned so as to bookend each candidate location. Not only does this approach allow both the upstream and downstream elevations of the micro-dam to be established as part of the same optimization process, but also allows the option of not placing a micro-dam at all by simply setting the bookend elevations equal (e.g., no grade change means no micro-dam). Because the algorithm considers the option of not placing a micro-dam at every candidate location, the number of micro-dams does not need to be pre-selected, as is required in the Bakhtyar, Mousavi, and Afshar (2007) stilling basin optimization approach. In principle, within a desired tolerance, any number of locations along an entire reach could be considered as part of the same model. Finally, constraints to the optimization algorithm are developed in terms of site specific parameters, such as land cost and construction challenges; as well as global challenges, such as total elevation change of the canal reach being considered. Altogether, the developed approach should benefit both public and private sector engineers involved with canal system management, particularly at the planning stage of development.

MODEL

Variables

The simplified model of a canal system is defined as a sequence of M candidate locations, each identified by m as shown in Figure 1. The independent variables are the grade elevations that bookend each candidate location. These grade elevations are denoted as y_n, where n identifies the relative downstream position, varying from 0 to N. Thus, m corresponds to n as

$$m = \begin{cases} \frac{1}{2}n + \frac{1}{2}, \; n \; odd \\ \frac{1}{2}n + 1, \; n \; even \end{cases} \tag{1}$$

By identifying each candidate location by two closely spaced grade changes, the decision to place a micro-electric dam and the size of the dam can be established by grade elevations alone: if these two elevations are equal, no dam is to be constructed, whereas if these two elevations differ, a dam is to be installed with the given drop height. For example, Figure 1 shows a micro-electric dam installation with drop inferred by the grade change between y_n and y_{n+1}.

Note that at the limits of the model as defined by Figure 1, a candidate dam location has is considered at the upstream location (y_0) but not at the downstream end (y_N). This decision was arbitrary, of course, and can be modified for a given configuration. However, for any model the endpoints y_0 and y_N are always boundary conditions.

Given the distances between candidate stations (defined as x_n as measured from the upstream boundary), Figure 1 also defines the slope s_m along the canal reaches.

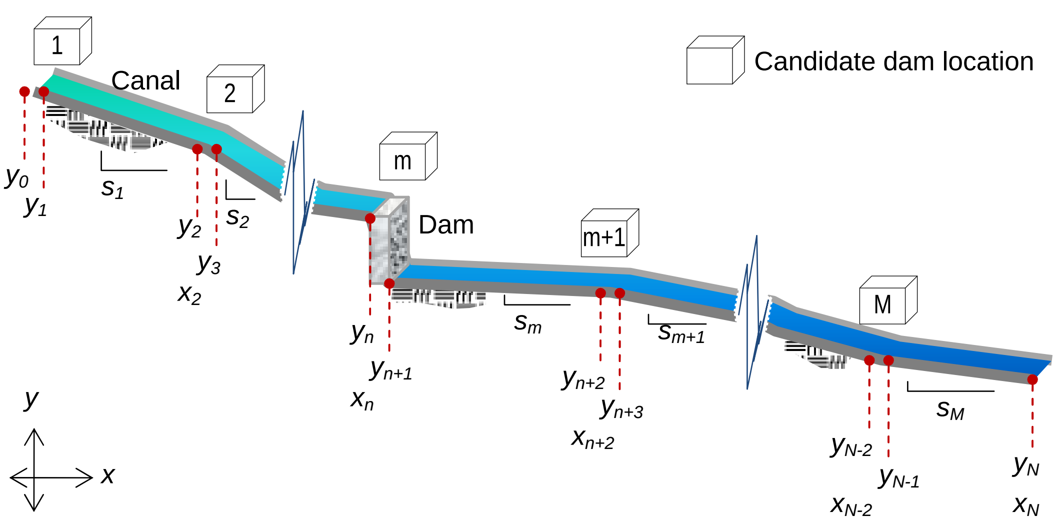

Figure 1. Canal Model Schematic

Constraints

The constraints on the system are as follows:

$$\Delta y_{cut} \le y_n - g_n \le \Delta y_{fill} \tag{2}$$

$$s_{\min} \le s_m \le s_{\max} \tag{3}$$

$$y_{n+1} \le y_n \tag{4}$$

Equation (2) establishes the limits on cut and fills at the candidate locations as Δy_{cut} (a negative value) and Δy_{fill} (a positive value), respectively. These limits may be driven by available construction methods, geotechnical considerations, embankment road construction issues, safety concerns, or other factors. The existing ground elevation is denoted g_n.

Equation (3) requires that the canal slope not exceed a given minimum ($s_{\min}$) or maximum ($s_{\max}$) value. Note that this constraint insures that the elevations are always decreasing.

While Equation (3) ensures that elevations are always decreasing between dams, it does not ensure the obvious requirement that, if installed, the vertical drop at the dam must be in the downstream direction. Equation (4) establishes this constraint.

Algorithm

Dynamic programming (DP) was used to optimize the network. The state variable is y_n. The cumulative profit function p_n is made up of four other functions in the form

$$n_{odd} : p_n = p_{n-1} + R_{\text{dam}}\left(y_{n-1}, y_n\right) \tag{5}$$

$$n_{even} : p_n = p_{n-1} - C_{\text{cut}}\left(g_{n-1}, g_n, y_{n-1}, y_n, x_{n-1}, x_n\right) - C_{fill}\left(g_{n-1}, g_n, y_{n-1}, y_n, x_{n-1}, x_n\right) \tag{6}$$

where R_{dam} is the expected net revenue function (expected annual profit minus annualized installation and maintenance) of a dam with head difference $y_n - y_{n+1}$; and C_{cut} and C_{fill} are the construction cost functions. The economic functions express results in equivalent present value for a selected return period. Not only does this ensure fair comparison, but (as will be shown) often has a profound effect on expected profit for a given return period. Note that the boundary condition for the profit function is $p_0 = 0$.

In practice, the functions identified in Equation (5) are often complicated and non-linear; and sometimes discontinuous. The advantage of DP is that it can incorporate any single valued function, so this complexity does not pose a problem.

The constraints can be met by either limiting the range of y_n values considered at each location (if the constraint is independent of the other variables) or by imposing a penalty to the profit function.

Thus, the general optimization function, denoted Z, is

$$Z \equiv \max\left\{\sum_{n=1}^{N} p_n\left(y_n\right)\right\} \tag{7}$$

Here, a forward recursive technique is utilized, and so the recursive optimization function is

$$f_n^*\left(y_n,k_n\right)=\begin{cases}\max\limits_{k_n}\left\{p_n\left(y_n,k_n\right)\right\}, & \textit{for } n=1\\ \max\limits_{k_n}\left\{p_n\left(y_n\right)+f_n^*\left(y_n,k_n\right)\right\}, & \textit{for } 1<n\leq N\end{cases} \tag{8}$$

The decision variable k_n corresponds to the optimal elevation at position n, selected from K_n different candidate elevations. The DP model starts at n = 1 and works forward along n, calculating the cumulative profit for each of the different candidate elevations and identifying the best decision. At $n = N$, the optimal profit is identified, and a traceback establishes each optimal elevation. Figure 2 presents the solution scheme in pseudo-code.

The foregoing is only a brief overview of dynamic programming. For more information, see Hiller and Lieberman (2010). For application of DP specific to water resources, see Mays and Tung (2002).

// Dynamic programming algorithm	
// $n, k,$ and j are counters	
// N is the total number of candidate locations with K_n the number of sizes considered at each one	
// profit(n, j, k) is the profit function; p_{test} and p_{best} are the test and best profits, respectively	
// $p_{n,k}$ is the profit matrix, $d_{n,k}$ is the policy matrix, & r_n is the decision vector of the best elevations	
for n = 1 to N	// for all candidate locations...
for k = 1 to K_n	// for all sizes at location n...
$p_{n,k} \leftarrow 0,\ d_{n,k} \leftarrow 1$	// intialize max profit & best policy matrix
for j = 1 to K_{n-1}	// for all sizes at location $n-1$
$p_{test} \leftarrow p_{n-1,j} + \text{profit}(n, j, k)$	// test the net profit for decision j given k...
if $p_{test} > p_{n,k}$ then $p_{n,k} \leftarrow p_{test},\ d_{n,k} \leftarrow j$	// ...if better, update max profit & policy matrix
next j	// loop to next j
next k	// loop to next k
next n	// loop to next n
$p_{best} \leftarrow 0,\ k_N \leftarrow 0$	// initialize overall best profit & final decision
for k = 1 to K_N	// for all possible decisions at N...
if $p_{N,k} > p_{best}$ then $p_{best} = p_{N,k},\ r_N = k$	// ...if better, update max profit and decision vector
next k	// loop to next k
for $n = N$ - 1 to 1	// begin traceback
$r_n \leftarrow d_{n,r_{n+1}}$	// record the decision at n given it at $n+1$
next n	// loop backward
return r	// return the decision vector

Figure 2. Dynamic Programming Pseudo-Code

DESIGN EXAMPLE

Input

A hypothetical canal is used for a design example. This example assumes a new canal will be constructed, thus allowing complete control over the slope, size of the canal, location of check structures, etc. In this example, the R_{dam}, C_{fill}, and C_{cut} functions (all in \$) are taken to be

$$R_{dam}\left(y_{n-1}, y_n\right) = PV\left[i_{rate}, t_{period}, -c_{\$}c_{kW}\left(y_{n-1} - y_n\right)^w\right] - u_{dam} - u_m \tag{9}$$

$$C_{cut} = c_{cut}\mathcal{V}_{cut,n} \tag{10}$$

$$C_{fill} = c_{fill}\mathcal{V}_{fill,n} \tag{11}$$

where:

- PV is the present value function;
- c_{kW} and w are constants that convert the depth term to power, and $c_{\$}$ is the conversion factor for power to profit per year;
- u_{dam} is the installed unit price of the dam, u_m is the site specific installation costs (e.g., property, zoning, fees, etc.);
- i_{rate} is the interest rate over return period t_{period};
- c_{cut} and c_{fill} are the costs to cut a volume $\cancel{v}_{cut,n}$ and fill a volume $\cancel{v}_{fill,n}$ measured between stations x_{n-1} and x_n.

The assumed values of all of the constants are shown in Table 1. The general form of the equations represents highly simplified but generally consistent with methods employed by Natal Energy, Inc. (Blankenship, personal communication 2010). Units in feet were used for the elevations (y) and stations (x).

The canal was assumed have a rectangular cross-section with a 10 foot width. Of course, in practice, a trapezoidal cross-section would be more likely, wherein the sideslopes of the channel would have a direct and possibly significant effect on cut volume, fill volume, and required property cost. Future work will consider other cross-sectional geometries.

Table 1. Assumed Costs in the Example Problem

Factor	Value	Units
w	1.4	-
c_{kW}^{*}	5.67	$kW/ft^{1.4}$
$c_{\$}$	150	\$/kW/yr
u_{dam}	315,000	\$
Δ_{cut}	20	ft
Δ_{fill}	20	ft
i_{rate}	4	%
s_{min}	0.05	%
s_{max}	5	%
c_{cut}	3	$\$/ft^3$
c_{fill}	3	$\$/ft^3$

*The fractional units for this coefficient result from the R_{dam} function as defined by Equation (9)

Ten different candidate locations are identified, and their specific characteristics of the (existing elevations, stations, and dam installation unit costs) are shown in Table 2. As noted earlier, any number of candidate locations can be considered at a linear cost to time needed to execute the optimization. For this hypothetical application, however, ten candidate locations was deemed to be both sufficient in order to present the procedure while not overwhelming this article by reporting excessive input data.

Ten different elevations were tested at each of the ten candidate locations. Annual return periods from 1 to 50 years were evaluated.

Table 2. Assumed Values in the Example Problem

m	x_m (ft)	y_m (ft)	u_m ($)
0	0	3100.0	$40,000
1	5,000	3065.0	$30,000
2	10,000	3030.0	$30,000
3	15,000	2995.0	$20,000
4	20,000	2970.0	$20,000
5	25,000	2960.0	$10,000
6	30,000	2955.0	$10,000
7	35,000	2950.0	$5,000
8	40,000	2935.0	$5,000
9	45,000	2920.0	$10,000
10	50,000	2905.0	$20,000
11	55,000	2902.5	-

Results

The DP algorithm was executed by spreadsheet on a standard laptop computer; computing time was several seconds. Figure 3 shows the results of the DP optimization considering 5, 10, 25, and 50 year return periods. The present value profit as a function of return period is shown in Figure 4.

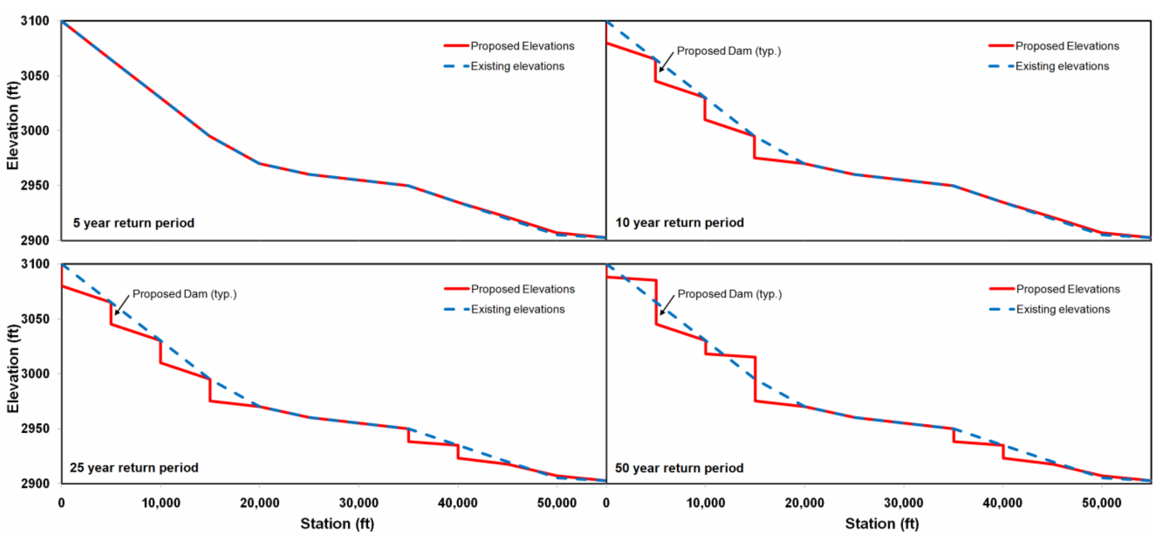

Figure 3. Example Results for 5, 10, 25, and 50 Year Return Periods

From Figure 3, it is seen that the four return periods have different optimal policies. The cost of dam installation at any of the candidate locations is too much to achieve profit for a return period of only 5 years (thus, the optimization routine does not suggest any micro-dams be installed). Interestingly, while the algorithm considers multiple drop sizes for each location, the 10-year period return was optimized with three identically sized micro-dams. For the 25-year return period, an additional two micro-dams (with a smaller elevation drop) were added. The 50-year return period plan is quite different than the other return periods, with the greatest profit resulting from a different upstream configuration, with two large micro-dams with one smaller micro-dam located between them rather than three the same size.

Figure 4 indicates that for return periods greater than six years, profit nonlinearly increases, reflecting the non-linearity of the present value calculation, the non-linear power / depth relationship, and variation in the number and sizing of the micro-dams.

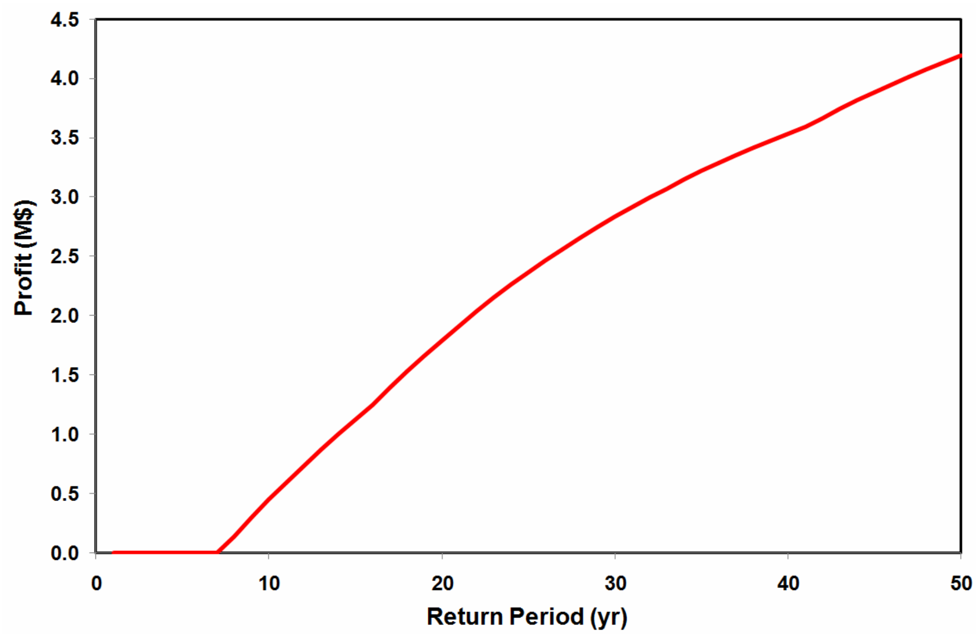

Figure 4. Equivalent Present Value Profit in Millions of Dollars (M$) Versus Return Period for the Example Application

CONCLUSIONS

When optimized, the expected return on micro-dam installation is seen to be particularly sensitive to evaluated return period. This is likely a consequence of the need for a relatively large initial investment. Targeting a specific return period appears to be essential before installing micro-dams, as it appears that optimal dam sizes and locations can significantly vary depending on the return period selected.

The DP algorithm presented here is much more versatile than the simple example problem demonstrated. More realistic cut and fill functions are easily incorporated. The costs of exporting or importing material can also be included, refining the design to balance cut and fill. Also, the example problem considered only one type of micro-dam, but the DP algorithm can consider multiple micro-dam sizes or even micro-dams in parallel without difficulty. Indeed, the best dam (or dams) at any one location can be designed by simply optimizing the various configurations within the profit function.

The developed model considers only a few of the many factors that must be considered when locating hydroelectric generators, such as the distance to the commercial grid, which can be a critical consideration, especially for long canals. Other costs should be considered as well, including component replacement costs, depreciation, etc. The model is easily extended to account for these factors, as will be demonstrated in a later paper extending the present work.

It must be noted that the DP algorithm does not guarantee global optimality. It does, however, asymptotically approach global optimality as the range of candidate locations and sizes become large. Thus given the high speed of the algorithm, it is likely that the input ranges can be made large enough to ensure confidence in the results.

The example application considered new canal construction, but the adjusting the constraints allows the algorithm to determine optimal placing of micro-dams on existing canals as well. For example, if installing a micro-dam would require the relocation of a check gate and the mechanism for delivering water to the farmers, these costs would simply be added to the other costs considered in the model.

Finally, the presented method can also be applied to other applications, such as siting dams on existing rivers, or even inverting the problem to locate the best pump or lift locations.

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REFERENCES

Bakhtyar, R., Mousavi, S.J., and Afshar, A. (2007). “Dynamic programming approach to optimal design of Cascade Stilling Basins”, *J. Hyd. Eng.,* 133(8), 949 – 954.

Cobaner, M., Haktanir, T., and Kisi, O. (2008). “Prediction of hydropower energy using ANN for the feasibility of hydropower plant Installation to an existing irrigation dam”, *Water Res. Mgmt*., 22, 757 – 774.

Goodwin, P., Falte, M., and Betss, A.D.K. (2000). “Managing for unforeseen consequences of large dam operations”, contributing paper to Howard, C.D.D. (2000), *Operations, Monitoring, and Decomissioning of Dams, Thematic Review V IV.5*, prepared as an input to the World Commission on Dams, Cape Town, www.dams.org.

Hiller, F.S., and Lieberman, G.J. (2010). *Introduction to Operations Research*, 9th ed., The McGraw-Hill Companies.

Jacquot, J. (2009). “Dams, from Hoover to Three Gorges to the Crumbling Ones”, *Discover Magazine*, March.

Mays, L.W., and Bedient, P.B. (1982). “Model for optimal size and location of detention.” *J. Water Resources Planning and Management*, ASCE, 108(3), 270-285.

Mays, L. W., and Tung, Y-K. (2002). *Hydrosystems Engineering and Management*. Water Resources Publications, Highlands Ranch, Colorado.

Özbay, E., and Gençoğlu, M.T. “Self-tuning fuzzy pi controlled system model for small hydro power plants”, *Proceedings of the 10th International Conference on Clean Energy (ICCE-2010)*, Famagusta, N. Cyprus, September 15 - 17, 2010.

Travis, Q.B. (2010). *Ebb and Flow: Preserving Regulated Rivers through Strategic Dam Operations*, Doctoral Dissertation, Arizona State University, Tempe.

Travis, Q.B., and Mays, L.W. (2008). “Optimization of retention basin networks”, *J. Water Res. Plng. and Mgmt*., 134(5), 432 – 439.

Uitto, J.I. (2008). “Small hydel for environmentally sound energy in remote areas: Lessons from the Indian Himalayas”, *Focus On Geography,*51(2), 1 – 8.

PROTOTYPE HYDROPOWER GENERATION DESIGN AND CONSTRUCTION FOR LOW-HEAD APPLICATIONS

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ABSTRACT

The increasing costs of electrical energy generation have forced irrigation districts to consider innovative approaches to reduce and subsidize energy-related costs to their constituency. Irrigation canals offer a renewable source for hydropower generation that has gone unnoticed until recently, due to high capital costs. Present incentives created by establishment of renewable energy credits, REC, create a more competitive economic environment for small hydropower systems. The Elephant Butte Irrigation District, EBID, has received funding from the New Mexico Energy, Minerals and Natural Resources Department, NM EMNRD, to assess alternative means to reduce initial capital costs for hydropower generation for low-head applications. Computational fluid Dynamics, CFD, for low-head conditions found in irrigation canals (6 – 10 ft) lead to the design of a fixed pitch 24-inch diameter turbine. This turbine was fabricated and installed by EBID personnel during the replacement of an existing drop structure. The double-impeller turbine generated up to 7.2 kW at approximately 30% overall (water-to-line) efficiency. Total capital cost for fabrication and installation of this unit was about $16,000, resulting in capital cost outlay of approximately $2.22 per watt, a cost that is considerably lower than equivalent solar or wind generation projects. The payback period using solar New Mexico solar REC rates for this project is 3.5 years, assuming a project duration of 10 years.

INTRODUCTION

Description of the Elephant Butte Irrigation District

EBID is a quasi-municipal entity of the state of New Mexico. The district operates under New Mexico statutes §73-10-1 through §73-10-47 Irrigation District Cooperating with United States under Reclamation Laws; Formation and Management, and §73-11-1

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through §73-11-55 Irrigation Districts Cooperating with United States under Reclamation Laws; Fiscal Affairs; Local Improvements and Special Powers.

There are 90,640 acres of land within the EBID boundaries that have authorized water rights, with an estimated 7,900 water users; the majority of them are agricultural users. The Rio Grande Project covers 130 miles of land located in the Lower Rio Grande Basin from Caballo Dam to El Paso, Texas. The EBID jurisdiction extends from Percha Dam, at the Southern end of Sierra County, NM and it traverses Doña Ana County, in Southern New Mexico. The irrigation district ends near the border that separates El Paso County, Texas, Doña Ana County and the Republic of Mexico. Geographical location of the EBID (Texas Compact) is shown in Figure 1.

Snowmelt runoff from the Rocky Mountains in southern Colorado provides the bulk of the water that reaches the Rio Grande. Water delivery into the Elephant Butte and Caballo Reservoirs is guaranteed by the terms of the 1939 Rio Grande Compact. The irrigation season for the Rio Grande Project typically runs from mid- March to mid-October. The average annual Rio Grande flow to Elephant Butte Reservoir is 937,570 acre- feet of water, but this flow can be erratic, ranging from 114,100 to 2,831,000 acre-feet per year. In full supply years, the District allots 3 acre-feet per acre to 90,640 acres, or about 272,000 acre-feet. In 2003 and 2004, drought reduced the diversion allocation to about 150,000 acre-feet.

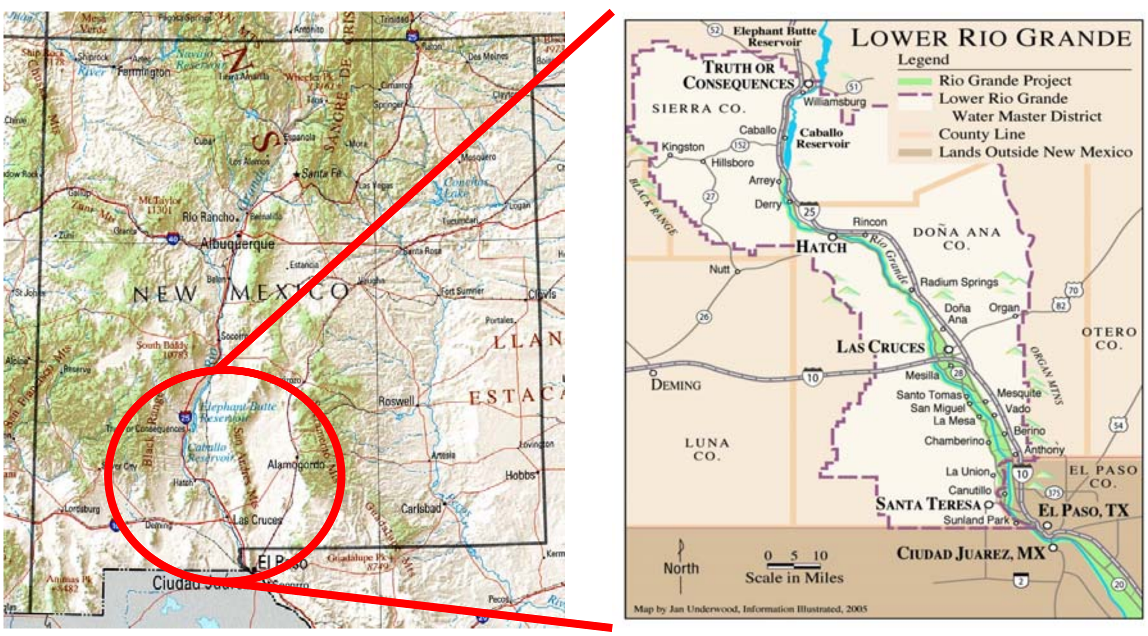


Figure 1. Geographical Location of the EBID (Texas Compact)

Water available to Elephant Butte Irrigation District is stored in the Elephant Butte and Caballo reservoirs until it is ordered for release by the irrigation district. Water delivery in the EBID system was engineered as a gravity flow process using canal systems to convey the water from three diversion dams located along the Lower Rio Grande (Percha, Leasburg and Mesilla). The total elevation gradient through the EBID system, from the upper reach at Caballo Dam to the lower end at the American Dam (near the Mexico border) is 430 feet. Average slope for the system is approximately 4.0 feet per mile for a horizontal river distance of 107 miles. Figure 2 depicts the gradient loss throughout the EBID system from Caballo Reservoir to the American Dam.

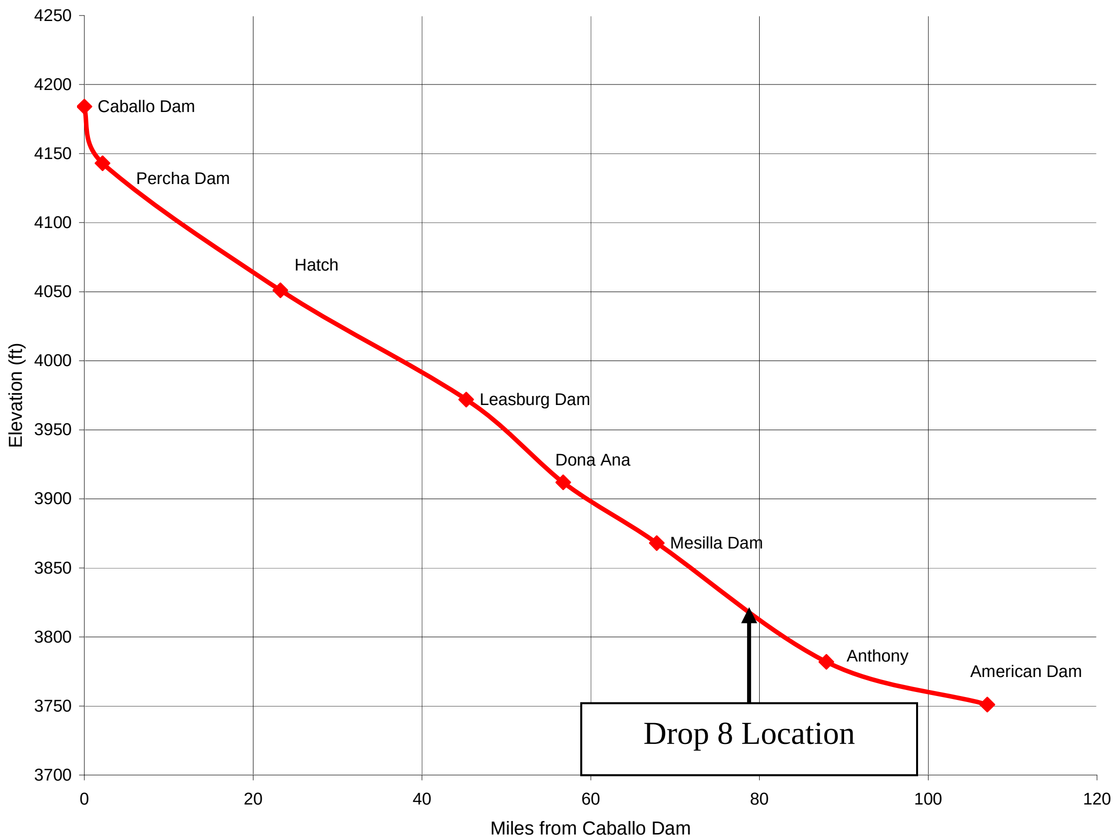


Figure 2. Hydraulic Profile within the EBID System

Drop structures are used to dissipate excess energy in irrigation canals, where back-water elevation is controlled by adjustment to gate openings. It is at one of these points of energy dissipation (Westside Canal, Drop 8) that EBID installed a prototype hydro power generation station using resources from EBID and the New Mexico Energy, Minerals and Natural Resources Department, NM EMNRD. The station is located near La Mesa, NM between Mesilla Dam and Anthony, NM as shown in Figure 2.

Selection Criteria

The Westside Canal has at over 10 drop structures where energy could be recovered as electric energy. This Drop 8 site was selected among the available drop structures based on the following criteria: at least six feet available hydraulic head, ample space to append the hydropower station to the drop structure, flowrate of at least 100 cfs and proximity to El Paso Electric's (the local electric company) distribution grid. This site was preferred since the Drop 8 site handles an average flow rate of 300 cfs and dissipates approximately eight feet of head. Additionally the EBID right of way allowed for construction of a lateral structure to house the hydropower station without interfering with local traffic while the electric distribution grid is approximately 100 feet away from the site. The original sliding drum structure at Drop 8 was built in the early 1900's and was replaced by a new, more functional, radial gate structure. The aerial photograph in Figure 3, taken at around the time of site completion, highlights the main reasons for site selection. This figure shows the new radial gate structure adjacent to the old one, which was left on site for historical preservation purposes.

Figure 3. Aerial View of Drop 8 Site

The technical challenge to recover energy from hydraulic systems increases as the available potential energy (i.e., hydraulic head) decreases. Recovered energy is lower than the theoretical energy (the product of head and flowrate) due to efficiency losses in the hydraulic to electrical conversion process. For this reason, it is common for commercial units to have overall efficiencies below 50% for heads lower than 10 feet. Low power generation may be offset by a proportional increase in flowrate through the turbine, resulting in turbines that are typically voluminous and expensive. An extensive search for an off-the-shelf turbine revealed that manufactured turbines for low head applications are quite expensive with prices starting at $125,000 for a 50 kW unit. It is important to consider that most of these units are manufactured upon request in either Europe or Asia; thus, delivery costs can be considerable and their delivery could be a slow process resulting in untimely project completion.

Our first attempts at harnessing the hydraulic energy at Drop 8 using two enclosed paddlewheel designs were quite disappointed, as explained later in this publication. We also explored using a pump as a turbine (PAT) for this project. Though, this approach could have been more economical than the off-the-shelf turbine approach, we were warned and discouraged by pump manufacturers that use of their pumps in a reverse mode (i.e., as turbines) would void warranty on their equipment. Additionally, our electrical engineering advisors were hesitant to pursue this venue because the electrical components might not function efficiently as generators. Our logical conclusion that a PAT was not appropriate for the Drop 8 site was based on these two criteria (high capital cost and uncertainty for success).

We concluded, at this point, that the paddle wheel and the PAT concepts were unfeasible for our needs due to low efficiency of the former and cost and uncertainty of the latter. Elimination of these two likely technologies for the Drop 8 site forced our engineers to consider design and fabrication of an appropriate and inexpensive turbine system, capable of producing power approaching 10 kW. Our intent was to replicate the successful turbine design, as needed, in order to generate a total of 50 kW at the Drop 8 site. Design and fabrication considerations are presented later in this publication. Our construction plans were reduced to two turbines at the Drop 8 site due to last-minute financial constraints within the EMNRD, which led to partial funding on the original budgeted support by the agency.

Design of electrical interconnection to the commercial grid has become a relatively simple process, since there are many commercially available products that are used in small solar and wind generation facilities. Our electrical engineering consultants concluded that the mechanical energy at the turbine should be converted to alternating current (AC) 220 V, using an off-the shelf generator. Prices for the generator varied widely, from $300 (7 kW) to $2,500 (20 kW). After initial testing with the least expensive generator, we opted for the unit with the more robust and expensive unit, with the expectation of reducing operation and maintenance expenses later in the project.

Conveyance of generated power to the grid requires a perfect match in the generator frequency with the grid frequency (60.0 Hz in the USA). It became clear in our preliminary investigations that we could not maintain such consistent output frequency at the generator. Yet, this limitation can be overcome by converting the AC to direct current (DC), with back-conversion to filtered AC using an off-the-shelf inverter that is common in solar energy applications. We estimated the total cost of electrical energy conversion at around $5,000. We continue exploring other salient technologies for power conveyance at lower capital costs.

During 2010 our plans to convey electricity to the commercial grid were forestalled due to various legal constraints, which are discussed in the legal section of this publication. Thus, we have stopped our construction at the unfiltered AC level of the project at the present time.

ENGINEERING DESIGN AND CONSTRUCTION

Civil Works Design and Construction

The Drop 8 site required replacement of an obsolete drum gate system with a more functional radial structure to be built immediately upstream from the old structure, as shown in Figure 3. A reinforced concrete drywell was built adjacent to the new drop structure to house the hydropower station. All civil works required for drop replacement and drywell construction were performed by EBID personnel. The CAD drawing in Figure 4 illustrates the new drop structure with the adjacent hydropower drywell.

Flow from the Westside Canal is controlled into the two turbines in Figure 4 by an equal number of 24-inch Armco gates. Water passing through the turbines is returned back into the downstream section of the canal via a 5-foot diameter corrugated metal pipe (CMP) that serves as the raceway. Excepting these CMP pipes and gates, all other conveyance piping shown in Figure 4 was manufactured to specifications by EBID metal fabricators.

Mechanical and Hydraulic Design

The engineering staff at EBID performed several engineering designs for the turbines at the Drop 8 site. A 36-inch diameter, enclosed paddle wheel concept was used during the first and second design stages in this project. The first paddlewheel design was tested by reducing the 24-inch inlet into a 16-inch pipeline, with the expectation that the resulting increased water velocity would enhance paddlewheel rotation. Maximum power at the generator using the first paddlewheel design generated a disappointing 150 watt. Power generation was doubled during testing of the second paddlewheel design by applying the water toward the outside of the wheel, but even this 300 watt output fell quite short of expectations.

Our mechanical engineering consultants suggested using an axial flow turbine which could be optimized using computational fluid mechanics in future design improvements. To expedite the proof of concept, a $500, 15-inch diameter boat propeller was installed in the horizontal portion of the 16-inch steel pipe, as illustrated in Figure 5. Maximum water-to-line power for this design was 600 watts. This power generation improvement led to the logical addition of a second, in-line propeller on the same shaft, placed a few inches upstream the initial one. The stacked propeller turbine shown in Figure 5 was capable of generating up to 1.5 kW.

The favorable results from the axial turbine concept led to the next improvement alternative: increasing the flowrate by using a 24-inch pipeline equipped with two 23-inch commercially available propellers. However, the cost for each boat propeller, $2,500, proved to be prohibitive during the experimental phase of this project. Our mechanical engineering consultants suggested in-house fabrication of a simplified axial propeller, in lieu of the commercial propellers. Based on the successful runs with the axial flow concept, they suggested using a fixed-pitch Kaplan-type propeller, which would be relatively easy to manufacture.

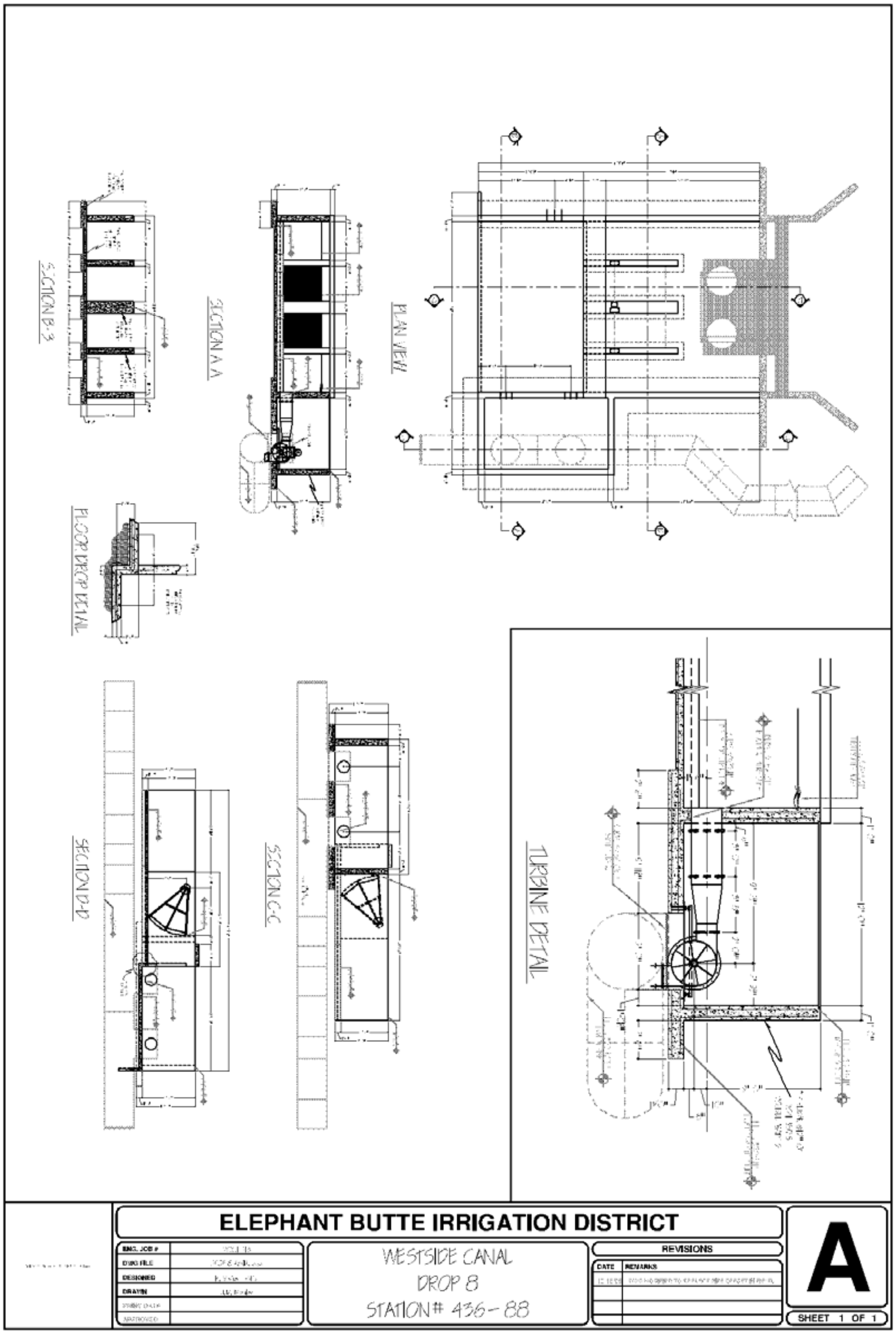
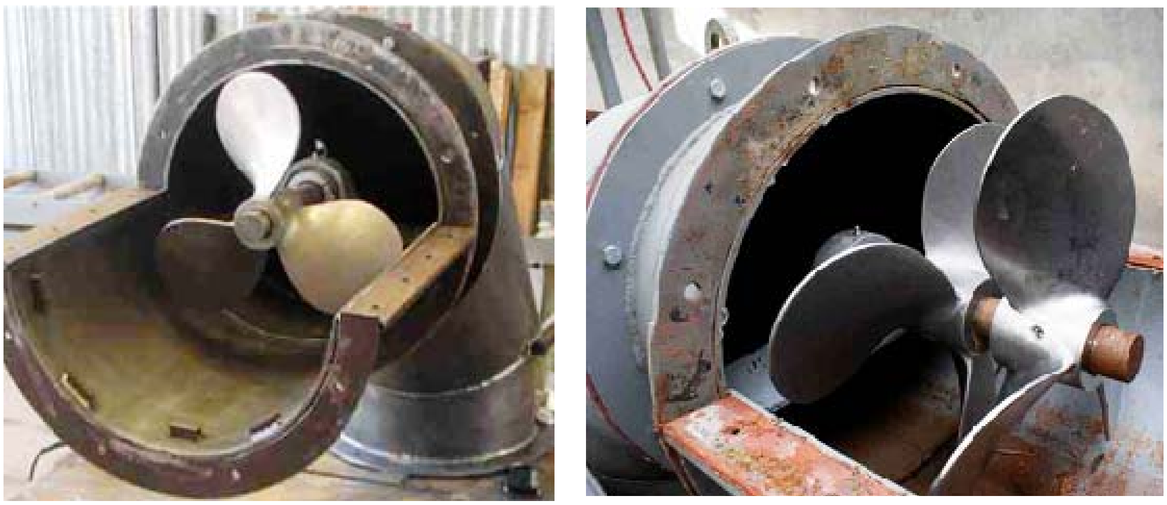


Figure 4. Engineering Drawings for the Drop 8 and Hydropower Structure
Note: A 36-in, Enclosed Paddlewheel Concept is shown in this Drawing.

Design optimization of a propeller is a difficult and labor intensive fluid mechanics task. Computational fluid dynamics software (CFD) provides the means to bridge this shortcoming. The CFD software chosen for this project was Pumplinx developed by Simerics, which uses Navier Stokes code, specifically for fluid pumps and motors. Since a turbine is simply a pump running in reverse, Pumplinx was a good fit for the micro-hydro project.

Pumplinx was used to find the theoretical optimum blade angle and geometrical configuration. For the 24-inch cases blade angles of 20^o, 25^o, 30^o and 35^o, were investigated with and without stators for the 23-inch Kaplan propellers[6]. Table 1 summarizes the main CFD results for the fixed-pitch, 6 blade, 24-inch Kaplan turbines subjected to a 10 foot head drop. This head, though larger than the 8 ft presently available at Drop 8, may be achieved in the future by increasing the upstream bank elevations.



a. Single-Propeller Alternative

b. Double Propeller Alternative

Figure 5. Three-blade, 16-inch, Commercial Propeller Turbines

It may be observed in Table 1 that the overall (i.e., water-to-line) efficiency decreases with increasing pitch angles. However, energy production increases by augmenting the pitch angle from 20^o to 25^o and then it decreases for larger angles at the expense of greater flowrate utilization for the higher pitch turbines. A similar set of CFD evaluations resulted in maximum power output with a two-30^o pitch blade using four straight stators, as shown in the last row of Table 1. Water utilization for this 2-propeller configuration is also predicted to be lower than the single turbine case, as the obstructing surface opposing water flow movement is increased. From CFD work, this double propeller should produce 12.2 kW with a 46.9% overall efficiency, a value that is close to

[6] Stator blades are fixed blades placed between the rotating blades of the turbine to achieve a particular inlet flow condition for the next set of blades. Simplistic stator blades are straight and only straighten out the flow of the fluid, but more complicated stator blades are curved. Our limited machining capabilities forced us to assess the straight stators only.

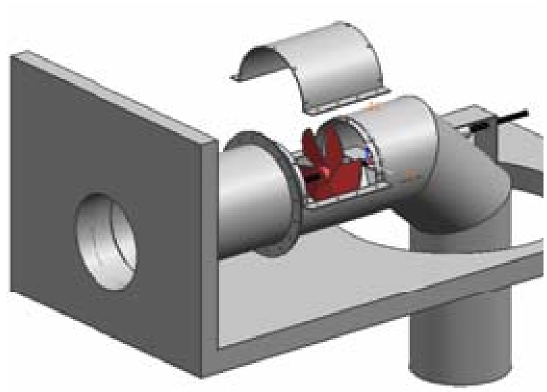
the known efficiencies of commercial turbines in low-head applications. Engineering assessment of these data led to the selection of two 30°-pitch turbines with stators for experimental testing.

Table 1. CFD Results for the 24-inch Kaplan System

Number of Props.	Pitch Angle (°)	Angular Speed (rpm)	Torque (N m)	Flowrate (m^3/s)	Power (kW)	Overall efficiency (%)
1	20	280	266	0.82	7.7	31.5
1	25	247	312	0.95	8.2	28.9
1	30	213	357	1.06	7.9	24.9
1	35	182	377	1.14	7.2	21.1
2 - stators	30	263	438	0.87	12.2	46.9

The experimental testing of the CFD optimized turbines was conducted in two consecutive phases. Power production of the single propeller system was tested initially. Figure 6 shows the turbine during in-house manufacture and the mechanical CAD design for this single-propeller configuration. Maximum tested power output for this design was 6.2 kW, or 86% of the CFD predicted value in Table 1. This value is consistent with model expectations considering that the actual head was 8 ft, rather than the simulated 10 ft (i.e., 80% of the theoretical head).

a. 30° Propeller during Manufacture



b. Complete, Single Propeller with Open Hatch

Figure 6. Fixed-blade 30° Kaplan Turbine Concept

A second, identical turbine was fabricated and installed during the second phase of the fabricated turbine testing. A set of four straight stators was also installed within the hatch compartment expecting that the power generation would be greater than in the single-propeller case. Measured power output for this improved turbine arrangement was 7.2 kW, which is considerably lower than the CFD predicted value. This discrepancy between model and actual results may be due to inadequate flow redirection by the stators.

ECONOMIC ASPECTS

Table 1 itemizes the capital costs for manufacturing the fixed-pitch turbine system (2 propellers and stators). This table excludes the cost of drywell construction, since it was built as an integral component of the drop structure, which was completely replaced with different funding sources.

Table 2. Capital Manufacturing Costs for Single-Turbine
(Double-Propeller, Fixed Kaplan and Stators, excluding drywell construction costs)

Item	**Cost**
Materials	\$2,500
Generator	\$2,500
AC-DC-AC Converter	\$5,000
Labor	\$3,000
Miscellaneous	\$3,000
Total	\$16,000

El Paso Electric, EPE, the local electric utility, is searching to supplement its power needs with green, renewable energy. The approved purchase rate has two additive components: the Renewable Energy Credit (REC) and the non-renewable generation rate. The small system renewable energy purchase rate is available to customers owning renewable generation rated 10 kW or less (this legal constraint represents a secondary reason for maintaining power output below 10 kW at each of the turbines). The most recently approved REC rates by the New Mexico Public Regulation Commission, NM PRC for solar power and wind power are \$0.12 and \$0.08 per kW-h. The NM PRC has not established equivalent rates for small hydropower systems, as to our knowledge no other hydropower generators have pursued this economic venture, to date. The non-renewable rate varies over season and whether production occurs over peak demand. The average annual rate for this component is approximately \$0.06/kW-h.

Table 3 presents the main assumptions made for the engineering economics for a single-turbine (fixed Kaplan, dual propeller with stators). The EBID canals convey irrigation water from about March through October, depending on water availability in the upstream storage reservoirs (Caballo and Elephant Butte). We assume a downtime of 25% in this table to account for non-irrigation time and maintenance. The maximum experimentally-confirmed power generation of 7.2 kW is assumed in the computations, as shown in Table 3. Annual power generation approaches 47 MW-h/yr using these assumptions. The engineering economics summary in Table 4 assumes that the power rate sale is equivalent to the solar energy case, \$0.12 plus \$0.06 per kW-h for non-renewable rate, as illustrated in Table 3.

According to Table 4, the annualized cost (i.e., annual replacement cost) required to recover the total capital cost in Table 3 in 10 years is \$1,973 for this project, at an interest rate of 4% per annum. Based on our experience at Drop 8, we estimate \$2,000 annual cost for operation and maintenance, O&M. The sum of these two annual costs yields a total annual cost of \$3,973.

Table 3. Assumed Input for Engineering Economics Study for Singe-Turbine Case

Item	**Value**
Capital Cost	\$16,000
Interest Rate	4.00%
Lifetime (yr)	10
Downtime	25%
Power Generation (Watt)	7,200
Annual Production (kW-h)	47,304
Unit Price (\$/kwh)	\$0.18

Estimated annual revenue, due to generated electricity sales, is \$8,515 for a single-turbine. Thus, the net annual revenue is \$4,542, as shown in the same table. The payback period using these assumptions is 3.5 years, which is approximately one-third than the estimated lifetime of the project. Thus, the economics of this project are quite favorable. It is important to notice that the payback period would increase to 6.0 years if the REC rates adopted by the NM PRC were similar to the ones for wind energy (\$0.08/kW-h), making this project feasible, but to a lower extent, than for the higher \$0.12/kW-h for solar energy. The minimum REC required to make the project feasible within the 10 year project lifetime is \$0.02 for a total energy sale value of \$0.08/kW-h.

Table 4. Engineering Economics Summary for Single-Turbine Case

Costs		**Revenue**	
Annualized Capital Cost	$1,973	Annual Revenue	$8,515
O&M Costs	$2,000	Total Annual Costs	($3,973)
Total Annual Cost	$3,973	Net Annual Revenue	$4,542

LEGAL ASPECTS

In the past, the regulatory restrictions imposed by the Federal Energy Regulatory Commission, FERC, made the application process for small hydropower onerous and slow, discouraging entrepreneurial of small hydropower sites. FERC has recently relaxed many of these requirements by granting exemptions to small hydropower systems with total generation capacity below 5 MW unless the applicant's project:

1. Is located on a navigable waterway of the United States;
2. Occupies lands of the United States;
3. Uses surplus water or waterpower from a government dam; or
4. Is located on a stream over which Congress has Commerce Clause jurisdiction, is constructed or modified on or after August 26, 1935, and affects the interests of interstate or foreign commerce.

We believe that we are exempt from all restrictions above and thus meet the criteria required by FERC above. For this reason, EBID has started the accelerated exemption application process with FERC but it is premature for us to determine the regulatory fate of the project. Besides these legal limitations, a second legislative hurdle that must be overcome is the establishment of a REC rate for hydropower generation within the State of New Mexico by the NM PRC. Obviously, the feasibility of the project hinges on issuance of an exemption by FERC and adoption of favorable REC rates for small hydro projects by the NM PRC.

CONCLUSIONS

Up to date, the economic feasibility of hydropower generation in irrigation canals has been marginal at best due to high capital costs for off-the shelf turbines and the lack of subsidized renewable energy credits, RECs. The introduction of RECs in many states has favored economic reevaluation of many renewable energy methods, including hydropower generation in irrigation canals. This study demonstrates that it is technically possible to build a functional turbine using in-house fabrication, at a cost that is considerably lower than commercial devices. In spite of the relatively low efficiency of these turbines, it is possible to recover their cost within a few years, as long as favorable REC rates are available to subsidize these projects. Project success also depends on exemption from the formal hydropower license process by FERC for these small hydro projects.

ACKNOWLEDGMENTS

The authors wish to acknowledge the invaluable participation of Mr. Daniel Marquis and Drs. Satish Ranade and George Mulholland, who served as consultants on this project. This project was partially funded by the NM Energy, Minerals and Natural Resources.

BLUE IS THE NEW GREEN

Ed Gerak, P.E[1]

ABSTRACT

In the last decade, the United States has become increasingly aware of the need for environmental stewardship. What once was considered extreme ("tree hugger") has quickly gained acceptance in recent years. Thanks to media attention, most Americans have a heightened social conscience.

All this attention on environmental awareness has spurred people to educate themselves on the benefits of living green. This awareness has also enlightened people about the cons of green technology. For example, certain power is only generated when the wind blows or the sun shines. Also, the amount of land and water needed to generate electricity from these technologies is considered detrimental. These factors have contributed to a broader view of other options, including a closer look at hydroelectricity.

Hydroelectricity was viewed during the reclamation projects in the west from a strictly an economic perspective, but people are now becoming acutely aware of the benefits of hydroelectricity from an environmental impact standpoint.

This focus on green energy has opened the way for an acceptance of hydroelectricity as an acceptable green energy. On March 24, 2010, the Department of Energy, Department of Interior and the US Army Corps of Engineers signed an MOU recognizing hydro as a renewable energy source and called for the evaluation, promotion and possible development of hydro on existing federal and non-federal facilities (with proper environmental consideration).

This paper will highlight the increased attention placed on hydroelectric potential over the last 3 years.

INTRODUCTION AND BACKGROUND

Buckeye Water Conservation and Drainage District (BWCDD) is an irrigation district that operates as a municipal corporation in the state of Arizona. The District provides water and power to approximately 22,000 acres. The District's mission is to provide reasonably priced water to the agricultural and urban lands in the District.

BWCDD has an allocation of Hoover power, which aids in keeping our water and electrical costs affordable. This allocation expires in 2017, and legislation is currently ongoing to renew the contracts. However, the District initiated a pilot project in an attempt to replace the Hoover power if it did not get reallocated.

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When the Buckeye Irrigation Canal was surveyed in 1909, potential locations for hydro sites were identified. Due to the drop sizes, it would not have been practical to develop a hydro unit until recently. With improvements in low-head technology, we decided to investigate if this was now feasible.

We contacted K.R. Saline, our electrical engineering contractor, to inquire if they had any experience with low-head hydro units. They connected us with Natel, the manufacturer of the SLH (Schneider Linear Hydro) engine. We were impressed by the technology and decided to partner with the inventor on the first commercial installation of the SLH.

Since the decision to install a low-head hydro unit, we are well acquainted with the process, including the political process for hydroelectric units.

EVOLVING TIMELINE

As Congress began to wrestle with the global climate change debate, multiple ideas came to the surface. The most prominent idea to surface has been cap and trade legislation. Another discussion has been to take the same approach as some states, and establishing a federal renewable energy portfolio standard. Whatever the outcome, it is apparent that Congress is spending considerable time exploring possible solutions.

This focus on environmental stewardship has opened the door to examine all technologies; including hydro (and even nuclear) to combat this perceived threat of global warming. A seemingly partisan division on hydroelectricity (big dams vs. the environment) has evolved into bipartisan support for hydro possibilities, with proper consideration of environmental impacts.

Rodgers Letter

In a letter-dated May 1st, 2008 from Cathy McMorris Rodgers-Congresswoman from Washington, to Grace Napolitano, Chairwoman of the Water and Power Congressional Subcommittee, Mrs. Rodgers requested that the Chairwoman hold an oversight hearing on the "historical and future role of clean and renewable hydropower in meeting consumer electricity needs."

Congresswoman Rodgers refers to Chairwoman Napolitano's support (by vote) of an amendment to classify classic hydropower as renewable (later removed from the bill). She also referred to hydro's ability to be a base load and back-up wind or solar projects.

Conduit Exemption

While we went through the small conduit exemption application to FERC, it became very evident that the process is heavily influenced for large dams and that the process has not been updated to accommodate the new advances.

For example, our application was for a 9.6 kW capacity engine in an existing conduit with no environmental impact, and our application took 9 months for approval.

While we were waiting for approval, a grass roots effort started in an attempt to modify the low-head hydro process. In discussions with FERC, they agreed that the process should be streamlined, even suggesting "blanket certifications," like in the natural gas industry, as a possible solution.

A white paper was drafted with regards to a possible exemption of the FERC process if the project was under a certain capacity (1.5 MW). These efforts resulted in the introduction of "The Small-Scale Hydropower Enhancement Act of 2010'' by Congressman Adrian Smith of Nebraska.

<u>Signed MOU[2]</u>

A Hydropower Workshop hosted by the Department of Interior on March 24th, 2010, caught people's attention when the DOI, Department of Energy and Army Corps of Engineers signed a Memorandum of Understanding on Hydropower.

What stood out in the MOU was that three departments of the federal government all publicly agreed that hydropower was a minimal emission, low-cost source of renewable energy. They pledged to work synergistically for the identification, addition and improvement of generation capacity on existing and potential hydro power sites on both federal and non-federal lands, while ensuring sustainability and protecting the environment in the process.

In articles following the MOU, Interior Secretary Ken Salazar and Energy Secretary Steven Chu reinforced the need for the MOU and its potential impact, while appeasing the anti-dam crowd.

Estimates vary on the amount of power untapped hydro sources might yield, but Energy Secretary Steven Chu estimated it could range between 16,000 and 25,000 megawatts. "This is a lot," Chu said. "It's clean power, renewable energy on demand.
This is what we're trying to focus on."

Interior Secretary Ken Salazar said. The following-"This is not ushering in a 21st century new dam era…This is taking a look at existing facilities and low-impact hydro. This is an examination of what we can do with hydropower that does not necessitate the building of new dams."

What was most intriguing to me was that FERC was not at the table.

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[2] Memorandum of Understanding for Hydropower Among the Department of Energy, the Department of Interior And the Department of the Army, March 24th, 2010

FERC Streamlining Process

On April 15th, 2010, FERC also announced that it was working to streamline the administrative process for small hydropower projects. It also said it was updating a Memoranda of Understanding with other agencies to improve coordination and avoid duplication of efforts, along with a new outreach and education program for potential small hydro developers.

Chairman Jon Wellinghoff said: "To address the US's energy challenges, we must ensure that we are both making the most efficient use of our existing hydropower resources and promoting smart investments in new hydropower resources and innovative technologies."

Commissioner Philip Moeller related: "Small and micro hydropower has enormous potential, but these projects often cannot be developed under traditional licensing methods. By our action today, the Commission is working to ease the regulatory burden of harnessing this clean and renewable form of energy."

Vermont Calls Hydroelectric Dams Renewable[3]

In a move called both a dangerous precedent and a financial victory, Vermont declared that power generated from large hydroelectric dams counts as a renewable resource in June of 2010.

The bill signed into law by Republican Gov. Jim Douglas generated applause among utilities, which said they would be able to provide lower-cost electricity to Vermont's customers from Hydro-Québec, the Canadian supplier of the state's hydropower. Hydroelectric facilities currently provide about a third of the state's electricity.

Hydro's Star Rises in D.C.[4]

The drumbeat for renewable hydropower is growing louder. The Administration has directed the Corps of Engineers and the Energy and Interior departments to work together to facilitate its development. Bipartisan legislation promoting hydro has been introduced in the Senate and hearings are now being held. Hydro is finally getting its due!

In March, Secretary of Energy Steven Chu said, "I'm for hydropower because I'm an environmentalist."

Murkowski Legislation [4]

The Hydropower Improvement Act (S. 3570) says the United States should "substantially" increase the capacity and generation of clean, renewable hydropower. Among other things, it directs the Department of Energy to make competitive grants for

[3] Climate Wire Trial (06/08/2010) Christa Marshall, E&E reporter
[4] Current Reflections (July – 2010) Terry Flores

existing hydro facilities' efficiency and capacity improvements and to add generation at dams that currently do not have it. It also gets on the bandwagon for pumped storage and developing hydropower in existing conduits, like irrigation channels.

A second bill, the Hydropower Renewable Energy Development Act of 2010 (S. 3571), expands the definition of renewable energy to include hydropower, including conventional hydro like our Northwest dams. These bills have bipartisan support. Cosponsoring S. 3570 are Senators Murray and Cantwell of Washington, and Senators Crapo and Risch of Idaho.

Water and Power Subcommittee

On July 29, the House Water and Power Subcommittee held a hearing on *"Investing is Small Hydropower: Prospects of Expanding Low-Impact and Affordable Hydropower Generation in the West."* The hearing indicated that there is broad, bipartisan support for measures that will remove barriers and promote development of small hydropower. Republicans on the subcommittee also used the opportunity to vociferously denounce efforts to remove dams on the Snake River and elsewhere.

Rep. Adrian Smith (R-NE) cited the experience of the Buckeye Irrigation District in Arizona, which spent a lot of money and almost nine months getting a FERC exemption for its proposed small hydro project. Smith noted that FERC is trying to speed up its exemption process, and said he planned to introduce a bill that would exempt all small scale project 1.5 MW or less from the FERC licensing requirements.[5]

The Small-Scale Hydropower Enhancement Act of 2010

On July 29th, 2010, Congressman Adrian Smith from Nebraska introduced H.R. 5922. The "Small-Scale Hydropower Enhancement Act of 2010" intends to exempt any conduit-type hydropower project generating less than 1.5 megawatts from FERC jurisdiction.

State of Colorado Signs MOU with FERC

Developed as a pilot program, FERC and Colorado sign a MOU on August 24th, 2010 allowing Colorado latitude in streamlining small conduit and small project exemptions.

FUTURE

With the political upheaval of the November 2010 elections, it will be interesting to see how everything is resolved. Regardless of the outcome, we are all encouraged that both sides can agree on the need for increased hydro electric generation.

[5] Memo, Morgan Meguire LLC, Debroah Sliz, 7/30/2010

REFERENCES

Flores, Terry, Current Reflections, July / 2010

Marshal, Christina, Climate Wire Trial, 06/08/2010

Sliz, Debroah, Morgan Meguire LLC Memo, 7/30/2010

LOW-HEAD HYDRO CASE STUDY — SEC PROJECT

Donovan Neese, P.E., MBA[1]
Jennifer M. Torpey[2]

ABSTRACT

The changing mandates in the energy industry have increased the demand for renewable energy. Irrigation districts looking to generate renewable energy have an opportunity to exploit a relatively untapped resource, i.e., the hydropower generation potential of their own canal systems. This case study discusses one irrigation district that was able to make use of this resource and how they did it.

Buckeye Water Conservation and Drainage District identified a site on its canal system suitable for the installation of a low-head hydroelectric generation facility. K.R. Saline & Associates, PLC and Stantec Consulting Services Inc. were retained by the District to install a low-head hydroelectric engine designed and built by Natel Energy, Inc. Integrating the regulatory process with the civil process required an additional level of collaboration between the project participants, both pre- and post-installation. In addition, reconciling the timelines of the two processes proved challenging--installation could not commence until a small conduit exemption was granted by the Federal Energy Regulatory Commission. Following installation, the project faced the final hurdle of interconnection with the local utility, which required compliance with an additional layer of standards and specifications, as well as follow-up interaction with the Federal Energy Regulatory Commission.

Ultimately, the successful conclusion of the project created a facility which is part of a new wave of clean, renewable low-head hydropower generation. The experience of developing a project of this nature and the ability to recognize potential pitfalls will prove invaluable in the development of future low-head hydropower projects.

INTRODUCTION AND BACKGROUND

Project Participants

In recent years, volatile wholesale energy prices and a changing business climate have made alternative sources of energy more attractive and affordable. One of the resources enjoying re-emergence is low-head hydroelectric generation. In 2007, low-head hydro developer Natel Energy, Inc. ("Natel") began exploring the opportunity of an installation in Buckeye, Arizona. Their partner in the endeavor was the Buckeye Water Conservation and Drainage District ("BWCDD" or "the District").

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BWCDD is an irrigation and water conservation district established under Title 48 of the Arizona Revised Statutes in 1922 with a history dating back to 1887. The District owns and operates water delivery facilities, including a canal known as the South Extension Canal. As a part of its regular system improvements, BWCDD identified an irrigation drop structure on the South Extension Canal that was in need of repair and permanent improvement. Concurrently, BWCDD and Natel were discussing the potential for installing a low-head hydroelectric generating facility in the District canal system. With the opportunity for accomplishing two goals at once, BWCDD forged ahead with the design of a low-head installation, referred to as the SEC Project ("the Project").

The first order of business was identification of the parties who would be involved. On the civil side of the Project, the District's on-call civil engineering firm Stantec Consulting Services Inc. ("Stantec") was engaged. Stantec was to be responsible for all civil design work and production of drawings. On the regulatory side, energy consulting firm K. R. Saline & Associates, PLC ("KRSA") was retained. KRSA's role was to identify the process the District would need to go through in order to obtain regulatory approval for the Project and to coordinate preparation of the necessary paperwork.

Project Description

Physical Specifications. As installed, the Project consists of a 12-foot-long, 3-foot-square steel penstock leading to a Schneider Linear HydroEngine unit having an installed nameplate capacity of 18.65 kW, and appurtenant facilities. As noted above, the Project is located on the District's existing irrigation canal system. The water in the irrigation canal flows south through an upper irrigation canal, over a 10- foot-high irrigation drop, and into a lower irrigation canal. The irrigation drop contains two automated gates, which control the normal flow of irrigation water through the canal. The Project diverts water from the upper canal through one of the automated gates into the penstock. The water in the penstock then drops approximately 5 feet to the generator. After exiting the generating unit, the water goes through a draft tube into the lower canal. All of these facilities are housed in a concrete vault.

Vault Installation. The Project site prior to construction consisted of a masonry unit drop, as depicted in Figure 1.

Figure 1. Original Drop Structure

The new hydro-generation vault sits just downstream of the original drop structure, as shown in Figure 2.

Figure 2. Installed vault prior to generator installation

The precast vault houses the engine, valves, conduits and power inverter. The interior dimensions of the vault are 9’ high, 10’ wide and 16’ long with two access hatches in the top. One access hatch is positioned over the intake valves and other is over the engine for convenient maintenance. The two inlet openings on the upstream end are connected with square conduit to the openings on the downstream end to convey flow either through the engine or through the by-pass. The valves maintain a constant upstream pool elevation, but a control box mounted to the top of the structure offers a manual override. The channels upstream and downstream were lined to accommodate the pools necessary for the engine to operate. The majority of the construction was performed by the District and proceeded with few complications.

Electrical Interconnection. An electrical transmission line runs parallel to the downstream channel. The power line is owned and operated by Arizona Public Service Company ("APS"). Conveniently, connection to the line did not require extensive infrastructure. The proposed connection required an interconnect permit and submission of electrical drawings showing interconnect wiring and equipment. APS then required an inspection of the installed facilities prior to final connection. The final interconnection layout is shown in Figure 3. Permitting and plan review through the local power company was the final process to Project success.

Figure 3. Electrical Interconnection

Regulatory Requirements

The nature of the installation necessitated approval from the Federal Energy Regulatory Commission ("FERC") for a small conduit exemption. A small conduit exemption applies to facilities (not including a dam or other impoundment) constructed, operated, or maintained for the generation of electric power and located on non-Federal lands. It must use for generation only the hydroelectric potential of a manmade conduit, which is operated for the distribution of water for agricultural, municipal, or industrial consumption (and not primarily for the generation of electricity). Small conduit exemptions are limited to 15 MW or less for non-municipalities, and 40 MW or less for municipalities.

A small conduit exemption follows a specific timeline and contains specific components, all outlined in the Code of Federal Regulations (“C.F.R.”). A multi-stage public process is also required, although it is possible to compress some of the process if adequate consultation has occurred in earlier phases, and if agreement can be obtained from all affected resource agencies.

PROJECT CHALLENGES

Collaboration

The first challenge of the Project was coordination of the efforts of all the parties involved. Working towards the installation were the District, Stantec, KRSA and Natel. Each party then had its own contacts and organizations with whom it had to interact in order to achieve its individual goals. Managing the diverse efforts of the multiple parties involved was challenging. The District had to manage its expectations for budgeting, allocation of manpower, and meeting its own timeline for when installation could be completed. Stantec had to develop construction drawings that would retrofit the existing District drop structure, with said drawings eventually becoming exhibits to the exemption application. KRSA had to complete the required application preparation, public outreach, and interaction with FERC.

Regulatory Compliance

The second challenge was compliance with all the regulatory requirements associated with a small conduit exemption. Generally speaking, an application for a small conduit exemption is required to include:

- An Introductory statement;
- Exhibits known as A, E, F and G;
- An Appendix containing documentary evidence showing that the applicant has the real property interests for the project site; and
- Identification of all Indian tribes that may be affected by the project.

The Introductory Statement provides basic information on the applicant and the location of the proposed project. Exhibit A must describe the project in detail while Exhibit E is an environmental report. Exhibit F is a set of drawings showing the structures and equipment of the small conduit hydroelectric facility, and Exhibit G is a map of the project and boundary. The form and content of these items is dictated by Title 18 Chapter 1 Part 4 of the C.F.R. However, there is an inherent challenge in reviewing the regulations, as multiple types of hydro projects share certain core requirements. KRSA had to thoroughly review the relevant sections to ensure all requirements pertaining to small conduit exemptions were met while superfluous or irrelevant requirements disregarded. KRSA also had to apprise Stantec of the portions of the regulations that pertain to Exhibit F and G. This involved extensive collaboration with Stantec on what FERC specifically expected for the drawings. In addition, few exemptions were available for use as guidelines since only 11 small conduit exemptions had been granted in the five years immediately prior to the time the Project’s application was prepared. Further narrowing of the available examples was also necessary, to identify those which were similar enough to the SEC Project to be useful, as well has have been identified by FERC as good models.

Furthermore, as noted above, there are specific time lines required for a project of this kind, including a public process. According to regulations, consultation with affected agencies must occur before the application for exemption can be filed with FERC. Time must be allotted for comments to be submitted by the affected agencies, and for any comments to be addressed. A public meeting, to which the affected agencies must be invited, must also be publicly noticed and held, and be audio recorded or documented through written transcripts. Upon completion of this first stage of consultation, a second stage is required. If approval is obtained from the affected agencies, the second stage can be eliminated; however, this is not guaranteed. A third stage of consultation is also required, and is initiated when the application is filed and accepted by FERC. A period of time is then required to allow any final comments to be made before an exemption can be issued.

Preparation of the actual application took several months of data collection and organization while consultation with jurisdictional agencies was simultaneously occurring. Although the relatively limited scope of the Project meant that extensive collaboration with external agencies was unnecessary, KRSA and the District were required to communicate with U. S. Fish & Wildlife, Arizona Game & Fish, the State Historic Preservation Office, and the Arizona State Museum. Fortunately, no issues of concern were identified by any of the agencies, and no mitigating measures were necessary aside from the agreement to cease construction if archaeological remains were discovered.

Appropriate Drawings

Exhibit F to a small conduit exemption application should consist of general design drawings and supporting information used as the basis of design[3]. They must show all major project structures with sufficient detail to provide a full understanding of the project, including: (i) plans (overhead view); (ii) elevations (front view); (iii) profiles (side view); and (iv) sections.

Exhibit G to a small conduit exemption application is a general site map showing the location of the project, project boundary and principal features[4] (including land ownership details), and must be stamped by a registered land surveyor. Each sheet of Exhibit G must show three reference points labeled with latitude/longitude or state plane coordinates. The project boundary encloses all of the project works required for operation and maintenance of the project as well as the impoundment area. In addition, Exhibit G must be submitted electronically in a geo-referenced format such as an Arcview shape file.

[3] See 18 C. F. R. § 4.41 (g).
[4] See 18 C. F. R. § 4.41 (h).

Both Exhibit F and G are subject to specific additional requirements outlined in 18 C. F. R. § 4.39. Particularly, both Exhibit F and G must be produced on silver or gelatin 35mm microfilm mounted on Type D (3 1/4" by 7 3/8") aperture cards. This is in addition to full size prints or electronic files submitted. Furthermore, the aperture cards, prints and electronic submittals must be prepared for both the original application submission, and for the as-builts submittal made following the conclusion of construction.

Due to budgetary and time constraints, preparation of Exhibits F and G required Stantec to retrofit existing documents used in the construction of the Project. This entailed multiple changes and modifications to meet all of FERC's requirements. Careful review of the regulations and ingenuity was necessary in order to accommodate both purposes. Figure 4 depicts the Exhibit G submitted as part of this Project.

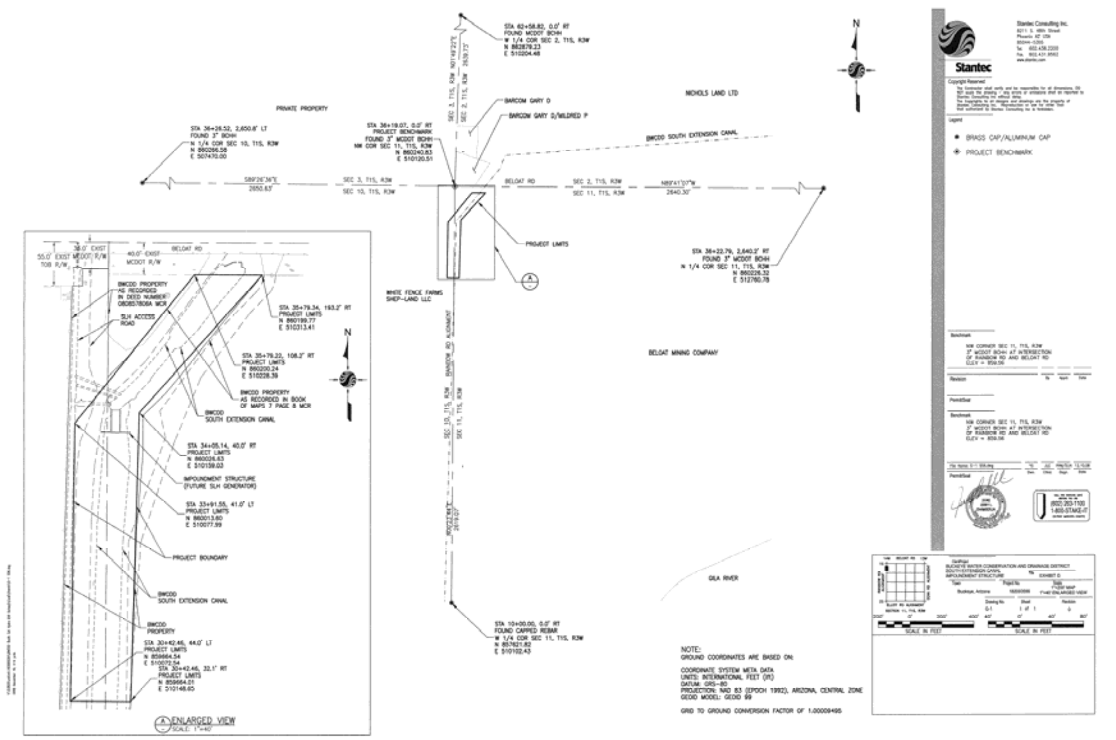

Figure 4. Exhibit G – Project Boundary Map

Scheduling Conflicts

The largest challenge was integration of the different schedules involved. First, the District had a specific window in which the hydro unit could be installed, during its two-week long seasonal dry-up. All construction needed to take place during this timeframe. All materials needed to be on-site prior to the dry-up including the automated valves, valve control system, by-pass conduit and the pre-cast vault. Equipment also had to be reserved to install the vault and grade the site. The schedule constraints did not allow the District much flexibility, and meant that all FERC approvals had to be obtained before this window.

Second, the FERC exemption submittal process extended the project schedule significantly. The initial application for exemption was submitted to FERC in December 2008, with the conclusion of the public process submitted in February 2009. Although the District was able to bypass the second stage of consultation and proceed to the third stage by filing the application, there was still a significant delay between the time the application was filed and when the exemption was granted. During this time, FERC took no formal action on the application aside from accepting it for filing. No official communications were made to the District or to KRSA explaining the delay or requesting clarification. It was only through extensive follow-up by both parties that the source of the delay was discovered: FERC had concerns over how the irrigation drop structure was "named" in the application. Although the Project was well and clearly within FERC's requirements, this led them to withhold approval of the application until their internal concerns were addressed.

Even following issuance of the exemption, final drawings and a construction work plan were required by FERC, and formal authorization to construct would not be granted until they were approved by FERC. This required the District to interact with the appropriate FERC regional office, which had its own specific protocols and requests. Many of these requirements were not known by the District until after the exemption was granted; the information was generated directly by the FERC regional office and was not published in the C.F.R. Finally, interconnection still had to be obtained with the local utility. This necessitated the production of more drawings and the completion of yet another application, which was, of course, specific to the utility involved.

All together the Project began in mid 2007 and was not completed until early 2010. The sequence of events progressed as follows:

- June 2007 — Natel met with BWCDD to discuss options.
- December 2007 — Site survey completed by Stantec.
- July 2008 — Site civil design initiated.
- December 2008 — Draft exemption application submitted to FERC.
- February 2009 — Final exemption application submitted to FERC.
- September 2009 — FERC granted the exemption.
- December 2009 — Vault construction completed.
- February 2010 — Electrical drawings submitted to APS.
- April 2010 — APS granted permission to parallel the grid.
- May 2010 — Commissioning event held.

Although the process spanned three years, the ideal schedule for a project of this kind would have been half as long, about eighteen months.

LESSONS LEARNED FOR FUTURE PROJECTS

Site Selection

Site selection is critical. Prospective generation sites must consider the following items:

- Municipal Jurisdiction
- Project Boundary Impacts

- Proximity to Electrical Transmission Infrastructure
- Potential Hydraulic Drop
- Conveyance Flow Rates
- Local Power Company Requirements
- Potential environmental issues
- Potential archaeological impacts
- Cost of permitting and construction

Once a site is chosen, planning and scheduling the installation should also reconsider each of these issues.

Realistic Schedule

In developing a project of this kind, it is imperative to have a clear understanding of the various timelines and potential pitfalls that could interfere with time-specific goals. This includes allowing sufficient time for pre-collaboration before a project application is even filed, as an applicant has a better chance of obtaining FERC authorization if there are no objections filed by affected agencies and a clean application on the environmental front. It is also important to consider the regional office requirements and interconnection requirements of the local utility, as they will add time and work following granting of an exemption. Finally, it should be noted that FERC is not required to act on applications within any given timeframe. However, in recent months FERC has redoubled its efforts to promote and streamline the processing of small conduit exemptions and other low head hydro projects, which has included the goal of issuing an exemption or license within a relatively short window of the filing of an application.[5]

In retrospect, the duration of the process is not surprising for a pilot project where the parties involved were unfamiliar with the processes. For future projects, a schedule more on the order of a year and a half would not be unrealistic. The engineering process and municipal review takes approximately five months, while concurrently initial resource agency consultation could be occurring. Following the agency consultation the exemption application could be filed. During the FERC review process the site construction can begin, interconnection design completed and installed and the site is simply waiting for the engine to be installed. Once complete the local power company must inspect the interconnect and grant permission to parallel the grid.

Communication

Communication of the exact nature of the Project is key. Making it clear to resource agencies, to FERC, and to those individuals working on the Project what exactly was needed or intended was the only way to make the Project happen. As noted above, FERC has since made steps to improve its own resource information and customer outreach, and has set its own goals for improving the turn around on small conduit exemptions. However, it behooves the applicant to do all their homework first and do

[5] See http://www.ferc.gov/industries/hydropower/gen-info/licensing/small-low-impact.asp for FERC resources.

whatever is necessary to ensure a smooth application process, including communication with FERC before an application is even filed. Any hiccup will only result in costly delays.

CONCLUSION

The CEO of Natel Energy has been quoted as saying that low-head hydro-electric power is the low hanging fruit in this energy crisis. Much of this power could be captured off of exising infrastructure from diversion dams, irrigation canals or water treatment plant outfalls[6]. The future of low-head power development is being ushered along through federal grants and regulatory changes are on-going to streamline the application process.

The current interest in alternative energy sources and increased funding opportunities has created an environment friendly to low-head hydropower. With a clear vision, adequate preparation, and patience, achieving the successful installation of a low-head hydro project is very feasible and wasted head can be turned into a revenue stream. Assembling a qualified, knowledgeable team and taking full advantage of resources available to potential applicants will increase the probability of your project success. With these tools in hand, irrigation districts can turn the hope of a low-head hydro installation into reality.

[6] Testimony of Gia Schneider before the Committee on Science and Technology, Subcommittee on Energy and Environment, U.S. House of Representatives, "Investment in Small Hydropower: Prospects of Expanding Low-Impact and Affordable Hydropower Generation in the West", July 29, 2010.

THE FRESNO LOW-HEAD HYDROPOWER PROJECT — A CASE STUDY

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Brock Buche, PE[3]

ABSTRACT

The City of Fresno Water Division is planning to develop a low-head hydropower project on a new 60-inch diameter pipeline that delivers water to their surface water treatment plant. The hydropower plant will take advantage of excess pressure at the pipe terminus and generate electricity for use at the treatment plant. Flows through the pipeline will be a constant 46 cfs, with an excess head of 40 feet, from 2011 to 2020. The flow will increase instantaneously to 93 cfs, and the excess head will reduce to 18 feet, when the treatment plant is expanded in 2020. The City was challenged in finding a low-head turbine that can accommodate both flow and pressure conditions, while maintaining high efficiency. Seven turbine configurations were evaluated that considered variations in the number and type of turbines, operating during one or both of the two flow conditions, operating at low efficiencies during one flow condition, and modifying or replacing turbines when flow conditions change. The final configuration includes a 130-kw tubular unit with adjustable runner blades that can accommodate a range of flow and head conditions. Due to a wide range of possible economic, hydrologic and design conditions, two economic analyses were performed including a conservative and less conservative case. The benefit cost ratios ranged from 0.9 to 3.2. Other project challenges include finding a domestic turbine supplier, evaluating net metering opportunities, and seeking government grants for low-head hydropower. The project is still in the planning and design stage.

INTRODUCTION

In 2008, the City of Fresno, California evaluated several alignment corridors and performed a preliminary design for a 'Raw Water Pipeline' from the Friant-Kern Canal to the City of Fresno Surface Water Treatment Facility (SWTF). The study recommended a 60-inch diameter pipe to accommodate future anticipated flows of 60 MGD, and also to match the diameter of a limited section of the Raw Water Pipeline that had already been constructed under a new development. Flows are anticipated to be 30 MGD from 2011 to 2020, and then increase to 60 MGD. Under both flow scenarios, the water will have excess energy (head) at the pipeline terminus, providing an opportunity to generate hydropower.

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In 2009 the City of Fresno performed a study to evaluate the potential for installing a small hydroelectric powerplant at the terminus of the Raw Water Pipeline. The goal is for the powerplant to take advantage of excess head in the pipeline and generate power for use at the treatment plant.

This paper discusses the technical and economic challenges in developing a small in-line hydropower project. The primary challenge was finding a turbine and project configuration that could accommodate the instantaneous change in head and flowrate expected in 2020 when the treatment plant is expanded.

SITE DESCRIPTION

Fresno Surface Water Treatment Facility

In 2004, the City of Fresno completed construction of a Surface Water Treatment Facility (SWTF). The SWTF began treating surface water in June 2004 and currently delivers between 15 and 30 percent of the water supply to the City's water distribution system. The City had previously relied solely on groundwater for its potable water supply. The SWTF has a capacity of 30 MGD, and is expected to be expanded to accommodate 60 MGD by 2020. The proposed Raw Water Pipeline will terminate at the SWTF.

Proposed Raw Water Pipeline

The SWTF is currently supplied with Kings River and Central Valley Project (CVP) water conveyed by the Fresno Irrigation District's Enterprise Canal. Raw water from the canal is diverted under gravity flow to the SWTF raw water pump station and is then pumped to the water treatment headworks. Due to capacity limitations and water quality concerns in the Enterprise Canal, the City is proposing to construct a 60-inch diameter pipeline directly from the Friant-Kern Canal to the SWTF. The pipeline will be called the Raw Water Pipeline. The proposed Raw Water Pipeline will replace the Enterprise Canal as the primary conveyance facility to the SWTF. Several pipeline alignments from the Friant-Kern Canal to the SWTF were considered (see Figure 1). A short section of the pipeline has already been constructed under a recent school development. This was done early to avoid disturbing the new school facilities when the entire pipeline is constructed. A 60-inch diameter pipeline was selected for this section. Consequently, a 60-inch diameter pipeline was also selected for the entire pipeline. The size of the pipeline allows the water to have excess energy (head) when it reaches the SWTF. The proposed powerplant would be located at the treatment plant where the Raw Water Pipeline will terminate.

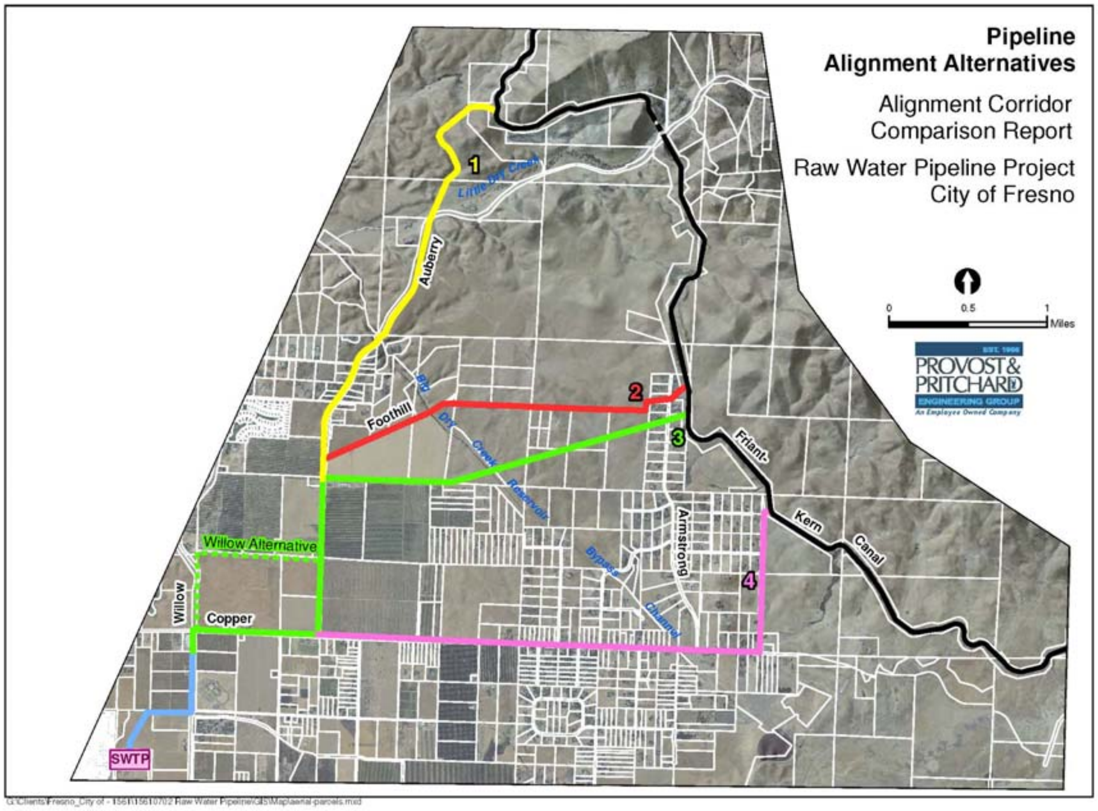

Figure 2. Pipeline Alignment Alternatives

ENERGY USAGE AND DEMANDS

The proposed powerplant will provide approximately 80,000 to 90,000 kwh/month. In comparison, monthly energy demands at the SWTF varied from 326,500 kwh to 843,000 kwh in months when water was treated. Future energy demands will approximately double when the volume of water treated is increased from 30 MGD to 60 MGD in 2020. Therefore, the hydropower plant could only provide a portion of the SWTF's energy demands, but it could offset a significant portion of the electricity bill for the SWTF.

DESCRIPTION OF ALTERNATIVES

Flows to the treatment facility are expected to increase from 46 cfs to 93 cfs in 2020. At the same time the excess head will decrease from 40 to 18 feet. This will require that the initial turbines installed in 2011 be replaced or modified in 2020. A variety of turbine installation alternatives were considered to accommodate these changes and are listed in Table 1**.**

Table 1. Turbine Installation Alternatives

Option	Description	0-10 Years	After 10 Years
1	Operate 46 cfs Turbine for 10 Years	Install 46 cfs turbine	Remove and salvage powerplant
2	Install 46 cfs Turbine and Divert Excess Flows through Bypass	Install 46 cfs turbine	Keep in place and operate at lower efficiency, divert new 46 cfs through bypass line
3	Install 46 cfs Turbine and Modify in 2020	Install 46 cfs turbine	Modify turbine to accommodate higher flow and lower head
4	Install 46 cfs Turbine in 2011 and Second 46 cfs Turbine in 2020	Install 46 cfs turbine	Install second 46 cfs turbine
5	Install 93 cfs Turbine in 2011	Install 93 cfs turbine and operate at low efficiency	Continue operating 93 cfs turbine now at higher efficiency
6	Install Turbine that Can Accommodate Range in Conditions	Install single turbine that can accommodate full range of flows and heads	Continue using turbine that can accommodate full range of flows and heads
7	Install 93 cfs Turbine in 2020	Nothing	Install 93 cfs turbine

Option 6, Install Turbine that Can Accommodate Range in Conditions, was identified as the most practical and economical alternative. A discussion on each of these alternatives is provided below.

Option 1 – Operate 46 cfs Turbine for 10 Years

This option includes installing a turbine that can accommodate the low flow, high head conditions (46 cfs and 40 feet). When the flowrate changes in 2020, the system would be removed and salvaged. No attempt would be made to generate electricity after 2020. The economic analysis shows that the project cannot be paid off in less than 10 years, even under the most optimistic assumptions presented. Furthermore, the salvage value of the turbine would probably be low, and it could be difficult to find someone looking for a used turbine of the same size. As a result, this option was eliminated from consideration.

Option 2 – Install 46 cfs Turbine and Divert Excess Flows through Bypass

This option would only generate power using 46 cfs under both flow conditions. After 2020, the new 46 cfs of flow would be diverted, thus missing the opportunity to generate some potential hydropower. This option would not be economical because the revenue would be too low after 2020, and therefore was eliminated from consideration.

Option 3 – Install 46 cfs Turbine and Modify in 2020

This alternative is similar to Option 6, but this alternative would involve major modifications in 2020, while Option 6 would require minor or no modifications in 2020. This alternative was eliminated from consideration because a turbine was found that could accommodate the range in flows and heads with only minor modifications (Option 6).

Option 4 – Install 46 cfs Turbine in 2011 and Second 46 cfs Turbine in 2020

This option would require the installation of two separate turbines. This would incur higher capital costs than installing a single turbine (Option 6) and therefore was eliminated from consideration.

Option 5 – Install 93 cfs Turbine in 2011

Under this option the turbine would be designed for 93 cfs, operate at a low efficiency from 2011-2020, then operated at a high efficiency after 2020. This option was also eliminated from consideration because no turbine was found that can operate under both flow conditions without modifications.

Option 6 – Install Turbine that Can Accommodate Range in Conditions

This option would be the most economical because it would minimize change-over costs in 2020 when flow conditions change, and the turbine would operate at fairly high efficiencies under the two different flow conditions with only minor modifications. A turbine that can accommodate both flow conditions was found and was used in the economic analyses.

Option 7 – Install 93 cfs Turbine in 2020

Waiting until 2020 to install a turbine is not necessary because a turbine was found that can accommodate the range in flow conditions (Option 6). In addition, it would probably be more difficult to connect a hydropower plant to the Raw Water Pipeline in the future, when it is operating, than in 2011, while it is being constructed. Therefore, this option was dropped from consideration.

POTENTIAL ENERGY GENERATION

Energy generation at the proposed powerplant was calculated using flow data, excess hydraulic head values, and equipment performance data. The energy generation was estimated under both conservative and less conservative assumptions for comparison purposes. The assumptions used in each analysis are show in Table 2. Both scenarios are considered to include reasonable assumptions.

Table 2. Energy Generation Assumptions

Description	Conservative Case	Less Conservative Case
Flowrate (2011- 2019)	30 MGD (46 cfs)	30 MGD (46 cfs)
Flowrate (2020+)	60 MGD (93 cfs)	60 MGD (93 cfs)
Available Head (2011-2019)	40 feet	40 feet
Available Head (2020+)	17 ft to 14 ft[1]	18 feet
Turbine Efficiency	90%	90%
Generator Efficiency	90%	95%
Powerplant Downtime	3%	2%
Water Supply Availability	94%	100%
Inflation of Energy Costs above Overall Inflation	0%	0.5%/year

1 – The available head is assumed to decline 1 foot every decade

For the purpose of this analysis, a project life of 50 years was used. However, with proper maintenance, the power plant should perform well beyond a 50-year period.

The potential energy generated for the conservative scenario is about 1,000,000 kwh/year in 2011 tapering down to about 710,000 kwh/year after 50 years, due to an increase in pipe roughness. For the less conservative case, the generation is about 1,100,000 kwh/year the first ten years, and then a steady 1,040,000 kwh/year for the remaining 40 years. A summary of the estimated revenue is provided in Table 3. The revenue is based on 2009 electricity and demand rates for Pacific Gas & Electric utility.

Table 3. Estimated Annual Revenue

Years	Conservative Case	Less Conservative
2011-2020	$102,000	$118,000
2020 -2030	$88,000	$112,000
2030-2040	$82,000	$118,000
2040-2050	$77,000	$123,000
2050-2060	$72,000	$128,000
Average	$84,000	$120,000

PROJECT DESIGN

Following is a discussion on the project's civil, mechanical and electrical facilities. A schematic drawing of the design can be found in Figure 2.

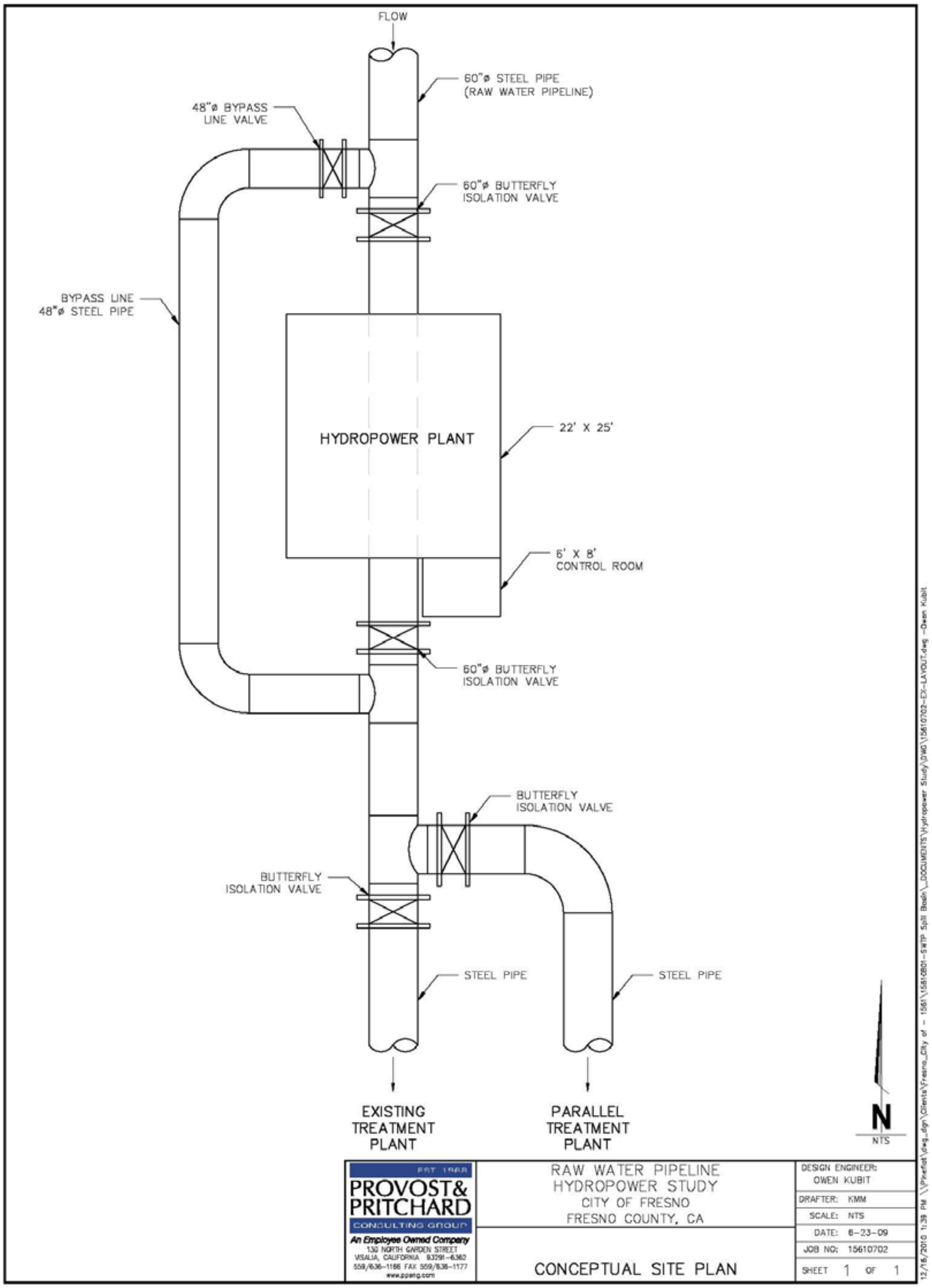

Figure 1. Conceptual Site Plan of Hydropower Plant

Turbine

Turbines are normally selected based on very specific characteristics of the site for optimum performance. The initial and final flow/head conditions for this site fit two different categories from the manufacturer's perspective. The initial condition (40 feet of head) falls in a low-head category while the final condition (18 feet of head) is in the very low-head category. Significant research was performed to find a domestic turbine manufacturer. Nine companies were contacted for equipment and cost information. The City found that turbine sales engineers often have little or no interest in customers seeking their smaller units, and are more focused on selling large turbines. In addition, most companies did not make or supply equipment that would meet the site conditions of this project, or could only meet the initial flow/head conditions. Turbines meeting the head and flow requirements therefore must be found from foreign suppliers. The lack of a domestic manufacturer is probably attributed to the minimal development of small scale hydropower in the United States, and lack of government incentives to develop small hydropower during the last 20 years. France, England, Japan and China are a few countries that have worked extensively with low-head hydropower, particularly due to government promotion, and are more likely to manufacture smaller turbines. International bidding will therefore be required for the equipment. Bidding, design, manufacturing and shipping will require at least 16 months.

Voith Hydro of Japan, a joint venture of Voith Siemens Hydro Equipment makes a turbine that will operate under both the initial and final flow/head conditions with only minor modifications when conditions change in 2020. That appears to be the most economic equipment to use for this site and, therefore, was used to establish the economics for this feasibility study.

The turbine manufactured by Voith Hydro is a tubular unit (L-type) with adjustable propeller blade runner. The turbine drives the generator with a timing belt rather than a gear assembly, which makes it easier to change speed ratios necessary for the two flow/head conditions. Maintenance of the turbine includes replacement of the timing belt every six months and pulleys, bearing, and water seals every five years. At the 10-year maintenance period, the pulley size would change to meet the speed ratio necessary for the final flow/head operating condition. This would require adjusting the runners, changing the pulley ratio, and possibly installing different belts.

Other Facilities

Additional facilities in the project design include the following:

- Induction type generator
- Reinforced concrete pit
- 48-inch diameter bypass pipeline with flow control valves
- 60-inch diameter butterfly isolation valves
- Hydraulic power units to operate the valves
- Cinder block powerhouse building
- SCADA system

Connection to Electrical System

The generated electricity could either be used on-site or delivered to the electrical grid for sale. Using the electricity on site is recommended because a grid connection would have no economic advantage at this time and would be time consuming to implement. A detailed list of reasons for using the power on site is provided below:

1. FERC Approval. Connecting to the electric grid would require a permit from the Federal Energy Regulatory Commission (FERC). This would be a time consuming and costly process.
2. Power Purchase Agreement. The City would need to prepare a power purchase agreement with the local electric utility (Pacific Gas and Electric) or another power provider. This too would require additional costs, could be time consuming, and may commit the City to a long-term agreement and performance requirements. In addition, some power purchasers may have little interest in the project due to its low power generation (150 kW).
3. Contract Management. The City would have on-going administrative costs for dealing with the power purchase agreement.
4. Capital Facilities. Connecting to the grid would require additional capital facilities, including a distribution line, step-up transformer, and additional switchgear for line/utility protection.
5. Transmission Losses. Some of the power would be lost as it is conveyed from the powerplant to the grid, which is several hundred feet away.

Based on their current electric rate schedule, the City pays a blended rate of about 10 cents per kwh. The price the local utility pays for wholesale power is negotiated, but, on average, it is about 3.5 to 4 cents per kwh. Higher prices of 5 to 6 cents per kwh can be found on the spot market. The local utility does offer some special rate schedules for renewable energy that offer about $0.10/kwh for a 10-year contract and $0.11/kwh for a 20 year contract. While these rates were competitive in 2009, there is no provision for escalation or inflation, and during the term of the agreement the rates will not increase. As a result, when inflation is considered, these rate schedules will probably be a poor option compared to displacing the electricity on site.

Electrical Facilities

The facility was evaluated on the basis that it will use all power on-site and not feed the grid. The electrical facilities required will include a feeder to the nearest load center which has the capacity and voltage corresponding to the capacity of the generator. A protection relay scheme is needed to prevent reverse power flow into the grid, ground fault protection to protect the utility from generator grounds and standard generator protection. No step-up transformer will be needed. An existing motor control center has the ability to receive the 130 kW of generated power.

ECONOMIC ANALYSIS

An financial analysis was performed that considered capital costs, operation and maintenance costs, benefit to cost ratios, various interest rates, and other factors that may impact the project economics in the future. Total capital costs were estimated to be $1.3 million, and operation and maintenance costs were estimated at $16,400/year.

A financial analysis was performed with loan periods of 10, 20 and 30 years and interest rates of 0%, 3% and 5%. Project benefits were estimated based on a 30-year and 50-year project life span. Using these variables, and the conservative and less conservative energy generation assumptions (see Table 2), benefit cost ratios were estimated to range from 0.9 to 3.20. Both the conservative and less conservative case are considered to have reasonable assumptions, suggesting that more detailed investigations are needed to refine the economics. Some important unknowns are foreign exchange rates when the turbine will be purchased, and the future market value of renewable energy.

No viable grant funding was identified for hydropower projects, with renewable energy funding focusing on wind solar and biomass. Grant funding could significantly improve the project economics and reduce uncertainty regarding the benefit cost ratio.

CONCLUSIONS

The following conclusions and lessons can be learned from the investigation:

- No domestic suppliers were found that manufacture turbines that can accommodate the range in flows and head. This probably reflects the low level of small hydropower development in the United States.
- Some turbine sales engineers showed little interest in assisting the design team with a small hydropower project.
- Grant funding for small hydropower is currently lacking, with renewable energy funding focused on other areas.
- The benefit cost ratio varied from 0.9 to 3.2, even though the range in assumptions was considered reasonable. This suggests that more detailed investigations are needed to refine the benefit cost ratio.
- For this project, using the electricity on site is a far superior alternative to net metering due to the low fees paid by the local utility for electricity generated, and the permitting and infrastructure requirements needed to establish a system interconnection
- A variety of alternatives are available for a project that has a range of flow and head conditions. In this case the best alternative was a tubular unit with adjustable propeller blade runner. The turbine drives the generator with a timing belt rather than a gear assembly, which makes it easier to change speed ratios necessary for the two flow/head conditions.

REFERENCES

Gordon, J. L, PE, *Turbine Selection for Small Low-Head Hydro Developments*, Waterpower XIII, July 29, 2003.

Hydro Info Systems, *TURBNPRO Version 3 User's Guide*, 2000.

Navigant Consulting, Inc., *Statewide Small Hydropower Resource Assessment, PIER Final Project Report*, June 2006.

Provost & Pritchard Engineering Group, Inc., *Alignment Comparison Report for the City of Fresno Raw Water Pipeline from the Friant-Kern Canal to the Surface Water Treatment Facility*, Final Draft, May 2008.

Provost & Pritchard Engineering Group, Inc., *City of Fresno Water Division – Raw Water Pipeline Hydroelectric Feasibility Study*, June 2009.

Warnick, C. C., *Hydropower Engineering*, 1984.

CREATING HYDROELECTRIC OPPORTUNITIES WITH IRRIGATION CANAL PIPING PROJECTS

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ABSTRACT

Coupled with increasing energy costs, a growing public interest in renewable energy and water conservation, and increased funding availability for developing sustainable renewable energy projects, there has been a resurgence of interest in irrigation districts to develop hydroelectric power projects. While many of the previously investigated projects were not economically feasible at the time, some projects are now becoming cost-effective, especially when coupled with canal piping projects that are driven by other incentives and benefits. The cost of developing ancillary facilities for multi-function projects can then be shared, thus reducing the capital cost burden for evaluating possible net returns on hydropower development. Balanced against these opportunities, there are several design, operational, and funding challenges to developing hydropower projects that must be carefully considered before embarking on such a project. The factors that affect the feasibility of these projects include the ability to provide suitable flow and head; the added cost of improving upstream pipelines; necessary equipment to control system hydraulic surges; the value of power generated versus cost to develop and maintain the facility; utility requirements for interconnection; power sale agreements offered by the utility; federal, state, and local regulations; and long-term operations and maintenance requirements for operating the facility. In this presentation, these considerations will be discussed in context of two operational hydropower facilities that were developed in conjunction with canal piping projects in the Pacific Northwest: (1) the 0.75 megawatt (MW) Swalley Irrigation District's Ponderosa Station and (2) the 3 MW Yakima-Tieton Irrigation District's Orchard Avenue and Cowiche Plants.

INTRODUCTION

0.75 MW Swalley Irrigation District's Ponderosa Hydroelectric Station

The Swalley Irrigation District has its origins in the late 1800s and serves irrigation water to over 4,000 acres of rural land in central Oregon in the vicinity of the city of Bend. Growth in the community brought urban and suburban development adjacent to a 5-mile portion of Swalley's main canal system, which included a diversion site on the Deschutes River located near downtown Bend. In the early 2000s, multiple stakeholders and Swalley coordinated their efforts to plan the conversion of this 5-mile portion of the open Main Canal to a high density polyethylene (HDPE) pipeline. The goal was to conserve water, reduce the safety hazards associated with open ditches, and eliminate maintenance issues from debris such trash, leaves, and aquatic weeds entering the canals.

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The elevation difference along the canal alignment was great enough that the gravity-pressurized pipeline system would result in pressure of a little less than 100 pounds per square inch (psi) at the end of the 5-mile pipeline. The end of the new pipeline feeds the continuation of the original open canal. A pressure reducing device at the end of the pipeline was required to dissipate or extract that energy before it could be delivered to the open canal. It was desirable to maintain pressure in the pipeline to allow pressurized irrigation water deliveries to upstream customers.

This need to dissipate energy presented an opportunity to develop hydropower as a component of the project. Typically, the expense involved in building a new hydropower site makes such projects difficult to fund on their own unless they can be constructed in conjunction with other features. For example, a hydropower generation facility comparable to the one completed for the Swalley Irrigation District would cost approximate $2 million for the generation facility and approximately $11 million for 5 miles of pipeline needed to transport the water and capture the energy. In addition, if power lines were not present nearby, further expense would be involved to build power lines to the site. In this case, the pipeline component and energy dissipation were needed anyway, and power lines existed fairly near the site, putting all of the pieces in place to incorporate hydropower cost-effectively into the project.

Because uninterruptable seasonal irrigation requirements drove the sequence of construction, the Swalley hydropower project was built in multiple parts over several winter construction seasons. First, an energy dissipater bypass designed to maintain irrigation flows was installed the first winter (2007-2008), when only about 2 miles of the pipe installation had been completed. That first winter the designer recommended that Swalley order long lead-time items such as electrical switchgear, the transformer, and the turbine/generator.

The next winter (2008-2009) additional pipeline was installed. The final winter (2009-2010), the hydropower facility and the most upstream segment of the pipeline were constructed. The facility was commissioned in early spring of 2010. It operated as designed during the 2010 irrigation season.

The Ponderosa Hydroelectric Station has the capability of producing 0.75 MW of power during the peak irrigation flows of 65 cubic feet per second (cfs) that occur during the mid-summer months. Designed using an induction generator with basic automatic controls, the facility is simple for existing District water management staff to operate, requiring no increase in full-time equivalent (FTE) employees. The Swalley Irrigation District has the capability of serving limited amounts of water to its users during the winter months for livestock use. The pipeline and hydroelectric facilities are designed to be operable during cold weather to gain a small amount of additional annual power generation. Maintaining up-time of the generator is a key operational responsibility of the producer to achieve maximum revenue. Operation on peak power sales price days (versus off-peak days) is another consideration for maximizing revenue when the producer has flexibility in the time of water deliveries.

<u>3 MW Yakima-Tieton Irrigation District's Orchard Avenue and Cowiche Plants</u>

In the 1980s, the Yakima-Tieton Irrigation District located in Central Washington undertook a major project to convert their open canal system, which serves 27,000 irrigated acres, to a gravity pressurized pipeline system. With over 900 feet of elevation difference throughout the project, there were opportunities for hydroelectric generation and the need for energy dissipation. The $80 million project entailed installation of over 200 miles of pipelines ranging in size from 2 inches to 90 inches in diameter. At two locations, hydroelectric generators were installed as pressure regulators and energy dissipaters.

The Orchard Avenue Plant contains two 0.75 MW generators, and the Cowiche Plant has one 1.5 MW generator. All three synchronous generators are driven by Francis turbines. The outputs of the plants are connected to the local power utility under interconnection and power sales agreements.

PUBLIC POWER AGENCY AGREEMENTS

Before generating power, the producer needs to identify who will buy the power and at what price. While connection to the power grid may be made through a local public power utility, the power itself could potentially be sold to another company across the country through a "power wheeling" agreement. The supplier and buyer must first develop a power sales agreement. Connecting to the local power system also requires a separate interconnection agreement that spells out the technical details of how the connection will be made, type of equipment, and protection of the utility and supplier systems from each other's electrical systems. Even if the power sales agreement and interconnection agreement involves the same power utility, they typically involve very different parts of the same organization, and separate agreements are needed for power sales and interconnection of facilities.

In the case of the Swalley project, Pacific Power was both the local power utility operator and also purchaser of the power generated. However, the two agreements involved different parts of the company that did not necessarily work together. Fortunately, revenue from power sales can provide a good value for the irrigation district. By federal law, a power company is required to pay the "avoided cost" of the power they buy from renewable sources. This is the rate they would have had to pay if they constructed a new generation plant. The avoided cost rate is a slightly better rate than the standard rate schedule for both peak and non-peak rates. These rates are spelled out in the power sales agreement over some period, typically 20 years, at which point they are renegotiated or re-set based upon current law and market conditions. The rate also usually increases annually (but can decrease) based on the utility's calculations of projected costs.

REGULATORY CONSIDERATIONS

Regulatory requirements can be particularly complicated for these types of hydropower projects simply because they are not constructed very frequently. The agencies involved are often inexperienced in the required processes. On the Swalley project, the current staff at regulators such as Deschutes County had very little experience with this type of project. Because of their rarity, hydropower projects can require modifications to local land use and permitting ordinances to specifically allow power production in a rural setting zoned for agricultural use, something not normally addressed in most local ordinances.

The regulatory requirements involved in developing new hydropower facilities are likely the second most difficult challenge to new projects after cost. One of the common, but not always true, assumptions is that all hydropower projects will negatively impact fisheries since they typically involve natural waterways. In this case, however, the installation of a pipeline in an existing irrigation system resulted in no such impacts. Even though the project had no negative impact to fisheries, fish passage improvement funding by the District was required by state regulators as a condition of project approval. The absence of negative environmental impacts was a significant factor in determining the feasibility and approval of the project.

The Federal Energy Regulatory Commission (FERC) is a major regulator involved in licensing hydropower plants. All new plants that are connected to a public utility must have a facility design and an agreement with the power utility that have been approved by FERC. The application requires detailed, meticulous, and precise organization of the data submitted. The approval process is typically very slow moving. There is a Conduit Exemption option that involves a slightly less rigorous process, as was the case for the both the Swalley and Yakima-Tieton projects.

FERC licensing has a critical impact on project schedule, as the project must have a FERC license or exemption before any equipment can be installed. Because the FERC process was so slow, the approval came down to the last hour to get equipment installed on the Swalley project. The District awarded the construction/installation contract but did not issue a Notice to Proceed (NTP) until all approvals were in hand. FERC approval was required before the Oregon Department of Fish and Wildlife (ODFW) would issue its approvals, and in turn, the Oregon Water Resources Department would not issue the necessary water right for hydroelectric purposes nor would the County issue approval of the Land Use Application until it received ODFW's approvals. It was found that the requirements of the various approving agencies were not necessarily coordinated. A district considering development of new hydropower should have a thorough understanding of the relationships between required permits and build such understanding into the project's critical path schedule, including appropriate provisions in construction contracts to protect the district from potential delay claims that are beyond the control of the district or contractor.

The bottom line is that regulatory approval for this type of project is a complex process that can cause delays and needs to be addressed from the beginning of the project.

DESIGN

The Swalley hydropower facility has a 0.75 MW—or 750 kilowatts (kW)—generator rated for the peak water flow and pressure. It produces 0.7 MW due to minor losses in the system. It uses a Francis turbine connected to an induction generator and is well suited for intermediate pressures (100 psi range) at this flow range. It can power approximately 300 households.

The Yakima-Tieton system also uses Francis turbines. At the Cowiche plant, there is a single 1.5 MW turbine, and the Orchard Avenue plant uses two 0.75 MW turbines, for a total combined output of 3 MW.

Generator choice will be governed by the availability of a nearby reliable power utility grid connection. An induction generator, as used at the Swalley project, basically uses a three-phase electric motor that is turned by a turbine to make power. Turning it slightly faster than its normal running speed produces power that can be delivered to the utility. This type of generator must be connected to a reliable and substantial power utility grid because it requires power input before new generation can occur. This was the generator type used for the Swalley facility. It is preferable because it is less expensive and simpler to operate and build than the synchronous generators used on the Yakima-Tieton project. As operators of an open ditch system, the Swalley operations staff was not experienced in complex electric/electronic equipment so a more straight forward installation was better suited to the District. Also, substantial nearby power utility connections made this a viable choice.

If a substantial power utility grid connection is not available at the site, then the design must use a synchronous generator. The Yakima-Tieton projects use synchronous generators. These could be stand alone systems that can make power without being connected to the power utility grid for power input if the correct governing systems are installed and there is a user for the power. Synchronous generators require more controls that are more complex to operate and therefore are more expensive to purchase and construct, making this a less preferable option. However, if nearby power connections are not available, it is the option that must be used.

Converting an irrigation facility to one that includes a hydropower facility creates unique operational requirements that involve an in-depth hydraulic analysis to prevent flow restrictions or hydraulic transient phenomena. It is recommended that an irrigation district hire an Owner’s technical representative to ensure that pipeline design and hydropower design do not work at cross purposes. In the case of Swalley, the District hired an independent Owner’s Representative through a competitive process to make sure all elements were coordinated properly and served the District’s best interests.

OPERATIONS

Operational requirements for new irrigation district hydropower facilities present both challenges and opportunities. Typically, operations staff at an irrigation district will not have the skills or experience in electric facilities operations and will require training and potentially demand expertise beyond their current capabilities. Where multiple agencies are involved in the project, there is the opportunity to cross-train among facilities—either casually or through formal agreements. For example, the Grand Coulee Project Hydroelectric Authority was formed to address hydroelectric operations for multiple irrigation districts in the Columbia Basin and can be a resource for cross training. In the case of Swalley, the neighboring Central Oregon Irrigation District has qualified staff and is available to work with them to help operate the new facility.

FUNDING

As discussed earlier, most hydro projects are not feasible simply because of the high costs involved, unless they are coupled with another project that will be built anyway to supply water to the hydro station. Additional requirements by funding agencies and regulators can impose specific conditions such as a required completion date and a Commercial Operation date for producing power, making schedule adherence critical. This may require temporary financing by the district to complete the project in a timely manner. Although there was some risk involved, another district neighboring Swalley decided to use a short term “bridge loan” to finance their recently constructed plant until certain incentive funding was released after plant was commissioned. This allowed the bridge loan to be paid off.

There are also a variety of federal and local matching grants and Federal ARRA funding opportunities that were utilized for the Swalley project. The District secured funding from several sources including:

- US Bureau of Reclamation
- Oregon Watershed Enhancement Board (OWEB)
- Oregon Department of Environmental Quality
- National Fish and Wildlife Foundation (NFWF)
- North Rim Foundation

The Deschutes River Conservancy, a non-profit group formed to “restore stream flow and improve water quality in the Deschutes Basin,” was very involved in securing funds from the OWEB and NFWF for the Swalley project. This combination of funding sources made for challenging project accounting and administration, and the District has undergone multiple accounting audits to confirm appropriate use of the funds. An important observation from Swalley’s project is that creative funding acquisition, engagement of environmental advocacy groups to leverage funding from third parties, and rigorous project accounting are often necessary to make a project successful.

It’s important to note that while power sales can help generate funding to pay for a project, irrigation district hydropower plants are not big money-makers. Depending on the scale of the project, hydropower revenue can off-set some of the relatively fixed costs of the district’s operations, thereby helping minimize increases in annual patron assessments. There are also liabilities involved if the plant does not deliver the amount of power defined in the power sales agreement. For example, if a plant could not produce power for a period of time, the district could be considered in default on the power sales agreement. The default remedy costs could be significant and are determined by the current market power purchase rates.

REVENUE

The revenue from a hydroelectric project is highly dependent on the availability of water and the power purchase rates as negotiated in the power sales agreement with the purchaser. Swalley was able to enter into a long term agreement for purchase of all power produced. The quantity of water available is governed by the District's water rights which vary throughout the irrigation season in relation to the demand for irrigation water. Very small amounts of water are delivered to meet the livestock water requirements during the winter months.

The power sales agreement generally sets the power purchase rates for each year. These rates vary depending on the utility's projections of future generating costs and the value of power on a daily and hourly basis. The 2010 rates are $0.0721 per kWh for "peak power" periods and $0.0559 per kWh for "non-peak" times. The non-peak times are defined as legal holidays and nighttime hours.

During 2010, which was the first season of operation, the District generated approximately 2.2 million kWh at a value of about $150,000. The District has not attempted to compute a payback time schedule since the pipeline and hydroelectric projects were mostly funded through grant programs.

CONCLUSIONS

Determining the feasibility of creating a hydropower facility as part of a canal to pipeline conversion project involves several important considerations. First, the cost to integrate the hydropower elements of the project must be quantified, as well as other factors such as the length of the operating season, value of the power, agency approvals, and proximity of an existing connection to the power utility grid. If the feasibility evaluation is positive, then crucial elements of successful project delivery will involve:

- Power sales agreement with the power purchaser
- Interconnection agreement with the connecting utility
- Thorough licensing, permitting, and regulatory coordination planning are needed to ensure minimal negative impacts to the schedule and budget
- Design consideration based on available power supply and operational simplicity
- Funding requirements that may be tied to the completion schedule
- Planning for sustained long-term facility operations and maintenance.

Ultimately, these projects can provide an enhanced asset and additional funding resources to an irrigation district if they are carefully considered and executed. This requires careful project planning and evaluation of project development and life cycle costs prior to embarking on such a project. Evaluation of recently developed similar facilities can help to avoid possible pitfalls along the way.

CASE STUDY OF SMALL HYDROPOWER PLANT BY ORANGE COVE IRRIGATION DISTRICT

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ABSTRACT

The Orange Cove Irrigation District has owned and operated the Fishwater Release Powerplant since 1991. The powerplant is located at the toe of Friant Dam, a Central Valley Project facility that impounds the San Joaquin River. The powerplant is a FERC licensed facility located on United States Bureau of Reclamation (USBR) property. The powerplant diverts water to a horizontal Francis turbine capable of generating 523 kW. The powerplant incorporates an inlet mixing valve, horizontal induction generator, a bypass valve, controls, switchgear and a unit substation. The powerplant generates electricity using a constant 35 cfs flow diverted to the California Department of Fish & Game's San Joaquin River Fish Hatchery located one River mile downstream. Maintaining a constant flow to the Hatchery is crucial for fish health. The powerplant is fed from one or both of two inlets that withdraw water at different reservoir elevations, and can blend the two water sources to optimize discharge temperature according to the Hatchery's needs. The Orange Cove Irrigation District delivers water to the powerplant under a non-consumptive water right. The Fishwater Release Powerplant illustrates a small hydropower project that an irrigation district conceived, designed, constructed and has successfully operated using District staff for over 20 years. The Orange Cove Irrigation District has also successfully developed and operated low-head hydropower on the Friant-Kern Canal, and has investigated an expansion to the Fishwater Release Powerplant using mandated environmental flows.

INTRODUCTION AND BACKGROUND

Landowners within the Orange Cove Irrigation District (District) have been cultivating the land for over a century. From the late nineteenth century settlers have come to this area attempting to provide for themselves and their families. Due to the nature of the soils, availability of water supplies and the other local conditions it is a constant struggle to make a subsistence on farming. It has been engrained on the local growers to constantly look for ways to reduce cost, and embrace opportunities that may allow them to reduce future costs so that farming continues to be a viable economic activity. To this end the District has taken actions in recent years to; modernize the distribution system to allow for on-demand delivery and highly accurate measurement of flows at any

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individual turnout while eliminating losses, constructed hydropower facilities that have performed well and allowed a reduction of annual costs and continues to look for opportunities to invest in environmental projects recognizing that regulations could be one of the largest cost factors in the coming years.

Location

The District is located in Fresno and Tulare Counties, approximately 30 miles southeast of the City of Fresno and 20 miles north of the City of Visalia. A map showing the location is included as Figure 1. The 152 mile long Friant-Kern Canal is the District's main source of water, with 15 turnouts located from milepost 35.87 to milepost 53.32. The District comprises a strip of land approximately 3 miles wide and 14 miles long along the western foothills (first alluvium) of the Sierra Nevada Mountains.

The population within the District is estimated to be about 1,000 people with approximately another 11,000 people living in the City of Orange Cove, which is encompassed within the boundaries of the District but not a part thereof.

For the most part, the terrain slopes to the West at about five to ten feet per mile. Some of the lands, within the coves along the easterly side of the District, have terrain slopes of 15-20 feet per mile. The depth of the alluvium for most of the District area is less than 100 feet. Most soils within the District range in the texture classification from sandy loam to clay loam. The intake characteristics range from moderate to moderately low.

The location of the District lands make it well suited for the crops grown. The area is often referred to as the "Citrus Belt" due to its microclimate adjacent to the Sierra Nevada Foothills. The microclimate is very important for citrus frost protection during the winter months, and is associated with the air movement adjacent to the foothills. Otherwise, the area is typical of the San Joaquin Valley with its hot dry summers and cool damp winters.

Climate

The Climate in the District service area is semi-arid. The average annual precipitation is about 13.5 inches measured at the District office since 1963. The low, which measured 7.61 inches, occurred in 1977 and the high, which measured 26.74 inches, occurred in 1983. The mean temperatures range from 45° F. in January to 80° F. in July, with minimum and maximum temperatures of approximately 25° F and 105° F respectively. The average frost free period is about 252 days occurring ordinarily between March 15 and November 22.

Figure 1. Location Map

History

The District is a political subdivision of the State of California formed for the purpose of delivering water to growers within the district. The present District was organized in February 1937 to comprise an area of 12,587 acres. The Navelencia and East Orosi areas were annexed in March of 1946, and with minor inclusions and exclusions has increased to the present total of approximately 28,000 acres. Included as Figure 2 is a district map showing the boundaries of the District. In the early years the District lands relied exclusively on groundwater pumping for water supply.

The District was formed to import surface water into the area due to fairly extensive cropping acreage reductions that occurred because of inadequate groundwater supplies. In the mid 1930's, an extensive effort was made to get a 250 cfs diversion entitlement from the Kings River. This effort was abandoned when an opportunity arose to contract for the Central Valley Project's (CVP) Friant Division water.

The District entered into a 40-year contract contract for CVP water on May 20, 1949 and started deliveries that same year. The District entered into a renewal contract dated May 23, 1989, again for a 40-year term. On Novermber 18, 2010 the District executed a Repayment Contract that will allow the District to pay its financial obligations to the Federal government by January 31, 2014. The opportunity for this contract change arose from the San Joaquin River settlement agreement which commits the United States Department of Interior to a number of requirements for restoration of an anadromous fishery on the San Joaquin River.

After signing the original CVP Contract in 1949, the District began looking at constructing a water distribution system to deliver water to the landowners. The original plan was to use a system designed by the USBR. However, landowners rejected that proposal because they felt it was too expensive. Many claimed the costs for the CVP Contract and a water distribution system would mean financial ruin for landowners in the District.

As an alternative, the landowners opted to construct their own water distribution system. As a means to do this, the landowners formed individual Improvement Districts under the California Water Code. Ultimately, 23 Improvement Districts were formed wherein each were operated and maintained and records kept as 23 mini-districts.

The water distribution systems selected by the landowners were primarily low-head mortar-joint concrete pipe, a system more typical of an on-farm system than an irrigation district. The life expectance of this type of system is generally accepted to be about 20 years but the District used it for more than 40 years. Maintenance costs were escalating rapidly and at a rate much higher than the normal inflation rate and dependable water deliveries were failing in critical times of the year.

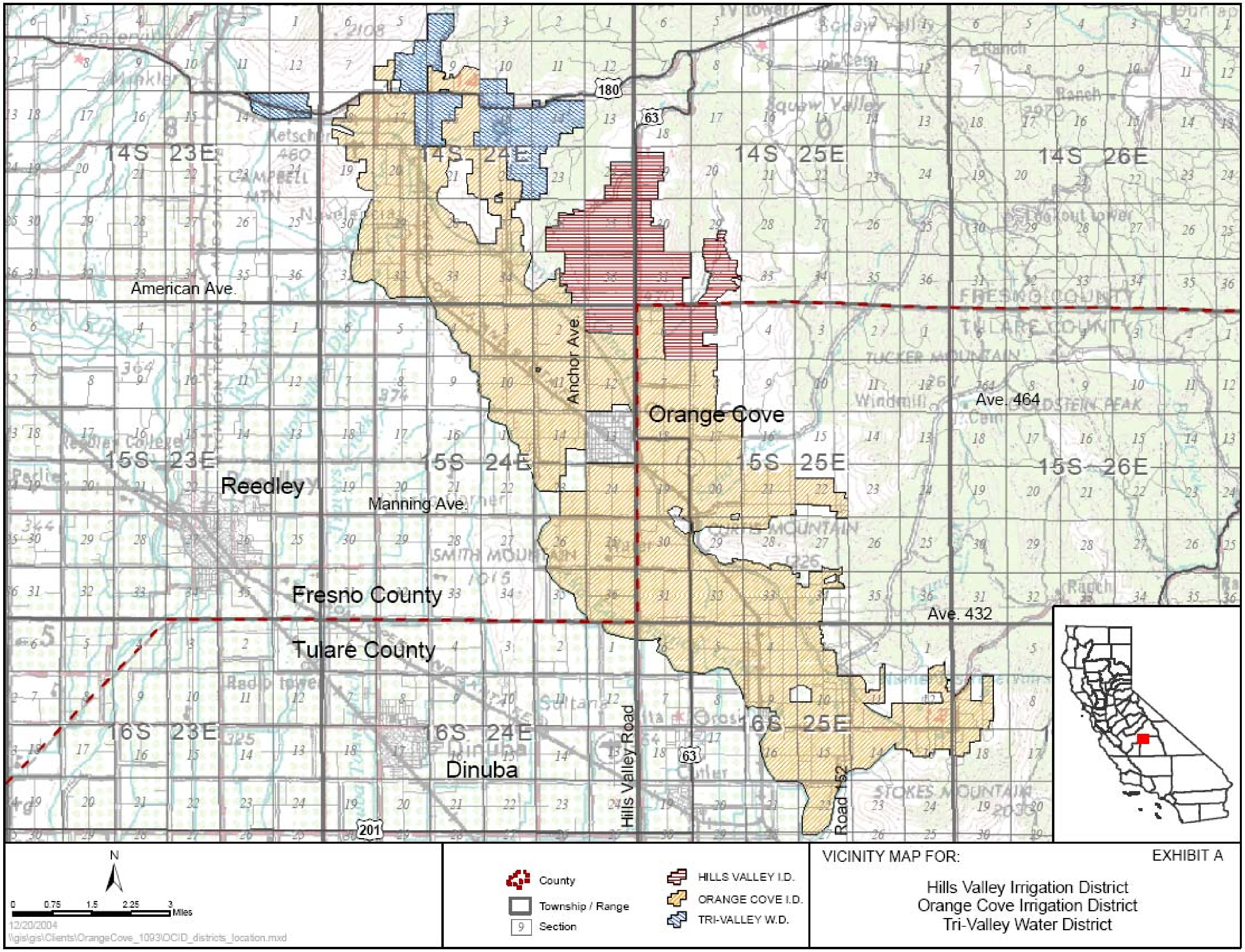

Figure 2. District Map

Water deliveries upslope from the Friant-Kern Canal were pumped through multiple pump stations due to the low-head distribution system installed. A total of 110 pumps were used District-wide. Energy consumption and costs were very high with an average pumping plant efficiency of 28 percent.

In the late 1970 and early 1980's, the District Board of Directors began looking at the high costs to operate and maintain the District's (Improvement Districts') water distribution facilities. Power costs were increasing rapidly and projected to increase greatly over the coming years. The District began looking at opportunities to generate its own power as a means to offset escalating rate increases. In 1984, the District joined with seven other districts that had formed the Friant Power Authority (FPA), an entity formed for the purpose of developing hydroelectric power on Friant Dam. The District also obtained a Federal Energy Regulatory Commission (FERC) license to construct a low-head hydroelectric plant on the Friant-Kern Canal at River mile 28.52.

In the late 1980's, the District began looking at ways to replace the water distribution systems built under the Improvement District concept. In 1991, the District Board of Directors elected to move forward with a rehabilitation program financed with bond sales through Certificates of Participation and constructed by a work force hired by the District. Public hearings were held to determine the level of support from the landowners. Enormous support was voiced by the landowners who now recognize that a dependable cost-effective water supply was the lifeblood of the community.

Agricultural Production

In the early years of the District, crops consisted mainly of dry land grain, irrigated field crops, and vegetables. Over the years the cropping has changed. Since 1975, the cropping pattern has remained fairly stable with about 90 percent of the cropped acreage in permanent plantings. Of the permanent crop about 84 percent or about 20,500 acres is dedicated to citrus. The second largest single crop acreage is grapes at around 2,000 acres. Other permanent crops consisting of deciduous and subtropical orchards, olives, and nut crops with a small mix of vegetables and field crops comprising about 4,300 acres. About 1,200 acres of non-cropped land such as farmsteads, packinghouses, equipment yards, and roads make up the balance. It is estimated that approximately 65,000 acre-feet annually is needed for crop production in the District but the District does not monitor water pumped by the landowners.

Water Supply

Groundwater - The historic water supply was pumped entirely from the groundwater. The aquifer in the area is not large in that the District is on the edge of the valley and extends to the edge of the foothills. Thus, the aquifer "thins" to the east. Often several wells may be drilled in the eastern portion of the District to capture an adequate supply of water. These are hard rock wells and many yield no water. However, if a well yields 20 gpm it is put into service for a dry-year supply. Well capacities increase toward the western side of the District but few produce more than 100 gpm. Groundwater recharge to the area comes partially from surface water runoff into and adjacent to the area that percolates into the ground and through the local streams of Sand Creek and Wahtoke Creek. Seepage from the portion of the Alta Canal and its service area also contribute to the groundwater supply.

Safe yield calculations performed by USBR staff, dated August 12, 1988, show a safe yield in the District of 26,800 acre-feet. Other reports show the safe yield to be about 27,800 acre-feet. At best, the District's safe yield does not exceed 28,000 acre-feet.

The District neither operates any ground water wells nor undertakes ground water recharge because of the existing ground water conditions. Landowners in the District practice their own conjunctive use out of necessity. They have mastered the art of pumping in coordination with District deliveries in order to produce their crops and sustain their permanent crops.

Surface Water – As stated earlier, the District was formed to acquire surface water supplies due to a recognition that farming the local area was unsustainable with local supplies. In 1949, the District signed a contract and starting importing supplies that year. The District has a contract with the USBR for 39,200 acre-feet annually of Class I water from the Friant Unit of the Central Valley Project, California. There is no anticipated change in the contract quantity provided the District continues to demonstrate beneficial use of the resource in accordance with State law. The following lists the surface supplies that have been imported since its inception. Note that through the years the District’s need for additional supplies does not vary significantly even though there has been significant hydrologic variability in both rainfall and surface runoff of the local stream systems.

Table 1. Orange Cove Irrigation District
Surface Water Diversions from the Central Valley Project

Year	AF Delivered	Year	AF Delivered	Year	AF Delivered	Year	AF Delivered
1952	18,440	1969	28,117	1986	38,600	2003	37,893
1953	23,083	1970	43,304	1987	30,346	2004	39,200
1954	27,808	1971	32,496	1988	37,046	2005	44,583
1955	30,472	1972	46,063	1989	36,080	2006	36,407
1956	16,178	1973	33,724	1990	33,944	2007	26,672
1957	31,109	1974	40,165	1991	32,262	2008	37,701
1958	29,287	1975	39,434	1992	38,784	2009	38,401
1959	37,197	1976	49,240	1993	37,969	2010	36,627
1960	30,007	1977	15,771	1994	28,961		
1961	34,239	1978	28,207	1995	42,832		
1962	31,609	1979	48,634	1996	38,710		
1963	41,672	1980	40,498	1997	37,017		
1964	37,775	1981	39,040	1998	22,161		
1965	39,590	1982	34,100	1999	39,200		
1966	40,336	1983	33,517	2000	36,276		
1967	30,467	1984	47,525	2001	38,385		
1968	46,246	1985	38,173	2002	36,761		

Reclaimed Water - The District, in cooperation with the City of Orange Cove, had implemented the use of reclaimed water. The program was in place for several years beginning in 1989 and generated about 260 acre feet per year of additional water for the District until the City's treatment plant failed to meet water quality standards. The City is in the process of upgrading their facility and the District may enter into a new agreement with the City to make use of this and potentially an expanded available water supply beyond that previously used.

The District has also participated in a water transfer program. Typically, if the District has an unused portion of its surface supply, that water is transferred to another District for banking with the intended purpose of recalling a portion of that supply in dry years. This program has been in use on the Friant-Kern Canal since its construction and is credited for getting the District through the 1987-1994 drought.

The District's need for a firm water supply cannot be over emphasized. The aforementioned development to permanent crops and a limited safe yield of ground water further emphasize the need for a firm surface supply. When the District entered into its original contract with the USBR, it elected to contract for Class I only because of the need for a firm supply. In fact, initially the Class I water for the Friant Unit was referred to as the "firm supply". After 61 years of operating history, it is now understood that while Class I water for Friant is generally a reliable supply, it is by no means a firm supply, particularly with newly required environmental flows to restore the San Joaquin River.

District Issues

The District has a formal set of rules and regulations by which it delivers water to its landowners. In general, the District allocates water in accordance with the California Water Code for those that purchase their annual allocation of water by the cut-off date of February 20th of each year. Water purchases after that date are subject to availability and require Board of Directors approval.

Distribution System

The Friant-Kern Canal severs the District and as such it is the main conveyance facility through the District. There are 4 turnouts from the canal that serve gravity laterals and 11 turnouts that serve pump laterals. All systems used low-head mortar joint pipe for their water delivery to the landowners'. Due to the use of low-head pipe, multiple lift stations were required to move water upslope with some lifts greater than 80 feet. Orifice plates were used to measure flow. The annual water loss that could not be accounted for was as high as 14 percent. The District used 110 pumps to move water which had an average, District-wide, pumping plant efficiency of 28 percent. Maintenance cost were approaching $40 per acre in some of the Improvement Districts. District operations were labor intense and did not provide for flexible deliveries. District rules permit only one change in a 24 hour period barring an emergency. From a District staff of 13, 9 people were used for operation and maintenance. The annual power consumption required for this distribution system was about 4,700,000 KWh.

In 1991 the District was approved for a financial package of $19 million for complete rehabilitation of the water distribution system and construction of the Fishwater Release Hydroelectric Project. Irrigation infrastructure construction started in 1992 and was completed by the end of 1996 except for some unfinished regulation reservoir work. 105 miles of new pipelines were installed predominately using Class 80 polyethylene pipe. 40 high efficiency pumps and motors were installed to move water upslope of the Friant-Kern Canal with only one system using an intermediate lift station. 646 propeller type flow meters, certified for +/-2% accuracy, were installed to measure flow and totalize delivery. Regulating reservoirs were installed at the top of four systems having the higher overall lifts to deliver water during peak power periods to avoid electrical cost differentials. Landowners are now permitted to operate their deliveries provided they order water from the District so that flow in the Friant-Kern Canal can be reasonably matched to demand. The new system is designed for on-demand deliveries and can be operated as such if and when the Friant-Kern Canal can accommodate on-demand deliveries. The original distribution system's power requirement of 4,700,000 KWh is approximately 170% of today's average power requirement of 2,800,000 KWh. At today's power rates, this translates to an annual power savings with a value of approximately $300,000.

Figure 3. Pipeline Rehabilitation

The rehabilitation program was accomplished by force account work where the District hired the work force and the District Manager had continuous full control of the program. This was done not only for the costs that could be saved but, more importantly, to maintain good landowner relations with those that were directly impacted by tree removal for installation of the new system. As such, the program was completed on schedule and under budget. The District then reorganized its staff for field operations going from 9 to 2 people who operate and maintain the whole system.

After completing the system rehabilitation program, the District began looking at variable frequency motor controllers, which were becoming popular and proving to be an effective means of regulating pump motors to meet load demands. After investigating this option, the District elected to install a SCADA system with variable frequency controllers on all pumped laterals with funds left over from the rehabilitation program. This greatly enhanced District operation by providing remote monitoring and control of the systems as well as providing alarms for system failures. With home computers for operation staff, nighttime alarms could be analyzed and most often solved without field visits.

On farm - low-pressure drips, mister or microjet systems were becoming the prominent means to apply crop water before the rehabilitation program and rapidly accelerated following program completion. Many landowners were able to take advantage of the pressure provided by the District pipeline and did not have to install pumps for their system. It is estimated that 74 to 80 percent of the permanent crop land is now irrigated by low-pressure systems. The remainder is irrigated by furrow except small acreage of pasture and hay. The District does not currently have an inventory of systems by crop but is in the process of surveying the system with the annual land use survey.

Pump and Electric Use Information

Listed below are the specifics of the stations including flow and cost for operation.

Table 2. Orange Cove Irrigation District
Pump Station Operational Data - Energy Use and Cost

Turnout	Energy Consumption (MWh)	Cost ($ 1,000s)
1	434	93
2	18	3
3	239	39
4	50	9
5	130	23
6	348	48
7	28	4
8	735	86
9	72	11
10	51	9
11	183	27
12	82	14
13	430	58

Hydroelectric Generating Facilities

The District has been progressive in identifying potential projects and programs to help defray ongoing operational costs. To this end the District's decision to get involved in hydroelectric power generation has proven to be a wise decision. In 1985, FPA completed power plants on Friant Dam at outlets for the Friant-Kern Canal, Madera Canal and the San Joaquin River outlet. 1986 was a banner year but revenues during the drought of 1987-1994 put the bond debt in default. The bonds were sold to an independent power producer and as revenues returned following the drought, FPA was able to regain control through a bond refinancing. Those bonds were paid off in 2001with partial payment from member district funds. Orange Cove Irrigation District has the smallest member share but the project, now debt free, generates an average of $350,000 per year to the District.

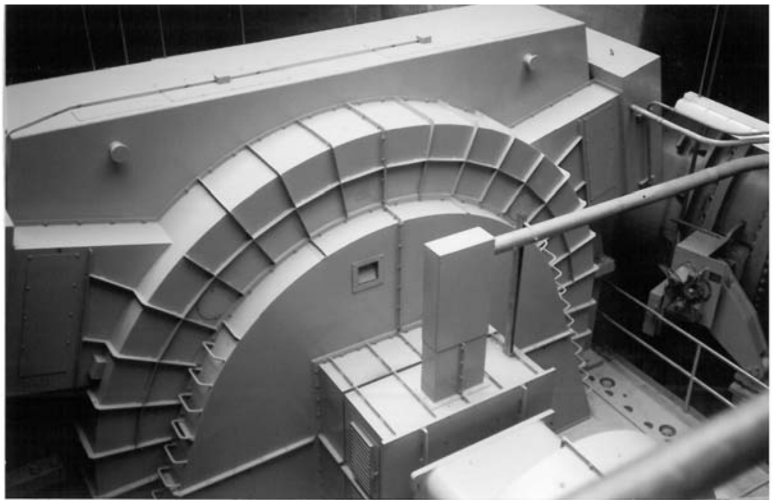

Figure 4. Friant-Kern Canal Power Plant

In 1988, the District entered into an agreement with a private developer to construct the now licensed Kings River Siphon Hydroelectric facility. The project is a low-head hydro unit built at the water control structure on the Friant-Kern Canal at Milepost 28.52 at the inlet to the Kings River Siphon, a water conveyance facility constructed under the Kings River. The project was constructed under a turnkey agreement where the District shares net revenues during the first 25 years and will own and operate the project commencing August 2015.

Figure 5. Kings River Siphon Hydro Project

Later in 1988, the FPA Board of Directors made the decision to abandon the development of the Fishwater Release Hydropower plant which was part of the overall plan licensed to develop power on Friant Dam. After completing feasibility studies performed by two major engineering firms, FPA considered the project unworthy of development considering a host of other issues they had to deal with. FPA offered the project to member districts if any opted to build it. Orange Cove Irrigation District with three other districts elected to pursue development of the project. When a call for upfront funds was made, the other three districts dropped their membership leaving the District as the only entity to develop the project. The District again entered into a turnkey agreement but took over the project when the developer could not obtain sufficient funds to complete the project.

Figure 6. Friant Dam Facilities

Operational History

The Kings River Siphon Hydro (KRSH) was constructed using four turbine/generator sets capable of flows of 300 cfs each. Water is siphoned from the Friant-Kern canal passing through the turbines and returned to the Canal immediately below the check gate.

This presented a potential problem for operations because the Friant-Kern Canal control structure at this location consisted of a single radial-arm gate. An offline trip by the power plant would attempt to return in excess of 1,200 cfs to the Canal over a very short time period. Most likely a task the check control gate could not accommodate. Furthermore, in case of gate failure or backup generator failure to start during a power failure, flows in the Canal would be interrupted causing catastrophic lining failure downstream due to rapid dewatering of the Canal. This problem was solved by using a vacuum regulator valve controlled by the speed (revolutions per minute) of the generator which replicates flow through the turbines as if they were online. This creative solution appears to be the first ever for this technique that was used to control water flow in a siphon system.

Figure 7. Intake from the Friant Kern Canal

The KRSH project is very sensitive to head and has maximum production when flows are at or near 1,200 cfs. The plant shuts down when Canal flows reach about 2,500 cfs due to insufficient head to produce power. For a normal water-year, this means about three months during the peak water demand periods when Canal flows approach or exceed 4,000 cfs the powerplant is not operational. On the average, gross revenues range from about $150,000 to $200,000 per year. Far less than projected by the feasibility studies. The major problem is the head loss in the tailrace due to the confined space and therefore friction losses associated with return of water to the Canal.

Figure 8. Kings River Siphon Power Plant

Operations of the Fishwater Release Hydroelectric Project were based on a continuous flow of 35cfs from Friant Dam to the San Joaquin River Fish Hatchery. The turbine/generator is rated for 523kW. Peak production is obtained when the reservoir is full and declines as the reservoir level drops. The District licensed an additional 30cfs that was being released to the San Joaquin River over and above the flows that passed through the FPA River Power plant. This was done to generate more power by passing more water through the turbine as water levels in the reservoir dropped. It was the District's understanding at the time the turbine had the capacity to handle most of the additional flow. It was later determined that production peaked at about 10-15cfs additional flow or 45-50 cfs maximum flow at the lower head levels. Both the KRSH and this project had Standard Offer 4 contracts with the local utility and provided premium power rates for the first 10 years of production. As such, the Fishwater Hydro Plant generated more revenue in the first four years than the $2.5 million cost of development. The project currently generates gross revenues in the range of $300,000 - $350,000 per year.

Figure 9. Fishwater Powerplant

Figure 10. Turbine at Fishwater Powerplant

Financial Analysis

The District has invested about $3.5 million in power producing projects which now return a revenue stream of about $750,000. That number will increase by more than $100,000 following the ownership transfer of the KRSH to the District in 2015. By financing the Fishwater Hydro Plant with the District's rehabilitation program, the District elected to use net revenues generated by the project to pay a portion of the landowners' repayment cost. The net results are that the rehabilitation program cost the landowners $22 per acre over and above the average maintenance costs for the duration of the bond repayment period which ends 2016.

FLOW MEASUREMENT CAPABILITIES OF DIVERSION WORKS IN THE RIO GRANDE PROJECT AREA

Brian Wahlin, Ph.D., P.E., D.WRE[1]

ABSTRACT

Releases from Rio Grande Project storage are made on demand by the U.S. Bureau of Reclamation for diversion into Elephant Butte Irrigation District (EBID), El Paso County Water Improvement District No.1, and Republic of Mexico canals and laterals. The diversions are charged against each district and Mexico's annual diversion allocation. As the Rio Grande Project implements and refines new operating procedures and the State of New Mexico continues efforts to implement Active Water Resource Management in the Lower Rio Grande, it is essential to have a high degree of confidence in the measurements of the water diverted from the Rio Grande.

With this mission in mind, the New Mexico Interstate Stream Commission (NMISC) initiated a study to evaluate the Rio Grande Project diversion works, flow measurement facilities, and flow measurement methodologies in the Rincon and Mesilla Valley portions of the Rio Grande Project. More specifically, the NMISC was interested in understanding the measurement accuracy limitations presented by the diversion structures themselves, and whether improvements to those structures and/or methods could improve measurement accuracy. WEST Consultants, Inc. (WEST) evaluated flow measurement techniques at Elephant Butte Dam, Caballo Dam, Percha Diversion Dam, Arrey Main Canal, Leasburg Diversion Dam, Leasburg Canal, Mesilla Diversion Dam, the East Side Canal, the West Side Canal, and the Del Rio Lateral. EBID is making a significant effort to accurately measure flows despite the advanced age of many of the structures in the Rio Grande Project. All measurements in these areas were made following typical protocols and standards.

This paper outlines the accuracy estimations, describes the flow measurement techniques used and analyses conducted, and provides suggestions for improving the flow measurements in some difficult locations.

INTRODUCTION AND BACKGROUND

Up until the 1980s, the Rio Grande Project was operated as a single project and was not concerned with state boundaries (i.e., determining the amount of water diverted by each state was not important). Now, releases from Rio Grande Project storage at Caballo Dam are made on demand by the U.S. Bureau of Reclamation for diversion into Elephant Butte Irrigation District (EBID), El Paso County Water Improvement District No.1, and Republic of Mexico canals and laterals. The diversions are charged against each district and Mexico's annual diversion allocation. As the Rio Grande Project implements and refines new operating procedures and the State of New Mexico continues efforts to

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implement Active Water Resource Management in the Lower Rio Grande, it is essential to have a high degree of confidence in the measurements of the water diverted from the Rio Grande. Allowable groundwater diversions are a function of annual surface water allocations made by the U.S. Bureau of Reclamation (USBR), and calculation of a comprehensive Lower Rio Grande water budget for administrative purposes necessarily depends on accurate diversion data for both surface water and groundwater.

Personnel from WEST and NMISC inspected the diversion works, flow measurement facilities, and flow measurement methodologies in the Rincon and Mesilla Valleys. WEST assessed the current flow measurement methodologies used at various sites and made suggestions to improve the measurements. A field visit was conducted on June 8-9, 2006. The site visit started at Elephant Butte Dam and preceded downstream to Caballo Dam, Percha Diversion Dam, Leasburg Diversion Dam, and Mesilla Diversion Dam (see Figure 1).

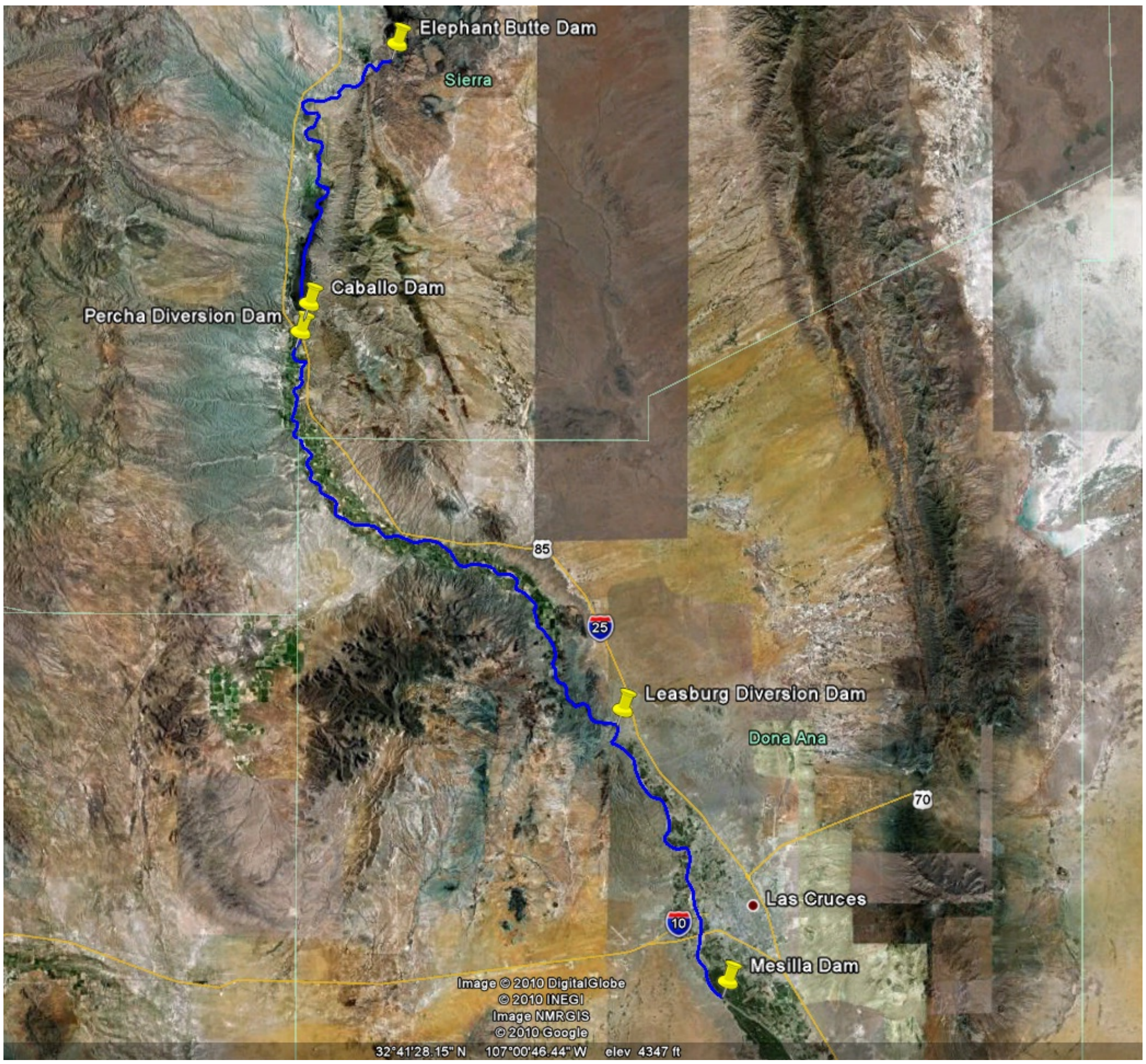

Figure 1. Project location map

ELEPHANT BUTTE DAM

The Elephant Butte Dam (originally called Engle Dam) is on the Rio Grande River, 125 miles north of El Paso, Texas. A concrete gravity dam, it is 301 feet high, 1,674 feet long (including the spillway), and contains 618,785 cubic yards of concrete. It was completed in 1916, but storage operations began in 1915. The resulting Elephant Butte Reservoir can store up to 2,210,000 acre-feet of water to provide irrigation supply and year-round power generation.

Flow is measured downstream of Elephant Butte Dam at the Rio Grande Below Elephant Butte Dam, NM stream gaging site (ID 08361000) that is operated and maintained by the U.S. Geological Survey (USGS). This gage has been in operation since 1916. A measurement is performed once every two weeks when releases are made from Elephant Butte Dam. While a stream gage measurement is being performed, flows are also measured using the sluice gates (it was not possible to observe these sluice gates during the field visit). The flow measured by the sluice gates and the stream gaging typically agree with each other. However, there can be additional releases from Elephant Butte Dam that are not passed through the sluice gates. In these cases, the two measurements do not agree.

The Elephant Butte stream gaging site appears to be in a very good location for maximizing measurement accuracy (see Figure 3). The approach channel is straight and there is no noticeable debris to obstruct the flow. The cobbled canal bed is relatively smooth and stable. The only problem with this site is that weeds often become entangled with the current meter (see Figure 2) and must be removed by hand. The USGS uses a 3-foot vertical spacing when performing the stream gaging and start their verticals 6 feet from the bank.

For the most part, the flows are measured using the guidelines provided by the USGS for accurate stream gaging measurement (Rantz, 1982). Based on previous studies [Sauer and Meyer (1992) and Clemmens and Wahlin (2006)], an individual current-meter discharge measurement at this site will be accurate to approximately 2-3%.

Figure 2. Weeds can be an issue at the Elephant Butte stream gaging site

Figure 3. Stream gaging site downstream of Elephant Butte Dam

CABALLO DAM

Caballo Dam, constructed by the USBR and completed in 1938, is located 25 miles downstream from Elephant Butte Dam. The dam is an earthen structure 96 feet high and 4,590 feet long with a capacity of 343,990 acre-feet of water. Water discharged from the Elephant Butte Power Plant during winter power generation is impounded at Caballo Dam for irrigation use during the summer. Caballo Reservoir also serves an important flood control role, particularly in the summer. The reservoir receives inflow from several significant tributaries which drain large areas between the Rio Grande River and the crest of the Black Range Mountains to the west.

Flow is measured downstream of Caballo Dam at the Rio Grande Below Caballo Dam, NM stream gaging site (ID 08362500). This stream gaging site is operated and maintained by the USBR since 1938. A measurement is performed once a week. In addition, EBID also performs stream gagings at this site twice a week. The measured flow at the site is used to set the opening on the radial gates at the Percha Diversion Dam, as explained in the next section. The Caballo stream gaging site appears to be in a very good location to maximize measurement accuracy (see Figure 4). The approach channel is straight and there is no noticeable debris or vegetation to obstruct the flow. The canal bed appears to be smooth and stable. The USBR uses a 5-foot vertical spacing when performing the current-meter discharge measurement, and they appear to be following the streaming gaging guidelines defined by the USGS (Rantz, 1982).

Figure 4. Stream-gaging site downstream of Caballo Dam

One problem with this site is that backwater effects can extend to this site from the Percha Diversion Dam downstream, resulting in extremely low velocities, which makes flow measurement less certain. Regardless, the methodology itself used to measure flows at Caballo Dam stream gaging site is acceptable. For the most part, the USBR is following the guidelines provided by the USGS for accurate stream gaging measurement (Rantz. 1982). Based on previous studies (Sauer and Meyer, 1992, and Clemmens and Wahlin, 2006), an individual current-meter discharge measurement will be accurate to approximately 2-3%, although this does not account for the potential complications resulting from the backwater effects.

PERCHA DIVERSION DAM

The Percha Diversion Dam is located about two miles downstream from Caballo Dam (see Figure 5). It was constructed by the USBR from 1914-1919 and diverts water into the Arrey Main Canal along the west bank of the Rio Grande River. Although the dam is still owned by the USBR, it is operated by EBID. The diversion dam is a concrete ogee weir with embankment wings. There are two radial gates on the west side of the diversion dam. Based on the observed water marks on the downstream side of the radial gates, it appears that the radial gates are always in the free-flow condition. Although some flow goes through the radial gates, most of the flow in the Rio Grande River goes over the diversion dam. The exception to this is when there are no releases from Caballo Dam in which case the entire Rio Grande River flows through the gates. The radial gates are not currently being used to measure the flow rate in the river. Instead, they are being used to keep the water level upstream of the Percha Diversion Dam at a given elevation so that the flow Q (cfs) into the Arrey Main Canal will be constant. Once a stream gaging has been made at the Caballo gaging station, the gate openings on the two radial gates on the Percha Diversion Dam are set according to the following rules:

- If $Q < 300$ cfs, then open one radial gate 6 inches.
- If 300 cfs $<= Q <$ 700 cfs, then open both radial gates 6 inches.
- If 700 cfs $<= Q <$ 2,500 cfs, then open both radial gates 14 inches.
- If $Q >=$ 2,500 cfs, then open both radial gates 16 inches.

The origin of these rules is unknown. They are attached to the controls for raising and lowering the radial gates. Once the gate opening is set on the radial gates, this information is called into EBID headquarters and recorded.

Figure 5. Percha Diversion Dam with radial gates in the foreground

ARREY MAIN CANAL (OFF OF PERCHA DIVERSION DAM)

The Arrey Main Canal, which carries water for the irrigation of 16,260 acres in the Rincon Valley, is 28.1 miles long and has a capacity of 350 cfs. The head gates to the Arrey Main Canal consist of 10 submerged sluice gates. Near the head of the canal, the bottom width is 12 feet and the side slopes are 2.5:1. The channel depth is 6 feet. The canal extends 4 miles downstream from the Percha Diversion Dam where it connects to the Garfield Canal.

Currently, there appears to be no flow measurement devices at the head of the Arrey Main Canal. The submerged sluice gates at the head of the Arrey Main Canal will not provide very accurate flow measurements. Installing a Replogle flume in the Arrey Main Canal will be an economical way to achieve accurate flow measurements. The Replogle flume is accurate to within about 2% and can be easily and inexpensively constructed in a concrete-lined or earthen canal. These flumes can easily pass floating debris and can be designed to pass sediment transported by open channels with subcritical flow. Head loss is minimal. Each flume can be computer calibrated, producing an accurate rating table even if the flume is not constructed exactly to the design dimensions. For more details on Replogle flumes, see Clemmens et al. (2001).

It has been reported that a flow measurement device is available approximately 100 yards downstream of the head gates in a concrete portion of the canal. Unfortunately, the existence of this site was not known until after the field visit, and hence no assessment was made of its effectiveness.

LEASBURG DIVERSION DAM

The Leasburg Diversion Dam is located on the Rio Grande 62 miles north of El Paso at the head of Mesilla Valley. The USBR started construction of this dam in 1906 and completed it in 1908. The Leasburg Diversion Dam is a concrete ogee weir with embankment wings (see Figure 6). This structure diverts water into the Leasburg Canal for the upper 31,600 acres of the Mesilla Valley irrigation system. There are submerged sluice gates on the east bank of the Rio Grande. Water passes through these gates and enters the Leasburg Canal.

The Leasburg Canal, constructed in conjunction with the Leasburg Diversion Dam, conveys irrigation water to Mesilla Valley, is 13.7 miles long and has a capacity of 625 cfs. On average, the bottom width of the canal is approximately 34 feet, the side slopes are 1:1, and the depth is about 4 feet. The canal is deeper and wider near the head gates. There are 7 highly submerged sluice gates at the head of the Leasburg Canal as shown in Figure 7. The water level in the Leasburg Canal is typically very high as shown in Figure 8. There are significant sediment problems that occur near the head of the Leasburg Canal. To alleviate the sediment problem, the Leasburg Canal Wasteway 1-A was installed approximately a mile downstream of the head gates.

The highly submerged nature of the diversion head gates makes flow measurement difficult and inaccurate using the sluice gates. Indeed, there are instrumentation casings installed upstream and downstream of the head gates that probably once held pressure transducers to measure depth but that were subsequently abandoned when the method was found to be unreliable and inaccurate. Thus, both known limitations and field evidence suggest that using the highly submerged head gates to measure the flow is not a viable option.

Figure 6. Leasburg Diversion Dam

Figure 7. Submerged sluice gates at the head of the Leasburg Canal

Figure 8. Leasburg Canal from the head gates

One way to measure flows in the Leasburg Canal would be to install a Replogle flume somewhere along the canal. However, because the water depths in the canal are so high, care would need to be taken to ensure that the flume operates correctly and that the canal does not overtop.

In addition, installation of a Replogle flume may cause excessive sediment accumulation upstream of the flume. To reduce sediment problems, several steps can be taken (including minimizing upstream backwater effects, minimizing head loss, limiting the Froude number to 0.5 at maximum flow and maximizing it at low flows, and contracting the flume from the sides only (Clemmens et al., 2001). Even with all of these measures, however, sediment may still accumulate upstream of the flume, which would require the sediment to be removed at regular intervals.

Unfortunately, a preliminary Replogle flume design for this location indicated that it is not a viable option. The Froude numbers at the flume are predicted to be near 0.1, which is not high enough to actively pass the sediment. Even at the lowest flume height possible, the Froude numbers do not approach 0.5 in this slow moving canal. Of course, additional data are needed to verify this conclusion.

Another alternative is to further investigate the area to look for a more appropriate flow measurement location farther downstream past the Leasburg Canal Wasteway 1-A. Other locations may have higher Froude numbers allowing a Replogle flume to be installed without causing sediment accumulation.

A more viable option may be to use the bridge shown in Figure 8 to perform stream gaging measurements. While the section around the bridge appears to be significantly influenced by backwater, thus limiting flow measurement accuracy by stage-discharge relationship development, accurate flow measurements could be made using standard stream gaging techniques. Further investigation is needed to determine whether or not a reliable record could be obtained at this site using stream gaging.

Finally, there may be ways to reduce the sediment load in the Leasburg Canal using settling tanks, sediment ejectors, vortex sand traps, or vortex chambers. Details of these devices are described by Raudkivi (1993). Further study is necessary to determine if any of these devices could be feasibly implemented on the Leasburg Canal.

MESILLA DIVERSION DAM

The Mesilla Diversion Dam is located on the Rio Grande 40 miles north of El Paso, TX. It was constructed by the USBR during the same time period as the Percha Diversion Dam (1914-1919). The Mesilla Diversion Dam consists of a low concrete weir with 13 radial gate structures, 22 feet high, flanked by levees as shown in Figure 9. This structure diverts water into the East Side and West Side Canals (also constructed from 1914-1919) for the lower 53,650 acres of the Mesilla Valley irrigation system. The structure also diverts water in to a smaller lateral called the Del Rio Lateral. Under normal operating conditions, only 2 of the 13 radial gates on Mesilla Diversion Dam are opened. The remaining 11 gates are kept closed unless sediment needs to be flushed through the structure or there is a flood event. Unlike the other diversion dam structures on the Rio Grande River, all of the flow goes through the radial gates; none of the flow goes over the diversion dam.

Gate 1 is located on the east side of the Mesilla Diversion Dam. This gate is typically kept at a constant opening and the gate position is not changed. Gate 2 is located on the west side of the diversion structure. This gate is adjusted up and down depending on the flow in the Rio Grande River. According to Wayne Treers, then of the USBR, who was interviewed during the field visit, there is a local flow controller on Gate 2 that automatically adjusts the gate opening to maintain a constant flow through Gate 2. According to a ditch rider for EBID, who was interviewed during the field visit, the local upstream water level controller on Gate 2 operates according to the following rules:

- If the water level upstream of Mesilla Diversion Dam is greater than 7 feet, then Gate 2 is opened.
- If the water level upstream of Mesilla Diversion Dam is less than 6.75 feet, then Gate 2 is closed.
- If the water level upstream of Mesilla Diversion Dam falls in between 6.75 and 7 feet, then the gate position is not changed.

Figure 9. Upstream face of the Mesilla Diversion Dam

Further investigation is needed to know exactly what type of controller is actually being used on Gate 2. Gate 2 transmits data (i.e., gate opening, water level, and flow rate) back to EBID's headquarters automatically. EBID can monitor Gate 2 remotely, but they cannot control it remotely (as of 2006). Thus, if EBID wants to make additional changes to Gate 2 (besides the local controller changes), a ditch rider must be sent out to manually adjust the gate.

Both Gate 1 and Gate 2 appear to be operating under free-flow conditions. The high water marks on the downstream sides of the gate suggest that these radial gates are never submerged or in the transition zone. Because of this, accurate flow measurements are possible using Gate 1 and Gate 2. Currently, EBID calculates the flow through the radial gates by assuming that one inch of gate opening is equal to 28 cfs. It is uncertain how this rule was initially developed. To obtain more accurate flow measurements, it is suggested that the USBR's WinGate program be used to calculate the flow through the gates. This program uses a newly developed algorithm for calculating flows through radial gates based on research performed by the U.S. Arid Land Agricultural Research Center, which is part of the Agricultural Research Service in the U.S. Department of Agriculture. A summary of the methodology used in this new radial calibration appears in a paper by Wahl (2005).

If WinGate is used to calculate the flows through Gates 1 and 2, a further investigation would be needed to ensure that the upstream head is measured correctly. Currently, the head is measured using a stilling well on the west bank of the Rio Grande. The stilling well appears to be in a good location. However, the stilling well should be examined to verify that the zero is set correctly and that the stilling well is installed correctly (e.g., pipes not clogged, taps drilled correctly, properly zeroed). This is probably routinely done by EBID, but was not verified.

EAST SIDE CANAL (OFF OF MESILLA DIVERSION DAM)

The East Side Canal is 13.5 miles long and has a capacity of 300 cfs. The head gates are sluice gates. The flow passes under the sluice gates and into a concrete rectangular box before transitioning into the canal. The flow in the concrete box is quite turbulent as can be seen in Figure 10. Near the head gates, the canal bottom is approximately 40 feet wide and its depth is 9.5 feet. The canal side slopes are 1.15:1.

It appears the flow measurement using the East Side Canal head gates would be difficult. Another option would be to install Replogle flumes just downstream of the head gates but still inside the rectangular concrete box. This option would require some flow conditioning to reduce the turbulence after the gate and improve the flow distribution so that the flume would measure flow accurately. Some possible flow conditioning devices include flow straightening vanes and wave suppressors (Replogle, 1997). A third alternative would be to install a Replogle flume about 500 feet downstream of the head gates, where an abandoned concrete structure could be converted into a Replogle flume.

WEST SIDE CANAL (OFF OF MESILLA DIVERSION DAM)

The West Side Canal is 23.5 miles long and has a capacity of 650 cfs. The head gates on the West Side Canal are submerged sluice gates. Just downstream of the head gates, there is a 90° bend in the canal. Water shoots out from the sluice gates and it is extremely turbulent as shown in Figure 11. There is a strong jet of water that runs along the north side of the West Side Canal which hits the side of the canal where it bends, causing even more turbulence. The canal has a bottom width of approximately 58 feet and a depth of 8.2 feet. The side slopes are 0.67:1.

Flow measurement accuracy is unlikely using the sluice gates at the head of the canal because the water downstream of the gates is so turbulent, making it difficult to obtain a downstream water level. An equipment casing housing had been installed in an attempt to measure the downstream water level; however, the EBID ditch rider reported the measurements obtained were not good and so the measurement site had been abandoned. An alternative would be to install a Replogle flume downstream of the bend in the canal near where the stream gaging station is located as shown in Figure 12.

DEL RIO LATERAL (OFF OF MESILLA DIVERSION DAM)

Very little information was available on the Del Rio Lateral. This canal receives water from the east bank of the Rio Grande upstream of the Mesilla Diversion Dam. The canal passes under the East Side Canal via a siphon before it continues south parallel to the Rio Grande River. It has a bottom width of 6 feet, a depth of 7.25 feet, and side slopes of 1.5:1. The Del Rio Lateral is an earth lined canal; however, there is a short portion that was concrete lined in an attempt to measure the flow using stream gaging techniques (see Figure 13). There was a PVC pipe installed on the side of the concrete portion of the Del Rio Lateral. This pipe was probably used to get a measurement of the stage in the lined section in order to develop a rating curve for this canal.

This canal is problematic from a flow measurement standpoint because sediment accumulates in the short lined portion of the canal. Installation of a Replogle flume may result in excessive sediment accumulation upstream of the flume. While the Clemmens et al. (2001) methods could be implemented to reduce the accumulation (see the Leasburg Canal section), it would probably not completely alleviate the problem, and sediment would still need to be removed at regular intervals.

The PVC pipe along the sides of the lined portion of the Del Rio Lateral (see Figure 13) is a static pressure tube that can be used to determine the depth of water. Note the pressure taps on this tube (shown in a close up view in Figure 14) are drilled such that the holes face upstream. As a consequence, the depth of water inside the pipe will be influenced by the energy of the water directly hitting the pressure taps. This will lead to water surface elevations inside the pipe that are higher than the water surface elevation outside the pipe. An alternative design for a static pressure tube that avoids such problems is shown in Figure 15 (from Replogle, 1997).

Figure 10. Turbulent flow under the sluice gates at the head of the East Side Canal

Figure 11. Turbulence downstream of the head gates on the West Side Canal

Figure 12. West Side Canal near the stream gaging bridge

SUMMARY

In general, flow measurements are made on the Lower Rio Grande following standard procedures despite the advanced age of some of the structures. Accuracy of the individual stream gage measurements is in the 2-3% range. Most of the laterals off of diversion dams are not gaged. It is would be desirable to have some sort of flow measurement information at the heads of each of these laterals. At some sites, such as the Arrey Main Canal, a flow measurement device would be relatively straightforward. At other sites, such as the Leasburg Canal and the Del Rio Lateral, installing a flow measurement device will be tricky. Suggestions as to possible flow measurement devices to use on these canals were given.

Figure 13. Concrete lined section of the Del Rio Lateral

Figure 14. Close up view of pressure taps

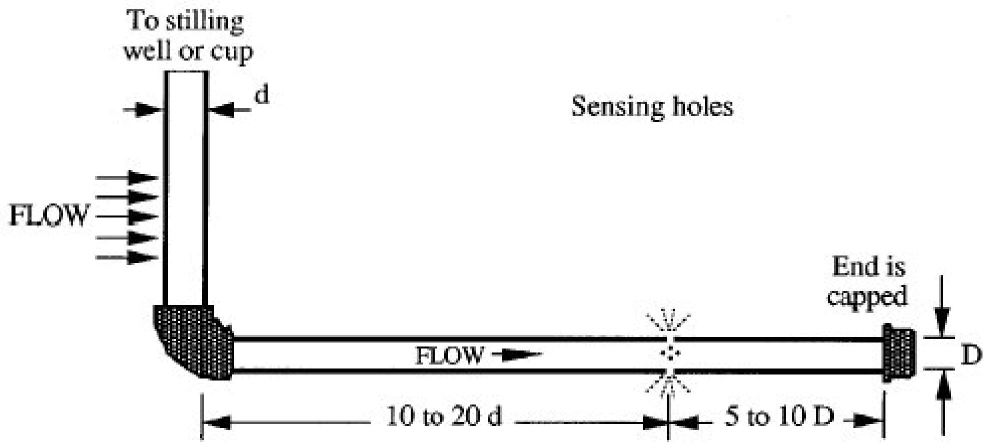

Figure 15. Suggested design for a static pressure tube (from Replogle, 1997)

REFERENCES

Clemmens, A.J., Wahl, T.L., Bos, M.G., and Replogle, J.A. (2001). *Water measurement with flumes and weirs.* International Institute for Land Reclamation and Improvement publication 58, Wageningen, The Netherlands.

Clemmens, A.J., and Wahlin, B.T. (2006). "Accuracy of annual volume from current-meter-based stage discharges." *J. Hydrologic Engr.*, 11(5), pp. 489-501.

Rantz, S.E. (1982). *Measurement and computation of streamflow: volume 1. Measurement of state and discharge.* USGS Water-Supply Paper 2175, Washington DC.

Raudkivi, A.J. (1993). *Sedimentation: Exclusion and removal of sediment from diverted water.* International Association for Hydraulic Research Hydraulic Structures Design Manual 6, Rotterdam, the Netherlands.

Replogle, J.A. (1997). "Practical technologies for irrigation flow control and measurement." *Irrigation and Drainage Systems*, v. 11, pp. 241-259.

Sauer, V.B., and Meyer, R.W. (1992). *Determination of error in individual discharge measurements.* USGS Open-File Report 92-144, Norcross, GA.

Wahl, T.L. (2005). "Refined energy correction for calibration of submerged radial gates." *J. Hydraulic Engr.*, 131(6), pp. 457-466.

ACOUSTIC DOPPLER CURRENT PROFILER (ADCP) EXPERIENCES IN NEW MEXICO

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ABSTRACT

The New Mexico Interstate Stream Commission (NMISC) recently acquired a Rio Grande model acoustic Doppler current profiler (ADCP) and the accessories required to deploy the instrument. Accessories include a tethered boat, an onboard field computer, an optional non-survey grade depth sounder, and an optional global positioning system (GPS).

ADCPs use sound waves to measure stream discharge accurately. In addition to stream discharge, other related types of measurements are possible including flow velocity and stream cross-sectional (bathymetric) profiles. The NMISC's statewide uses for the ADCP include measuring flood flows, improving stream gage rating curves, performing seepage studies, verifying or supplementing discharge measurements made by other methods, and conducting bathymetric surveys of river channels as part of habitat studies for threatened or endangered aquatic species.

ADCPs are versatile tools that allow measurements to be made in situations such as during flood events that would be difficult or impossible to measure using other approaches. For example, the device can be deployed when water is too deep, wide, or swift for traditional wading measurements. In other circumstances, the stream bottom may too silty or soft to permit traditional wading measurements. The ADCP may be deployed from a motorized or tethered boat, from a traditional or bank-operated cableway, or from a temporary or permanent rope and pulley system. System advantages and limitations are presented along with selected study results.

INTRODUCTION

The New Mexico Interstate Stream Commission (NMISC) is a statutory agency of the State of New Mexico charged with protecting, conserving, and developing the waters and stream systems of the state. Part of the agency's mission is to protect New Mexico's surface water allocated under eight interstate stream compacts. In addition, the agency monitors compliance with water delivery obligations under each compact.

As a tool to improve administration of surface water resources in the state, the NMISC acquired a 1200 kHz Rio Grande model acoustic Doppler current profiler (ADCP)

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(Figure 1). The ADCP is a device that uses sound waves to measure stream discharge, water velocity, and stream cross-sectional profiles.

Figure 1. Photograph of Riverboat in Canal near Albuquerque, NM

ADCP AND ACCESSORIES

The NMISC typically deploys the ADCP in a tethered Riverboat, a 4-foot long by 3-foot wide trimaran specifically designed for the Rio Grande unit. Included in the Riverboat is a water-tight compartment to accommodate an on-board field data acquisition computer known as the OysterPE, the control unit for an optional global positioning system (GPS), and a 12 volt marine battery. The Riverboat also comes equipped with the required connections for an optional non-survey grade depth sounder to supplement the bottom tracking feature built into the ADCP. A shore-based computer with a wireless connection to the OysterPE allows the operator to monitor the operation of the unit throughout each measurement. The OysterPE and shore-based computers use a Microsoft Windows-based data acquisition software package called WinRiver II.

ADCP APPLICATIONS

The primary use of the ADCP is to perform surface water discharge and velocity measurements. The ADCP can be used to confirm discharge measurements at existing stream gage locations or to measure discharge at locations where no stream gages exist. The NMISC was motivated to purchase the ADCP to corroborate stream gage data for interstate compact compliance. In a situation, for example, when a powerful flood renders an existing stream gage inoperable, the ADCP unit can be deployed to measure portions of the flood hydrograph that otherwise would not be available. This additional information may prove valuable in determining how much water New Mexico delivered to a downstream state during the flood and ensuring that New Mexico gets the appropriate credit for delivering that water.

The NMISC plans to conduct seepage studies on selected reaches of rivers in the state to learn more about locations and amounts of surface water gains and losses. This surface water – groundwater interface information will provide guidance for agency modelers to improve existing and future surface water and groundwater models. Another important use of the ADCP is for bathymetric surveys. These surveys will assist water resource managers in the agency to delineate river cross-sectional characteristics for use in fish habitat studies.

ADCP ADVANTAGES

ADCPs offer several advantages over traditional wading measurements. ADCP measurements are relatively quick and are generally repeatable. Discharge and velocity measurements can be made in places that might not be or are not suitable for traditional methods. For instance, in some locations the water is too deep to wade (Figure 2).

Figure 2. Photograph of an ADCP Measurement Using the Riverboat

Other locations have stream bottoms that are too silty and soft to wade safely. Water flow may be too rapid to wade safely in certain areas. Once mastered, ADCP measurements are faster than traditional wading measurements.

ADCP LIMITATIONS AND SUGGESTED SOLUTIONS

The NMISC discovered that the standard Riverboat wiring harness was not particularly robust. To address the issue, the original harness was replaced with one composed of higher strand count wire and studier connectors. Initially, we encountered wireless connectivity issues between the shore-based laptop computer and the onboard OysterPE, characterized by inter-device communication that was intermittent or could not be established at all. The problem was solved by using a newer laptop computer with a better wireless card. With a laptop computer that is several years old, the laptop battery life is a concern. The NMISC found that the battery charge lasted only a few hours. To solve the problem, NMISC uses a power inverter connected to the field vehicle's power outlet and also carries an extra laptop battery. Netbooks with battery life as long as 9 hours will also address this concern. The Netbook needs to have an operating system such as Windows XP or a Windows XP emulator to run the WinRiver II data acquisition and processing software.

The water depth in the river must be sufficient for this instrument; otherwise another type of ADCP instrument may be more appropriate. The Rio Grande model appears to work best in flows deeper than 2 feet. This presents a challenge because such flows rarely occur in NM unless confined in a narrow channel or during high flows or flooding events.

Moving bed issues must be identified and addressed. The NMISC has not encountered this situation to date, but expects to face this challenge in the future.

Another concern is how to measure during floods with a rope and pulley system at locations with no rope and pulley system in place. The main problem is how to install a rope and pulley or other system so that no safety hazard is created. Limiting or avoiding vandalism to an existing rope and pulley system is also an issue.

A final concern is related to the rope stretching during a measurement. A potential solution is to use steel wire instead of rope to minimize this concern.

SELECTED MEASUREMENT RESULTS

NMISC personnel deployed the ADCP unit from a highway bridge near the USGS 08317400 Rio Grande below Cochiti Dam stream gage. The results of NMISC's preliminary ADCP measurements show relatively good agreement with USGS provisional real time data (Table 1). Figure 3 is a plot comparing NMISC's preliminary ADCP measurements with the provisional USGS real time data.

Table 1. Comparison of NMISC Preliminary ADCP and Provisional USGS Results

Measurement Time on September 16, 2010	Provisional USGS Discharge Result (cfs)	Preliminary NMISC ADCP Discharge Result (cfs)	Difference between Discharge Results (cfs)	Percent Difference between Discharge Results
10:15 AM	736	755	+19	+2.6%
12:45 PM	755	694	-61	-8.1%
1:30 PM	785	771	-14	-1.8%
1:45 PM	795	736	-59	-7.4%

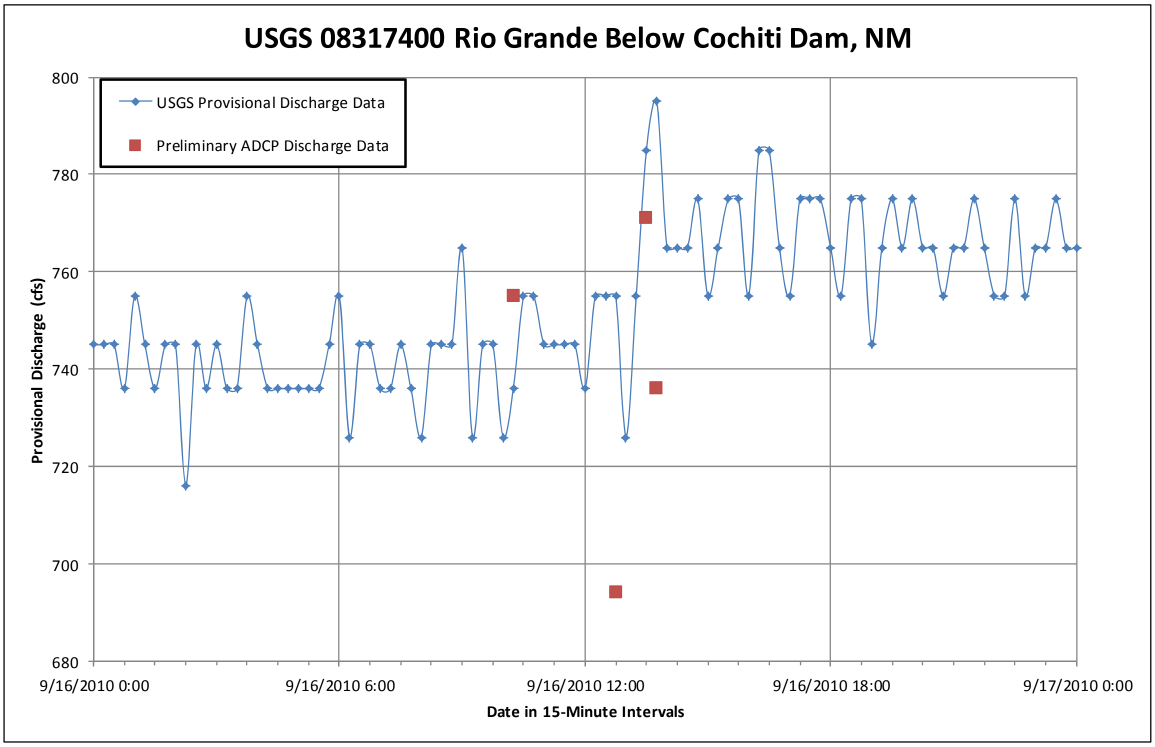

Figure 3. Comparison of Preliminary ADCP and Provisional USGS Discharge Results

The NMISC also deployed the ADCP in the following locations with mixed results:

- Pecos near Pecos, NM – NMISC found that the water was too shallow and the stream was too rocky to obtain good measurements.
- Pecos River below Sumner Dam – NMISC discovered that the water was too shallow and the channel was too rocky to make reasonable measurements.
- Middle Rio Grande Conservancy District (MRGCD) canal near Central Avenue NW in Albuquerque – NMISC obtained consistent results, but they were difficult to verify because of lack of independent gage measurements nearby.
- MRGCD Peralta Main canal on Isleta Pueblo – NMISC observed some variation in repeated measurements. These discharge variations may be real based on an upstream structure that caused some turbulence in the water.

ADCP TIPS

The NMISC found the following suggestions helpful.

- Invest in extra batteries for both the ADCP and the laptop computer.
- Bring extra fuses for the Riverboat equipment.
- Move slowly and steadily when deploying the ADCP from a bridge or via a cableway. Maintain a boat velocity that is slower than the water velocity. This can take some practice.
- Read and get as much training and experience with the equipment and accessories as possible.
- If available, bring a table, chair, stool, and sunshade to the field.
- For office testing and set up of the equipment, acquire a 120 volt to 12 volt converter to minimize difficulties caused by drainage of the 12 volt marine batteries.
- A multimeter is useful for diagnosing electrical connection problems.

CONCLUSIONS

Overall, the ADCP has performed relatively well for NMISC. However, it took longer than expected to learn to use this device properly. The equipment vendors have been fairly responsive to our questions and concerns. Measurement personnel must be able to devote a sufficient amount of time to learn to use the equipment and get appropriate training. It is essential to have the right equipment for the flow conditions to be measured. As with all discharge measurement techniques, a suitable measurement section is required.

ACKNOWLEDGEMENTS

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OPEN-CHANNEL AND PIPE FLOW MEASUREMENT AT MOHAVE VALLEY IRRIGATION AND DRAINAGE DISTRICT USING VENTURI TECHNOLOGY WITH BUBBLER SENSORS

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Alan Jackson[3]

ABSTRACT

Water for irrigation at Mohave Valley Irrigation and Drainage District (MVIDD) is all pumped from the alluvial aquifer along the Arizona eastern side of the Colorado River. This groundwater pumping is administered as diversion from the Colorado River under a contract between the MVIDD and the US Bureau of Reclamation (Reclamation). Previous efforts to measure pumped flows have been largely unsuccessful due to multiple factors including corrosive agents present in the water, limited head availability and/or limited space availability for proper installation and operation of traditional canal measurement structures.

Corrosive agents present in the pumped water have limited the service life for the various flow measurement technologies tried on District pumps. Open channel measurement structures that have been installed at selected sites as part of flow measurement demonstration efforts have met with limited success. Insufficient space between pump discharge and field turnouts or lateral off-takes is a problem for open channel structures at multiple sites. Available freeboard along lined canal sections has proven insufficient for even long-throated flumes, which pose the least head requirements of all critical-flow open channel flow measurement structures.

In an effort to address this challenge, Reclamation's Water Conservation Field Services Program of the Yuma Area Office (YAO) worked with Reclamation's Hydraulic Investigation and Laboratory Services group (HILS) to devise a plan for measuring flow from each well within MVIDD using technologies that would have an acceptable service life and be cost-effective for an agricultural water system. A combination of venturi-type pipe meters and open channel flumes utilizing the venturi solution for either critical flow or submerged operation – all using bubbler sensors to minimize potential for sensor degradation due to contact with corrosive agents in the water – was devised for MVIDD. Installation of measurement equipment was performed by MVIDD staff in Spring and Summer of 2010. Reclamation began performing calibrations of measurement sites during Fall of 2010. Calibrations are expected to be completed by summer of 2011. When calibrations are completed, flow data will be telemetered real-time by radio from each site to the MVIDD office.

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BACKGROUND

Mohave Valley Irrigation and Drainage District is located in western Arizona along the east side of Colorado River a few miles south of the southern tip of Nevada. All MVIDD waters are pumped from the aquifer fed by the Colorado River. Pumped MVIDD flows are administered as diversions from the Colorado River. By virtue of an agreement dated November 14, 1968 between MVIDD and the US Bureau of Reclamation, MVIDD holds entitlement for use of 41,000 acre feet of Colorado River Water annually. Figure 1 is a site map of the district.

Figure 1. Mohave Valley site map

YAO has worked with MVIDD for over a decade in seeking reliable means of measuring pumped flows. Water quality issues and layout of canal systems have proven to be challenging obstacles for measurement technologies traditionally suitable for agricultural water delivery systems.

A high concentration of iron oxide in pumped flows results in deposition of a rust-reddish coating on canal linings and structures that becomes a slimy film when wet and dries to a fine gritty powder. Propeller meters and paddle-type meters that are often utilized for measuring agricultural pipe flow have been shown to have limited life expectancy with this water quality issue. Service life of paddle meters has been on the order of one month while propeller meters typically fail within two seasons. In demonstration tests, affordable acoustic doppler flow meters have performed poorly due to the low suspended solids in the well water.

Issues with canal system layout include limited freeboard with concrete-lined canals, short distances between pump outlets and field turnouts, along with multiple sites where pumped flows entering a canal may be routed in more than one direction to deliver flow to field turnouts. Measuring open channel flows with a standard flume or weir requires a drop in surface elevation at the measurement structure. The required magnitude of this drop varies among standard measurement structures, but must be sufficient to ensure that submergence at a structure does not exceed the modular limit. The additional upstream elevation that would be needed for a critical-flow structure exceeds available canal lining freeboard at numerous sites in the district.

Short distances between pump discharge and field turnouts are also problematic for standard open channel measurement structures due to excessive turbulence in flow approaching the structure, and due to potential asymmetric flow paths downstream. Sites where flow may be routed in more than one direction would at a minimum require investment in multiple measurement structures for measuring flow from a single well. It may also be necessary to follow specific operational procedures at multi-directional sites to ensure the quality of measurement data – which may or may not be tasks hired irrigators would carry out reliably.

Given the issues encountered with flow measurement technologies commonly used for agricultural water delivery systems, MVIDD has utilized a system of monitoring pump run time as a means of tracking the amount of water pumped. This practice has been reasonably straight forward for wells with electric motors where documentation of power usage is readily available. For wells powered by internal combustion engines, YAO cooperated with MVIDD in examining various tracking systems, including installation of vibration sensors linked to data loggers on selected engine-powered wells. Ultimately, this method proved to be too labor-intensive for MVIDD.

In 2008, YAO arranged for Reclamation HILS engineers from the TSC to provide technical assistance in re-visiting options for direct measurement of pumped flows throughout the district. One objective of this effort was to identify a system or systems that would be within the means of MVIDD to operate and maintain. Additionally it was desired (to the extent feasible) to configure a system that streamlines data collection tasks to simplify generation of the required water use reports

PROJECT INITIATION

Armed with knowledge of the assortment of flow measurement ideas that have come up against obstacles at MVIDD, YAO suggested that a selected technology (or technologies) be field tested at a selected site (or sites) before proceeding to develop a district-wide implementation plan. Both pipe flow measurement systems as well as open channel measurement systems were included in feasibility assessments based on recognition that site-specific conditions would likely favor one over the other.

In the case of pipe flow measurement, systems with no moving parts were considered. Flow may be determined using a venturi by measuring the difference in pressure heads measured at the full pipe diameter approach section and measured at the reduced cross-section throat section. Venturi meters meet the criteria of a device with no moving parts in contact with the water. Among differential pressure pipe measurement systems, venturi meters are also known for producing comparatively small head loss. HILS proposed using a bubbler pressure sensing system linked to a solenoid valve bank as a means of measuring head differential with a venturi meter. The rationale behind this idea was that air bubbles being emitted from pressure taps might serve to prevent contaminants in the water from plugging tap orifices over time.

For open channel measurement YAO opted to look at a long-throated flume configuration. YAO has an extensive history with installing long-throated flumes throughout the service area. Based on long-term operating observations YAO had noted that sediment accumulation behind ramp-type long throated flumes had become a maintenance issue at multiple flume sites. To address this problem some of YAO's more recent installations had been configured as laterally contracted structures to minimize potential for upstream sediment accumulation.

FIELD TESTS

Two sites were selected to test flow measurement systems. Site selection factors included landowner/cooperator interest in participating, suitability of site for assessing the technology, and installation simplicity. Both field test sites selected were on lands owned by the MVIDD Board President. The pipe measurement site was readily accessible from a well-maintained road and was in close proximity to the District office. The open channel site was a similar distance from the District office and was in a location where a single measurement structure could measure all discharge from two wells.

Pipe Flow Field Test

The pipe flow measurement field test site (MVIDD well 23) had a 12 in. PVC discharge pipe. The US Department of Agriculture's Water Conservation Laboratory (Replogle & Wahlin, 1994) conducted a study using venturi meters which were constructed from plastic pipe fittings as cost effective flow measurement devices. Following concepts presented in this paper, a venturi meter for this field test site was constructed by installing a bell reducer near the discharge end of the pipe and reducing to a downstream pipe diameter of 10 in.

A rock pile under the pipe allowed limited access to a section of the outlet pipe closer to the well. For this reason the venturi metering section was installed within approximately four feet of the pipe outlet. Initially this meter was installed without an expansion section back to 12 inches for testing purposes. Figure 2 shows the pipe flow measurement field test site.

Figure 2. Pipe flow measurement field test site at MVIDD well 23

The pole-mounted electrical enclosure (Figure 2) houses the bubbler sensor/solenoid valve system, along with the RTU which is programmed to operate the bubbler and valves. Flow measurement data is logged onsite. A base unit at the MVIDD office can be operated to periodically poll data from field sites and write to a data file on the hard drive of a PC linked to the base unit. Bubbler lines are routed from the enclosure through a pipe conduit to tap locations on the pump discharge line. A temporary manometer system was set up to calibrate sensor offsets for each bubbler tap location.

Open Channel Measurement Field Test

Flow approaching the open channel field test site travels approximately ¼ mile from the nearest well. Flow from a second well travels approximately $^3/_8$ of a mile to the open channel measurement site (MVIDD Wells 18 & 19). A long-throated flume that was installed for the field test was prefabricated at the YAO shops using plastic lumber. The canal reach is concrete lined with a trapezoidal cross section. Canal side slopes are 1.25:1 and canal bottom width is 2.00 ft.

The flume itself was designed as a compound contraction with a crest elevation 1.00 ft above the approach section invert and laterally contracted walls with 1:1 side slope. The base of the contracted flume walls meet, leaving a flume with a V-shaped throat section. The field test flume was constructed with a 4 ft long converging section, a 4 ft long throat section and an abrupt expansion. An ultrasonic level sensor was installed in a stilling well to measure canal stage. An RTU unit was programmed to calculate flow based on the electronically-sensed canal stage. The RTU had on-board datalogging capability along with a data display. Figure 3 shows the open channel flow field test site.

Figure 3. Open channel measurement field test at MVIDD wells 18 & 19 measurement site.

YAO Water Conservation Field Services Coordinator Mark Niblack (now retired) is examining the compound contraction long throated flume in Figure 3. The iron oxide deposit seen on the white plastic panels of the flume was accumulated after a single irrigation cycle. As the photo shows the flume caused an increase in upstream water level as evidenced by the comparatively bright colored iron oxide deposits overlaying the darker colored cumulative iron oxide deposits from operations prior to the flume installation. In contrast, recent and cumulative iron oxide deposits on the canal lining below the flume appear to be at the same elevation.

Field Test Findings

Performance of equipment at the pipe flow field test site was encouraging. The site operated from August of 2008 thru March of 2009 without needing service or adjustment. [Farm production at MVIDD is continuous year-round.] A measurement accuracy check was performed by YAO using a Price AA current meter to measure pump discharge in the canal at the time the test site was established. Agreement between the pipe venturi measurement and the current-metered check was within accuracy limits of the current meter method. Based on observations of this field test, the bubbler-sensed venturi meters were selected as the preferred pipe flow measurement technology for the MVIDD flow measurement project.

Feedback for field test flume installed at the open channel field test site was less positive. The flume as designed performed well with no concerns regarding submergence. The increased upstream water level created by the flume narrowed the remaining canal freeboard to a margin the land owner was not comfortable with. The limited amount of available freeboard would be a limitation for conventional long-throated flumes at this and other MVIDD sites where open channel flow measurement was being contemplated.

Another field demonstration project that was concurrently being carried out by YAO with HILS assistance at multiple sites near Yuma featured a technology for open channel flow measurement that would be feasible given the constraints present at MVIDD. In this project long throated flumes that frequently or occasionally operated with submergence in excess of modular limits had been equipped with multiple stilling wells to enable measurement of water levels both in the flume approach section and in the throat section. Using the two measured levels, the same concept that was used for calculating flow rate with a pipe venturi meter could be applied to flow through long-throated flumes ("venturi solution" flumes) under normal or excessive submergence.

MVIDD SYSTEM-WIDE FLOW MEASUREMENT PROJECT DESIGN

Factoring in the accumulated institutional knowledge acquired from a decade of working with the district to devise a credible system for measuring and reporting water usage, a design concept for a flow measurement at MVIDD was formulated. In the design process YAO staff considered each pump site individually.

Where feasible, installation of a flume would be preferred since flumes would have no direct or perceived impact on the operation of a landowner's well(s). At sites suitable for a flume and where sufficient upstream freeboard was not in question, a conventional long-throated flume would be installed. Where a flume could effectively measure flow, but where freeboard was in question, a long-throated flume equipped for venturi solution measurement would be installed. At remaining sites a venturi pipe meter would be installed in the discharge pipe from the well.

In the YAO design for the twenty-six operating wells in MVIDD, one site was identified where a conventional long-throated flume would be suitable for measuring the output from one well. Four more sites were identified where venturi solution long-throated flumes could be installed. Two of the four venturi solution flume sites were in locations where one flume could measure discharge from two wells. Thus the system design called for discharge from seven of the twenty-six wells to be measured by flumes while venturi meters would be installed in the discharge pipe of the remaining nineteen wells.

YAO would assist MVIDD with the installation of flumes. Contractors hired by MVIDD would install the pipe venturi meters. YAO and HILS would assist in performing pressure/level sensor calibrations at all sites and in programming for the RTU units to calculate and record flow measurements.

In order to standardize the system and to simplify installation tasks the YAO-developed plan called for use of commercially available "wafer" type venturi meters that are flange mounted and extend in both directions inside the pipe from a single flanged connection. The cost of wafer venturi units would be higher than the cost of materials for venturi meters constructed of pipe fittings similar to the field test venturi at MVIDD well 23 shown in Figure 2. YAO reasoned that opting to use commercially produced venturi meters would diminish quality control concerns associated with venturi systems that would be contractor installed. Given that the MVIDD staff overseeing the meter installations had no experience with venturi technology prior to set up of the MVIDD well 23 field test site, along with the likelihood that available local contractors might have little or no prior experience with venturi meter installation, this could potentially be a significant concern. Figure 4 is a sketch of a wafer venturi.

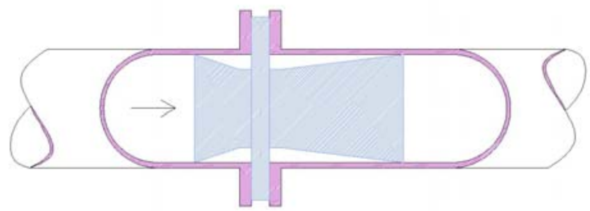

Figure 4. Sketch of a “wafer” venturi meter

MVIDD FLOW MEASUREMENT SYSTEM IMPLEMENTATION

Flumes at MVIDD were installed with the assistance of YAO and HILS in November, 2009. In the Yuma tests of venturi solution flumes it was determined that for this methodology it is crucial to be able to determine level differential between the flume approach and throat sections with a high degree of precision. A water level measurement configuration utilized in the Yuma demonstration testing included stilling wells for both the flume approach and throat taps. Valves that could be operated from above ground were installed in the pipes connecting each stilling well with the respective canal tap. A pipe linking the stilling wells equipped with a similar valve was also installed.

This plumbing system for the stilling wells greatly simplifies sensor calibrations. The stilling wells may readily be isolated from canal flow by closing valves in the tap lines. With stilling wells isolated from the canal, sensor slope calibrations may be performed without impacting canal operations by adding water to, or removing water from, the stilling wells. By opening the valve in the line between wells while at least one of the valves in the canal tap lines is closed, the stilling wells will come to a common level enabling sensor offsets for the respective wells to be accurately calibrated to a common datum. This stilling well configuration was included as part of the venturi solution flume installations in the MVIDD project.

Figure 5 shows the venturi solution long-throated flume that replaced the field test flume shown in Figure 3. The elevation of iron oxide stains seen on the white plastic flume material compared with the elevation of stains on the canal lining on the approach side of the flume (foreground in the photo) show the considerable reduction in upstream level created by this flume compared with the level created by the test flume shown in Figure 3 previously installed at the same location.

Figure 5. Venturi solution long-throated flume at MVIDD 18 & 19

The two stilling wells for measuring water levels in the flume approach and throat sections may be seen at the left of the flume in Figure 5 (white arrows). Standpipes for accessing valves in the pipes connecting the stilling wells to each other and to the canal taps are also shown (black arrows). The venturi solution methodology produces valid flow measurement rates for submergence rates within the flume's modular limit for critical-flow operation as well as for excessive submergence conditions. The venturi solution flumes installed at MVIDD were designed to create a small increase in upstream canal level to minimize problems associated with limited canal freeboard. The result of limiting the upstream stage increase will be high submergence operating conditions that exceed the flume's critical-flow modular limits.

The implementation of the MVIDD project overlapped a staff transition in the YAO Water Conservation Field Services Program. Shortly after overseeing flume installation in the MVIDD project, Mark Niblack retired from Reclamation in January 2010. The post was temporarily vacant until Alan Jackson joined Reclamation mid March 2010. MVIDD was without a Reclamation Point of Contact for the project during the time that arrangements were being made for contractors to install the pipe venturi meters.

After MVIDD had received price quotes for the insertion (wafer) venturi meters, the MVIDD Board of Directors decided to compare the quoted prices with the cost of materials needed to fabricate pipe fitting venturi meters similar to the field test site shown in Figure 2. The Board ultimately decided in favor of installing custom built pipe fitting venturi meters. MVIDD entered into contract agreements with two local firms for fabrication and installation of the venturi meters.

Most of MVIDD wells have steel discharge pipes that run above ground for a short distance between the well and the point of outflow into a canal. Flow from other wells is routed through underground pipelines before emerging above ground and discharging into canals. The well-to-canal conveyance routes differ from a discharge pipe length of about ten feet to over two thousand feet of buried pipe between the well and canal. MVIDD's decision to opt for pipe-fitting venturi meters, the Reclamation Point of

Contact staff transition, the use of multiple installation contractors and the wide variance in well-to-canal conveyance systems all contributed to a high degree of diversity in the resulting MVIDD pipe venturi installations.

At well sites with above ground steel discharge lines a pipe fitting venturi was constructed by installing weld-in bell-shape transitions to reduce to a venturi throat then expand back to the original diameter. The transition was one standard pipe size smaller than the original diameter. Wells with buried pipelines were equipped by the contractors with venturi meters in a variety of configurations. Pipe fitting venturi meters were installed in underground sections near the discharge end of the pipeline at four sites. At another site, the pipe fitting venturi was installed above ground in a pipe emerging from the ground at approximately a 45^{o} angle. At two other sites, pipe fitting venturi meters were installed above ground near the respective wells at locations upstream of buried line sections.

Figure 6 shows a pipe fitting venturi meter at an MVIDD well with an elevated discharge pipe. The installation shown in Figure 7 is at a well with a discharge pipe at the ground surface.

Figure 6. Pipe venturi at MVIDD 11

Figure 7. Pipe venturi at MVIDD 15

At MVIDD site 29 (shown in Figure 8) a pipe fitting venturi was installed on an underground pipeline. Lines connected to the venturi pressure taps were installed prior to backfill, however venturi meter was buried before sensor offset calibration was performed. In Figure 8 the pipe “daylights” just downstream of the buried venturi. Flow is seen discharging into a vertical “riser” section of concrete pipe.

Figure 8. Underground venturi at MVIDD 29

For the well measurement demonstration field test site shown in Figure 2, a simple manometer system consisting of two lengths of $^3/_8$ inch ID clear vinyl tubing each attached to a venturi pressure tap was assembled in the field to calibrate sensor offsets. Heights of the respective water columns were measured from an arbitrary datum. With this simple manometer apparatus, a small but constant fluctuation in water level was observed in water columns in each manometer. Water column values recorded for the calibration were the average of the observed high and low levels.

To calibrate MVIDD's multiple pipe venturi meters, a double manometer instrument was configured that incorporates stilling wells for each tap. Stilling wells were constructed using 2" PVC pipe. Segments of $^3/_8$ inch ID poly tubing were connected from each venturi tap to a port at the bottom of a stilling well. Clear vinyl sight tubes plumbed to a second port at the bottom of each stilling well extend upward along the outside of the stilling wells. Both stilling wells are secured to a metal bracket such that the sight tubes are positioned approximately 1.5 inches apart. Figure 9 shows the manometer apparatus set up for a venturi sensor offset calibration at the MVIDD 20 well.

Calibrations were performed with wells in operation as seen in Figure 9. At most sites the venturi is installed near the pipe outflow. For these installations tap pressure at the venturi throat is below atmospheric. The bubbler sensors for the MVIDD pipe meters were ordered with absolute pressure sensing transducers specified. A more detailed view of the double manometer is seen in Figure 10. The measured head differential of just over three inches shown in Figure 10 represents the head loss being measured across the venturi at MVIDD well 22.

Figure 9. Double manometer calibration apparatus (arrow) set up at MVIDD well 20

Figure 10. Close-up view of the double manometer apparatus

As a calibration is performed, the bubbler system is initially turned off. Poly tubes are installed linking a manometer stilling well to each venturi tap. The manometer linked to the upstream tap is allowed to fill from the tap. For the low pressure throat tap, water is poured into the top of the stilling well and the well is filled to a level higher than the manometer connection at the throat tap. The throat manometer is then allowed to draw down to a level representing a pressure in equilibrium with the tap. Once both manometers reach static condition, water column levels are read. The base of the metal rail to which both stilling wells are attached serves as a convenient arbitrary datum for measurement of each water column. After water column data has been recorded, the bubbler system is activated, and appropriate tap offset values may be determined for the bubbler sensor.

At sites with elevated discharge lines between the well and canal such as MVIDD Well 11 shown in Figure 6, setup for sensor calibration was a straight-forward task. MVIDD Well 15 shown in Figure 7 has the discharge pipe routed along the ground surface. At this and similar sites some minor excavation was necessary for setting up the manometer apparatus due to the fact that the calibration water column surface for the venturi throat tap was below ground level.

CALIBRATION ISSUES

Venturi meters for four MVIDD wells, including Well 29 shown in Figure 8 were installed below ground near the location where the pipelines daylight and discharge into a free surface condition. Pipes and fittings used to construct the venturi meters were not well documented by the installing contractors. After installations had been completed, HILS was able to view remnants of some of the pipe materials used to fabricate the venturi meters. Beyond that a verbal description provided by the contractors of pipe materials used at the respective sites was the extent of information available for devising calibration methodologies.

At the MVIDD Well 17 location the venturi meter was installed near the well at the upstream end of a ¼ mile long pipeline. The venturi tap pressures at this location – which included the energy needed to account for downstream pipeline transit losses – were problematic for use of the bubbler sensing system. Tap pressures were sufficient to push water up the bubbler tubes through the solenoid valves. Iron oxide deposits left behind as water evaporated between irrigations left valves inoperable during the time interval between system installation and sensor calibration. MVIDD has opted to relocate the meter for this well to a location near the discharge end of the pipeline to eliminate potential for water coming into contact with the bubbler sensing equipment.

Multiple issues are present at the MVIDD Well 7 venturi installation seen in Figure 11. As installed none of the upstream, throat or downstream pipe sections have co-linear lines of axis. Outflow at the end of the pipe is approximately half-pipe full. A check of tap pressures was made by connecting the calibration manometer apparatus. The water column linked to the venturi throat tap was slightly higher than the water column linked to the upstream tap. This would suggest that the throat venturi is positioned to be impacted by dynamic head in the flow. At the time of this writing, MVIDD is re-assessing measurement options at the MVIDD 7 site.

Figure 11. Poorly aligned pipe fitting meter at MVIDD Well 7

Accurate knowledge of the cross sectional flow area at the upstream and throat tap locations is key information need for measuring flow using a venturi meter. As noted above, information provided by the contractors regarding pipe materials utilized for venturi installations was limited. For sites where better information was not available, outside pipe circumferences were measured for each tap location. Using the measured circumferences, pipe OD dimensions were calculated and compared against standard pipe dimension tables to identify the "likely" pipe type and ID dimension. Cross section flow areas were then calculated using the presumed ID dimensions.

SUMMARY

An array of issues has impacted the MVIDD flow measurement project with respect to completion schedule along with pipe flow meter quality. At the time of submission of this paper flow measurement systems are measuring discharge from approximately 75% of the MVIDD wells. Most of the remaining sites are expected to be fully operational once meter calibrations are performed. Two of the sites have not yet been calibrated because the pump power units are awaiting diesel motor repairs. One site is in need of pipeline repairs. Issues with measurement at Wells 7 and 17 are noted above.

Flow measurement verifications will be a final task for this project. Verification will be carried out by comparing the flow measurements being generated by the respective MVIDD measurement devices with canal flow measurements obtained using stream gauging techniques. The high degree of diversity in the "as-built" pipe venturi units place added importance on the measurement verifications compared with the level of performance reliability anticipated for the "wafer" venturi units called for in the project design developed by YAO.

Despite cited issues that contribute to flow measurement uncertainty, completion of this project will mark a dramatic improvement in MVIDD's ability to accurately monitor and manage water usage throughout the district. The yet-to-be verified flow data being produced at sites (both flumes and pipe venturi meters) where calibrations have been performed is encouraging. Based on known canal geometry and approximation techniques used to estimate flow velocities, information being produced at all functioning measurement sites appears to be reasonable.

REFERENCES

Replogle, J. A., & Wahlin, B. T. (1994). Venturi Meter Constructions for Plastic Irrigation Pipelines. *Applied Engineering in Agriculture , 10* (1), 21-26.

Gill, T., & Niblack, M. (2009). Flow Measurement with Long-Throated Flumes Under Uncertain Submergence. *Proceedings.* Irrigation District Sustainability – Strategies to meet the Challenges. USCID-Reno, Nevada. June 3-6, 2009

USING AN ADCP TO DETERMINE CANAL SEEPAGE LOSSES IN THE MIDDLE RIO GRANDE CONSERVANCY DISTRICT

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ABSTRACT

Seepage from earthen irrigation canals represents substantial water loss in irrigation districts. Historically, the determination of canal seepage was accomplished using the inflow-outflow method with propeller and electromagnetic type flow meters. This method was difficult, time consuming, and limited by measurement device accuracy. In recent years, advances in technology have lead to the widespread use of Acoustic Doppler Current Profilers (ADCP) for discharge measurements in streams and rivers. Even though ADCP use has become widespread for stream discharges, studies to determine canal seepage using this new technology are limited. Using an ADCP, extensive field measurements were conducted in the Middle Rio Grande Conservancy District. This paper describes the ADCP measurement protocol used to measure irrigation canal seepage and presents predictive equations for determining canal seepage based on flow rate and canal geometry.

INTRODUCTION

According to an Interagency Task Force, the average off-farm water conveyance efficiency for irrigation in the United States is 78% (ITF, 1979) and conveyance loses account for 104 million cubic meters per day (Herschy and Fairbridge, 1998). This seepage represents ten times the daily U.S. domestic water use (Herschy and Fairbridge, 1998). In the Lower Rio Grande Valley, canal seepage accounts for 30-36% of the total diverted water (Fipps, 2001). The major factors that affect seepage rates in irrigation canals are soil permeability, canal length, length and shape of wetted perimeter, water depth, depth to the groundwater table, and presence of other constraints such as wells, drains, and impermeable soil layers (Akbar, 2005; Alam and Bhutta, 2004; Swamee et al. 2000). Some less significant factors include sediment load and size distribution, age of the canal, presence of aquatic plants, viscosity, and salinity of the canal water (Akbar, 2005; Alam and Bhutta, 2004; Swamee et al. 2000).

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Determining canal seepage is usually a difficult undertaking. Fluctuations in canal levels as well as groundwater levels can lead to variations throughout a year and within an irrigation season. Additionally, the amount lost to seepage often falls within the discharge measurement errors of traditional methods.

Overall, the inflow-outflow method has been the preferred method for determining seepage (Alam and Bhutta, 2004; Skogerboe et al. 1999), but is limited by measurement accuracy, time required for measurement, and canal depth and discharge fluctuations. Through the use of an Acoustic Doppler Current Profiler (ADCP) the limitations of the inflow- outflow method can be addressed resulting in high quality, replicable, and efficient measurements of canal seepage.

ADCPs allow for rapid flow rate and velocity measurements in rivers and other open channels (Shields and Rigby, 2005). An ADCP measures the Doppler shift of acoustic signals that are reflected by suspended particles in the water (Rennie and Rainville, 2006; Shields and Rigby, 2005). In recent years the ADCP has become the standard for measuring river discharges as well as velocity distribution (Rennie and Rainville, 2008; Mueller et al. 2007) and ADCP measurements have been shown to be more accurate and as reliable as traditional measurement**.**

One of the primary advantages of an ADCP is the speed and detail in which data can be collected (Carr and Rehmann, 2007). The amount of data that can be collected about velocity characteristics for a given measurement location greatly exceeds traditional methods and techniques, such as propeller or electromagnetic meters (Carr and Rehmann, 2007; Shields and Rigby, 2005). A significant benefit of the ADCP over traditional meters is that no intrusion into a water body is required, which decreases the risk to operators and increases the overall usefulness of the device (Nystrom et al. 2007).

To date ADCPs have not been extensively used for determining canal seepage although they have found widespread implementation for measuring streamflow. This paper presents the use of an ADCP in the Middle Rio Grande Valley to determine canal seepage rates.

Middle Rio Grande Conservancy District

The Middle Rio Grande Conservancy District (MRGCD) was formed in 1925 in response to flooding and the deterioration of previously constructed irrigation works. The district stretches over a distance of approximately 193 kilometers in the Middle Rio Grande Valley in Central New Mexico with 25,000 ha of irrigated agriculture. Water is conveyed in the MRGCD by gravity flow through primarily earthen canals whose total length exceeds 2,400 kilometers.

Only limited measurements of canal seepage have been previously conducted in the Middle Rio Grande Valley and no equations have been developed to predict seepage loss. Because canal seepage losses can represent a significant portion of diverted water and the MRGCD is focused on improving efficiency, a measurement study was conducted to determine canal seepage rates throughout the MRGCD. Through the availability of an ADCP, this study provided the unique opportunity to apply advanced technology in determining irrigation canal seepage rates under normal operating conditions.

MATERIAL AND METHODS

The ADCP model used for this study was the Teledyne RD Instrument StreamPro. The StreamPro is designed to make moving boat discharge measurements in flow depths from 2.36 cm to 2 meters (AuBuchon et al. 2008; Rehmel, 2006) and has a 2,000-kHz frequency with a small four beam transducer head. The processing software provides velocity profile data over an entire cross section (Figure 1).

The inflow-outflow method using an ADCP was chosen for the determination of canal seepage rates in the Middle Rio Grande Valley. This method was selected because it allows for measurement during normal operating conditions, it is a non-intrusive measurement technique, and previous studies have established this as the preferred method in determining canal seepage (Alam and Bhutta, 2004; Skogerboe et al. 1999). The inflow-outflow method is based on creating a water balance in an irrigation canal where inflow and outflow are measured a certain distance apart. These measurements were taken while ensuring that no water is being diverted of introduced into the measurement reach. The use of an ADCP in tandem with pressure transducers ensured that measurement errors associated with fluctuations in water level were addressed. Coordination with water managers was essential to guarantee that inflow-outflow measured sections had no withdrawals through headgates during the measurement period. A previous study conducted by the MRGCD determined that open water evaporation from the canal system was negligible and therefore evaporation was not incorporated into the analysis of the canal seepage water balance determination.

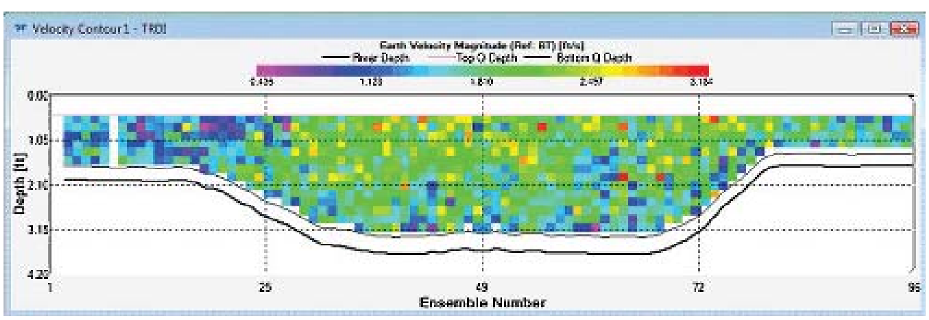

Figure 1. Velocity Profile Measured by ADCP in the Middle Rio Grande Conservancy District (MRGCD)

MEASUREMENT PROTOCOL

The measurement protocol used for the collection of canal seepage data followed the standard USGS ADCP data collection method (Oberg 2005; Simpson, 2001; Morlock 1996). A bank-operated rope and pulley system was deployed and used to move the StreamPro across the channel and back for each transect measurement (Figure 2). Bank-operated pulley setups allow for a more uniform pull, reduced boat motion, and consistent edge measurements.

Figure 2. Bank-operated rope and pulley system with operator and ADCP

All data were collected using the ADCP water mode 12 (WM 12). This is a general purpose mode recommended by the manufacturer (RD Instruments) for high-resolution flow measurements in rivers, streams, and other bodies of water.

In order to verify that storage in the canal was not changing, pressure transducers and temporary staff gages were used during inflow and outflow measurements to monitor water level fluctuations. The pressure transducers used were HOBO brand data loggers manufactured by Onset Incorporated. This data made it possible to determine the exact fluctuation in canal water level.

Once the initial setup and edge data collection was complete, four transects were collected using the USGS ADCP measurement guidelines (Oberg 2005; Rehmel, 2004; Simpson, 2001; Morlock 1996). If the standard deviation between the measurements exceeded 5% of the average, four more transects were collected following the standard USGS protocol. Measurements were conducted on three main canals, three lateral canals, and three acequia (tertiary) canals at three separate times during the irrigation season totaling 25 seepage measurements. The time span of the study was from June11th to October 23rd 2008 with an early, middle, and late season measurement conducted for each canal to address seasonal variability.

The measurements were taken at the upstream inflow and downstream outflow along a significant distance of canal. For each canal, a measurement site was established where a significant length of canal was available for inflow and outflow measurements without water diversions or additions to the flow. To ensure that all irrigation had ceased on the canal, all of the headgates along the canal were checked to see if they were closed. The distance between upstream and downstream measurements was made as long as possible to ensure that a measurable amount of canal seepage could be detected. In most cases this distance exceeded 3.2 kilometers (Table 1). GPS coordinates were taken at both the upstream and downstream measurement locations so that the exact distance between the two stations could be determined using Geographical Information System (GIS) software and maps were created for each canal section measured.

RESULTS

The data collected and subsequent analysis resulted in a database that contained the following information for each seepage measurement location: maximum change in water level, percent change in flow depth, upstream flow rate, downstream flow rate, canal length over which seepage was measured, total change in flow rate across the measured distance, upstream wetted perimeter, upstream flow area, maximum depth upstream, upstream top width, upstream average flow velocity, and percent loss of the inflow rate per mile. The upstream data were chosen for the database so that predictive seepage equations could be applied to upstream channel characteristics. Upstream channel characteristics are well defined for automated measurement sites throughout the MRGCD, and upstream characteristics are also required for determining seepage in the DSS used for scheduled water delivery (Oad et al. 2009). Table 1 displays the database developed from the measurement matrix. Two measurements were removed because of water deliveries from the canal: the Albuquerque Main Canal on 8/20/2008 and on the New Belen Acequia on 7/2/2008. This resulted in a total of 25 seepage measurements.

From the collected data it was determined that main canals exhibited the least amount of seepage with an average seepage rate of 0.64% per kilometer. Lateral canals and Acequia canals exhibited similar seepage rates with an average rate of 1.93% per kilometer and 1.84 % per kilometer, respectively. It was also found that no statistically significant difference in seepage rates existed throughout the season for the nine study canals as the variation fell within the standard deviation. The seepage loss rates obtained resemble results obtained by Fipps (2001) for canal seepage in the Lower Rio Grande Valley. The results also correspond well with a study in a Utah irrigation district that found seepage rates of 2% per kilometer (Napan et al. 2009). The suspected reasons for lower seepage rates in main canals include sedimentation, groundwater and maintenance. The main canals in the MRGCD are all directly connected to the Rio Grande and receive significant fine sediment loads. As water is conveyed down the main canals the sediment eventually settles out in the main canals reducing sediment load in lateral and acequia canals. The settling out in main canals results in soil pores being clogged with finer silt and clay sediment, thereby reducing overall seepage. Another reason for reduced seepage in main canals is the close proximity to the river and subsequent groundwater. Since the main canals originate at the Rio Grande they are not elevated above the river and could be connected to groundwater. Such close proximity to the groundwater would result in a small or negligible gradient for seepage from canal bottoms and to groundwater. Finally, the main canals in the MRGCD receive the most attention when it comes to maintenance and dredging. The main canal shapes in the MRGCD most closely represent the optimized canal sections for minimized seepage presented by (Swamee et al. 2000) and the continued maintenance of these main canals results in a more efficient canal shape and optimized water conveyance.

Table 1 shows the collected seepage data displaying canal name, measurement data, maximum change in water level, upstream and downstream flowrates, canal length over which seepage was measured, total change in flowrate, upstream wetted perimeter, upstream flow area, upstream maximum depth, upstream top width, upstream average flow velocity, and % loss per km.

Table 1.

Canal	US Flowrate (m^3/s)	DS Flowrate (m^3/s)	Length (km)	Total Change (m^3/s)	US Wetted Perimeter (m)	US Flow Area (m^2)	US Max Depth (m)	US Top Width (m)	US Average Flow Velocity (m/s)	Percent Loss per km
Main Canals										
Belen Highline	6.29	6.22	7.04	0.08	16.86	106.09	1.01	15.74	0.70	0.17
Belen Highline	6.54	6.39	7.04	0.15	14.59	127.90	1.09	13.69	0.55	0.32
Belen Highline	4.50	4.21	7.04	0.29	13.80	105.43	1.06	12.28	0.46	0.91
Socorro Main	6.46	6.23	4.44	0.23	10.75	105.72	1.56	8.67	0.68	0.81
Socorro Main	4.57	4.43	4.44	0.14	9.20	78.64	1.08	8.27	0.63	0.67
Socorro Main	3.95	3.85	4.44	0.10	9.65	80.86	1.13	8.78	0.54	0.58
Albuquerque Main	3.98	3.91	2.65	0.07	9.20	72.73	1.35	7.99	0.60	0.70
Albuquerque Main	2.95	2.88	2.65	0.08	8.53	55.28	0.96	7.93	0.59	0.97
Lateral Canals										
New Belen Acequia	1.01	0.95	3.36	0.07	7.07	26.03	0.72	6.49	0.42	1.91
New Belen Acequia	0.81	0.75	4.38	0.06	7.35	20.15	0.36	7.10	0.44	1.81
Bernalillo Acequia	0.80	0.71	5.90	0.09	5.91	32.04	1.06	4.71	0.28	1.84
Bernalillo Acequia	0.78	0.69	5.90	0.09	5.54	34.03	1.07	4.88	0.26	1.95
Bernalillo Acequia	0.84	0.76	5.90	0.09	5.63	32.57	0.92	4.89	0.28	1.75
Barr Main Canal	1.46	1.22	6.72	0.24	6.20	33.39	0.71	5.51	0.46	2.43
Barr Main Canal	1.66	1.40	6.72	0.26	6.70	35.00	0.68	6.12	0.50	2.33
Barr Main Canal	1.43	1.29	6.72	0.14	6.56	34.13	0.66	5.89	0.44	1.44
Acequia or Branch Canals										
Peralta Acequia	0.57	0.51	5.87	0.06	5.40	14.64	0.48	4.89	0.44	1.79
Peralta Acequia	0.76	0.68	5.87	0.07	5.87	24.99	0.76	3.83	0.34	1.65
Peralta Acequia	0.55	0.50	5.87	0.05	5.69	19.45	0.63	4.22	0.31	1.54
Sili Main	0.47	0.41	5.73	0.06	5.50	19.51	0.48	5.05	0.25	2.21
Sili Main	0.58	0.51	5.73	0.07	5.07	21.34	0.54	4.62	0.28	2.15
Sili Main	0.64	0.58	5.73	0.06	5.14	23.09	0.57	4.49	0.28	1.67
Williams Lateral	0.61	0.58	2.87	0.03	3.50	12.75	0.68	2.89	0.55	1.88
Williams Lateral	0.67	0.63	2.87	0.04	3.57	13.94	0.67	3.10	0.55	1.94
Williams Lateral	0.69	0.66	2.87	0.03	3.47	14.74	0.72	2.90	0.53	1.71

Further analysis of the data showed that trends in canal seepage rate existed for upstream flow rate, and the three canal geometry properties of upstream wetted perimeter, upstream flow area, and upstream top width. The data showed that as canal inflow rate decreased the seepage increased. For the wetted perimeter, flow area, and top width data, the seepage increased as these values decreased. In order to develop predictive equations, the characteristics of the upstream cross section were related to the percent loss per mile.

Correlation between Seepage Loss and Flow Rate

Analyzing the data for seepage rate versus upstream flow rate exhibited an exponential trend (Figure 3). This relationship exhibited a coefficient of determination (r^2) of 0.80 and is displayed in Figure 3 as well as Equation 1.

$$S = 2.34e^{-0.28Q} \qquad \text{Equation 1}$$

Where S= percent seepage loss per kilometer (%)
Q = inflow discharge (m^3/s)

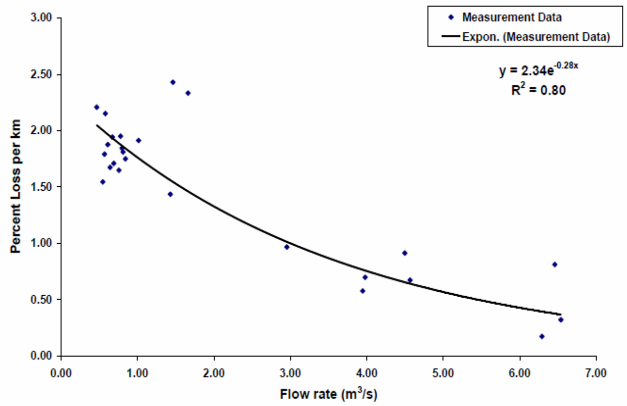

Figure 3. Relationship between upstream flow rate and percent loss per km

Correlation between Seepage Loss and Canal Geometry

In addition to analyzing the inflow rate versus seepage loss, geometric properties of the inflow canal were plotted against the seepage rate. The three geometric properties that exhibited the most significant predictive equations were wetted perimeter, flow area, and channel top width. The data for seepage rate versus upstream wetted perimeter exhibited an exponential trend (Figure 4). The exponential relationship developed exhibited a coefficient of determination (r^2) of 0.79 and is displayed in Figure 4 as well as Equation 2.

$$S = 4.54e^{-0.17P} \qquad \text{Equation 2}$$

Where S = percent seepage loss per kilometer (%)
P = wetted perimeter (m)

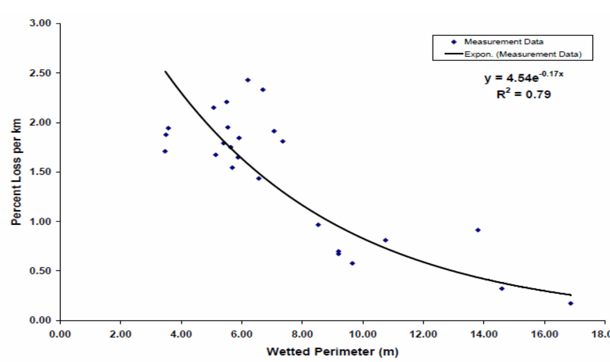

Figure 4. Relationship between wetted perimeter and percent loss per km

The data for seepage rate versus upstream flow area also exhibited an exponential trend (Figure 5). The exponential relationship developed exhibited a coefficient of determination (r^2) of 0.76 and is displayed in Figure 5 as well as Equation 3.

$$S = 2.70e^{-0.18A} \qquad \text{Equation 3}$$

Where S = percent seepage loss per kilometer (%)
A = inflow area (m^2)

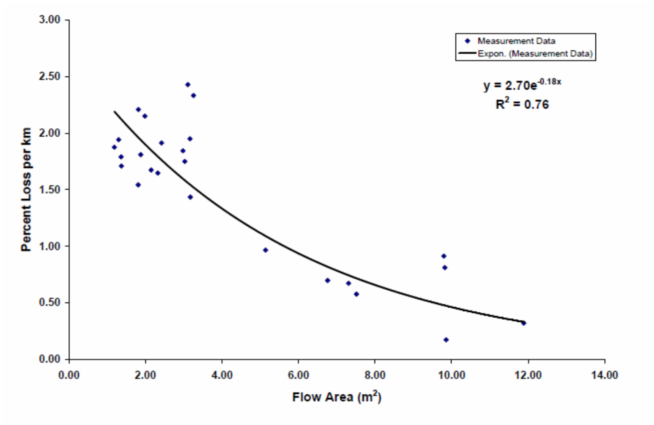

Figure 5. Relationship between flow area and percent loss per km

The data for seepage rate versus upstream top width also exhibited an exponential trend (Figure 6). The exponential relationship exhibited a coefficient of determination (r^2) of 0.78 and is displayed in Figure 6 as well as Equation 4.

$$S = 4.10e^{-0.18T} \qquad \text{Equation 4}$$

Where S = percent seepage loss per kilometer (%)
T = top width (m)

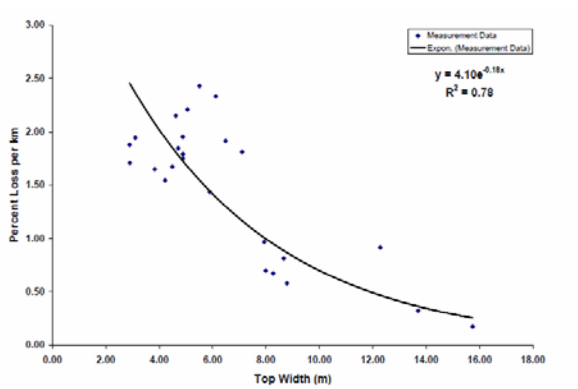

Figure 6. Relationship between top width and percent loss per km

Although the equation for top width is a function of velocity and cross sectional area it will be useful to the MRGCD as ditch-riders and water managers will be able to predict seepage using only the top width of a canal.

These equations present the opportunity to predict canal seepage losses based on the four easily measured parameters of inflow rate, wetted perimeter, flow area, and top width. These equations should only be applied to similar systems and to canals that are comparable in size to the ones measured during this study. The developed equations display r^2 values similar to other published studies. A study by (Hotchkiss et al. 2001) in Nebraska was able to develop predictive canal seepage equations with coefficients of determination of 0.64 and 0.77. Another study in Australia by (Akbar, 2005) developed numerous predictive seepage equations with coefficients of determination ranging between 0.40 and 0.93. Through the development of the equations for the MRGCD, district managers are able to predict seepage. Using the developed seepage equations the total seepage in the MRGCD for 2008 was calculated to be 72,000 acre-feet which is 20% of the total diversion. A similar seepage rate of 15% of the total diversion was found in an Alberta irrigation district (Iqbal et al. 2002).

DISCUSSION AND CONCLUSION

The completed study to examine canal seepage in the MRGCD provides the framework for using technology in the form of an ADCP to determine canal seepage in an irrigation district. ADCPs offer the benefit of reducing measurement error, measurement time, offer high resolution data collection, are non intrusive, and allow for the collection of canal seepage data during normal canal operation. Coupled with a pressure transducer to ensure that canal fluctuations are limited, the presented methodology offers the opportunity to determine canal seepage quickly, accurately, and efficiently.

The developed equations only apply to the Middle Rio Grande Valley or to irrigation systems that are geologically and hydrologically similar. Although the data collected to develop the equations showed no significant seasonal variation there is the possibility that seepage varies from year to year and further investigation is necessary. The two most useful equations to the MRGCD will most likely be Equations 1 and 4 which relate canal inflow and top width to seepage loss rate, respectively. The variables of canal inflow and canal top width are easily obtainable and require minimal effort for data collection. The MRGCD utilizes a network of automated measurement stations (Gensler et al. 2009) which will aid in determining canal inflow, which can then directly be related to a canal seepage rate. Determining the canal top width will be straightforward because many bridges exist across canals allowing ditch-riders and water masters to measure the canal top width to estimate canal seepage.

Using diversion records obtained from the automated measurement network, the MRGCD will also be able to quantify the aquifer recharge from the canal system in the Middle Rio Grande Valley. The length of each canal as well as the inflow for said canal is well defined and the developed equations will allow for calculation of canal seepage rate. The benefit to the MRGCD will be proving the amount of water that the canal system recharges to the regional aquifer. The city of Albuquerque and several smaller communities pump from the regional aquifer, and it is believed that aquifer levels are maintained through the seepage from the Rio Grande and MRGCD irrigation canals. Quantifying the amount of seepage that occurs from the MRGCD canals indicates the benefit that the canal network has on the local aquifer and aids the MRGCD in water rights litigation. Application of the developed equations may help to determine areas where canal maintenance or lining would have the greatest benefit in water saving.

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REFERENCES

Akbar, S., 2005. Measurement of losses from on-farm channels and drains. Cooperative research centre for sustainable rice production. Yanco Agric. Institute. Yanco.

Alam, M.M., Bhutta, M.N., 2004. Comparative evaluation of canal seepage investigation techniques. Agric. Water Manage. 66, 65-76.

AuBuchon, J., Benoit, V., Donnelly, C, Devergie, S., 2008. ADCP field tests comparison: utilizing RD Instruments StreamPro ADCP. Bureau of Reclamation. Technical Services Division, Albuquerque, New Mexico. River Analysis Group.

Carr, M.D. and C.R. Rehmann. 2007. Measuring the Dispersion Coefficient with Acoustic Doppler Current Profilers. J. of Hyd. Eng. 133, 977-982.

Fipps, G., 2001. Potential water savings in irrigated agriculture for the Rio Grande Planning region (Region M). Rio Grande Regional Water Plan. Final Report. December.

Gensler, D. Oad, R. Kinzli, K., 2009. Irrigation system modernization: a case study of the Middle Rio Grande valley." ASCE J. of Irrig. and Drain. 135(2).

Herschy, R.W., Fairbridge, R.W., eds., 1998. Encyclopedia of water resources, Kluwer Academic, Dordrecht, The Netherlands. 442, 777–778.

Hotchkisss, R.H., Wingert, C.B., Kelly W.E.. 2001. Determining irrigation canal seepage with electrical resistivity. ASCE J. of Irrig and Drain. 127, 20-26.

Interagency Task Force (ITF)., 1979. Irrigation water use and management. U.S. EPA Interagency Task Force Rep., U.S. Department of Agriculture, Washington, D.C.

Iqbal, Z., Maclean, R.T., Taylor, B.D., Hecker, F.J., and D.R. Bennett. 2002. Seepage losses in irrigation canals in southern Alberta. Canadian Biosystems Engineering. 44, 22-27.

Morlock, S.E., 1996. Evaluation of acoustic Doppler current profiler measurements of river discharge. U.S. Geological Survey Water Resources Investigation Rep. 95-4218, Denver.

Mueller, D.S., Abad, J.D., García, C.M.,Gartner, J.W., García, M.H., Oberg, K.A., 2007. Errors in acoustic Doppler profiler velocity measurements caused by flow disturbance. J. of Hyd. Eng. 133, 1411-1420.

Napan, K.M., Merkley, G.P., Neale, C.M.U. 2009. Seepage Evaluations in Cache Valley Irrigation Canals. Proc. Fifth International Conference on Irrigation and Drainage. USCID, Salt Lake City, UT.

Nystrom, E.A., Rehmann, C.R., Ober, K.A.., 2007. Evaluation of mean velocity and turbulence measurements with ADCPs. J. of Hyd. Eng. 133, 1310-1318.

Oad, R., Garcia, L., Kinzli, K., Patterson, D., Shafike, N., 2009. Decision support systems for efficient irrigation in the Middle Rio Grande valley. ASCE J. of Irrig. and Drain. 135. 177-185.

Oberg, K.A., Morlock, S.E., Caldwell, W. S., 2005. Quality assurance plan for discharge measurements using acoustic Doppler current profilers. Scientific Investigations Rep. 2005-5183, U.S. Geological Survey, Denver.

Rehmel, M.S., 2006. Field Evaluation of Shallow-Water Acoustic Doppler Current Profiler Discharge Measurements. World Environmental and Water Resource Congress. ASCE Conference Proceedings, 200, 162.

Rennie, C.D., Rainville, F., 2008. Improving precision in the reference velocity of ADCP measurements using a Kalman filter with GPS and bottom track. J. of Hyd. Eng. 134, 1257-1266

Rennie, C.D., Rainville, F., 2006. Case study of GPS differential correction strategies: influence on ADCP velocity and discharge estimates. J .of Hyd. Eng. 132, 225-234.

Shields, F.D. Jr., J.R. Rigby., 2005. River habitat quality from river velocities measured using acoustic Doppler current profiler. Environ. Manage. 36, 565-575.

Simpson, M.R., 2001. Discharge Measurements Using a Broad-Band Acoustic Doppler Current Profiler: Sacramento, CA, United States Geological Survey, pp. 123.

Skogerboe, G.V., Aslam, M., Khan, M.A., Mehmood, K., Mehmood, S., Khan, A.H., 1999. Inflow-outflow channel losses and canal lining cost-effectiveness in the Fordwah

Eastern Sadiqia Project. Report No. R-85. International Water Management Institute, Lahore, June. pp. 3-8.

Swamee, P.K., Mishra, G.C., B.R. Chahar., 2000. Design of minimum seepage loss canal sections. ASCE J.of Irrig. and Drain. 126, 28-32.

WHAT'S IN YOUR RADIO COMMUNICATION TACKLE BOX?

Dan Steele[1]

ABSTRACT

The sporting anglers today have the latest gear and lures available to help catch that prize fish in their favorite fishing spot. They have a lot of help on the equipment side with the current technology that is available today. The new rods are much more flexible, lighter, stronger and the same applies to the reels that hold the fishing line; the use of technology is seen everywhere, right down to the fishing hooks and lures that today's anglers can use.

We see the same things with the wireless communications market. Radios have become faster, smarter and easier than ever to program and upgrade firmware. The biggest advancements are happening on the Input/Output (IO) side of the pond. We see more options and frequencies that can be used today than ever before. The same is true with cell modems and satellite radios to competitive radio hardware. There are new software programs that act like a Swiss army knife – it can program the radio, update firmware, create network design templates and gather diagnostic information all from the same software. Someone can even create a network design and program the radio for each specific location and have peace of mind that it has all the correct settings required. This paper will explore the infinite possibilities and present some applications others have already tried and proven in the field or pond.

INTRODUCTION

Figure 1. Tackle Box

Radio Communication Tackle Box Contents

It might consist of a laptop computer; diagnostic software – ToolSuite, Wireshark and others; Bird 43 watt meter; radio spectrum analyzers; test radios; Omni and Yagi antennas; RF coaxial cable; cable jumpers; lightning protection; weather proofing and grounding kits; radio path studies; diagnostic tools; DC power supplies; radio antenna

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masts and towers; and knowledge of radio types and SS and licensed frequencies with Serial and/or Ethernet devices and cables.

Programs like Tool Suite (aka: Swiss army knife) have a design template that can be used to create a new network and later the user can program the radios from this template. You can program a new radio, update firmware, perform both local and network diagnostics, plus it has the spectrum analyzer tool that can show you the noise floor in the radio frequency that is being used. If a customer has been using the older diagnostic software tools (Comm-Control and Radio Config.) they can import those networks into Tool Suite and perform a reverse design template and have a template of their existing network to use with the new software.

With the release of several new I/O (input &output) and IP or Ethernet products in both 900MHz and 2.4GHz, users have more options today at a lower price point than ever before utilizing wireless I/O and IP Supervisory Control and Data Acquisition (SCADA) applications. They can do wire replacement mode or use Modbus protocol.

Figure 2. Tool Suite diagnostic software and Bird Model 43 watt meter

IMPORTANT CONSIDERATIONS FOR RADIO COMMUNICATIONS LET'S GO FISHING!

Figure 3. Developing plans and goals before you start are critical to your success

- Preparation and knowledge are essential!
- What are the short and long term plans/goals?
- Who is the master/administrator and does he/she knows what is expected?
- Who's going to perform the work?
- What is the time table and time of completion?
- Do you have a radio path study – either in field or computer generated?
- Do you have a network design and list of end devices and connection types? Will there be I/O devices either wire replacement or using Modbus protocol?
- Are you looking at hybrid networks using Ethernet and serial radios? Will there be IP or Ethernet security cameras?

The leading radio manufacturers are developing several new products to help with this demand. There are all types of I/O radios and a new series of radios that include I/O expansion modules that utilize Modbus protocol, which is the industry standard. A new hybrid radio has 4-RJ-45 ports, two for Ethernet and two for serial communications. New cathodic protection radios have the ability to connect to rectifiers from 110-480VAC with a one step down transformer that will drop the voltage down to 12VDC. It can accept either positive or negative voltage shunts as well and bring in your pipe to soil test stations. The ease of adding I/O into existing networks that already have serial radios makes this task quite easy and if repeaters are needed they can be added to the radio network. There also are new software programs that can program the radio, update firmware, create network design templates and gather diagnostics all from the same software. You can create a network design and program the radio for each specific location and know that it has all the correct settings required.

Different Topologies, Network Sizes & Data Requirements

Each installation is unique and the user must take into consideration several factors to ensure they have selected the appropriate topology, network size and meet the necessary data requirements for the system to run properly. Some of the specific items include:

- Field size and location – where is the data going?

- Topology and vegetation types and challenges
- Feel the need for speed? What are you bringing back to the host and are you trying to tie into the network from the field?
- Video, IP, Programmable Logic Controllers (PLCs), Wi-Fi and other Ethernet devices
- Combining different types of data to your SCADA network
- New field challenges and how to get your data back to the host
- Microwave – spread spectrum or licensed radios, ring or self healing link architectures?
- Cellular networks – where and why you might use them
- Satellite networks – for those really remote areas….

System Needs and Long-Term Goals

First and foremost, one of the most critical steps to instituting a reliable and comprehensive radio communication network system is to understand the current and future needs of your system's requirements. Compiling a set of standards and requirements provides a nice overview of where the system's capabilities are currently and will give some insight as to where the system needs are headed. It is imperative to involve experts in the field and all stakeholders, because of the various perceptions and ideas that people will bring to the table. Developing a set of long-term needs and criteria for a radio communications network system with as much input as possible will better equip your company to meet its changing needs and requirements.

Second, it is important to understand your organization's long-term communication goals. What is the company trying to achieve with this communication network? Do these goals align with the future needs of your system? Is the company going to need this communication infrastructure to be shared for different applications? How do the long-term goals of your communication network apply and compare to the long-term goals and vision of the company in general? Failure to completely understand all of these considerations in the short-run will increase the odds of future complications and can even lead to a system failure.

Detailed Planning

Once long-term goals are established and everyone has mapped out the needs of future system requirements, it is time to get into the detailed planning stages of a radio communication network. For instance, how frequently does the data acquisition need to be conducted for your organization?
There are many options to consider such as: On-demand, hourly, daily, on exception, changes in status, etc. How is the data supposed to be delivered? Do you want raw data or data packet sizes? Do you want large or small data packets? Do you want the data to be streamed, polled or reported by exception? What are the latency or performance requirements? What aspects of communications are important to ensure, so that the application or component does not take too long to complete a user task? By considering these questions and thought processes, it will help to identify the appropriate technology

choices early on in the design. By applying the defined goals for the future and in considering the current status of your system, the answers to the above questions will help lay out the overall technological needs of the radio communication network.

Know the Market

Competition brings about the creative nature of everyone involved. When you consider installing or adding to your radio communication network, it is important to pay attention to what your competitors and neighbors are doing. This information will provide better understanding as to what they are trying to achieve and how the overall landscape of the market is changing. The focus should first and foremost be on your core competencies. Therefore, it is important to make your choices based on your individual needs and goals and not what your competitor or neighbor is doing. When entering into a new realm of possibility, always verify the performance of your supplier, contractor and manufacturer. Don't blindly trust market buzz or advertising – get as much information as you can and confirm the credentials of your technology partner. The name of the game in communication networks is reliability if you want to make sure you are going to have all your needs met, in order to reach the goals you set for yourself and your company.

Selecting the Appropriate Technology

Selecting the technology that best fits your requirements obviously is an important decision to consider now that the overall goals and system requirements need to be determined. As with any big decision, you should take the time to learn about the various technology options. Learn their advantages and limitations and how each option could play into the future development of your communication system. It is beneficial to be a skeptic in this instance because one size doesn't fit all when it comes to communication networks. Each system is different, because of its location or topography, data speed requirements, performance or polling times, and backward compatibility issues. Because every communication network is unique, the option of hybrid networks may offer the best technical approach to solving diverse needs and requirements.

Budget Considerations

The budget usually plays a very large role in determining what kind of technology you decide to introduce for the communications network. Obviously, cost is a driving factor for many decision-makers. Consider a radio communication network an investment in technology as a capital expenditure, because it is an upfront investment for the future, or, in other words, it is an expenditure that is creating future benefits for you and your company. In addition, it is crucial to consider the operating expenditures because there are recurring charges to consider based on the technology options you choose.
Lastly, there are various maintenance, repairs and service charges that will need to be taken into consideration, so you aren't over budget in the future. Financing is a viable option for these new communication technology assets that are being introduced into your company. Therefore the depreciation method (tax deduction) can be used to recover the costs of these assets. By mapping out these budgetary considerations, you can also get

a snapshot of the benefits and savings you can accrue by making certain choices over others. As long as you match your choices with your needs and budget, everything will fall into place. Perhaps this viewpoint will help you to consider a hybrid solution for your radio communication network.

Case Study Example: Utilizing Radio Communication Tools to Solve SCADA Network Issues

For the City of St. George and Washington County Water Improvement Districts in Utah, getting reliable and accurate data from hundreds of I/O points scattered throughout a very large and diverse topographical area was a significant issue. Some of the challenges they faced were:

- Wider area for source and use of water
- Network type(s) and arrangement
- Protocols and Conversions

To minimize cost without sacrificing performance, FreeWave Technologies, a wireless data radio provider designed a system using lower-cost FreeWave FGRIO and FGR Series radios to handle single and low I/O data points using the ModBus protocol. Additionally, satellite cells were coupled together through a network of its HT-Plus Ethernet radios to the main control PC that graphically displays the data and logs the important points at pre-determined intervals. The HT-Plus radios have two serial ports which allow the City of St. George and Washington Country to 'back link' the serial data radios to them and send data without converting it to an Ethernet protocol. They also use PLCs at control points and configure the PLCs to communicate through serial ports to ModBus devices using the FGR radios and then connect to the backbone through their TCP/IP ports.

By using these methods of coverage, the water improvement districts now have a backbone system that is more than 100 linear miles in length, and branches covering many more miles. The I/O count is in the thousands, with more than 200 data sites. The sites are a mix of utility (A/C powered) and solar-powered devices, with the solar sites designed to perform for several cloudy days without interruption of service. The combination of hardware with the ModBus protocol also has minimized the need to replace or upgrade field hardware.

By carefully evaluating their wireless options in their "communication tackle box," these water improvement districts were able to fully leverage available radio communication tools and now have reliable, secure coverage for their SCADA network.

CONCLUSION

The key to building a successful radio communication network is to examine all the tools in your communication tackle box. The options today are endless with continually improving radios, firmware and software, plus the option of hybrid solutions to optimize your communications requirements. In order to have the most reliable system possible, it is essential to plan your network, understand the technology, be aware of the total budget and know the market. Being aware of these critical factors will open users to the infinite possibilities available and let them learn about applications others have already tried and proven in the field or pond. Last - enjoy the Fishing!

REFERENCES

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VENTURI METERS CONSTRUCTED WITH PIPE FITTINGS: AN UNDER-APPRECIATED OPTION FOR MEASURING AGRICULTURAL WATER

Tom Gill[1]
Brian Wahlin[2]
John Replogle[3]

ABSTRACT

Increasing competition for limited water supplies, improved technology for managing water delivery systems, and a growing importance in being able to document use of water supplies are all factors driving interest in establishing the capability to measure flow at an expanded number of locations in agricultural water delivery systems. Pipe Venturi meters are widely recognized as a measurement technology in piped systems offering a high degree of accuracy while imposing comparatively small head loss. Researchers at the Agricultural Research Service have documented their efforts in using off-the-shelf PVC fittings to produce "constructed Venturi meters" as a low-cost option for measuring water in agricultural systems. These devices can achieve an accuracy on the order of ±2% for a cost of about $180.

Despite many attractive attributes of this flow measurement concept, this technology has seen a limited degree of adoption. This paper examines field installations where constructed Venturi meters have been used to measure flows over a range of magnitudes and under a variety of data collection methodologies using a case study format. Guidelines for construction and installation are also presented.

INTRODUCTION

Replogle and Wahlin (1994) introduced the idea of creating low head loss Venturi meters constructed from plastic pipe fittings. True Venturi meters do not have stagnant zones, are more tolerant of upstream conditions, have lower head loss, avoid fouling problems, and are more accurate than most others meters. However, true Venturi meters are quite pricey and are typically beyond the means of most irrigation districts. The plastic pipe fitting Venturi meters suggested by Replogle and Wahlin (1994) avoid the issue of high cost while maintaining the other benefits associated with Venturi meters. The original paper described experiments in which 15 Venturi-type meters were constructed using plastic pipe fittings that had symmetrical configurations (i.e., similar converging and diverging cones). By reversing flow through the meters, 30 configurations were available to assess the construction capability to make appropriate piezometer taps that responded the same to flow in either direction. With the 30 Venturi meters, an attempt was made to

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evaluate the statistical variability due to construction techniques and manufacturing differences in commercially available plastic pipe fittings. The results of the experiments indicated that the Venturi meters could be constructed for about $180 and could be constructed in about 2 hours. Using a standardized rating curve developed as part of the experiments, the accuracy of these meters is approximately ±2%, not including the errors associated with the readout method.

THEORY

Venturi meters represent one of the oldest and most reliable of the differential head meters. These devices are well defined in the literature and little new information is available (see ASME (1971) and Brater et al. (1996) for a more complete treatment). Certain angles of convergence and divergence must be observed for standard Venturi-meter behavior. The conduit walls should converge at about 20° and diverge on the downstream side at about 5 to 7°. The approach piping requirements are similar to those for orifices; however, they can be relaxed somewhat with few detrimental effects. A frequently used Venturi meter is the Herschel-type Venturi tube. It has a converging cone of 21° ±1° and a diverging cone of 7 to 8° (see Figure 1). The throat length of these meters is equal to the throat diameter. This is considered by many users to be the "standard" or "classical" Venturi meter. The angle of the diverging cone does not influence the calibration coefficient, but it does have an effect on the overall head loss through the tube. Commercially produced Venturi meters claim a primary device accuracy of ±0.5% (ASME, 1971).

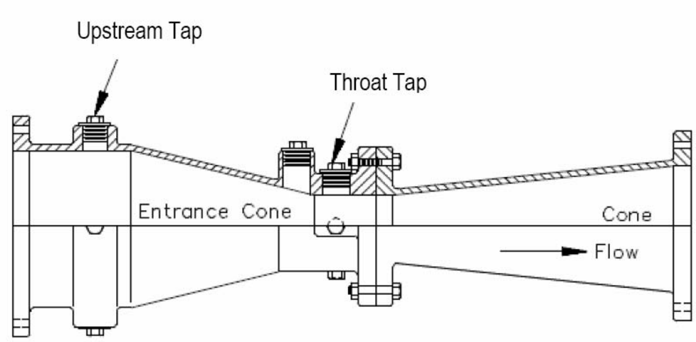

Figure 1. Schematic Diagram of a Standard Venturi Meter

The basic expression for discharge, *Q*, is derived from the classical Bernoulli Equation and can be written in a form that is applicable to round pipes or other conduit shapes as:

$$Q = C_d \frac{A_p A_t}{\sqrt{A_p^2 \quad A_t^2}} \sqrt{\frac{2g}{\alpha}(h_p - h_t)}$$

where C_d = discharge coefficient (typically between 0.96 and 0.99 for standard Venturi meters)
A_p = area of approach piping
A_t = area of contracted throat section
g = gravitational constant
α = velocity distribution coefficient (assumed to be 1.02)
h_p = upstream pressure tap reading
h_t = throat pressure tap reading

EXPERIMENTAL SETUP

A schematic diagram of the plastic pipe Venturi meter is shown in Figure 2. These devices were constructed using commercially available PVC pipe and fittings. The total construction cost is about $180 US (2010) for the materials plus the cost of about two hours of labor. Once the meters were constructed, they were calibrated using a weigh-tank-and-timer system that is accurate to about ±0.1%. Initially, three Venturi meters were constructed with different throat lengths to determine the optimal throat length. In addition, there were two types of converging fittings that were tested: one with 15° contraction and one with a 25° contraction. The need for multiple pressure taps around the throat section was assessed by installing four pressure taps at 90° intervals around the center of the throat section. These taps were hydraulically connected for one series of tests, to give an average pressure reading for the group. Next, they were grouped into two opposite pairs, and, finally, they were separated and read individually. Once the throat length and pressure tap locations were determined, 12 more meters were constructed and calibrated. All the meters then had the flow direction reversed and were calibrated again. Thus, the 30 unique calibrations obtained from the various Venturi meters were used to determine the scatter of calibration for these plastic devices.

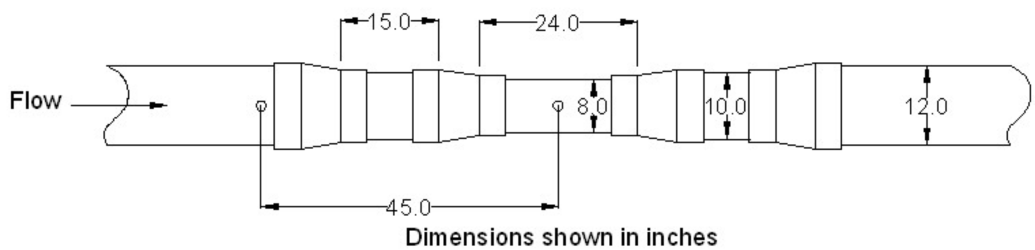

Figure 2. Schematic Diagram of Plastic Fitting Venturi Meter

EXPERIMENTAL RESULTS AND MANUFACTURING RECOMMENDATIONS

Plastic pipe fittings of the kind usually used by the irrigation industry can be fashioned into suitable Venturi meters with an expected accuracy of ±2%, not including the errors of the readout method. The discharge coefficient for the plastic fitting Venturi meter is given by:

$$C_d = 0.964 - 0.0466 e^{\left(-R_n/284{,}000\right)}$$

where C_d is the discharge coefficient and R_n is the Reynolds Number based on pipe diameter. The experimental discharge coefficients for the plastic pipe Venturi meters ranged from about 0.92 to 0.96, slightly less than the discharge coefficients for true Venturi meters. Other conclusions from Replogle and Wahlin (1994) include:

- It is recommended that a throat length of three times the throat diameter be used for plastic Venturi meter construction. Shorter throat lengths appear to cause difficulties in pressure detection due to flow separation. Longer throat lengths produce excessive head loss.
- The rate of contraction of the reducer fittings (i.e., 15° versus 25°) caused no significant change in C_d, and thus the meter calibration. However, the fittings with the 25° contraction rate exhibited a greater total head loss through the meter than those with the less severe 15° contraction rate.
- The most important construction factor is the fabrication of the pressure taps and the immediate connections. They should be drilled with appropriate backing blocks to reduce burrs and with a guide to assure that they are constructed perpendicular to the pipe wall. It is recommended that the pressure taps be installed on the sides of the meter to prevent air bubbles from entering the pressure lines. It is not necessary that the pressure taps be on the same horizontal line and the meter can be mounted at any angle.
- Slow-setting PVC cement should be used to allow workers sufficient time to uniformly assemble and adjust large pipe parts.
- The cost of pipeline parts is within the economic range of most irrigation applications. [Costs are about $180 U.S. (2010) for the pipe and fittings, plus about two hours of labor, per meter.]

FIELD INSTALLATION CASE STUDIES

Three field installation case studies are presented that show the versatility of pipe-fitting Venturi systems for measuring flow either as stand-alone low-tech installations, or as part of an automated data collection network. The first two case studies presented document field demonstration projects that were established to examine performance over time in terms of reliability and long-term cost effectiveness. The third case study included is a brief discussion of a temporary measurement installation where a pipe-fitting venturi was utilized to measure flow as part of an irrigation research project.

Pioneer Irrigation District

The Pioneer Irrigation District (PID) diverts flow from the North Fork Republican River in Yuma County, CO and has historically delivered irrigation water to farmlands in extreme eastern Yuma County and in western Dundy County, NE. During the 2003 and 2004 irrigation seasons, the Water Conservation Field Services Program of Reclamation's Nebraska-Kansas Area Office (NKAO) arranged for engineers from Reclamation's Hydraulic Investigation and Laboratory Services group (HILS) in Denver, CO to provide technical assistance to the PID in establishing flow measurement capability at each operating farm turnout.

Site-specific constraining conditions dictated use of multiple flow measurement technologies in the project. The pipe-fitting Venturi system developed by Replogle and Wahlin (1994) was proposed to PID as a cost-effective measurement option that may be applicable for some of the PID turnouts. PID agreed to work with HILS engineers to set up a demonstration site using a pipe-fitting Venturi.

The site selected for the demonstration project featured a 12 in. pipe turnout from the PID canal. A pipe-fitting Venturi was constructed using PVC bell reducer fittings in a configuration similar to that shown in Figure 2 to reduce from 12 in. to 10 in. then from 10 in. to an 8 in. diameter Venturi throat. Downstream from the throat the pipe was expanded through two steps back to 12 in. diameter using a mirror image configuration of the fittings used for the contraction. Metering taps were installed in the 12 in. pipe just upstream of the initial reducing fitting, and at mid-length of the throat section. For the demonstration site, a third tap was installed in the downstream section after pipe diameter was expanded back to 12 in as a means of showing the head loss through the meter. Figure 3 shows the installation of the venturi meter at PID the demonstration site.

Figure 3. PID Ditch Rider Dennis Waggoner (l) and Ditch Superintendant Dan Korf (r) assisting with the May, 2003 pipe-fitting Venturi demonstration site installation

A specialized manometer board was fabricated for the PID demonstration site that featured a sliding scale. The scale was marked to show head differential in feet and also to show flow rate in gallons-per-minute (the flow rate measurement units historically utilized by PID). To determine flow rate or head differential, the sliding scale would be raised until the zero line on the scale was even with the height of water in the low pressure manometer tube linked to the throat tap. Flow rate and head differential could then be read as the values from the respective scales that lined up with the water level manometer linked to the higher pressure upstream section tap. A manometer tube attached to the downstream tap was installed on the manometer board adjacent to the upstream manometer tube. Comparison of water levels in the upstream and downstream tubes provided visual evidence of head loss experienced by flow passing through the Venturi meter. Figure 4 shows the manometer board at the PID demonstration site.

Figure 4. PID demonstration pipe-fitting Venturi and manometer

Flow conditions being measured in Figure 4 show an approximate 800 gpm flow rate with meter head loss of ~ 0.35 ft. The throat tap plumbing may be seen near the bottom of Figure 4. A tee fitting at the tap is oriented such that a valve is installed in the branch of the tee oriented normal to the Venturi throat while the manometer line leaves the tee in a direction parallel to the throat. With this configuration, the valve on the end of the tee may be opened to allow insertion of a thin rod or wire to clean debris that may clog the pressure tap from time to time with this canal-fed system.

HILS engineers and PID staff agreed that the pipe fitting Venturi meter demonstration showed that the technology is cost competitive, can provide suitable flow measurement accuracy, and imposes comparatively modest head requirements. The land-owner whom this turnout served however could not be convinced that the 12 in. to 8 in. pipe size reduction was not severely limiting his ability to receive water from the canal. A short time after the Venturi demonstration site was set up the land owner removed it and installed a suppressed rectangular weir which required a water surface level drop in the range of one foot for non-submerged operation.

At the request of PID staff, the PVC Venturi meter was later re-installed in 2004 at a different turnout. The new location was higher in the PID delivery system within the Colorado delivery area. Figure 5 shows re-installed Venturi.

Figure 5. PVC Venturi meter and manometer installed at the second PID site

At the second installation site the PID canal outflow pipe previously discharged into a concrete lined field canal. The Venturi meter had to be installed on top of the canal lining. The "dog leg" configuration of two 45^{o} pipe bends that served to raise discharge to a suitable discharge height for the initial demonstration site shown in Figure 4 was also utilized at this site to ensure pipe-full flow through the Venturi. There is sufficient head available at this site to maintain the normal delivery flow rate. As may be seen in Figure 5 the fall from the "dog leg" represents a significantly greater head loss than the measured ~ 0.35 ft head loss through the Venturi meter discussed above.

Flow measurement with the PVC Venturi meter at the second PID site was also fated to a limited time of operation. Ramifications from a US Supreme Court case on water usage in the Republican Basin involving Colorado, Nebraska and Kansas has led Colorado Republican Basin well users to seek augmentation water to offset stream-flow injury resulting from well operations. Beginning in 2008 the Colorado irrigators on the PID system have entered a long-term lease for use of their share of PID water to an upstream well-users group and for the present have discontinued their PID irrigation operations. It is unclear whether this turnout will again be in service.

Mohave Valley Irrigation District

The Mohave Valley Irrigation and Drainage District (MVIDD) lies in Arizona along the east bank of the Colorado River a short distance downstream from the southern tip of

Nevada. All water utilized by the district is pumped from the shallow groundwater aquifer fed by the river, and is administered as diversion from the Colorado River. MVIDD had encountered problems with a range of previously tried flow meter technologies due in a large part to high concentrations of iron oxide present in the pumped flows.

In 2008, the Water Conservation Field Services Program of Reclamation's Yuma Area Office (YAO) requested technical assistance from HILS to identify cost-effective flow measurement methods that could function reliably over time given the water quality issues present along with other site-specific constraints at MVIDD. An automated data collection system was also a desired capability for the flow measurement system capability. YAO and MVIDD agreed to set up a demonstration site configured with a pipe fitting Venturi as a preliminary step in design of a flow measurement system. Figure 6 shows the MVIDD demonstration site.

Figure 6. MVIDD pipe fitting venturi meter demonstration site

Interest in the pipe-fitting Venturi meter concept was based on comparatively low installation costs, lack of moving parts, and relatively low head requirements. Keeping pressure tap orifices un-obstructed would be a concern given water quality conditions at MVIDD. As a means of keeping tap orifices cleared, a prototype sensing system utilizing a bubbler sensor linked to a solenoid valve bank was configured. Operation of the bubbler and solenoid valves was controlled by a programmable RTU that has on-site datalogging capability along with a radio communications link to a base unit at the MVIDD office. The RTU, bubbler and solenoid valve equipment may be seen in the electrical enclosure in the foreground of Figure 6.

The MVIDD demonstration site differed from the Replogle-Wahlin design sketch shown in Figure 2 in two key respects. First, the existing 12 in. pump discharge pipe was reduced by only one pipe size to 10 in. (as opposed to the double reduction to 8 in. pipe as shown in Figure 2 and as employed with the PID Venturi meter). The demonstration site well motor is always operated at the same speed and produces a comparatively high discharge for the pipe size (~ 9 ft^3/s). For the near-constant discharge at this site, a suitable pressure differential may be observed for appropriate measurement resolution with the single drop in pipe size.

The second design deviation was the absence of an expansion section back to the original 12 in. pipe size downstream from the Venturi meter. A downstream expansion can be a means of converting much of the increased dynamic (velocity) head seen in the reduced diameter pipe of the Venturi throat back to static (pressure) head. The lack of an expansion and associated increased discharge velocity results in an increased energy loss but does not impact flow measurement. For limited-term operation as a field test, an expansion section was not initially installed at this site.

Measured discharge rate at the MVIDD demonstration site was compared against a flow measurement obtained using a stream gage technique with a Price type AA current meter in the downstream canal. Agreement between the Venturi measured flow and the stream gage measured flow were found to be within the accuracy limits of the stream gage measurement.

The MVIDD demonstration site has now been in operation for approximately 30 months. Over this period of operation, the bubbler-sensed Venturi has experienced no maintenance issues or interruptions in service. This technology appears to be suitable for maintaining flow measurement capability with the water quality problems present which have proven problematic for multiple previously tested meter technologies at MVIDD. Following the initial 6 months of "field test" operations a pipe expansion section was added to the venturi meter at this site.

Palo Verde Irrigation District Deficit Irrigation Study

During 2008, a University of California Cooperative Extension Service field study examining impacts of deficit irrigation on alfalfa under the direction of Dr. Khaled Bali was being conducted at Palo Verde Irrigation District. As part of this study, a cost effective means of measuring field runoff of irrigation water was needed. Dr. Bali contacted Mark Niblack, the Water Conservation Field Services Program Coordinator at Reclamation's Yuma Area Office for assistance in measuring the field runoff flows.

Runoff from the test field is conveyed under a field road though a pipe culvert that discharges into a drain canal on the opposite side of the road. This culvert pipe entrance is several inches below the grade of the alfalfa field. An elevation survey of the culvert pipe revealed a slight upwards slope of the pipe that would ensure pipe full flow at the inlet any time water is being discharged into the drain. With pipe full flow assured, a

Venturi constructed of pipe fittings installed on the drain culvert inlet was suggested for measuring the irrigation runoff at this site.

Figure 7 and Figure 8 show the pipe-fitting Venturi and solar powered datalogging system being installed to measure and record field runoff. A small solar charging system was set up to power a differential pressure transducer linked to a datalogger at the site.

Figure 7. Runoff measurement venturi at Palo Verde

Figure 8. Installation of logging system at Palo Verde runoff venturi

SUMMARY

Venturi meters constructed of pipe fittings can be a practical means of measuring flow with reliable accuracy for a range of applications. As presented in the field demonstration sites cited above, the technology can be configured as low-tech stand alone measurement sites based on reading water column elevations, or may be readily incorporated into an automated data collection or SCADA system. While the pipe fitting venturi meters are by definition a closed conduit measurement instrument, it may be feasible to install one in an intermediate closed conduit link in what is essentially an open channel conveyance system. For all applications, pipe-full flow must be ensured as flow passes through the Venturi section.

Increased head loss in the downstream expansion section may be measurably greater with a pipe fitting Venturi than for an engineered Venturi. However in comparison with numerous other commonly employed measurement structures in agricultural water systems, the pipe fitting Venturi head losses are comparatively small. For water districts with the in-house fabrication and installation capabilities, pipe fitting Venturi meters can represent a cost competitive measurement alternative in comparison alternative flow measurement technologies including commercially available Venturi products.

None of the demonstration site case studies presented cover a time span that would approach a desired life expectancy for operation of a flow measurement system. Thus assessment of long-term reliability and cost effectiveness would require some degree of extrapolation. Given the extremely basic functionality of Venturi meters, along with fact that the Venturi solution for measuring flow is an analytic (as opposed to a calibrated) relationship, expectations for an appealing life expectance should be quite high. Pipe fitting Venturi meters are a technology that should be factored into the thinking of any water delivery entity seeking an economical means of expanding flow measurement capabilities.

Additional items of interest regarding use of venturi meters that are not brought out in the case studies cited are worth noting: Venturi meters do not require horizontal installation. The static head components that may be measured as a water column at the respective upstream and throat cross sections represent a combination of pressure head and elevation head. For an installation where Venturi taps are not in the same horizontal plane an increase (or decrease) in pressure heads (compared with a horizontal installation) is exactly offset by the differing elevation heads. Thus the measured water columns for either case will be the same.

When manometers are used to measure Venturi static heads, the manometers do not necessarily need to be vented to atmospheric pressure. A manometer system may be constructed with the tops of both the upstream and Venturi throat manometer tubes plumbed together in a manner that allows an increase in air pressure or a vacuum pressure to be present above the water column surfaces. Adjusting air pressure above the water surfaces with this manometer configuration allows the water columns to be read at a more convenient level than would be the case for manometers vented to the atmosphere.

Actual head loss through a Venturi meter is commonly quite small compared with opportunities for head recovery that might exist at a pipe exit. As an example, for the site shown in Figure 6, addition of pipe fittings to create an underwater discharge would enable a recovery of static head currently being lost that could easily represent an amount several times the head requirement presented by the Venturi meter.

REFERENCES

American Society of Mechanical Engineers. (1971). *Fluid Meters: Their Theory and Application* (6th ed.). (H. S. Bean, Ed.) New York, NY: American Society of Mechancial Engineers.

Brater, E. F., King, H. W., Lindell, J. E., & Wei, C. Y. (1996). *Handbook of Hydraulics* (7th ed.). Boston, MA: McGraw Hill.

Replogle, J. A., & Wahlin, B. T. (1994). Venturi Meter Constructions for Plastic Irrigation Pipelines. *Applied Engineering in Agriculture , 10* (1), 21-26.

IMPROVING CROP WATER USE DETERMINATION USING ADJUSTED EDDY COVARIANCE HEAT FLUXES

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ABSTRACT

Eddy covariance (EC) systems are being used to measure sensible heat (H) and latent heat (LE) fluxes in order to determine crop water use or crop evapotranspiration (ET_c). However, EC derived H and LE fluxes must be adjusted because EC systems systematically tend to underestimate actual H and LE heat fluxes. Thus, the energy balance does not close well generally for EC heat fluxes. A standard procedure for adjusting EC fluxes takes environmental conditions into consideration. This procedure allows for an improved determination of ET_c values on an hourly and daily basis. The objective of this study was to evaluate different adjustment methods and determine which one or combination of adjustment methods improved performance. For this purpose, two EC systems were installed near two large monolithic weighing lysimeters on irrigated cotton fields at the USDA-ARS Conservation and Production Research Laboratory at Bushland, Texas in the Texas High Plains during the months of July and August of 2008. A total of eight days (four in July and four in August) of EC data from two adjacent fields were post-processed and results were compared with the lysimetric ET_c. The evaluation included an analysis of the EC energy balance (EB) closure and residuals. Results indicated that the adjusted LE heat fluxes (converted to an equivalent amount of evapotranspirated water depth) underestimated the measured ET_c with an average error of 19-21% compared with 25-27% for the LE fluxes that were not adjusted. The residual errors occurred mostly during nocturnal measurements. The EB closure for the adjusted daytime fluxes was 88-97% which was an improvement over the 83-89% closure before adjustments. The EC frequency response adjustment had the largest impact in improving the EC-based ET values. This adjustment increased the LE fluxes by an average of 5.0-5.5%, and thus it was the most significant adjustment. Therefore we recommend that the combined adjustment method described be consistently applied when using EC to properly measure ET_c using EC systems for irrigated cotton in the Texas High Plains.

INTRODUCTION

The methods or approaches by which crop or vegetation water use or evapotranspiration (ET_c) can be measured are hydrological, meteorological, or based on plant physiology

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(Rose and Sharma, 1984). The hydrological approach is an indirect method in which ET_c is calculated as the residual of the soil water balance. For instance, lysimeters were used to measure water mass loss (therefore crop/soil ET) with high accuracy, according to Howell et al., (1995). However, lysimeters give only a single point measurement of ET_c due to its spatial limitations and cannot be easily moved to another location due to its large size and installation nature. The micrometeorological approach is a direct method in which ET_c is a function of climatic variables. The eddy covariance (EC) method is a micrometeorological approach based on the direct measurements of the product of vertical wind velocity fluctuations (w') and a scalar concentration fluctuation (c'), like water vapor or air temperature, producing latent (LE) [Eq. (1)] and sensible heat (H) [Eq. (2)] fluxes, respectively, assuming the mean vertical velocity is negligible:

$$LE = \rho_a \lambda \overline{w'q'} \tag{1}$$

$$H = C_p \rho_a \overline{w'T'} \tag{2}$$

where LE and H are the latent and sensible heat fluxes (W m^{-2}), $\overline{w'q'}$ is the covariance between fluctuations of vertical wind velocity, w' (m s^{-1}), and air specific humidity, q' (kg kg^{-1}), $\overline{w'T'}$ the covariance between fluctuations of w' and air temperature, T' (°C), ρ_a is the air density (kg m^{-3}), C_p is the specific heat of dry air at constant pressure (J kg^{-1} K^{-1}), and λ is the latent heat of water vaporization which varies with air temperature. Under good turbulent atmospheric conditions and a homogenous surface, the eddy covariance method can yield a representative spatially distributed estimate of ET_c. The EC system has the advantage that it can be easily relocated to another place. However, it has been documented that eddy covariance systematically tends to underestimate surface scalar fluxes and thus fails to close the energy balance (Mahrt, 1998; Twine et al., 2000). Energy balance is a fundamental principle based on the law of energy conservation. The major components of the energy balance are net radiation R_n (W m^{-2}), soil heat flux G (W m^{-2}), H, and LE and can be expressed as:

$$R_n - G = H + LE \tag{3}$$

where the right side in Eq. (3) is defined as available energy (R_n-G) and the left side as the turbulent fluxes (H+LE) with the signal convention of positive away from the surface with the exception of R_n.

The objectives of this study are: (1) to assess the agreement between EC measured ET_c values and measured lysimetric ET values; (2) to evaluate energy balance closure of EC measurements; and (3) to investigate and determine appropriate adjustment methods for EC measurements.

MATERIALS AND METHODS

Raw EC data were filtered for quality assurance and adjusted in an effort to improve H and LE fluxes and improve the energy balance closure. The following sub-sections describe the data collection setup, heat flux adjustment and quality control procedures, verification of resulting ET_c values against the large weighing lysimeter values, and the statistical procedures used.

Site description and field measurements

The research data collection was conducted during the 2008 cotton cropping season at the USDA-ARS, Conservation and Production Research Laboratory (CPRL) located at Bushland, TX. The geographic coordinates of the CPRL are 35°11'N, 102°06'W, and its elevation is 1,170m above mean sea level. Soils at and around Bushland are classified as slowly permeable Pullman clay loam. The major crops in the region are corn, sorghum, winter wheat, and cotton. Wind direction is predominantly from the south/southwest direction. Annual average precipitation is approximately 562 mm. However, only 280 mm of precipitation occurs during the nominal cotton growing season while about 670 mm of water are needed to grow cotton (New, 2005), thus irrigation needs to provide about 390 mm of timely water for a successful cotton harvest. The site was irrigated with a lateral move irrigation system. In addition, the long-term annual microclimatological conditions indicate that the study area is subject to very dry air and strong winds. Annual averages for air temperature, air water vapor pressure deficit, and horizontal wind speed are 14°C, 0.3 kPa, and 4.9 m s^{-1}, respectively.

Large monolith weighing lysimeters

Two precision weighing lysimeters (Marek et al., 1988), 3 x 3 x 2.3 m, were used to directly measure cotton ET_c. Each lysimeter contained a monolithic Pullman clay loam soil core. The lysimeters were located at the center of the north and south (East) experimental fields [210 m wide (East-West) x 225 m long (North-South) each]. The change in lysimeter water mass was measured by load cell (SM-50, Interface[4], Scottsdale, AZ) and recorded by a datalogger (CR7-X, Campbell Scientific Inc., Logan, UT). The signal was sampled at 0.17 Hz frequency. The high frequency load cell signal was averaged for 5 min and composited to 15-min means. The lysimeters were calibrated using techniques as explained in Howell et al. (1995). The lysimeters mass measurement accuracy in water depth equivalent was 0.01 mm, as indicated by the root mean square error (RMSE) of calibration. Each lysimeter field was equipped with one net radiometer (REBS Q*7.1, REBS, Radiation and Energy Balance Systems, Bellevue, WA) at about 1.5 m above the ground in the center of the N lysimeters side facing to the S.

[4] The mention of trade names of commercial products in this article is solely for the purpose of providing specific information and does not imply recommendation or endorsement by the U.S. Department of Agriculture or Colorado State University.

Eddy covariance energy balance system

Two identical EC systems were deployed approximately 15 m North-East of the lysimeters and each system consisted of a 3-D sonic anemometer (CSAT3, Campbell Scientific Inc., Logan, UT), an open path infrared gas analyzer (LI-7500, LI-COR Inc., Lincoln, NE), a fine wire thermocouple (FW05, Campbell Scientific Inc., Logan, UT), an air temperature/humidity sensor (HMP45C, Vaisala Inc., Woburn, MA), and a datalogger (CR3000, Campbell Scientific Inc., Logan, UT). Component wind vectors (u, v, and w) and scalar measurements of temperature (T), water vapor (H_2O) concentration, carbon dioxide (CO_2) concentration, and atmospheric pressure were measured at a frequency of 20 Hz. This equates to a rate of 20 samples (or readings/records) per second. The data derived from the high frequency data sampling were called “time series data.” Both systems were installed and kept at a height of 2.5 m above the ground level. The CSAT3 sensor was oriented toward the predominant wind direction, with an azimuth angle of 225° from true North. Installed about 4 m East from each EC system were two soil heat flux plates (HFT-3, REBS, Radiation and Energy Balance Systems, Bellevue, WA), two pairs of soil thermocouples (TCAV, Campbell Scientific Inc., Logan, UT), and two soil water reflectometers (CS616, Campbell Scientific Inc., Logan, UT) for measuring soil heat flux, soil temperature, and volumetric water content; and to calculate soil heat storage to the depth of soil heat flux plates installation. Soil heat flux plates (SHFP) were installed at 0.08 m depth within and between crop rows. Soil thermocouple pairs were installed at 0.02 and 0.07 m depths close to the SHFP locations. Soil water reflectometers were installed at an approximate angle of 13 degrees across the 0.01-0.1 m depth to measure the volumetric soil water content within this depth zone.

Eddy covariance data processing and filtering

Eddy covariance data were post-processed for the following selected eight days of the year (DOY), during the months of July and August: 203, 205, 206, 208, 234, 238, 239, and 240. Eddy covariance does not perform well in intensive rain or irrigated conditions and therefore the days slelected for comparison were days when no precipitation or irrigation events occurred.

EC data were post-processed with the EdiRe® software package (Clement, 1999) following the guidelines described in Lee et al. (2004) and Burba and Anderson (2007). Before the covariances were calculated, spikes of six standard deviations (SD) from the population mean were removed from the time series and replaced with running means. If four or more consecutive points were detected to be about the six SDs, they were not considered as a spike. Time delay between the CSAT3 and LI-7500 was removed using a cross-correlation analysis. Although the terrain for the site was virtually flat, the CSAT3 cannot be perfectly leveled, such that the vertical component (w) is perpendicular to the mean streamline plane. For this reason, the coordinates were rotated using the double rotation (2D) method of McMillen (1988) and Kaimal and Finnigan (1994). According to Lee et al. (2004) this method is suitable for “ideal sites” with little slope and fair weather conditions. The effects of density fluctuations induced by heat fluxes on the measurement of eddy fluxes of water vapor using the LI-7500 were corrected according to Webb et al. (1980) procedures; henceforth, referred as WPL corrections. Spectral loss in the high frequency band due to path-length averaging, sensor separation, and signal processing was corrected after Moore (1986). Schotanus et al. (1983) recommended correcting air temperature calculated by the sonic anemometer (T_s) for crosswind and humidity effects, commonly referred to as the heat flux correction (HFC). The CSAT3 implements the crosswind correction online and therefore the heat flux only needed to be corrected for humidity fluctuations. The sonic temperature flux $\overline{w'T_s'}$ was converted to actual temperature flux $\overline{w'T'}$, Eq. (4), following Schotanus et al. (1983).

$$\overline{w'T_s'} = \overline{w'T'} + 0.51\ T\overline{w'q'} \tag{4}$$

where, T_s is the sonic temperature (°C), T is the actual air temperature (°C), w is the vertical wind velocity (m s^{-1}), q is the specific humidity, and the overbars and primes denote mean and fluctuating parts, respectively.

A dimensionless parameter that characterizes the processes in the surface layer is the atmospheric stability parameter (ζ), Eq. (5), which is the ratio of the convective production to the mechanical production of turbulent kinetic energy (Campbell and Norman, 1998):

$$\zeta = -\frac{0.4 z_m g H}{(T+273.15)\rho_a C_p u_*^3} \tag{5}$$

where, z_m is the wind observation height above the zero-plane displacement (d, m), g is the gravitational acceleration (m s^{-2}), H is the sensible heat flux (W m^{-2}), T is the air temperature (°C), ρ_a is the air density (kg m^{-3}), C_p is the specific heat of dry air at constant pressure (J kg^{-1} K^{-1}), and u_* is the friction velocity (m s^{-1}). Positive ζ represents a stable stratification, negative ζ represents an unstable stratification, and ζ≈0 represents a neutral stratification.

The sensible and latent heat fluxes were calculated using an averaging period of 15-min. Each adjustment step/procedure, mentioned above, was performed separately and then all steps were combined with the frequency response, WPL, and HFC adjustments being applied iteratively due to their interdependence. The sequence of adjustments is shown in table 1.

Table 1. Post-processing procedure using the software package EdiRe®.

Procedure		EdiRe Commands
1.	Extract raw time series data	Extract
2.	Calculate wind direction	Wind direction
3.	Remove spikes	Despike
4.	Calculate and remove lag between instruments	Cross correlate, Remove lag
5.	Rotate coordinates	Rotation coefficients, Rotation
6.	Calculate means, standard deviations, skewness, and kurtosis	1 chn statistics
7.	Calculate covariances and fluxes	Latent heat of evaporation, Sensible heat flux coefficient, 2 chn statistics
8.	Calculate friction velocity and stability	User defined, Stability - Monin Obhukov
9.	Calculate and apply frequency response corrections	Frequency response
10.	Calculate and apply Schotanus H correction	Sonic T - heat flux correction
11.	Calculate and apply WPL correction	Webb correction
12.	Iterate steps 8-11 two times	
13.	Convert LE to ET (mm h^{-1})	User defined
14.	Calculate roughness length	Roughness length (zo)
15.	Calculate stability	Stationarity

The canopy heights for both the Northeast (NE) and Southeast (SE) fields were measured five times during the study on the following DOYs: 171, 182, 200, 210, and 220. The canopy height shortly after emergence (DOY153) was assumed to be 0.01 m. The canopy height (h_c, m) and DOY was plotted and a curve was fitted using SigmaPlot 11(Systat Software Inc., San Jose, CA) to determine the canopy height as a function of the DOY. The zero-plane displacement height was assumed to be 65% of the canopy height (Campbell and Norman 1998). The following functions correspond to the canopy height (m) for the NE and SE fields, respectively:

$$h_{c\_NE} = \frac{0.76}{\left(1+e^{-(DOY-195.78)/11.95}\right)} \tag{6}$$

$$h_{c\_SE} = \frac{0.84}{\left(1+e^{-(DOY-196.06)/12.25}\right)} \tag{7}$$

Wind direction, heat flux source area, stationarity, and integral turbulence tests were applied to filter out any data that did not meet minimum quality control standards. Only fluxes for wind directions between 142° and 322° from North were included in this study because the predominant wind direction and larger fetch were obtained from the south-southwest sector of the field. These parameters were set for two reasons: to achieve optimum inter-comparison between the EC systems and lysimeter ET_c values and to exclude flow that may be distorted by the instrumentation.

The approximate analytical heat flux source area (i.e. footprint) model proposed by Hsieh (2000) was used to calculate the 70% effective fetch. The distance upwind from the measurement location representing a fraction, f, of the contributing source area, X_f (m), was calculated as:

$$X_f = \frac{-D|L|^{(1-P)} z_u^P}{k^2 \ln(f)} \tag{8}$$

with

$$z_u = z_m \left(\ln\left(\frac{z_m}{z_o}\right) - 1 + \frac{z_o}{z_m} \right) \tag{9}$$

where z_o is the roughness length (m), z_m is the wind measurement height (m), k is von Karman's constant (0.41), and D and P are the coefficients found by regression of Calder's analytical solution (1952) to the results of Thompson's Lagrangian model (1987) for unstable (D=0.28, P=0.59), near-neutral (D=0.97, P=1), and stable stratification (D=2.44, P=1.33). To ensure reasonable horizontal homogeneity (i.e., that heat fluxes belonged within the cotton field), fluxes with effective fetch greater than the boundaries of the field were excluded. Measurements of heat fluxes via the eddy covariance method are based on simplified forms of the Navier-Stokes equations for certain atmospheric conditions (Stull, 1988). These conditions are not always met and therefore must be tested. Tests for stationarity and integral turbulence were performed following methods outlined in Foken and Wichura (1996) and Thomas and Foken (2002). For the stationarity test, covariances between the vertical wind speed (w) and the horizontal wind speed (u), the air temperature (T), and the water vapor (q) scalars for the averaging period (i.e., 15-min) were compared to covariances of consecutive 5-min intervals within the same period. The periods where deviations, Δ_{st}, were greater than 30% were considered unstationary and excluded from the study:

$$\Delta_{st} = \frac{100 \left| \overline{w'x'_s} - \overline{w'x'_o} \right|}{\overline{w'x'_o}} \tag{10}$$

where, x is u for momentum flux and the scalar of interest (T or q) for scalar fluxes and "5" and "o" are subscripts for the 5-min and averaging period covariances, respectively. Integral turbulence tests are used to determine if the turbulence is well developed. With weak turbulence the measuring methods based on surface layer similarities may not be valid (Foken et al., 2004). The integral turbulence test was done by comparing similarity functions for vertical wind speed (ϕ_w) and temperature (ϕ_T) with modeled functions developed by Thomas and Foken (2002) as follows:

$$\phi_w = \frac{\sigma_w}{u_*} \tag{11}$$

$$\phi_T = \frac{\sigma_T}{T_*} \tag{12}$$

where, σ_w and σ_T are the standard deviations of w and T, respectively, and T_* is the dynamic temperature (°C). Any periods with deviations between measured and modeled similarity functions, Δ_{ITT}, greater than 30% were excluded using the following:

$$\Delta_{ITT} = \frac{100\left|\phi_{model} - \phi_{measured}\right|}{\phi_{model}} \tag{13}$$

Energy balance residual and statistical analysis

The effectiveness of each method of adjustment was determined by evaluating the energy balance residual and comparisons between the lysimeters ET_c-measured values and EC-measured/adjusted ET_c values. The energy balance residual (ε) is defined as:

$$\varepsilon = R_n - G - LE - H \tag{14}$$

Energy balance closure is achieved when the residual is equal to zero. Also, the energy balance ratio (EBR) is defined as:

$$EBR = \frac{\sum H + LE}{\sum R_n - G} \tag{15}$$

The statistical analysis was performed following Willmott (1982). The mean bias error (MBE, Eq. 16), RMSE (Eq. 17), index of agreement (d, Eq. 18), and linear regression analysis based on the least squares method for comparison of fitted equation slope and intercept were used for the comparison of ET_c values and evaluation of the entire adjustment procedure.

$$MBE = \frac{1}{N}\sum_{i=1}^{N}[X(M)_i - X(O)_i] \qquad (16)$$

where, N is the number of pairs compared, $X(M)_i$ is the measured EC-based ET_c value, and $X(O)_i$ the observed lysimeter-based ET_c expressed in mm h^{-1} and percent (% of the observed) .

The RSME and d were computed as follows:

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}[X(M)_i - X(O)_i]^2} \qquad (17)$$

$$d = 1 - \left\{\frac{\sum_{i=1}^{N}[X(M)_i - X(O)_i]^2}{\sum_{i=1}^{N}[|X(M)_i - \overline{X(O)}| + |X(O)_i - \overline{X(O)}|]^2}\right\} \qquad (18)$$

RESULTS AND DISCUSSION

The individual and combined adjustment schemes were first evaluated by comparing EC and lysimeters measured ET_c values. The level of energy balance closure was then evaluated for each adjustment scheme. The MBE of EC-based hourly ET_c before any adjustments were applied was -0.13 mm h^{-1} (-26.9%) for site EC1 and -0.11 mm h^{-1} (-24.9%) for site EC2. A negative MBE means that the EC system underestimated ET_c.

Adjustment scheme evaluation

The WPL adjustment increased the MBE of EC-based hourly ET_c to -0.13 mm h^{-1} (-29.5%) for site EC1 and -0.11 mm h^{-1} (-28.8%) for site EC2. Nighttime ET was 7-10% of the whole day ET_c (Tolk et al. 2006a, b) and therefore even small variations will appear large in terms of percent. As shown in Figure 1, the hourly ET_c values obtained after applying the WPL adjustment resulted in a nighttime adjustment that was negative and in a daytime adjustment that was positive. Liebethal and Foken (2003) showed that the diurnal pattern of WPL adjustments correlated well with the diurnal pattern of vertical wind velocity. Although, the WPL adjustment increased the MBE while the RMSE decreased by a small amount. Considering only daytime EC-based ET_c values (~14 h), the MBE after WPL adjustments was -0.19 mm h^{-1} (-29.8%) with RMSE of 0.25 mm h^{-1} (27.4%) for EC1, and -0.16 mm h^{-1} (-27.2%) with RMSE of 0.19 mm h^{-1} (23.1%) for EC2. As shown in Liebethal and Foken (2003), the WPL adjustment had a much larger impact on CO_2 flux. However, in our study the impact of WPL adjustments on ET_c was significant.

The 2D rotation adjustment improved hourly ET_c for both EC sites having a more significant impact on site EC1. For site EC1 the 2D adjustment reduced the MBE to -0.11 mm h^{-1} (-21.2%) and RMSE to 0.18 mm h^{-1} (20.7%). As shown in Figure 1, the 2D adjustment was larger earlier in the day (morning hours). Although, the diurnal pattern was similar for both sites the magnitude was larger for EC1. This inconsistency could be due to a slightly different topography/micro-topography between the two sites or because one of the sensors was not leveled identical to the other.

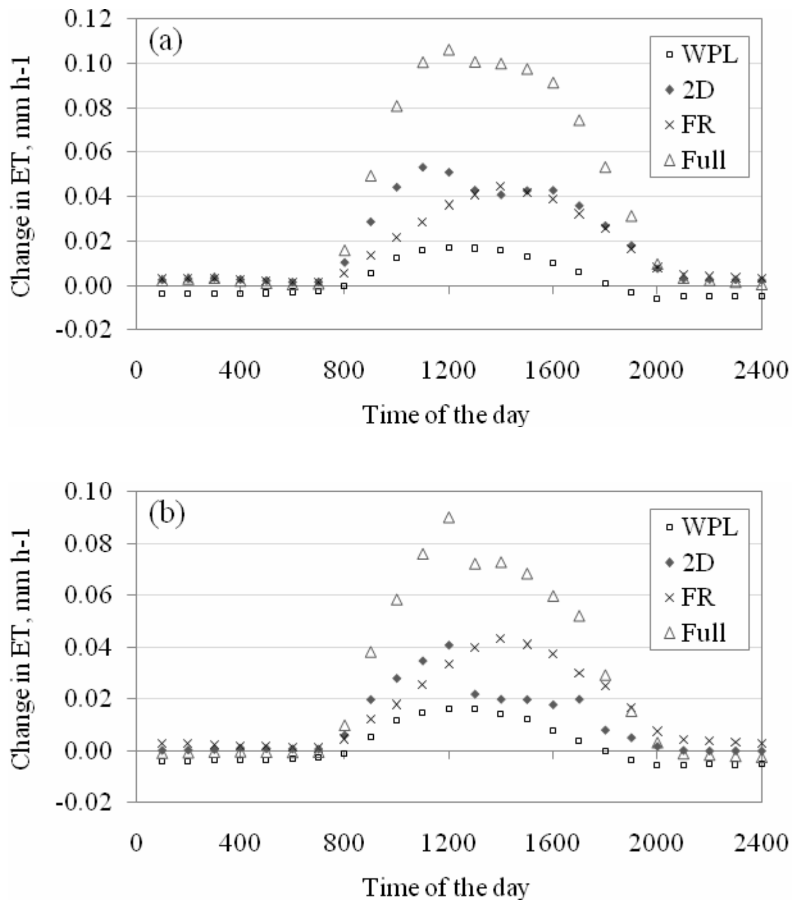

Figure 1. Hourly average change of EC-based ET_c due to Webb density (WPL), double coordinate rotation (2D), frequency response (FR), and iteration of all adjustments (Full) from (a) northeast (NE) and (b) southeast (SE) fields for entire data set

The frequency response (FR) adjustment had the most significant impact on EC-based hourly ET. The MBE and RMSE each decreased to -0.11 mm h^{-1} (-21.9%) and 0.18 mm h^{-1} (21.1%) for EC 1 and -0.09 mm h^{-1} (-19.5%) and 0.13 mm h^{-1} (17.3%) for EC2, respectively. The FR adjustment is mostly dependent on the sensor setup and method used to detrend the data, which was identical for both sites, and therefore similar results could be expected when using the same sensor setup and methods at different sites. As shown in Figure 2, there was a strong correlation between change in flux due to frequency response adjustment and the atmospheric stability. As the atmosphere became more stable the impact of the adjustment increased. It should be noted that for unstable conditions (i.e. stability less than zero) the change remained the same, just above 5%.

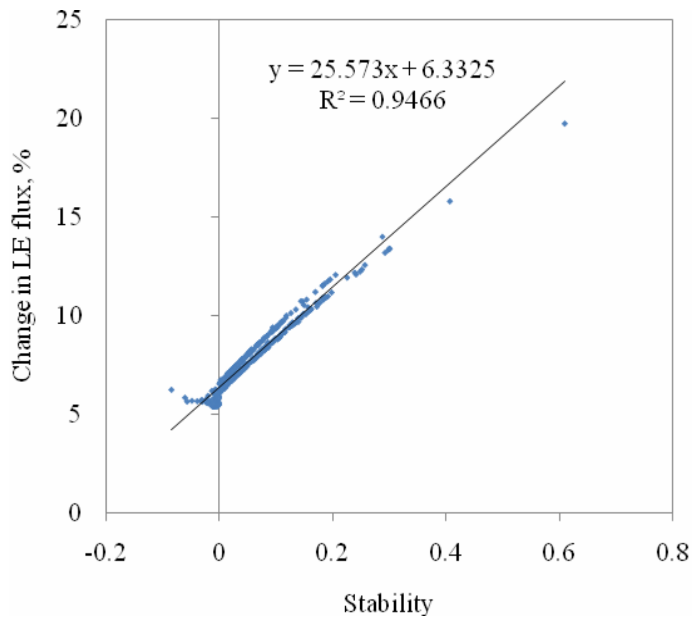

Figure 2. Change in latent heat flux (%) due to frequency response adjustment versus atmospheric stability

The full adjustment of ET included 2D, FR, HFC, and WPL with the last three corrections being iterated two times due to their interdependence. Though the HFC doesn't directly affect the ET_c, it does affect H which was used in the calculation of the WPL adjustment. The full adjustment of ET resulted in a MBE of -0.09 mm h^{-1} (-19.0%) for EC1 and -0.08 mm h^{-1} (-21.0%) for EC2. Table 2 details the ET_c errors in mm h^{-1}, percent, and the corresponding correlation parameters for both EC systems and for ET_c including/excluding different adjustment schemes. Similar analyses of ET errors were performed for 15-min average ET_c derived from daytime (~14 h) and are shown in Table 3. Also, the daily progress of energy fluxes and ET_c rates during DOY 239 are illustrated in Figures 3 and 4, respectively.

Table 2. Comparison of EC-based ET_c with lysimetric ET_c using the entire day (24 h) 15-min averaged data and different adjustment schemes

Site	Adjustment	N	MBE (mm h^{-1})	MBE (%)	RMSE (mm h^{-1})	RMSE (%)	Slope	Intercept (mm h^{-1})	d
EC1	None	505	-0.13	-26.9	0.20	23.5	0.65	0.018	0.90
EC2	None	484	-0.11	-24.9	0.15	19.8	0.70	0.001	0.91
EC1	WPL	505	-0.13	-29.5	0.20	22.9	0.67	0.014	0.92
EC2	WPL	484	-0.11	-28.8	0.15	19.3	0.72	-0.004	0.95
EC1	2D	505	-0.11	-21.2	0.18	20.7	0.70	0.021	0.93
EC2	2D	484	-0.10	-22.1	0.14	18.2	0.74	0.001	0.94
EC1	FR	505	-0.11	-21.9	0.18	21.1	0.69	0.020	0.88
EC2	FR	484	-0.09	-19.5	0.13	17.3	0.75	0.002	0.89
EC1	Full	505	-0.09	-19.0	0.16	18.2	0.76	0.018	0.95
EC2	Full	484	-0.08	-21.0	0.12	15.8	0.80	-0.004	0.96

where, ET_c is crop evapotranspiration, N is the sample size, MBE is the mean bias error, RMSE is the root mean squared error, d is the index of agreement, EC1 is eddy covariance system 1, EC2 is eddy covariance system 2, WPL is Webb density correction, 2D is double coordinate rotation, FR is frequency response, and Full is the combination and iteration of all adjustments.

Table 3. Comparison of EC-based ET_c with lysimetric ET_c using daytime (~14 h) 15-min averaged data and different adjustment schemes

Site	Adjustment	N	MBE (mm h^{-1})	MBE (%)	RMSE (mm h^{-1})	RMSE (%)	Slope	Intercept (mm h^{-1})	d
EC1	None	320	-0.20	-30.8	0.25	28.2	0.61	0.050	0.82
EC2	None	290	-0.17	-28.2	0.20	24.0	0.71	-0.001	0.84
EC1	WPL	320	-0.19	-29.8	0.25	27.4	0.62	0.050	0.85
EC2	WPL	290	-0.16	-27.2	0.19	23.1	0.72	-0.003	0.89
EC1	2D	320	-0.16	-24.8	0.22	24.7	0.65	0.060	0.85
EC2	2D	290	-0.15	-24.4	0.18	22.0	0.73	0.002	0.88
EC1	FR	320	-0.17	-26.3	0.23	25.3	0.65	0.052	0.77
EC2	FR	290	-0.14	-23.1	0.17	20.9	0.75	0.000	0.79
EC1	Full	320	-0.13	-19.3	0.19	21.4	0.70	0.062	0.89
EC2	Full	290	-0.12	-18.9	0.15	18.5	0.80	0.000	0.92

where, ET_c is crop evapotranspiration, N is the sample size, MBE is the mean bias error, RMSE is the root mean squared error, d is the index of agreement, EC1 is eddy covariance system 1, EC2 is eddy covariance system 2, WPL is Webb density correction, 2D is double coordinate rotation, FR is frequency response, and Full is the combination and iteration of all adjustments.

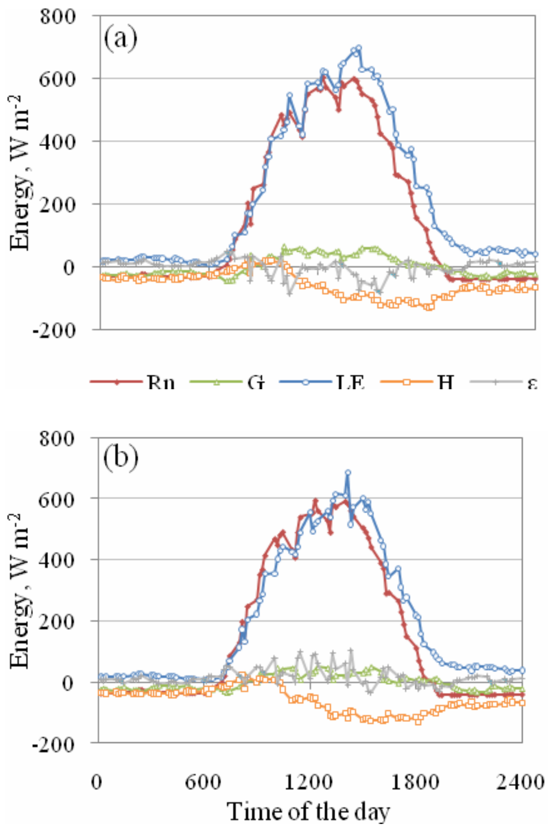

Figure 3. Diel net radiation (Rn), soil heat flux (G), latent heat flux (LE), sensible heat flux (H), and residual (ε) (DOY 239) from (a) northeast (EC1) and (b) southeast (EC2) fields

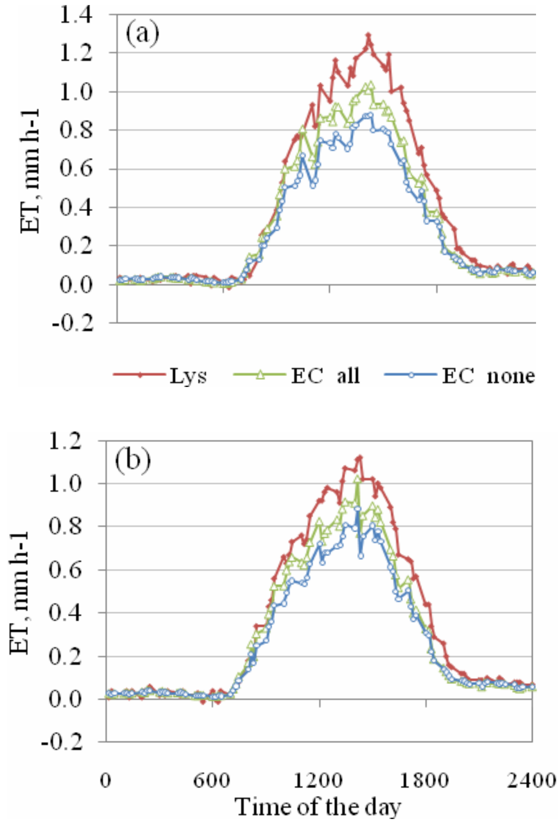

Figure 4. Diel lysimeter ET_c (Lys), eddy covariance with no adjustment (EC_none), and eddy covariance with iteration of all adjustments (EC_all) (DOY 239) from (a) northeast (EC1) and (b) southeast (EC2) fields

Energy balance evaluation

Energy balance closure for non-adjusted fluxes was 89% and 83% for EC1 and EC2, respectively. Each adjustment increased closure, with the 2D and FR adjustments having the most significant impact on improving the energy balance closure. The overall energy balance ratio for adjusted fluxes was 97% and 88% for EC1 and EC2, respectively. Table 4 shows the energy balance ratio, correlation parameters, and residual statistics in W m^{-2} for daytime (~14 h) fluxes. The daytime values of H were frequently negative as shown for DOY 239 in Figure 3. Irrigated land in the Texas High Plains is surrounded by very dry fallow lands and therefore the advection of dry heat can significantly increase ET_c (Tolk et al., 2006b). However, the energy balance closure did not reach 100% after adjustments were performed. Twine et al. (2000) discussed a method of closing the energy balance by preserving the Bowen ratio that could further improve the agreement between EC and lysimeter ET_c values.

Table 4. Comparison of available energy (Rn-G) and turbulent heat flux energy (H+LE) using daytime (~14 h) 15-min averaged data and different adjustment schemes

Site	Adjustment	N	EBR	d	Slope	Intercept ($W m^{-2}$)	Residual Mean ($W m^{-2}$)	Residual s_d ($W m^{-2}$)	Residual Max ($W m^{-2}$)	Residual Min ($W m^{-2}$)
EC1	None	320	0.89	0.97	0.85	13.02	38.1	53.5	253.0	-113.5
EC2	None	290	0.83	0.95	0.79	13.36	57.8	57.1	295.2	-154.0
EC1	WPL	320	0.90	0.97	0.88	9.43	32.4	51.1	239.8	-115.4
EC2	WPL	290	0.84	0.96	0.81	9.69	52.4	53.6	282.9	-152.8
EC1	2D	320	0.96	0.98	0.90	19.50	14.3	55.2	239.3	-133.8
EC2	2D	290	0.87	0.96	0.82	16.50	44.3	54.7	304.1	-161.3
EC1	FR	320	0.94	0.98	0.89	16.76	19.1	52.9	231.1	-131.4
EC2	FR	290	0.89	0.96	0.83	17.34	38.3	56.7	249.0	-170.3
EC1	Full	320	0.97	0.98	0.92	14.75	11.4	52.4	225.3	-144.3
EC2	Full	290	0.88	0.97	0.84	11.96	41.1	51.0	296.8	-158.0

where, ET_c is crop evapotranspiration, N is the sample size, MBE is the mean bias error, RMSE is the root mean squared error, d is the index of agreement, EC1 is eddy covariance system 1, EC2 is eddy covariance system 2, WPL is Webb density correction, 2D is double coordinate rotation, FR is frequency response, and Full is the combination and iteration of all adjustments.

CONCLUSION

Accurate crop evapotranspiration measurements are important for irrigation management and model verification. Eddy covariance measurements of LE can be improved considerably when adjusted for vapor density, coordinate rotation, frequency response, and heat flux conversion. Double coordinate rotation and frequency response adjustments had the largest impact on properly adjusting EC-based ET. The coordinate rotation adjustments improved the MBE between EC-based hourly ET and lysimeter ET values by 5.7% and 2.8% for site EC1 and EC2, respectively. The frequency response adjustment improved MBE by 5.0% and 5.5% for site EC1 and EC2, respectively. The iteration of the entire combined adjustments improved the EC-based hourly ET by 3-8% and reduced the energy balance residual by 20-50%.

After adjustment EC still underestimated ET by 20%. It seems that further adjustments are needed beyond the procedures presented in this study. Large eddy flux contribution can only be measured if the averaging period is sufficiently long. This study was limited to a 15-min averaging period and an optimum averaging period in which most of the flux can be measured without violating stationarity limits needs to be determined by testing longer averaging periods. Techniques to fill the gaps left by quality control procedures along with forcing energy balance closure could also be employed to further improve the EC-based ET_c results. It is the recommendation of the authors that the combined adjustment scheme described be consistently applied when using EC to properly measure ET_c using EC systems.

REFERENCES

Burba, G, and Anderson, D (2007). Introduction to the Eddy Covariance Method: General Guidelines and Conventional Workflow. LI-COR Biosciences, http://www.licor.com , 141 pp (accessed 5 February 2010).

Campbell, GS, and Norman, JM (1998). An Introduction to Environmental Biophysics (2nd ed.). Springer, New York, 286 pp.

Clement, R (1999). EdiRe Data Software (Version 1.5.0.10) [Computer software]. Edinburgh, England: University of Edinburgh. Retrieved 1 March 2010. Available from http://www.geos.ed.ac.uk/abs/research/micromet/EdiRe

Foken, T, and Wichura, B (1996). Tools for quality assessment of surface-based flux measurements. Agricultural and Forest Meteorology, 78:83-105.

Foken, T, Gockede, M, Mauder, M, Mahrt, L, Amiro, B, and Munger, W (2004). Post-field data quality control. In X Lee, W Massman, & B Law, Handbook of Micrometeorology: A Guide for Surface Flux Measurement and Analysis. Kluwer, Dordrecht, 181-208.

Howell, T, Schneider, A, Dusek, D, Marek, T, and Steiner, J (1995). Calibration and scale performance of Bushland weighing lysimeters. Trans ASAE, 38:1019-1024.

Hsieh, C, Katul, G, Chi, T (2000). An approximate analytical model for footprint estimation of scalar fluxes in thermally stratified atmospheric flows. Adv. Water Resour. 23:765-772.

Kaimal, J, Finnigan, J (1994). Atmospheric Boundary Layer Flows: Their Structure and Measurement. Oxford University Press, New York, 289 pp.

Lee, X, Massman, W, and Law, B (2004). Handbook of Micrometeorology: A Guide for Surface Flux Measurement and Analysis. Kluwer, Dordrecht, 250 pp.

Liebethal, C, and Foken, T (2003). On the significance of the Webb correction to fluxes. Bound.-Layer Meteor. 109:99-106.

Mahrt, L (1998). Flux sampling error for aircrafts and towers. J Atmos. Oceanic Technol. 15:416-429.

Marek, T, Schneider, A, Howell, T, and Ebeling, L (1988). Design and installation of large weighing monolithic lysimeters. Trans ASAE, 31:477-484.

McMillen, R (1988). An eddy correlation technique with extended applicability to non-simple terrain. Bound.-Layer Meteor. 43:231-245.

Moore, C (1986). Frequency resopnse corrections for eddy correlation systems. Bound.-Layer Meteor., 37:17-35.

New, LL (2005). Agri-Partner irrigation result demonstrations 2005. Texas Cooperative Extension Servic, Texas A&M University. Available online at: http://amarillo.tamu.edu/programs/agripartners/irrigation2005.htm

Rose, C and Sharma, M (1984). Summary and recommendations of the workshop on evapotranspiration of plant communities. Agric. Wat. Manag. 8:325-342.

Schotanus, P, Nieuwstadt, F, & De Bruin, H (1983). temperature measurement with a sonic anemometer and its applications to heat and moisture fluxes. Boundary-Layer Meteorology, 26:81-93.

Stull, RB (1988). An Introduction to Boundary Layer Meteorology. Kluwer, Dordrecht 666 pp.

Thomas, C, & Foken, T (2002). Re-evaluation of integral turbulence characteristics and their parameterizations. Proceedings of the 15th Symposium on Boundary Layers and Turbulence (pp. 129-132). Wageningen, The Netherlands: Am. Meteorol. Soc.

Tolk, J, Howell, T, Evett, S (2006a). Nighttime evapotranspiration from alfalfa and cotton in a semiarid climate. Agron. J. 98:730-736.

Tolk, J, Evett, S, Howell, T (2006b). Advection influences on evapotranspiration of alfalfa in a semiarid climate. Agron. J. 98:1646-1654.

Twine, TE, Kustas, WP, Norman, JM, Cook, DR, Houser, PR, Meyers TP, Prueger, JH, Starks, PJ, Wesley, ML (2000). Correcting eddy covariance flux underestimates over a grassland. Agric. For. Meteorol. 103(3):229-317.

Webb, E, Pearman, G, & Leuning, R (1980). Correction of flux measurements for density effects due to heat and water vapour transfer. Quart. J. Roy. Meteorol. Soc., 106:85-100.

Willmott, CJ (1982). Some comments on the evaluation of model performance. Am. Meteor. Soc., 63:1309-1313.

PERFORMANCE EVALUATION OF TDT SOIL WATER CONTENT AND WATERMARK SOIL WATER POTENTIAL SENSORS

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ABSTRACT

This study evaluated the performance of digitized Time Domain Transmissometry (TDT) soil water content sensors (Acclima, Inc., Meridian, ID) and resistance-based soil water potential sensors (Watermark 200, Irrometer Company, Inc., Riverside, CA) in two soils. The evaluation was performed by comparing volumetric water content (θ_v) data collected in the laboratory and in fields near Greeley, CO, with values measured by the sensors. Calibration equations of θ_v were then developed based on the laboratory and field data. Statistical targets to determine accuracy of the equations were ±0.015 $m^3\ m^{-3}$ mean bias error and a root mean square error of less than 0.020 $m^3\ m^{-3}$.

Under laboratory and field conditions, the factory-based calibrations of θ_v did not consistently achieve the required accuracy for either sensor. Field tests indicated that using the calibration equation developed in the laboratory to correct data obtained by TDT and Watermark sensors in the field at Site A (sandy clay loam) was not consistently accurate. Using the laboratory equations developed for the Watermark sensors at Site B (loamy sand) accurately measured θ_v.

Field tests found that a linear calibration of the TDT sensors (and a logarithmic calibration for the Watermark sensors) could accurately correct the factory calibration of θ_v in the range of permanent wilting point (PWP) to field capacity (FC). Furthermore, the van Genuchten (1980) equation was not significantly more accurate than the logarithmic equation, and the additional work of deriving the former equation did not seem worthwhile, within the range of soil water contents analyzed.

INTRODUCTION AND BACKGROUND

Due to competition for water from urban growth, drought and changing climate conditions, irrigated agriculture needs to improve its water management methods (Cooley et al., 2009). One technique uses soil water content sensors to closely monitor a wide range of field soil water content conditions. These measurements can potentially be used

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to accurately determine irrigation amounts and timing. Most soil water content sensors are a simple, cost-effective way to closely monitor soil water conditions in the crop root zone. Using these sensors, an irrigation manager can determine irrigation timing and amount. Irrigations can then be scheduled whenever the soil water content is depleted to a management allowed level (previously-set critical level).

Soil water content sensors are gaining increased federal support. The U.S. Department of Agriculture recently awarded the White River Irrigation District in Arkansas $4.45 million to install water measurement and monitoring technology, which includes soil water content sensors (NRCS, 2009). Furthermore, since 2006 the U.S. Air Force has been introducing Watermark soil potential sensors to farmers throughout rural Afghanistan (Kapinos, 2006). Yet Hignett and Evett (2008) warn that some soil water content sensors are being used in applications for which they are not suited, producing results that have little relation to actual field conditions. These and other examples indicate that soil water content sensors are achieving widespread use and swift measures should be taken to assess them in specific soil types.

This study evaluates the performance of digitized Time Domain Transmissometry (TDT) soil water content sensors (Acclima, Inc., Meridian, ID) and resistance-based (Watermark 200, Irrometer Company, Inc., Riverside, CA) soil water potential sensors. A handful of studies have been performed on these sensors, but few have been conducted for particular soils in the state of Colorado. Performance evaluations and specific calibrations have not been carried out on irrigated (surface and sprinkler) coarse-loamy to silty-clay soils in eastern Colorado. It is hypothesized that the accuracy of the sensors in these soils will be different than the results found by the sensor manufacturers. Hignett and Evett (2008) warn that a “manufacturer’s calibration is commonly performed in a temperature controlled room, with distilled water and in easy to manage homogeneous soil materials (loams or sands) which are uniformly packed around the sensor. This produces a very precise and accurate calibration for the conditions tested.” In the field, though, factors such as rocks, roots, and variations in clay content, temperature and salinity may mean that “the manufacturer’s calibration is rarely applicable.”

Therefore, a thorough evaluation and the development of a family of soil-specific sensor calibration curves are highly desirable. These can improve farmers’ abilities to track soil water status and therefore improve irrigation water monitoring and irrigation scheduling. The result will translate into water savings, improved crop yields, and protection of groundwater from potential agro-chemical contamination.

MATERIALS AND METHODS

This study took place during the 2010 growing season and included soils from two agricultural fields in eastern Colorado. Laboratory and field tests were performed on the TDT soil water content and Watermark soil water potential sensors between mid-July and early-October, 2010. The first field was an experimental field cooperatively operated by the United States Department of Agriculture – Agricultural Research Service (USDA-ARS), Regenesis Management Group, and Colorado State University (CSU). Corn was grown at this location and was irrigated using furrows. This field is located near the City of Greeley airport and is hereafter referred to as Site A. The second field was a commercially-operated alfalfa field near La Salle, with the research coordinated through the Central Colorado Water Conservation District (CCWCD). This field was irrigated using a center pivot sprinkler system and is hereafter referred to as Site B. Geographic coordinates, bulk density and soil texture for the soils at each site are presented in Table 1. Bulk density was obtained using a Madera Probe (Precision Machine, Inc., Lincoln, NE). The porosity was estimated using the sampled bulk density from each field and an assumed particle density of 2.65 g/cm^3. Soil textures were determined by a particle size analysis (Hydrometer Method; Gavlak, et al., 2003).

Table 1. Site Name, Geographic Coordinates, Porosity (φ), Dry Soil Bulk Density (ρ_b), and Soil Texture in the 10 - 30 cm soil layer

Soil	Lat. (N)	Long. (W)	ρ_b (g/cm^3)	φ (%)	Sand (%)	Silt (%)	Clay (%)	Class
A	40°26’	104°38’	1.46	45	65	10	25	Sandy clay loam
B	40°15’	104°40’	1.68	37	85	3	12	Loamy sand

Factory Calibrations

The TDT soil water content sensor is pre-calibrated by the sensor manufacturer, which enables it to give a direct reading of volumetric soil water content (θ_v), soil temperature (°C), and electrical conductivity (EC, dS/m). The Acclima (2010) states volumetric water content accuracy of ±1% (full scale) under temperature conditions of 0.5 to 50 °C and bulk EC of 0 to 3 dS/m. Laboratory and field tests were conducted to test this claim of accuracy.

The Watermark sensor directly measures voltage excitation (in mV) which is converted to electrical resistance (in kOhms) through the datalogger’s internal program (Campbell Scientific, 2009). Soil water potential (SWP, kPa) is then estimated using the electrical resistance through another internal correction. The equations used in the dataloggers in Site A are shown in Equations 1 and 2.

$$R_s = V_r / (1 + V_r) \tag{1}$$

$$SWP = 7.407{*}R_s / (1 - 0.018{*}(T - 21)) - 3.704 \tag{2}$$

where V_r (mV) is the ratio of the measured voltage divided by the excitation voltage, R_s (kOhms) is the measured resistance, T (°C) is the soil temperature measured by the TDT sensor, and SWP (kPa) is the soil water potential. SWP is directly related to θ_v through water retention (or release) curves, which vary by soil type. The manufacturer of the Watermark sensor recommended relating the SWP to θ_v through soil water release curves for general soil types similar to those presented by Ley et al. (1994). (These are generalized soil water release curves originally published by the NRCS, and Ley et al. (1994) noted that specific soils will deviate from these generalized relations.) This curve was generalized using equation 3.

$$\theta_v = \alpha X^{\beta} \tag{3}$$

where α and β are coefficients and X is the sensor-based soil water potential (millibars, mb). The α and β coefficients for the soil in Site A are 104.63 and -0.19, respectively, and coefficients for the soil at Site B were 38.14 and -0.14, respectively.

<u>Laboratory Calibrations</u>

Laboratory calibrations were performed using soil samples collected from the upper 0-30 cm layer from sites A and B from the locations shown in Figure 1.

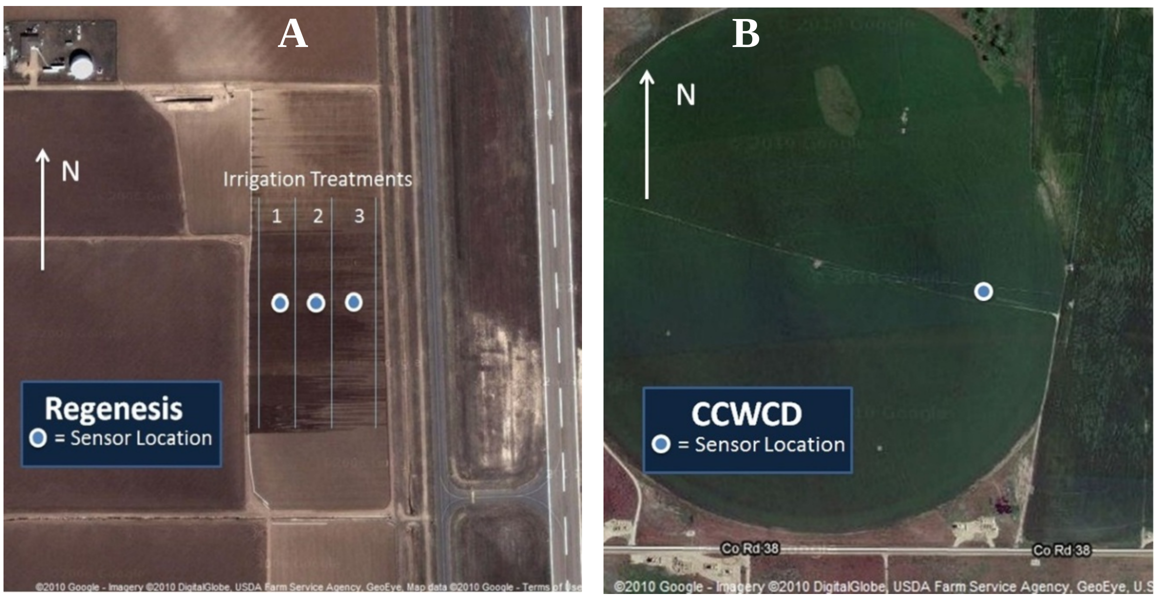

Figure 1. a) Approximate Locations of Sensors at Site A. (This field, near Greeley, CO, was split into three sections that received water in different amounts and frequencies.) b) Approximate Location of Sensors at Site B (La Salle, CO)

The laboratory calibration for the TDT sensor was based on the procedure proposed by Starr and Paltineanu (2002) and Cobos (2009). Soil collected from each field was air-dried until it could pass through a 2-mm sieve. It was then packed in a 19 L container to approximate field bulk density. The sensor was then inserted vertically into the soil, and several sensor readings were taken over an interval of at least 20 minutes. After each sensor was read, gravimetric samples were taken from the container and oven-dried at 105 °C for 24 hours. The volumetric water content was then computed by multiplying the gravimetric water content by the soil bulk density obtained from the field. The soil from the container was then wetted with 500 mL of water and mixed thoroughly. The above procedure was repeated, each time repacking the container, taking multiple sensor readings, and adding another 500 mL of water until the soil reached saturation.

A total of sixty data points (n=60) were used in the analysis of the soil from Site A, and volumetric water contents ranged from 10.7 to 35.9%. Six samples (n=6) that ranged in θ_v from 9.3 to 23.2% were used in the analysis of the soil from Site B. Fangmeier et al.

(2006) reported permanent wilting points (PWP) and field capacities (FC) for soils that were in the same textural groups as those tested in the laboratory as 16 to 26% (by volume) for Site A and 7 to 16% for Site B. Using these estimates, the range of water contents in the laboratory studies fully covered the PWP to FC range for each soil, but in no soil was saturation achieved.

A linear calibration equation was developed for each soil by plotting the probes' readings versus the volumetric water content derived from the gravimetric method. These equations were developed using Microsoft Excel® Regression Analysis, based on the sensor-based θ_v. They take the form of equation (4, below.

$$\theta_v = \alpha_0 X + \alpha_1 \tag{4}$$

where α is a coefficient and X is the sensor-based θ_v (dimensionless). During these tests, the TDT sensor registered bulk EC in the range of 0.00 - 1.60 dS/m (0.69 dS/m average) in the soil from Site A and never registered a bulk EC reading in the soil from Site B. The soil temperature was nearly constant (~21 °C) throughout the entire study.

The laboratory calibration procedure using the Watermark sensor was different from that of the TDT because water tension in the Watermark sensor must equilibrate with that of the surrounding soil before an accurate reading could be taken. Therefore the sieved soils from the previous tests were separated into multiple smaller buckets of different water contents. One Watermark sensor was placed in each bucket and left for an average of three days to equilibrate with the soil. Gravimetric samples were then taken from each bucket, oven-dried and converted into θ_v using the dry soil bulk density obtained from field samples. A total of seven samples (n=7) were used in the analysis of the soil from Site A and three samples (n=3) were used in the analysis of the soil from Site B.

Two types of calibration equations were developed by plotting θ_v versus the SWP sensor output. The logarithmic equation is shown in equation 5 below.

$$\theta_v = \alpha \ln|X| + \delta \tag{5}$$

where α and δ are coefficients and X is the sensor-based soil water tension (millibars, mb).

The van Genuchten (1980) equation was also used to relate the sensor-based SWP to measured θ_v, shown in equation 6.

$$\theta_v = \theta_r + \frac{(\theta_s - \theta_r)}{[1 + (\alpha h)^n]^{1-\frac{1}{n}}} \tag{6}$$

where θ_s is the saturated soil water content, θ_r is the residual soil water content, h is the absolute value of the soil water tension (cm H_2O), and α (cm^{-1}) and n are soil-specific coefficients. When fitting the van Genuchten (1980) equation to the laboratory and field data, θ_s was estimated for each soil using the assumed porosity at each location.

However, θ_r was assumed for each soil using the values recommended by Schaap & Leij (1998): 0.063 and 0.079 for the sandy clay loam (Site A) and loamy sand (Site B), respectively. The α and n coefficients were then derived using Microsoft Excel® Solver. To analyze the accuracy of the calibration equations obtained from the laboratory procedure, the 'laboratory equations' were applied to the field sensors' readings and results were compared with the field-sampled θ_v.

Field Calibration

During the summer of 2010, TDT and Watermark sensors were installed at Site A. This site had three differing irrigation treatments, as shown in Figure 1. Each treatment contained one TDT sensor and treatment 1 had one Watermark sensor. The sensors were installed under the crop row, roughly 0.2 m apart from each other, at a uniform depth of 10 cm below the average elevation of the row height and the furrow bottom. These sensors were installed by digging a shallow trench and inserting the sensors horizontally into the wall, then backfilling the trench. Data collection for the sensors began in mid-July.

At Site B, one Watermark sensor was placed at 61 cm and another at 91 cm below the surface. These sensors were installed by creating a small vertical hole with a soil auger, then lowering the sensor to the desired depth. Also at this location, a thermocouple was installed 30 cm beneath the surface to monitor soil temperature (°C). The Watermark and temperature sensors came into service in the end of July of 2010.

From the time of installation until the first week of October, 2010, automated sensor readings were recorded at Site A every five minutes. At Site B automated readings were recorded every eight hours, until the third week of October, 2010. Readings were compared with periodic gravimetric measurements, totaling eleven from each irrigation treatment in Site A and five at each depth from Site B.

The gravimetric samples were taken using a soil auger approximately 1-2 meters away from each sensor location. These samples were immediately placed in sealed containers inside a cooler and taken directly to a laboratory to be weighed, oven-dried, and weighed again. The gravimetric samples were then converted into θ_v using the dry soil bulk density field values. During the times of gravimetric field sampling at Site A, soil temperatures ranged from 15 - 22 °C in irrigation treatment 1, 15 - 24 °C in treatment 2, and 16 - 30 °C in treatment 3. Bulk EC ranged from 0 - 1.23 dS/m in treatment 1, 0 - 1.31 dS/m in treatment 2, and 0 - 2.12 dS/m in treatment 3. At Site B, soil temperatures ranged from 13 - 20 °C.

Sensor-specific linear calibration equations were developed for the TDT sensors based on the θ_v read by the sensor. This equation is shown in equation 4, above. For the Watermark sensors, the logarithmic and van Genuchten (1980) equations (shown in equations 5 and 6, above) were derived.

Statistical Analysis

Four statistical measures were computed to compare and evaluate each model-predicted (*P*) equation with the observed (*O*) gravimetric samples taken from the field and laboratory soils. These include the coefficient of determination (R^2), mean bias error (*MBE*; Equation 7), root mean square error (RMSE; Equation 8), and index of agreement (κ; Equation 9) as defined by Willmott (1982).

$$MBE = n^{-1}\sum_{i=1}^{n}(P_i - O_i) \tag{7}$$

$$RSME = \left[n^{-1}\sum_{i=1}^{n}(P_i - O_i)^2\right]^{.5} \tag{8}$$

$$\kappa = 1 - \left[\frac{\sum_{i=1}^{n}(P_i - O_i)}{\sum_{i=1}^{n}(|P'_i| - |O'_i|)^2}\right] \tag{9}$$

where n is the sample size, $P'_i = P_i - \bar{O}$, $O'_i = O_i - \bar{O}$, and $\bar{O}$ is the average observed value. The units for MBE and RMSE are volumetric water content (%), and κ is dimensionless.

Hignett and Evett (2008) point out that in most agricultural and research applications the measurement accuracy needs to be within 0.01 to 0.02 $m^3\ m^{-3}$. Therefore MBE under 2.0% and RMSE less than 3.5% fit this criterion. The scale of κ ranges between 0-1, with higher numbers representing greater correlation between the model prediction and observations.

RESULTS AND DISCUSSION

Factory Calibration Evaluation

This study found that, under laboratory and field conditions, the factory-based calibrations of θ_v did not achieve the required accuracy within the PWP to FC range of water content for any sensor. The statistical values (see Table 2) for the TDT sensor indicate that, in the laboratory, the factory calibration underestimated θ_v by 1.5% in the sandy clay loam (Site A), and overestimated by 6% in the loamy sand (Site B). However, the RMSE was greater than 3.5% in all soils, so the factory calibration did not meet the criteria for any soil. These less-favorable values may be attributed to a lower number of samples or problems with the sandy soil not packing correctly around the sensor's metal loop. Under no laboratory tests did the Watermark sensors achieve the required accuracy.

Table 2. Comparison of the Factory Calibration-Based θ_v (%) with Laboratory Measurements of θ_v (%) for the Different Soils in the Study

Soil Type	Sample Size (n)	R^2	MBE (%)	RMSE (%)	κ
TDT					
Sandy clay loam	60	0.94	-1.2	3.9	0.95
Loamy sand	6	0.98	6.1	6.7	0.75
Watermark					
Sandy clay loam	7	0.93	20.5	21.1	0.32
Loamy sand	3	0.65	8.2	8.8	0.61

In the field tests, the MBE and RMSE of applying the factory calibration to the data from the TDT sensor in treatment 2 were within the limits (0.7% and 2.3%, respectively), but the MBE's in treatments 1 and 3 were 2.7% and -2.2%, respectively. The Watermark's factory calibration overestimated θ_v in the three treatments at Site A by 11.2% and at both depths at Site B by 10% (Table 3).

Table 3. Comparison of the Factory Calibration-based θ_v (%) with Field Measurements of θ_v (%) at Sites A and B

Soil Type	Location / Depth (cm)	Sample Size (n)	R^2	MBE (%)	RMSE (%)	κ
TDT						
Sandy clay loam (A)	1	11	0.73	2.1	3.0	0.85
	2	11	0.83	1.8	2.9	0.92
	3	12	0.77	-1.8	3.3	0.90
Watermark						
Sandy clay loam (A)	1	15	0.87	11.2	12.6	0.48
Loamy sand (B)	61	5	0.85	10.5	10.5	0.27
	91	5	0.33	10.4	10.6	0.32

<u>Laboratory Calibration Evaluation</u>

Soil-specific calibration equations developed in the laboratory yielded high levels of accuracy, well within the targeted statistical parameters, for both sensors. The MBE, RMSE and κ parameters, shown in Table 4, were each better than the parameters representing the factory calibrations. In both soils, the logarithmic and van Genuchten (1980) equations developed for the Watermark sensor produced similar levels of accuracy. In the soil from Site B, both equations developed for the Watermark sensor had higher RMSE values than in the soil from Site A, most likely due to the smaller sample size.

Table 4. Comparison of the Laboratory-based Calibration of θ_v (%) versus Laboratory Measurements of θ_v (%)

Soil Type	Eqn. Type	Sample Size (n)	R^2	MBE (%)	RMSE (%)	κ
TDT						
Sandy clay loam	Linear	60	0.94	0.0	1.9	0.98
Loamy sand	Linear	6	0.98	0.0	0.7	0.99
Watermark						
Sandy clay loam	Logarithmic	7	0.94	0.0	1.1	0.98
	van Genuchten	7	0.93	0.0	1.2	0.98
Loamy sand	Logarithmic	3	0.60	0.0	3.3	0.86
	van Genuchten	3	0.75	-0.2	2.6	0.93

Table 5 displays the results of comparing the use of the laboratory-derived calibration equations with field-measurements of θ_v (%). The large MBE (> ±2.0%) and RMSE (> 3.5%) values indicated that the laboratory-derived calibration equations for the both sensors were not consistently accurate. When compared with the TDT's factory calibration, the TDT's laboratory calibration yielded comparable MBE and RMSE values, and was accurate only in treatment 2. The laboratory equations for the Watermark sensor at Site A were less inaccurate than the factory calibration, and the accuracy of the laboratory-derived van Genuchten (1980) calibration equation was similar to the accuracy of the laboratory-derived logarithmic equation. The laboratory equations developed for the Watermark sensors at Site B accurately predicted θ_v at the 61- and 91-cm depths (RMSE = 1.4% and 2.4, respectively). At both depths, the laboratory-derived van Genuchten (1980) calibration equation performed nearly identically to the laboratory-derived logarithmic equation. This is evidence again that the van Genuchten (1980) equation was not significantly more accurate than the logarithmic equation for this application, and that the additional work of deriving the parameters for the former equation did not seem worthwhile, within the range of soil water contents analyzed.

Table 5. Comparison of the Laboratory-based Calibration of θ_v (%) versus Field Measurements of θ_v (%) at Sites A and B

Soil Type	Location / Depth (cm)	Eqn. Type	Sample Size (n)	R^2	MBE (%)	RMSE (%)	κ
TDT							
Sandy clay loam (A)	1	Linear	11	0.76	2.8	3.3	0.78
	2	Linear	11	0.83	0.8	2.1	0.93
	3	Linear	12	0.74	-2.0	3.7	0.86
Watermark							
Sandy clay loam (A)	1	Logarithmic	15	0.81	-3.0	3.6	0.82
		van Genuchten	15	0.90	-2.6	2.8	0.87
Loamy sand (B)	61	Logarithmic	5	0.83	1.3	1.5	0.87
		van Genuchten	5	0.88	1.2	1.6	0.78
	91	Logarithmic	5	0.30	0.6	2.4	0.73
		van Genuchten	5	0.38	1.6	2.4	0.61

Field Calibration Evaluation

The field-based calibration equations developed for both sensors, within the PWP to FC range of water contents, showed higher levels of accuracy than the factory- or laboratory-derived equations. As shown in Table 6, the RMSE values were consistently low (and κ values high) for both sensors in both fields, and well within the ideal statistical targets. This also agrees with research conducted by Dr. Steve Evett (Personal Communication, 2010), that “a linear soil-specific calibration would suffice to correct [the TDT] to be useful in scheduling [irrigations] according to” management allowed depletion. When comparing the complex van Genuchten (1980) equation with the simpler logarithmic equations, it appears that the van Genuchten (1980) calibration equation is trivially more accurate (RMSE decrease by 0.5%) than the logarithmic calibration equation.

Table 6. Comparison of the Field-based Calibration of θ_v (%) versus Field Measurements of θ_v (%) at Sites A and B

Soil Type	Location / Depth (cm)	Eqn. Type	Sample Size (n)	R^2	MBE (%)	RMSE (%)	κ
TDT							
Sandy clay loam (A)	1	Linear	11	0.73	0.0	1.9	0.91
	2	Linear	11	0.83	0.0	1.9	0.95
	3	Linear	12	0.74	0.0	2.4	0.93
Watermark							
Sandy clay loam (A)	1	Logarithmic	15	0.81	0.0	1.6	0.94
		van Genuchten	15	0.86	0.0	1.4	0.96
Loamy sand (B)	61	Logarithmic	5	0.83	1.0	1.3	0.90
		van Genuchten	5	0.89	1.0	1.4	0.82
	91	Logarithmic	5	0.30	0.6	2.4	0.73
		van Genuchten	5	0.36	1.6	2.4	0.60

An analysis of the factory-, laboratory-, and field-derived calibrations of θ_v (%) is not complete without a visual inspection of the data in graphical form. In Figure 2, the derived equations are applied to the TDT sensor in treatment 1 at Site A. This field was surface irrigated with application times exceeding 12 hours, so it is assumed that the soil around the sensors reached saturation. Assuming a porosity of 45%, the TDT's factory calibration measured impossible levels of water content, while the laboratory- and field-derived equations indicated saturation. It is evident in Figure 2 that the TDT responded well to small amounts of rainfall (for example, ≈3 mm on August 19^{th}), and all equations measured water content levels similar to the gravimetric field measurements.

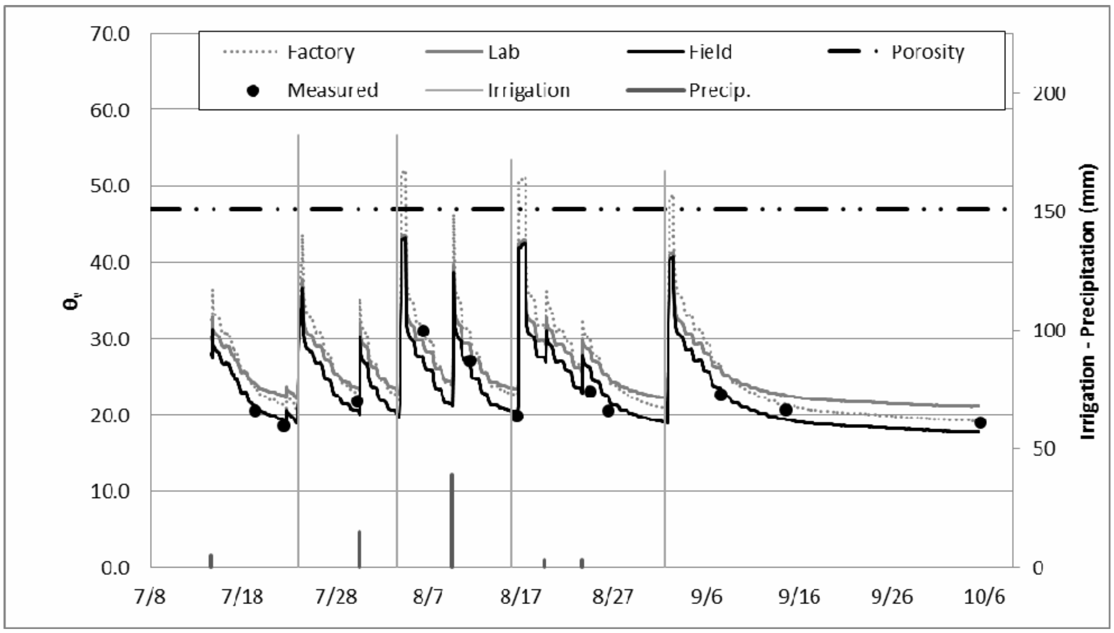

Figure 2. TDT Calibration Equations for Site A, Treatment 1

In Figure 3, it is clear that none of the Watermark's calibration equations adequately represented the full range of water contents. The Watermark's factory calibration reported water contents much greater than porosity, and the other equations did not report saturation during irrigations. It is assumed that if gravimetric measurements would have been made immediately after irrigations ended, the derived equations also would have reported saturated conditions. The field measurements in Figure 3 show that the field-derived van Genuchten (1980) equation was the best in measuring water contents in the ranged of PWP to FC. This coincides with the data presented in the previous tables.

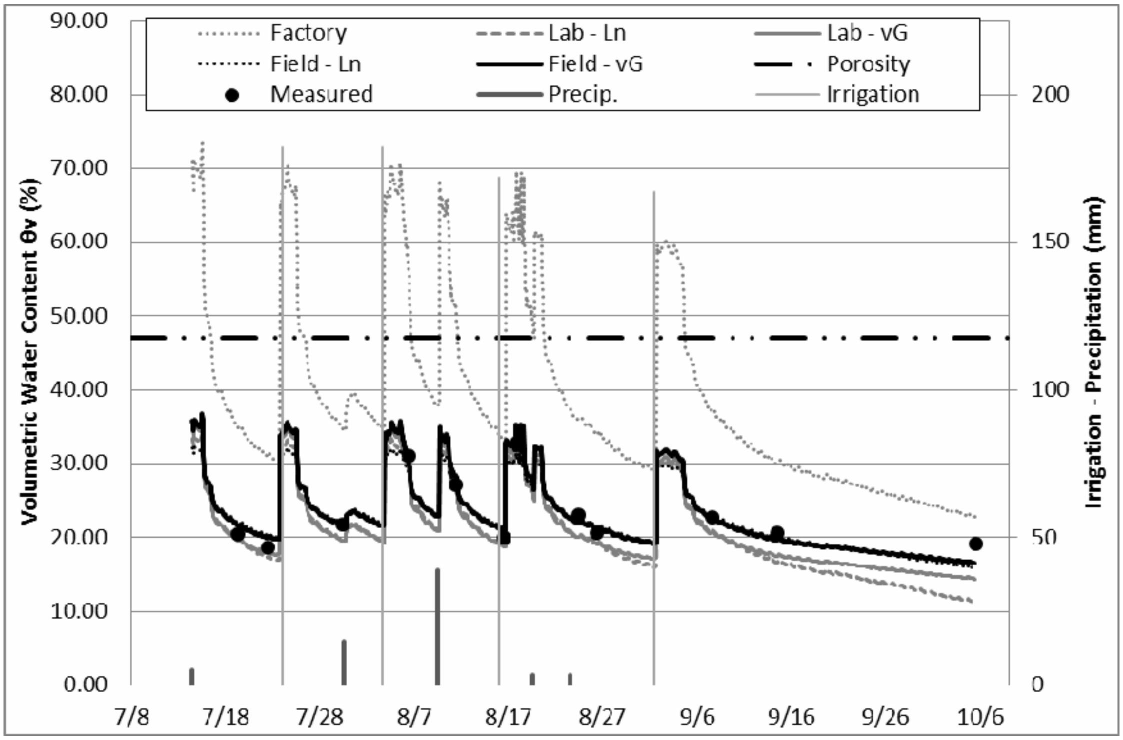

Figure 3. Watermark Calibration Equations for Site A, Treatment 1

CONCLUSIONS

This research evaluated the performance of Watermark soil water potential and TDT soil water content sensors under laboratory and field conditions in sandy clay loam and loamy sand soils. Measured soil water content/potential values were compared with corresponding values derived from gravimetric samples, ranging in water content from permanent wilting point (PWP) to field capacity (FC). Linear calibration equations were developed for the TDT sensor. For the Watermark sensor, calibration equations taking the form of van Genuchten (1980) and logarithmic calibration equations were developed. These equations were compared against each other and with factory-recommended calibrations. Statistical targets for these tests were ±2% (units in θ_v expressed as a %) MBE and less than 3.5% (units in θ_v expressed as a %) RMSE.

In the laboratory tests on the soils from Sites A and B, we found that the TDT's factory-recommended calibration was not suitable for either soil. Laboratory tests on the same soils also found that the Watermark's factory-recommended calibration overestimated θ_v by 10-11%, for both soils. The laboratory data was used to develop various calibration equations that improved the accuracy of the factory calibrations, and all equations reached the required statistical parameters.

During the summer of 2010, TDT and Watermark sensors were installed in irrigated agricultural fields near Greeley, CO. The factory-recommended and laboratory-derived calibration equations were applied to these sensors, and compared against periodic gravimetric samples. At Site A, the factory calibration for the TDT sensor was accurate in treatment 2, but not treatments 1 and 3 (MBE of 2.7% and -2.2%, respectively). The laboratory calibrations for the TDT sensors were not consistently accurate in every treatment. At Sites A and B, the Watermark's factory-recommended equations overestimated θ_v by 10-11%. The Watermark's laboratory-derived equations underestimated the field-measured θ_v (MBE: -3.8%) at Site A, but at Site B, the laboratory-derived equations applied to the Watermark sensors were within the statistical goals.

Field-derived calibration equations developed for both sensors in the fields returned higher accuracy than the factory- or laboratory-derived equations. The RMSE for the TDT sensors at Site A were ≈2% and for the Watermark sensors RMSE ranged from 0.5% to 2.2% at both sites.

REFERENCES

Acclima, 2010. Cut Sheet, TDT Soil Moisture Sensor. Acclima, Inc., Meridian, ID. Revision 3/10. Available online at: http://acclima.com/wd/acclimadocs/MoistureSensor/TDT%20Moisture%20Sensor%20Cut%20Sheet.pdf (verified 13 Feb. 2011).

Campbell Scientific, 2009. Instruction Manual: Models 253-L and 257-L (Watermark 200) Soil Matric Potential Sensors. Revision 3/09.

Cobos, D.R., 2009. Calibrating ECH2O Soil Moisture Sensors. Application Note. Decagon Devices, Inc.

Cooley, H., J. Christian-Smith, and P.H. Gleick. 2009. Sustaining California Agriculture in an Uncertain Future. Pacific Institute, Oakland, CA. Available online at: http://www.pacinst.org/reports/california_agriculture/ (verified 7 Jan. 2011).

Evett, S., R.C. Schwartz, N.Th. Mazahrih, M.A. Jitan, and I.M. Shaqir. 2010. Personal Communication.

Fangmeier, D.D., J.E. William, S.R. Workman, R.L. Huffman, G.O. Schwab, 2006. Soil and Water Conservation Engineering, 5th Ed. Thomson Delmar Learning, Clifton Park, NY. p. 337.

Gavlak, R.G., D.A. Horneck, and R.O. Miller, 2003. Plant, Soil and Water Reference Methods for the Western Region, 2nd ed. WREP 125.

van Genuchten, M.Th., 1980. A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils. Soil Sci. Soc. Am. J. 44:892-898.

Hignett, C., and S. Evett, 2008. Direct and Surrogate Measures of Soil Water Content. In: Evett, S.R., L.K. Heng, P. Moutonnet, and M.L. Nguyen, editors. Field Estimation of Soil Water Content: A Practical Guide to Methods, Instrumentation, and Sensor

Technology. IAEA-TCS-30. International Atomic Energy Agency, Vienna, Austria. ISSN 1018-5518. p. 107-109.

Kapinos, J. 2006. Irrigation Sensors Benefit Afghanistan Farmers. United States Air Force, 455th Air Expeditionary Wing Public Affairs, Camp Cunningham, Bagram Airfield, Afghanistan.

Ley, T.W., R.G. Stevens, R.R. Topielec, and W.H. Neibling, 1994. Soil Water Monitoring and Measurement. PNW 475, a Pacific Northwest Publication. Washington State University, Pullman, WA.

National Resource Conservation Service (NRCS). 2009. Agricultural Water Enhancement Program Project Proposals Approved for Fiscal Year 2009. U.S. Department of Agriculture, Washington, DC.

Schaap, M. G., and F. J. Leij. 1998. Database related accuracy and uncertainty of pedotransfer functions. Soil Sci. 163:765-779.

Starr, J.L., and I.C. Paltineanu. 2002. Methods for Measurement of Soil Water Content: Capacitance Devices. p. 463-474. In J.H.Dane, and G.C.Topp (ed.) Methods of Soil Analysis: Part 4 Physical Methods. Soil Science Society of America, Inc., Soil Science Society of America, Inc.

Willmott, C.J. 1982. Some Comments on the Evaluation of Model Performance. Bull. of Am. Meteorol. Soc., 63, 1309-1313.

MEETING WATER CHALLENGES IN IDAHO THROUGH WATER BANKING

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ABSTRACT

Idaho authorized water banking in 1979. Today, a statewide water bank functions as well as local rental pools. Stored water and natural flows are traded. The water bank and local rental pools are used to meet the needs of irrigators suffering from drought induced water shortages, to meet instream flow needs of endangered species, and to meet the needs of water users having junior priority surface or ground water rights. Both lessors and lessees have benefited from water rentals.[3]

This paper will focus mainly on recent experiences in the Upper Snake (the Snake River above Milner Dam near Burley, Idaho) and Payette Rental Pools, the two most active in the State. Both Rental Pools have been very successful. Particularly in the Payette Basin, income from rentals has enabled water users to upgrade their irrigation systems with resulting significant improvements in water management. The Upper Snake Rental Pool, while also experiencing significant rentals and opportunities for water users, has had to deal with drought induced competition for water that seriously challenged rental pool managers. Setting prices in changing economic conditions, addressing impacts to non-participating water users, and determining priorities among prospective uses were all addressed. Conflicts have not ended, but it is fair to say that through the persistence and dedication of rental pool managers and participants, the challenges were successfully addressed.

BACKGROUND

In Idaho, rentals of stored water first took place in the 1930's. In 1976 the Idaho Water Resources Board, in its State Water Plan, recommended the establishment of a water supply bank. In 1979 the Idaho legislature formally established a water supply bank. That year the Water Resources Board appointed the Committee of Nine, an advisory committee to the Watermaster of Idaho's Water District One, to manage the Upper Snake Rental Pool[4] which has functioned since that date. In 1988, with the Bureau of

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[3] Rigby presented two previous papers on or touching on flow augmentation. At the 1989 conference he presented a paper entitled *Water Banking in Idaho.* In 1996 he presented a paper *Acquiring Water For Flow Augmentation*. Some of the information in those papers will be summarized here for the sake of completeness.
[4] The term "Upper Snake" has different meanings. In Idaho water circles the term refers to the area of the Snake River served from primarily surface diversions above Milner Dam near Burley Idaho. In the

Reclamation's encouragement, a rental pool was established in the Boise River Basin. A rental pool was established in the Payette Basin, also with Reclamation's encouragement, in 1990. Today 6 local rental pools function under the management of local committees. Local pools cover specific geographic areas. Four of the six local rental pools only trade stored water. The State Water Bank continues to operate for all areas not covered by local rental pools and involves the trading of surface and ground water supplies.[5]

THE BUREAU OF RECLAMATION'S INVOLVEMENT WITH IDAHO RENTAL POOLS

For many decades the Bureau of Reclamation was an interested bystander in discussions about renting water from one reservoir spaceholder to another. Reclamation's role was to deliver stored water to contracting entities. Under the terms of reservoir *spaceholder* contracts, largely unique to Idaho, the onus of dealing with water shortages is mainly the responsibility of the spaceholders. Each spaceholder contract provides carryover privileges for contracted reservoir storage. Spaceholders decide how they will manage their available water supplies. They may be aggressive in their use of water this year, or be more conservative and save water in storage for possible use next year. The reservoirs are more likely than not to fill in a given year, so water saved this year has a good chance of spilling past the reservoirs with the next spring freshet. Spaceholders have three basic choices in a given year: (1) use the water they need without regard to next year; (2) conserve storage in case the reservoirs don't fill next year; or (3) rent some of their stored water, often at the urging of a needy user, and thereby help a neighboring user while improving one's financial condition. Reclamation may be consulted, but decisions about the use of stored water rest with the boards of directors of contracting entities—they, not Reclamation, get to explain their decisions to disgruntled users if they guess wrong.[6]

In 1991 Idaho Sockeye Salmon were listed under the Endangered Species Act (ESA).[7] Other species of anadromous fish were listed in the following years. Today 13 species of salmon and steelhead listed as threatened or endangered under the ESA are considered to be impacted by Reclamation's Idaho projects.[8] As a result, Reclamation's Pacific

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vernacular of Reclamation's Endangered Species Act consultations discussed herein, the term "Upper Snake" refers to the Snake River above Idaho Power Company's Hells Canyon Complex on the Idaho/Oregon Border. Perception matters, and from Portland the Upper Snake starts at Hells Canyon. In Idaho it starts at Milner. This paper uses the Idaho definition.

[5] http://www.idwr.idaho.gov/WaterManagement/WaterRights/WaterSupply/ws_default.htm

[6] Reclamation does get plenty of advice in managing the reservoirs to make sure they fill. Many irrigation water users understandably have a fill and spill mentality. The fact is Reclamation has multiple project purposes to implement, including flood control and power. While most people would probably agree that Reclamation errs on the side of filling reservoirs, it attempts to balance needs for instream flows and hydroelectric power generation during the winter storage season.

[7] Federal Register of November 20, 1991

[8] Species of anadromous fish addressed in NOAA Fisheries May 5,2008 Biological Opinion on the operation of Reclamation's Snake River projects are: Snake River fall chinook, Snake River spring/summer Chinook, Snake River sockeye, Snake River steelhead, Upper Columbia River spring chinook, Upper Columbia River steelhead, Middle Columbia River steelhead, Columbia River chum, Lower Columbia River Chinook, Lower Columbia River coho, Lower Columbia River steelhead, Upper Willamette River Chinook, and Upper Willamette River steelhead. As if that lengthy and far ranging list were not enough, NOAA also addressed the affects of Reclamation's upper Snake operations on southern resident killer

Northwest Region has become intimately familiar with what had been previously considered to be arcane provisions of the Endangered Species Act. Two farm boys from South Dakota and Utah with degrees in agricultural engineering and economics and who joined Reclamation to help tame the Wild West learned about things they could never have imagined when they joined Reclamation. Concepts like *may affect*, *reasonable and prudent measures*, and *adverse modification of critical habitat* became part of their everyday vocabulary.

While these changes were dramatic to Reclamation, they were a severe shock to the system of Idaho's irrigation community and threatened to seriously strain relationships as the idea of providing water from Reclamation reservoirs located hundreds of miles from the nearest salmon became a reality.[9] In 1991 at the request of the governor's of Idaho, Montana, Oregon, and Washington, Reclamation first provided water for flow augmentation. NOAA Fisheries' Biological Opinions beginning in the early 1990s called on Reclamation to provide up to 427,000 acre-feet for flow augmentation, from willing sellers and in accordance with state water law. Changes in State law were made to accommodate the use of water for flow augmentation, and specified that all water provided must go through the respective rental pools. This accommodation agreed to by Reclamation further cemented the willing seller policy and secured for Reclamation state approval and protection of flow augmentation deliveries. Otherwise the water released could have been considered part of the natural flow and subject to diversion. The volume to be provided was increased to up to 487,000 acre-feet (also from willing sellers and in accordance with state water law) with completion of the Nez Perce tribal water rights settlement in 2004).[10] The authors participated in the negotiations and wanted to avoid battles with the water user community over rates. Therefore, they sought and secured acceptance of a state-wide rate for rentals. Rental rates started at $14 through 2012 and increase to $23 for the years 2023-2030. Rates are inclusive of administrative fees.[11]

Reclamation has provided water for flow augmentation requested in Biological Opinions issued since 1993.[12] Flow augmentation usually comes from four sources: (1) water stored in project reservoirs but never contracted for use that has been rededicated for flow augmentation; (2) reservoir storage specifically reacquired for flow augmentation; (3) annual rentals from the Upper Snake, Payette, and to a lesser extent the Boise Rental

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whales, or orcas, which feed on salmon in the ocean and which are deemed to be possibly affected by the operation of Reclamation's Snake River Projects

[9] Other local species listed under the ESA also impact the operation of Reclamation projects. Bull trout occupy project reservoirs in the Boise Project. Listed snails live in the sediments of the Snake River in and below Reclamation Minidoka Project reservoirs.

[10] http://www.idwr.idaho.gov/waterboard/WaterPlanning/nezperce/default.htm

[11] Mediator's term sheet, page 21. The rates specified pertain to water rented from spaceholders. The state statute specified that all water provided must go through the rental pools, and this applied to space never contracted to water users or reacquired by Reclamation. This water in Reclamation space was obliged to pay water district *administrative fees*, which are currently $0.75, $0.80, and $1.00 on the Boise, Upper Snake, and Payette Basins respectively.

[12] NOAA Fisheries and the Bureau of Reclamation's first status report (Dated October 3, 2006) on the remand of NOAA Fisheries Upper Snake Biological Opinion, found at: http://www.salmonrecovery.gov/Files/BiologicalOpinions/2006/Final%20_First_Remand_Joint_status_report.pdf

Pools; (4) a thirty year lease with the Idaho Water Resources Board for 60,000 acre-feet of natural flows, which was arranged pursuant to the Nez Perce water rights settlement, plus natural flows acquired in Oregon.[13] The rental pools have been essential to Reclamation's success in providing water for flow augmentation and always will be. Except for the extreme drought years of 2001-2004, Reclamation has been able to meet the volume identified by NOAA Fisheries.

PAYETTE RENTAL POOL

The Payette Basin is probably noted as much for its recreation opportunities as anything. The basin has a wide range in elevation over its' fairly short course which helps make the Payette River below Cascade Dam a highly popular whitewater rafting and kayaking area, with a class V designation. Logging was a large part of the economy for a period but agriculture has persisted as an important part of the area's economy. The basin is covered by Idaho's Water District 65. The District's Rental Pool governs the rental of water stored in Reclamation's Cascade and Deadwood Reservoirs. These reservoirs are features of the Payette Division of the Boise Project. Together they store more than 800,000 acre-feet of water. Some 120,000 acres are irrigated in the Payette Division and are nearly evenly split between those receiving a full supply and those which receive a supplemental supply.[14]

The Payette River Basin is characterized by a reasonably large and productive watershed with a limited irrigated area. Consequently, while the basin has suffered water shortages in the past it is relatively speaking the most water rich of Reclamation project areas in the State of Idaho. Consequently, Reclamation has relied on the Payette in providing water for flow augmentation for proportionately more water than the Boise or Upper Snake rental pools.

In a typical year Reclamation may release up to 95,000 acre-feet of water which was never contracted and has been reassigned to flow augmentation. In addition, water users typically rent to Reclamation up to 65,000 acre-feet of water for flow augmentation. Historically, from 150,000 to 175,000 acre-feet of water have been provided for flow augmentation from the Payette Basin. The larger volume was provided when conditions elsewhere were dry and when Reclamation elected to release water from Deadwood Reservoir that had been dedicated to maintenance of a minimum reservoir pool. Recent consultations with the Fish and Wildlife Service on ESA listed bull trout have placed greater emphasis on the pool level at Deadwood, so it is unlikely that the minimum pool will be available for salmon and steelhead flow augmentation in the future. In addition, while some 336,000 acre-feet in Cascade Reservoir have never been contracted, a 300,000 acre-foot minimum pool has been adopted as an acceptable volume to deal with late summer water quality problems at the reservoir.

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[13] Bureau of Reclamation, Appendix C to August 2007 Biological Assessment, Bureau of Reclamation's Operation and Maintenance in the Snake River Basin above Brownlee Reservoir, pp C3-C6. It also describes the use of water stored in so-called power head space that is used when the other sources do not yield 427,000 acre-feet.

[14] See Reclamation's description of the Payette Division located at: http://www.usbr.gov/projects/Project.jsp?proj_Name=Boise Project

With the exception of the extremely dry year of 1977, the Payette River Basin remained essentially unregulated.[15] The Water District 65 Rental Pool was established in 1990.[16] In 1991 the patrons of the Water District hired the first permanent watermaster. Motivation for a rental pool arose from water users' awareness that opportunities existed to generate revenue from rentals and recognition that aging facilities wouldn't last forever without significant upgrades or outright replacement. The timing of the rental pool was fortuitous. Within a few short years Reclamation's flow augmentation efforts were underway and rental pool activity quickly expanded to current levels. Today while most rentals are dedicated to flow augmentation for listed salmon and steelhead, irrigators in the basin rent about 8,800 acre-feet in a typical year and the Idaho Power Company rents up to 10,000 acre-feet in some years.

The Water District 65 Rental Pool has truly achieved the aspirations that led to its establishment. Recognizing the need for system upgrades, the water users agreed at the outset to dedicate one-third of the revenue generated from water rentals to infrastructure improvements. In 1997 part of the "administrative fees" (currently $1.00 per acre-foot) applicable to all rentals were identified as a source of funds for improvements. The water users have been frugal in operating the water district and revenues from the administrative fees became sufficient to dedicate part of them to an incentive program. Since 1997 nearly $1.1million of administrative fees has been expended on system improvements on a cost share basis. The total value of improvements under this incentive program is nearly $4.5 million. Project features include canal lining and piping, headgate automation and telemetry, automated check structures and automated spill structures.

Water measurement has improved significantly as the Payette River has transformed from a basin with relatively few measured diversions to the current situation where nearly all major diversions possess constant remote monitoring and in most cases automated remote control. Supervisory Control and Data Acquisition via telemetry are a good fit for the basin. Operating an accounting system requires good measurement and monitoring in order for the accounting to be accurate. Not only has measurement improved, nearly all significant diversions possessing constant remote monitoring and in most cases automated remote control.

Over the last 14 years, 48 separate sites have been equipped with constant monitoring. Most of these sites include automatic water control to achieve a preset flow or stage requirement. This results in constant canal deliveries in contrast to historic conditions where diversions rose and fell with changes in river stage. Data from these sites is recorded hourly and downloaded to the water district office daily where it is fed into the water accounting program of the Idaho Department of Water Resources. Additional sites continue to be added with the consequence that water accounting becomes more precise

[15] The authors are indebted to Water District 65 Watermaster Ron Shurtleff for much of the factual information in the remainder of this section.
[16] The current Water District 65 (Payette) Rental Pool Procedures are on the web at: http://www.idwr.idaho.gov/WaterManagement/WaterRights/WaterSupply/PDFs/WD65_2005_Procedures.pdf

every year. To date 15 separate irrigation districts and canal companies have participated by adding supervisory control and data acquisition (SCADA) to their system. In addition to the 48 automation sites mentioned above, 22 canal check structures have been added or modified to be motorized, allowing for nearly infinite adjustment. These check structures have been installed throughout three of the larger canal companies in the valley. They were built to afford automation at a later date as funding allows.

One of the largest projects made possible because of water marketing was replacement of a diversion dam. The outdated ridged structure was replaced with an inflatable rubber dam. It is capable of automatically maintaining a preset stage as the river flows fluctuate. It had a total cost of $1,578,547.00. The canal company was able to finance the dam's installation and by utilizing proceeds from water banking activity, the structure will be paid off in 2016.

As a result of these improvements the water supply in the Payette Basin has become more secure. Automated control and telemetry, accurate water measurement, and real time monitoring has resulted in water savings. Water saved is now available for rental to generate additional revenue in good years and to extend supplies in dry years. Therefore, the conservation of water through these improvements comes without impacting the water supplies available to irrigation water users. This is a significant unexpected benefit.

These improvements have been achieved in many cases while holding water assessments to canal patrons comparatively flat. Companies once struggling to just keep up with the needs of aging facilities are now able to maintain a safety cushion of funds which allows them to undertake projects on their own and keep their system functionally sound.

UPPER SNAKE RENTAL POOL

The Upper Snake River is a thriving agricultural area. The Snake River rises in the mountains of Yellowstone National Park and is the longest and largest tributary of the Columbia River. Underlying the Upper Snake Basin is the enormous Eastern Snake Plain Aquifer (ESPA), which in the memory of the authors was described as an inexhaustible resource. Some two million acres are irrigated in the basin split approximately equally between surface and ground water diversions. The aquifer discharges into the Snake River at springs in various locations in the basin, most notably in the Thousand Springs area, where several thousand cubic feet per second discharge into the river. The water emanating from the Thousand Springs is clean and cold, and ideal for fish propagation. Idaho's aquiculture industry is located mainly in the Thousand Springs area and leads the nation, at least in terms of water use.[17]

The Upper Snake Rental Pool involves the exchange of stored water from one private reservoir and seven Bureau of Reclamation storage reservoirs located in Idaho and Wyoming. These reservoirs store some 4.1 million acre-feet of water and provide a full

[17] United States Geological Survey, Estimated Use of Water in the United States, Circular 1344, page 30

or supplemental supply of water to over 1 million acres of land.[18] This rental pool has seen considerable activity through the years. In 1985 the Idaho Power Company rented 350,000 acre-feet of water for summertime hydroelectric power generation at its power plants at American Falls and downstream. That same year a total of 12,169 acre-feet were rented by irrigators.[19] In 1988, the year dry conditions led to massive forest fires that ravaged much of Yellowstone National Park, snowpack was between 71-82% of average,[20] and irrigation rentals hit 136,000 acre-feet, while Idaho Power rented 50,000 acre-feet.[21] In the even drier year of 1992, snowpack ranged from 50-73% of average.[22] That year, for the first time since the Upper Snake Rental Pool was established in 1979, insufficient water was made available for rental to meet the total irrigation requests of 52,779 acre-feet.[23] Only 9,954 acre-feet were provided from the rental pool, all for irrigation.[24]

While 1988, 1992, and 1994 saw relatively dry conditions, they did not compare to the period starting in the year 2000 which experience record drought conditions in the Upper Snake. According to Karl Dreher, former Director of the Idaho Department of Water Resources:

> Based on the 2-year, 3-year, 4-year, and 5-year moving averages of unregulated (corrected for reservoir storage) natural flow in the Snake River at the USGS stream gage located 2.4 miles upstream of Heise, Idaho ("Heise Gage"), since the year 2000 the Upper Snake River Basin has experienced the worst consecutive period of drought years on record.[25]

This drought impacted aquiculture in the Thousand Springs area. Spring users had seen the spring discharge decline from peak flows experienced in the 1950s[26] and made calls by 2003 for priority delivery of water, seeking to regulate groundwater pumping from the ESPA. Negotiations were undertaken to resolve the issues,[27] but they ultimately failed and existing calls were renewed and new ones initiated.[28] By 2005 the largest aquiculture

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[18] See: http://www.usbr.gov/projects/Project.jsp?proj_Name=Minidoka+Project
[19] Water District One, 1985 Annual Report, Water District 1, page 69
[20] Water District One, 1988 Annual Report, Water District 1, page 1
[21] Ibid, page 72
[22] Water District One, 1992 Annual Report, Water District 1, page 1
[23] Ibid, pp 2, 80
[24] Ibid, page 78
[25] Idaho Department of Water Resources, Director's Order of April 19, 2005 in responding to a water call from the Surface Water Coalition, page 17
[26] Charles M. Brendecke, June 18, 2007 Affidavit filed before the Department of Water Resources, Blue Lakes and Clear Springs delivery calls, page 7
[27] http://www.idwr.idaho.gov/News/WaterCalls/ESPA_Agreement/default.htm
[28] Billingsley Creek Ranch, March 16, 2005 letter to Karl Dreher, Director, Idaho Water Resources Department at:
http://www.idwr.idaho.gov/Browse/News/WaterCalls/1000/archive_PDFfiles/billingsley%20creek%20call.pdf

spring users made calls.[29] Surface water users diverting from the lower reaches of the Upper Snake, also filed a delivery call on January 14, 2005.[30]

These numerous water calls resulted first in administrative hearings before the Director of the Idaho Water Resources Department or an appointed hearing officer. Multiple hearings have been held. The interface of ground and surface water, while essentially undisputed, is complex. Final orders addressing delivery calls require findings with respect to model accuracy and application, timing of impacts from pumping, and application of Idaho law. Once a matter has run its course through the administrative process, the Director's final order is typically appealed to the courts, first to the District Court in Twin Falls, which handles Idaho's Snake River Basin Adjudication, then to the Idaho Supreme Court. The Idaho Supreme Court's first rulings on specific call related findings by the director are anticipated in the spring of 2011. It is anticipated that the process will go on for several additional years before final certainty is achieved.

One of the outcomes from the water calls is mitigation obligations of ground water pumpers. With respect to calls from the Thousand Springs area, mitigation to date has been associated with buy out of spring users, voluntary curtailment of ground water pumping, conversions from groundwater to a surface water source of supply, and recharge.[31] Much of the water for recharge and conversions needs to be acquired from the rental pool. Mitigation associated with the Surface Water Coalition call has largely relied on the acquisition of stored water that can be made available to impacted Surface Water Coalition members. The first mitigation obligations arose pursuant to orders issued by the Director in 2005. The courts have weighed in to require the Director to assure that ground water users secure mitigation water early in the season for use if needed by Surface Water Coalition members. As a result, the Interim Director required ground water users to secure 84,300 acre-feet as a contingency to meet potential shortages in 2010.[32] That volume was based on low snowpack conditions and predicted resulting shortages to Surface Water Coalition members. Conditions improved dramatically due to heavy spring precipitation, and the water was ultimately not needed.

To summarize, since inception of the Upper Snake Rental Pool in 1979, new demands for rental pool supplies have arisen from:

1. Bureau of Reclamation flow augmentation. Reclamation reacquired about 23,000 acre-feet of storage in the Upper Snake from willing sellers, and relies on the Rental

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[29] The Thousand Springs area calls are extensively documented at the Idaho Department of Water Resources web page:
http://www.idwr.idaho.gov/News/WaterCalls/1000Spring%20Users%20Calls/default.htm#AD

[30] Surface Water Coalition Letter dated January 14, 2005 to Karl Dreher, Director, Idaho Department of Water Resources. The Surface Water Coalition calls are also extensively documented. See:
http://www.idwr.idaho.gov/News/WaterCalls/Surface%20Coalition%20Call/default.htm#Admin

[31] See the Interim Director's Order of July 19, 2010 at:
http://www.idwr.idaho.gov/News/MitigationPlan/ESPA/PDF/20100719_Final-Order.pdf

[32] Idaho Department of Water Resources, Order Regarding April 2010 Forecast Supply, April 19, 2010, pp 2-3

Pool for significant supplies. In recent years 150,000 to 180,000 acre-feet of water has been rented.

2. Mitigation for ground water pumping. While the volume required in 2010[33] was the largest since the 2005 water calls, it is not a worst case scenario. The drought expected in April 2010 followed a near normal 2009. In the second sequential year of drought the acre-feet required for mitigation could be in the hundreds of thousands.

3. The Idaho Power Company. The Company rented water in the decade of the 1980s but dropped from participation in the 1990s. In recent years the Company has sought to rent water, but no rentals through the rental pool have been achieved.[34] Water users know the Company has relatively deep pockets and some would like to rent water to the Company at attractive rates.

The demands for storage have required hard work by Rental Pool managers. It became apparent soon after the water calls were issued that without extraordinary efforts, the Upper Snake Rental Pool would cease to function and among other things, the water users' commitments to provide flow augmentation that were articulated in the Nez Perce Settlement would be unfulfilled.

The rental pool has faced several challenging questions:

1. What about impacts to other spaceholders? Storage is not equal since some reservoir space is essentially guaranteed to fill while other space may not fill for several years in a row.[35] Spaceholders with junior space feared that rentals by senior spaceholders could impact the storage available to them in future years.

2. In light of potential impacts to non-participating spaceholders, should spaceholders' ability to rent water be limited? Spaceholders with senior storage believed that once stored, the owner has the unfettered right to market it. Junior spaceholders held fast to the concept that the reservoir was a system to benefit all spaceholders, and that while a user had the ability to use as much stored water as needed to irrigate his crop, the right did not extend to renting water to others, because rentals made one year reduced system carryover and made the reservoir system more difficult to fill.

3. How would sufficient water be made available to meet the water users' commitments to provide water for flow augmentation? Under the rental pool procedures that applied prior to implementation of the Nez Perce water rights settlement it was necessary for willing lessors to formally consign water to the rental pool. During the

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[33] 84,300 acre-feet associated with the Surface Water Coalition call plus an additional volume, probably about 20,000 acre-feet, for conversions and recharge to mitigate for spring users' calls

[34] The Company did rent 45,716 acre-feet of water stored in American Falls Reservoir from the Shoshone Bannock Tribes. See the Company's October 29, 2009 filing with the U.S. Securities and Exchange Commission located at http://www.faqs.org/sec-filings/091029/IDAHO-POWER-CO_10-Q/, pp 28 and 51

[35] The probability of refill relates to the storage priority date for the space in question and the location of the reservoir. For example, American Falls Reservoir on the Snake River near the bottom of the system having a 1921 priority will fill before Ririe Reservoir upstream on Willow Creek with a 1969 priority.

drought many spaceholders suffered from water shortages. It was apparent that spaceholders were adopting a more conservative approach to making water available for rental. The potential existed that every time a spaceholder was negatively impacted from flow augmentation rentals, it would decline to make water available for an extended time into the future. That situation could result in an ever declining pool of suppliers and was considered to be a serious problem.

4. What about price? As demand has risen in recent years, the value of water has increased significantly. There are anecdotal reports that price has increased tenfold within the last two decades. Spaceholders with water to rent thought the price should be high to properly recognize the value of water. Those anticipating they might rent water thought the price should be low, to spread the benefits of the storage system as widely as possible.

5. How would new demands for water to mitigate for ground water pumping[36] be met in light of bright memories about insufficient water supply during the recent drought and questions whether the drought was indeed over?[37]

The challenges were addressed as follows:[38]

1. Reclamation and the watermaster of Water District One collaborated to develop a chart that defined how much water would be made available for rental for flow augmentation under specified conditions. The factors are November 1 reservoir carryover for the prior year and the April 1 forecast for April-September flows at Heise. In years where the combined carryover and forecast are low, no water would be made available and Reclamation would need to rely on water stored in powerhead space to attempt to provide at least 427,000 acre-feet. This partially resolved water users' concerns by assuring them that Reclamation would not demand water when there wasn't enough to meet irrigation demand. It was also in keeping with the longstanding principle that water would be provided only from willing sellers (lessors). Under the chart up to 205,000 acre-feet could be provided in the best conditions. In addition, 55,000 acre-feet would be available every year for spaceholders (50,000 acre-feet) and "small users," (5,000 acre-feet) who individually

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[36] Probably few spaceholders consider the need for water to mitigate for ground water pumping to be an *obligation*, but many are willing to accommodate the need if it can be done without impacting storage spaceholders too severely.

[37] Mother Nature has not been as cooperative as she could be in recent years, as demonstrated by conditions in 2007 and 2010. In the winter of 2007 Reclamation was on target to completely fill the system reservoirs. Conditions turned very dry and the April 1 Heise forecast (April-July) was only 67% of average. The system failed to fill. Actual runoff for the period was a dismal 54% due to continued dry conditions. Water supply conditions in 2010 looked very poor and the April 1 Heise forecast was 54% of average. Spring precipitation was well above average and actual runoff was 73% of average. The system filled.

[38] The Upper Snake Rental Pool Procedures are available on the web at: http://www.idwr.idaho.gov/WaterManagement/WaterRights/waterSupply/PDFs/2010-RentalPool_WD1.pdf

demand no more than 100 acre-feet per year.[39] All water rented to Reclamation for flow augmentation plus the 55,000 acre-feet rented to spaceholders and small users was deemed to come from a common pool. Spaceholders were each given the opportunity to "opt out" of the rental pool. Essentially all spaceholders remained participants in the rental pool.

2. Payments to spaceholders are based on a formula that includes each spaceholders percentage of total system capacity (space, whether filled with water or not), and actual storage (water stored).

3. Seventy percent of rental pool payments, less Water District One administrative fees, are paid each year to participating spaceholders. The remaining 30% will be retained in an "impact fund."

4. Each spring or summer after the reservoir system has attained maximum storage the watermaster computes the storage available to each spaceholder. The new rental pool procedures call for a second accounting in years when the system fails to fill. The parallel accounting will compute the amount of storage that each spaceholder would have had absent rentals from the common pool the preceding year. Spaceholders impacted from prior year rentals are entitled to a payment. If the spaceholder's storage is insufficient to meet internal needs, the spaceholder may use the impact payment to rent stored water from the common pool. Impacted spaceholders have priority to rent water from the common pool. This procedure isn't perfect and it is hard to imagine how a perfect system could be structured, other than to eliminate all rentals. To the credit of the spaceholders, they accepted this approach as a good enough pragmatic approach.

5. A tiered pricing structure was developed for rentals from the common pool. In years when the main storage reservoirs fill, the price would be $6.30 per acre-foot to the irrigation lessee. In years when the main storage reservoirs fail to fill but water is provided to the Bureau of Reclamation for flow augmentation pursuant to the chart identified above, the price to irrigation renters would be $14.00. In years when no water for flow augmentation is provided pursuant to the chart, the price to irrigation renters would be $20.60.

6. Notwithstanding the flow augmentation values contained on the chart developed by Reclamation and the watermaster to govern flow augmentation leases, the Committee of Nine may elect to make more water available to the Bureau of Reclamation. In three recent years the water supply situation improved after the April 1 forecast and this provision was used.

7. Provision was made for private leases. This was necessary because with a vivid memory of previous drought conditions, the spaceholders couldn't see their way clear

[39] In administering water rights on the Snake River, it has frankly proven more efficient to rent water to small users rather than devote the resources necessary to make sure they stay strictly within their water rights. It can take as much time and effort by the watermaster to regulate a small user as a large canal.

to expand the common pool by 100,000 acre-feet or more to mitigate for ground water pumping. Private leases may be negotiated with any spaceholder at an agreed upon price. The space associated with the private lease is last to fill the next year, thereby assuring non-participating spaceholders that they are not impacted.

8. Recent revisions to the rental pool procedures have made a place for the Idaho Power Company. As a last priority after assuring that all other needs are met, Idaho Power has the option to rent any remaining water in the 50,000 acre-feet common pool designated for spaceholders. The price is subject to negotiation but may be as much as $35.00 per acre-foot, plus an infrastructure fee of $5.00. The infrastructure fee will go into an account designated for system improvements.

FINAL OBSERVATIONS

The changes in water management in the Payette Basin over the past 20 years are nothing short of remarkable. The watermasters and the water users are to be commended for their vision, persistence, and skills as they have used the Payette Rental Pool as a tool to modernize the water delivery infrastructure in the Payette Basin. Along the way they have met external needs and contained costs incurred by the water users.

In contemplating recent rental pool changes in the Upper Snake, the Upper Snake Rental Pool has proven amazingly resilient. However, it is very difficult to fully appreciate the work done by the Upper Snake Rental Pool in grappling with drought, water calls, and flow augmentation demands. These seemingly intractable water problems were “solved” by referring them one at a time to the Upper Snake Rental Pool—a group of Idaho farmers. These farmers are obviously smart and resourceful, but they are far from experts in the esoteric provisions of federal ESA law or state water rights administration. Having worked with these farmers in the trenches as they grappled with exceedingly difficult conflicts, the authors stand in admiration of their willingness to stay engaged, their attention to detail, their pragmatism, and their willingness to compromise.

WATER TRANSFERS IN CALIFORNIA: 20 YEARS OF PROGRESS, VIEW TO THE FUTURE

Steve Macaulay, P.E.[1]

ABSTRACT

Throughout the 1980's the California Legislature authored changes to California law that encouraged market-based water transfers as an alternative to development of new water supplies. At that time, and continuing to the present, projections in the California Water Plan were that many regions throughout the State would be short of water by 2020. The belief was that market reallocation of existing developed water supplies would reduce environmental impacts associated with water supplies and allow water to go to higher economic uses. Notwithstanding the new legislation, water supply conditions were not severe enough to trigger the need for water transfers until 1991, the fourth year of a prolonged drought.

This paper provides a background on water in California, and a summary of the practice of market-based water transfers in California with an emphasis on short-term transfers (defined as one year or less). Transfers begin with the 1991 State Emergency Drought Water Bank and continue to the present. Historical data and several case studies are provided for illustration. The paper addresses the future direction of this important water management tool for providing increased water supply reliability for both agricultural and urban water users. This includes several examples of long-term market-based water transfers that are either underway or being contemplated.

INTRODUCTION AND BACKGROUND

Physical and Legal Setting

California has a temperate Mediterranean climate and abundant water resources. It has a population of 38 million people, and more than 9 million acres of irrigated farmland. Roughly 2/3 of the population is in the southern part of the state, and 2/3 of the water resources in the northern part.

It rarely rains during summer months. Consequently large surface reservoirs have been developed to capture water during the wet months (November through April) for release in the dry months (May through October). California is fortunate to have natural "reservoirs" in the form of extensive groundwater basins and seasonal snow pack in the Sierra Nevada Mountains. All of these water resources contribute to a diverse, large water resources mix. Figure 1 is a map showing California's principal agricultural regions, river systems and water projects.

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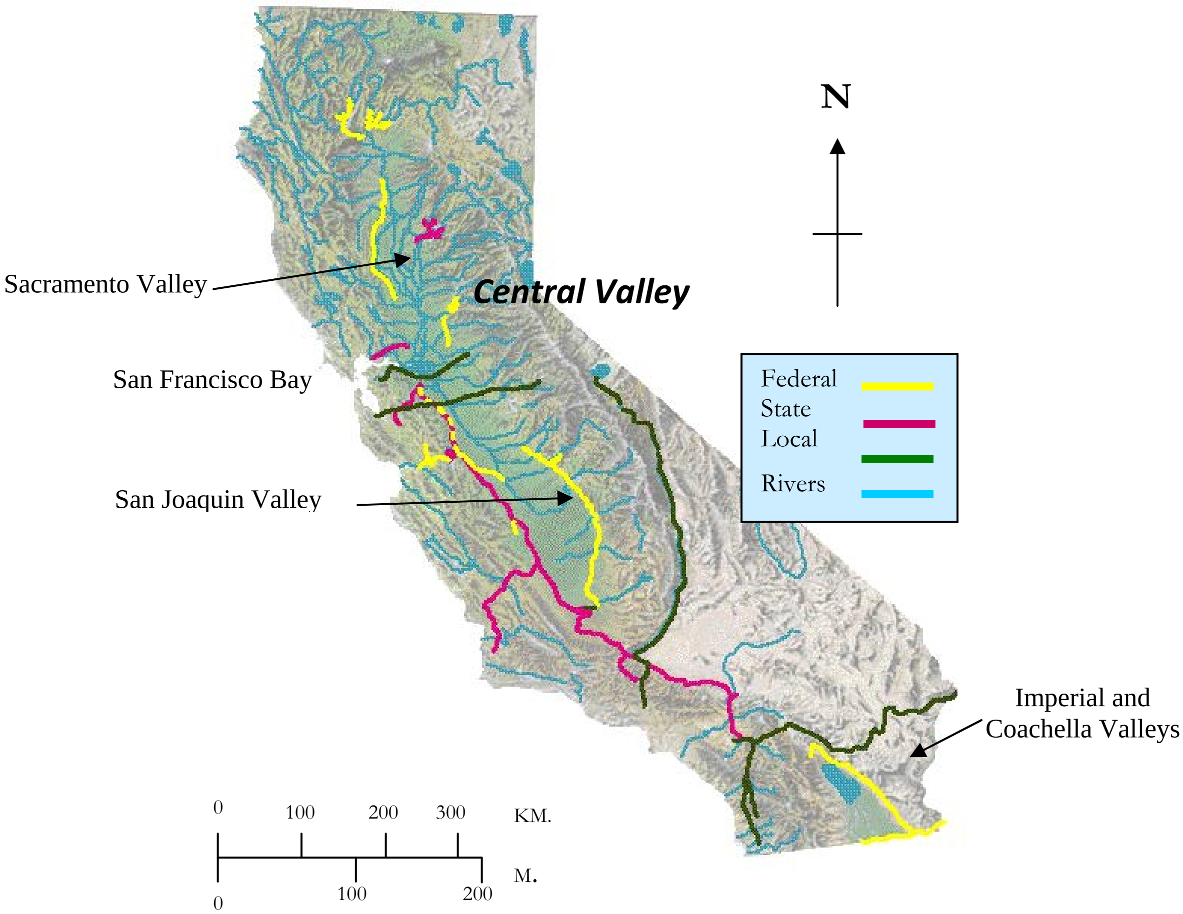

Figure 1. California River Systems and Water Projects, Principal Agricultural Areas

The Sacramento Valley is in the water-abundant north, and most of the region has ample water supplies in even the driest of years except for periods of extraordinary drought (e.g. 1977). There are more than 2 million acres of irrigated farmland. Return flows from irrigated agriculture are available for downstream users, and consequently water conservation measures are generally not considered for water supply reasons. A major crop is rice. The San Joaquin Valley has extensive productive farmland with a wide mix of crops. Agriculture benefits from local and imported surface water as well as extensive groundwater supplies.

Southern California is very dry, and most of the cities rely on water imported from Northern California and the Colorado River to the east. Cities near San Francisco Bay get their water supplies either directly from sources in the Central Valley or from reservoirs located in the Sierra Nevada on the east side of the Central Valley. Most water projects are designed to sustain a recurrence of the droughts of 1928 - 1934 and 1987 - 1992. Northern California has adequate water supplies in most years, while the large population in Southern California is very susceptible to drought.

Much of California's developed surface water supplies are in the Central Valley, drained by the Sacramento River in the north and the San Joaquin River in the south. These two rivers join in the Sacramento-San Joaquin Delta (Delta) and flow into San Francisco Bay. The Delta is a tidal estuary where salinity is maintained by releases of fresh water from upstream reservoirs.

Several major water projects store water in reservoirs on these rivers, and divert water from the Delta. Salinity concentrations are managed at levels sufficient for both urban and agricultural uses in the export areas, as well as local uses in the Delta itself.
Water is pumped from the Delta to other areas of California, including cities in the San Francisco Bay Area and Southern California. Water is also pumped to farms in the southern portion of the Central Valley. The region where these two rivers join is important not only to California's water supply, but also to the state's natural environment. The Delta is home to more than 750 species of plants and animals, including resident and anadromous fish. Conveyance of water through this artificially-managed tidal estuary was intended to be a short-term measure prior to development of a fresh water canal to divert water around the end of the Delta to the urban and agricultural water users to the south and west. Significant environmental conflicts, changing public values, and years of litigation have stalled a long-term solution to water conveyance through or around this estuary. As human demands for water increased over the past 20 years, as well as the understanding of environmental water needs (quality and quantity), there has been the continuing erosion in the reliability of water supplies for all purposes.

California surface water rights were initially riparian and self-enforced. Towards the end of the 19th century appropriative water rights were added to the institutional framework, and such rights have been administered by state government since 1914. As to groundwater, no rules have been developed other than the general rule that overlying landowners have a higher priority to use of underlying groundwater than water users located elsewhere. Overlying landowners have the right, similar to an ownership right, to withdraw groundwater for use on their property.

Major Water Supply Infrastructure

The U.S. Bureau of Reclamation's (USBR) Central Valley Project (CVP) has five large reservoirs and more than 500 miles of canals. The CVP has water supply contracts to deliver more than 8 million acre-feet per year of water to more than 100 water and irrigation districts in the Sacramento and San Joaquin Valleys. Most of the water is for irrigation, although it delivers some urban water supplies within the Sacramento and San Joaquin Valleys and the San Francisco area. More information on the CVP can be found at: http://dataweb.usbr.gov/html/cvp/html/. The Department of Water Resources' (DWR) State Water Project (SWP) consists of 28 reservoirs, 500 miles of aqueducts, and other facilities. Total contractual commitments are to deliver 4 million acre-feet of water per year to 29 public water agencies. The SWP delivers 75 percent of its water to cities throughout California and the remaining 25 percent to farms in the San Joaquin Valley. More information on the SWP and DWR can be found at: http://wwwdwr.water.ca.gov.

California has a substantial number of regional surface water projects, developed primarily by regional or local water utilities. Such agencies include the Metropolitan Water District of Southern California (one of the world's largest urban water utilities) with its diversions from the Colorado River, East Bay Municipal Utility District and its diversions from the Mokelumne River in northern California, and the San Francisco Public Utilities Commission with its diversions from the Tuolumne River.

While California has very extensive surface water resources and infrastructure, water users also rely on extensive groundwater withdrawals to meet demands. Much of the state's groundwater use is within the Central Valley. For much of the last 30-40 years, the average overdraft (pumping more groundwater than is naturally recharged) has approached 2 million acre-feet per year with much of that in the San Joaquin Valley portion of the Central Valley.

What is a Water Transfer, Early Driving Forces

So what exactly is a water transfer? Water transfers typically involve changes in the place, purpose and/or method of water use. California water transfers have typically been from agricultural use to urban use, although transfers to high-value agriculture are increasingly common and in recent years transfers have been developed from agricultural use to environmental uses. The term "water transfer" is used in this paper to refer to the more modern market-based water transfers, and not to the historic permanent transfer of water from one basin to another that was typically associated with large water projects and extensive infrastructure. Market-based water transfers were developed initially to meet emergency needs during drought conditions, as described below. They had a policy basis in laws passed by the California Legislature during the 1980s to encourage transfers and remove legal/institutional barriers. These laws had substantial support from key environmental groups as a means of reallocating existing developed water supplies as an alternative to building more surface storage facilities.

Both short-term and permanent water transfers have used the same market mechanisms: buying water from other water users. This kind of program requires three components: (1) the institutional support for buying and selling water; (2) adequate water infrastructure to transport the water from the buyer to the seller; and (3) the ability and willingness to solve problems affecting other parties that may result from the transfer. Implementation requires most of the elements of any commodity market, as well as a layer of public policy since water is considered a prominent public resource. No water transfer can be implemented without adequate infrastructure to move water from the seller to the buyer. California's successful programs have been possible due to more than a century of water development resulting in the water resources infrastructure pictured earlier in Figure 1.

STATE EMERGENCY DROUGHT WATER BANKS, 1991-1994

1991: A Large-Scale Experiment

Since the 1980s, California water law has allowed and encouraged water to be sold from one user to another, but there was very little practice of these market-based water transfers until 1991. In January 1991, California was in the middle of a severe drought, and for the first time implemented a large-scale water transfers program. The California Drought Water Bank acted as a central buyer for transfers. Sellers were primarily farmers in the Sacramento Valley growing relatively low-value crops. State government forced all transfers to go through the drought water bank, and effectively prevented transactions outside the drought water bank. This was done initially since there was no effective water market. The lack of institutional history and an emergency need to assure that public health and safety water needs would be satisfied, resulted in this centralized approach. This approach was taken for the drought years 1991, 1992, and 1994, but left open the possibility of different water market models for the future. Further, drought water banks were created for a duration of one year. This limited risks to sellers and buyers, and helped to emphasize that the centralized program was designed for emergency water supply conditions.

More than 150,000 acres of irrigated farmland was idled to support the program. Other farmers sold their surface water supplies and relied on greater withdrawals of ground water. Still other water districts sold surplus water from smaller regional surface water reservoirs. The Drought Water Bank was managed by DWR in cooperation with the U.S. Bureau of Reclamation, and was staffed by 100 people as an emergency effort (the author was manager of this program).

The Experiment Continues

The Drought Water Bank continued in 1992 and 1994, and provided substantial institutional experience for large-scale water transfers and marketing. The price for water in 1991 was as high as $125 per acre-foot, but dropped to less than half that amount in 1992 and 1994 as the water market gained more experience and water supplies were less severe. The $125 per acre-foot price was also based initially on crop idling, which was not employed in 1992 or 1994.

Water transfers came about in the early 1990s for several reasons. Foremost was the long hydrologic drought, with record low reservoir storage levels after four consecutive critically dry years. A second factor was the feeling throughout California that the drought was a water supply crisis, and historic regional water conflicts were set aside for the common good. A third factor was the appeal of the economic theory of market-based reallocation of water, which had been promoted by many environmental groups and academic researchers over the prior decade as an alternative to building new reservoirs. A fourth factor was the extraordinary cooperation of a large number of federal and state regulatory agencies, and their assurances to both sellers and buyers that they would support a water market through expedited regulatory approvals. A final factor was the high price for the water that the buyers were willing to pay, and a willingness on their part to pay in advance for the water rather than waiting until the water was delivered.

While the drought ended in 1995, water transfers continued to play a role in helping to meet periodic water shortages. A summary of interbasin water transfers from agricultural water users to urban water users for the period 1991 through 2001 is shown in Table 1. Note that these are market-based transfers between different hydrologic basins, and are in addition to substantial transfers that may have occurred within each basin as well as long-term transfers among basins due to facility development and/or long-term agreements. For example, this does not reflect the long-term transfer agreements between Imperial Irrigation District and both the Metropolitan Water District of Southern California and San Diego County Water Authority, or the water transfers and exchanges that occur frequently among water users on the Sacramento River. The amounts varied from year to year due to changes in hydrology, as well as storage and water delivery conditions for the primary sources of long-term supplies.

Table 1. Summary, Agricultural to Urban Transfers in California, 1991-2001
(Source: internal records, DWR, August 2001)

Year	Number of Transfers	Amounts, Acre-feet
1991	670	349,000
1992	300	31,000
1993	245	31,000
1994	360	25,000
1995	160	1,000
1996	260	1,000
1997	480	35,000
1998	200	2,000
1999	170	2,000
2000	170	2,000
2001	740	50,000
TOTAL	3,755	529,000

The sources of many of these transfers (but not all) are water users in the Sacramento Valley. In addition to the transfers shown in this table, there were substantial "Agriculture to Agriculture" transfers largely from low value to high value crops. In 1991 the source crops were primarily alfalfa, pasture and field corn. Due to changes in crop commodity prices and other factors, a substantial amount of rice was fallowed in 2001. Similar conditions returned in 2008 and 2009, driven by high international rice commodity prices.

This history reflects that California water users look to the Sacramento Valley as a source of market-based transfers to supplement their reduced dry year supplies, although there are water-short areas in northern California as well. A principle adopted by DWR in 2001 was that local needs should be met first before transfers from the Sacramento Valley proceed.

From 1995 through 2007, California experienced a number of normal and dry years. Following the completion of the 1994 Drought Water Bank, water transfers continued to occur through transactions directly arranged by sellers and buyers without centralized management.

Conditions changed dramatically in 2008, a second consecutive dry year during which reservoir levels began to drop to low levels. Water users throughout California began discussions regarding potential water transfers. It was clear early in the year that DWR would not organize and operate a drought water bank. Indications are that transfers were in the range of 125,000 to 250,000 acre-feet. Transaction costs were fairly uniform between sellers and buyers, at $175 per acre-foot. This was based largely on the value of water to the sellers, who would otherwise use the water to irrigate crops.

The lack of direct involvement from state government, coupled with a pending change in the leadership of the federal government (national elections were held in November 2008), resulted in poor coordination among state and federal water, fish and wildlife agencies. Many of these agencies have regulatory roles regarding water transfers, and there was no central point of coordination. This greatly increased transaction times and costs over what they had been in the 1990s. In addition, declining budgets for many of these regulatory agencies substantially reduced the staff available to review proposed water transfers. A final concern was the lack of centralized, uniform rules. Many potential sellers were frustrated over the lack of clear rules, and one of the largest water users in the Sacramento Valley – the Glenn Colusa Irrigation District – decided not to sell water during 2008 for this reason.

Reservoir water throughout California continued to drop in the summer and fall of 2008, bringing attention by the Governor to continuing drought conditions. In July 2008, the Governor declared a “drought emergency” and called for the creation of a drought water bank to match potential water sellers and buyers together to help meet critical water needs in the following year. This timing was very important since most sellers are farmers who typically harvest their crops in late summer, and begin planning in October for the following year. Conditions at this time were different than they were in 1991 when a “drought water bank” was first created. Based on experience with subsequent water market transactions by various urban and agricultural water utilities, the Governor did not require that all transactions go through a centralized program although a state Drought Water Bank was again developed. There was some confidence that many sellers and buyers would make their own transactions without direct intervention by state government. The 2009 Drought Water Bank was identified to meet three purposes: (1) develop and sustain a robust water transfers market; (2) provide water for critical health and safety needs; and (3) coordinate and facilitate compliance with regulatory (primary environmental) requirements.

Table 2 summarizes all transfers during 2009 that were known to DWR (personal communication, State Drought Coordinator Wendy Martin, August 24, 2009). DWR initially expected that most water transfers during 2009 would go through their process.

In fact, only about 15 percent of transfers went through the Drought Water Bank, although all transactions had to go through some form of DWR regulatory approval.

Table 2. 2009 Water Transfers
(Source: Personal Communication, State Drought Coordinator Wendy Martin, August 24, 2009)

Water Transfers	**Amount, Acre-feet**
Drought Water Bank	81,275
Private, north to north	80,640
Private, north to south	250,500
Private, south to south	200,000
TOTAL	612,415

These results require some explanation. The drought water banks of the early 1990s occurred during an era where there was little transaction experience. It was also a time when state government essentially forced all transfers to go through the centralized function of the drought water bank. Over time, sellers and buyers gained enough knowledge to negotiate and implement transactions themselves. Such direct transactions also provided opportunities to speed the transactions and reduce costs.

The "north to north" transactions were all within the Sacramento Valley, keeping the water supplies in the local region to assure that all needs would be met within the selling region. The "north to south" transfers were similar to the drought water bank – farmers in the Sacramento Valley selling to water users in the San Joaquin Valley and Southern California. These transfers did not go through the drought water bank, and most of this water went to urban Southern California.

2010 promised to be another intense water transfers year due to continuing drought conditions. However, abundant rains throughout the spring filled a number of larger reservoirs with the exception of the SWP's Oroville Reservoir. Consequently many short-term transfers in 2010 were limited to SWP water users. No data on quantities was readily available at the time this paper was prepared.

TRANSITION: EMERGENCY SUPPLIES TO LONG-TERM RELIABILITY

Trends in State Water Policy

A major change in modern California water resources planning came about with the 2005 Update to the California Water Plan, updated even further in the 2009 Update. The original California Water Plan in 1957 was a framework for water development, and contained large-scale plans for developing reservoirs, canals and pipelines throughout California to meet the needs of a growing population. State law requires the Plan to be updated every four to five years, and the subsequent updates have largely been refinements on the original water development plans with a principal focus on reservoirs and canals. The 2005 Update took a much different approach, recognizing that water needs can be met in many different ways. To a large extent this reflected actions already being taken at the local level since large water infrastructure was becoming increasingly difficult to implement due to environmental and cost considerations. The 2005 and 2009 Updates have a strong emphasis on integrated approaches to solving water resources problems, with a full range of water management tools needed to meet California's long-term agricultural, urban and environmental needs. One of the key water management tools identified was water transfers.

Other than development of large water infrastructure programs, the planning horizon for water and land resources historically has been relatively short – 20 years or less – and generally associated with financing time frames. This has changed in recent years, with increasing attention on long-term sustainability of water supplies to meet urban water needs. In 1990, state law was changed to require preparation of "Urban Water Management Plans" (UWMPs) by most water utilities, to be completed and updated every five years. Over time these plans have become increasingly important as a useful planning document as well as a potential target for litigation. This has focused the water community on the need to assure that such plans are as technically rigorous as possible. The trend is clear. Urban water utilities are increasingly being required to demonstrate that they have reliable water supplies well into the future. More and more urban water utilities are including water transfers in their mix of water management tools, in addition to more aggressive water conservation, wastewater recycling and distribution system improvements.

Long-Term Transfers, Water Transfer Lessons

Long-Term / Permanent Water Transfers. It was clear from the drought water bank transfers in 1991, 1992, and 1994 that this could be a new tool to aid in urban water supply reliability, since purchase of developed water supplies from agricultural water users was competitive in cost to development of additional water supplies. It was also clear from past experience that such purchases would be too expensive in the long term for agricultural water users except for high-valued crops. Consequently, long-term and/or permanent water transfers are typically from irrigation districts to cities.

Five examples of long-term and/or permanent water transfers are summarized in the paragraphs below. Locations of these individual programs are shown in Figure 2.

Figure 2. Location of Example Long-Term / Permanent Water Transfer Programs

1. **Metropolitan Water District of Southern California.** In the mid-1990s the Metropolitan Water District of Southern California (MWD) adopted a three-part strategy for transfers:

- Permanent or long-term transfers, providing water every year.
- An "options" agreement, where a seller would be given a small payment every year under the condition that MWD could exercise an option to purchase water in any year.
- Purchase water in only dry years on the market, often referred to by economists as the "spot market". Each of these fits well into a typical urban water agency's water supply portfolio.

A number of urban water agencies have adopted this general water purchase strategy. In the case of MWD, they have developed a long-term options agreement with the Palo Verde Irrigation District along the Colorado River in California. This agreement requires that MWD make an annual payment to the sellers for the right to purchase water in a dry year. In years when the option is exercised, the parties agree on a market purchase price. MWD has developed a number of other water transfers and exchanges throughout California in furtherance of their 3-part water transfers strategy.

2. **City of Tracy.** The City of Tracy (City) had increasing concerns regarding the reliability of their existing 10,000 acre-feet per year of surface water supply from the federal CVP and 9,000 acre-feet per year of groundwater, particularly during below normal and drought years. At the same time, this growing city near San Francisco Bay had increasing potable water demands. The City developed a comprehensive water supply strategy that included securing 10,000 acre-feet per year of supplemental surface water supply. Based on this strategy, the City identified potential entities with surplus supplies, and negotiated a long-term agreement with two local irrigation districts that had service area boundaries near, or overlapping with, the City's water service area. Due to timing and cash flow issues, the City paid for and acquired 7,500 acre-feet per year of water supply immediately, and subsequently negotiated a set price for the transfer of the remaining 2,500 acre-feet per year. Payment for this second amount of water was deferred for three years to allow time for cash reserves to buildup. This deferral also worked well with the timing of increased water demands from new development.

   These supplemental supplies significantly enhanced the reliability of the City's existing water supply, particularly during periods of reduced dry year surface water deliveries. The City's water supply strategy also identified other opportunities to diversify the City's portfolio of supply sources. The City participated in an entirely new treated surface water supply project (South San Joaquin Irrigation District Project) that receives its source water from a different, remote watershed not adjacent to the City. The City acquired an additional supply of up to 10,000 acre-feet per year, meeting their goal to further diversify its water sources. Consequently, together with other actions taken in the past few years, the City has two different treated water supply sources, groundwater supplies and recycled water supplies.

3. **Stockton East Water District.** This water district serves much of the urban area of the City of Stockton, in Northern California. It has negotiated two 5-year water transfer agreements with local irrigation districts as a trial program to see if the District wants to include transfers in their water supply portfolio. The combined purchases are about 11,000 acre-feet per year. These agreements allow Stockton East Water District to evaluate how the supplemental water supplies integrate into their water project operations, as well as determine if the institutional relationships will be good for a long-term period.

4. **State Water Project Contract Purchases.** As indicated earlier in the paper, the State Water Project delivers up to 4 million acre-feet per year of water to 29 public water agencies throughout California. In 1994, the water users agreed to allow purchases of contract supplies among the water users, principally from agricultural water use to urban water use. This is a form of market water transfers since the price is developed by both the seller and buyer, and is a transfer of contract water supplies. The limit agreed to by SWP water users was a maximum of 130,000 acre-feet per year, every year through 2035 (when all SWP water contracts are subject to re-negotiation with DWR). Prices for this water, as a one-time purchase cost, have been up to $6,000 per acre-foot in the past few years. This appears to have set a new market price for long-term urban water supplies, at least from this source.

5. **San Diego County Water Authority.** More than 25 years ago, MWD negotiated a long-term transfer of 100,000 acre-feet per year from the Imperial Irrigation District IID. The transfer was made possible through investments by MWD in more efficient irrigation within IID. This program developed water that could be transferred, since irrigation return flows from IID normally flow into an inland high-salinity lake (Salton Sea). About ten years ago, the San Diego County Water Authority began negotiations with IID to transfer an additional 100,000 acre-feet per year to the San Diego region of Southern California. While not without controversy, a long-term agreement has been reached that will provide for transfer of the water for the next 75 years.

In addition to those transfers summarized above and shown in Figure 2, a notable long-term water transfer was negotiated at the end of 2010 that may be an important precedent for the future. The cities of Davis and Woodland are located in the Sacramento Valley immediately west of Sacramento. Davis and Woodland have begun developing a surface water supply project to replace most of their historical local ground water supplies due to water quality concerns. In addition to acquiring a new state appropriative water right (approved on March 1, 2011), the cities need to acquire additional surface water supplies during summer months when water is no longer available for new appropriations in most years.

The two cities have negotiated an exchange with Conaway Ranch, an 18,000-acre nearby farm with rice as the predominant crop. The framework of the deal is outlined in a press release found on the website of the Woodland-Davis Clean Water Agency (Agency), the joint powers authority implementing the water project: http://www.wdcwa.com/detail/news/board_approves_agreements_to_purchase_water_rights_and_joint_intake_fa. The Agency has negotiated the right to acquire up to 10,000 acre-feet per year of additional surface water starting in 2016, when the new surface water facilities (water intake, treatment plant, pipelines) are scheduled to be completed. The agreement is complex and addresses issues of concern to all parties, including a shared, new water intake with state-of-the-art fish screens. The press release indicates that the deal includes ultimate transfer of a portion of the underlying water right from Conaway Ranch to the Agency.

As of March 2011, the abundant rainfall and snowpack will likely decrease the need for short-term water transfers this year. However, the increasing need for urban water supply reliability in California continues to increase pressures for long-term water transfers. Early this year the U.S. Bureau of Reclamation and a consortium of its water supply contractors in the San Joaquin Valley and San Francisco Bay Area had begun a process to pursue a ten-year water transfers program starting in 2012. Details of this proposed "Long-Term Water Transfer Program" as they become available can be found at this web site: http://www.usbr.gov/mp/cvp/ltwt/.

There are many more examples of successful water transfers in California, as well as examples of efforts that were not successful. The lessons learned to make transfers a useful tool for future water resource management activities are described below.

Lessons from California Water Transfers. The State Drought Water Banks in the early 1990s worked very well for a number of reasons. The extraordinary drought conditions over five to seven years were in the minds of the public, and regional political conflicts were set aside temporarily for the common good to meet critical health, safety and economic water needs. The feeling of a mutual crisis was widespread, and there was extraordinary institutional cooperation at all levels of government. This was also prior to severe environmental restrictions on water diversions out of the Delta, and prior to the severe depletion of endangered fish species populations. Finally, all transfers in California were forced to go through the drought water bank.

The 1991 Drought Water Bank was an extraordinary, large-scale water management experiment. As expected, there were a number of unintended consequences: some good, some bad. Critical water needs were met for urban water uses, irrigated agriculture and the environment. Unintended consequences included: (1) adverse impacts to groundwater levels in some regions selling water, resulting in some non-participants having increased groundwater pumping costs; (2) some unforeseen environmental problems; and (3) some negative economic impacts in regions selling water, resulting from the idling of agricultural lands. Strategies, rules and laws have been developed to avoid these problems in the future.

Water transfers have turned into a very important water resource tool for regional and local water agencies. The individual short case studies above show that urban water utilities are willing to invest in short-term and long-term transfers. For the most part, buyers and sellers can find each other and can implement transfers without additional institutional help – although normal institutional approvals will still be required. It is also clear that water transfers are important to sellers as an important source of revenue, particularly for investing in their own water systems. Indications from both buyers and sellers are that they would like to see a lesser centralized government role – this was already reflected in the results of 2009 water transfers. It is also clear that centralized regulatory control is important to assure protection of environmental resources that may be affected by water transfers, particularly any potential impacts to endangered fish and wildlife species. DWR has formed an Office of Water

Transfers to offer centralized technical advice and support to all parties wanting to pursue a transfer.

California is dealing with water supply and reliability shortages in dry years, and it is projected that there will be shortages in average years with a forecasted increase of 15 million people over the next 20 years. We have the further legal requirement, added in 2002, that construction of new housing subdivisions will require a certification that water supplies will be adequate to meet the additional water needs. There is very strong incentive to improve the reliability of our water supplies. The problems we are dealing with are similar in many respects to problems elsewhere. It is becoming increasingly clear that the pressures to increase urban water supply reliability are very great, and that market-based water transfers will be one of many tools to be considered in meeting future urban water demands. To some extent we may continue to see similar investments by farmers with high-value crops. California's water market is here to stay.

Finally, while not addressed specifically in this paper, more attention is being given to long-term water supply reliability of both surface and ground waters in all regions of the state. One concern is the long-term interrelationships between surface and ground water, and how that relates to water transfers that are based of the transfer of surface water by a farmer and a shift to groundwater use. Another concern (or opportunity) is the continued "banking" of surface water in groundwater basins to support future water transfers. There are increasing technical, environmental and institutional concerns in water transfer source areas, which become even more important for long-term transfers than for past short-term transfers. The past 20 years have brought market-based water transfers to the mainstream as an important water resources tool. The next 20 years are likely to see more advancement in the areas of technical knowledge and water management institutions.

California's major hydrologic regions are undertaking efforts to rely as much as possible on local and regional water supplies, recognizing that water imported from other regions is becoming increasingly problematic although it will remain an important part of the overall water supply mix. Due to the emergence of market-based water transfers, it is also clear that many regions will continue to depend on other regions during times of drought and other severe water shortage conditions, and to an increasing extent for adding to long-term urban water supply reliability.

THE ENERGY FARM

Joe E. Blankenship[1]

ABSTRACT

The 20th century saw explosive growth in the world's population, from 1.65 billion at the beginning of the century to over 6.0 billion at its end. Current projections are for the Earth to be home for 9.5 billion people by 2050. Today, as we prepare for the 22nd century, the overriding questions facing everyone on the planet are:

- How can we achieve economic growth that meets the needs of all of Earth's citizens?
- Can we economically provide the food and energy necessary for all of the citizens?
- And, can we preserve and improve the environment as we meet the first two objectives?

The 20th century was powered by fossil fuel sources; coal, petroleum and natural gas. Much of the 21st century will likely follow the same energy path, since, in the year 2010 we do not have the dedicated renewable natural resources, the ready and economic technologies, nor at the moment, the political will in the United States to rapidly transform the energy profile for the United States to be sustainable in the long term.

What America does have is the most productive farm system on Earth; the most educated farm and rural population and an entrepreneurial culture that promotes and rewards innovation. These factors are necessary for change and improvement of supply and efficiency in the use of renewable materials, energy and manpower. This paper will explore how the U. S. Farm infrastructure can accelerate the adoption of renewable fuels.

USING CURRENT RENEWABLE ENERGY TECHNOLOGIES: SOLAR, WIND AND LOW HEAD HYDRO GENERATION

A quick way for farm communities to lower their dependence on fossil fuels is to incorporate current renewable energy technologies. Finding ways to produce electricity with photovoltaic arrays, wind turbines or hydroelectric generation will speed the reduction in fossil fuels. This electricity can be for use on the farm or the farmer can use farmland to integrate these technologies into the terrain and geography to become another cash crop. These installations can be developed through land rental to outside developers or developed by the farmers themselves. Innovative structural designs and optimum placement can put these resources where there is minimal obstruction to tilling and harvesting and where shading is not a problem.

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Pole Mounted CPV with work around space

Utilizing existing farm structures for PV

Wind turbines and cattle grazing in Iowa

Drops at irrigation reservoirs or simple drop structures for energy dissipation can provide opportunities for energy recovery.

CONVERSION TECHNOLOGIES FOR GAS AND SOLID FUELS WITH ENERGY FARM SOURCES

Ethanol and biodiesel have been the primary focus for crops dedicated to producing fuels for transportation. However, both gas and solid fuels can be available from farm resources for heat and electricity generation. The "other" fuel sources have not gained attention because of lack of adequate and reliable supply and because of the abundance of cheap petroleum fuels for transportation and low cost coal and natural gas for electricity generation. New conversion technologies hold the promise of making these farm supplied forms of fuel readily available, with consistency of quality and quantity and at competitive prices, which the market demands.

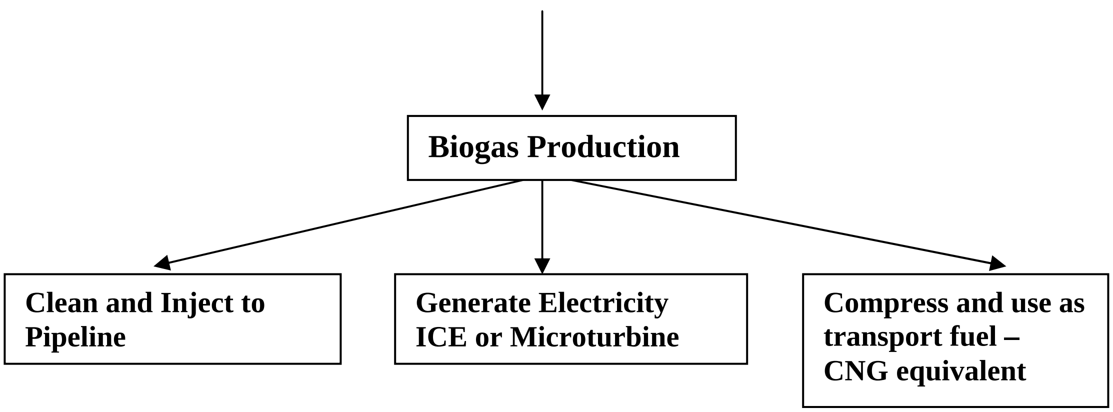
Anaerobic Reactors

Anaerobic reactors or “digesters” use a biological process to break down organic substances into biogases. Currently used mostly for conversion of animal wastes, (Dairies, cattle feed lots and swine growing operations), anaerobic reactors can also be used with other agricultural waste, particularly vegetable and grain crops that might otherwise be considered over production or poor quality and would ordinarily be plowed under or composted for a soil amendment. The digestion of animal manures can form a good base for inclusion of other source separated organic wastes to co-digest thus increasing the amount of gas produced from anaerobic reactors.

Anaerobic reactors are widely used in wastewater treatment to treat sludge before it goes to land application. In addition to the generation of a very useful gas, the process kills pathogens, removes odors, and generally provides a much more beneficial soil amendment than if the sludge is directly applied without the biological treatment.

The methane from an anaerobic reactor, when separated from other gases, has the same flexibility for use as natural gas. The highest value use of that gas will be dictated by local available markets and distance to connections to pipelines or transportation fueling stations.

Farm Based Anaerobic Reactor



Anaerobic digesters can also be supplied with energy crops grown for dedicated biogas production. In Germany and continental Europe these facilities are referred to as *biogas* *plants*. A *co-digestion* or *co-fermentation* plant is typically an agricultural anaerobic digester that accepts two or more input materials for simultaneous digestion.

While fuel supply is the primary objective, anaerobic digestion technologies also help to reduce the emission of greenhouse gasses in a number of key ways:

- Replacement of fossil fuels
- Reducing methane emission from landfills
- Displacing industrially-produced chemical fertilizers
- Reducing electrical grid transportation losses by having distributed generation

Digestate liquor can be used as a fertilizer, supplying vital nutrients to soils by replacing chemical fertilizers that are more energy intensive, require transportation and therefore produces more carbon dioxide. The solid, fibrous component of digestate can also be used as a soil conditioner. This solid digestate may be used to boost the organic content of soils. In countries where there are organically depleted soils, the markets for the digestate may be just as important as the biogas.

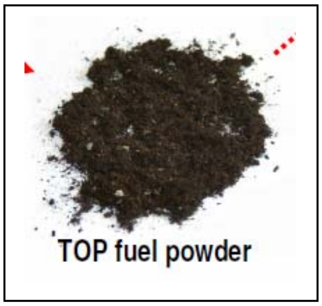
Anaerobes break down biomass material to varying degrees of success. Short chain hydrocarbons such as sugars are converted quickly while cellulose and hemicelluloses take much longer periods of time, making post digester processes more efficient. Anaerobic microorganisms are unable to break down long chain woody molecules such as lignin.

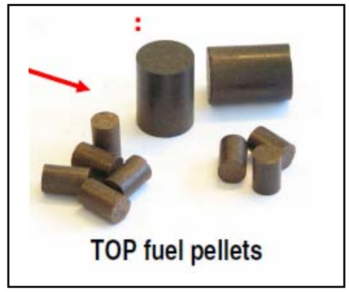
Anaerobic digesters were originally designed for operation using sewage sludge and manures. Sewage and manure may not be the material with the most potential for anaerobic digestion as the biodegradable material has already had some of the energy content taken out by the animal that produced it.

ANOTHER CONVERSION TECHNOLOGY — TORREFACTION

Torrefaction is an old process given a new application. Coffee roasting uses torrefaction to condition the beans to give them additional flavor but just as importantly, to make them easy to grind. Torrefaction is a thermal treatment in the relatively low temperature range of 225^0 to 300^0C using an oxygen free environment. The process produces an increased energy density, ease of grindability and makes different kinds of biomass more homogeneous for use in combustion or gasification operations. Torrefaction also can be used to produce biochar that can be used as a high grade soil conditioner.

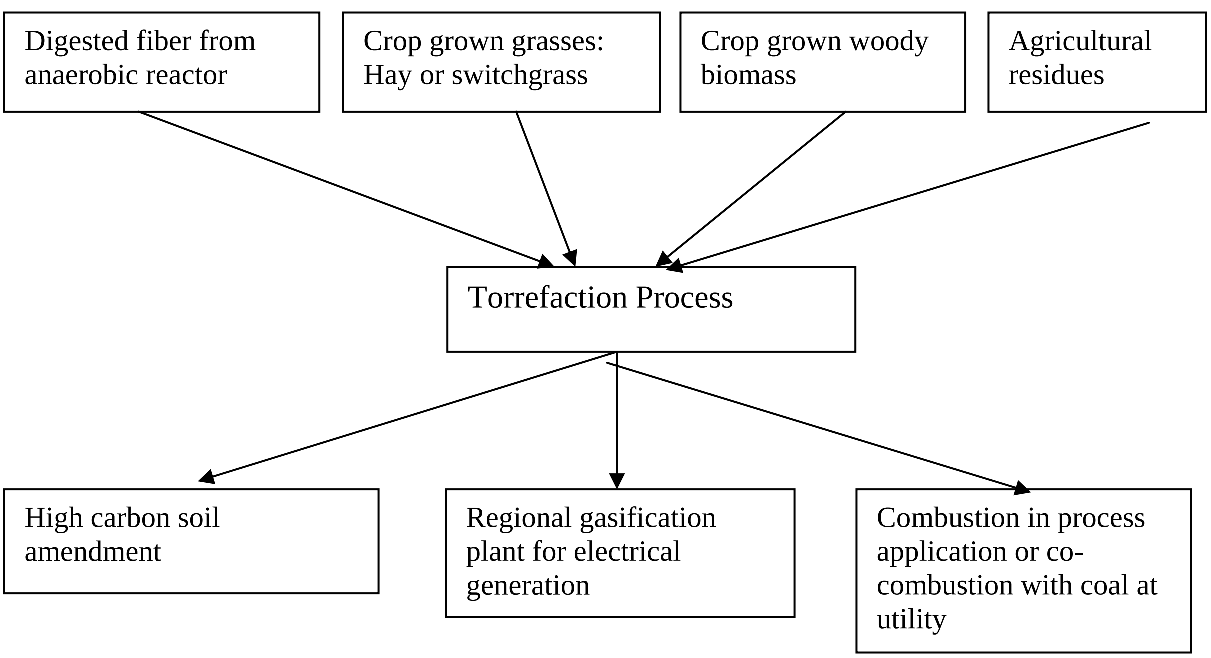
In the process of torrefaction the biomass will lose some energy value, generally about 10%, but becomes much more dense, losing between 20 -30% of its weight and volume. The resulting material does not absorb moisture as the original product does and is much more brittle, lowering cost for grinding or size reduction by as much as 90%. The product is a high carbon content fuel that is easy to transport, will not rot or retain moisture and can be pelletized for easy handling.





The heat content of torrefied fuel is comparable to a medium grade coal – i.e. 10,000 BTU/lb. The pellets can be combusted in a boiler for process heat, gasified to produce synthesis gas or shipped to utilities to co-combust with coal. The value of the torrefied fuel will mostly be determined by the value of the fuel that it is replacing, considering freight charges to the point of end use. The best strategy may be to have a number of smaller torrefaction plants supplying a larger gasification plant or coal utility where scale is achieved in the final conversion process.

One great benefit of the torrefaction process is the ability to input several types of materials and generate a relatively consistent product for use in gasification and/or combustion processes. There might be some variation in process time depending on the source materials and their size, but all of the output could be directed toward the same application.



CONVERSION PROCESS — GASIFICATION OF BIOMASS

Gasification is a high temperature pyrolysis process that converts all the organic matter in biomass to a low BTU gas, generally referred to as syn-gas. With a heat value of 300 – 500 BTU/SCF the gas cannot be taken to a pipeline, but can be used in internal combustion engines (ICE) or microturbines to generate electricity, or directly fired in boilers for process heat or steam. Biomass gasifiers are in wide use in Europe, particularly Northern Europe where there is plenty of woody biomass from forests and papermaking processes.

The ability to use gasification in certain processes is dictated by the temperature of the reaction. Very high temperature reactions create a clean gas that can be readily combusted in ICEs and microturbines. A lower temperature process will leave condensable tars in the gas that will tend to gum up an engine or turbine. This gas is good for direct combustion in a boiler for process heat or steam. The low temperature process is widely used in the United States for wood drying kilns at lumber operations.

Lower temperature reactions are easier and less costly to construct and control, but finding ways to prevent the condensation of tars in the subsequent use of the gas has been difficult to achieve. There are a few suppliers of equipment that have conquered this problem. However, the process is still not in wide application. The emphasis on renewable energy plus the availability of better, more consistent fuels and the desire to dispose of environmentally harmful organic wastes will help in making this process more generally used.

The photo below is of a demonstration plant that does gasification, gas cleaning and uses the gas in an internal combustion engine to generate electricity. This unit was fed with prepared and pelletized municipal solid waste (MSW) and successfully operated to generate at a capacity of one megawatt (1,000 kW).

Gasification of municipal solid waste to generate Electricity

BUILDING THE FUEL FARM INFRASTRUCTURE

Current agriculture infrastructure is built around farmers growing and selling food products, either for people or animals. In recent years a small portion of that infrastructure has been dedicated to supplying corn for ethanol. The production of ethanol from corn has been criticized for being too fuel intensive, i.e. questioning whether as much energy is used to grow, harvest, process and distribute ethanol as energy is available as a substitute for petroleum based gasoline. In addition to ethanol, some attention was given to crops directed toward biodiesel, but that effort never received the sustained tax subsidy or investment in conversion (refining) facilities to develop a meaningful contribution toward replacement or assured source of supply. The focus for biodiesel production has shifted toward algae grown in specialized industrial facilities rather than on farms.

To begin to further develop a reliable, sustainable and cost effective farm based source of fuel there is always the question: Which comes first, the fuel source or the conversion, distribution and market for the product grown? Obviously, the answer is that the two must grow in parallel, advancing reasonably close in supply, conversion and use.

For the farm economy, the best solution is to see the technology of conversion and market development done by the industrial and commercial sectors. Generally this means the investment in technology development, conversion facilities and distribution infrastructure are all done by venture capital so that the farmer can continue his (or her) traditional role of planting, harvesting and selling, but adding an additional source of revenue – crops for fuels. For the farmer to decide which crop to grow the conversion process must be proven and readily available and the general characteristics for a fuel source should be available.

A parallel development of fuel source and fuel conversion is possible and highly likely, using industrial and municipal waste streams as initial fuel sources and gradually adding farm grown supplies that meet the energy requirements of the conversion technologies and market need.

Why this route? The industrial and municipal sectors produce waste streams that can utilize technologies that are compatible with farm fuel sources and have reliable sources of supply. Incremental farm supplies may be added until the market is big enough to accommodate only farm supplied fuel sources. One example is anaerobic digestion of cattle manure with co-digestion of source separated organics; restaurant greases and food, grocery store spoilage, organic process waste streams, etc. The fuel products of this process are readily adapted to conversion to electricity or compressed natural gas (CNG) which can be used in transportation or added to the pipeline. For solid fuels, they must be processed into a form that is compatible with current fuel handling systems and meet user BTU specifications. To accomplish this requirement, new process technologies will have to be integrated into the supply chain. Torrefaction and gasification conversion technologies, discussed earlier, will be brought into general use.

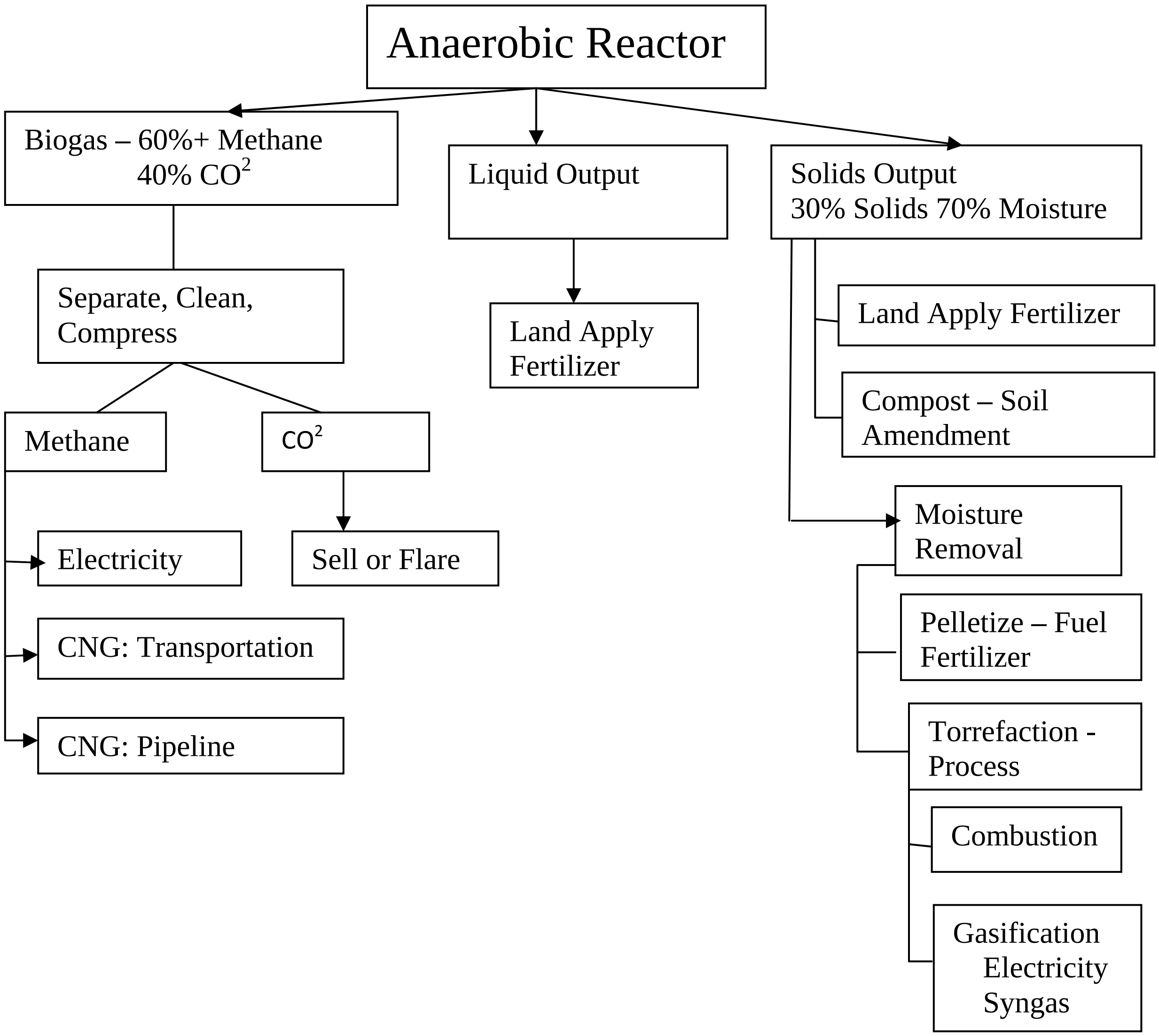
The following flow charts illustrate the integrated nature of a wet conversion process that produces both gaseous and solid fuels. Process refinements or equipment availability will improve in the following areas as the use of biomass fuels increases:

- Drying of biomass to make it acceptable for pelletizing, torrefaction or gasification.
- Commercial availability of torrefaction equipment.
- Biomass gasification and/or design for torrefied fuel.

POSSIBLE ANAEROBIC DIGESTER FLOW CHART

This chart illustrates the potential for revenue from several different by-products of the anaerobic digestion process. It does not reflect the potential of several different inputs for codigestion. Some of those inputs may allow for collecting a tipping fee for disposal.

By reducing the amount of methane, a potent Green House Gas (GHG), being released into the atmosphere, the operation may also qualify for carbon credits or electricity generated may be eligible for Renewable Energy Credits to meet a utility's Renewable Portfolio Standards.

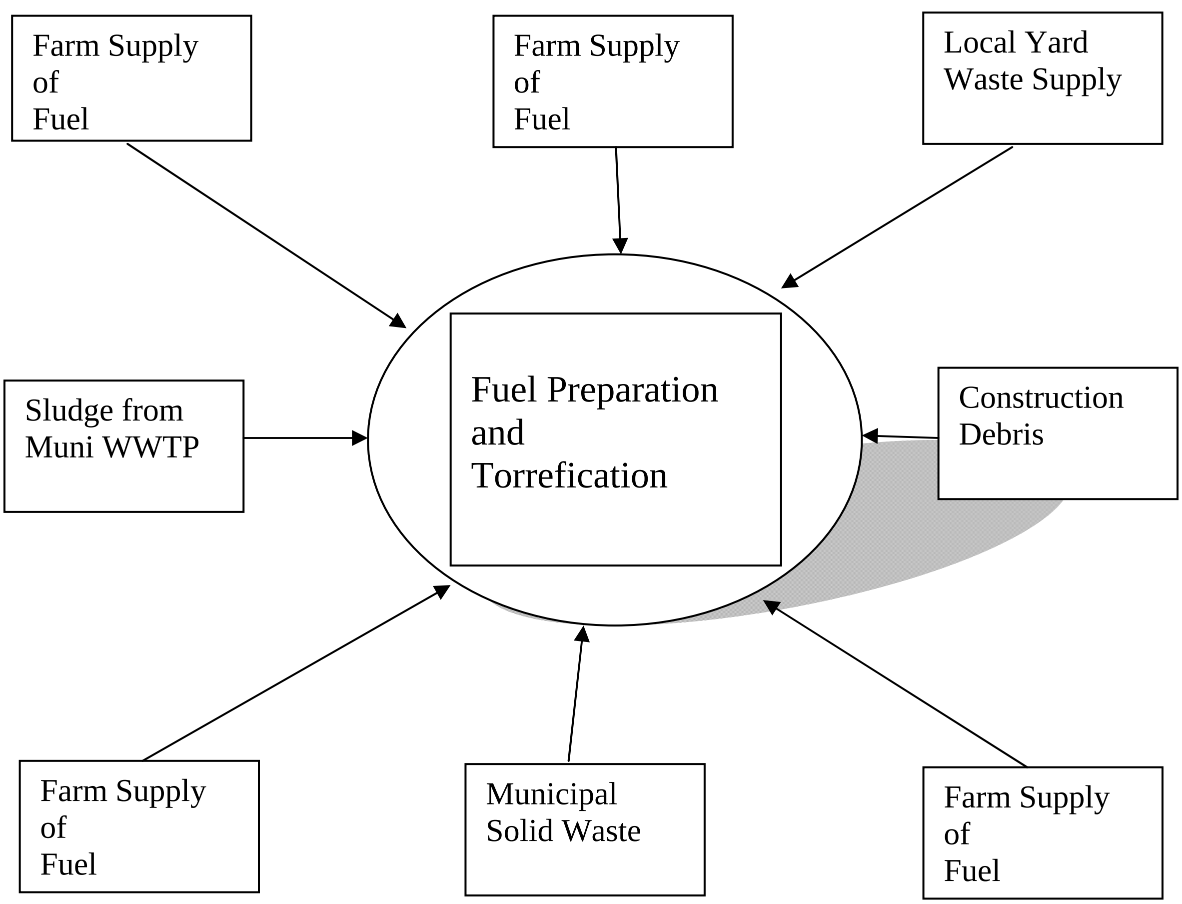


Anaerobic Reactor
Biogas – 60%+ Methane 40% CO2
Liquid Output
Solids Output 30% Solids 70% Moisture
Separate, Clean, Compress
Land Apply Fertilizer
Land Apply Fertilizer
Compost – Soil Amendment
Methane
CO2
Moisture Removal
Electricity
Sell or Flare
Pelletize – Fuel Fertilizer
CNG: Transportation
Torrefaction - Process
CNG: Pipeline
Combustion
Gasification Electricity Syngas

LOCALIZED FUEL PREPARATION AND TORREFACTION

It is generally recognized that biomass supply needs to be within 50 miles of a process plant to keep freight costs reasonable. Localized facilities that can receive, process and torrefy a variety of fuels from several sources will likely provide the best economic returns while diversity of supply increases reliability. The farmer can then maintain his flexibility to grow the crops that meet his soil conditions and equipment capabilities. Farmers may receive payment based on dry BTU content and fuel conditions such as moisture content and particle size. The torrefication plant may also receive tipping fees from industrial, construction and municipal wastes to provide revenue in addition to product sales.

<u>Multiple Farm Production and Supply — One Fuel Conversion Plant</u>



SCALE UP FOR GASIFICATION AND/OR COMBUSTION OF TORREFIED PRODUCT

Localized process plants can feed a centralized generation or combustion facility for further system efficiencies, bringing scale to bear in fuel handling and conversion, permitting and interconnection.

<u>Multiple Torrefication Plants Feeding Larger Generation Facilities</u>