The Agriculture/Urban Water Interface — Conflicts and Opportunities

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Preface

The papers included in these Proceedings were presented during the **USCID Water Management Conference**, held October 22-25, 2013, in Denver, Colorado. The Theme of the Conference was ***The Agriculture/Urban Water Interface — Conflicts and Opportunities.*** An accompanying book presents abstracts of each paper.

The last few decades have seen high rates of population growth across many western states and the world. For example, Colorado's population has grown approximately 51 percent in the last two decades, while Arizona's population increased by 73 percent. The Conference theme recognizes not only the effect this population growth has had on irrigated agriculture, but also investigates how urban and irrigation interests can work together to meet future water needs given limited supplies.

Historically, growth in water demand was met by the development of water resources projects and storage reservoirs. Now, the prospects for permitting and constructing new large storage projects and storage reservoirs are poor, leading to the current situation in which increased urban water supply is often in direct conflict with established agricultural uses. In addition, there is growing need for water in order to sustain and/or improve ecosystem services, recreation and tourism, and the ever-present threat of reduced supplies due to climate change.

The Conference served as a forum to discuss issues relating to the theme, as well as other issues relating to technology, water conservation, water quality and disaster mitigation.

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

USCID and the Conference Co-Chairmen express gratitude to the authors, session moderators and participants for their contributions. Thanks, also, to the Rocky Mountain Section, American Society of Agricultural and Biological Engineers, the Cooperating Organization for the Conference.

Gerald A. Gibbens
Berthoud, Colorado

Luis A. Garcia
Burlington, Vermont

Conference Co-Chairmen

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ITRC METERGATE CALIBRATION TESTING FOR FARM TURNOUT DELIVERY

Daniel J. Howes[1]
Ryan Fulton[2]

ABSTRACT

Accurate flow rate and volumetric information to farm deliveries from irrigation district conveyance systems is critical for good on-farm water management, irrigation district conveyance operation, and for billing and allocation purposes. In California, Senate Bill x7-7 requires irrigation districts to measure, within specified levels of accuracy, the flow and volume of water delivered to farmers. The most common turnout flow measurement device in California is the Armco-type metergate, which are calibrated based on discharge tables originally created by the Fresno Irrigation District and later by the USBR in the 1950's. A calibration facility has been constructed at the Cal Poly Irrigation Training & Research Center to recalibrate these gates under a variety of situations that were outside of the testing conditions used by the USBR but are commonly found in the field. Initial testing has been conducted to check the original calibration ratings under similar conditions as the USBR tests. In addition, tests have been conducted on other gate designs and on metergates constructed in situations outside of the original testing ranges (low upstream water levels, higher head differentials, with the gate oriented perpendicular to flow in the supply canal, etc.). This paper will discuss comparisons of the original gate ratings with current results, issues related to gate installation, operation, and the computation of opening area, and the calibration of square gates. Initial results on the influence of supply canal velocity on gate rating when the gates are oriented perpendicular to the flow will also be presented.

INTRODUCTION

Monitoring water consumption is crucial in California as water allocations for environmental, urban, industry, and agricultural needs are often at odds. For good on-farm water management, growers should have an accurate estimate of how much water they are applying. Recently, the state of California enacted Senate Bill x7-7 (SBx7-7), which provides regulations regarding measurement of irrigation deliveries from agricultural water suppliers. While many districts had already been estimating flow rates delivered, SBx7-7 mandates this for districts over a specific size in California. The bill requires a volumetric accuracy of ±12% for existing turnouts. For new or replacement measuring devices a volumetric accuracy of ±5% is required if being certified in a laboratory. If a new or replacement device is being certified in the field using a non-laboratory certification process, then a ±10% by volume accuracy is required (California Code of Regulations 2009).

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The vast majority of turnout flow measurement devices for California irrigation district turnouts are circular Armco-type metergates or calibrated orifices (ITRC 2002). Many irrigation district turnouts consist of a calibrated undershot gate (metergate) that opens into a circular pipeline, which travels under the district canal bank and delivers water to the farmer. The standard metergate has a hole in the top of the pipe, 12 inches downstream of the gate, which supplies water to a stilling well. Water level differential measurements are made by measuring the head difference in the downstream stilling well and the water level in the supply canal.

Accurate flow rate and volumetric monitoring is critical for district conveyance operations, billing, and water allocations. Canal operators must match the flow rate from the water source to the overall flow rate of all the operating turnout points to reduce canal spills and to provide excellent service to water users. District turnout measuring devices can help prevent growers from taking more than their allocated amount of water and can be used to verify that growers are billed appropriately for their water consumption. By documenting water use and the area of land each turnout serves, the district along with the growers can monitor the amount of water that is being applied to crops. In turn, fields with poor irrigation practices can be targeted and growers can take steps to improve irrigation management.

The Cal Poly Irrigation Training & Research Center (ITRC) is currently recalibrating gates under various conditions that were not covered in the past, but are commonly found in the field, including:

1) Initial testing to verify original rating tables
2) Gate designs, including Armco-type (circular) and square gates
3) The effects of water flowing in the supply canal perpendicular to the turnout
4) Location of stilling well downstream of the gate
5) Low upstream water levels, higher head differentials, etc.

This paper addresses the feasibility of using metergates as a method to satisfy SBx7-7 under standard installation conditions and non-standard conditions that are often found in the field but were outside the scope of past calibration studies. The current results of the ITRC testing will be discussed, as well as issues related to gate installation, operation, and the computation of opening area used to empirically calculate flow rate.

BACKGROUND

Metergate Discharge Tables

In the 1920's, Fresno Irrigation District (FID) developed discharge tables for Armco Model 101 metergates, ranging in diameter from 8 inches to 24 inches (Fresno Irrigation District 1928). FID found that metergates could be used for both flow control and flow measurement while maintaining a high degree of accuracy. In the 1950's the United States Bureau of Reclamation (USBR) partnered with Colorado A & M College to expand on FID's earlier study (Summers 1951). Included were metergates ranging in diameter from 8 inches to 48 inches (Ball 1961). The USBR found that even with the

more advanced testing equipment the discharge tables that were published earlier only needed slight adjustment (Armco Steel Corporation 1975). Both studies used Armco metergates with the gates positioned so that the flow in the supply canal approaches the gate straight on. While Armco is no longer manufacturing these gates, Fresno Valve and Casting (Selma, CA) purchased the patent/design from Armco and now manufactures these types of gates.

Flow Rate vs. Volumetric Accuracy

As mentioned earlier, SBx7-7 requires agricultural water suppliers to volumetrically measure and account for water being released to individual turnouts within a certain level of accuracy. The ITRC metergate study targets flow rate accuracy of orifices, which is related to volumetric accuracy. *Flow rate* refers to the rate at which water is delivered; common units are cubic feet per second (cfs) and gallons per minute (gpm). *Volumetric* readings refer to the total volume, or amount, of water delivered over a specified time period. Common units are acre-feet, or AF.

The maximum acceptable device flow rate error can be calculated using the following equation (Burt and Geer 2012):

$$Max.\,flow\ rate\ error = \sqrt{\left(1 - \frac{VA}{100}\right)^2 - ARD^2 - CBP^2 - CWLF^2} \qquad (1)$$

The maximum flow rate error depends on the desired volumetric accuracy, VA; accuracy of recording durations, ARD; changes in backpressure, CBP; and canal water level fluctuations, CWLF. In situations where there is a high level of water level control (i.e., long-crested weirs and flap gates) the effect of a changing water level is negligible. The desired volumetric accuracy is set in SBx7-7 depending on the certification process and if the gate is currently installed or if it is a new/replacement gate. As shown in equation (1), to satisfy accuracy requirements the maximum flow rate error must be less than the desired volumetric accuracy. A full explanation of this topic can be found in Burt and Geer (2012).

Orifice Flow Rate Equation

Flow rate through an orifice can be computed using the following equation:

$$Q = C_d A\sqrt{2g\Delta H} \qquad (2)$$

where Q is the flow rate, C_d is the discharge coefficient, A is the pipe opening area, ΔH is the head differential, and g is the gravitational constant. This equation is applicable to submerged gate conditions (water depths upstream and downstream are above the top of the pipe downstream of the gate). The head differential is computed as the difference between the upstream and downstream water levels.

Installation and Operation Guidelines

Figure 1 shows the recommended installation guidelines by the United States Bureau of Reclamation for circular metergates with corrugated pipe (Cadena and Magallanez 2005).

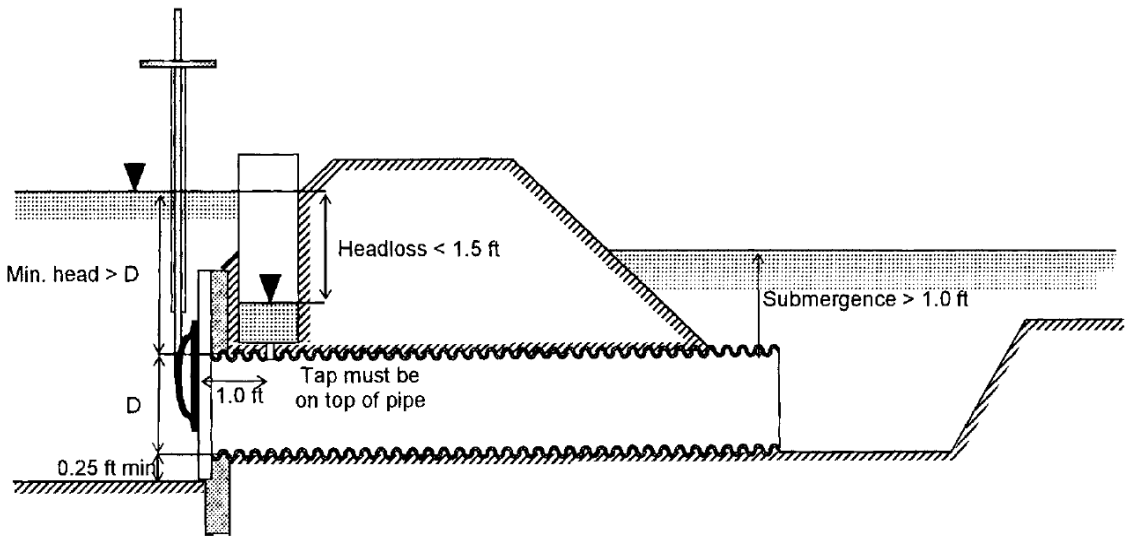

Figure 1. Suggested installation guidelines for Armco-type meter (Cadena and Magallanez 2005).

The installation guidelines by the USBR were used to calibrate and create the published rating tables. Often in the field these guidelines are not met, potentially decreasing the accuracy of the predicted flow rate. For example, standard conditions place the stilling well 1 foot downstream of the gate. Due to constraints like roads or safety concerns it is sometimes not feasible to install stilling wells at the specified location. Work is currently underway at the Cal Poly Irrigation Training & Research Center (ITRC) to explore what effects the downstream stilling well location has on determining flow rate by placing stilling wells at different distances up to 16 feet downstream of the gate, and comparing results. Factors that are being investigated include: stilling well location, position of gate with respect to supply canal, upstream submergence, varying supply canal flow rates, and differences in circular versus rectangular gates.

There are four inaccuracies that commonly occur when measuring flow rate through calibrated orifices: incorrect zero reference, incorrect downstream water level measurement, incorrect gate opening geometry, and non-standard entrance and exit conditions (Burt and Geer 2012). To calculate flow rate the distance the gate moved vertically from zero reference must be known. Zero reference is the point at which water begins to leak from the gate; a notch in the shaft should be made at this point. Having an accurate downstream water elevation reading is important. The stilling well access hole must be correctly sized and located on the top of the corrugated pipe to prevent plugging.

METHODOLOGY

The Cal Poly ITRC has constructed a turnout gate testing facility at the Cal Poly Water Resources Facility at California Polytechnic State University, San Luis Obispo. The gate testing facility is attached to the flow measurement flume on the site and is capable of handling up to 30 cfs. The testing facility allows for the gates to be positioned so that the supply flow is straight-on or perpendicular to the turnout gate. The flow rates through the gates are measured using magnetic meters (18-inch and 24-inch) that were calibrated with a weighing tank at the downstream end of the flume.

For the perpendicular flow testing the gate is attached to the side of the flume with corrugated metal pipe connecting the gate to the main spill box as shown in Figure 2. The downstream water level is maintained in the spill box by adjusting butterfly valves downstream of the magnetic meters, which are in pipelines connecting the spill box back to the drain. From the drain the water can be recirculated to the head of the flume through a variable frequency drive (VFD)-operated pump at the downstream end of the flume. Figure 3 shows photos of the operation testing facility.

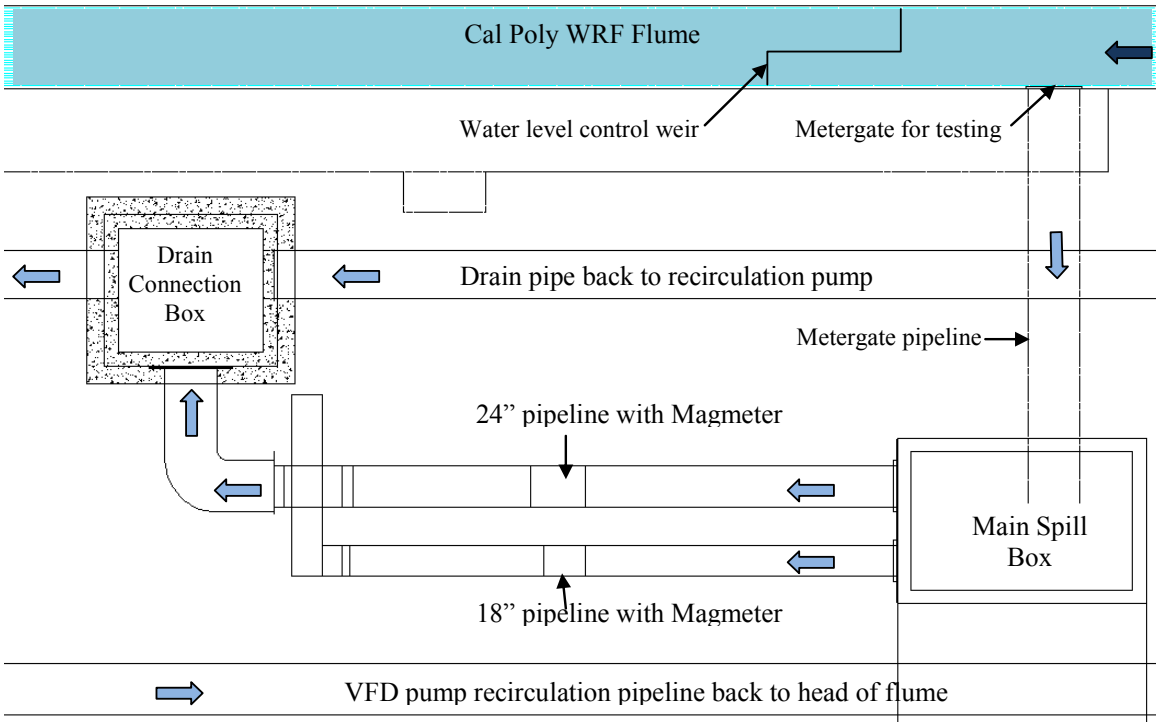

Figure 2. Top view of metergate testing facility at the ITRC facilities to test metergates with supply flows that are perpendicular to the metergate.

Figure 3. Photos of metergate testing facility and the Cal Poly Water Resources Facility.

Straight-on testing was conducted by placing the gate directly in the flume with the magnetic meter downstream of the gate. The water level was adjusted by changing the flow coming into the head of the flume through the VFD-operated pump. In the future, the side testing facility will be modified so that straight-on testing can be conducted using the new arrangement.

Gate Testing

Tests are being conducted over a variety of situations including variable gate openings, water level in the supply channel, head differential across the gates, and supply channel velocities. For the 18-inch and 24-inch gates, tests under each scenario are conducted at gate openings every 2 inches from wide open to closed. Three different supply channel velocities are examined for each testing scenario. The head differentials are examined at multiple levels depending on the upstream water level. The upstream water level is examined at 4 to 5 levels from a small head (~1/2 of a gate diameter) up to a very high head (2-3 times the gate diameter). The standard head above the top of a gate is one gate diameter. Currently, approximately 45 individual tests are conducted for each gate size, and 9 to 12 gate openings are examined for each test.

Calibrated magnetic meters are used to measure the flow rate passing through the gates. Head along the pipeline is measured at set distances of 6, 8, 12, 24, 48, 96, 144, and 192 inches downstream of the downstream metergate face. In addition, the head is measured upstream of the gate in the supply channel. The head is measured at each location by connecting flexible (1/2-inch) tubing from each location to 6-inch stilling wells.

Two methods are used to measure the head differential from the water level upstream of the gate to each of the downstream locations. Manual readings from staff gauges attached to the stilling wells are used as a check. The primary differential measurement uses a differential pressure transducer (SMAR LD301) connected directly to a personal computer. If the difference between the differential pressure transducer and the manual readings is different by more than 0.2 inch (less than a quarter of an inch), the test must be redone.

The flow rate in the main supply channel is measured through a 30-inch calibrated magnetic meter on the recirculation pipeline. This flow rate is used to compute the flow and the cross sectional velocity in the channel just upstream of the metergate. Again, three different velocities levels are examined: a low (approximately 2-3 cfs more than will be turned into the metergate), medium (~20 cfs), and high (>25 cfs). Depending on the upstream head the actual velocity will vary.

Opening Area Computation

Opening area equations are used when solving for flow rate empirically using Equation 2. When using the Armco discharge tables, having an accurate area computation is not important because the area term is already built into the tables. There is a different discharge table for each gate size and the flow rate is selected using the net gate opening. The opening area is crescent-shaped and depends on the gate diameter, pipe diameter, and vertical gate displacement. There are several different approaches to calculating opening area, but this research examined two methods: advanced mathematics ("ITRC Method") and a simplified approach used in a past study.

ITRC Method. The procedure below outlines the method developed by the ITRC to calculate opening area:

Variables (also shown in Figure 4):

- Offset is the distance from the center of the orifice to the center of gate [inches]
- Vertical Gate Displacement, y, is the vertical distance the gate is moved from "zero reference" [inches]
- Gate Radius, R_g, is the measured diameter of the gate divided by two [inches]
- Radius of Pipe, R_p, is the measured inside diameter of the pipe divided by two [inches]
- Point of Intersection, P, is the vertical distance from the center of the orifice to where the circles' edges intersect [inches]
- Net Opening Area is the area of the crescent-shaped orifice used in the orifice equation [inches^2]

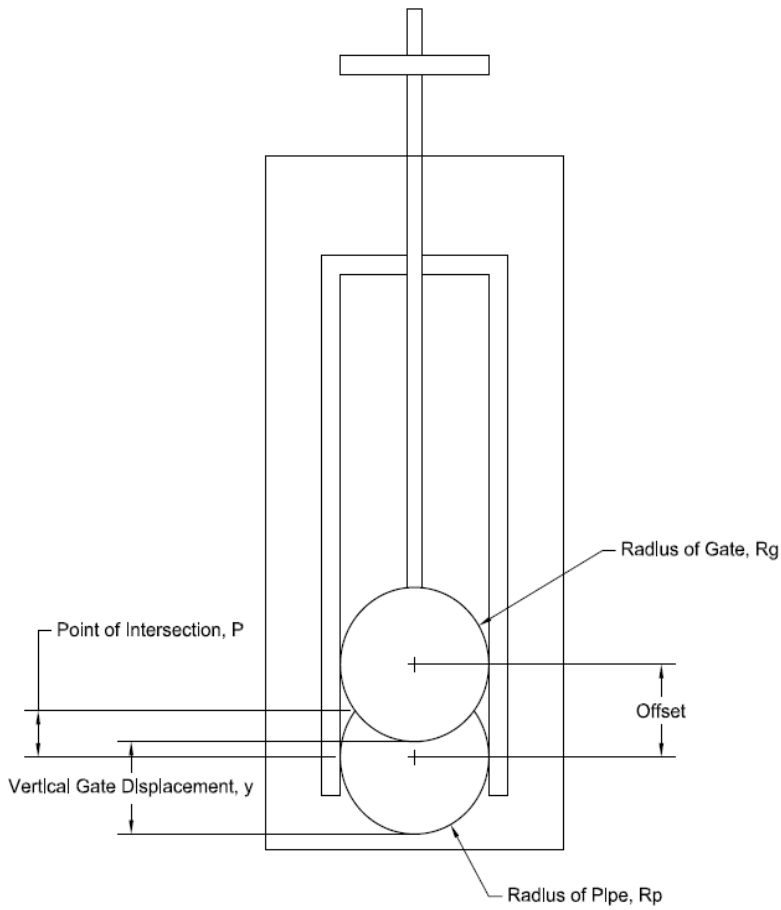

Figure 4. Metergate dimensions needed for ITRC opening area calculation.

$$Offset = y + R_g - R_p \tag{3}$$

$$P = \frac{R_g^2 - R_p^2 - Offset^2}{-2 * Offset} \tag{4}$$

$$Area\ Raw = \frac{\pi R_p^2}{2} + P * \sqrt{R_p^2 - P^2} + \sin^{-1}\left(\frac{P}{R_p}\right) * R_p^2 \tag{5}$$

$$\begin{aligned} Area\ Subtracted \\ &= \frac{\pi R_g^2}{2} + \sqrt{R_g^2 - P^2 + 2P(Offset) - Offset^2}\,(P - Offset) \\ &+ \sin^{-1}\left(\frac{P - Offset}{R_g}\right) R_g^2 \end{aligned} \tag{6}$$

$$Net\ Opening\ Area = Area\ Raw - Area\ Subtracted \tag{7}$$

If the gate is fully open, the opening area can be computed using the following equation:

$$Opening\ Area = \pi R_p^2 \tag{8}$$

Simplified Method. In a previous study on metergates, the following simplified opening area equation was used to calculate the area of the crescent-shaped orifice (Cadena and Magallanez 2005):

$$Opening\ Area = 0.95 * D * y \tag{9}$$

where D is the inside pipe diameter in inches, and y is the vertical gate displacement in inches. This simplified method, as noted by the authors, is intended to provide a rough estimate of gate openings. As will be shown, the rough estimate approximates the area well in the mid-openings but can be off significantly at lower and higher openings.

RESULTS AND DISCUSSION

Comparison of Opening Area Equations

Figure 5 shows the percent error comparing the ITRC Method's opening area to the simplified equation used by Cadena and Magallanez (2005). An 18-inch metergate and a 24-inch metergate were analyzed at various gate positions. Percent error is computed as:

$$\text{Percent Error} = \frac{\text{Simplified Method} - \text{ITRC Method}}{\text{ITRC Method}} \times 100 \tag{10}$$

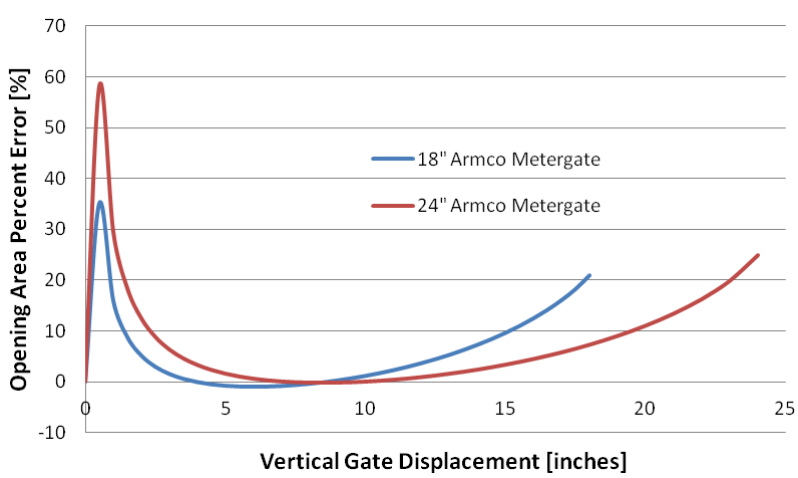

Figure 5. Percent error comparing the calculus-based approach and the simplified method to solve for opening area.

The different opening area computations delivered similar results (within ±5%) when both gates were opened 15 to 65 percent of full vertical gate displacement. For example, an 18-inch gate displaced vertically 9 inches from "zero" reference would be opened 50% of full vertical displacement. Therefore, the opening area accuracy would fall within ±5% of the true value. The inaccuracy associated with the opening area can potentially have a direct influence on flow rate accuracy. If a 24-inch metergate is opened 2 inches the percent error associated with using the simplified opening area is 12.4%. Using the same inputs to compute flow rate with Equation 2, only varying the area, the flow rate would be overestimated by 12.4%.

The method presented by Cadena and Magallanez (2005) used flow rate data collected from the USBR metergate study (Summers 1951; USBR 1953). No new measurements were made by Cadena and Magallanez; they relied solely on the data from the USBR. The 2005 study showed that the method is able to predict flow rates within ±6% with 95% confidence while using Equation 9 (the simplified area computation). The error was lower than the potential error due to the fact that the Cd values computed in the Cadena and Magallanez method are a function of the area. Therefore, the Cd values incorporate and account for the error. The difficulty with this method is that evaluating factors that influence the Cd values becomes very difficult. In addition, the Cd values are only valid for gates with exactly the same design as those used in the USBR metergate study.

Original Armco Rating Tables and Straight-On Testing

The flow rate error curves, shown in Figure 6, suggest the Armco rating tables are consistent with the ITRC's data collection for the "straight-on" condition under standard installation and recommended operation (e.g. downstream head measurements 12 inches downstream of the gate face).

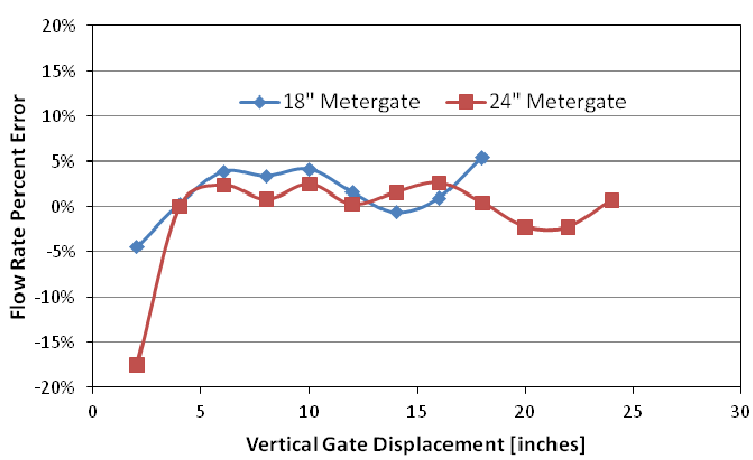

Figure 6. Percent error comparison between Armco discharge tables and ITRC flow tests.

The values in the "head" column on the Armco discharge tables range from 1 inch to 18 inches with increments of ¼ to 1 inch. To determine the Armco flow rate given the ITRC's experimental flow data, flow rates were generally interpolated from Armco tables. In a few cases where the conditions were outside of those shown in the Armco

tables, extrapolation was used. In one case an outlier was found that could have been attributed to either the extrapolation or measurement error. In Figure 4, for the 24-inch metergate an outlier flow rate error occurred when the head differential exceeded the 18 inch maximum found on the Armco discharge tables when the vertical gate displacement was at 2 inches. For the data shown in Figure 6, the average percent error is 3% with a standard deviation of 3.7%. Neglecting the outlier, the average error is 2.1% with a standard deviation of 1.6%, which is consistent with the results from past metergate studies (USBR 1953; Cadena and Magallanez 2005).

Figure 7 shows a flow comparison for standard and non-standard conditions. Non-standard conditions include gate submergence and head differential being less or greater than that recommended by the USBR (1997). For this analysis the downstream head readings were taken at the standard stilling well location that is 12 inches (1 foot) downstream of the downstream face of the gate (Figure 1).

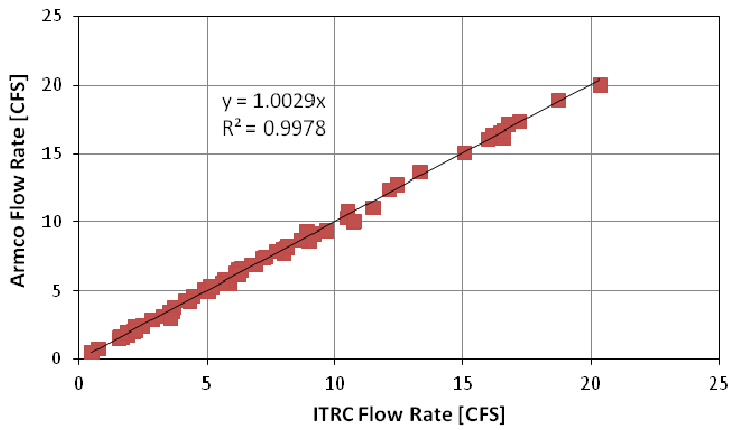

Figure 7. Results for standard and non-standard installations with the supply flow straight-on in relation to the gate. Downstream head measurements taken at the standard 12-inch location.

Even combining non-standard conditions, the original rating flow rates from the original rating tables showed a good relationship with the measurements made in this study. The degree of "non-standard" conditions were limited due to gate orientation in this testing. Additional testing to increase the severity of the conditions to match common field installations is addressed when the gate is perpendicular to the supply canal.

Turnout Perpendicular to Supply Canal

District turnouts are often located on the side of a canal, where the supply flow is perpendicular to the turnout flow. One of the major objectives of this study was to investigate the effects of this perpendicular flow on the gate calibration. Other studies have found that the velocity in the supply channel will influence the hydraulics surrounding how the water enters the gate (Hussain et al. 2010; Hussain et al. 2011). Preliminary results indicate that Armco discharge tables are not as accurate when the metergate is positioned perpendicular to the canal, as shown in Figure 8. The level of

error will depend on the cross sectional velocity in the supply canal. This relationship will be constructed through future tests.

Figure 8 shows a comparison of the same gate size and scenario for straight-on and perpendicular flow tests. A slope of 1.0 would indicate that the Armco flow rates and the ITRC flow rates are equivalent (perfect accuracy). The slopes for the straight-on and perpendicular gate positions are 1.03 and 1.13, respectively. The higher slope seen by the perpendicular gate indicates that the change in orientation decreases accuracies.

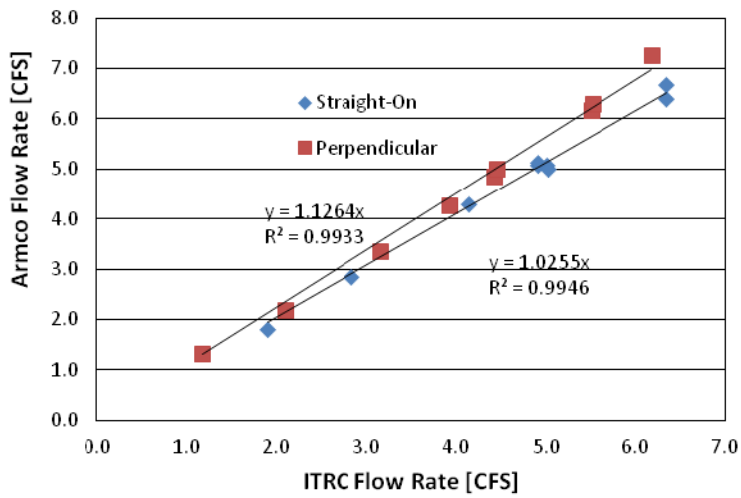

Figure 8. Flow rate results comparing straight-on and perpendicular gate positions relative to supply canal.

Figure 9 shows the variability in accuracy at different gate openings. On average, the Armco discharge tables overestimate the flow rate by 11% when the gate is positioned perpendicular to the supply canal. The percent error increased to a maximum error of approximately 17.6% as the vertical gate displacement increased. To satisfy the volumetric accuracy criteria set in SBx7-7, discharge rating tables will need adjustment. Further testing will be done on different gate sizes and shapes.

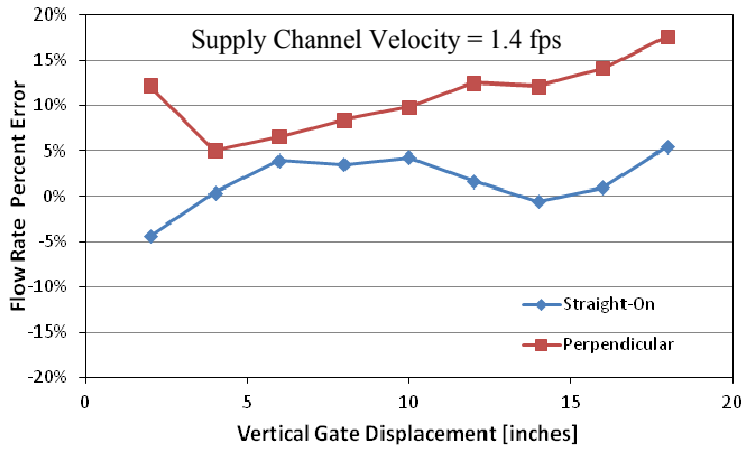

Figure 9. Flow rate accuracy for the Armco discharge table when the gate is positioned straight-on and perpendicular to the supply canal. An 18-inch metergate was evaluated.

Stilling Well Location

Downstream head measurements were recorded at eight different locations, ranging from 6 to 192 inches behind the gate. Due to hydraulic conditions within the corrugated pipeline the head measurements will vary, which in return will decrease the accuracy of the flow rates obtained using the original Armco rating tables. This is illustrated in Figure 10.

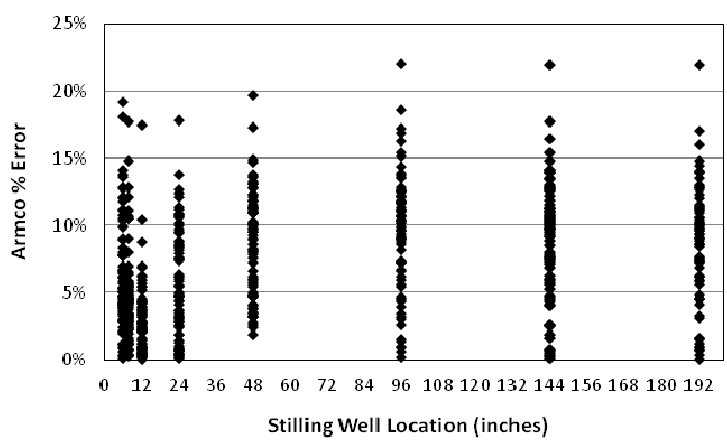

Figure 10. Percent error in rated gate measurement using head differentials taken at different downstream stilling well locations.

The lowest percent errors are seen at the stilling well located 12 inches behind the gate (standard location). This is to be expected since the Armco tables were created based on this location. There seems to be an outlier, but in general the error is less than 10% at this standard location. Flow rate accuracy decreases as the downstream head is measured at different locations downstream of the gate. This would result in significant error if the stilling well location is somewhere other than 12 inches downstream of the gate. Creating

new rating tables with a built-in adjustment factor is a potential option to obtain higher accuracies at varying stilling well locations. Further testing is in progress.

CONCLUSION

The incorrect opening area computation can significantly decrease flow rate accuracy by as much as 58% depending on gate size and displacement when using the orifice flow rate equation. This is not a problem with the existing metergate rating table since the table provides a direct reading of flow rate. However, it does create issues when trying to develop a mathematical relationship between flow rate, gate opening and head differential. A new, more advanced method of computing opening area has been presented.

The ITRC testing results were consistent with the current rating tables for standard gate conditions (e.g., straight-on, downstream stilling well 1 foot from gate, standard bulkhead) with straight-on testing. The test results verify that the original work by Fresno ID and the USBR was accurate, and also that the testing facility at the ITRC is functioning correctly.

After switching the turnout perpendicular to the supply canal, keeping all other factors the same, the flow rate accuracy significantly decreased. The average flow rate percent error jumped from 3% to 11% with common upstream velocities (~1.4 fps). In the future, different supply channel velocities will be examined as well as different entrance conditions (i.e., entrance conditions that have been set back from the channel).

Due to physical constraints in the field, downstream stilling wells may not be installed at the standard 12-inch location behind the gate. This can significantly affect flow rate accuracy. The ITRC is currently researching methods to adjust flow rates to account for the change in stilling well location. This may mean developing new ratings tables with a built-in adjustment factor for each downstream stilling well location.

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The authors and the ITRC would like to thank Fresno Valve and Casting for donating the gates that were used for testing. The authors would also like to acknowledge Evan Geer, former graduate student and employee of the ITRC, for his work in developing the gate opening area equation for the Armco type metergate.

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USING WINGATE TO DEVELOP PRACTICAL DISCHARGE EQUATIONS FOR GATED OUTLETS AT YELLOWTAIL AFTERBAY DAM

Tony L. Wahl[1]

ABSTRACT

Three gated outlet systems at Yellowtail Afterbay Dam were analyzed to develop discharge curves and equations needed for an automated gate control system that will regulate flow rates in the Bighorn River and the Bighorn Canal. Radial gates mounted on the dam's ogee crest spillway were analyzed using established empirical methods, while vertical slide gates in the river sluiceway and canal sluiceway systems were analyzed using the WinGate computer program. Analytical results were used to develop simplified gate-rating equations that will be practical to apply in the gate control system.

INTRODUCTION

Yellowtail Afterbay Dam is a 72-ft high concrete gravity dam with earthen embankment wings located on the Bighorn River in southern Montana, 2.2 miles downstream from Yellowtail Dam near St. Xavier, MT. The dam provides reregulation storage below Yellowtail Dam and Powerplant to stabilize flows in the Bighorn River, and also is the point of diversion for flows into the Bighorn Canal, which conveys irrigation water along the east side of the valley of the Bighorn River as it proceeds toward Hardin, MT.

Three gated outlet systems release water from the dam into two waterways:

- An **overflow weir** section equipped with five 30-ft-wide by 13.5-ft-tall radial gates releases water into the Bighorn River. Flows through this structure can be gate-controlled, or the gates can be raised out of the water to allow free weir flow over the dam crest, which is at elev. 3179.50 ft. Maximum discharge past the overflow weir is about 25,000 ft^3/s at reservoir elevation 3192.0 ft.
- The **river sluiceway** uses three 10-ft-wide by 8-ft-tall vertical slide gates to release water into the Bighorn River at the right end of the overflow weir. The gate sill elevation is 3157.0 ft, and the maximum discharge capacity is 8,100 ft^3/s. Flow through the river sluiceway is always gate-controlled.
- The **canal sluiceway** contains two 10-ft-wide by 8-ft-tall vertical slide gates that release water to the Bighorn Canal. The gate sill is at elev. 3167.0 ft. Maximum discharge capacity of the canal sluiceway is 750 ft^3/s. Flow is normally gate-controlled, but at unusually low reservoir elevations, weir flow is possible.

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Figure 1. Aerial view of Yellowtail Afterbay Dam showing the three outlet systems that release water to the Bighorn River and Bighorn Canal.

Flows in the Bighorn River below the dam and in the Bighorn Canal are measured at river gage sites operated by the Bureau of Reclamation and incorporated into the U.S. Geological Survey gaging network. Gage 06287000 (Bighorn River near St. Xavier, MT) is located in the Bighorn River about 780 ft downstream from the Afterbay Dam. This gage has provided daily discharge data since October 1, 1934. A second gage is located in the Bighorn Canal approximately 720 ft downstream from the end of the canal sluiceway stilling basin. Reclamation's Hydromet system provides daily average flows at both gages as well as reservoir elevations for the Afterbay Dam pool from October 1, 1985 to present. Hourly and 15-minute interval data are also available for many parameters through the project's SCADA system.

The original operational scheme for Yellowtail Afterbay Dam called for the dam outlets to be operated to maintain a steady discharge in the river downstream from the dam, as indicated by the river gage. However, seasonal growth of algae in the river and other factors cause the river gage rating to shift significantly over time, making it difficult to maintain steady outflows from the dam. There is also a history of high total dissolved gas levels in the Bighorn River below the Afterbay Dam. The river sluiceway is the primary generator of the high dissolved gas concentrations. To control total dissolved gas levels, it is desirable to operate the dam outlets to generally maintain a 25% – 75% split of flows released through the river outlet works and the overflow weir, respectively, although this flow split ratio is subject to adjustment.

These two issues prompted the development of a new Afterbay Automated Gate Control System (AAGCS) that was designed to set river flows by calculating discharges through the gated outlets. The system was also intended to maintain the desired flow split for

control of total dissolved gas in the river. Unfortunately, during commissioning of the new system, fluctuations in the river elevation exceeded allowable limits during power peaking, and gates operated excessively while attempting to maintain the set point.

As a result, implementation of the AAGCS was suspended and Reclamation's Technical Service Center was asked to review the control system and make recommendations for modifications that would improve system performance. A preliminary review identified issues related to control algorithms, control stability, and flow measurement accuracy. Sources of potential inaccuracy in the existing discharge equations were identified for each of the gated outlets and the report recommended a review of the gate-flow equations and development of accurate, physically-based equations as needed. The project to develop new discharge equations for all three outlet facilities is described fully in Reclamation research report HL-2013-01 (Wahl 2013). This paper focuses on the application of the WinGate computer software to the two sluiceway systems.

BACKGROUND INFORMATION

The investigation began with a review of polynomial equations that had previously been developed for the automated control system. For each of the three outlet systems, these equations compute discharge as a function of gate settings and relevant water levels. In addition, the Standing Operating Procedures for the dam provide graphical rating curves for the overflow weir and river sluiceway, based in part on a 1:24-scale physical model study of the dam (Arris 1965) performed during its original design. A second 1:24-scale physical hydraulic model study of the river sluiceway (Young 1982) also provides an alternative rating curve for the river sluiceway gates. No rating curves were known to exist for the canal sluiceway gates.

The Bureau of Reclamation Hydromet system provides operational data for the site using station name BHSX (Bighorn River near St. Xavier, MT). The system can be accessed on the Internet at http://www.usbr.gov/gp/hydromet/. Daily values of several useful parameters are available from October 1, 1985 to present. Key variable names are shown in Table **1**.

Table 1. Available Hydromet data for Yellowtail Afterbay Dam.

	Real-time data	**Historic daily values**
Forebay elevation	FB	FB
River gage height	GH	GD
River gage shift	HH	HH
River discharge	QR	QRD
Total discharge (river + canal)	Q	QD
Canal gage height	CH	GJ
Canal gage height shift	HJ	HJ
Canal discharge	QC	QJ

OVERFLOW WEIR

The Standing Operating Procedures for Yellowtail Afterbay Dam (SOP) provides a rating curve dated July 1966, showing both free weir flow and gate-controlled flow. The free weir flow curve (gates out of the water) is based on a 1:24-scale physical hydraulic model study that included the weir crest, but did not include functional gates (Arris 1965). The source of the curves for gate-controlled flow is unknown.

Because the engineering basis for the gate-controlled flow equations is not known, the decision was made to develop new equations using either existing operational data or an accepted analytical method. Unfortunately, a review of the available data raised questions that could not be resolved regarding the accuracy of historic gate position measurements. This uncertainty made it impossible to utilize currently available historical data for discharge calibration purposes and forced the application of an analytical method.

The method used to develop new discharge equations for the overflow weir was that described in U.S Army Corps of Engineers Hydraulic Design Criteria Charts 311-1 to 311-5 and also presented in *Design of Small Dams* (Reclamation, 1987). For details on the application of this method to Yellowtail Afterbay Dam, see Wahl (2013).

RIVER SLUICEWAY

Existing Methods for Determining Discharge

Three methods for computing discharge through the river sluiceway are currently available. The SOP provides a rating curve based on the original hydraulic model study (Arris 1965) and field data of unknown origin, a second rating curve was developed during the 1982 physical model study (Young 1982), and polynomial equations were developed for the automated gate control system.

Figure **2** shows the SOP discharge curves, the 1982 model study curves, and the curves produced by the AAGCS polynomial equation. For gate openings of 4 ft or less there is good similarity between the SOP and 1982 model study curves, but for higher gate openings the latter model study curves indicate lower discharges, which is attributed to the inclusion of a larger forebay in the 1982 model. The polynomial equations indicate significantly higher discharges than the other two sets of curves, especially at high reservoir levels and large gate openings. There is a huge disparity for gate openings of 6 ft or more. There is no plausible explanation for the polynomial curves indicating that the sluiceway discharge at an 8-ft gate opening is almost double that of a 6-ft opening.

Effective Discharge Coefficients

Discharge curves for vertical sluice gates should generally exhibit the behavior of an orifice flow equation:

$$Q = C_d G L \sqrt{2gH} \tag{1}$$

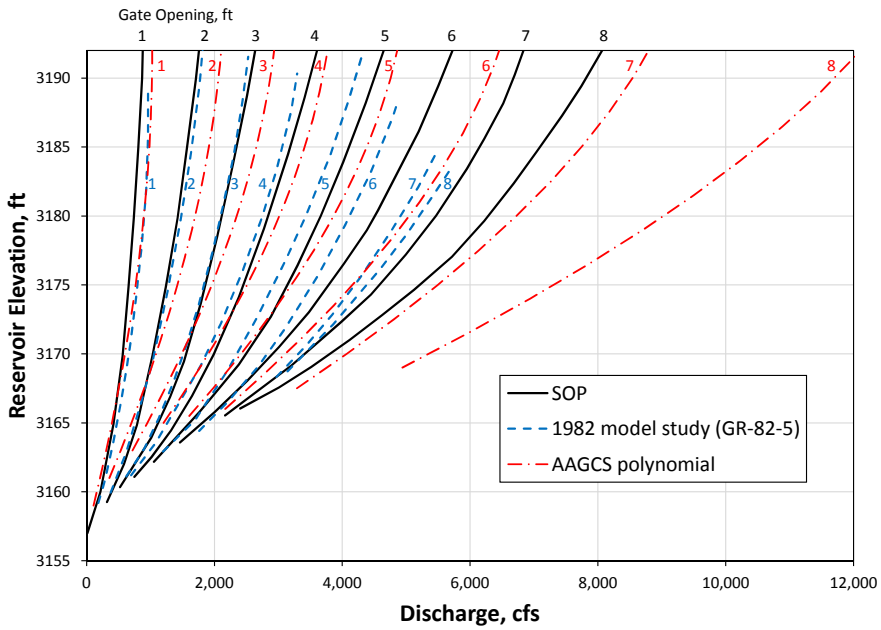

Figure 2. River sluiceway discharge curves.

with a discharge coefficient that is relatively constant near a value of 0.6. To further evaluate the discharge curves shown in Figure **2**, effective discharge coefficients were computed from data points digitized from the original curves. The head term in the orifice equation was computed relative to the center of the gate opening. The variation of these discharge coefficients with relative gate opening is shown in Figure **3**.

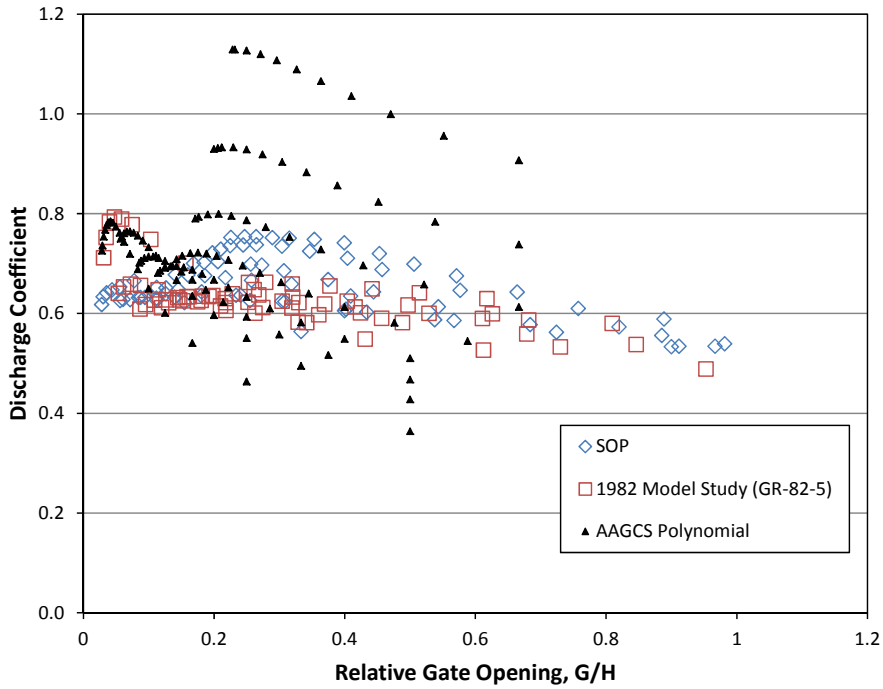

Figure 3. Effective discharge coefficients for previously established river sluiceway rating curves.

The discharge coefficients for the SOP and 1982 model study curves behave somewhat as expected. The model study data in particular show a steady value close to 0.6, except for the data points associated with the 1 ft gate opening, which was noted to have a suspicious shape in the 1982 model study report. In contrast, the AAGCS polynomial shows dramatic variation of the discharge coefficient and values exceeding 1.0 that are not physically realistic.

Analytical Discharge Curves

The discharge curves and equations compared in Figure **2** all base the gate discharge on just the gate opening and the upstream reservoir elevation, so they are assuming that free flow occurs through the sluice gates. However, a review of the river gage records and the resulting tailwater conditions below the dam shows that river stages could be high enough to cause submergence of the sluice gates, since the gate seat is at elevation 3157.0 ft and tailwater levels vary from about 3157 to 3165 ft. Although submergence is possible, tailwater levels above the gate seat elevation do not guarantee submerged flow, since the momentum of the flow through the gate opening may be strong enough to sweep the tailwater away from the gate exit and allow the gate to flow free. The most likely condition for gate submergence would be combinations of high discharge through the overflow weir (causing high tailwater in the river) and low discharge through the sluiceway. Future operations will maintain a steady ratio of the sluiceway and overflow weir discharges, and it is not immediately apparent that this will yield submerged flow conditions at the sluiceway.

To evaluate the potential for submergence, flow through the sluiceway gates was analyzed using the new WinGate computer program (Wahl and Clemmens 2012; Clemmens et al. 2012). This program was developed for the purpose of calibrating radial gates and vertical slide gates in canal check structures using the energy-momentum (E-M) method. The program can be applied to these sluice gates since they seat on a horizontal surface, as opposed to an ogee crest. The program solves the energy equation from the upstream pool to the orifice opening beneath the gate and the momentum equation from there to the downstream canal. The combined use of the energy and momentum equations makes the method well suited to accurate modeling of flow in the transition zone from free to submerged flow.

The WinGate analysis made use of historic Hydromet data to model a realistic set of circumstances. Daily values of total river flow, river gage height, and upstream reservoir elevation were obtained from 10/1/1985 to 2/3/2013. These data were filtered by comparing the river flow rates and net river gage height data (gage height plus gage shift) to the river gage rating equation. About 87% of the net gage height values matched the values expected for the corresponding river discharges within ±0.01 to ±0.03 ft, while 13% varied from the expected values by amounts up to ±2 ft. The latter data were excluded from analysis on the assumption that the mismatch to the river gage rating curve indicated that operations on that day were highly transient, and thus average daily values did not accurately represent the conditions.

For the data retained, the total river discharges were used to compute the flow through the overflow weir and sluiceway that would have been set on that date, if the 25% – 75% flow split rule had been applied. WinGate was then used to solve for the sluice gate openings needed to obtain the computed sluiceway flow, using the river gage height as the tailwater elevation just downstream from the sluiceway stilling basin. This neglects the small head loss that occurs in the river reach between the end of the stilling basin and the river gage location, but is a reasonable approximation. The WinGate analysis considered that the reservoir upstream from the sluice gates was only as wide as the gates themselves, so that the velocity heads in the reservoir approaching the sluice gates would be realistic, since WinGate was modeling only the sluiceway and not the simultaneous flow through the overflow weir structure.

The analysis showed that the required sluice gate setting for about 80% of the cases produced a submerged, gate-controlled flow condition, while 20% produced free gate-controlled flow (meaning that the tailwater level was too low to affect the discharge, even though it may have been above the gate seat elevation). When total river discharge was greater than 5225 ft^3/s (sluiceway discharge > 1306 ft^3/s) the sluiceway was always in free flow, and when the river discharge was less than 2475 ft^3/s (sluiceway discharge < 619 ft^3/s) the sluiceway was always submerged. When the river discharge was between these limits, either flow condition was possible. The exact threshold for submergence is a complex function of the gate opening, upstream head, and tailwater elevation and requires a momentum analysis like that performed in WinGate. The situation is also complicated by the fact that the river gage elevation for a given river discharge is not constant due to algae growth in the river and other factors that create the need for adjusting the river gage relation seasonally through the use of shifts. Finally, if flow

splits other than the 25% – 75% condition were used, the ranges in which free and submerged flow are possible would vary. It should be emphasized that although this analysis was performed assuming the 25% – 75% flow split, this assumption was made only to obtain a realistic range of operating conditions; the results should be applicable to other flow split ratios.

For the submerged flow cases, WinGate also provides output of the calculated discharge if free flow had existed, and an analysis of these data showed that the effects of submergence were typically small; the median discharge reduction from the free flow value was only 0.29%, and 99.5% of the cases had errors smaller than 2.5%. This shows that when submergence occurs, the river tailwater levels are typically just above the threshold needed to cause submerged flow. Although ignoring submergence would cause mostly small errors, the maximum error was 4.73%, and all errors were in the same direction, so it seemed worthwhile to seek a way to account for submergence.

The sluiceway discharges computed with WinGate were used to back-calculate effective discharge coefficients for use in basic orifice equations for free and submerged flow. For free flow the head is measured relative to the center of the gate opening, and for submerged flow the head is the difference between the reservoir level and the river level at the gaging station. Figure **4** shows the variation of the discharge coefficients as a function of the relative gate opening. Note that the relative gate opening is defined as the ratio of the gate opening to the upstream head *relative to the gate sill elevation* (not the gate centerline). Although the orifice equation for computing free discharge uses head referenced to the center of the gate opening, the discharge coefficient proved to be more closely related to the sill-referenced head.

The variation of the discharge coefficient for free flow is very slight, and this is consistent with experimental and numerical simulations of flow through vertical sluice gates (Belaud et al. 2009). The submerged flow discharge coefficient varies more significantly and in a different manner, increasing with relative gate opening. This is partly due to the change in the definition of the head term for submerged flow, and also due to the fact that this coefficient is accounting for the lumped effects of several empirical factors affecting submerged flow (e.g., momentum effects in tailwater channel) that are included in the WinGate analysis.

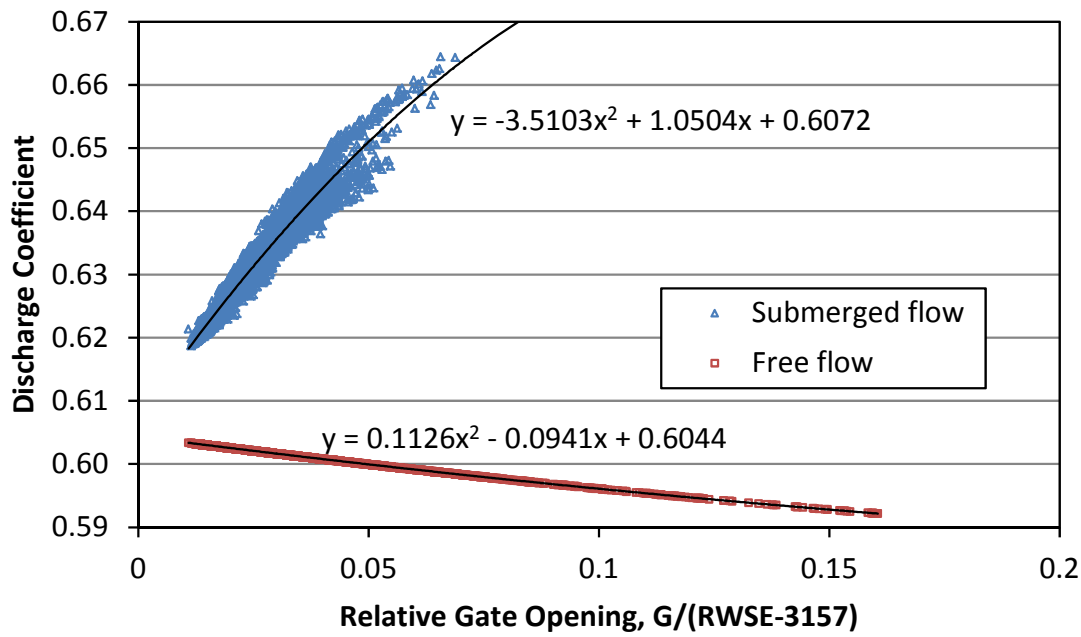

Figure 4. Discharge coefficients for river sluice gates based on the WinGate analysis.

To compute discharge through the sluiceway using the information in Figure **4**, the following steps can be performed:

1. Compute relative gate opening

$$G^* = \frac{G}{RWSE - 3157} \tag{2}$$

2. Compute free-flow discharge as

$$\begin{aligned} C_{d,free} &= 0.6044 - 0.0941(G^*) + 0.1126(G^{*2}) \\ Q &= (3)(C_{d,free})(10)G\sqrt{2g(RWSE - 3157 - G/2)} \end{aligned} \tag{3}$$

3. Compute submerged-flow discharge as

$$\begin{aligned} C_{d,submerged} &= 0.6072 + 1.05(G^*) - 3.51(G^*)^2 \\ Q &= (3)(C_{d,submerged})(10)G\sqrt{2g(RWSE - h_{gage})} \end{aligned} \tag{4}$$

where h_{gage} is the elevation of the water surface at the river gaging station. The final result is the minimum discharge computed by the two methods. Figure **5** shows the resulting discharge prediction errors and demonstrates that this approach does a good job of distinguishing between free and submerged flow and provides a reasonable representation of the flows predicted by the WinGate analysis.

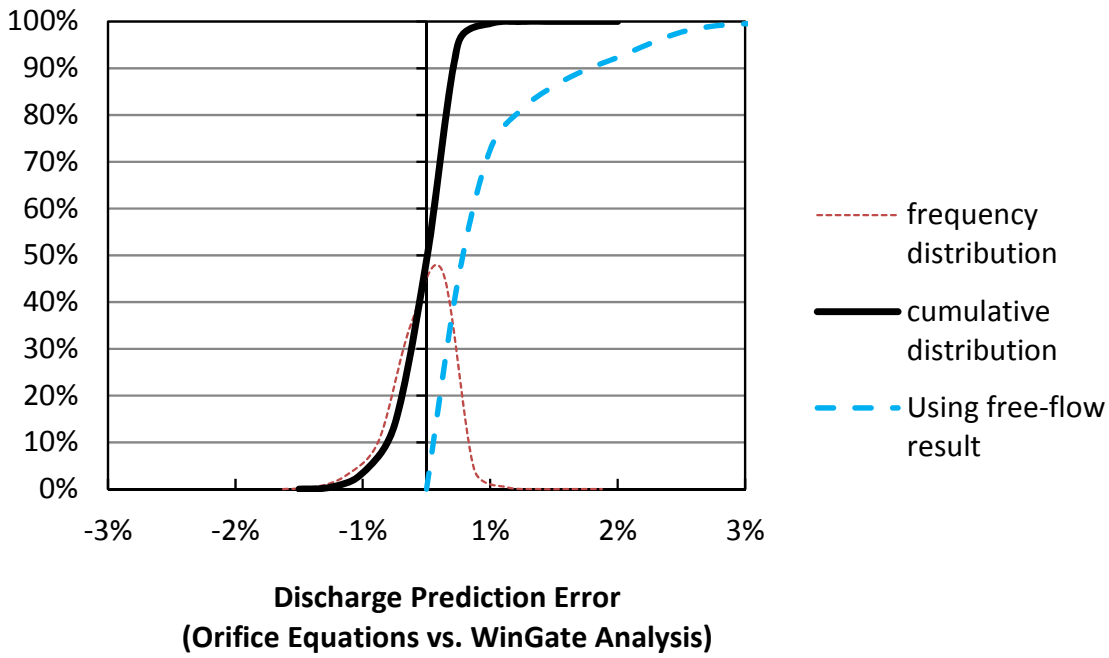

Figure 5. Discharge prediction errors using minimum of free-flow and submerged-flow orifice equations. The errors that would occur by always assuming free flow are also shown.

The control system will also need to solve for the gate setting required to obtain a target discharge. For this purpose, free flow conditions should be assumed and the discharge coefficient can be set to 0.6 as an initial value. The gate setting can then be determined from the free-flow orifice equation. Once this has been done, the discharge coefficients for free and submerged flow can be refined and the equation controlling the final result can be determined. Multiple iterations may be needed to reach convergence.

Figure **6** shows a new set of discharge curves that was generated for the sluiceway using the results of the WinGate analysis. Since there is a range of ambiguity for free versus submerged flow and the effect of submergence is slight in the vast majority of cases, the curves were generated using only the free flow equations. The resulting curves most closely match the 1982 model study curves and indicate somewhat lower discharges than all of the previously developed curves. When submerged flow conditions exist, the discharges would be reduced further, but only by a small amount in most cases (0 to 2%).

There is significant difference between the various discharge curves beginning at gate openings greater than 4 ft, but the practical importance of large gate openings may be small. The WinGate analysis using the 1985-2013 Hydromet data showed that the sluice gate setting to achieve the 25% – 75% flow split objective would be less than 4 ft on 99.5% of the days analyzed (see Figure **7**).

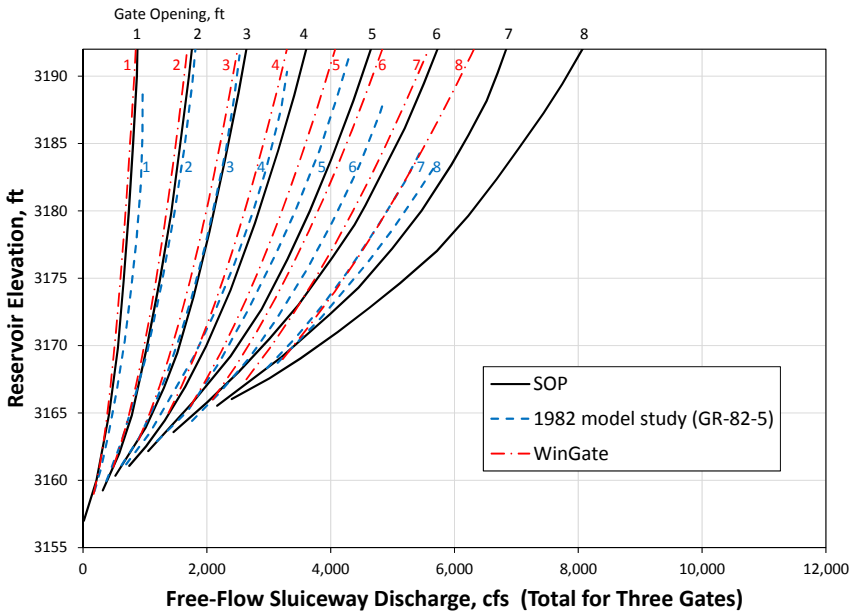

Figure 6. Discharge curves generated from WinGate analysis.

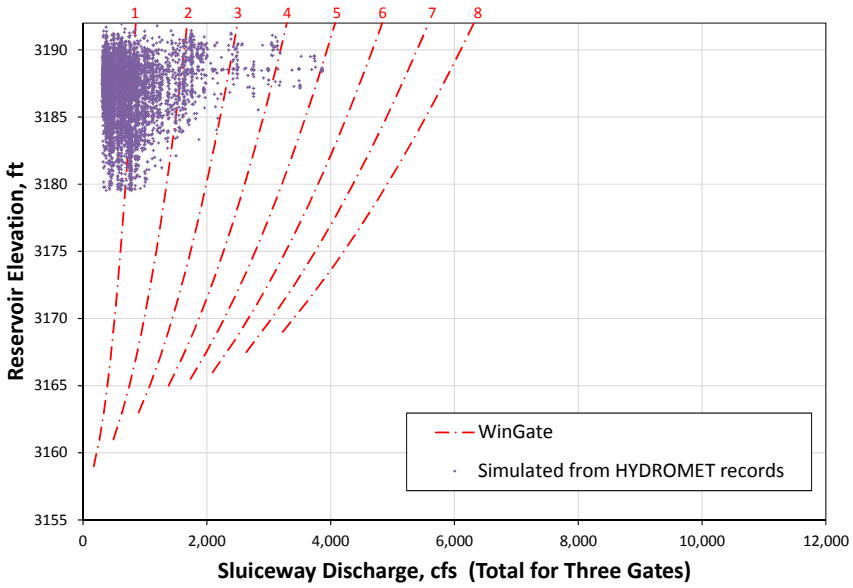

Figure 7. Sluiceway operating conditions simulated for 1985-2013 period from Hydromet data.

Comparison to Field Data

There are few field data values available for testing the discharge equations and curves. The SOP discharge curve provides 7 data points, and an additional 6 data points are available, which were reportedly used to develop the AAGCS polynomial equations, but their original source is unknown. **Error! Not a valid bookmark self-reference.** shows

the available data and the discharges computed by different methods. The WinGate discharges are significantly lower than the discharges predicted by the other methods, as expected.

Table 2. Field data for discharge through the river sluiceway.

Reservoir elevation, RWSE, ft	Gate opening, G, ft	Observed discharge, ft^3/s (three gates)	Predicted discharges and % differences from observed		
			AAGCS Eqn.	WinGate	SOP
Data used to develop AAGCS equations (unknown origin)					
3186.38	0.838	896	807 (-10%)	653 (-27%)	670 (-25%)
3186.38	1.483	1509	1501 (-1%)	1146 (-24%)	1225 (-19%)
3185.99	3.44	2943	3030 (+3%)	2572 (-13%)	2750 (-7%)
3185.33	4.074	3492	3497 (+0%)	2983 (-15%)	3250 (-7%)
3185.80	4.643	4100	4063 (-1%)	3406 (-17%)	3800 (-7%)
3184.00	5.41	4750	4786 (+1%)	3788 (-20%)	4400 (-7%)
Data from SOP discharge curve					
3169.25	2	1013	1032 (+2%)	956 (-6%)	1000 (-1%)
3184.42	3	2445	2623 (+7%)	2189 (-10%)	2300 (-6%)
3174.75	4	2424	2483 (+2%)	2251 (-7%)	2450 (1%)
3169.75	5	2424	2376 (-2%)	2254 (-7%)	2450 (1%)
3189.42	5	4460	4721 (+6%)	3901 (-13%)	4450 (0%)
3179.83	6	4481	5023 (+12%)	3779 (-16%)	4500 (0%)
3174.83	7	4552	5450 (+20%)	3731 (-18%)	4600 (1%)

Despite the large differences from the few field observations, the WinGate discharge curves are believed to offer the best estimate of discharge for the sluiceway. The inconsistent behavior of the discharge coefficients in the other methods has already been discussed and gives good reason for discrediting them. The WinGate curves are physically-based and reflect the most current research on sluice gate discharge characteristics. There are potential sources of error in the WinGate analysis, such as unique site-specific approach flow conditions and head losses that may not be accurately accounted for, but most such factors would tend to reduce the discharges, and WinGate is already predicting lower flows than the other methods. The one factor that could cause the WinGate discharges to be too low for a given reservoir level is not correctly accounting for high velocity head in the reservoir due to the simultaneous operation of the overflow weir with the river sluiceway. If a consistent bias between the new equations and field-measured flows is found in the future, adjustments could be made to the WinGate-based discharge curves.

BIGHORN CANAL SLUICEWAY

The canal sluiceway contains two 10-ft-wide by 8-ft-tall vertical slide gates that release water into the Bighorn Canal. The gate sill elevation is 3167.00 ft, and the maximum discharge capacity of the canal sluiceway is 750 ft^3/s. Flow is normally gate-controlled, but for unusually low reservoir elevations, weir flow may be possible.

A rating curve for the sluiceway gates is not provided in the SOP for Yellowtail Afterbay Dam. Discharge in the canal is measured at a gaging station located about 720 ft downstream from the start of the canal. Hydromet records define the rating curve at the gaging station. About 90% of the available daily readings fit the rating within ±0.25 ft, with the other 10% scattering widely around the curve defined by the bulk of the data.

A complex polynomial equation developed for use with the automated gate control system was reviewed and found to be seriously flawed in several ways (see Wahl 2013 for details).

WinGate Analysis

Hydromet data for the canal sluiceway were obtained from October 1, 1985 to October 10, 2012. The data set was filtered to retain only those data that fit the rating curve defined by the bulk of the data, with outliers considered to be days on which steady flow did not prevail, so daily average values provided a poor representation of real conditions. With the filtered data, WinGate was used to compute sluice gate settings that were required to obtain the recorded value of canal discharge with the given upstream reservoir and downstream canal water levels. As described previously, WinGate performs a momentum analysis that can accurately account for the effects of gate submergence. The results from WinGate were then used to compute effective discharge coefficients for a simplified submerged orifice equation that would be practical for use in the automated gate control system. It is notable that the data set contained no records of conditions for which weir flow was likely.

The WinGate analysis showed that the gates always operate in a submerged flow condition, and the reduction of discharge due to submergence varies from about 5% to 28% of the free-flow discharge.

The WinGate results were used to compute submerged-flow discharge coefficients for a basic orifice equation:

$$Q = C_d G L \sqrt{2g\Delta H} \tag{5}$$

where Q is the discharge in ft^3/s, C_d is the discharge coefficient, G is the gate opening, L is the gate width, g is the acceleration due to gravity, and ΔH is the difference in elevation between the upstream reservoir and downstream canal.

The discharge coefficients were related to both the relative gate opening, G/H_1, and the submergence ratio, H_3/H_1, where H_1 is the upstream head and H_3 is the downstream head, both measured relative to the gate sill elevation. Thus, H_1=*RWSE*-3167 and H_3=Y_{canal}-3167, where *RWSE* is the upstream reservoir water surface elevation and Y_{canal} is the water surface elevation in the canal. Note that this submergence ratio is a simple parameter that does not perfectly reflect the submergence conditions at the gate itself, since the water level at the back side of the gate leaf will be different from that in the downstream canal, but it is straightforward to compute and useful for operational computation purposes. Figure **8** and Figure **9** show the relationships to each parameter.

Both relations appear to be promising for predicting discharge coefficients, but an even better relation was found using an equation fitting software tool designed for analysis of 3D surface functions, TableCurve 3D. This relation is:

$$C_d = \frac{1}{1.6503 - 1.769\sqrt{G/H_1} + 0.7717\left(H_3/H_1\right)} \tag{6}$$

Figure **10** shows the results when predicting C_d using equations based on G/H_1, H_3/H_1, and both parameters together. The only region where the two-parameter relation performs poorly is when the predicted C_d value is less than 0.62. When this is the case, the equation shown in Figure **8** based on G/H_1 should be used.

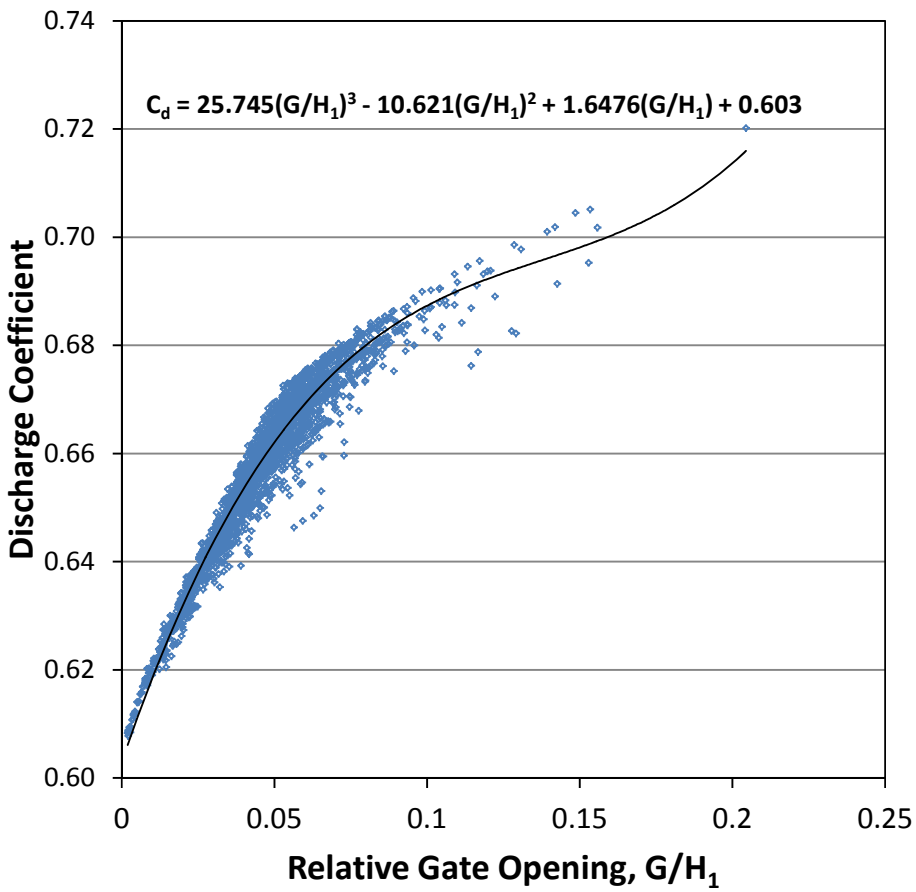

Figure 8. Relation between canal sluiceway discharge coefficients and relative gate opening.

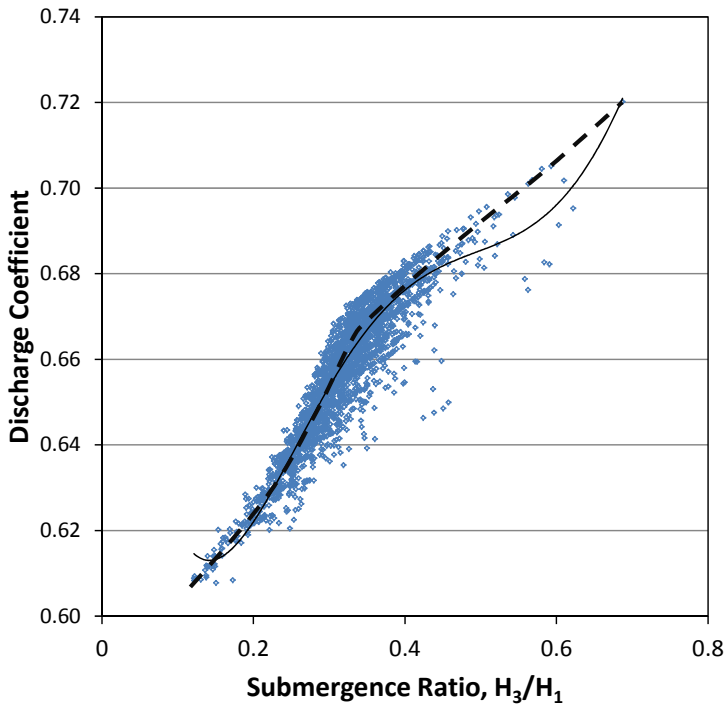

Figure 9. Relation between canal sluiceway discharge coefficient and submergence ratio. The solid line is a fifth-order polynomial curve fit, and the dashed line is a manual curve fit (by eye).

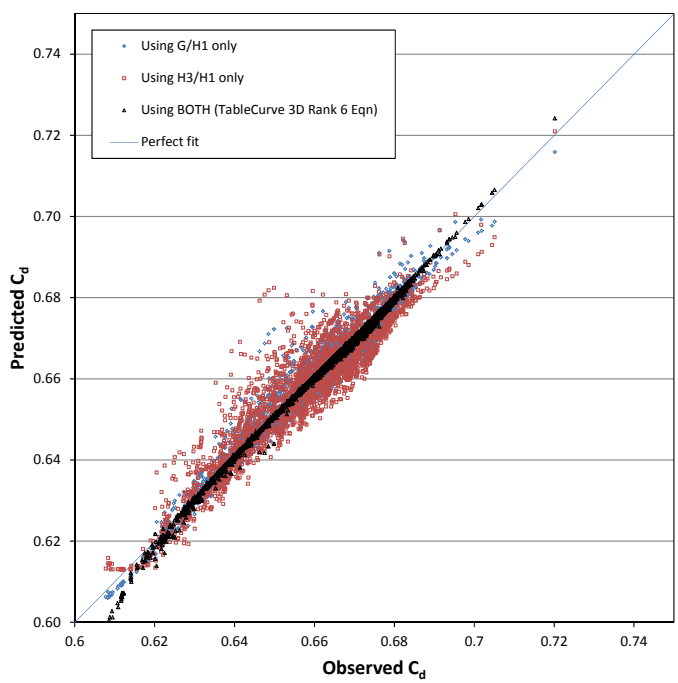

Figure 10. Performance of several functions that can predict values of discharge coefficients for the canal sluiceway.

Sources of Uncertainty in WinGate Analysis

The WinGate computer program is a physically based model designed to calibrate canal sluice gates for accurate discharge measurement. It applies the energy equation to the upstream side of the gate and includes empirical factors that account for energy loss approaching the gate, assuming a streamlined approach channel. At the gate opening, the program applies empirical relations for estimating the contraction coefficient of the flow through the gate opening. Downstream from the gate, the momentum equation is applied, with empirical relations that estimate the hydrostatic and drag forces on downstream channel boundaries. Each of the empirical relations is a source of uncertainty, as are the assumptions of streamlined approach flow and no interaction with the adjacent river sluiceway and overflow weir. The greatest source of uncertainty in the model results is probably the estimation of flow forces on the sloped apron downstream from the gate, which leads into the stilling basin. The canal check gates that WinGate was designed to analyze typically do not have such sloped aprons or discharge into a stilling basin, so this specific downstream channel configuration has not been studied during the development of WinGate. If the forces on this surface are not accurately modeled in WinGate, there will be a systematic error in the computed discharge. The relative size of this error should be consistent throughout the operating range, so a future calibration adjustment is possible.

The analysis used to develop the new gate equations made use of the prevailing relationship between canal discharge and canal depth, which establishes the tailwater condition below the sluice gates. It was noted that the prevailing tailwater curve is about 2 ft higher than that which was expected based on the original design parameters of the canal. The reason for this difference is presently unknown. A second analysis was carried out in which the tailwater curve was set at the design level, and in this case the sluiceway would experience free orifice flow for a significant range of operating conditions. Thus, it should be noted that if tailwater conditions at the site change in the future due to canal maintenance activities or canal rehabilitation projects, that may change the equations needed to accurately predict sluiceway discharges.

Weir Flow

Although the Hydromet records indicate that weir flow through the canal sluiceway is highly unlikely, the potential for it does still exist. If the reservoir level upstream from Yellowtail Afterbay Dam is extremely low, it may be necessary to raise the canal gates out of the water to deliver as much water into the canal as possible. However, a simple estimation of free weir flow ($Q=3.09LH^{1.5}$) through two gates or through one gate (assuming the other gate is closed) shows that in either case the tailwater levels produced in the canal are higher than the reservoir levels needed to obtain a given flow rate. Thus, it is impossible for free weir flow to exist, and the weir will always be submerged by the tailwater if the gates are raised out of the water. In this condition, flow will actually be controlled by the canal cross section and the canal gaging station will offer the best means for estimating the flow rate. The present rating equation for the gaging station was obtained from analysis of the Hydromet data:

$$Q = -0.2182h^3 + 60.91h^2 - 5278.5h + 145915 \quad (7)$$

where h is the net (shifted) canal gage height relative to elevation 3100 ft. (i.e., if the canal water level is at elevation 3170.0 ft, h=70.0).

Recommended Equations for Canal Sluice Gate Discharge

Discharge through the canal sluiceway should be computed using the orifice equation, Eq. (6), with the discharge coefficient computed from Eq. (7), unless the value computed there is less than 0.62. In that case, the discharge coefficient should be computed instead with the equation shown on Figure **8**.

If the reservoir elevation and canal elevation are both lower than the gate lip elevation, then the gates will not control the flow and the discharge should be determined at the canal gaging station using Eq. (8).

When it is necessary for the control system to compute the gate opening needed to deliver a target discharge into the canal, an iterative solution of the orifice equation is required, since the discharge coefficient is dependent on the gate opening. The current value of the gate opening could be used as an initial guess at the new gate opening and the calculations then proceed as described above. Alternately, the discharge coefficient could be assumed to have a value of 0.66 for the first cycle of calculations (the median of the values obtained from the WinGate analysis).

CONCLUSIONS

The WinGate computer program was effectively used in this study to develop discharge rating equations for two sets of vertical slide gates. The results of the WinGate analysis were used to develop simplified rating equations that will be suitable for use in the automated gate control system. The river sluiceway gates can operate in either free or submerged flow, and a method was developed for identifying the controlling discharge equation. The canal sluiceway gates will operate in a submerged condition at all times (assuming current tailwater levels are maintained). An empirical equation was developed to predict the submerged flow discharge coefficient as a function of both the relative gate opening and relative submergence.

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MATLAB / SIMULINK NONLINEAR HYDRAULIC MODELS FOR TESTING CANAL GATE CONTROL ALGORITHMS

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ABSTRACT

Two unsteady-flow open-channel hydraulic models were developed in Matlab/Simulink for testing canal process control algorithms. These models solve the Saint Venant equations using 4-point implicit and Runge Kutta methods. Both methods produce results that are similar to simulation results from the RootCanal model. Initializing the 4-point implicit or Runge Kutta methods for solving the equations in Matlab/Simulink is tedious but these algorithms have helped to evaluate control routines. Matlab/Simulink have numerous control methods that are readily available, so it takes little effort to test these methods once the open-channel models have been programmed. These algorithms can be used to test both local and global canal gate control routines.

INTRODUCTION

The need for more water and the conservation of existing water supplies is becoming increasingly important. Increasing populations as well as expanding urban consumption places pressure on agriculture to produce more with less water. Hydrologists, water managers, and scientists generally agree that this increasing need places great responsibility on developing new and improved water conservation methods.

One method that has been studied to improve water conservation is canal gate automation. Numerous efforts have been made to determine a method that is best suited for the canal automation. Empirical methods have been developed (Burt 1983; Parish 1994; UMA Engineering 1989), as well as methods based on process control (Balogun et al. 1988; Clemmens et al. 1997; Litrico and Fromion 2006; Ploss 1987; Reddy et al. 1992; Schuurmans 1997; Wahlin 2004; Wahlin and Clemmens 2002).

Testing canal gate control algorithms that will work on actual systems require an accurate hydraulic model, but defining such a model is difficult. Control engineers indicate that determining an adequate mathematical model is one of the most difficult tasks in designing a feedback control system. Furthermore, most processes in nature have nonlinear characteristics that make modeling difficult. This forces control engineers to simplify system models to develop control algorithms which can be used to manipulate a real system. This problem is further compounded by the fact that commonly used control methods are based on the development of a linear model to simulate system response,

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resulting in a linear controller. In many cases, this type of control manipulates the system adequately but problems may occur in instances where the system is highly nonlinear.

When a linear controller is developed and implemented, difficulties can arise during the initial testing, and in some cases catastrophes occur (Phillips and Harbor 2001). Even though the controller can be proven to be mathematically stable, it must be remembered that this stability is proven for a simplified linear system and not for the actual case. Extensive tests may be performed on the linear model, but a control engineer cannot be entirely certain that the designed controller will operate as desired until the controller is extensively tested on the actual system. It should also be noted that in many cases a linear controller can perform the desired control response, but one cannot be sure about controller performance until extensive testing is completed.

Canal system hydraulics is an example of a nonlinear system and can be modeled by solving the Saint Venant equations (Strelkoff 1969). Even though these equations are simplified from more general equations, they are nonlinear and involve wave phenomena. If they are used to design control algorithms, they must be further simplified. As mentioned above, linear equations are required for process control design techniques. After designing a system controller, it can and should be tested on a linear model, but it is very useful to test the algorithm on a nonlinear model before performing tests in the field, thereby saving time.

This testing phase is even more important for the development of a canal controller. Canals are often long and have large operational time delays. Computer simulations of hydraulic behavior may be used to investigate canal response in much less time. A canal response which takes several hours or days can be investigated in a few minutes via computer simulation. When mathematical models are used, no water is wasted and water users do not experience interruptions in deliveries. Extensive simulation can be conducted on a controller and the nonlinear canal model throughout the entire operation range before installation in the field.

There are a few available canal models that can be used to simulate canal systems, but extensive computer coding and debugging is required to test control algorithms. On the other hand, there is existing control system development software which is very useful and convenient for testing various control routines before implementing the routines on the canal system. One product which is available and extensively used in the control industry is Matlab/Simulink (2013). Much of control engineering makes use of linear models to develop system controllers, but Matlab can be programmed to model nonlinear processes. This model can be programmed using 4-point implicit solving routines, ordinary differential equation (ODEs) routines, or any other programming method that is desired. This software has the ability to display system response using several readout functions. Matlab has several preprogrammed solving routines that will solve ODEs. In addition, it has the capability to easily implement the latest control methods used in research and by industry.

MODEL EQUATIONS

The Saint-Venant equations are nonlinear hyperbolic partial differential equations. Several methods have been developed to solve these equations (Strelkoff 1970; Merkley 1988, Akan 2010). Balogun (1985) demonstrated a method where the Saint-Venant equations are converted to ODEs using finite differencing. When this is accomplished, the equations are modified to model several nodes in a canal reach. For the upstream node, the continuity equation takes the following form:

$$\frac{dy}{dt} - -\frac{1}{\Delta x T_{1,m}}\left(Q_2 - Q_g\right) - q_{1,m} \tag{1}$$

For the interior nodes, the continuity equation is as follows:

$$\frac{dy}{dt} = -\frac{1}{2\Delta x T_{j,m}}\left(Q_{j+1,m} - Q_{j-1,m}\right) - q_{j,m} \tag{2}$$

and the momentum equation is:

$$\frac{dQ}{dt} = -\frac{Q_{j,m}}{\Delta x A_{j,m}}\left(Q_{j+1,m} - Q_{j-1,m}\right) + \frac{Q^2{}_{j,m}}{2\Delta x A^2{}_{j,m}}\left(A_{j+1,m} - A_{j-1,m}\right) - \frac{gA_{j,m}}{2\Delta x}\left(y_{j+1,m} - y_{j-1,m}\right) + gA\left(S_o - S_f\right) \tag{3}$$

Finally at the downstream node, the continuity equation is modified slightly and takes the following form:

$$\frac{dy}{dt} = -\frac{1}{\Delta x T_{N,m}}\left(Q_{out} - Q_{gN,m}\right) - q_{N,m} \tag{4}$$

where y = depth of water (m); Δx = length segment (m); T = top width of the water surface (m); Q = flow rate (m^3/s); q = ratio of outflow/inflow per unit length of channel (m^2/s); A = flow cross-sectional area (m^2); g = ratio of weight to mass (m/s^2); S_0 = canal longitudinal bed slope; S_f = energy loss gradient; m = node index (distance); j = step index (time); and, N = index of the last node at the downstream end. There is no need to apply the momentum equation for the upstream and downstream nodes because this requirement can be satisfied by specifying inflows and outflows, respectively. This is alternatively accomplished by specifying flow rate gate equations, and it results in an equal number of unknowns and equations.

Balogun (1985) linearized these equations for testing a Linear Quadratic Regulator (LQR) controller; however, the nonlinear ordinary differential form of these equations can be easily programed into Matlab and Simulink and solved using Runge Kutta or 4-point implicit methods.

As mentioned above, the Saint-Venant equations can be solved using a 4-point implicit method. This method is considered preferable to many other solution techniques, despite the fact that it takes more computing time and more computer memory than explicit methods. Akan (2008) shows a 4-point implicit method to solve the Saint-Venant equations. In this method the continuity equation takes the following form:

$$\frac{(A_{i+1}^{n+1}+A_i^{n+1})-(A_{i-1}^{n}+A_i^{n})}{2\Delta t}+\frac{\theta(Q_{i+1}^{n+1}-Q_i^{n+1})+(1-\theta)(Q_{i+1}^{n}-Q_i^{n})}{\Delta x}=0 \tag{5}$$

The momentum equation is modified to the following form:

$$\frac{(Q_{i+1}^{n+1}+Q_i^{n+1})-(Q_{i+1}^{n}+Q_i^{n})}{2\Delta t}+\theta\frac{\left\{\left[\frac{(Q_{i+1}^{n+1})^2}{(A_{i+1}^{n+1})^2}-\frac{(Q_i^{n+1})^2}{(A_i^{n+1})^2}\right]\right\}}{\Delta x} \tag{6}$$

$$+(1-\theta)\frac{\left\{\left[\frac{(Q_{i+1}^{n})^2}{(A_{i+1}^{n})^2}-\frac{(Q_i^{n})^2}{(A_i^{n})^2}\right]\right\}}{\Delta x}+g\theta\frac{(A_{i+1}^{n+1}+A_i^{n+1})}{2}\frac{(y_{i+1}^{n+1}-y_i^{n+1})}{\Delta x}$$

$$+g(1-\theta)\frac{(A_{i+1}^{n}+A_i^{n})}{2}\frac{(y_{i+1}^{n}-y_i^{n})}{\Delta x}+g\theta\frac{(A_{i+1}^{n+1}+A_i^{n+1})}{2}\frac{(S_{f_{i+1}}^{n+1}+S_{f_i}^{n+1})}{2}$$

$$+g(1-\theta)\frac{(A_{i+1}^{n}+A_i^{n})}{2}\frac{(S_{f_{i+1}}^{n}+S_{f_i}^{n})}{2}-g\theta\frac{(A_{i+1}^{n+1}+A_i^{n+1})}{2}S_o-g(1-\theta)\frac{(A_{i+1}^{n}+A_i^{n})}{2}S_o=0$$

where all the variables were previously defined except θ = weighting factor between 0 and 1; i = node index (distance); and, n = step index (time).

Both the Runge Kutta and 4-point implicit methods were programed into Matlab and Simulink to test various control routines. Both methods provided similar results.

TEST PARAMETERS

The parameters used in determining a test model for this research were for a trapezoidal canal section with a bottom width of 7 m, a side slope of 2 to 1, a Manning roughness of 0.03, a length of 9.7 km, and a longitudinal bed slope of 0.00012. Manning's equation was used in the model to approximate the S_f variable in the momentum equation. The Matlab model was composed of 50 nodes.

Both solving methods were programed into Matlab and then placed in a Simulink program as a coded function. In addition, the Runge Kutta method was programed into Simulink using the visual codding method. The Simulink toolbox uses a visual coding

method to implement ordinary differential equations. For example, the continuity equation for the upstream node is programed as indicated in Figure 1, where the blocks that perform multiplication are designated by *1, *2, *3, and *4, while addition or subtraction symbols indicate blocks where values are summed. In the diagram, the depth value is fed back to perform the required top width calculation. This method requires a starting depth for an initial condition, which is input into the integrator block. The unit outflow/inflow component of the continuity equation has been omitted, but a relationship for this component could easily be incorporated into the program. In this diagram, the flow rate into and out of the node are specified by the block with the Q_2-Q_g label. The flow into the node may be specified as a constant value, a gate equation, or as the input from a control algorithm. Constants that are required for the calculation are specified by a numerical value on the block, while the integration block is denoted by a "1/s." The block labeled by "1/u" computes the reciprocal of the input value.

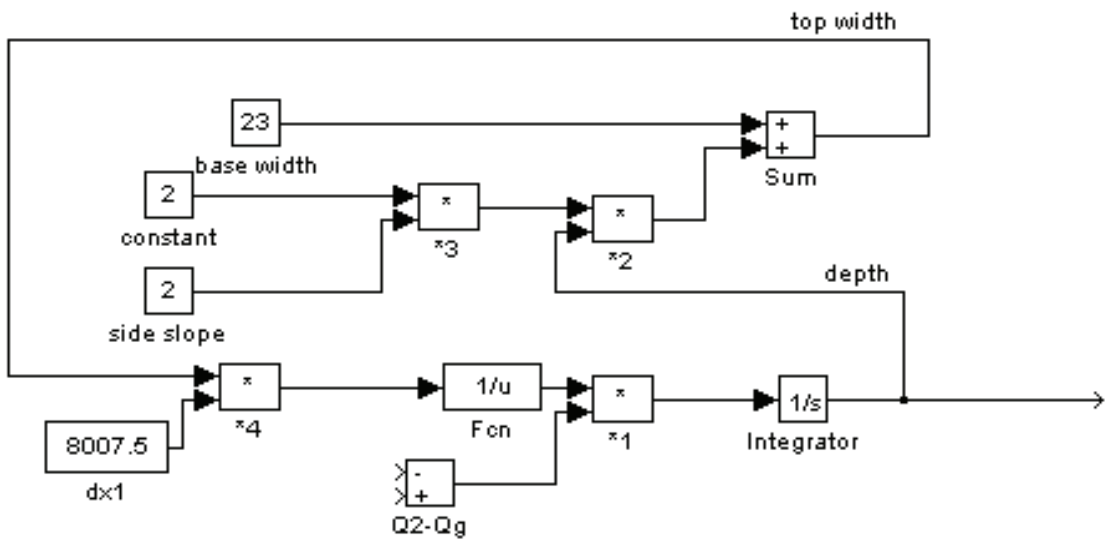

Figure 1. Simulink Diagram for an Upstream Node Continuity Equation for Modeling Unsteady Flow in a Canal.

When the interior nodes are programed into the model, the momentum equation is implemented in the same manner as in Figure 1 along with the continuity equation. However, because there are more variables, the block diagram is much more complex and is not given here. This equation must be combined with the continuity equation for the central nodes. The momentum equation is only required for the central nodes. As all the various nodes are combined, a canal system can be simulated for control algorithm testing. A weir equation was programmed at the end of the reach where the flow rate and water level at the weir were determined.

In Figure 2 the 4-point implicit and the Runge Kutta methods are programed into the block that is labeled canal or canal reach. The majority of the blocks on the left side of the diagram provide input flow values to the canal reach block. When various controllers

are tested, they are substituted into the blocks on the left side of the diagram. The blocks that are located below the canal block are set up to model a duckbill weir but blocks can be changed to model any flow measurement structure. The rectangular blocks that are labeled Qup, yup, and ydn are data readout blocks that display values from these variables as the model is running. There are several rectangular blocks that have smaller rectangles drawn inside. Some of these blocks have a circle drawn in the inner rectangle. These blocks graph the results from different variables as the model is running. All of these data displays provide readout values that allow the user the opportunity to monitor model and gate controller performance as the model is running.

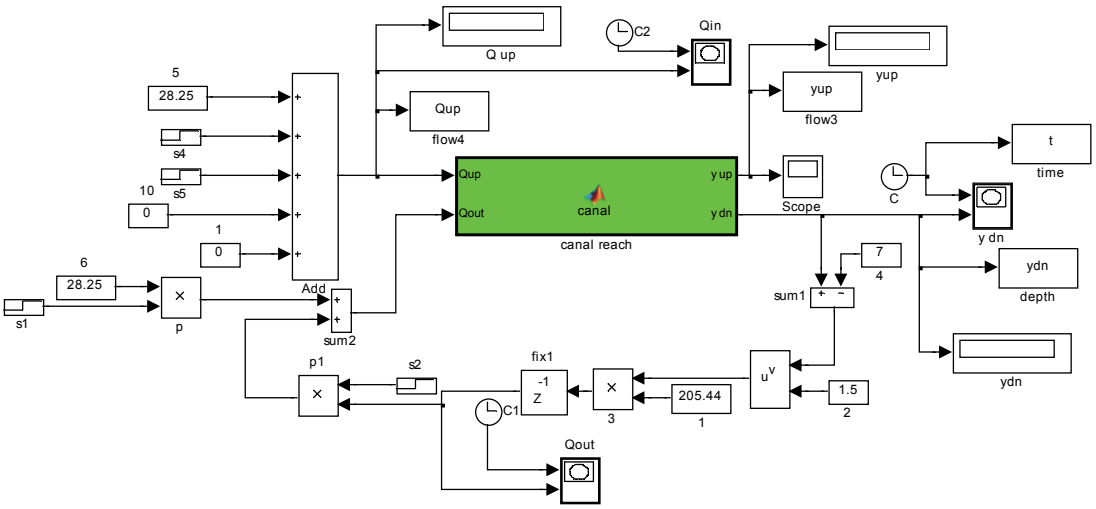

Figure 2. Simulink Diagram for a Canal Reach. The block with "canal reach" written below it can have either a 4 point implicit or Runge Kutta solving routine within it.

RESULTS

The validity of using these methods for simulating canal hydraulics was determined by comparing the results to the RootCanal model (Merkley 2006). Because RootCanal has been validated extensively in the field, it can be used to validate the Matlab models. In one test, the flow rate into the reach was varied and the resulting downstream depths and the times that they occurred were compared to the results from RootCanal. In the first test, the flow into the reach was set at 8.5 m^3/s (300 cfs), and then after 100 minutes, the flow was increased to 10.2 m^3/s (360 cfs). The results from RootCanal and the two Matlab models are given in Figure 3.

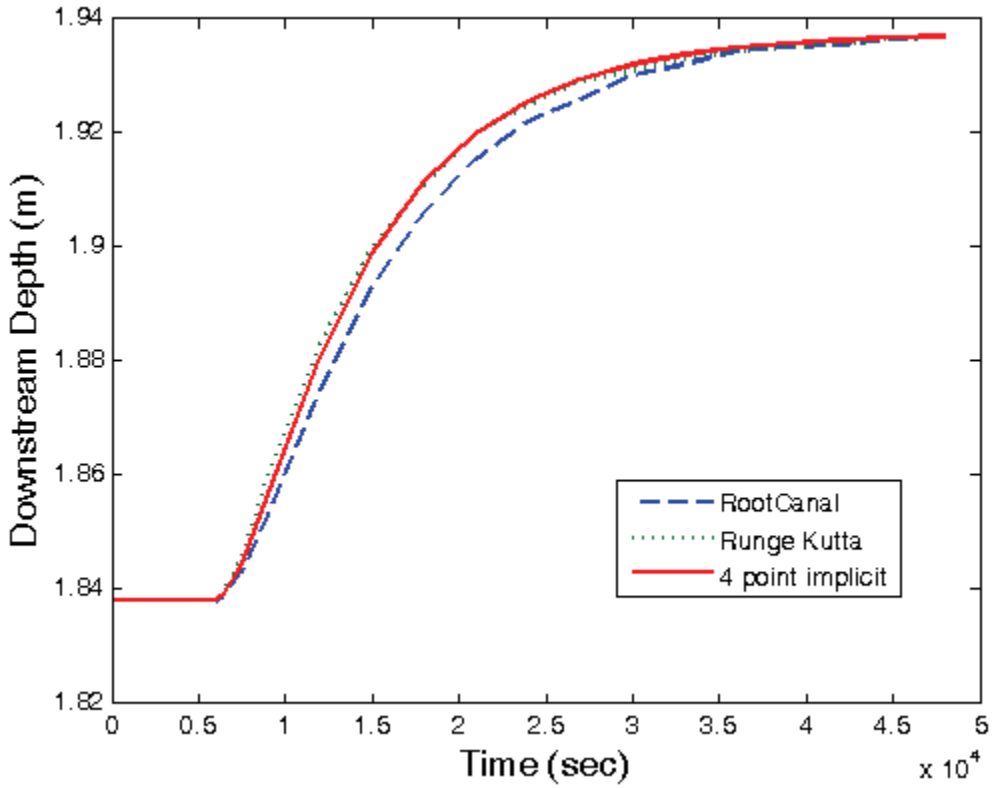

Figure 3. Downstream Flow Depth for a Flow Change from 8.5 m^3/s to 10.2 m^3s

The Figure indicates that the water depth results from the RootCanal model lag slightly behind the Matlab model results. The maximum percentage difference in water levels at any time during the interval shown in Figure 3 is less than 1%, and as both models reach steady-state conditions, the water surface levels converge to the same value. In a second test, the water flowing into the reach was set at 8.5 m^3/s (300 cfs), then at 100 minutes, the flow was reduced to 5.7 m^3/s (200 cfs), and finally at 1300 minutes, the flow was increased to 7.1 m^3/s (250 cfs). As indicated in Figure 4, the results are similar but the downstream water level from the RootCanal model again lags behind the downstream water levels from the Matlab models. The maximum difference between the two final water levels is still less than 0.5%.

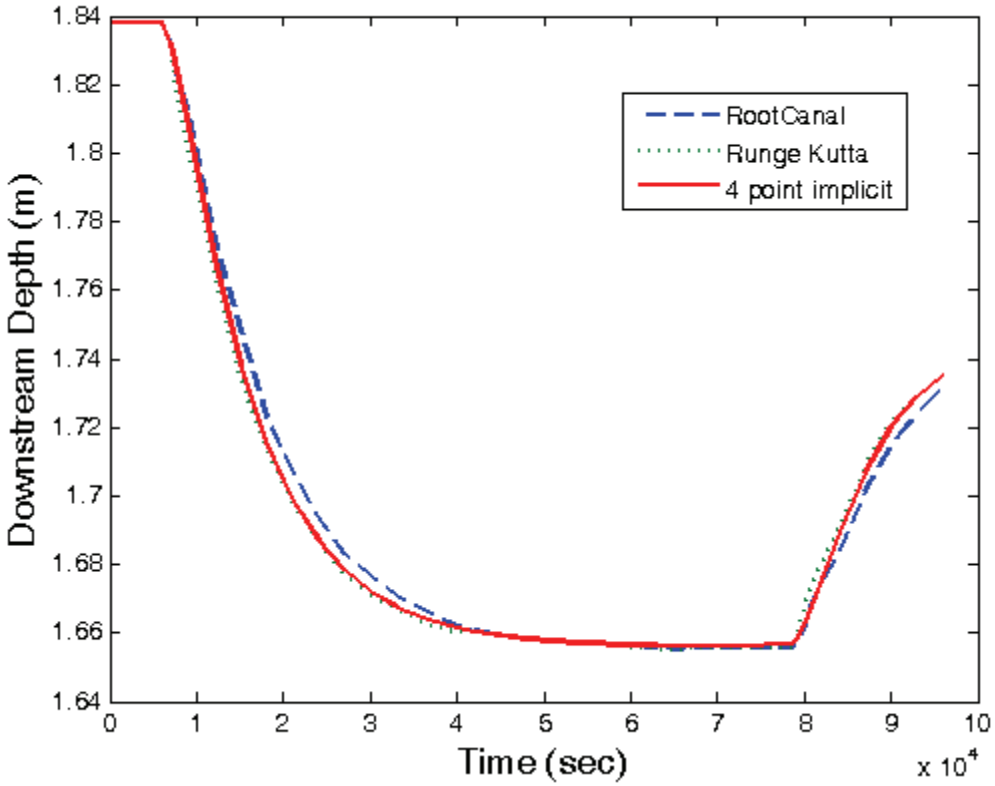

Figure 4. Downstream Depth for a Flow Change of 8.5 m^3/s to 5.7 m^3/s to 7.1 m^3/s

Finally, the inflow was set to 4.7 m^3/s (166 cfs) and the outflow was reduced to zero. Figure 5 once again demonstrates a similarity in response between the models. The downstream water level from RootCanal tracks the Runge Kutta method but it lags the Matlab 4-point implicit downstream water level. The graph indicates that the difference increases as the model is run for longer periods. The modeling error that exhibited the differences in the preceding tests compounds in this case because the canal does not reach a steady-state condition.

Considering that the major goal in canal automation testing is to obtain steady-state conditions, this final test only serves to show that some problems can occur. There is no practical application for this test.

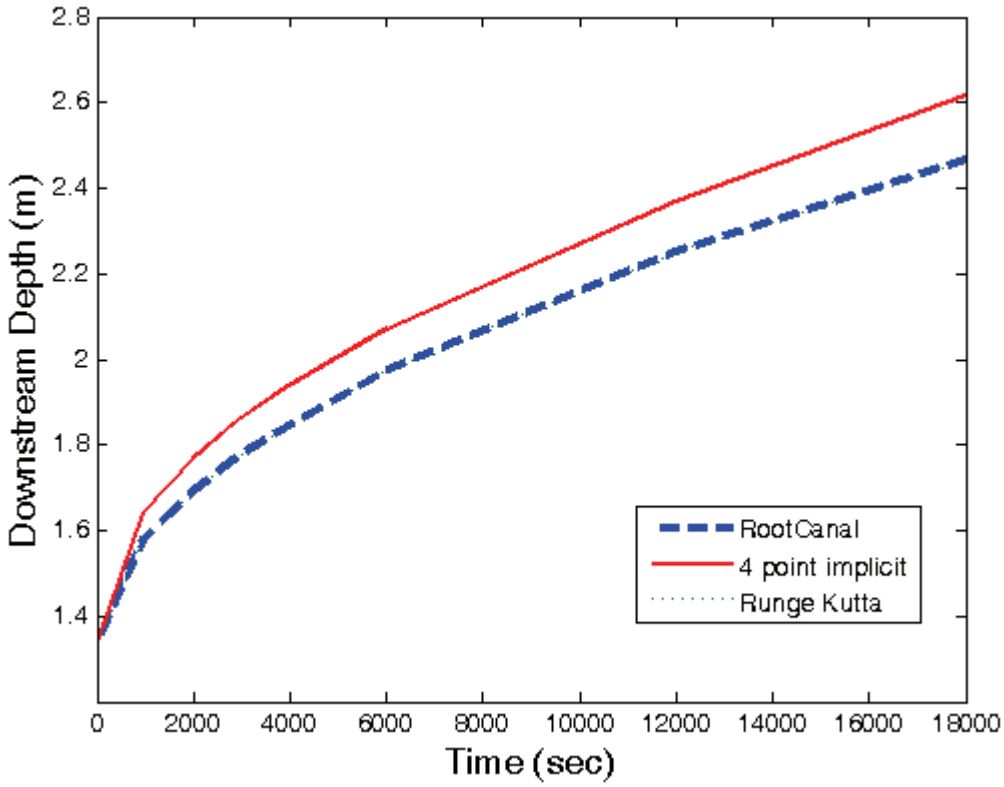

Figure 5. Downstream Depth for a Flow of 4.25 m^3/s. Flow Is Dead Ended at the End of the Reach. Note: RootCanal and Runge Kutta results are plotted on top of each other.

SUMMARY AND CONCLUSIONS

The Matlab methods presented above have demonstrated that they can be used to predict canal hydraulic response. The slight differences between the RootCanal and Matlab models are not significant for the tested cases, especially considering the fact that there will always be differences between model results and the field measurements. The similarities between the response of the models illustrate that the Matlab models can simulate canal behavior and can be used in gate control algorithm testing. This gives a control engineer a convenient tool for testing control algorithms for canal regulation. More extensive tests on this model may reveal further limitations, but the model is demonstrating that it can simulate canal hydraulic behavior.

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CANAL MODERNIZATION, MEASUREMENT, AND REMOTE MONITORING TO MANAGE DISTRICT-WIDE WATER USE EFFICIENCY AND EFFECTIVENESS

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Lupe Chavez[3]

ABSTRACT

Consolidated Irrigation District (CID) has worked to implement Efficient Water Management Practices (EWMPs) as adopted by the Agricultural Water Management Council, including measurement of diversions and deliveries; automation of canal structures; construction and operation of water supplier spill and tailwater recovery systems; and optimization of conjunctive use of surface and groundwater. Specifically through this Water Use Efficiency project, CID applied Supervisory Control and Automated Data Acquisition (SCADA) technology to monitor performance of the irrigation delivery systems to improve both the efficiency (water conservation) and effectiveness (the ability of the systems to respond to the irrigator's demands for water). The SCADA systems improved measurement and accounting of water diverted, applied, and recharged in the CID area. Funding was obtained by CID from the California Department of Water Resources (DWR).

The Canal Modernization, Measurement, and Remote Monitoring Project (Canal Project) was the first step to modernize facilities at CID. This first step was a pilot project to design, install, and test field data acquisition platforms and office hardware and software necessary to capture, manage, and apply the real time field data on a portion of the CID system. The project developed an approved approach for instrumentation of the CID system; prioritizing future sites for field instrumentation and automation; and served to initiate development of a fully operational Supervisory Control and Automated Data Acquisition (SCADA) system.

INTRODUCTION

Consolidated Irrigation District (CID) is working to implement Efficient Water Management Practices (EWMPs) as adopted by the Agricultural Water Management Council, including measurement of diversions and deliveries; automation of canal structures; construction and operation of water supplier spill and tailwater recovery systems; and optimization of conjunctive use of surface and groundwater. Specifically through this Water Use Efficiency project, CID applied Supervisory Control and Automated Data Acquisition (SCADA) technology to monitor performance of the irrigation delivery systems to improve both the efficiency (water conservation) and

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effectiveness (the ability of the systems to respond to the irrigator's demands for water). The SCADA systems improved measurement and accounting of water diverted, applied, and recharged in the CID area. Funding was obtained by CID from the California Department of Water Resources (DWR) Proposition 50 Water Use Efficiency Grant Program.

BACKGROUND

The 2007 Upper Kings Integrated Regional Water Management Plan (Upper Kings IRWMP) and 2009 CID Groundwater Management Plan identified overdraft conditions in the Kings Groundwater Basin. CID is one of 28 members of the Kings River Water Association that diverts and beneficially uses Kings River Water for agriculture and municipal uses. CID also recovers water released from Pine Flat Dam that intended to enhance and preserve fishery resources below the dam. Opportunities for diverting uncontrolled flood water for groundwater recharge exist and such diversions need to be measured. The cities in CIDs jurisdictional area are growing and increased development relies on groundwater rather than surface water. The cities are partnering with CID to develop additional recharge to mitigate for groundwater impacts of new development. There is a need to better account for all diversions (historical rights, flood flows, fishery releases, water banking) and for recharge of surface water to groundwater through conjunctive use operations.

PROJECT DESCRIPTION

The Canal Modernization, Measurement, and Remote Monitoring Project (Canal Project) was the first step to modernize facilities at CID. This first step was a pilot project to design, install, and test field data acquisition platforms and office hardware and software necessary to capture, manage, and apply the real time field data on a portion of the CID system. The project developed an approved approach for instrumentation of the CID system; prioritizing future sites for field instrumentation and automation; and served to initiate development of a fully operational Supervisory Control and Automated Data Acquisition (SCADA) system.

PROJECT GOALS AND OBJECTIVES

CID developed the canal modernization, technical support project, with an overall goal to begin implementation of the monitoring part of a SCADA system with automated water delivery measurement and to assist in conserved water documenting, conjunctive use, and groundwater banking benefits.

The canal modernization project objectives were to:

- Compile and map information on the CID's conveyance systems;
- Design, install, and test water measurements and recording devices;

- Establish a base automation and measurement system to support evaluation of potential water savings, verify water conserved, and quantify groundwater recharge; and
- Develop an implementation plan for automating system operations.

SCADA systems have two major components, measurement and automation. Measurement is to obtain data to support operational decisions and to document operating results. Automation involves installation of equipment to communicate and control the use of water delivery facilities from a remote location (CID offices).

The first step for CID was to evaluate, select, and begin instrumenting priority locations within the delivery system to collect data and improve measurement of key parameters (flow, stage). The SCADA system will be used to "benchmark" and document existing operations, and subsequently, to identify how water can be saved through improved systems operations. SCADA has a high initial investment, but over the long term, SCADA can improve operations, reduce operating costs and improve operating efficiency. Due to the high initial costs, this project is the beginning of a multi-phase effort to instrument measurement sites and automate operations; future funding may allow for installation of additional instrumentation and automation equipment at prioritized sites.

During the Project, an adjustment was made to shift some funding from developing an implementation plan to installing recording devices in the field. The main reasons for the adjustment were because the local compilation of the map information on CID's conveyance system was initiated prior to the grant award contract, and the development team quickly reached a consensus on the priority field sites to instrument. CID also recognized it was important to install several SCADA components at key locations for the Board of Directors to see the remote monitoring system working prior to investment in subsequent phases.

DESCRIPTION OF BENEFITS

The Kings River watershed and the Kings River groundwater basins were the targeted areas to receive benefits from the project. The program is intended to attain the following benefits:

1) Improve performance of the irrigation delivery system for both:
   a) Efficiency (water conservation); and
   b) Effectiveness (the ability of the system to respond to irrigators' demands for water).
2) Implement Efficient Water Management Practices (EWMPs) adopted by the Agricultural Water Management Council, including:
   a) Measurement of diversions and deliveries;
   b) Automation of canal structures;

c) Construction and operation of water supplier spill and tail water recovery systems; and
d) Optimization of conjunctive use of surface and groundwater.

Specific benefits realized from implementing this project include the following:

1. Several priority water measurement sites were automated; Staff does not need to travel to sites to record water measurement.

2. CID Staff developed a method to advance automation in the district by forming a working group of District Operating and Management Staff, a local SCADA System Integrator, a District Engineer and a Project Consultant. Future projects can be approached using the collaborative team.
3. Continuous data records for each site; data gets delivered into Excel file for reporting.

4. Information from remote site is obtainable through I-Phone and laptop computers.

5. Improved safety; alarms are now used to notify operation staff of possible flooding and spills.

6. Reduced staff time to monitor canal system during wet periods.

7. Automated measurement reduces pond overfilling and spills near the end of the conveyance system.

8. The success of this project has encouraged the Board of Directors and District Management to consider implementing additional SCADA systems equipment.

TASKS AND STATEMENT OF WORK

Pre-project conditions and baseline data were reviewed and considered to develop a conceptual model of the preliminary system design, which was called the “first-phase” SCADA system. This included preparing GIS maps; assessment of the CID's system operations under a range of hydrologic conditions; definition of the water budget components; and development of final objectives for automated monitoring. Rather than develop a thick, heavy report to place on a shelf at CID, the technical team approach was to optimize grant funding towards instrumenting priority sites. A list of priority sites was developed and then prioritized for implementation of the “first-phase” of SCADA system, based on the available DWR grant funds and local CID resources. The technical team visited each potential, high priority field site and discussed SCADA improvements for each location. Installation and site development costs and equipment specifications were then prepared. Procurement was consistent with CIDs local plans and policies. The resulting SCADA system installed provides for collection of data that is useful and supports development of a hydraulic model of the CID systems. Developing a hydraulic model was not part of the scope of this work plan, however, the technical team considered how each field site, once instrumented, could provide information to a

hydraulic model. The following tasks were undertaken in three related groups to meet the projects objectives and to allow tracking of grant funds in accordance with project tasks.

- Group 1 Tasks
- Task 1: Contract Administration and Project Management
- Task 2: Develop Geographic Information Systems (GIS) Base Map and Collect Baseline Data.
- Task 3: Develop a Conceptual Model and Preliminary System Design
- Group 2 Tasks
- Task 4: Specification and Acquisition of Equipment
- Task 5: Installation and Testing
- Task 6: Data Acquisition, Collection, Reporting and Analysis
- Group 3 Tasks
- Task 7: Technology Transfer and Training
- Task 8: Grower Outreach/Public Information
- Task 9: Development of Long Term Strategy and Final Report

The technical team concluded the project could make use of the GIS information developed as part of a separate effort that was completed prior to the start of this project. The water control and facilities base map provides a geo-referenced link to other available CID data sources and allowed CID to further describe and document the CID system. CID staff added secondary canal lateral information (attributes) into the data base. ESRI's ArcPad was used to draw in the alignment of over 200 laterals into the GIS data layer using aerial photo as a background. Once all of the District features were entered into the GIS, KRCD linked the physical features to the District's STORM database. The map views developed for the water control and facilities base map are linked to the available water monitoring and measurement information by station ID and are available for use in the SCADA base station views.

Collectively, the GIS base map and linked information are used to manage and document and catalogue water control and conveyance facilities and their condition. A component of the project was to ensure the SCADA system design was a conceptual model of the CID system and operation. The Historical operations data, staff expertise and the reports listed in the annotated references provide the basis for the conceptual model.

PRELIMINARY SYSTEM DESIGN — "FIRST-PHASE" SCADA SYSTEM DEVELOPMENT

Following the project Kick-Off meeting to implement the project, a technical team approach was implemented that would work in coordination with a local SCADA system integrator. A project meeting was held on December 11, 2008 to decide on the high-priority monitoring and measurement sites for initial installation phase. Initially, 21 sites were identified for possible SCADA system improvements. An initial site visit with the SCADA System Integrator, District Operating Staff, and Consultant assisting with the DWR Ag WUE grant, was conducted for 12 of the 21 sites. Based on the field inspection and initial cost estimate, approximately 4-6 sites were identified as within the realm of

the budget for SCADA instrumentation. "First Phase" SCADA installation included a base station, software, communication towers, and monitoring sites. The monitoring sites were constructed in the non-irrigation season to avoid disturbing the canals during water delivery operations. The field visit of the priority sites was to refine the design and the purpose for each site; and to prepare initial cost estimates for site improvement, construction and equipment costs. The need for radio repeaters was also evaluated. Concepts in Controls, the SCADA Systems Integrator selected by CID, prepared a preliminary cost estimate in cooperation with CID Staff, dated January 21, 2009. The technical memorandum "First Phase Supervisory Control and Data Acquisition System for Monitoring and Measurement Instrumentation" was prepared to document the priority sites and costs, help CID plan budgets, and issue work orders for staff. Once the priority sites and cost estimate was obtained, CID Staff coordinated the timing of the field work with the SCADA System Integrator and worked with the DWR regarding revising the schedule.

A map of the existing and proposed facility locations are shown on the GIS base map, Figure 1. Prior to installation of instrumentation, about half of the sites identified required design and construction estimates for modifications to structures at the measurement site. Due to the cost of site preparation and as a result of budget constraints, the SCADA equipment needs for the whole district will be phased in over time. Ultimately, the sites were pared down into two sets of priority sites listed below. The six sites numbered 1, 2, 3, 5, 6A, and 6B were completed over two periods. First, sites 1, 2, and 3 were completed. Sites 5 and 6 were completed in the following year as the budget allowed (Site 6 had two locations that were instrumented). Photos of the six sites are included as figures in this paper.

No.	Site	Purpose
1	District Yard Office	Display, record, and control water level and flows from district yard office.
2	CID Headgates [Main District Diversion of Surface Water]	Monitor and record water level and gate position for District water budget and operations; remotely control gate position to be considered and added under future funding opportunities.
3	Fowler Switch Canal Head gate, Mule Weir	Monitor and record water level to obtain flow information for water balance.
4	C&K Canal at Cole-Slough Head gate was considered as a site for radio repeater.	Site 4 was too expensive to instrument under this project; the site was considered for location of a radio repeater, if needed, for communication between Site 1 and Site 2.
5	Fowler Switch Divide, Elkhorn Canal	Monitor and record water level and provide flow measurement for effective District water management. An added function and benefit is to provide an alarm for Fowler Switch during operations.
6 - A	Head of Harlen Stevens Canal, which is also the End of the Fowler Switch canal	Monitor water level and provide flow measurement at Harlan Stevens Canal at Minniwawa Ave. for effective District water management.
6 - B	Harley Stevens Canal at Head Gate, which is the Head of Willow Spill	Monitor water level and flow measurement at Harlan Stevens Canal at Head Gate for effective District water management.

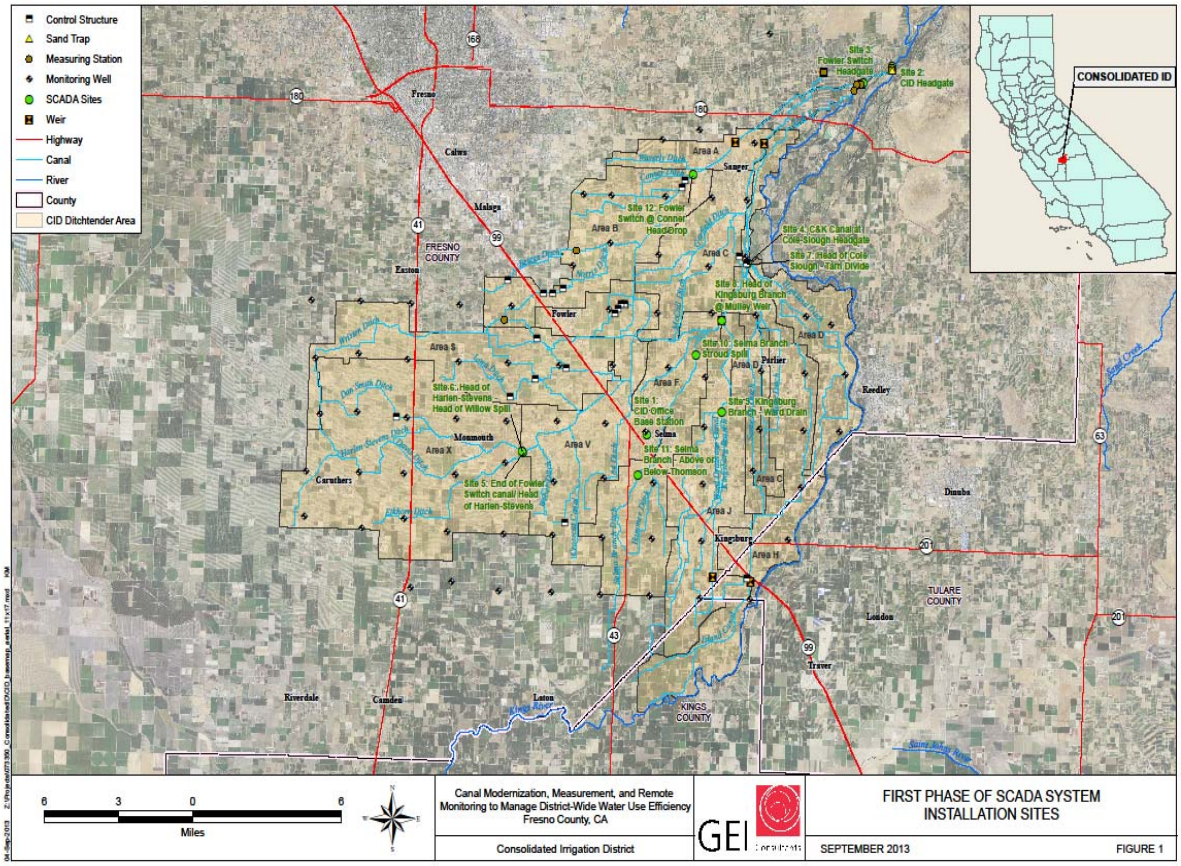

Figure 1. Proposed facility locations and GIS basemap.

Figure 2. Site 1 - Yard Office Site - Removal of Old Antenna

Figure 3. Site 1 - Yard Office Site - Installation of New Radio and Antenna

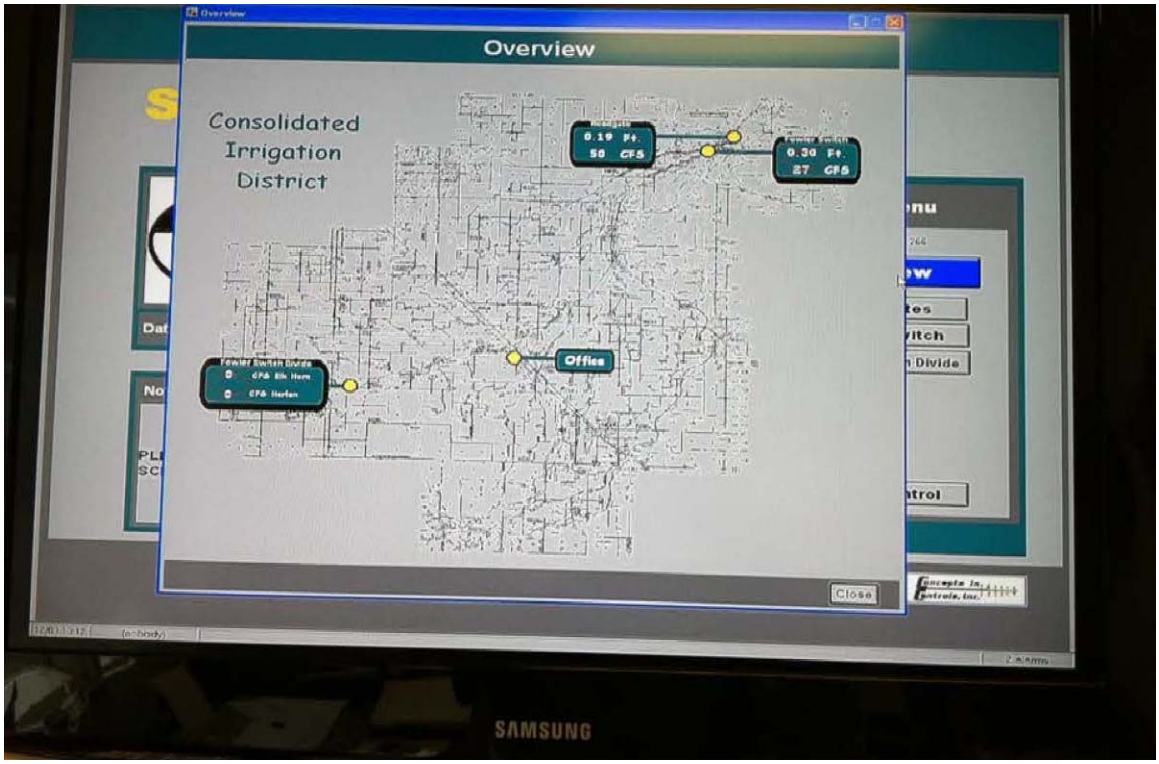

Figure 4. Site 1 - Yard Office Site - Monitoring Screen CID Overview

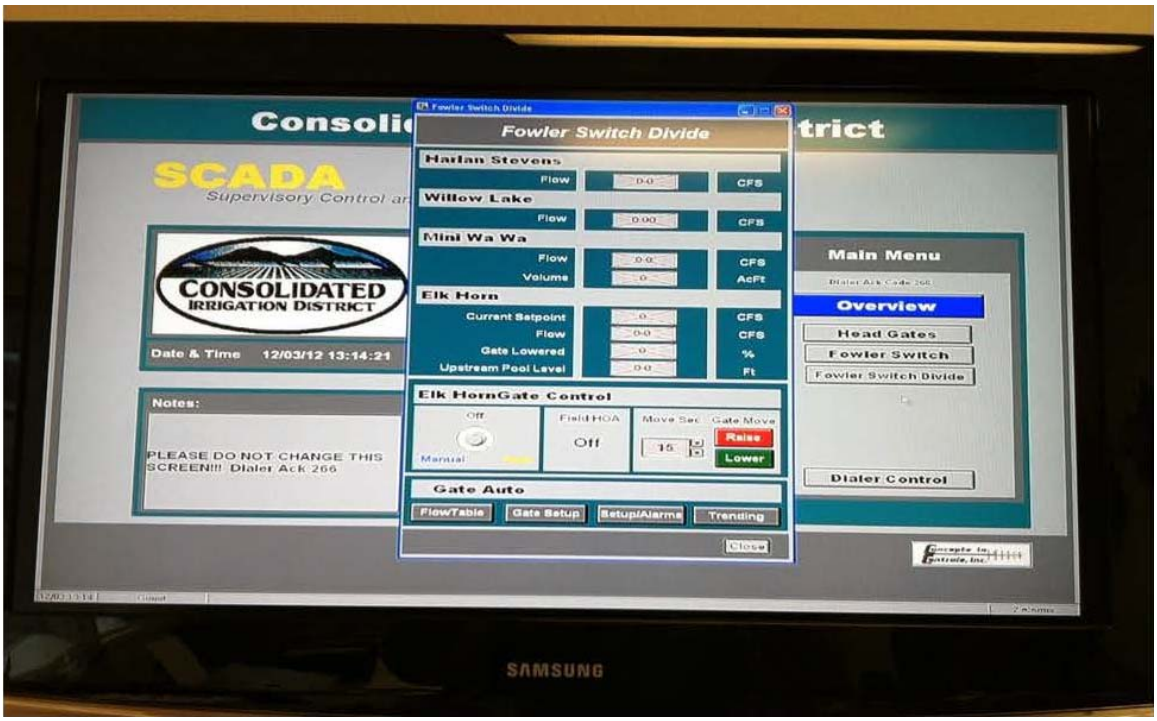

Figure 5. Site 1 - Yard Office Site - Monitoring Screen Fowler Switch Divide Site No. 5 Elkhorn Canal and No. 6 Harlan Stevens Canal

Figure 6. Site 2 - CID Headgate - Installation of Radio and Antenna

Figure 7. Site 2 - CID Headgate

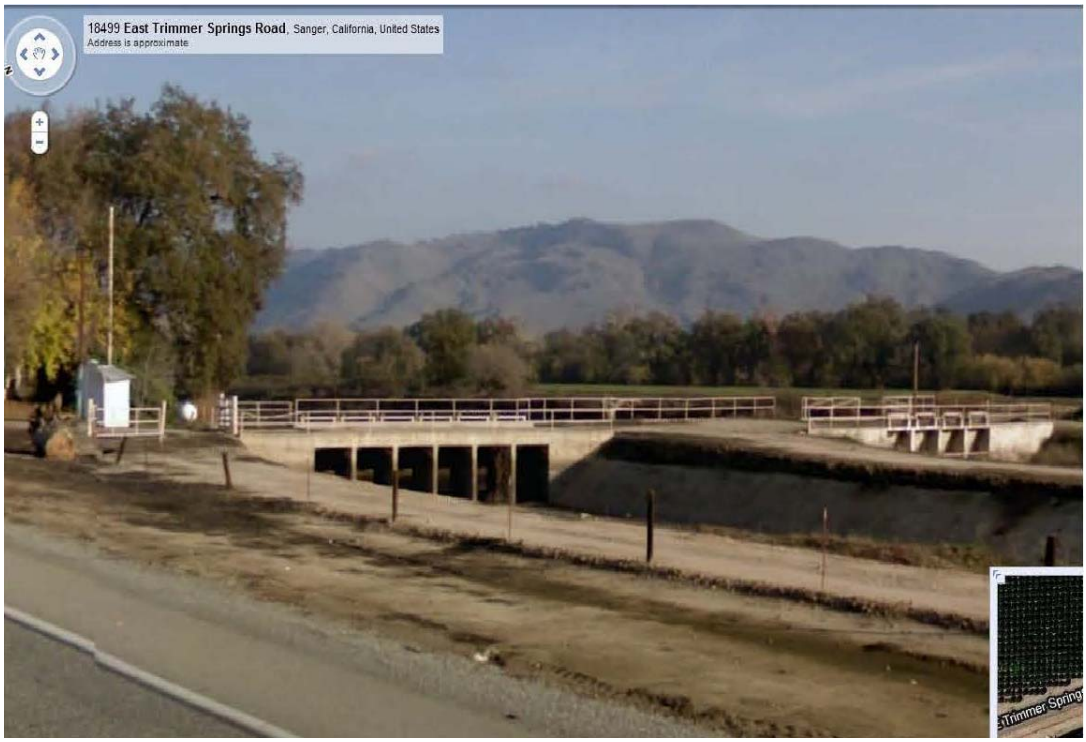

Figure 8. Site 3 - Fowler Switch Headgate - Prior to Construction

Figure 9. Site 3 - Fowler Switch Headgate - View of Truck during Installation

Figure 10. Site 3 - Fowler Switch Headgate - View of Boom during Construction

Figure 11. Site 5 - Elkhorn Canal - Bank View Prior to Construction

Figure 12. Site 5 - Elkhorn Canal - Canal View Prior to Construction

Figure 13. Site 5 - Elkhorn Canal - Canal View after Construction

Figure 14. Site 5 - Elkhorn Canal - Antenna View after Construction

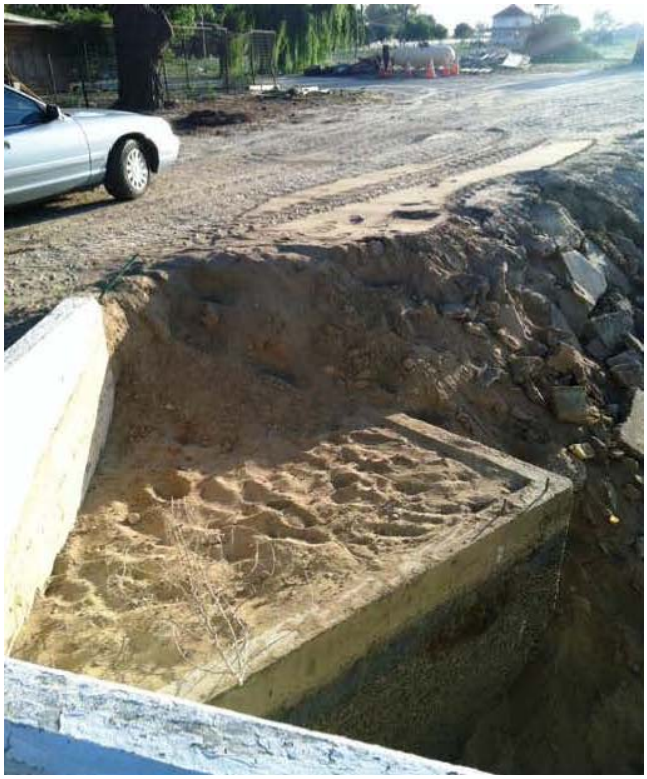

Figure 15. Site 6 - Harlan Stevens Canal - Below Minniwawa Avenue Prior to Construction

Figure 16. Site 6 - Harlan Stevens Canal - Below Minniwawa Avenue after Construction

SPECIFICATION AND ACQUISITION OF EQUIPMENT

The District's approach included obtaining a cost estimate from a local SCADA Systems Integrator for purchase and installation at the priority sites. CID coordinated the construction with the SCADA Systems Integrator to further prioritize and plan how to best use the mix of DWR grant funds and local resources. The planned installation was a coordinated effort between CID and Concepts in Controls, with CID providing in-house labor for all final site design or modification and construction. After preliminary costs were obtained, CID decided to utilize the grant funds to obtain the equipment necessary to implement the highest priority sites rather than expend resources on consulting to complete an assessment for an idealized network design. The District Board obtained the greatest benefit by installing equipment to demonstrate the SCADA benefits and build community support for further expenditures in subsequent phases. The intent was to also allow the CID Staff to gain experience, test equipment, develop skills and plan and budget for additional SCADA equipment in subsequent phases. A team approach of CID Management, CID Operators, Consultants, and the SCADA System Integrator was used to complete the project and can be used in the future for additional SCADA improvements.

The SCADA system equipment is non-proprietary and open architecture so the system can be expanded. It uses standard Modbus communication protocol which allows for maximum interchangeability of parts with other equipment manufactures. The office configuration used the existing CID radio mast and building adjacent to the antenna to house radio and computer equipment for the SCADA system. Access to the SCADA information screens and field data will be via a connection to the main CID office Ethernet system using wireless communications.

INSTALLATION, TESTING, TRAINING DATA ACQUISITION, COLLECTION REPORTING AND ANALYSIS

Base station installation and testing began in 2010. The SCADA system integrator completed installation of base station equipment. During the latter part of 2010, the SCADA system integrator completed installation at sites #1, #2, and #3 and CID began to use the base station and collect information. CID staff also began initial reporting and analysis of the collected data.

One of the main reasons for failure of SCADA systems is related to a lack of operator training after installation. Technology transfer was accomplished by the SCADA systems integrator by engaging CID staff in the project after each step of the equipment installation. The in-kind local match costs were related to these activities with the majority of the "in-kind" staff costs for site preparation, construction and installation of the equipment. CID has been successfully operating the SCADA system since the base station was first installed two years ago.

OUTREACH

Two grower outreach meetings were held to communicate the progress made installing the base station, collecting data remotely, sending control signals to the head gate, and for preparing reports. Meetings were held on Thursday, February 17, 2011 at 1:30 pm and Wednesday, February 23, 2011 at 1:00 pm. Additional project progress reports were made to the Board of Directors at their regularly schedule and noticed public meetings.

ACKNOWLEDGEMENTS

GEI provided project management and support services throughout the project delivery. CID led the field effort and partnered with Concepts in Controls to develop specific site improvements and install equipment. Concepts-in-Controls serves as the CID selected systems integrator and all work was conducted in accordance with the purchase orders and requisitions issues and local acquisition policies and procedures. CID's engineering support contractor, Summers Engineering, also supported the project and participated in meetings to specify priority sites. KRCD provided GIS support services at the beginning of the project.

CONCLUSIONS

The project objectives to design, install, and test field data acquisition platforms and office hardware and software necessary to capture, manage, and apply the real time field data on a portion of the CID system were met utilizing the state grant and local resources committed to the project. The initial steps included:

- Map information was compiled for the CID conveyance systems;
- Initial SCADA sites were designed, installed and tested, staff was trained and the system is operational;
- CID accomplished two years of experience collecting baseline data to evaluate potential water savings, document operational benefits, verify water conserved. These first steps serve to:
    - Document benefits of current operations or any improved operations (e.g.; construction of additional recharge ponds);
    - Account for diversions that could increase the yield of the conjunctive use operations (Kings River diversions that recapture fishery flows, capture flood flows),
    - Document beneficial use of CID water rights for recharge, agricultural, domestic and municipal uses; and
    - Support measurement of any water banking operations.
- A near term priority implementation plan for instrumenting and measuring priority locations was developed.

The project provided a critical first step in the long-term, multi-phase effort to instrument the CID canal network, improve measurement and verify conserved savings and

deliveries. The instrumentation procured and installed will help the Districts apply an adaptive management strategy and will support the ongoing efforts to improve the distribution systems efficiency.

The project met the objectives by serving as a "proof-of-concept" and demonstration project. Early on in the project and as a result of Board of Directors, staff and grower input, it was decided that it was important to prove the concept and install and test as much equipment as could be obtained with the available budget under a preliminary plan, document the benefits and opportunities of SCADA technology, demonstrate near term success, allow time for staff to become familiar with SCADA and develop the Board awareness and grower support to further invest in SCADA instrumentation and measurement, while moving towards automation at key locations. Additional plans are being developed by CID to continue to move forward and implement additional SCADA facilities. CID, like all irrigation districts, faces budget constraints. Further facilities will need to be budgeted for over time. This project demonstrated results and identified them using a "team" of knowledgeable district staff operators, District Engineers, Consultants, and a SCADA system integrator provided a practical approach that will support long term planning and development to prioritize SCADA systems instrumentation and to automate facilities where cost effective.

REFERENCES

KRCD, 1993. Consolidated Irrigation District Surface Water Study. Fresno, CA. Provided a conceptual model of canal operation losses (recharge to groundwater) and evaluated individual canal reaches.

WRIME, 2005. Hydrologic Modeling in Kings Basin, A White Paper. Prepared for the Upper Kings Water Forum. Sacramento, CA. A non-technical publication to support the Upper Kings Water Forum in defining the purpose, need, and approach to developing modeling tools.

A CRITICAL ANALYSIS OF THE EPA WATERSENSE® WBIC PROGRAM

Michael Davidson[1]

ABSTRACT

There are 40.5 million acres of irrigated turf lawn in the United States making grass more widely irrigated than the eight following irrigated crops combined (Diep, 2011). The U.S. Environmental Protection Agency's (EPA's) WaterSense® program for weather-based irrigation controllers (WBIC) is the U.S. Government's response to conserve water and energy in the household landscape sector. It is designed to promote and enhance the market for commercial and residential irrigation controllers that create or modify irrigation schedules based on landscape attributes and real-time weather by labeling efficient irrigation system control technologies (EPA, 2009). "The EPA anticipates, in full consideration of the research studies on weather-based controllers, realizing, on average, at least 15% saving of applied irrigation water after installation of weather-based irrigation controllers " (EPA, 2009), and further, that the annual net savings equate to nearly 120 billion gallons of water and almost $450 million (EPA, 2011). This paper asks if the EPA can empirically state and reliably infer from the data provided by the eleven research studies on weather-based controllers, cited as evidentiary in the WaterSense Specification (EPA, 2011), that WBICs save ≥ 15% more water than traditional controllers for its nationally, targeted population. A meta-analysis of these studies shows that the authors of the cited studies did not design experimental studies for the purposes of testing hypotheses that would substantiate or disprove EPA assumptions. This paper provides strong evidence that the data of the studies derived by the EPA cannot be generalized for the purpose of providing a reference point for the EPA WaterSense® program and the assumptions and predictions of the EPA regarding the potential savings of WBICs are invalid.

INTRODUCTION

The rationale for the development of the WaterSense specification is based on the assertion of the EPA that "irrigation demand is the single largest end use of water in the urban sector in California" (Mayer, DeOreo, Hayden, & Davis, 2009, p. ix), "forecasted to reach 58% by the year 2020" (Hunt, et al., 2001, p. 4). Moreover, "as much as half of this water is wasted due to evaporation, wind, or runoff often caused by improper irrigation system design, installation, maintenance, or scheduling" (EPA, 2011, p. 1). The U.S. Environmental Protection Agency's (EPA) WaterSense program is designed to address irrigation scheduling for residential and light commercial applications by labeling efficient irrigation system control technologies. Over a period of four years the EPA, in collaboration with irrigation controller manufacturers, water utilities, and irrigation industry representatives, developed the *WaterSense Specification for Weather-Based Irrigation Controllers*, releasing the final iteration in 2011 (EPA, 2011).

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Irrigation controllers are to be tested in accordance with the Smart Water Application Technologies™ (SWAT) test protocols for climatologically based controllers utilizing climate data and some form of evapotranspiration data as a basis for scheduling irrigation. The SWAT protocol established the method by which controllers are tested and provides two output measures of performance: irrigation adequacy and irrigation excess. Irrigation adequacy is a measure of how well the plant's consumptive water needs are met and irrigation excess is a measure of water applied in excess of the plant's landscape consumptive needs (EPA, 2011). Required supplementary features are primarily utilitarian.

The foundation for the WBIC specification is the result of eleven research studies conducted from 2001-2009 on the efficacy and efficiency of weather-based controllers. This paper examines the explicit assertion by the EPA that, "In full consideration of the findings of these (eleven) numerous studies, WaterSense anticipates seeing overall water savings of approximately 15 percent after installation of weather-based irrigation controllers" (EPA, 2011, p. 12). This statement, and the exchange between the author of this paper and the WaterSense product lead in the Conclusion portion of this paper, imply a causal relationship between the operation of WBICs and water, cost and energy conservation. In order to support the assertion that WBICs cause reductions in water, cost and energy it is necessary to carefully and thoroughly analyze the data upon which the assertion rests. The EPA does not conduct its own testing or evaluations of weather-based controllers. It relies on third party studies (Tanner, 2012).

This paper posits that the research studies which comprise the foundation for the inferences derived by the authors of the EPA WaterSense program do not provide a degree of probability that justifies the rationale for the EPA WaterSense program. In particular, this paper examines the conceptual frameworks of the research study within the particular context of generalizability. Generalizability asks to what extent a sample of measurement generalizes to a universe of measurement. The principal result of a generalizable study is a set of estimated random effects variance components for the universe of admissible observations. In other words, were the research studies appropriately and purposefully constructed for the purposes of the EPA WaterSense program and can we extend the results of the study to the population at large? The conclusion of this study is that neither question is satisfied and the EPA WaterSense program is without a valid foundation.

The EPA declares that ≥15% of household water can be saved by using WBICs instead of standard time-based controllers, and asserts that this is a generalizable inference that can be derived from the research studies and applied to the 12,825,000 households in the United States who are candidates for WBICs (EPA, 2011). This paper will show that although the studies cited by the EPA for WBICs explicitly examine the effectiveness and/or efficacy of weather-based controllers, the implicit objective of these publicly funded projects is to reduce irrigation water in specific service areas. The majority of the studies cited target the highest water users in the service area under study and this selection of non-random sample sets partially explains why their results are not generalizable. In addition, no empirical baseline was provided in any of the studies to

determine pre-intervention water usage in landscape irrigation. Approximations were utilized to estimate irrigation water consumption based on assumptions of percentages of total household or commercial property water use. Inferences that the studies are generalizable are invalid because the data collected and analyzed in the studies served different objectives and are components of a different conceptual framework. The purpose of the studies was to utilize the WBICs as a tool to reduce water consumption within a limited population— not to determine if, when applied to the general population, WBICs reduce water consumption in the landscape. In other words, the studies were not conducted to serve as evidence that overall, national, water savings could be achieved by using WBICs.

STUDY APPROACH

The approach of this study is to conduct a meta-analysis of the research studies that serve as the foundation of the EPA WaterSense program with an emphasis on the generalizability of the studies. That is to say, can EPA WaterSense apply the results of the studies to the wider population it serves? The evidence of this analysis shows that the data are not reliable because water quantities were not empirically measured and that non-probability sampling was used in virtually all studies. Evidence will show that researchers engaged in judgmental sampling, deliberately selected their populations because of time or monetary limitations, in a minority of cases, or, in the majority of cases, because they were attempting to prove the efficacy of WBICs within a limited population. In no case, did the researchers randomly sample their populations and therefore, the results of their research cannot be used as generalizable to the entire population. This paper does not ascribe any normative values to the studies nor does this paper evaluate the individual studies beyond an examination of their generalizability for the objective population of the EPA WaterSense program. As part of this analysis it is unavoidable to touch upon the reliability of data and sampling bias in the studies within the context of the EPA objectives. Similarly, this paper does not address the details of the specification of the WBICs nor does it address the performance or robustness of the controllers except for illustrative purposes.

The paper is organized as follows: the second section of the paper provides a short narrative summary of each of the research studies, examining them for reliability and generalizability. The third part of the paper will present an explanatory table of the salient parameters of the studies; the fourth part of the study discusses the assumptions of EPA WaterSense and the calculations that were derived to justify the WaterSense program. The final section provides conclusions about the WaterSense program for WBICs and ends with recommendations for a different, more robust, research design that could lead to generalizable results.

THE RESEARCH STUDIES

In the 2009 Appendix A of the WaterSense Draft Specification for Weather-Based Irrigation Controllers Supporting Statement, the fourth assumption states that "large-scale, long-term studies have shown that on average, weather-based irrigation controllers

have the potential to save at least 20 percent of applied irrigation water" (EPA, 2009). In 2011, the anticipation of EPA WaterSense was to estimate water savings of 15 percent, based on the same eleven studies and an additional study of California WBIC programs (Mayer, DeOreo, Hayden, & Davis, 2009). The referenced studies are as follows:

Aqua Conserve (2002)

"Residential landscape studies using Aqua ET Controllers manufactured by Aqua Conserve were conducted in partnership with Denver Water, Denver, CO, and two adjacent water districts in Northern California: the city of Sonoma; and, the Valley of the Moon Water District, in 2001" (Addink & Rodda, 2002). The reported average water savings per participants in Denver was 21.47%; the average outdoor water savings per participant in Sonoma was 7.37%; and, the Valley of the Moon Water District average outdoor water savings per participants was 25.1% (Addink & Rodda, 2002).

The study concludes, that "Based on the results from these studies, homeowners who install an Aqua ET Controller can expect an average water savings of from 7% to 25%, with two of the three studies indicating 21% to 25% average water savings. Additionally, Water Districts whose customers install Aqua ET Controllers can expect a significant reduction in water demand" (Addink & Rodda, 2002)

Aqua Conserve provided a list of high volume water users interested in the study project to the Sonoma County Water Agency and the Valley of the Moon Water District. Aqua Conserve personnel installed controllers at: 27 residential sites in the City of Sonoma; 37 in the Denver metropolitan area; and, at 10 residential sites in the Valley of the Moon Water District. All controllers were equipped with temperature sensors. Water usage during 2001 was compared to pre-installation historic usage for two previous years for Sonoma and for the previous five years for Valley of the Moon. If excessive wilting of the grass or brown spots developed, the users could press a button and add an additional scheduled watering (Addink & Rodda, 2002). There was substantial variation in the results—some participants had extremely high water savings some had no water savings and even a few had an increase in water usage due to the addition of sprinklers and improper controller settings. Outlier data was included in the final calculations.

In summary: the manufacturer of the WBIC provided the agencies the list of candidates so the sampling of the population was not unbiased; high volume users were selected as candidates; participants were allowed to manipulate their controllers manually; and, participants were allowed to add or subtract sprinklers.

Aqua Conserve is no longer manufacturing or selling product (Aqua Conserve, 2011).

Aquacraft (2003)

The Aquacraft, Inc. 2003, research study consisted of ten controllers installed in Colorado of which nine were residential and one commercial. Seven of the participants volunteered for the study and three were selected based on their high water usage. Overall savings averaged about 20%, however, post-installation water usage increased at

four of the sites which was explained by researchers as sites where volunteers had historically under-irrigated (U.S. Department of the Interior, 2008).

In summary, while the results appear to be positive, it is important to note that seven of the ten sites were voluntarily selected and the remaining three were selected because they were high water users. Volunteers for the study were provided a free controller.

Aquacraft (2009)

The Aquacraft, Inc. evaluation of California Weather-Based "Smart" controllers was designed to maximize potential water savings so the targeted sample selected for the Northern California portion of the study were historically high outdoor water users who were identified by historic billing data (Mayer, DeOreo, Hayden, & Davis, 2009). In Southern California, the target sample group were "interested and motivated customers" (Mayer, DeOreo, Hayden, & Davis, 2009, p. xiv). This study is quite broad and reflected the efforts of a collaborative group of agencies: California Department of Water Resources; California Urban Water Conservation Council; Metropolitan Water District of Southern California (MWD); the twenty-six member agencies of MWD in southern California; a consortium of six water agencies in northern California; and, the East Bay Municipal Utility District. There were 2,294 sites in this study, 3,112 controllers. There were three distribution methodologies used: rebate and vouchers; exchange programs; and, direct installations.

In summary, the results indicated an overall savings in one-half the population but the results are not generalizable for the EPA population target because customers in the Aquacraft (2009) study were either paid to switch out old controllers or were selected because they were high water users.

Cbilling data

The Carlos experiment in Northern Nevada consisted of four treatments: intuitive irrigation; manually ET scheduled irrigation; manually ET scheduled irrigation with management training; and, ET satellite controlled irrigation. Preliminary results indicate a potential of 15-30% water savings using satellite technology. Estimates range from 50% to 70% of the total water supply is used for outdoor irrigation during the summer months and unpublished data suggests that in non-drought years residents typically apply anywhere from 2 to 10 times more water for landscape irrigation than is actually needed (Carlos, Miller, Devitt, & Fernandez, 2001). The study is a 4 x 2 factorial experiment with three replications in a completely random management design. The experiment utilizes localized data generated from weather stations to control the duration and frequency of outdoor irrigation. Weather station data are sent to a PC unit cellularly where ET_0 is computed then sent via satellite dish to an orbiting satellite. The satellite then beams the signal down to an irrigation controller individually located at the consumer's place of residence on a weekly basis. The controller opens the irrigation valve and automatically sets the duration and frequency of irrigation based on a pre-

assessed application rate and distribution efficiency of the irrigation system. The 2001 study does not report any results.

In summary, the Carlos study is scientifically robust but two issues make its results ungeneralizable for the EPA WaterSense program. First, the scope of the study is limited to the efficacy of satellite technology to manage landscape irrigation water, and, second, while the experiment is conducted randomly, each experimental unit is controlled for similar turf variety and uniform cultural and management practices.

Devitt (2008)

The Devitt study is a mixed landscape experiment conducted on 27 residential sites in Las Vegas to quantify water savings associated with satellite controllers (Devitt, Carstensen, & Morris, 2008). Seventeen sites were equipped with ET satellite irrigation controllers and ten sites were designated as control sites and retrofitted with non ET-based controllers—five received seasonal irrigation scheduling information and five received no educational information. Sites were selected based on an extensive evaluation of landscape plant materials, irrigation system performance, homeowner level of interest in participating, and the presence of tall fescue in the front yard. All homeowners in the control group were provided a two-page flier every three months on landscape water use and irrigation scheduling recommendations and tips. Electronic water meter-reading devices were installed on each residential water meter and irrigation was restricted to the hours between 10:00 PM and 5:00 AM. Water use (meter readings) at all residential sites, was compared with historical data for each site obtained from the local water purveyor. Indoor use was estimated by subtracting outdoor use (10 PM to 5 AM) from the total meter readings. Historical water use was for total water with no separation between indoor and outdoor use. The average water savings for all smart controller sites is reported to be approximately 20%, and individual savings ranged from 61.6% to -68.1% (U.S. Department of the Interior, 2008).

Results showed that 13 of the 16 ET Based controller sites saved water compared to four of ten of the non ET-based control sites. Statistical difference occurred between the control and ET based group (ET-based =+20% savings) ($p<0.05$).

In summary, the Devitt study was designed to examine the impact of WBICs in mixed landscapes and concluded that the landscape plant material was not negatively affected by the ET-based controllers, and 81% of the variation in the total outdoor use could be described by the total turfgrass area at each site. The results, then, are generalizable but only in conditions where there is a preponderance of tall fescue turfgrass in an arid environment, controlling for the dissemination of information on seasonal irrigation scheduling.

Irvine Ranch Water District IRWD (2001)

The objective of the Irvine ET Controller project was to study as homogenous group as possible to improve the validity of the findings. To that end, test sites were selected from

Westpark Village, a development located in the city of Irvine, California. Test homes were targeted as per traditional water conservation programs. That is, the top 20% water users were selected. In the study area, residents with average annual consumption exceeding 200 Hundred Cubic Feet (HCF), derived from three years of billing data defined the top 20%. The residents of these 509 homes were sent letters requesting study volunteers. Over 130 households volunteered to participate. " From these volunteers 40 homes were selected"(Hunt, et al., 2001, p. 13). Three household groups were selected: a test group; a reference group to account for externalities; and, a postcard group (residents who received postcards as weather conditions changed with suggestions for irrigation schedule adjustments). All treatment group households were surveyed prior to the retrofits to gauge their irrigation knowledge and practices and to gauge their receptivity and willingness to pay for this technology. All test groups were selected from among the top 23% water users in the development. On an absolute basis, when savings were estimated through a statistical comparison of weather-normalized consumption before and after retrofit, WBICs "were able to reduce total household water consumption by roughly 37 gallons per household per day, representing a 7% reduction in total household use or a projected 16% reduction in estimated outdoor use" (Hunt, et al., 2001, p. 36) . The authors infer that by targeting roughly the top third of homes in terms of water use (approximately 10,000 homes) ET controllers might be expected to save roughly 57 gallons per household per day, a reduction of 10% in total water use or 24% in outdoor use.

In summary, the authors conclude that the total potential savings are suggested for illustration purposes only and that the study is not designed to generate widely generalizable inferences. The sample set was restricted to the top 23% water users in the development.

Los Angeles Department of Water and Power LADWP (2004)

The LADWP weather-based irrigation pilot study was targeted at large multi-family residential (homeowner associations) and small commercial sites. The study was implemented during 2002 and 2003 (Bamezai, 2004). The authors posit that, to date, several studies have examined the effectiveness of weather-based irrigation controllers in single-family residential settings, but virtually none have systematically examined how these controllers perform in other types of settings with medium to large landscapes. All twenty-five sites in the study were professionally installed and programmed. To avoid implementation delays, the study did not randomize the assignment of sites to the vendors. LADWP staff identified potential commercial, industrial, institutional sites with significant landscapes by examining summer-winter usage differentials. LADWP staff then contacted these sites to inform them about the pilot program, and to solicit participation. At the time of selection, careful attention was paid to the general condition of the irrigation system. Sites with irrigation systems in significant disrepair or sites where significant alterations had been made to the landscape in the prior two years were excluded.

In summary, the LADWP study is generalizable only in conditions where WBICs are professionally installed and programmed in commercial settings.

Municipal Water District of Orange County MWDOC (2004)

In the summer of 2003, MWDOC was awarded a Proposition 13 non-point-source pollution control grant from the California State Water Resource Control Board to provide funding asistance for the installations of a new irrigation timer technology (Berg, Hedges, & Jakubowski, 2009). The study had two primary objectives: to capture pre- and post-Smart Time installation data for water quality and runoff flow for two neighborhoods; and, evaluate water savings on the same Smart Timers installed in the program. The "Orange County's Weather Based Irrigation Timer Rebate Reimbursement Program" examined water savings for the entire program area by single-family residences, water savings by commercial installations, runoff flow patterns during pre- and post-interventions, and water quality changes resulting from WBIC installations. In addition, the study examined water savings by season, brand of Smart Timer and type of installer. The program wide savings of single family residences was about 0.7 Hundred Cubic Feet (HCF)/month (about 18.3 gallons/day (gpd) or 0.0045 gpd/sq ft of irrigated area). This estimate is arrived at by calculating the total change in water use in cases where water use changed significantly (increased or decreased, α=0.05) and averaging the net change by all the Smart Timers (899) that were qualified for evaluation. However, the amount of water savings will increase, according to the authors, to 1.4 HCF/month (35.7 gpd) if the estimates are made by averaging the net water change (significant increase or decrease) by only those Smart Timers (460) that contributed to significant change in water use (Berg, Hedges, & Jakubowski, 2009). Program wide savings in commercial settings averaged 7.6 HCF/month (about 190 gpd; 0.004 gpd/sq ft irrigated area). In 30% of cases water consumption significantly decreased, 11% increased, 60% had no change. The authors identified three distinct trends in the single-family residences retrofitted with Smart Timers. In about 33% of the accounts, the water consumption significantly decreased (α=0.05) after installation of Smart Timers. In about 18% of the cases the water consumption increased statistically significantly after installation of Smart Timers. In nearly 50% of the accounts, water use did not change significantly upon installation of Smart Timers. The selection process for the 500 single family residences in the study area consisted of a marketing campaign of directly-mailed postcards, letters and two weekends of direct door-to-door marketing by Boy and Eagle Scouts. Following the marketing campaigns, the fifty-three interested residents contacted the rebate program, purchased and installed an approved WBIC and then filed a rebate program application with MWDOC. Participation was a bit over 10% of the neighborhood (Kennedy/Jenks Consultants, 2008).

In summary, the authors advise that that this study, notwithstanding its extensive production of data, is limited because: the data were not normalized for weather with advanced statistical modeling; the results obtained were not compared to a control set of similar participants; and, the weather data used in the study was found to be inaccurate due to malfunctioning weather equipment such that all data are currently being re-run (Berg, Hedges, & Jakubowski, 2009). The recommendations of the authors for further

study include; the need for periodic readjustment (of crop coefficients) due to seasonal changes; proportionate installation of WBICs in various ET zones; and, random population selection. They conclude that proactive early adopters of the WBIC technology do a better job overall of water conservation (Berg, Hedges, & Jakubowski, 2009).

Santa Barbara County Water District (2003)

The Santa Barbara County Water District program involved six agencies (Santa Barbara County Water Agency, City of Santa Barbara, Goleta Water District, City of Lompoc, City of Santa Maria, and, the Vandenberg Village Community Services District). Each agency developed a list of high-water using customers who served as the target audience for the ET Controller Program. Average water use for January and February and average use for July, August and September for the prior three years was determined for each customer. The average amount of landscaping at residential properties in the study area was about one acre and it was estimated that approximately 50 percent of the water used at a residence goes to the landscape. These averages were used to create a ratio of the difference between summer and winter to determine highest irrigation use. ET Controller Program brochures and letters from the water purveyor were mailed to the top 100 high water users from these lists for Goleta Water District and City of Santa Barbara and the top 25 for the other three agencies (Litton, 2003). A marketing campaign and phone campaign to attract the highest users was conducted and participants had to pay $144 for a 3 year service plan up front. Site visits (6 hours per controller) for pre-screened customers were conducted by staff members and included a Customer To Do list which provided information on the required repairs and installer contact information. The WeatherTRAK ET Controller technology was chosen for the ET Controller Program because a study conducted by Irvine Ranch Water District was assumed to provide "conclusive evidence" (Litton, 2003) that the WeatherTRAK controller supplied accurate irrigation scheduling by automatically creating a weekly irrigation schedule based on real time evapotranspiration (ET) data from local weather stations (Litton, 2003). Preliminary data indicated that customers were reducing their monthly water use by approximately 26%, with a high of 59% savings and a low of 8% savings. The author further noted that using the factory settings for precipitation rates in the WeatherTRAK controller does not result in reliable savings. On average, the WBICs were over watering turf areas and under watered areas with drip systems.

In summary, the Santa Barbara study is not generalizable because of biased sample selection, reliance on data from earlier, ungeneralizable studies, the absence of a reliable baseline, and the manipulation of the controller settings by customers.

Saving Water Partnership (2002)

The Saving Water Partnership study was designed to test the savings potential and customer satisfaction of four types of irrigation controller devices: ET controller and sensor; wireless and hardwired rain sensor; ET controller without a rain sensor; and, irrigation scheduling service (Smith, 2003). Participant selection was based on a

customer's potential to save water. The study participants (including controls) used an average of 375 gallons per day during the peak season above their average daily winter use and are considered very high users. This list produced 2,000 names. Half were invited to participate and the other half would be used to select controls. According to the study, the 20 participants who received the ET controller with a rain sensor realized the greatest water savings because these customers had a high savings potential. The study estimated that that about 5% of all customers have "a total peak differential of over 44,880 gallons" (Smith, 2003, p. 6) which represents the total amount of water used from May 15-September 15 above the average winter use. The study estimates that, approximately ,7,875 customers have the 44,800 differential and an automatic irrigation system. The prediction of the study, is that if this population "installed the ET controller with rain sensor, the Saving Water Partnership could potentially save 1.2 million gallons per day or 3 billion gallons during peak season" (Smith, 2003, p. 6).

In summary, the Saving Water Partnership study is generalizable in areas with high water usage and considered "very high users" (Smith, 2003). There is evidence of a strong correlation between high water use and potential water savings.

University of Arizona (2006)

This is a field study that evaluated water savings resulting from installation of weather and soil moisture based controllers. Data were collected at 27 residential sites in Tucson, Arizona during August 2004 to July 2006. Devices were installed by a landscape professional with support from manufacturer representatives. The participants consisted of volunteers and high water usage was not a selection criteria. Reported average water savings are 25% for the WBIC and 3.2% for a second WBIC and 4.3% for the moisture sensor WBIC. (U.S. Department of the Interior, 2008).

In summary, the reported success of this study can be traced to the selection of voluntary participants. This study contains a small sample size (27 homes) and does not cite independent third-party review as to the methodology uses and the soundness of the conclusions (Dukes, 2012).

THE RESEARCH STUDIES

Table 1. Research Study Parameters

Study	Target	Marketing Strategy	Scope	Comments
Aqua Conserve 2002	Manufacturer provided list of high volume water users	Direct, targeted approach by manufacturer	37 WBICs	Users allowed to make adjustments
Aquacraft 2003	7 volunteer subjects; 3 selected as high users	Initial phone calls and follow up to qualify	10 WBICs	Reported savings of 20%

Aquacraft 2009	High water users in half study	Web, word of mouth, agency letter	2,294 sites	Water savings for about one-half sample
Carlos 2001	Identical turf users	Unknown	Unknown	Satellite based technology
Devitt 2008	Uniform landscape planting, presence of fescue	Free controllers	27 WBICs	Variation of results due to preponderance of tall fescue grass
IRWD 2001	Homogeneous group; top 20% billing data	Letters requesting volunteers	40 WBICs	Authors cite ungeralizability of studies
LADWP 2004	Large residential developments and commercial properties only	Solicited	25 WBICs	Installed meters on 60 of 83 acres
MWDOC 2004	Intense marketing campaign requiring customer to contact agency	Marketing campaign using Boy Scouts, direct mailing, door-to-door	1,222 WBICs	About half report signifcant water savings
Santa Barbara 2003	Residential customers with highest water use	Letters to top 100 water users	62 WBICs	Customers had to pay $144 service fee
Saving Water Partnership 2003	Residential customers with highest water use	Identified by water agency and directly contacted	106 WBICs	One-half water bills higher after first year
University of Arizona 2006	Voluntary participants	Landscape and Manufacturer professionals identified users	27 WBICs	Tested WBICs of two types and moisture sensors

ASSUMPTIONS AND CALCULATIONS

The EPA WaterSense program authors make a number of assumptions about the inferences that can be derived from the research studies. The assumptions are categorized under three headings: potential water savings; potential energy savings; and, cost effectiveness. The energy savings and cost effectiveness predictions rely on data generated from the water savings calculations which are explicated below.

Potential water savings

The first assumption is that average outdoor usage is approximately 58,000 gallons of water annually per household. This data is based on Table 5.14 of the Residential End Uses of Water (Mayer, DeOreo, Hayden, & Davis, 2009). However, the referred Table indicates an average outdoor use of 84,738 gallons which represents 58% of total usage. While this disparity is not central to the critical analysis of the conceptual framework of the EPA WaterSense program it does serve to illustrate the even greater potential savings of water for landscape irrigation and, thereby, emphasizes the criticality of conducting robust and sound analysis and evaluation of the program's framework and foundation. The potential payoff, should empirical results be scientifically evaluated by third parties, appears to be significant in terms of water, economic, and energy conservation.
The EPA assumes that 13,500,000 detached single family homes have automatic irrigation systems and that 95% of these systems are candidates for replacement. The research studies indicate a range of overall savings from 6-30%, first year savings according to the Evaluation of California Weather-Based "Smart" Irrigation Controller Programs (Mayer, DeOreo, Hayden, & Davis, 2009) were shown to be 6 percent and within a limited subset of controllers that were tracked for 3 years, overall savings were shown to be 16% in year 3.

"In full consideration of the finding of these numerous studies, WaterSense anticipates seeing overall water savings of approximately 15 percent after installation of weather-based irrigation controllers" (EPA, 2011, p. 12).

Based on the above data, EPA calculated that: annual water savings per controller is 8,700 gallons/year; number of candidates for installation is 12,825,000; annual national water savings is 111.6 billion gallons/year; and, national cost savings are $410.6 million/year (EPA, 2011, p. 12)

Potential energy savings

The potential energy savings calculation is drawn directly from the water savings data. The assumption is that 1,500 kWh are required to deliver 1,000,000 gallons of water to residences from public supplies (EPA, 2011). Therefore, taking the gallons saved times the energy costs equals a savings of 167.4 million kWh (EPA, 2011).

Cost effectiveness

The cost effectiveness calculations are drawn directly from the water savings data. The assumption is that $3.68 per kilo-gallon of water is the marginal cost and the product lifetime for WBICs is 15 years. Therefore, the estimated annual water cost savings from installing a WBIC is $32.02; the estimated payback period for average cost of a WBIC is 7.4 years; and, the estimated payback period for low-cost option WBIC and higher water use is 2.7 years (EPA, 2011)

EPA WaterSense Conclusion

The conclusion of the EPA WaterSense program for WBICs is posted on the EPA web site and says "Replacing a standard clock timer with a WaterSense labeled irrigation controller can save an average home nearly 8,000 gallons of water annually. If every home in the United States with an automatic sprinkler system installed and properly operated a WaterSense labeled controller, we could save $435 million in water costs and 120 billion gallons of water across the country annually from not overwatering lawns and landscapes. That's equal to the annual household water need of nearly 1.3 million average American homes" (EPA, 2013). In other words, the posted conclusion is reflective of the assumptions the EPA makes about the potential savings and the outcomes of the research studies that are cited above.

CONCLUSIONS

The overarching conclusion of this study is that the EPA WaterSense program authors relied on the outcomes of 11 research studies to justify the program. Evidence has shown that the data derived from the research studies are not generalizable and, for the most part, were not designed to be generalizable for the purposes of developing a national program for water conservation by using weather-based controllers. The studies made no pretense of using non-random, targeted sample sets that often singled out high water users in their respective study areas. Therefore, the EPA program is not built on a solid, science-based foundation and its conceptual framework has not passed the tests of reliability, validity and generalizability.

In a public meeting on the WaterSense Draft Specifications for WBICs in 2009 (EPA, 2009), the author asked what metric would be used to measure success of the WBIC program? Ms. Stephanie Tanner, WaterSense Products Lead, who was directing the meeting, replied that the metric to be used by the EPA to measure success of the WaterSense program will be the number of weather-based irrigation controllers sold (EPA, 2009). In other words, the outcomes of the research study are so incontrovertible that, rather than measuring a direct reduction in irrigation water, it is sufficient to proxy the savings by counting the number of WBICs sold in the United States.

Public policy programs require robust evaluations on two levels. First, the conceptual framework of the policy needs to meet the tests of good social science. The model that is postulated needs to be reliable, internally valid, externally valid or generalizable, parsimonious, falsifiable, and important. Second, when the program is implemented it must be cost-effective, efficient, cognizant of the interactions with externalities, and equitable. As a recommendation for future study, the EPA WaterSense program for WBICs needs to be thoroughly evaluated at both levels. This study was a first step to question the generalizability of the program by critically evaluating the foundation of its conceptual framework.

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WATER SETTLEMENTS INVOLVING AGRICULTURE AND URBAN WATER USERS — AN ENGINEER'S PERSPECTIVE

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ABSTRACT

Agriculture to urban water transfers or leases can be an important component in a water right settlement. Often water transfers or leases of are part of the solution to water right and economic problems. For more than a century water has been marketed in the form of leases, transfers, purchases of land with water rights, and purchases of shares of water in irrigation and water companies and districts. This paper reviews two cases based on behind-the-scenes engineering, negotiations, and decisions concerning the marketing of water from agriculture to municipal uses. One example discussed is the agreement between the Quechan Tribe of the Fort Yuma Indian Reservation and the Metropolitan Water District of Southern California and others. The agreement was instrumental in the 2006 Consolidated Colorado River Compact and solved the decades-long conflict and legal battles concerning the water rights of the Quechan Tribe. Another case study involves the Moapa Tribe, Muddy Valley Irrigation Company, Moapa Valley Water District, Southern Nevada Water Agency, and others. Elements of water settlement agreements are presented, along with the motivations for settlements. Negotiated water settlements provide opportunities for participants to present ideas and work on solutions that provide some benefits to all parties. The participants have more control of the outcome and can generally come to a quicker and less costly solution to complex water allocation problems.

INTRODUCTION

Agriculture to urban water transfers can play a role in the settlement of complex Indian water right issues. In the United States there are 565 Federally-recognized Indian Tribes (U.S. Department of the Interior, 2013). Many of these Tribes have non-quantified federally reserved water rights. In 1908, the Supreme Court of the United States (Supreme Court) in the *Winters v. United States* case stated, "That the government did reserve (water rights for the tribe) we have decided, and for a use that would be necessarily continued through years" (207 U.S. 564; format is Supreme Court of the United States volume U.S. and page). The "Winters Doctrine" was again upheld by the Supreme Court in 1963 in the *Arizona v. California* case (373 U.S. 546). In this case the Supreme Court also ruled that the reservations (on the lower Colorado River) were entitled to sufficient water to irrigate all "practicably irrigable acreage". Theses ruling coupled with the allocation of water supplies by state have resulted in numerous unresolved water right issues concerning Indian Tribes.

The Arizona v. California case is an example where the temporary use of agriculture water allocated to the Quechan Tribe by Metropolitan Water District of Southern California (MWD) helped facilitate a water right settlement. The settlement was reached

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during litigation and was instrumental in the consolidated Colorado River Decree that was entered March 27, 2006. Another example of a water right settlement is the Moapa Paiute Water Settlement Agreement in which the water users of the Muddy River in southern Nevada worked out a settlement based on existing water rights and state water law. The Moapa Paiute Water Settlement Agreement occurred during a time when agriculture irrigation water was being purchased and leased by the Southern Nevada Water Authority to help meet the urban demands in the Las Vegas area.

QUECHAN TRIBE WATER SETTLEMENT

A brief historical background to the Colorado River Consolidated Decree provides the setting for the negotiations that resulted in the agreement and settlement. The 1922 Colorado River Compact described the division of the water between the upper (Colorado, Utah, Wyoming, New Mexico, and Arizona) and lower basin (California, Arizona, and Nevada) states. The specific state allocations in the Lower Basin were established in 1928 as part of the Boulder Canyon Project, while the current specific allocations in the Upper Basin were established by the Upper Colorado River Basin Compact of 1948. The following are significant events that occurred from 1953 to 2006:

- In 1953, the State of Arizona filed a bill of complaint against the State of California and seven of its public agencies concerning the allocation of the lower Colorado River Water.
- In 1963 the Supreme Court upheld the original allocation, and a decree was issued in 1964 (373 U.S. 546).
- In 1966, the Supreme Court granted the joint motion of the parties to amend the decree, and so extend the time for submission of lists of present perfected rights (383 U. S. 268).
- In 1979, the Supreme Court filed an opinion granting the joint motion for entry of a supplemental decree, entered a supplemental decree, denied in part the motion to intervene of the Fort Mojave Indian Tribe, and otherwise referred the case and the motions to intervene of the Fort Mojave Indian Tribe and the Colorado River Indian Tribes, et al., to Judge Elbert Tuttle as Special Master (439 U. S. 419, 437).
- In 1983, the Supreme Court filed an opinion rendering a decision on the several exceptions and the recommendation that the Fort Mojave Indian Tribe, the Chemehuevi Indian Tribe, the Colorado River Indian Tribes, the Quechan Tribe, and the Cocopah Indian Tribe be permitted to intervene, and approving some of his further recommendations and disapproving others (460 U. S. 605, 609, 615).
- In 1984, the Supreme Court entered a second supplemental decree implementing the 1983 Court decision (466 U. S. 144).
- In 1989, the Supreme Court granted the motion of the state parties to reopen the decree to determine the disputed boundary claims with respect to the Fort Mojave, Colorado River, and Fort Yuma Indian Reservations (493 U. S. 886).
- In 2000, the Supreme Court filed an opinion rendering a decision on the several exceptions to the report of the Special Master, approving the settlements of the parties with respect to the Fort Mojave and Colorado River Indian Reservations and remanding the case to the Special Master with respect to the Fort Yuma

Indian Reservation. (530 U. S. 392, 418, 419–420). On October 10, 2000, the Supreme Court entered a supplemental decree (531 U. S. 1).

- After the 2000 ruling, the parties worked to settle the claims of the Quechan Tribe.
- In 2005, an agreement with Arizona, California, and Colorado water districts was reached and approved by the Supreme Court. The settlements provided the Quechan Tribe with 26,350 additional acre-feet of water per year as incorporated into the 2006 Consolidated Colorado River Compact (547 U.S. 150).
- Due to the duration of the case, from 1953 to 2006, five special masters were appointed.

Fort Yuma Indian Reservation

Fort Yuma Indian Reservation of the Quechan Tribe encompasses some 25,000 acres of disputed boundary lands not attributed to the reservation in earlier stages of the litigation. Figure 1 is map of the Fort Yuma Indian Reservation showing the general location of the new reservation lands. The south and east boundary has been disputed because of the changes in the location of the Colorado River which was initially used as the reservation boundary. The opinion of the Supreme Court of the United States, June 19, 2000 stated:

> "For the foregoing reasons, we remand the outstanding water rights claims associated with the disputed boundary lands of the Fort Yuma Indian Reservation to the Special Master for determination on the merits. Those claims are the only ones that remain to be decided in *Arizona* v. *California*; their resolution will enable the Court to enter a final consolidated decree and bring this case to a close (530 U.S. 419-420)."

On June 14, 2005, Special Master McGarr submitted his report recommending approval of the settlements of the federal reserved water rights claim with respect to the Fort Yuma Indian Reservation and a proposed supplemental decree to implement those settlements. The decree consolidated the substantive provisions of the decrees previously entered and described (376 U. S. 340 (1964), 383 U. S. 268 (1966), 439 U. S. 419 (1979), 466 U. S. 144 (1984), and 531 U. S. 1 (2000), implemented the settlements of the federal reserved water rights claim for the Fort Yuma Indian Reservation, which the Court approved, and reflects changes in the names of certain parties and Indian Reservations. The decree was entered in order to provide a single convenient reference to ascertain the rights and obligations of the parties adjudicated in the original proceeding, and reflects only the incremental changes in the original 1964 decree by subsequent decrees and the settlements of the federal reserved water rights claim for the Fort Yuma Indian Reservation (547 U.S. 150).

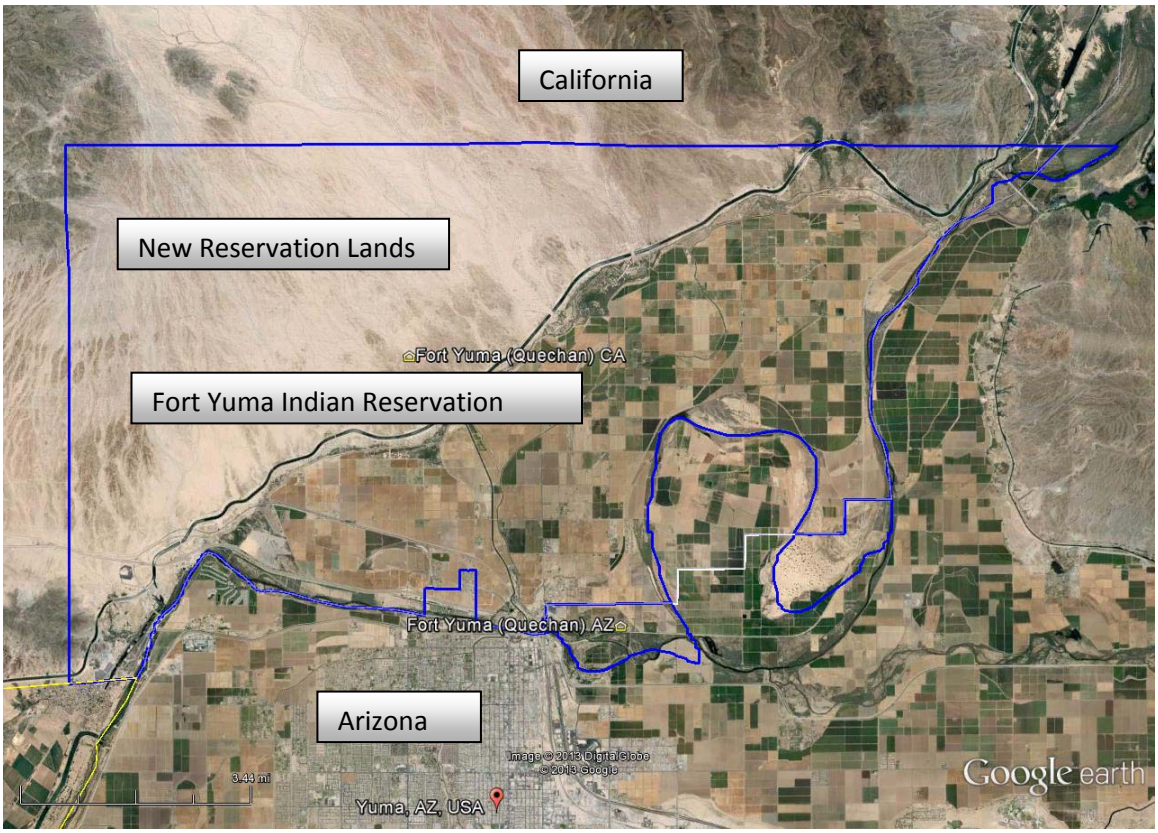

Figure 1. Fort Yuma Indian Reservation, California and Arizona.

Settlement Agreement

After the Supreme Court's ruling in 2000, the Quechan Tribe began working to quantify the water rights for 'practicably irrigable lands' on the their disputed boundary lands and other lands omitted from the 1964 Lower Colorado River Decree. The work was in preparation for litigation or a settlement. Because the allocation of the Colorado River to California was not to change, the allocation to specific water users would be reduced by the additional water rights received by the Quechan Tribe. The 1931 Boulder Canyon Project Agreement apportioned California's share of the Colorado River among seven California water agencies applicants (Seven Party Agreement) (Palo Verde Irrigation District, et. al, 1931). In the Seven Party Agreement, MWD and/or the City of Los Angles' allocation of 550,000 acre-feet per year was the fourth priority. The first three priorities were not to exceed 3,850,000 acre-feet per year. The Quechan Tribe's water right is a 'perfected right' (date of January 9, 1884) and has priority over allocations specified in the Seven Party Agreement, therefore additional allocation to the Quechan Tribe could reduce MWD's annual water allocation. For many years California Colorado River water users consumed in excess of the 4.4 million acre-feet per year from the Colorado River. As part of the Quantification Settlement Act, California developed the 4.4 Plan which limited California consumptive use from the Colorado River to 4.4 million acre-feet per year. Figure 2 illustrates California's consumptive use from the Colorado River as determined by the United States Bureau of Reclamation.

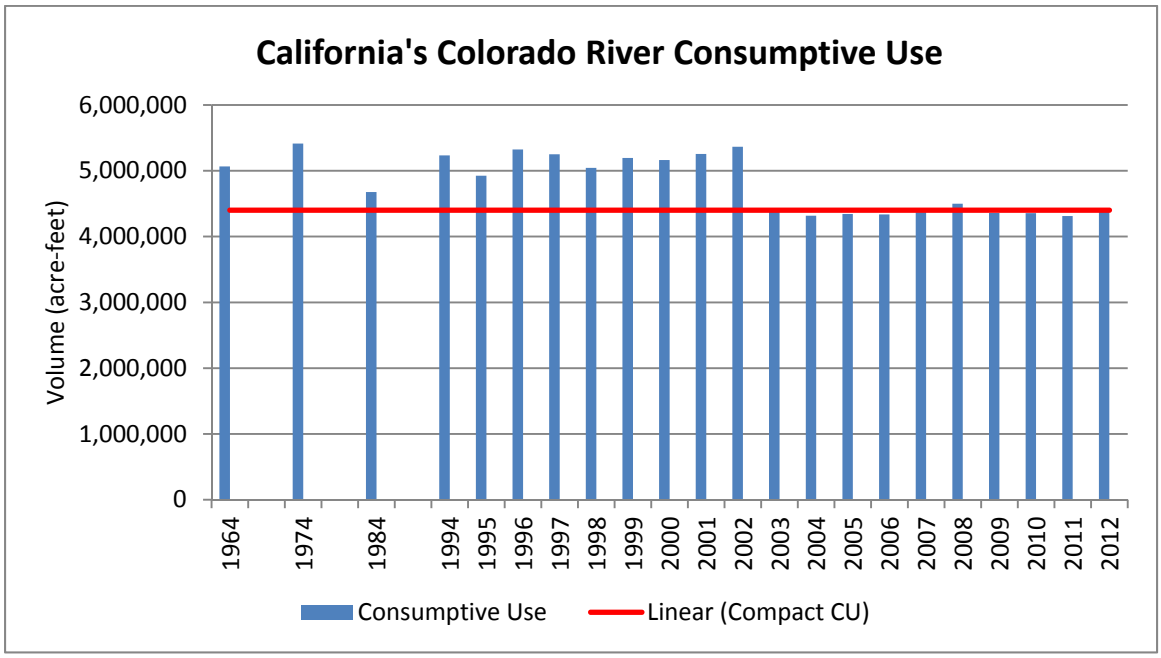

Figure 2. California's Colorado River Consumptive Use (from USBR Annual Colorado River Water Accounting Reports).

After several years of preparation for litigation, discussions, and meetings by the Quechan Tribe, the United States, and MWD a negotiation settlement meeting was held in 2004. At this negotiation meeting agreement in principles were developed and used to formulate the Settlement Agreement in Arizona v. California by and Among the Quechan Indian Tribe of the Fort Yuma Indian Reservation, The United States of America, The Metropolitan Water District of Southern California, Coachella Valley Water District, and the State of California, dated February 14, 2005 (Settlement Agreement). An important aspect of the settlement plan was MWD ability to use some of the Quechan Tribe unused allocation. This helped MWD with its water supply obligations, especially with their reduced diversions from the Colorado River.

The following are some of the general provisions of Settlement Agreement:

1. Clarifies the boundary of the Fort Yuma Indian Reservation;
2. The agreement resolves the water rights of the Fort Yuma Indian Tribe;
3. The agreement specifies the annual amount of water available for use by the Tribe on certain lands within the reservation:
    a. 20,000 acre-feet of diversions
    b. For use on 2,998.50 acres within California
4. The MWD and the Tribe agrees that water unused by the Tribe may be diverted by MWD for a payment;
5. Pursuant to the Quantification Settlement Agreement (QSA), Coachella Valley Water District (CVWD) agrees not to object to MWD's diversion and use of any water obtained from the Tribe, provided that CVWD retains all of its rights to the use of any water by MWD after the QSA has expired;

6. The parties agreed that the Tribe's Present Perfected Rights are not affected by this settlement agreement;
7. The United States agrees to resolve the consumptive use issues related to the Yuma Island water use;
8. The State of California accepts the boundary of the reservation in California for the purposes of determination of water rights, but reserves the right to contest the boundary of the reservation in any appropriate administrative or judicial proceedings; and
9. The State of California contends that the western-half of the natural bed of the Colorado River prior to the avulsion event of 1920 are sovereign lands and passed to the State of California in 1850 by operation of the Equal Footing Doctrine.

The only aspect of the Settlement Agreement that was incorporated into the Consolidated Colorado River Compact was the 20,000 acre-feet per year of additional allocation to the Fort Yuma Indian Reservation (Quechan Tribe) for use in California. The Fort Yuma Indian Reservation California diversion allocation increased from 51,616 to 71,616 acre-feet per year diversion and net acreage from 7,743 acres to 10,742 acres (the water right is limited the lesser of the diversion right or the acreage). Additionally, as a result of the negotiations the Quechan Tribe and the State of Arizona agreed to the Fort Yuma Indian Reservation receiving 6,500 acre-feet per year diversion right from the Colorado River for irrigation of up to 950 acres.

In accordance with the Settlement Agreement the Quechan Tribe and the Metropolitan Water District of Southern California agreed that until 2035 up to 13,000 acre-feet per year of the Quechan Tribe's unused water could be used by the MWD for a payment ($/acre-foot) to the Quechan Tribe. Additionally, that 7,000 acre-feet or the amount necessary to supply the consumptive use required for irrigation of 1,049.47 acres, and for the satisfaction of related uses, whichever is less, shall be allowed to pass through the priority system and be diverted by MWD. Beginning in calendar year 2035, and continuing thereafter, this water shall become available to the Quechan Tribe.

A separate settlement agreement was developed as a result of negotiations between the United States, the Tribe, and the Arizona parties. The Arizona settlement provides 6,350 acre-feet of water to the Tribe and establishes the boundary of the reservation in Arizona. The agreed upon additional water rights are included in 2006 Consolidated Colorado River Decree. In the 1964 Colorado River Compact the Fort Yuma Indian Reservation had no Arizona diversion rights.

MOAPA BAND OF PAIUTES

The 71,954-acre Moapa Band of Paiute Indians (Moapa Tribe) Reservation is located in Southern Nevada; a portion of the Reservation is along the Muddy River that flows into Lake Mead, a reservoir on the Colorado River (Figure 3). The Moapa Tribe entered into settlement negotiations with water users in the region; including the State of Nevada, Southern Nevada Water Authority, Las Vegas Valley Water District, United States Fish and Wildlife Service, Coyote Springs Investment LLC, Moapa Valley Water, and Muddy

Valley Irrigation Company. The Moapa Paiute Water Settlement Agreement (Moapa Water Settlement) includes the Memorandum of Agreements, resolutions, permits, certificates, applications, and legal documents to provide the Tribe with surface and groundwater water for the purposes of the reservation (Moapa Band of Paiutes, 2006). The Moapa Settlement Agreement included the following unique aspects:

- The transfer of existing applications for groundwater filed by Las Vegas Valley Water Authority to the Moapa Tribe.
- The long-term (198 years) leasing of decreed surface water with no rent charge to the Moapa Tribe from the Muddy Valley Irrigation Company
- The United State Department of Justice, the United State Bureau of Indian Affairs, and the United States Department of Interior are not signatures to all the agreements.
- The settlement was based on existing state laws and decrees.
- The Moapa Water Settlement provided the Moapa Tribe with water for agriculture and for commercial and industrial development without the new appropriation of water. The groundwater will take several years to be fully developed before proof of beneficial use can be submitted and water right certificate can be obtained.

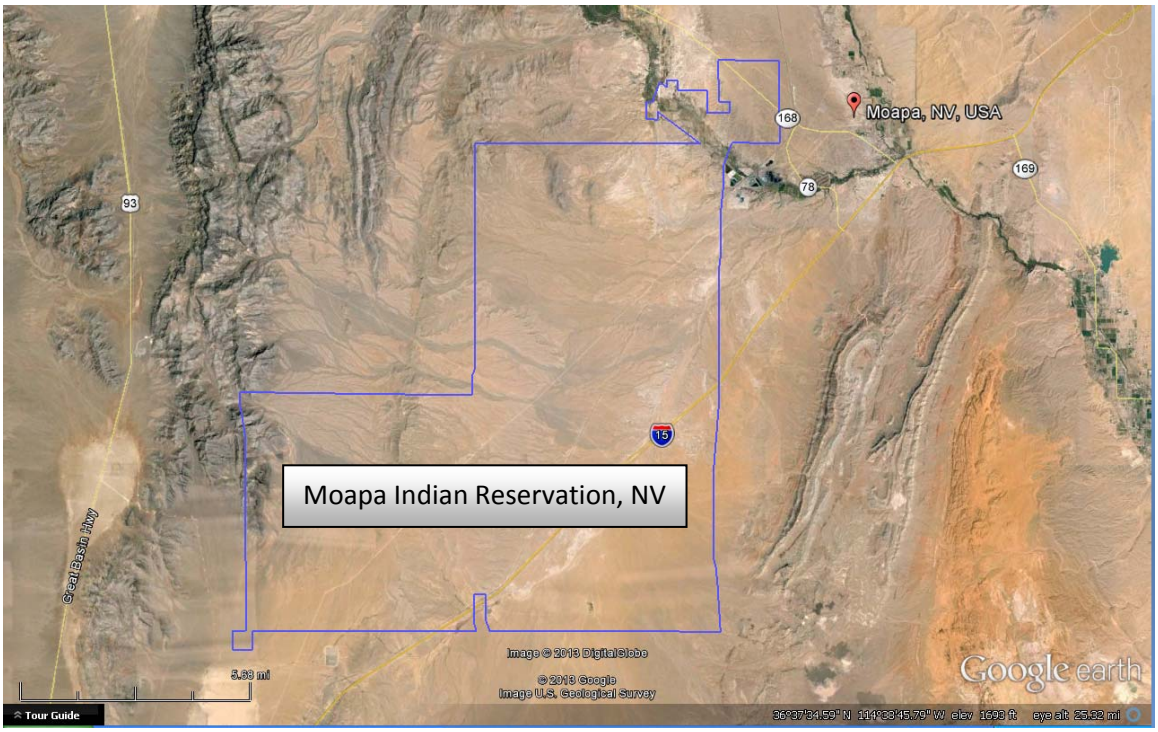

Figure 3. Moapa Indian Reservation, Nevada.

The groundwater is currently being developed and used for economic development. The groundwater is from groundwater permits issued by the State of Nevada to Las Vegas Valley Water District. The permits were transferred based on agreements and the necessary application filed with the Nevada State Engineers Office. The groundwater

permits were issued by the State of Nevada and are treated as other groundwater permits in the state. Since the groundwater is for current and future use and development, each year the Moapa Tribe report their water use and request and extension of time to accommodate future water development in accordance with state law. The priority dates for the groundwater permits are based on state law and not on the Moapa Reservation establishment date.

The Muddy River is the source of surface water for the Moapa Reservation. The Muddy River was fully allocated and decreed as a result of a 1920 judgment in *the Muddy Valley Irrigation Company, et al. v. Moapa and Salt Lake Produce Co., et al* case. In the 2006 Moapa Settlement Agreement, the Moapa Tribe did not receive a water right for waters from the Muddy River; however, they did obtain a no cost a 99-year lease and a right to a single 99-year lease extension for use of decreed water (consumptive use of 3,700 acre-feet per year). The lease is with the Muddy River Irrigation Company the water right holders of the decreed water. This agreement provided a way for the Tribe to have certainty in its water right and operate within the existing state laws and established water rights. The Muddy Valley Irrigation Company applied with the Nevada State Engineer the necessary changes in point of diversion, place of use, manner of use, etc. to accommodate the leases. The Moapa Tribe has no shares in the Muddy River Irrigation Company. The priority date of the water remains the same as the decree, but the Moapa Tribe has a contract seniority which provide the first right to use the water.

The Moapa Water Settlement is comprehensive and includes seven agreements each with a different purpose, plus resolutions by the parties, all the certificates of water rights and change applications. There are agreements in place to protect the Moapa Dace, an endangered species of fish, by providing habitat and minimum instream flows in the Muddy River and spring tributaries. The agreements also call for the measurement of surface water and groundwater diversions, monitoring of groundwater levels and spring flows. The measurement and monitoring is to protect the water rights in the Moapa Valley and on the Moapa Reservation by preventing over allocation of groundwater that may adversely impact surface flows and groundwater levels.

The alternative to the settlement would have likely been decades of litigation that would have hindered the Moapa Tribe from utilizing its water supply for economic development. It would have also made it more difficult for Southern Nevada Water Authority, Las Vegas Valley Water District, Muddy Valley Irrigation Company, and Moapa Valley Water District to make the many agreements and transactions for utilization of the water resources of the Muddy River.

The engineering aspects of this settlement were based in part on the legal principals of Indian Water Rights including ‘Winters Doctrine’, ‘historically irrigated lands’ and ‘practicably irrigable acreage’. As with other negotiations, compromises were made concerning the irrigation and diversion requirements and irrigated acreage.

Another benefit that resulted from the Moapa Water Settlement is that the Moapa Tribe had a recognized surface water lease and was able to sublease water to Southern Nevada

Water Authority. The 2007 *Lower Colorado River Basin Intentionally Created Surplus Forbearance Agreement* (State of Arizona, et. al, 2007) provided the mechanism that the Moapa Tribe could sublease water to Southern Nevada Water Authority within the Colorado River Decree and receive an economic benefit from their unused leased water lease. Without the agreement in place, the Tribe would not have had the opportunity to receive millions of dollars for Southern Nevada Water Authority use of the Moapa Tribe's unused surface water lease. Provisions of the 5-year sublease agreement provided an option that the Tribal farm could use a portion of their water lease for irrigation.

DISCUSSION

While each set of water right and water use problems have unique circumstances and needs, there are some general concepts that lead to successful settlements and/or agreements. For the Quechan Tribe Water Settlement a number of circumstances were in place that encouraged a negotiated settlement and facilitated aspects of water transfer and leasing. The circumstances included 1) the Supreme Court of the United States decision that the Quechan Tribe's water rights should consider the reservation lands not included in the 1964 Compact, 2) the directive that the State of California develop a plan to limit water diversions to the decree water right of 4.4 million acre-feet per year, 3) the agreement of all parties to allow a portion of the Quechan Tribe's unused water right to be used MWD. Soon after the consolidated decree was ratified, the Lower Colorado River Intentionally Created Surplus Forbearance Agreement was signed allowing specific water conserved or not used to be used by other parties. For the Moapa Water Settlement, the immediate water needs by the Moapa Tribe and Southern Nevada Water Authority helped facilitate a the agreement so that economic opportunities could be pursued by the Tribe.

Keys to Success

- Identify and Involve Participants – Make an assessment of the parties that are needed to achieve a negotiated settlement. Even successful negotiations among several parties can fail during the implementation stage if other affected parties are not involved. Remember that if you are not at the table; you may be part of the menu.
- Willingness to Participate in Discussions and Negotiations – In both examples described, the parties had motivation to achieve a settlement. At times it takes a lawsuit to get the attention of the parties, especially if the existing status offers a benefit to one or more of the parties. The directive from the federal government that the State of California develop a plan to reduce their Colorado River water use to the decreed amount of 4.4 million acre-feet per year provided an incentive to develop certainty in the water right allocation among the California Colorado water users.
- Define Goals then Develop a Plan with a Timeline – The plan should identify the legal and technical aspects that are a part of the negotiations. The plan of work should include tasks and timelines. The goals are often directly tied to issues under litigation.

- Technical Preparation – In both of the described cases a water right claim was prepared by consultants based on identification of historically irrigated lands and on 'Practicably Irrigable Acreage' criteria. This analysis included analysis of climate, irrigable soils, runoff and erosion, crops, irrigation system construction and operation costs, economic feasibility, crop market analysis, and similarly situated successful irrigation projects. The preparation was at a level sufficient for litigation purposes if a settlement was not reached.
- Hold Pre-Negotiation Meetings to Present Technical Findings – These meetings provide opportunities for technical experts to present methodology, data, and findings. The information from these meetings help parties understand and sometimes agree upon methodologies and analysis (for example crop water requirements, soil classification criteria, irrigation system design and costs, crop economics, and technical specifications). The meetings also help promote trust in findings and analysis that increase the success of a settlement agreement.
- Develop Settlement Options that Provide Benefits to all Parties – This involves creative ideas and sound legal and technical analysis. The proposals need to be within the framework of existing laws or be willing to spend the time and resources to develop new law. The settlement or agreements can solve several problems, but don't expect all problems to be solved. Sometimes adding other items into a settlement agreement provides benefits to a specific party and increases their willingness to accept other portions of the settlement. A balance is needed that includes enough items to benefits to all parties; but not extra items that complicate, confuse, and/or limit agreement. In the case of the Quechan Tribe and the other California parties involved in the Arizona v. California case the settlement agreement included provisions including; additional water rights for the Quechan Tribe, the ability for MWD to divert and use a portion of the Tribe's unused water, an income to Tribe for MWD's use of a portion of their unused water, and provisions to help resolved reservation boundary issues. In the Moapa Water Settlement benefits and certainty were provided to all parties.
- Prepare for Negotiations – Each party should internally discuss possible outcomes and acceptable terms of potential agreement. This could include a minimum and/or maximum water right quantity and acceptable provisions. All proposals are backed up by sound technical and legal analysis.
- Be Willing to Compromise – Negotiations and litigation are to resolve a problem or differences of opinion. Understand that seldom will the solution be exactly what the agency or individual wants. In the case of the Quechan Tribe's negotiation with California entities, the additional water right of 20,000 acre-feet per year was more than MWD experts had determined and less than experts for the Quechan Tribe has determined. Hence the round number of 20,000.
- Use an Arbitrator or Moderator for Decision Making Meetings - An effective arbitrator can keep the negotiation meetings moving forward, exchange information during party caucus meetings, and help bring closure to discussions.
- Provide Opportunity and Time for Party Caucuses – During the course of negotiations there are times when private discussions by parties are necessary to privately discuss options, proposals, counter proposals, compromises, and

strategies, and most importantly to make decisions. This was particularly helpful in the California Quechan Tribe water right negotiations.

- Have Informed Decisions Makers Involved in the Settlement Negotiations – The effectiveness of a negotiation session is increased if decisions can be made during the session. Having to suspend or put negotiations on hold can stall if not stop a settlement. The Quechan Tribe negotiation sessions in which the principles of agreement were developed was held over consecutive days until agreement was reached. The meeting was attended by Quechan Tribal Council, legal counsel representing governmental and water agencies, water agency manager, state water resource directors, state general attorney, and others. There were times when resolutions were drafted and voted on during the negotiation so that a settlement could be reached when all parties were present. Although agreements were reached and signed, every detail was not developed during the negotiation meetings. Technical experts were also attending to present proposals and answer questions.

Benefits of a Negotiated Settlement

Negotiated settlements are only an option if the parties are willing to participate. As an alternative to litigation, there are a number of benefits to a negotiated settlement. These benefits include:

- The judge, court, and/or special master are likely to accept a stipulated or agreed upon settlement when all parties have signed the agreement.
- The settlement can include items other than the issues listed in a lawsuit, complaint, or appeal. In the case of the Quechan Tribe and the California agencies the settlement included an income for the Tribe from MWD for MWD's use of water not used by the Tribe. The settlement is agreed upon by all parties and should include provisions that benefits to parties.
- The parties are more in control of the outcomes and are not limited to the decision or opinion of a special master and/or judge. Parties can withdraw from negotiations if acceptable terms are not presented. This is easier that appealing a decision from a court.
- Negotiations generally require less time and cost than litigation.
- Negotiations can provide increased opportunities for technical experts to interact to understand differences and similarities in methodology, analysis, results, and opinions.

CONCLUSIONS

Agriculture to urban water transfers can be an important part of negotiated water right settlements involving Indian Tribes, because many Indian water rights are for irrigated agricultural based on 'historically irrigated acreage' and 'practicably irrigable acreage'. There are opportunities for Indian Tribes to receive an income from urban water users who have critical demands and ability to purchase or lease water. The leasing of water by Indian tribes can provide them the needed time and resources to develop agricultural

irrigation, municipal, commercial, and industrial uses for their water rights. Negotiations require willing parties who consider the water needs, engineering analysis, and legal issues. The negotiation of a water settlement can be less costly, more acceptable, quicker, and provide more flexibility than litigation.

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HOW CONJUNCTIVE USE HAS CHANGED IN THE SOUTH PLATTE BASIN OVER 50 YEARS

Robert Longenbaugh[1]

ABSTRACT

Irrigation wells were constructed and have been used by farmers to supplement their inadequate and undependable surface water supplies beginning in the 1920's. The number of new wells increased over the next 45 years especially during the 1953-57 and 1963-65 droughts. Major water conflicts developed in the 1960's between senior surface water owners and well owners over the impact well pumping had upon river flows.

Hydrologic studies by Bittinger and Wright [3][2] in 1968 documented that wells impacted river flows, but supplemental pumped water provided a full water supply for the irrigators crops. The 1969 Colorado Legislature passed the Ground Water Administration Act which required the dual objective: ground and surface water had to be administered to prevent injury to vested water rights, but it must also maximize the beneficial use of both the ground and surface water resources. The concept of augmentation plans to allow junior well owners to pump out of priority, if they replaced stream depletions due to pumping to prevent injury to downstream rights, was developed. Conjunctive use, which had been used for many years previously, was born.

This paper briefly describes South Platte River Basin history for four different periods of water administration and management. During the first two periods conjunctive use flourished and the South Platte Basin was considered as a prime example of how conjunctive use of both ground and surface water maximized the available water supply. During the third period there was significant change in the water law and strict priority administration of both the ground and surface water rights was implemented. This period also included the 2002 drought. The fourth period restricts the use of the groundwater resource and conjunctive use, in my opinion, has retrograded back to pre 1965 conditions.

A brief discussion of some of the consequences of the water administration and management changes that occurred on January 1, 2006 is included.

INTRODUCTION

Colorado has a number of streams which flow through alluvial valleys, most notably the South Platte and Arkansas River Basins. These stream/aquifer systems are typical of similar hydrologic systems in the western United States and other countries. Development of irrigation using the stream flows for a water source was common, but those flows varied throughout the year due to fluctuation of precipitation or snow melt runoff. Often the stream flow and groundwater in the underlying alluvium are

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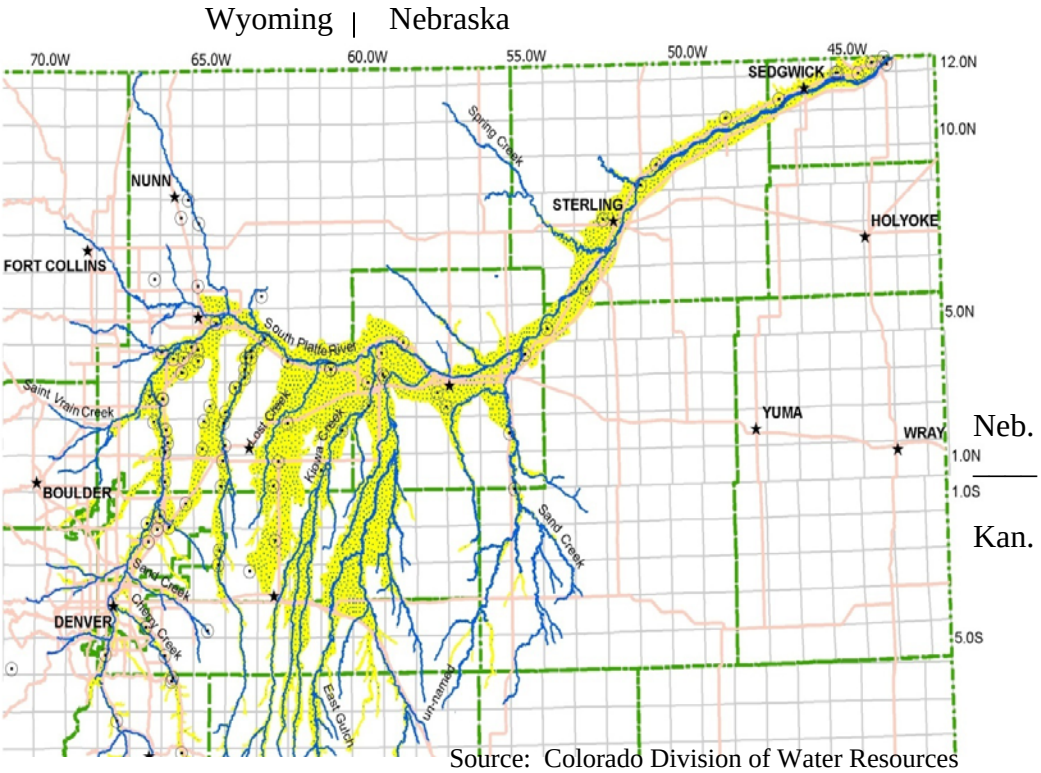
[2] Refers to publication number 3 in the Reference list.

hydraulically connected. These types of systems are prime candidates for the conjunctive use of both the ground and surface water resources.



Source: Colorado Division of Water Resources

Figure 1 Location map of South Platte River Basin in northeast Colorado including the boundaries of the alluvial aquifer, (yellow area).

This paper focuses on the South Platte River Basin in Colorado, see Figure 1 above. It will refer to historical data and events to illustrate how conjunctive use began in the Basin and describe how that has changed due to new water laws, water administrative policies, and court cases. The paper will focus on four different time periods where the amount and control of conjunctive use has changed significantly. Space restraints will limit the details describing the specific changes, but sufficient description is provided to illustrate that with appropriate water laws and water administration policies a more dependable and adequate water supply for irrigation is possible with conjunctive use. Conversely, when laws and water administration are changed without first studying the impact of such changes, then the dependable water supply can be greatly reduced.

A brief discussion is included that describes the hydrologic and economic consequences that have resulted since January 1, 2006 when all wells had to have a Water Court decreed augmentation plan. Irrigation wells are severely limited to when they can beneficially consume any water under their own priority.

CONSUMPTIVE USE DESCRIPTON

In the South Platte River Basin irrigation began in the late 1850's to provide food for the large number of settlers that migrated to Colorado participating in the gold rush. Water was diverted from the streams and applied by gravity furrow irrigation. Deep percolation of the irrigation water below the root zone recharged the underlying alluvial aquifer. Ground water levels rose dramatically and resulted in accretion of water to the South Platte River causing the river to become a perennial stream with increased flows each year. River gain studies by Parshall [15] document how the rise in ground water levels caused increased river flows. For the South Platte River downstream of Kersey the ground water return flows were the principal source of stream flow especially during the July-September period.

Stream flows were often inadequate during much of the irrigation season to satisfy the demand by the canals. Only during the snowmelt months of May and June were there sufficient flows to meet the canal demands. During drought years even the snow melt months might be short of water. Because of inadequate stream flows to meet the canal demands there were both small and large capacity surface reservoirs constructed which stored water in the winter time or during snow melt runoff. Water was then released from those reservoirs to supplement natural river flows thus increasing the amount of water available for canal deliveries. Even with the significant reservoir releases there were water shortages for both the early and late season months and especially during the drought years.

Individual farmers were aware that there was significant ground water stored in the alluvium and in the 1920's they began to dig large diameter wells and use centrifugal pumps to provide a supplemental supply for the inadequate and undependable canal and reservoir supplies. Thus the use of wells to supplement the surface supplies was the beginning of conjunctive use.

1920-1969 Conjunctive Use Period

Individual farmers paid and constructed their own irrigation well(s) to supplement their canal and reservoir water rights. Those wells were often located at the highest elevation on their property so that they could deliver the flows by gravity to the point of use. The water was pumped to the on-farm canal system and directly supplemented the surface water supplies.

William Code, Associate Irrigation Engineer, Colorado Agricultural Experiment Station, wrote several Experiment Station Bulletins that described how to construct irrigation wells, select pumps, and operate irrigation pumping plants. Probably his most useful report is one entitled "Use of Ground Water for Irrigation in the South Platte Valley of Colorado"[5]. Code and his assistants did extensive field work in 1940 and 1941 and found that there were 1,955 irrigation wells pumping an estimated 220,000 acre-feet/yr. Of these, over 80 percent supplemented the surface water rights owned by the irrigators.

Code concluded from this 1940-41 study: "The tremendous value of water from wells in this valley is universally admitted even by those whose river rights might possibly be affected. The tendency in late years is to recognize that wells are a source of immediate relief in case of water shortage and that, even if return flow is affected, the injury so engendered is probably of less importance than the total economic benefits derived…"

The number of irrigation wells constructed by individual farmers continued to increase especially after 1945. There were three specific reasons for this increase after World War II: 1) reverse rotary well drilling became available allowing a better and less expensive method of well construction; 2) turbine pumps and hollowshaft electric motors were developed during the war years; and 3) rural electric cooperatives expanded and provided inexpensive dependable three phase electric power. These improvements allowed individual farmers to drill and equip a new irrigation well for less than $10,000.

When the 1953-57 and 1963-65 droughts arrived, there were major increases in the number of irrigation wells constructed. These increases in well pumping, when stream flow from the mountains and precipitation falling on the watershed were greatly reduced, caused major conflicts to develop between senior surface water right owners and irrigation well owners. Senior water right owners sought an injunction against the State Engineer to curtail all irrigation pumping. That action was quickly withdrawn when some of the senior water right owners recognized that their irrigation wells would be shut off.

The Legislature in 1966 funded a study by Mort Bittinger and Ken Wright, consultant water engineers, to evaluate how irrigation well pumping affected stream flows and to make recommendations for new legislation. These consultants made their own hydrologic studies and also relied heavily upon the extensive U.S. Bureau of Reclamation studies for the proposed Narrows Dam Project. The Bureau for a fifteen year study period evaluated historic canal diversion records and used electric power records to calculate how much water was being pumped by irrigation wells. The Bureau prepared graphs for a number of canals diverting from the South Platte that showed how frequently there were water shortages over the 15 year period when only canal diversions and reservoir releases provided the irrigation water. They then added on the volumes of pumped irrigation water applied beneath each ditch which clearly showed how the use of irrigation wells when added to the surface water supply was providing a full water supply to meet the crop demands. The Bureau's analyses also showed that excess irrigation well capacity provided a safety factor to meet total crop demand.

Bittinger's and Wright's [3] studies confirmed the Bureau's findings. Their 1968 project completion report contained a number of conclusions and recommendations, but the two most significant findings were: 1) the irrigation well pumping did reduce river flows that could cause injury to senior vested water rights; however, 2) it was the use of irrigation wells that supplemented the inadequate and undependable surface water supply that was needed to provide a full water supply to meet the crop needs. They recommended that irrigation wells with their junior priority dates should be allowed to continue to pump out of priority, but they must take specific action to replace stream depletions that caused

injury to vested rights. The concept of well augmentation to replace depletions due to their previous pumping to prevent injury to vested rights was born.

The Bittinger and Wright study for the South Platte Basin had nearly identical findings, conclusions and recommendations to a similar study by Wheeler and Associates for the Arkansas River Basin [20]. The Colorado Legislature relied on those studies and passed the Ground Water Administration statute (SB-81) in 1969. That statute required that irrigation wells had to be incorporated into the priority system by obtaining their own court decree establishing their own appropriation date, flow rate, annual volume to be pumped and description of acres irrigated. In addition, the 1969 Act established a new dual-water administration requirement that both ground and surface water rights had to be administered to prevent injury to a senior vested right, but they must also be administered so as to maximize the beneficial use of both the ground and surface water resources. The new statute recognized that wells could be allowed to pump out of priority, if they augmented their stream depletions or in some other way prevented injury to vested rights. Provisions were made to allow Water Courts to decree augmentation plans, but that was not an absolute requirement.

The Bittinger and Wright studies and the new 1969 water law clearly codified the conjunctive use concept in Colorado. Irrigation well pumping was recognized for its importance to meet the needs of irrigated crops. By using the wells to supplement the inadequate surface water supplies there was a significant increase in the volume of water available to meet the irrigation needs.

From 1940 to 1965 there was significant development in our understanding and ability to quantify the interaction between ground water and surface water for a stream/aquifer system like the South Platte. The work by C.V. Theis in 1941 [19] provided mathematical equations that could be used to calculate the depletion of stream flow caused by pumping a nearby irrigation well. R.E. Glover and G.G. Balmer [8] published a paper in 1954 that advanced Theis's work and is often called the Glover method. C.T. Jenkins [10] from the U.S. Geologic Survey developed a stream depletion factor, SDF, method in 1964-68 which was a simple approximation for estimating stream depletions caused by well pumping. In 1965 Colorado State University [2]developed the first finite difference ground water model which was capable of simulating a complex stream/aquifer system and could compute both time and space variation in stream flows as a function of many hydrologic inputs. The science and technology had advanced by 1969 to where there were ways to compute how various water administrative decisions would impact both stream flows and the ground water system. Those tools could be used to better evaluate where, when, and how much impact was occurring to river flows due to nearby irrigation pumping.

1970-1999 Conjunctive Use Period

Following the passage of the 1969 Ground Water Administration Act, there was considerable activity by the State Engineer's Office; by both senior surface water right and well owners; by lawyers, engineers, scientists and other interested parties. They all

wanted to have input in the development of procedures and polices that would implement the Act so as to meet the dual objectives. The debate centered on how best to allow irrigation wells to pump while preventing injury to other vested water rights.

The State Engineer's first action was to adopt rules and regulations that would control and reduce well pumping from wells that did not include specific provisions to prevent injury. Those rules and regulations were contested by senior water right owners and were found to be illegal.

An attempt was made to have irrigation wells decreed as alternate points of diversion for ditch water rights. Although several decrees were entered granting wells as alternate points, this process did not prevent injury to downstream water rights when a ditch water right went out of priority and thus a well had to stop pumping. The delayed depletive affect caused by the well pumping still impacted river flows for a significant period of time.

Another approach pursued by a number of well users was to acquire through ownership, lease, rental or some other method sufficient quantities of surface water to replace all the computed depletive affects caused by pumping of wells owned by members of the group. These groups of well owners went to Water Court and obtained a court decreed augmentation plan. The methods used to compute the depletive affect of future and past well pumping were not standardized and this resulted in significantly different requirements in the augmentation decrees. Recent Court augmentation decrees have a more uniform method of computing depletions.

One of the major efforts undertaken by the State Engineer, Clarence Kuiper, in the 1970-73 period was to organize well owners in several different associations where the purpose of the associations was to acquire significant quantities of water that could then be administered by the Division Engineer to keep senior surface ditches satisfied so they did not place a priority call. If there was no priority calls senior to the well rights, then the wells could pump in their own priority. The State Engineer proposed to approve such operating plans each year pursuant to Colorado Revised Statue 37-80-120 CRS.

The two largest associations which were formed were Groundwater Appropriators of the South Platte, (GASP), and the Central Colorado Water Conservancy District, (CCWCD). These two associations were formally created in the 1972-74 period and proceeded to sign up memberships of individual well owners. Each well owner had to pay the respective association an amount of money each year based on an estimated amount of pumping for each well. This money was used by each association to then purchase, lease rent or otherwise acquire the amount of water needed by the Division Engineer to prevent senior priority calls. The amount of water needed in a dry year was more than for a wet year, because there was less stream flow and more pumping in the dry year.

Both GASP and CCWCD operated from 1974-2001 pursuant to these annual substitute supply plans. The annual amount of water which they had to acquire which was turned over to the Division Engineer for him to deliver to senior surface water right owners

varied between 5 percent for wet years and 10 percent for dry years. The 5 and 10 percent numbers are the percentages of the annual consumptive use pumped for all the wells included in each association.

Another important feature of how GASP operated was that they knew one of the most senior surface water rights which made priority calls was the Sterling #1 Ditch. To prevent that right from making a priority call, GASP drilled and operated 10 high capacity wells which pumped directly into the Sterling #1 Ditch. When the South Platte River flows were insufficient to meet the demand from the Sterling #1 Ditch, then one or more of the production wells would be turned on to satisfy the Ditch needs. Reports from the Sterling #1 Ditch owners claim that for the period when GASP operated the 10 supplemental production wells, they had the most dependable and adequate water supplies ever on record. This illustrates how conjunctive use of both ground and surface water results in a more dependable and maximum available water supply.

Another significant water administration policy which was in place that allowed irrigation wells to pump out of priority was what is referred to as “the gentlemen’s agreement for how surface reservoirs would be filled”. That agreement which originated in the 1940’s provided that following the normal irrigation season, surface reservoirs would begin storing river flows from the top down and not have a priority call lower in the system. This resulted in no winter-time call to fill surface reservoirs and the stream depletions caused by pumping the previous summer did not have to be replaced by the irrigation well owners. This was a major benefit for all well augmentation plans and for GASP and CCWCD.

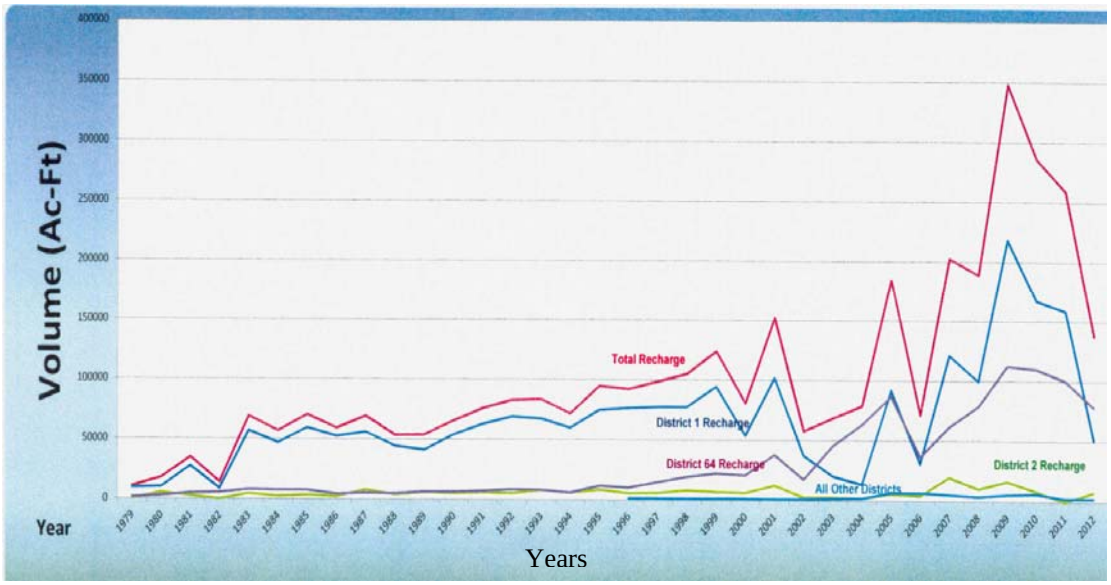
In 1974 a cooperative demonstration project between the South Platte Ditch Co., Colorado Division of Water Resources, Colorado Agricultural Experiment Station and GASP was started to demonstrate how artificial recharge of water diverted though the South Platte Ditch during winter non irrigation months would recharge the underlying aquifer. That water would later return as recharge accretions to the South Platte River. The project utilized a leaky reach of an abandoned irrigation canal and a couple of natural depressions to store the diverted water until it soaked into the alluvial aquifer.

The South Platte Ditch Recharge Demonstration Project lasted four years. Twenty three monitoring wells were constructed and measured biweekly and continuous recorders were used on measuring flumes and in the two ponds to determine the recharge rates. All of the observation well data and surface water data were used to prepare two progress reports [6,7]. Those data were later used by the Division of Water Resources staff to develop and calibrate a finite difference ground water model. That model was used to evaluate when, where and how much water returned to the South Platte River as artificial recharge river accretions. A 1979 professional paper in the Water Resources Research [12] publication describes the project model and concluded that about 73 percent of the artificially recharged water should be allowed as augmentation water to replace the well pumping depletions from a number of nearby irrigation wells.

In 1974 the South Platte Ditch Co. obtained the first artificial recharge decree from the Water Court for the South Platte Basin. In 1979 the Ditch Company obtained the first augmentation decree relying upon artificial recharge river accretions and utilized the 1974-78 demonstration project and DWR ground water model as a basis for that decree.

The South Platte Ditch Recharge Demonstration Project added another important dimension to the conjunctive use of ground and surface water. It showed the importance of aquifer storage using artificial recharge and the use of the recharge accretions to offset stream depletions caused by irrigation well pumping. That project clearly showed how the conjunctive use of both the surface and the groundwater could satisfy the dual water administration objectives of the 1969 Act.

Following the 1974-78 South Platte Canal demonstration project there were a number of other canal companies that initiated artificial recharge during the non irrigation season. The resultant river accretions are now commonly used to offset stream depletions resulting from well pumping.



Source: Colorado Division of Water Resources

Figure 2 Annual Volume (acre feet/year) of artificial recharge to the alluvial aquifer from Denver to the Nebraska State line, water years 1979-2012.

Data from the Division of Water Resources are available to document how the number of artificial recharge sites increased annually. Figure 2 shows the annual volume of natural recharge, acre-feet/year, that has occurred. The recharge facilities are often a shallow pond, a spreading basin, a leaky reach of canal or a land surface where water is allowed to pond and seep away. The existing irrigation canals are used to divert the water from the river to the recharge facility. All recharge structures are required to have a recording device that measure the amount and timing for the artificially recharged water. Also during this period there were a significant number of augmentation decrees entered by the Water Court.

It is important to note that in the late 1990's, major water administration problems developed in the winter non-irrigation season. The demand for artificial recharge water exceeded the supply and a year around call developed. Because of the demand for artificial recharge which was the source of accretion water needed to offset stream depletions for well pumping, it became impossible for the senior large surface reservoirs to fill with the "gentlemen's agreement." This caused those senior surface reservoirs to have to revert back to strict priority administration to fill their senior storage decrees. This resulted in a year around priority call that could not be satisfied by GASP and CCWCD. The change from the "gentlemen's agreement" to a year-around call happened in 1999 and resulted in a major change in water administration including problems for those well owners who relied upon GASP and CCWCD to allow them to pump in their own priority.

2000-2005 Conjunctive Use Period

During this period, there were several Colorado Supreme Court Cases dealing with well administration and new water laws were passed by the Legislature in 2002, 03 and 04 that had a dramatic impact on South Platte Basin water administration. These resulted in curtailment of irrigation pumping and put the 1969 Act's dual objective to maximize the beneficial use of both ground and surface water in jeopardy. In addition the 2001-2002 drought had a limited amount of river flow available for priority administration. Strict priority administration of both wells and surface rights occurred.

In the year 2000, there was a court case in the Arkansas Basin where the issue was whether the State Engineer had statutory authority to approve substitute supply plans pursuant to 37-80-120 CRS. That case did not involve well pumping. The Colorado Supreme Court in its "Empire Lodge" decision in 2001 for that case concluded that the State Engineer did not have the statutory authority to approve such substitute supply plans. This decision had direct implications for both GASP and CCWCD who had relied upon the State Engineer's annual approval of their substitute supply plans to allow their member wells to pump under their junior priority.

Both GASP and CCWCD had no recourse, but to consider obtaining a Water Court augmentation decree to replace the river depletions caused by the pumping of their member's irrigation wells. They had been able to operate for over 25 years by only having to supply 5-10 percent of the consumptive use to the Division Engineer to keep the priority of senior water right calls off the river. This coupled with the loss of the reservoir fill "gentlemen's agreement" in 1999 would mean that they would have to acquire much more water to satisfy their augmentation requirement.

GASP considered the issues they faced to obtain a Court decreed augmentation plan and after discussing those issues with its membership, the decision was made in 2003 to disband the Association. This meant that the approximately 3000 irrigation wells that had been GASP members would no longer be allowed to pump until they were covered by an augmentation plan. The water rights owned by GASP including the 10 wells that supplied the Sterling #1 Ditch were dispersed and the Association ceased to function.

CCWCD also had to evaluate whether it could continue to function. They decided to go to Court and seek their own augmentation decree, but they realized they did not own or control sufficient water to replace all the stream depletions caused by the pumping of their member's irrigation wells. They chose to restrict the pumping by their member's wells so that there was sufficient replenishment water to replace the stream depletions. They also chose to implement an aggressive program to acquire additional water that could be used to satisfy their augmentation needs. This included new ownership of senior historic rights, lease or rental of sewage effluents, construction of new artificial recharge sites, drilling and operating augmentation wells, and other miscellaneous sources. Individual members of CCWCD were assigned an annual allotment of water that could be pumped by each well. Because of the shortage of water to meet the total augmentation needs of CCWCD, this allotment was only 20-40 percent which translated to wells being allowed to only pump a few days per year. Each year and even during the irrigation season CCWCD would adjust the allotment.

The State Engineer, Hal Simpson, in 2002 again attempted to promulgate water administration "Rules and Regulations" to control irrigation well pumping to prevent injury to senior vested rights. His attempt to have "Rules and Regulations" was opposed by senior water rights and immediately litigated in the Water Court, and that decision appealed to the Supreme Court. A good description of that litigation and the simultaneous changes to the water laws by the Legislature can be found in a publication by Mike Shimmin [17].

As a result of the Colorado Supreme Court decision in the "Empire Lodge Case", the loss of the "gentlemen's agreement" concerning how reservoirs could fill, and the return of a year round priority call for surface water administration on the South Platte there was pressure for change to Colorado's water laws. Significant changes were made in each of 2002, 2003 and 2004. For this paper those changes can be summarized as follows:

- A revised statute 37-80-120-CRS clarified what type of water administrative issues could be approved by the State Engineer pursuant to a substitute supply plan.
- Specific legal notice had to be given and a hearing held if there were objectors to approval of a State Engineer's substitute supply plan.
- The flexibility for the State Engineer to make water administration decisions was eliminated and was replaced by the time consuming court systems requiring legal notice, objector's response time and hearings process that does not allow real time management decisions.
- All large capacity wells had to have a Water Court decreed augmentation plan by January 1, 2006. Because of the expected backlog in the Water Court process to obtain new augmentation decrees there was a provision allowing wells to operate pursuant to a substitute supply plan until the Water Court acted on their new application.

The new statutes were passed without any detailed analyses of how those changes would truly affect water use and availability. Would those changes assure the prevention of

injury to other vested rights? Would the 1969 Act dual objective be possible? Would conjunctive use of ground and surface water be achieved?

Nobody expected the severe drought of 2001-2002 or how it would impact ground and surface water use issues. The 2002 water year was one of the most severe droughts on record. Although precipitation and snow melt runoff returned to near normal levels in 2003 the actual water available for reservoir storage and canal deliveries was significantly below normal. Water levels measured in 185 observation wells between Denver and Julesburg in March of 2003 appeared to show, in my opinion, that the aquifer was still essentially full but some wells had been curtailed in 2002.

As a result of the year-around call on the river and the need to have a decreed augmentation plan before 1/1/2006, there was a major push to increase the development of artificial recharge projects to obtain river accretion credits for use in augmentation decrees. All of the artificial recharge structures were junior in priority to the decreed irrigation wells. During this period, the use of irrigation wells to supplement the inadequate and undependable surface water supplies diminished, artificial recharge increased and the water administration did not maximize the beneficial use of both the ground and surface water. The augmentation plan decrees not only required replacement of current but also all post-pumping depletions back to 1974. Those augmentation decrees also added a requirement that a decree owner had to have enough water in this years supply to replace the projected depletions for the next 3-5 years to cover the depletions caused by this years pumping. The new augmentation decrees required the well owner to have significantly more water to have an acceptable plan compared to previous years.

The technology and science used to compute depletions caused by well pumping and accretions due to artificial recharge relies upon mathematical equations developed by Glover, Jenkins and finite difference ground water models. Attempts to field check these to see if the natural hydrologic and physical processes actually match computed values are inadequate. Are augmentation plans over augmenting? Do they prevent injury?

2006-2013 Conjunctive Use Period

Although there was a concerted effort to get Court approved augmentation decrees prior to the January 1, 2006 deadline, there were still a large number of well owners that knew their well pumping would be restricted starting in 2006. This was true for all of CCWCD member wells.

Even those wells which obtained a court decreed augmentation plan several years earlier were impacted requiring them to have more augmentation water because of the year-around call that was now on the river. They were even required to replace post pumping depletions back to 1976 because of the year around call.

Early in 2006, the Division of Water Resources began to notify well owners who did not have either a court decreed augmentation plan or the temporary substitute supply plan

that they could not pump any water from their well. The Division Engineer reported that about 2,400 of the reported 8,000 decreed irrigation wells received such a notice. In addition because of inadequate replenishment supplies to offset computed depletions another large group of wells had their pumping volumes restricted. For example some of CCWCD's wells were allowed to pump only a small portion of their historic pumping rate. Approximately another 240 of CCWCD's wells. wells were not allowed to pump any water although they obtained a decree. In total there were approximately 4,000 of the irrigation wells that were either totally or partially curtailed.

The amount of curtailment which began early in 2006 caused severe consequences to farmers who had inadequate surface water deliveries and had historically depended upon the wells as their supplemental supply. Farmers began to report significant reduction in yield for specific crops, or in some cases, loss of all yield from certain fields. Some farmers began to leave fields uncultivated, changed back to dry land crops, or planted short season or less water demanding crops. Vegetable farmers were especially hurt because many of those crops can not stand any significant amount of drought.

The increase in use of artificial recharge to store water in the aquifer during the non irrigation season had been increasing steadily from 1974 to 2004, see Figure 2. When the 2002-2004 statute changes occurred and people recognized the need for more artificial recharge accretions to offset pump depletions, then nearly every canal company and even a fairly large number of individual farmers actively pursued the construction and operation of artificial recharge structures. The most common structures were shallow recharge ponds or spreading basins, where the stored surface water infiltrated into the soils, deep percolated to the underlying alluvial aquifer, and resulted in a rise in the ground water levels. In some instances the leakage from the irrigation canals during the non irrigation period was allowed as an artificial recharge source.

By water year 2009 the number of individual artificial recharge structures had increased to over 500. During that water year over 350,000 acre-feet of water was diverted and measured into those structures. The Division of Water Resources requires that there must be a measuring device with a recorder in order to know how much and when water was diverted into the recharge structures. That data are then entered into accounting forms and calculations made to determine how much accretion water returns to the river in time, place, and amount.

The individual court augmentation decrees contain sophisticated accounting programs based upon the Glover, SDF or Finite Difference methods for computing both stream depletions caused by the pumping and river accretions due to the artificial recharge. The Division Engineer uses these computed values to determine, if the river accretions, are larger than the computed depletions. If accretions exceed depletions, then the wells in that augmentation plan are allowed to pump. If depletions exceed accretions, then wells are prohibited from pumping in order to prevent injury to vested rights.

With over 150 decreed augmentation plans to administer and the need to make daily or weekly decisions on whether accretions exceed depletions for each of those plans, a

major responsibility for the Division Engineer and his staff is to process large amounts of data. There has been progress made to further automate the data collection and analyses process. A significant amount of the calculations are made by individuals or private companies and the net accretion and depletion numbers supplied to the Division Engineer on a prescribed time schedule. The Division Engineer's staff conducts audits of a few augmentation plans each year, but only a small fraction of the 150 plans are audited.

The State Engineer's method of analyses, to see if the augmentation plans are preventing injury, is to assure the computed accretions exceed the computed depletions in each plan. There doesn't appear to be any effort to evaluate physical data to determine, if the equations used to compute depletions and accretions actually match the gains or losses in the River. In all fairness to the Division of Water Resources, it may not be possible to conduct field experiments or studies that would allow one to determine whether the Court decreed augmentation plans reflect the physical system. Each of the technical procedures used to calculate either depletions or accretions is based on a number of hydrologic and geologic assumptions. If the assumptions are incorrect, then the computed depletions and accretions will be in error.

The Legislature passed a statute 37-92-304(6) CRS which requires each augmentation decree to have a retained jurisdiction provision to correct any inaccuracies in the way depletions or accretions are computed in the accounting forms. This author is not aware of any augmentation decrees where field data have been collected to verify the accuracy of the unit response functions (URF's) that are now fixed in each Court decree. We do not now have data available to answer the question about whether the computations over calculate or under calculate the amount of accretions needed to offset the true depletions to the river. If we are over augmenting, then the well owners are being injured because they are having to replace more water than necessary to offset their depletions. If the accretions are less than the computed depletions, then other vested water rights can be injured.

It may not be possible to conduct field studies to determine whether a single well or small group of wells is providing the correct amount of augmentation. It does appear that observation of river gains or losses between gaging stations coupled with collection of ground observation well data could be made, which would provide some answers on whether there is over or under augmentation. Some preliminary studies by Longenbaugh and Halepaska [13,14] suggest that there is significant over augmentation.

In the fall of 2007, there were a number of anecdotal reports from individual farmers, other water users, and water officials that the ground water levels were rising throughout a large portion of the South Platte Basin downstream of Denver. Those water levels were causing drainage problems in irrigated fields, failure of septic or sewer systems, flooding of basements in homes and ponding on the land surface. A check of data from existing observation wells supported those claims. Water year 2008 was dry in the spring and high ground water issues subsided.

Water years 2009, 2010 and 2011 were wetter than normal and large volumes of water were artificially recharged (350,000 acre feet in 2009 and over 250,000 acre feet in each of 2010 and 2011 water years). Additional efforts were made in the fall of 2009 to measure a larger number of the historic observation wells. Approximately 300 wells were measured in November or December of 2009 and have been measured in March/April and November/December time periods for each of the years 2010, 2011, and 2012. An analyses of those data showed that ground water levels rose in 2009-2011 and declined in the fall of 2012 because of the 2012 drought. For the 2009 through the 2011 period those water levels were near or above the all time recorded high levels except for an area north and west of Wiggins. Damages due to the high ground water levels were prevalent from Denver to Julesburg.

The cause of high ground water levels downstream from Kersey was likely, in my opinion, probably **a result of over augmentation** caused by the large increase of artificial recharge in those three years. From Denver to Kersey there was also a continuous rise in the water levels, but the cause is believed to be from the restriction on irrigation well pumping with an associated increase in direct flow canal diversions by the most senior South Platte ditches. Some of those canal diversions increased by 50 percent to account for the loss of irrigation well production.

During this 2006-2013 period there have been significant restriction of irrigation well pumping. Most of the wells have only been allowed to pump a limited amount of water under their own priority. Those that do have Court decreed augmentation plans are probably **over augmenting**. It is the restriction on well pumping and **over augmenting** of their depletion that could be the cause for the increased river flows downstream of Kersey which are responsible for the high artificial recharge rate and over augmentation. The water rights for the irrigation wells are all senior to the artificial recharge decrees. We do <u>not</u> now have strict priority administration.

Because of **over augmentation** as a result of excess river accretions the State Engineer has issued over 20 new well permits to drill new large capacity irrigation wells to irrigate lands never before irrigated in the vicinity of Sterling. A serious question arises on how can he permit new irrigation wells there while at the same time restricting senior well pumping above Kersey.

The large amount of artificial recharge has resulted in **over augmentation**, that cause high ground water levels that result in excess river flows. An analyses of the gaging station records at the Julesburg station shows that Colorado is passing more excess flow downstream to Nebraska during the non compact delivery months than it did prior to 2006. That is water that should be managed to be used by Colorado water right owners. Letting the water go to Nebraska is waste.

The high ground water levels have resulted in significant increases in consumptive use by phreatophytes. A rise in water levels of one to two feet in the band of phreatophytes along with river could easily increase the consumptive use by 100,000 acre feet/yr. That is waste of water.

Because the ground water levels are so high, it is now not possible to divert and store excess stream flows caused by snowmelt or thunderstorms. To utilize storage in the ground water alluvium the water levels must be lowered to obtain the unsaturated storage space. The large amount of artificial recharge that occurred during the 2009-2011 period clearly demonstrates how the aquifer can be used as a storage vessel using existing infrastructure. Better management of the ground water resource and changes in current water administration to meet the 1969 Act's dual objectives could easily reduce the need to import water from the west slope of Colorado.

SUMMARY

This section has been prepared to summarize some of the most important conjunctive use issue in each of the four time periods. Those points are the following:

- Individual farmers for nearly 50 years constructed irrigation wells that withdrew water from the alluvial aquifer to supplement their inadequate and undependable surface water supplies.
- The number of wells continued to increase until the 1965 Ground Water Management Act required the State Engineer to determine, if issuing a new well permit would injure other water rights. Because the South Platte River was over appropriated there weren't any new well permits issued after 1965 unless they had a court decreed augmentation plan.
- Wells were required to get a court decreed water right so they could be incorporated into the Colorado priority system for water administration.
- The 1969 Ground Water Administration Act recognized junior well rights could pump out of priority, if they replaced their stream depletions to prevent injury to other vested rights.
- The 1969 Act also mandated the dual objective should be applied for water administration: well pumping could not cause injury, but administration must also maximize the beneficial use of ground and surface water.
- The development of science and technology had advanced by 1969 to allow several different methods to be used to calculate: how, when, and where well depletions would impact river flows.
- Changes in water administration to implement the 1969 act took several different approaches. All of them had the objective to continue to allow junior wells to pump, but they must prevent injury to vested rights.
- Court decreed augmentation decrees were begun but the 1969 act did not require that as the only method of administration.
- One process which allowed junior wells to pump out of priority required well owner associations to acquire quantities of water that was then administered by the Division Engineer to keep senior water rights calls from occurring, thus allowing the junior well owners to pump in their own priority.
- The associations of wells owners, GASP and CCWCD, proceeded to operate for 25 years pursuant to their substitute water supply plans and to acquire water that was then administered by the Division Engineer to keep the senior call off the river.

- For over 25 years all the member of GASP and CCWCD as well as other well owners were allowed to pump under their junior priorities, because the senior water right calls were prevented.
- The amount of water GASP and CCWCD had to supply to suppress senior right calls was only 5-10 percent of their historic consumptive use and not the 100 percent now required by Water Court decreed augmentation plans because of the year around priority call.
- The South Platte Ditch recharge demonstration project illustrated that river accretions from the recharge could be managed to offset depletions to the river due to pumping. This was a different feature of conjunctive use.
- When the year around priority administration of both wells and surface diversions began in the 1999-2002 period this had a major impact on well pumping.
- Legislation in 2002-2004 removed the flexibility in water administration from the State Engineer and moved it to the rigorous and inefficient procedures of the Water Court. It also required all irrigation wells to have Court decreed augmentation plans by January 1, 2006 in order to pump any water.
- There wasn't any detailed hydrologic study done before these new laws were passed to evaluate the impact the new laws would have on all water users.
- Obtaining a Court decreed augmentation plan is expensive, time consuming, and may be too burdensome for irrigators. They may also be required to over augment for their well depletions.
- In 2006 approximately 4,000 of the decreed 8,000 irrigations wells were either totally or partially curtailed from pumping water.
- Artificial recharge increased slowly after 1974 and became a major component in augmentation decrees. After the 2002 drought the number of recharge sites and volumes recharged doubled or tripled.
- After the 2002 drought there has generally been a year around call except for the snow melt runoff, or runoff following a major thunderstorm event. Because of the excess river accretions in the reach from Sterling to the Nebraska state line there are free river conditions there.
- Excess river accretions are now occurring and these are being used to allow new irrigation wells to be constructed when over 4,000 decreed irrigation wells are totally or partially curtailed from pumping.
- Ground water levels through out much of the South Platte Basin downstream of Denver were at an all time record high in 2009, 2010 and 2011. Where are the cones of depression due to pumping between 1974 and 2009 that would have to be present to cause stream depletions? Post pumping depletions prior to 2009 should be canceled for most of the South Platte irrigation wells.
- There are major economic consequences of the current water administration. The total annual cost to all parties have been estimated to exceed one billion dollars.

CONCLUSIONS

In the previous section a number of summary statements were made resulting from specific changes in water administration, Supreme Court decisions and changes in the

water laws. This section focuses on some more generalized conclusions concerning conjunctive use in the South Platte River Basin. Those conclusions are:

- The South Platte River Basin is very dynamic and there are major changes in physical and hydrologic processes occurring regularly to meet the basins need for more water.
- The water administration policies and procedures changed drastically from 1999 to 2006. This resulted in significant change in how conjunctive use worked.
- Even the most senior surface water right can benefit from pumped irrigation water during droughts or at times during the irrigation season when stream flows are non existent.
- Strict priority administration of both the ground and surface rights will not result in the maximum beneficial use of both the surface and ground water supplies. This suggests that the total supply available to meet Colorado's water needs would be less, if strict priority administration was practiced.
- We do not now have strict priority administration. Wells are not allowed to consume water in their own priority while junior artificial recharge rights can divert.
- We do not know whether the science and technology now being used is accurately computing stream depletions due to pumping or river accretions due to artificial recharge. It is not possible to use existing computed values to know, if we are over augmenting or are we preventing injury to senior water rights.
- Senior water rights owners below Kersey have publicly stated that they will resist any change in water laws or water administration that would affect their current use of the excess river accretions. They are now benefiting financially from sale of excess river accretions.
- Changes in water administration must occur to return to the conjunctive use we had in 1970-1999 period. It will be a challenge to develop polices and procedures to implement the dual objectives of the 1969 Ground Water Administration Act.
- Some flexibility must be returned to how we administer water in a stream/aquifer system. There must be a way to allow additional ground water pumping during a drought followed by recharging the aquifer when there is excess supply.
- Failure to return to a more reasonable application of conjunctive use will have significant economic consequences forever. Farmers, county governments and State of Colorado are all being impacted.

RECOMMENDATIONS

It has been suggested that there is a definite need to make changes to how water in the South Platte Basin is now administered. Current water laws require that both the ground and surface water must be administered to maximize the beneficial use for the benefit of all Colorado citizens. New laws may not be necessary. To allow future water administration to be successful there are a few things that are needed:

- Adequate and reliable data must be collected and utilized to manage both the ground and surface water resources. Because of the hydraulic connection

between the stream and the aquifer, the two resources must be managed together; i.e. conjunctive use.

- A basin management authority could develop policies, procedures and make management decisions to maximize the beneficial use while preventing injury to vested water rights. The State Engineer could be the water administrator for the basin authorities decisions but should not be the basin authority. This would require cooperation from all water right owners.
- Real time basin management should be considered. Decisions on how much to pump or where to artificially recharge must consider the current status of all the water sources such as: surface reservoir storage, ground water levels, stream flows, projected snow melt runoff, expected precipitation, sewer out flows and accurate calculations of depletions and accretions. A Colorado example of such management is the Widefield channel on Fountain Creek south of Colorado Springs.
- We need to know, if the current Water Court decrees use of the URFs are accurately computing the stream depletions and accretions. If they aren't, then change should be made to those decrees using the "retained jurisdiction" provisions or else new statutes must be passed to accomplish that task.

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In preparation of this paper he wants to acknowledge the role Dr. James Ruff played to encourage him to prepare the paper. Other colleagues who reviewed the first draft and submitted comments and suggestions were: Dr. Regan Waskom, Dr. John Halepaska, Dr. Deanna Durnford and Mr. Reed Murray. There suggestions were quite valuable. Without the assistance of Mr. Lynn Kasten it would not have possible to prepare the final draft. Thank you for the assistance.

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MAPPING TURF EVAPOTRANSPIRATION WITH HIGH-RESOLUTION MULTISPECTRAL AERIAL IMAGERY AND METRIC

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ABSTRACT

Multispectral imagery with visible, near-infrared, and thermal wavebands was used to spatially estimate evapotranspiration (ET) in Berthoud, CO. The METRIC (Mapping Evapotranspiration at high Resolutions with Internal Calibration) energy balance model was used to estimate turf evapotranspiration from aerial multispectral imagery collected in 2011. The METRIC model was developed using Landsat satellite imagery but is adaptable to other satellite imagery with similar wavebands available. The METRIC model was also recently adapted for aerial imagery with limited reflective bands and a thermal band.

Following adaptation for aerial imagery, the METRIC daily ET for turf at Berthoud, CO was compared to daily ET from 44 mini-turf weighing lysimeters on 31 August 2011. Combinations of ETo (short crop or grass reference ET) vs. ETr (alfalfa or tall crop reference ET) and alfalfa vs. turf cold pixel resulted in mean percent error of 6.20 and 6.48%, for ETo and turf cold pixel and ETo and alfalfa cold pixel, respectively. METRIC agreement with lysimeter data using ETr as the reference ET ranged from 37.5% to 48.5% mean percent error for turf and alfalfa cold pixels, respectively. On 19 July 2011, mean percent error of METRIC ET and lysimeter ET using ETo and a turf cold pixel was 6.15%. METRIC applications for turf require ETo as the reference ET in the model.

INTRODUCTION

METRIC (Mapping Evapotranspiration at High Resolution using Internal Calibration) is a remote sensing model to estimate evapotranspiration (Allen et al. 2007b, c). METRIC has its roots in the SEBAL (Surface Energy Balance Algorithm for Land) model (Bastiaanssen et al. 1998a, b)

METRIC was developed as a method to estimate ET in irrigated agricultural regions from Landsat imagery. One strength was that it eliminated the need for a crop classification as it was based on well-established physical and biological parameters derived from the image bands and on-the-ground meteorological data. Recommendations for use included choosing a "cold" pixel that was part of a tall crop population. "Cold" pixels then assumed the value of 1.05x the tall crop, or alfalfa, reference ET value (ETr). For agricultural applications, ET is usually referenced to "tall" crop reference

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evapotranspiration (ASCE-EWRI, 2005), which is considered comparable to alfalfa reference evapotranspiration.

As METRIC applications have expanded, some potential limitations of the method have been encountered. Using multispectral aerial imagery, Chavez et al. (2012) found that using ETo (grass or short crop based ET) instead of ETr in a METRIC estimation in an advective environment improved agricultural crop ET agreement with lysimeter values. Also, in that study, alfalfa or other tall crop pixels were not cooler than grass (short crop) pixels in the limited aerial imagery frame and were not selected as the cold pixels.

On three dates in 2011, multispectral aerial imagery was acquired over Northern Water's Conservation Gardens in Berthoud, CO. The purpose of this paper is to compare turf mini-lysimeter ET to METRIC ET with various combinations of ETr, ETo, and alfalfa vs. turf cold pixel. Results of the comparison will be used in future analysis of turf ET at the Conservation Gardens site. Because full cover alfalfa or other tall crop fields were not consistently available in the extent of the aerial image, it was imperative to verify that turf, the main subject of the aerial campaign, could be used as a reference for the cold pixel, and to verify whether ETr or ETo was the appropriate reference ET value to use in METRIC. Turf ET is typically referenced to ETo, so it is consistent to attempt to use that value in a METRIC turf analysis.

METHODS AND MATERIALS

Imagery

Multispectral aerial imagery was acquired on 19 July, 12 August, and 31 August 2011 via the Utah State University airborne multispectral remote sensing system (Chavez and Taghvaeian, 2012). Table 1 shows time of acquisition (Chavez and Taghvaeian, 2012). Flight and camera details are also found in Chavez and Taghvaeian, 2012. Reflective band resolution was 0.2 m and thermal band resolution was 0.6 m.

Table 1. Image acquisition times (24-hr basis, Mountain Standard Time).

Date	Morning flight	Afternoon flight
19 July 2011	1131	1327
12 August 2011	1055	1242
31 August 2011	1045	1347

The aerial imagery was georegistered and radiometrically corrected. Reflectance panels were used to calibrate the reflectance bands. Minor adjustment of pixel alignment among image dates and thermal and reflectance bands standardized the images for subsequent GIS analysis.

Only the 31 August 2011 morning image was used in this analysis, as full cover alfalfa was present in the image only on that date. The afternoon flight on this date had significant cloud cover. Lysimeter and METRIC analysis from 19 July 2011 were used to independently check the model and parameters generated from 31 August 2011. The

lysimeters were hand-watered during the day on 12 August 2011, so lysimeter data were not available for that date.

Figure 1. Aerial false color image on 31 August 2011 of Northern Water's property near Berthoud, CO. The Conservation Gardens are outlined in blue. The clipped image is outlined in yellow. The alfalfa field in the analysis is located to the west of the Conservation Gardens. The orange point is the selected hot pixel; the dark blue point is the alfalfa cold pixel; the medium blue point is the turf cold pixel.

METRIC

Details of the METRIC model are found in numerous references (Allen et al., 2007a, b, c). Conceptually, METRIC calculates ET via energy balance algorithms based on short wave reflective and thermal waveband imagery, such as Landsat satellite images. The energy used for evapotranspiration is calculated as a residual of net radiation minus soil heat flux and sensible heat flux. Because the remotely sensed data are indicative of current crop status and the algorithms are based on well-established physical processes, METRIC provides a direct method of calculating ET.

While METRIC was originally developed with Landsat satellite imagery, recent applications of the model used aerial imagery and were adapted for the more limited wavebands of the aerial data (Chavez et al., 2012). In the Northern Water application, modifications were also made to accommodate the limited spectral bands of the aerial imagery.

In our application, METRIC surface albedo calculation was modified for the limited band aerial imagery following concepts and procedures in Tasumi et al. (2008), the METRIC manual (Allen et al. 2007a), and in (Brest and Goward, 2007). Atmospheric correction was based on SMARTS2 (Gueymard, 1994, 1995) output for each date and image acquisition time, following concepts in Tasumi et al. (2008) and Allen et al. (2007a).

Also in this analysis, the SAVI (Soil-Adjusted Vegetation Index, Huete, 1988) L factor (the soil-brightness adjustment factor) was set at 0.05 after testing with various values of L. As L approaches 0, the SAVI becomes equivalent to NDVI, the Normalized Difference Vegetation Index (Rouse et al., 1973). Allen et al. (2007a) recommended L = 0.1 for Idaho conditions.

The cold pixel was selected from coldest turf pixels in the Conservation Gardens and coldest alfalfa pixels in the west alfalfa field. The hot pixel was selected from the limited range of bare soil pixels in the image, primarily from the east or north edges of the north alfalfa field, where there is a narrow soil roadway and a narrow transition from field to roadway. A simple daily soil water balance was used to estimate ET of the bare soil so that $H = RN - G - ET_{bare\ soil}$.

To maintain consistency with the chosen reference ET methods, EToF and ETrF were used to extrapolate the instantaneous METRIC ET values to daily ET values.

Weather data

Weather data necessary for input into the METRIC model were obtained from Northern Water's Berthoud weather station, sited in the center of the Conservation Gardens. The weather station is maintained regularly and instrumentation checked or calibrated at least annually. More information about Northern Water's weather network, reference ET calculations, equipment, operating standards, and sites can be found here: http://www.northernwater.org/WaterConservation/BackgroundInfo.aspx.

Northern Water uses the ASCE-EWRI Standardized Reference methods (ASCE-EWRI, 2005).

Lysimeters

The 44 small turf weighing lysimeters were installed in 2009 and turf established in 2010, with 2011 the first full year of operation. Four replications with 11 turf mixes or blends were irrigated as for high quality turf. Briefly, lysimeters were 0.61 m deep, and 0.3 m in diameter, filled with a sandy loam soil. Each lysimeter was centered in a 1.22 m x 1.22 m plot of the same turf. The plot was large enough to have thermal pixels fully within the plot bounds. Areas within each lysimeter plot were digitized on pixel bounds for pixel data extraction in each plot area.

Weighing load cells were electronically logged at 15 min intervals. Details of construction and the first two years of the lysimeter study can be found in Crookston and Hattendorf (2012).

Irrigation was scheduled by soil water balance from the adjacent Berthoud weather station. A base irrigation was applied, and individual lysimeter plots were then hand-watered up to each individual lysimeter irrigation specification for that date.

Irrigation

The west alfalfa field was irrigated on several dates in 2011 (Figure 2). The cold pixels were located in a plot that was part of an alfalfa irrigation study. This plot was watered once after 1st cutting, with no irrigation after 2nd cutting. Irrigation resumed after 3rd cutting. This plot had temperatures consistent with temperatures in full irrigation plots, which could not be included in the clipped image.

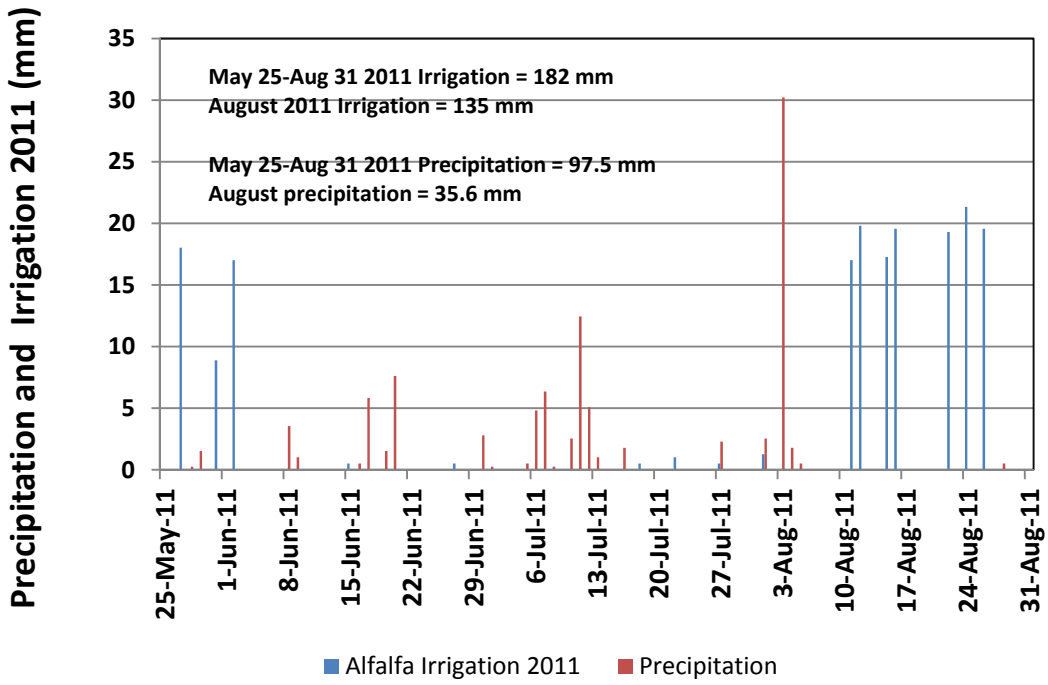

Figure 2. Precipitation and irrigation in the alfalfa plot where the alfalfa cold pixel was selected.

The lysimeters were irrigated 6 days prior to the 31 August flight. Figure 3 shows all rainfall, sprinkler irrigation, and hand irrigation in the lysimeters from 1 April 2011 to 26 Oct 2011.

GIS was used to extract individual lysimeter METRIC ET values for comparison to the weighing lysimeter data. Only data from 31 August, 2011 were used for these comparisons, as that was the only date from the three flights that had full cover alfalfa.

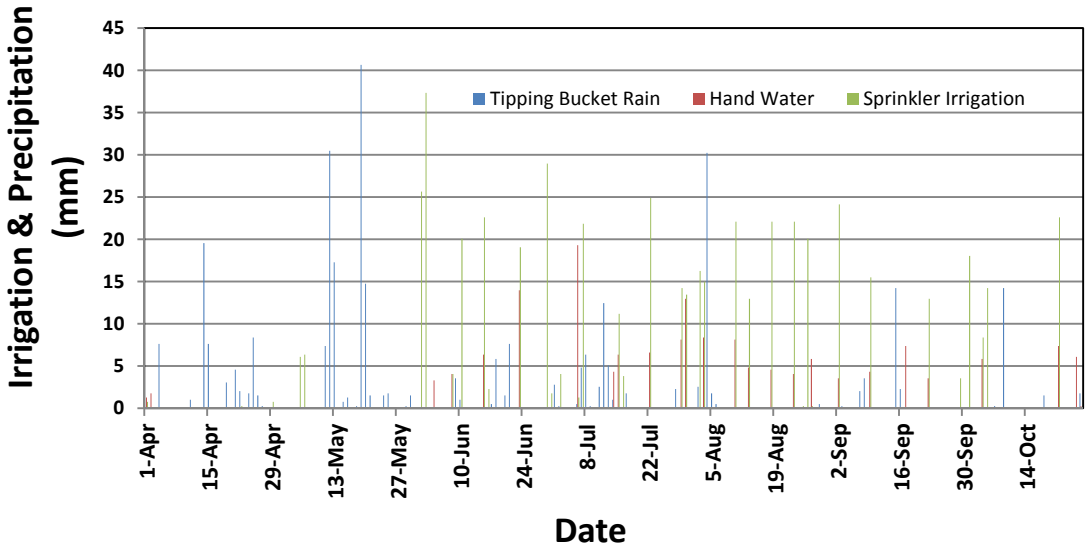

Figure 3. Lysimeter rainfall, sprinkler irrigations, and hand irrigations in 2011. Hand irrigations usually followed the base irrigation.

RESULTS

METRIC Analysis

The METRIC analysis was executed with the following cold pixel parameters (Table 2) with ETr and subsequently ETo as the reference ET used in METRIC. Each value was extracted with GIS analysis at the cold pixel point. The parameters are NDVI; LAI, leaf area index; Ts, surface temperature (deg K), and albedo. NDVI is calculated as (near-infrared [Band 4] – red [Band 3])/([near-infrared [Band 4] + red [Band 3]), with band numbers referenced to Landsat bands. NDVI is an indicator of green biomass—the higher the value, the greater the green biomass. Leaf area index is a dimensionless number commonly defined as area of leaves per unit area of ground surface. Surface temperature Ts (deg K) is obtained from the thermal band of the imagery product. Albedo is the unitless integrated surface reflectance across the full shortwave spectrum.

Table 2. Alfalfa and grass cold pixel parameters.

Cold pixel	NDVI	LAI	Ts (deg K)	Albedo
Turf	0.839	3.8	302.81	0.190
Alfalfa	0.896	5.7	299.22	0.282

Results of the METRIC analyses showed that using ETr with an alfalfa cold pixel overestimated ET values with a mean percent error of 48.54. Root Mean Square Error (RMSE) was 2.89. Error associated with all combinations of reference ET, date, and vegetation of cold pixel is listed in Table 3.

Table 3. Error associated with METRIC agreement with lysimeter data.

Date	Reference ET	Cold Pixel Vegetation	Mean % Error*	RMSE*
31August 2011	ETo	alfalfa	6.48	0.48
31 August 2011	ETo	turf	6.20	0.47
31 August 2011	ETr	alfalfa	48.54	2.89
31 August 2011	ETr	turf	37.46	2.24
19 July 2011	ETo	turf	6.15	0.42

*Mean % error = $[\sum(\text{abs}(y_{MET}-y_{LYS}))*100]/n$ RMSE= $\sqrt{[\sum(y_{MET}-y_{LYS})^2/n]}$
(MET = METRIC; LYS = Lysimeter)

Although the alfalfa cold pixel was 3.59 deg C cooler than the turf pixel, this was less important to the accuracy of the METRIC analysis than the choice of grass or alfalfa reference ET. METRIC and its parent model, SEBAL, both use a scaled dT function and thus escape the limitation of using explicit surface temperature as a driver for ET (Allen et al. 2007b, Bastiaanssen, 1998a). The upper and lower limits of evapotranspiration are effectively set by the reference ET and the multiplier chosen, in this case ETr x 1.05 or ETo x 1.05.

The ETo places a limit on the range of ET values that can be generated from an analysis, regardless of the cold pixel temperature. Choice of ETo over ETr effectively muted the maximum ET that could occur with the given parameters even for a substantially cooler pixel.

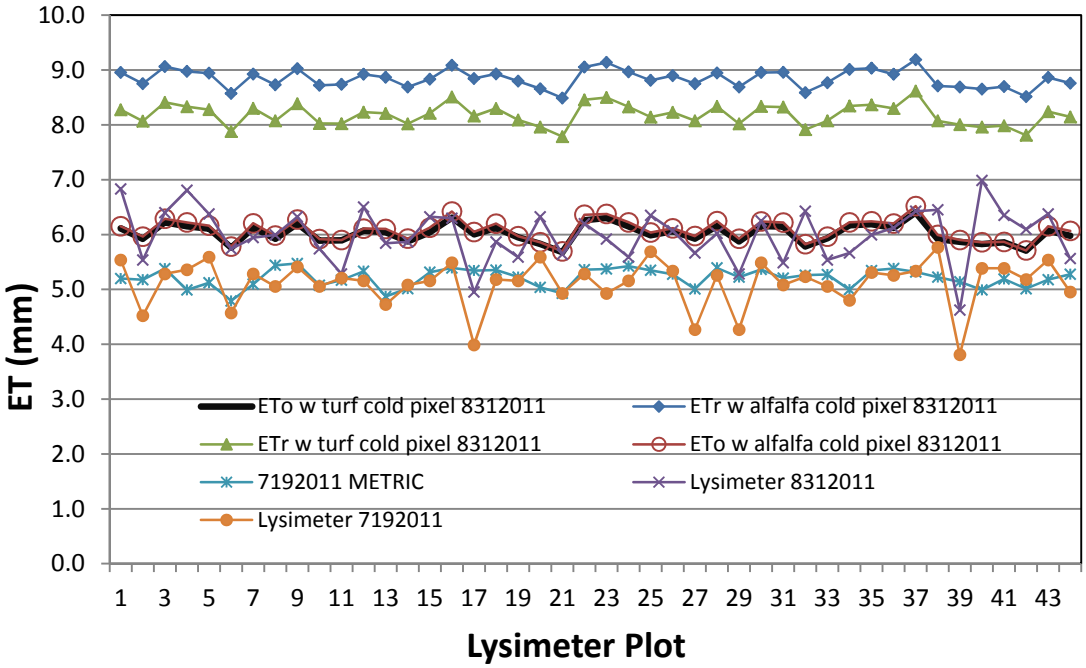

Figure 4 shows all extracted METRIC data points and the lysimeter data for 2 dates in 2011.

ETr with an alfalfa cold pixel systematically overestimated the 31 August 2011 lysimeter values. Choosing a turf cold pixel improved the estimate slightly, but METRIC still overestimated lysimeter ET. Standardized reference ET (ASCE-EWRI, 2005) is calculated at Northern Water; the separate calculations for "tall" crop and "short" crop were intended for the agriculture and landscape communities, respectively. It is clear from the results that a landscape turf application requires use of "short" crop reference ET.

Lysimeter ET for plots 17, 27, and 39 were lower than most of the other plots on both dates. These lower values were not well-tracked by METRIC. The turf in these plots (Ephraim Crested Wheatgrass) did not establish well. A factor that may have contributed to this non-agreement could be choice of SAVI L value. For this analysis, L = 0.05 yielded parameters within bounds of possible and appropriate values. This L value was more consistent with the recommended L value of 0.1 for Idaho conditions (Allen et al., 2007a) than with the original recommended L value of 0.5 (Huete, 1988).

Testing L = 0.5 led to daily ET values distant from the lysimeter values and implausible intermediate values of LAI and albedo in particular. However, further sensitivity analysis with the SAVI L value could establish a range of conditions, inputs, and plausible values. Other means of deriving the L factor, such as via MSAVI (Qi et al, 1994) might provide a more robust methodology, though this requires development of a soil line from the red and near-infrared bands.However, for specific sites, this might not be overly burdensome.

There may be other factors that weigh into lack of agreement of the METRIC-calculated Ephraim Crested Wheatgrass ET with lysimeter ET, but because L was extensively tested over a range of values for this analysis, further refinement in selecting this factor may be necessary.

CONCLUSION

Expanding applications of METRIC create challenges in choice of reference ET and cold pixel selection. METRIC must be run with ETo as the upper limit for turf applications, at least for the ASCE-EWRI ETo formulation. Using ETr as the upper limit generated METRIC ET much higher than lysimeter ET. It is therefore consistent to search for a well-watered turf cold pixel, but in this analysis, it worked nearly equally well to choose a cooler alfalfa pixel. ET even from a colder pixel than in well-watered turf is constrained by the ETo*1.05 ceiling.

When using ETr, separation of the alfalfa and turf cold pixel ET estimates are spread apart because ETr embraces the full range of ET available to the agricultural or turf world.

This is apparently a limitation of METRIC: while a crop classification may not be genuinely necessary in most contexts, regional ET where there are large acreages of turf or sod farms may be significantly overestimated. By extension, ET of "short" crops with aerodynamics similar to turf may also be overestimated.

Potential limitations of this application may include methods of selecting the SAVI L value. In this analysis, the METRIC ET output could be verified against the lysimeter ET after testing a range of L values. However, most applications will not have this level of validation available. Experimentation with various suggested SAVI L values, such as L = 0.5, yielded ET and other parameters out of bounds of established typical values. It may be worthwhile to explore different methods of acquiring an L value, such as MSAVI.

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REMOTE SENSING OF IRRIGATION REQUIREMENTS IN WINE GRAPES

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ABSTRACT

Most vineyards within California need to be irrigated in order to achieve optimum yield and quality levels. As vineyards compete with other crops for access to irrigation water, its availability is expected to become a major issue in the short term future. Traditionally, irrigation scheduling has been performed on the basis of replenishing a fraction of vineyard evapotranspiration, calculated via the use of generic potential crop coefficients (Kcp), which might not reflect the true characteristics of each wine growing region in terms of potential canopy growth and vine water demands. Remote sensing of vineyard water requirements stands as an opportunity to accurately estimate vine water use, providing real time estimates of vine actual and potential evapotranspiration (ETa and ETp respectively).

In this study, we first validated the energy balance algorithm at the land level (SEBAL) in several commercial vineyards located within the Central Valley of California. For this purpose, we used estimates of crop coefficients (ETa/ETo), potential crop coefficients (ETp/ETo), and NDVI, all derived by SEBAL North America from Landsat TM 5 images collected at different times during the 2011 season. Potential crop coefficients obtained from SEBAL were compared with field measurements of potential water use collected over several locations in multiple commercial vineyards, with the results showing a very good accuracy of the SEBAL model (error +/- 5% across multiple vineyards and varieties). Actual fraction of reference ET (ETa/ETo) for 876 commercial vineyards was plotted against NDVI to obtain a baseline for potential water use.

These findings will be used to improve water use efficiency via the development of site specific potential crop coefficients, considering factors such as cultivar, trellis system, and rootstock among others. We also aim to establish biomass thresholds for achieving intended yield and quality targets, while maximizing water use efficiency.

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INTRODUCTION

Most vineyards within California need to be irrigated in order to achieve optimum yield and quality levels. As vineyards compete with other crops for access to irrigation water, its availability is expected to become a major issue in the short term future, and vineyard water use efficiency will likely become a major driver of standard winegrowing management practices.

Traditionally, irrigation scheduling has been performed on the basis of replenishing a fraction of vineyard evapotranspiration, calculated via the use of generic potential crop coefficients (Kcp), which might not reflect the true characteristics of each wine growing region in terms of potential canopy growth and vine water demands. Remote sensing of vineyard water requirements provides an opportunity to accurately estimate vine water use, providing real time estimates of vine actual and potential evapotranspiration (ETa and ETp respectively). Although used in other horticultural crops, models such as the Surface Energy Balance Algorithm for Land (SEBAL, Bastiaanssen et al., 1998, 2005) or the Mapping EvapoTranspiration with high Resolution and Internalized Calibration (METRIC, Allen et al., 2007) are not widely used in viticulture.

Potential crop coefficients can be measured from the ground by estimating ground coverage (Williams and Ayars, 2005), but coverage of vineyard spatial and temporal variability is often a challenge for such methods.

Vegetation indices such as the normalized difference vegetation index (NDVI) have been widely used to characterize vineyard variability, although their use and practical application for irrigation scheduling has not been completely understood. The main use of this and other similar vegetation indexes may be the estimation of potential water requirements (Tasumi et al., 2005).

MATERIALS AND METHODS

Vine actual and potential evapotranspiration (ETa and ETp respectively) were estimated using SEBAL and used to calculate potential and actual crop coefficients. Potential crop coefficients were measured from the ground for comparison to the remotely sensed values. The following two sections describe the materials and methodology of the remote sensing and the ground based measurements.

Remote Sensing

SEBAL solves the energy balance at the Earth's surface, accounting for all major sources (net Radiation, R_n) and sinks (latent heat flux, LE; sensible heat flux, H; and soil heat flux, G) of energy (Figure 1). LE, which is equivalent to $R_n - G - H$, is converted to ET based on the latent heat of vaporization (λ) and density (ρ_w) of water.

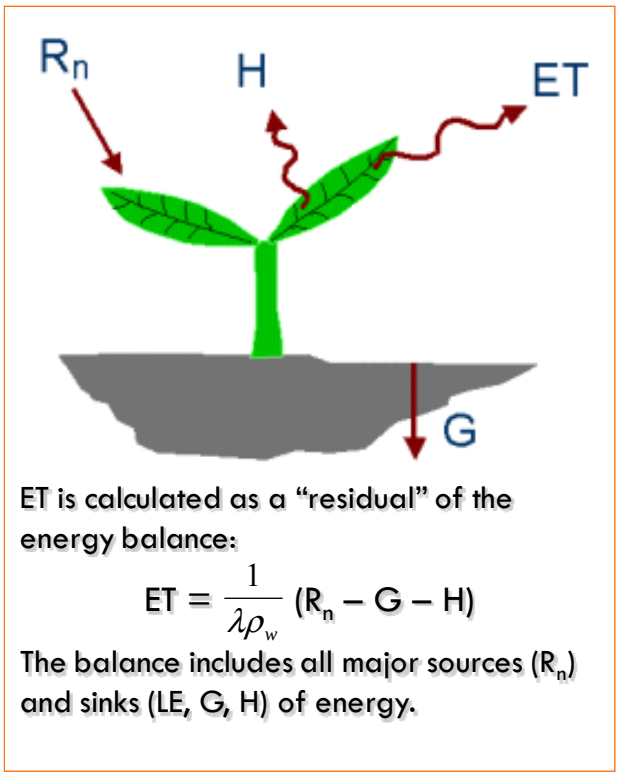

Figure 1. Conceptual Representation of Surface Energy Balance.

SEBAL is applied using multispectral Landsat satellite imagery representing the visible and infrared regions of the electromagnetic spectrum, a digital elevation model, surface roughness estimated from gridded land use data, and gridded weather data. The primary output is actual evapotranspiration (ET_a) at the pixel scale. For the analysis presented herein, four Landsat 5 images spanning the period July to September 2010 were selected and analyzed. Additionally, in 2011, six Landsat 5 images were analyzed from May through July.

SEBAL has been updated over time based on advances in surface energy balance science. These advances include both published and un-published refinements. The 2009 version of SEBAL used for this study includes the following updates from the originally published version of the model:

- Topographic correction of incoming solar radiation based on actual surface slope and aspect;
- Lapse rate correction of observed surface temperatures to normalize for elevation effects;
- Use of spatially distributed weather surfaces integrating multiple weather stations for improved representation of actual surface conditions;
- Advection correction based on comparison of instantaneous and daily evaporative fractions estimated for a grass reference surface at each pixel, which is used to compute a theoretical advection correction factor, which is then adjusted based on the actual instantaneous evaporative fraction;

- Atmospheric correction and calibration of albedo; and
- Improved soil heat flux estimation based on a combination of LAI and estimated soil moisture.

Ground-based Measurements

Four different plots within 42 drip-irrigated commercial vineyards covering multiple cultivars and located across the Central Valley of California were selected for this trial (Table 1). The experimental plots were selected on the basis of differences in vine vigor (i.e. NDVI) among them, with the objective of testing the SEBAL model against field measurements of potential water use during the 2010 season. Each plot was laid out as a square of 60 by 60 meters. Estimates of crop coefficients (ETa/ETo), potential crop coefficients (ETp/ETo), and NDV were derived by SEBAL North America from Landsat TM 5 images collected from May to July during the 2011 season, and used for the validation process. Since vineyard evapotranspiration (ETa) is affected by the interaction between current irrigation levels and atmospheric demand, both hard to estimate while covering a large area, we focused on potential water use (ETp) and potential crop coefficients (Kcp=ETp/ETo). Potential crop coefficients obtained from SEBAL were compared with field measurements of potential water use collected within each plot. All ground measurements of potential crop coefficients were performed when shoot growth stopped (maximum potential water use), and replicated ten times within each plot. Field estimates of potential water use consisted of measurements of ground coverage taken with a portable solar panel, with readings taken above and below the canopies (University of California Cooperative Extension, http://cesanluisobispo.ucdavis.edu/Viticulture/Paso_Panel/). Values were then used to estimate potential crop coefficients as described by Williams and Ayars (2005).

Table 1. Cultivars and # of vineyards used for the ground validation of SEBAL during the 2011 season.

Cultivar	*# of vineyards*
Chardonnay	8
Cabernet Sauvignon	10
Merlot	13
Petit Syrah	1
Pinot Noir	3
Riesling	2
Symphony	2
Syrah	3
TOTAL	42

Furthermore, NDVI and ETrF (ETa/ETo) values obtained from SEBAL North America between bloom and veraison were averaged on a vineyard basis, and then compared against each other for all dates. A total of 876 drip-irrigated commercial vineyards located across the Central Valley of California were used for this purpose, covering a wide range of the main white and red wine grape cultivars.

RESULTS AND DISCUSSION

SEBAL results for an image acquired June 23, 2011 are shown in Figure 2 for a group of commercial vineyards west of Lodi, California. In the figure, a true color, high resolution satellite image is provided, followed by SEBAL results for NDVI, ETa, ETrF, and Kcp.

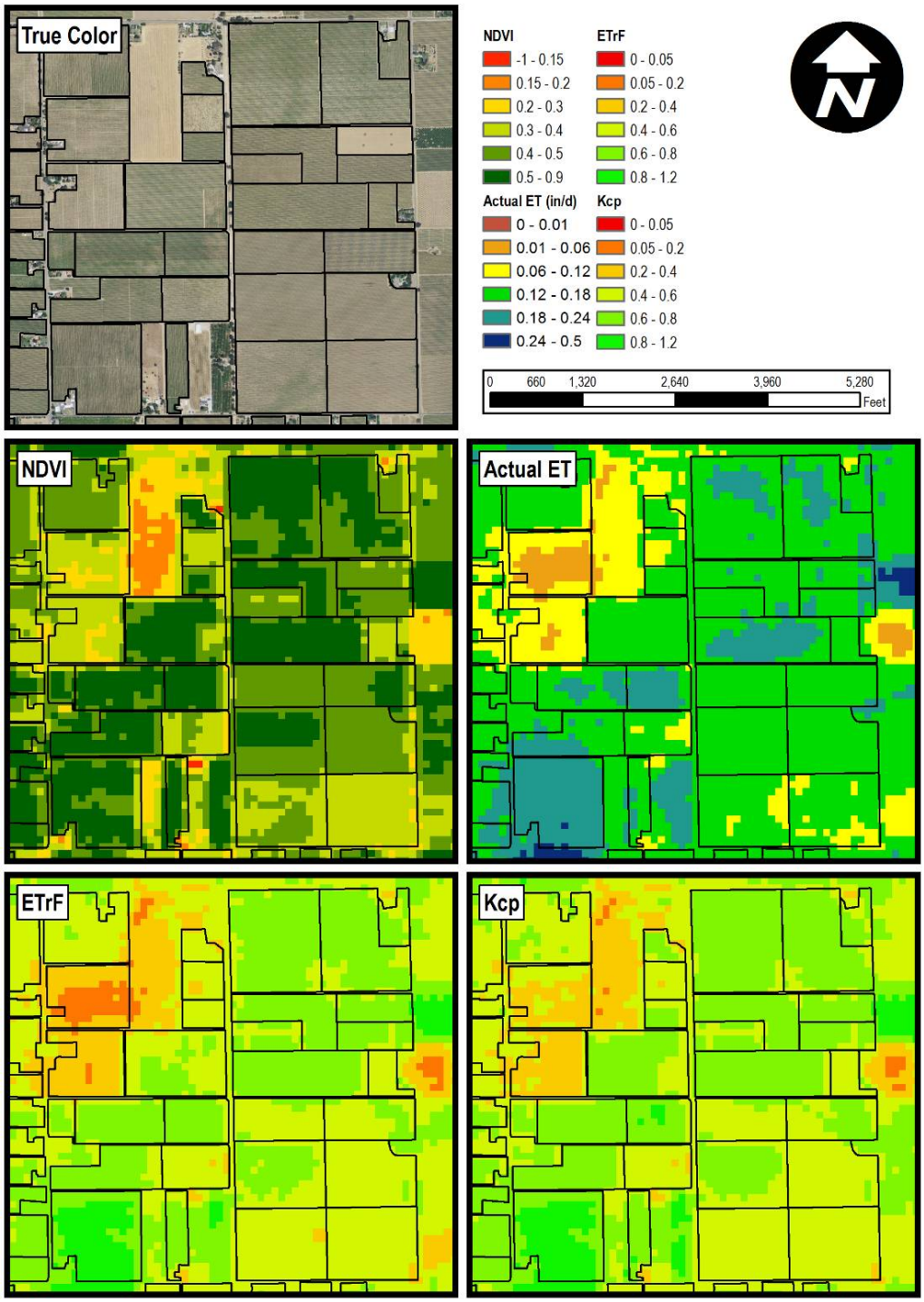

Figure 2. SEBAL Results for Commercial Vineyards West of Lodi, June 23, 2011.

We were able to show a good correlation between the reference model most commonly used in wine grapes for calculating potential water use (Williams, 2005) and the estimates obtained from SEBAL. There were no significant differences between Kcp values obtained from SEBAL and those estimated under field conditions for any of the cultivars under the scope of the study, and the overall error was +/- 5% (Figure 3). Furthermore, we were able to show a wide range of reference ET Fraction (ETa/ETo) among the commercial vineyards under the scope of the study (Figure 4). The relationship between ETrF and NDVI will be used to define baselines for potential water use by cultivar, trellis, and growing conditions.

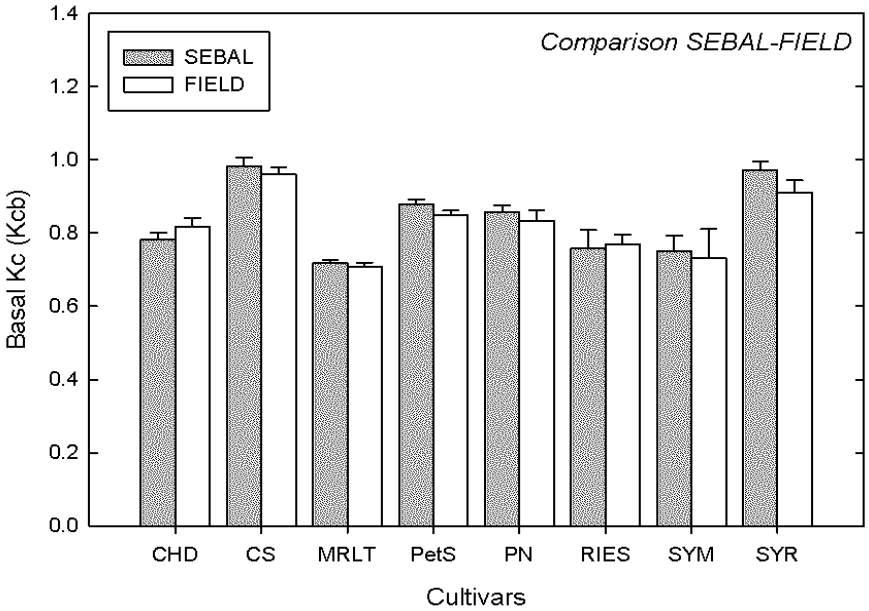

Figure 3. Comparison between potential crop coefficients obtained using SEBAL and field measurements obtained by using the method developed by Williams (2005). Error bars represent one standard error.

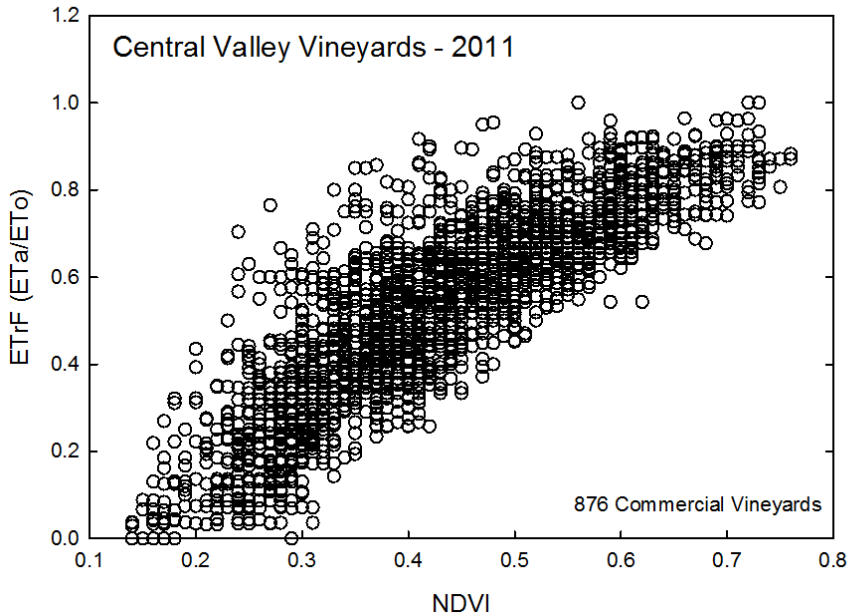

Figure 4. Reference ET Fraction (ETa/ETo) and NDVI for 876 commercial vineyards located within the Central Valley.

CONCLUSIONS

Remote sensing of water use is an accurate option for commercial vineyards. The values estimated by the SEBAL model were not significantly different from those measured at the field level using the standard method previously established for wine grapes (Williams, 2005). Vegetation indexes such as NDVI can be used to estimate basal crop coefficients in vineyards (Kcp). The findings of this study will be used to improve water use efficiency via the development of site specific potential crop coefficients, considering factors such as cultivar, trellis system, and rootstock among others. We also aim to establish biomass thresholds for achieving intended yield and quality targets, while maximizing water use efficiency.

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BENEFITS OF THE SOUTH SAN JOAQUIN IRRIGATION DISTRICT'S PILOT PRESSURE IRRIGATION PROJECT

Jeff Shaw[1]
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ABSTRACT

The SSJID board commissioned Stantec Consulting as a partner in developing an irrigation program that could improve delivery efficiency and service. A portion of one of the District's nine divisions – 3,800 acres in Division 9 – was chosen as the site for building, testing, and optimizing a pilot pressure irrigation project. The vision for the system included the following fundamental capabilities:

- Pressurization – pumping water from a 56-acre-foot pond to individual farms through 19 miles of pressurized pipeline
- Calculated use – letting farmers choose the time, volume, and flow rate of deliveries
- Automated/mobile access – developing a web-based tool that allows farmers to schedule deliveries from a computer, smart phone, or iPad based on current and past weather forecasts, previous water usage and historical evapotranspiration rates, and orchard moisture sensors.

This paper will focus on the realized benefits for the SSJID and the Division 9 farmers including but not limited to: improved service to crops, volumetric billing compliance, improved irrigation flexibility(duration, frequency, flow rate), water conservation, reduction in farmer energy costs, reduced groundwater pumping, improved air quality, improved yields, reduction in labor inputs, automatic delivery information for billing, increased pumping efficiency, increased District enrollment, protection of water rights, improved flood delivery service and efficiency, intelligent irrigation scheduling, and improved management of flows through a regulating reservoir.

INTRODUCTION

The South San Joaquin Irrigation District (SSJID) has historically delivered water to farmers through 400 miles of gravity-based canals and pipelines. Farmers drew from the network of laterals at scheduled times via flood irrigation or private pumps used for sprinkler or drip systems.

While the system works well for flood irrigation, the combination of flood and sprinkler usage on a single system becomes problematic. As a result, some customers did not buy water from the SSJID, opting instead to draw from their private, salinity stricken wells.

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The SSJID board commissioned Stantec Consulting as a partner in developing an irrigation program that could improve delivery efficiency and service. A portion of one of the district's nine divisions – 3,800 acres in Division 9 – was chosen as the site for building, testing, and optimizing a pilot pressure irrigation project.

The newly completed pressurized irrigation system is among the most water-efficient in the agricultural industry. Designed to be an industry model for water efficiency and provide area growers with individualized, automated irrigation access through the use of online and mobile technology, the new system was designed and constructed over a three-year period. Design of the new system was developed and implemented as a cooperative effort between Stantec Consulting and the SSJID.

The project consists of a 19-mile network of pipelines with flexible pressurization (currently set at 60 psi), a 56-acre-foot water storage basin, a 1,225-hp pumping station containing seven vertical turbine pumps capable of pumping a total of 23,500 gal/min (52.4 ft^3/s), and a total of 55 solar-powered Field Telemetry Units or FTU's controlling 77 customer connections. The FTUs consist of a PV panel, a flow control valve and meter, and a radio based supervisory control that communicates with data acquisition (SCADA) system in the pump control room.

With the new system, irrigation water is distributed to the customers across 3,800 acres of California's Central Valley through an automated channel. Using an online system similar to an airline ticketing platform, growers in the District's Division 9 are able to log-in and schedule water deliveries. Additional information on current and past weather forecasts, previous water usage, historical evapotranspiration rates, and real-time moisture sensor readings are also available on the website. Each farmer selects from available delivery dates and receives alerts via email and text message before and after delivery to confirm volume and flow rate data. To promote efficient water usage, moisture sensors placed in the ground on each grower's property will help indicate optimal ordering times when orchards are at their greatest need. This paper will focus on the realized benefits of the Division 9 Pressure System Project.

IRRIGATION SERVICE

An irrigation service study was conducted in 2012 by Davids Engineering in conjunction with the District's On-Farm Conservation Program to assess the current service quality the District is providing to its customers. Survey results related to irrigation water availability, flow rate, and duration are detailed below. As a comparison, customers of the pressure system are now receiving irrigation water at the exact time, flow rate, pressure, and duration they desire. In addition, the reduced number of customers using the gravity system has allowed the flood runs to be accomplished faster and more efficiently, with less stress on the previously overloaded gravity system and reduced long term maintenance costs.

Availability

Figure 1 shows the percent of surveyed SSJID farmers for each irrigation application method who irrigate based on surface water availability. With the Division 9 pressure system, zero farmers irrigate based on surface water availability; the farmers irrigate when their crops need it.

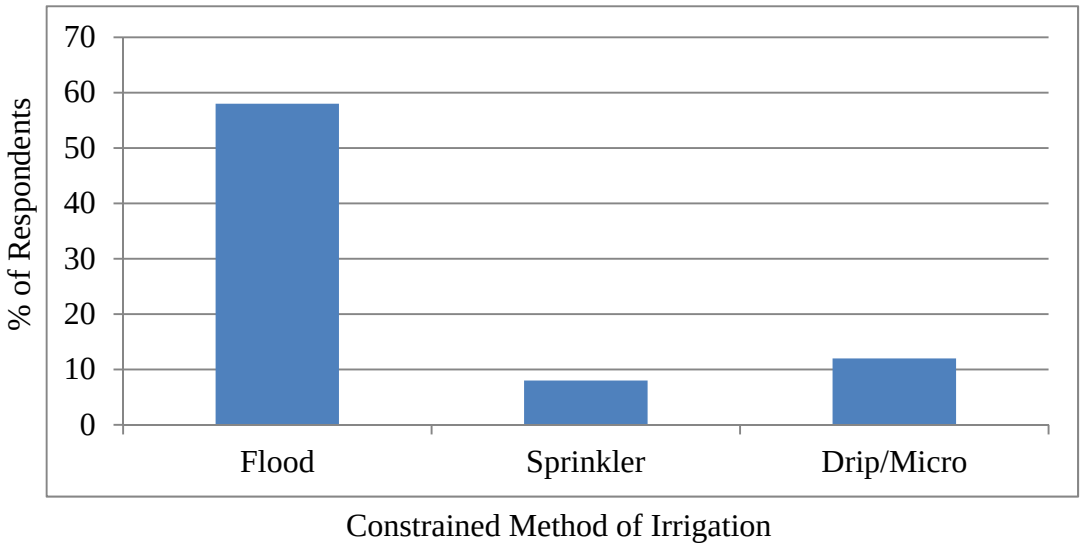

Figure 1. Percent of surveyed SSJID farmers for each irrigation application method who irrigate based on surface water availability.

Flow Rate

Figure 2 shows the percent of surveyed SSJID farmers for each irrigation application method who determine flow rate based on District infrastructure system flow constraints and/or District infrastructure turnout system capacity. With the Division 9 pressure system, zero farmers determine flow rate based on District infrastructure system flow constraints and/or District turnout system capacity; the farmers irrigate at the exact flow rate they desire.

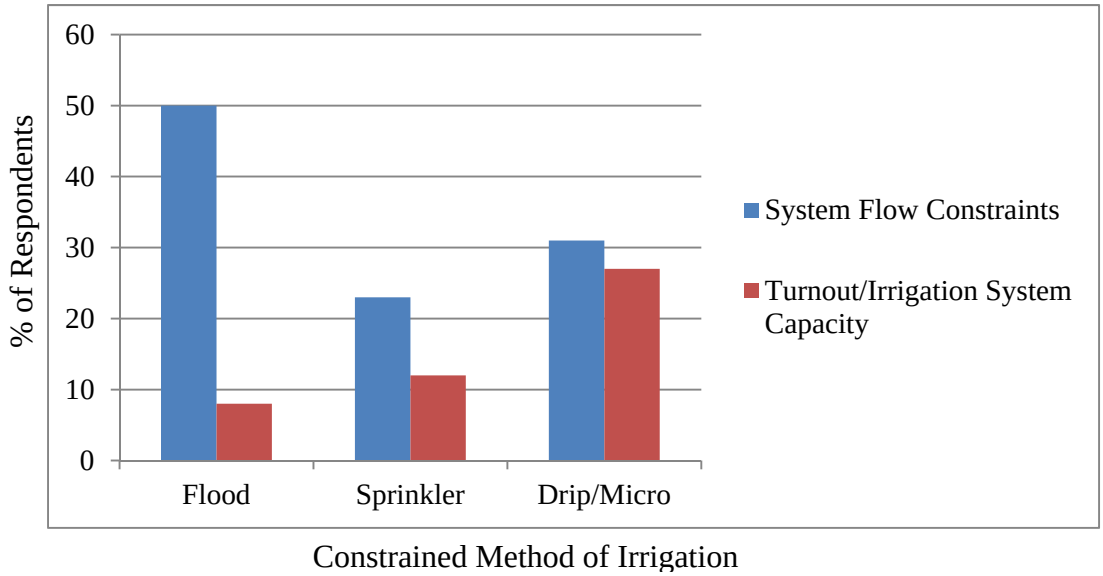

Figure 2. Percent of surveyed SSJID farmers for each irrigation application method who determine flow rate based on infrastructure limitations.

Duration

Figure 3 shows the percent of surveyed SSJID farmers for each irrigation application method who determine irrigation duration based on infrastructure constraints. With the Division 9 pressure system, zero farmers determine irrigation duration based on District infrastructure delivery system constraints; the farmers irrigate for the exact duration they desire.

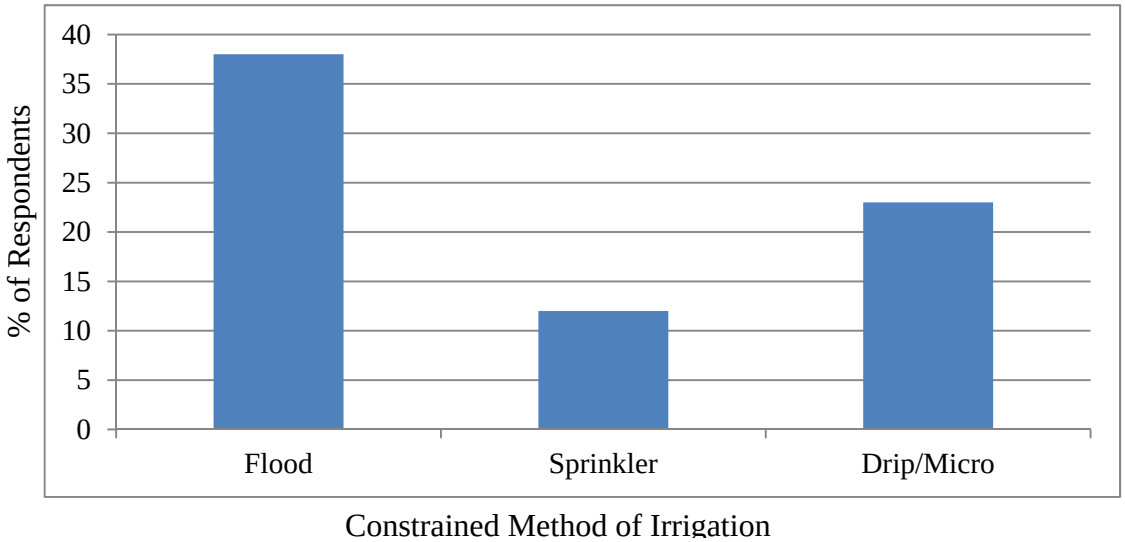

Figure 3. Percent of SSJID farmers for each irrigation application method who determine irrigation duration based on infrastructure constraints.

WATER CONSERVATION ACT COMPLIANCE

A growing population and competing demands for limited water resources prompted California to pass the Water Conservation Act. In addition to a 20% reduction in per-capita urban consumption by 2020, the law requires agricultural suppliers to "implement efficient water management practices" and volumetric pricing. With a statewide assessment of water use under way, the SSJID Board of Directors realized the issue posed a potential threat and approved the Division 9 pressure system upgrade to demonstrate the District is proactively addressing California's conservation goals. The Division 9 system efficiently manages water delivered by reducing water needs by up to 30%(Dunbar, 2012) and accounts for water use through magnetic flow meters at each customer connection.

VOLUMETRIC BILLING

The farmers in the SSJID service area have historically been charged a flat rate of $24/acre for irrigation water. To come into compliance with the Water Conservation Act, the SSJID is required to bill water deliveries volumetrically. A typical 40 acre orchard has numerous valve structures used to flood irrigate the land. This poses a very difficult and expensive challenge for the District to comply with due to the thousands of exit points off of the gravity system. With the Division 9 pressure system, each customer has one connection point, with a magnetic flow meter to measure and transmit water deliveries; historic data are automatically stored on the District's server for uploading into the District's billing software.

RELIABILITY AND ACCESS TO DISTRICT'S WATER SUPPLY

The District's fixed, 10-day delivery schedule does not provide an optimal water supply at the frequency needed to maximize yield of crops. The Division 9 pressure system's East Basin Pump Station doubles as a regulating reservoir, storing and pumping irrigation water to 77 customer connections on an on-demand basis.

CONVERSION FROM FLOOD TO SPRINKLER IRRIGATION

The introduction of the Division 9 pressure system induced a demand to convert from flood irrigation to sprinkler/drip application methods. Of the 77 customer connections in the system, 18 have installed sprinkler or drip systems immediately after the pressure system was available to serve their land. The increased use of sprinkler/drip increases the irrigation efficiency of the farming operation and contributes to the goal of maximizing beneficial use of the District's water rights.

RENEWABLE ENERGY

The abundant sunlight in the Central Valley of California is one of the reasons why agriculture is so successful. The Division 9 pressure system taps this readily available solar energy to meet the power demands of all of the customer connections. The solar

system powers the solenoids of the flow control valve, magnetic flow meter, moisture sensors, process logic controller, and radio communications to operate the turnouts and provide real time information on flow rate, crop moisture conditions, turnout pressure, control and battery component status, and delivery details(start time, end time, total hours irrigated, average flow rate, total water delivered).

WATER CONSERVATION

The Division 9 pressure system includes a number of conservation features that contribute to the District's water savings. These measures include drip and sprinkler conversions, a tail water recovery system, intelligent irrigation scheduling, and soil moisture monitoring.

From a conservation perspective, delivering the right amount of water(and nothing more) to the District's irrigation customers through 400 miles of gravity based pipelines spanning approximately 72,000 acres is very problematic and often times causes spills to the drain. During drought years, when water conservation is paramount, infrastructure that allows precise and accurate water deliveries that match a farmer's actual water needs is a crucial asset to ensure that the water needs of all of the District's customers can be met. Frank Avila, SSJID's telemetry and SCADA manager, reports that the new pressure system reduced spills to the drains in Division 9 by 5,000 acre-ft in the 2012 irrigation year.

The ten year average water supply (2002-2011) to the Division 9 pressure system customer base has been 7,528 acre-feet. The summation of water deliveries (calculated via magnetic flow meters at each customer connection) through the pressure system for the first year amounted to 4,695 ac-ft. Thus, a 2,833 acre-ft conservation has been achieved.

On a water delivered per acre basis, the savings are magnified because 50% of the customers of the pressure system were using their own wells prior to the pressure irrigation system being constructed. With the introduction of the Division 9 pressure system, the District was able to re-enroll these farmers and get them to reduce ground water pumping and use higher quality surface water. Prior to the pressure system, 19,924 acre-ft of water was delivered to Division 9 to support 3,151 acres, or 6.32 ft of water per acre. Water deliveries for the pressure system customer connection for the 2012 irrigation year amounted to 4,695 acre-ft to support 2,389 acres, or 1.96 ft of water per acre. In addition to the Division 9 water conservation Davids Engineering found that SSJID's On-Farm Conservation Program is also producing water savings using many of the same measures featured in the Division 9 project.

Table 1. Water conservation results from SSJID's On-Farm Conservation Program (Davids, 2012).

Conservation Measure	Fields Evaluated	% of Fields Evaluated for each Conservation Measure	Acres	% of Acres Evaluated for each Conservation Measure	True Point Deliveries, ac-ft (March - October)		Preliminary Conservation Estimate	
					2010	2011	ac-ft	inches
Drip Conversion	8	53%	379	54%	1093	719	374	11.8
Sprinkler Conversion	4	80%	220	90%	472	373	99	5.4
Tail water Recovery	0	NA	0	NA	NA	NA	NA	NA
Grower Proposed	1	11%	25	10%	100	101	-1	-0.6
Irrigation Scheduling	7	30%	278	30%	996	721	275	11.9
Soil Moisture Monitoring	47	61%	1,497	58%	5,242	4,695	547	4.4
Totals	67	51%	2,399	45%	7,902	6,608	1,294	6.5

FARMER OPERATING COSTS

A case study was conducted at the end of the 2012 irrigation season on three customer pressure system turnouts that previously elected to not take District water and use their own wells to pump water from the groundwater aquifer. When the current diesel fuel costs and their historical records of run time hours are compared to the $30/acre-ft the District charges for delivered pressurized water, the farmers at these three locations are experiencing between 34 – 67% reductions in costs for pressurized irrigation water. These numbers do not factor in costs the farmers historically incurred to maintain their personal pumps and the increased labor costs of a manual irrigation operation compared to the fully automatic pressurized water the District now supplies.

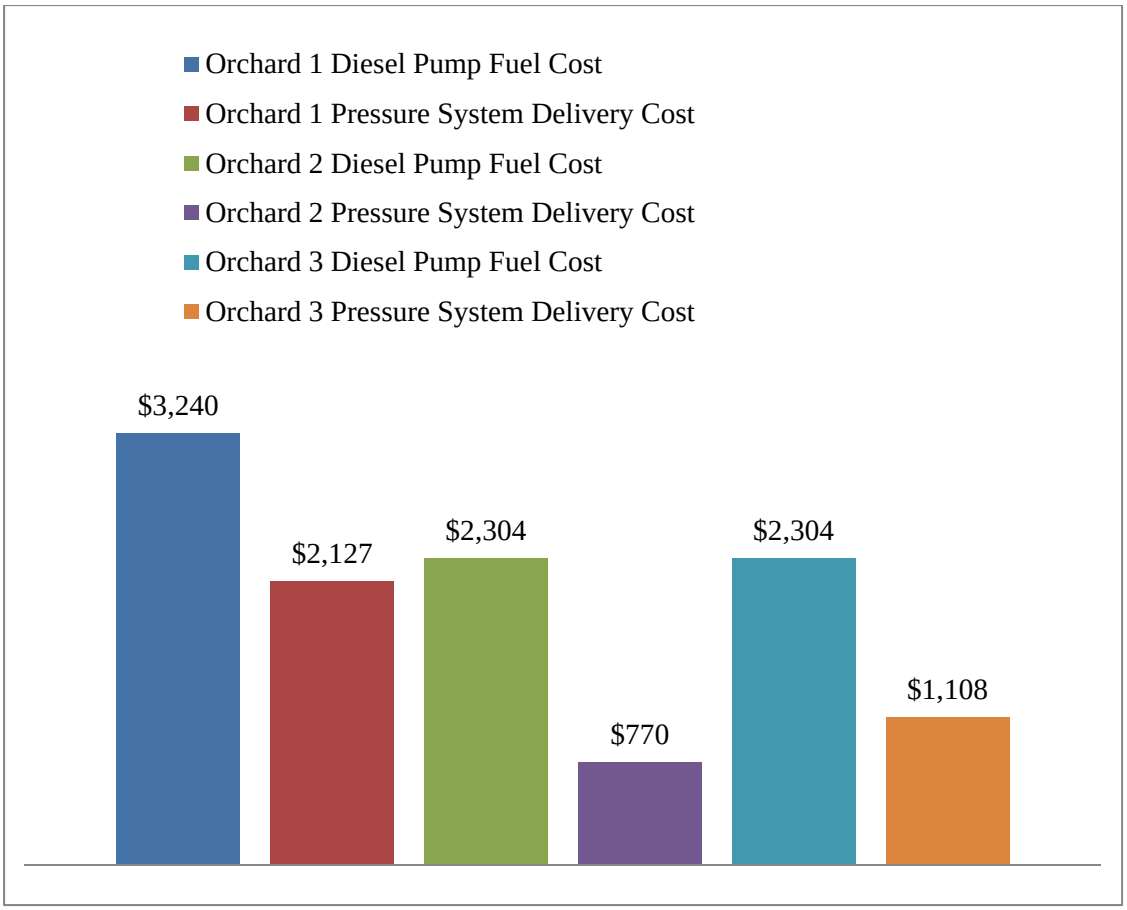

Figure 4. Comparison of farmer fuel cost to pump water from private well to meet irrigation demands vs. SSJID supplied pressurized water for a peak summer month.

Table 2. Farmer savings from pressurized system compared to energy costs to pump from private well.

	Farmer Savings (%)
Orchard 1	34%
Orchard 2	67%
Orchard 3	52%

GROUNDWATER PUMPING AND AIR QUALITY

The irrigation service component of this project was conceived because growers in this immediate area were coming into our office and complaining that the groundwater was getting too salty to apply to permanent crops and the cost of running pumps was chewing up profits," SSJID General Manager Jeff Shields said(Campbell, 2012). The Division 9 pressure system has reduced the acreage pumping from the ground water aquifer by 50% according to the Division 9 Manager Michael Donahue. Groundwater pumping in the

Division 9 area was primarily conducted using diesel driven pumps. The reduction in diesel emissions has improved the air quality, and the high quality surface water has improved crop (primarily almond and walnut orchards) health.

FERTILIZER APPLICATION

Farmers participating in the pressure system to a large extent are using direct injection of fertilizer at their filter stations and delivering chemicals directly to the root zone area; reducing the deposition of fertilizer in the local surface and ground water.

IRRIGATION SCHEDULING AND ACCOUNT MANAGEMENT

To provide a manageable pressure system for both the SSJID and the customers, a user-friendly software interface to replace the practice of phoning in delivery orders was crucial. Since creating an interactive tool required a two-way conversation, sizeable portions of early community meetings were devoted to gathering insight into the features, capabilities, and information farmers felt would be helpful for scheduling deliveries online. Through a web-based interface entitled “The Division 9 Irrigation Information Center”, each farmer has been assigned a unique platform to service all of their irrigation related needs. Tools at the farmers fingertips to plan their irrigations includes national weather service alerts for the area (including frost and wind alerts), weather forecasts, Doppler radar imaging, customizable and exportable/printable charts on past weather (rainfall, wind, temperature, humidity, evapotranspiration rates), water deliveries (time start, time end, total hours irrigated, average flow rate, and total water delivered), and moisture sensor information. After the farmer has analyzed all of the information, an irrigation can be completed with only three selections. Via an airline ticketing type calendar, a farmer selects a date, followed by the number of hours of irrigation desired. The web site immediately queries system capacity for the requested date, times, and flow rate, and displays available times for the requested day (order time options are normally available on the hour every hour). If there are times unavailable for the requested day, the system gives all time options 48 hours before and after the requested day for the farmer to select. After the farmer selects the optimal time, text and email alerts notify the farmer (24 hours and 1 hour prior to irrigation, and an irrigation delivery summary after the order has completed). Full account management is available through the web site to keep the customers apprised of their records on file at the District.

NEXT GENERATION OF FARMERS

While it’s a stark departure from the way that agriculture has been approached in the area for decades, younger generations that are set to take over operations at some point are more likely to pick up and embrace the new technology. It is something that SSJID Engineering Department Manager Sam Bologna said he has already seen playing itself out. “We had a father out here with his son that will more than likely take over the operation and he’s already up to speed on the system – we expected that would be the case with the younger generations that are savvier with technology,” Bologna said(Campbell, 2012).

YIELDS

Although it will take many years of data to figure the increase in yield due to the Division 9 pressure system project, evidence from other case studies leads the District to believe that farmers will see an increase in yield of up to 30% according to the General Manager of the SSJID, Jeff Shields.

PESTS AND DISEASE

With increased control of irrigation timing, duration, and application rate, there has been a marked decrease in pests and disease associated with water delivery.

PUMPING EFFICIENCY

A major benefit from the Division 9 pressure system project has been the consolidation of pumping operations. Historically, farmers used private motors and pumps that were diesel driven and provided much less water per unit of energy input. With the construction of the Division 9 East Basin Pump Station, seven 480V variable frequency drive vertical turbine pumps deliver water in a much more efficient manner. The pump station has two 50 hp pumps for low flow operation, one 125 hp pump, and four 250 hp pumps with a combined pumping capacity of 23,500 gpm.

CONCLUSION

The clear winner for this design innovation is the community of farmers that make up Division 9 of SSJID. For the first time, these farmers get water exactly when they need it at the pressure and flow rate they desire. Since the valves are automatic and the web based interface allow management through an Internet connection (smart phone and iPad compatible), farmers can concentrate on other aspects of their farming operation. Since gravity water was often not available when farmers needed it, groundwater pumping had become commonplace. Due to the new surface water based pressure system, there has been a considerable reduction in the pumping of salinity stricken groundwater, and the trees in Division 9 are already benefiting. With less groundwater pumping, air quality throughout Division 9 has improved due to the reduced use of diesel powered well pumps. With a pressure system available, farmers can reduce flood irrigation and utilize drip, micro, and solid state sprinklers to irrigate their land which improves crop yield, conserves water by up to 30%, and reduces erosion and deposition of fertilizer into local surface and groundwater. Finally, the system complies with new State regulations on volume based billing.

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Dale Kuil, SSJID
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Dave Kamper, SSJID
Forrest Killingsworth, SSJID
The farmers of Division 9
Natural Resources Conservation Service
Bureau of Reclamation

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EFFECT OF WATERCOURSES LINING ON WATER DISTRIBUTION EQUITY: A CASE STUDY FROM SINDH, PAKISTAN

Saleem Raza Samo[1]
Bakhshal Lashari[2]

ABSTRACT

Pakistan's irrigation system is supply based which runs round the clock. Distribution of irrigation water among the water users is being made through rotation (Warabandi) system. Since warabandi system was considered viable way of equitable water distribution. But, due to unlined watercourses and poor maintenance (Tertiary), the seepage losses and slow down of flow velocity have caused the inequitable distribution of water among water users. This paper is focusing on head-tail inequity, water losses, cropping intensity and evaluating the warabandi system at two sample watercourses (lined and unlined). Using field data, the Theil Inequity Coefficient (TIC) for lined and unlined watercourses was calculated. It was found that the TIC for lined watercourse was 0.22 and for unlined watercourse was 0.496, which show the efficacy of water distribution equity in lined watercourse 78 percent (though the lining is about 50% of total length) and 50 percent in unlined watercourse. Similarly, TIC between head-tail section were also calculated which shows overall variation more than double in unlined watercourse against lined watercourse. This water saving, improved flow pattern and increased water distribution equity in lined watercourse, the cropping intensity increased from 79 percent to 87 percent. Study has concluded that the lining of watercourses not only improving the water distribution equity and so as cropping intensity but also enhancing the confidence level among the farmers and reducing the water conflict among the water users. Therefore, it is suggested that the not only half of the watercourse be lined but full length of watercourses be lined. Further, watercourse associations which are major pillar of boasting agriculture productivity are found weak which need to be strengthened.

INTRODUCTION

Warabandi means fixing of turns for irrigation water for each farmer on a watercourse. There are two types of warabandi namely "Kacha" and "Pucca". The Kacha warabandi is arranged by the farmers themselves. Its varies from 10 to 15 days depending upon the number of farms on a given watercourse. In each chak or village, a watch keeper used to announce time for the benefit of each farmer through drum beating. With the passage of time, most farmers now own watches and can keep track of their turns and time. This system of water rotation has many problems. Big farmers exploit the small farmers and do not adhere to the agreed upon arrangement of water supplies. To overcome this problem, on the request of any farmer who is not satisfied with the system, the canal

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department regulates the supply of water and fixes the turn of each farmer in a given crop year. This is called Pucca warabandi. The farmers who receive water from the watercourse, in the area of what is called the "Chak", are on a seven-day rotation schedule. Each farmer is assigned what is called a proprietary right to a period of time e.g. from 10.00 a.m. to 12.00 noon every week for which he is entitled to all of the flow in the watercourse (*Seckler, D., 1988*). Evaluated that the warabandi system of irrigation prevalent in North West India and Pakistan. It argues that warabandi needs to be understood as a composite socio-technical system comprising a physical infrastructure and a corresponding institutional arrangement for rationing and sharing water. This has implications for efforts at replication the system in other parts of the region. An understanding of these features is also essential in assessing the prospects and potential for irrigation management reform in the region. Nara in concludes by identifying some challenges and opportunities for management reform in the warabandi system (*Vishal. N., 2008*). Reported that the feasibility of closing distributaries canals at night was investigated in a recently modernized surface irrigation system in Pakistan, the Upper Swat-Pehur High Level Canal system. Increased water supply, greater delivery capacity and the introduction of downstream control potentially allow more flexible service.

In the command area of Maira branch of this system, farmers are anyway abandoning night-time irrigation, as they can meet their needs from improved supply during the day. They practice night irrigation only during the times of peak crop water demand (*Zardari, N. H., et, al., 2009*). The rotational delivery system, known as warabandi in the sub-continent and Dauran in Arabian countries, has even broken down in the day in some parts of the command area. This is believed to be typical of systems with more than adequate water supply. A simulation study was undertaken using the Canalman software developed by Biological & Irrigation Department, Utah State University, Utah Logan, USA. Primary data collected in one distributary canal and the two minors connected to it was utilized for simulations. The feasibility of night-time closure depends on the speed of filling and emptying the canal each day, and the time required to meet full irrigation demand during the day. The results show that where canal lengths are less than 5 km, in this system, there is good potential to make savings, which can be realized at system level through reduced demand on supplemental supplies from Tarbela Dam (*Ghumman, A. R., 2009*).

STUDY AREA FROM SINDH, PAKISTAN

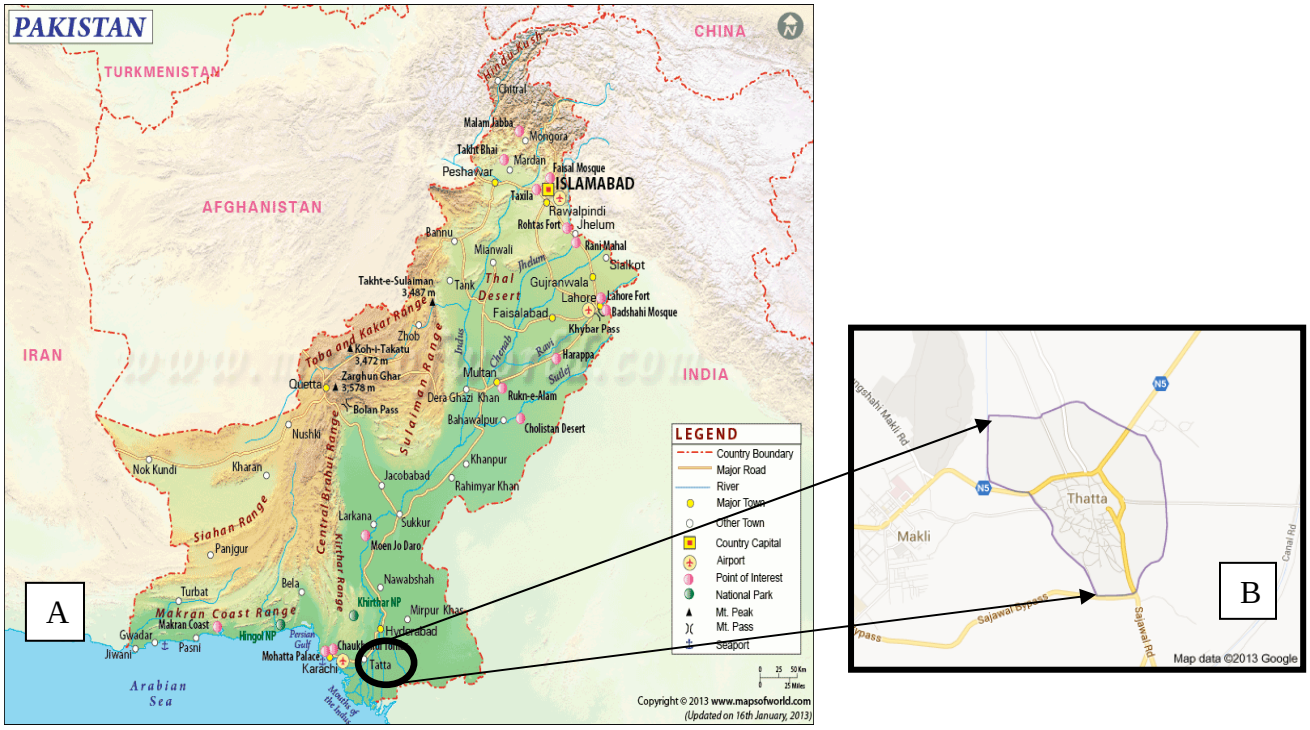

Figure: 1. Study Area

Methodology

Performance of Warabandi System

To evaluate the performance of warabandi system of above two watercourses of Laiqpur ex-Ali Bahar Minor, data regarding cultivable command area (CCA), net wetted area (NWA) and frequency of irrigation watering in Kharif 2010 season has been collected. Ratio of net irrigated area and total irrigated area to CCA were calculated.

The performance of the warabandi system has been evaluated by using index given by Theil (*Qureshi et al. 1994*):

$$\text{TIC} = \frac{\sqrt{\sum (TWA - TWA^*)^2}}{\sqrt{\sum (TWA^*)^2}}$$

Where

TIC	= Theil's Inequality Coefficient
TWA	= Actual Total Wetted Area
TWA*	= Predicted Total Wetted Area

Equity in Water Distribution

To evaluate the equity of water distribution through warabandi system, Theil's Inequality Coefficient (TIC) has been calculated separately for head, middle and tail sections of the selected watercourses. In this connection, the following data with respect to these three sections will also be collected separately.

- Actual wetted area of the section,
- Predicted wetted area of the section and
- Frequency of irrigation

In addition to above, the equity in the water distribution among water users has also been determined by computing of water losses and conveyance efficiency.

RESULTS AND DISCUSSION

To evaluate the performance of warabandi system of the selected two watercourses of Laiqpur Ex Ali Bahar Minor, the data was collected regarding cultivable command area (CCA), net irrigated area (NIA) net wetted area (NWA) and frequency of irrigation for various crops, in Kharif 2010. Actual total wetted area (TWA), predicted total wetted area (TWA*) were calculated. Using these values, Theil's Inequality Coefficient (TIC) was calculated. The TIC calculated for lined watercourse is 0.22 and for unlined watercourse is 0.496. Hence, the effectiveness of warabandi comes for 8CL lined watercourse and 11R earthen watercourse is 78% and 50.4% respectively. This shows that the effectiveness of warabandi is increased due to lining of the watercourse. This concludes that the performance of warabandi is amplified due to lining the watercourse of the same minor during the same period of Kharif 2010**.** In order to confirm our TIC values and the effectiveness of the lined and earthen watercourses, it is necessary to compare this coefficient with the already computed TIC values available in open literature. In this connection, the computed TIC values for the selected watercourses are compared with TIC values for the watercourses of Faisalabad, Pakistan and India (*Walker, H. H., 1980*).

Table. 1. The comparison of TIC values and effectiveness of warabandi in % percentage

Parameter	**Study by (*Qureshi, S. K., et al 1994*)**			**Present study (2011)**	
	Faisalabad Pakistan		**India**	**Thatta, Pakistan (Kharif 2010)**	
	Kharif	**Rabi**	**Crop year**	**Lined w/c**	**Unlined w/c**
Theil inequality coefficient (TIC)	0.56	0.52	0.20	0.22	0.496
Effectiveness of warabandi (%)	44	48	80	78	50.4
When TIC = 0, Performance is Perfect (100%) and When TIC = 1.0, no water being delivered and performance is zero.					

The above Table shows that the performance of lined watercourse of Thatta and that of India is excellent that is about 80%. However, the performance or effectiveness of warabandi of selected earthen watercourse in Kharif season and those of Faisalabad for Kharif and Rabi seasons is almost same (i.e. ranging between 44% and 50%). This shows that the warabandi system of these watercourses is not performing well. This suggests that for the earthen watercourses, there is need of improvement in the warabandi system (*Makin, I. W., 1987*)

Equity in Water Distribution

To evaluate the equity in water distribution among the water users, Theil's Inequality Coefficient (TIC) has been computed for head, middle and tail portions of the selected watercourses. The TIC, effectiveness and other parameters for head, middle and tail portions of both lined and earthen watercourses are tabulated in Table. 2. Theil index (TIC) comes 0.147, 0.165 and 0.319 for head, middle and tail portions of the one lined and two unlined portions of the lined watercourse respectively. However, the TIC for head, middle and tail portions of earthen watercourse is 0.21, 0.5 and 0.6 respectively.

Table. 2. Comparison of Cropping Intensity, TIC and Effectiveness of warabandi in Head-Middle-Tail of the watercourses

Parameters	**8CL Lined Watercourse**			**11R Earthen Watercourse**		
	Head	**Middle**	**Tail**	**Head**	**Middle**	**Tail**
CCA (acres)	31.07	31.76	28.35	91.35	96.25	97.34
Irrigated Area (acres)	30.15	28.45	20.59	80.535	61.16	50.64
Cropping Intensity (%)	97.04	89.55	72.63	88.17	63.57	52.02
Theil's Inequality Coefficient (TIC)	0.147	0.165	0.319	0.211	0.499	0.594
Variation in TIC	**0.172**			**0.383**		
Effectiveness (%)	85.3	83.5	68.1	78.9	50.1	40.6

This shows that the effectiveness of warabandi in lined portion of the 8CL watercourse is 85% that is performing at excellent level. Not only this but also it has increased the performance of warabandi at its tail portion i.e. 68% which is also more than that of the middle portion of earthen watercourse (i.e. 50%). However, the effectiveness of tail

portion of the earthen watercourse is less than the average that is 40%. This indicates that the lining of watercourse is increasing the performance of warabandi of the whole watercourse even it is lined up to 33%. Variation of TIC in lined watercourse is 0.17, where as this variation is increased to more than double that is 0.38; which clearly show that equity in water distribution in lined watercourse is rational, the variation of only 17%. However, the water is not equally distributed in earthen watercourse; variation of water distribution is 38%. More area irrigated at head portion and less area irrigated at tail portion of the watercourse. Further added that more area irrigated at lined watercourse and less area irrigated in unlined watercourse. Data shows that improvement of watercourses through remodeling and lining coupled with land leveling has reduced the losses in irrigation system, increase conveyance efficiency of irrigation system which increases the supply of water for farmers. Due to above reasons, cropping intensity increased and performance of warabandi proved more effective in lined watercourse and less effected in unlined watercourse (*Goldsmith, H., et, al., 1991).*

CONCLUSIONS

Following conclusions have been drawn:

- Evaluation of performance of warabandi system was assessed using Theil Inequality Coefficient (TIC). The TIC calculated for lined watercourse is 0.22 where as for unlined watercourse is 0.496. Hence, the effectiveness of warabandi comes for 8CL lined watercourse and 11R earthen watercourse is 78% and 50.4% respectively.

- From the comparison of present study with (*Qureshi. S. K., et al. 1994*), the performance of lined watercourse of Thatta (present study) and that of India is the excellent that is about 78 and 80% respectively. However, the performance or effectiveness of warabandi of selected earthen watercourse of the study area and those of Faisalabad is almost same (i.e. ranging between 44% and 50%). This shows that the warabandi system of these watercourses is not performing well. This concludes that for the earthen watercourses, there is need of improvement of warabandi system.

- The effectiveness of warabandi in lined portion of the 8CL watercourse is 85% that is performing at excellent level. Not only this but also it has increased the performance of warabandi at its tail portion having 68% which is much more than that of the middle portion of earthen watercourse (i.e. 50%).

- The effectiveness of tail portion of the earthen watercourse is less than the average that is 40%. Hence, it is concluded that lining of watercourse significantly improves the water distribution equity between head and tail portions.

- Variation of TIC within lined watercourse is 0.17, which show the equity within the water users is rational. However, inequity of water distribution is increased to 0.38 in earthen watercourse.

- The conveyance efficiency in lined portion of the watercourse is 98.76% while the conveyance efficiency in the unlined portion of the same watercourse is 90.60%. On the contrary, the conveyance efficiency in earthen watercourse is 70.59%.

- The cropping intensity of 8CL lined watercourse before and after improvement is 79% and 87%, and increase in cropping intensity of about 8%. However, the cropping intensity of earthen watercourse is 67%.

- Lining of watercourse is saving water i.e. about 4.9% per 1000ft length, hence increased the cropping intensity by 8%. However the cropping pattern remains same before and after watercourse improvement.

SUGGESTIONS

The following suggestions have been made.

- Lining of watercourse improve the cropping intensity, convenience efficiency and equity in water distribution. It is therefore suggested that 100% watercourses should be lined in all irrigated areas.

- In case of earthen channels, the extra allowance of water should be allocated and inbuilt in the warabandi system, so that inequity among the farmers due to seepage losses within the watercourse should be aboided.

- Proper marketing system may be developed so that the interest of land owners farmers could be increased in changing the cropping pattern.

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ANALYSIS OF PARTICIPATORY IRRIGATION MANAGEMENT IN SINDH PROVINCE OF PAKISTAN: ISSUES, OPTIONS AND WAY FORWARD

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ABSTRACT

In 1997, Pakistan started institutional reforms in the irrigation because the irrigation management systems had not been performing well and was deteriorating over the time. Reforms were based on experiences and successes of participatory irrigation management in various countries. The aim of reforms was to bring equity, efficiency, reliability and user satisfaction in irrigation water distribution and improve the water charges collection to fund system operation and maintenance and reduce subsidies. The irrigation reforms in Sindh province of Pakistan are covered under Sindh Water Management Ordinance 2002. Sindh Irrigation and Drainage Authority (SIDA), three Area Water Boards (AWBs) and 354 Farmers Organizations (FOs) are established under this law.

The paper analyzes performance of FOs in various categories including organizational development, service delivery, maintenance of channels, internal conflict resolution and water charges collection. Performance index calculated from the field data based on ranking and scoring indicates that on average 20 percent of the FOs are performing satisfactorily, 49 percent of FOs are unstable, and the remaining FOs are falling into the fragile category. Additionally, SIDA and Canal AWBs are facing many other challenges including functioning of regulatory authority, completion of transition process for other canals and service oriented performance of the canal area water boards. Essentially, two parallel organizations are undertaking irrigation management, resulting in duplication, institutional conflicts and inefficient service delivery.

The paper concludes that the unstable FOs can be moved to a stable level by providing capacity building and social mobilization. Furthermore, in the short term, policy and institutional changes are needed, including political support to the reformed institutions (SIDA, AWBs and FOs). It is also recommended that the government develop user-directed, autonomous, commercially-oriented public utilities (PUs) that will ensure operational transparency and cost recovery for the long term sustainability of the system.

INTRODUCTION

Agriculture plays an important role in the economy and livelihood of rural communities in Pakistan. It contributes over 20 percent to GDP, accounts for 60 percent of its exports, and provides employment to over 46 percent of the labor force (Government of Pakistan,

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2013). The key challenges to the agrarian economy of the country are providing sufficient food to a growing population and sustaining and improving its economic growth.

Agriculture in Pakistan largely depends upon the Indus Basin irrigation network which is one of the largest contiguous irrigation systems in the world. The network includes 3 storage reservoirs, 19 diversion barrages, 12 linking canals, 43 main canals and more than 100,000 watercourses. The annual average flow of the Indus River system is estimated to be about 140 Million acre -feet (World Bank, 2007). Besides, water is required for industrial and municipal uses also. With increasing population and climate change and weather variability, there is tremendous pressure on the land and water resources to meet the food and fiber requirements of the growing population of over 179 million. The country over time is moving towards water stress and scarcity which becomes even more complex with increased water management inefficiencies at various levels of water regulation and distribution.

The major issues for irrigation include low efficiency of surface water delivery and use, water distribution inequities, waste of water at the farm level, high delta (requiring more water) cropping pattern, poor operation and maintenance, low cost recovery, constrained investment and water logging and salinity. Water pricing may also have a relationship with water use inefficiencies and water losses.

The institutional weaknesses are one of the causes of the above mentioned inefficiencies. A study by International Water Management Institute (IWMI) in 1997 found that the state-managed surface irrigation in Pakistan had not been performing well and was deteriorating day by day due to financial, managerial and socio-political factors. Consequently, based on experiences and successes of participatory irrigation management in various developed and developing countries, Pakistan embarked upon institutional reforms in the irrigation sector in 1997. The provincial assemblies passed laws to implement a new approach of decentralization and management transfer of irrigation and drainage from the public sector to multitier institutions with farmers' participation. The objective of these reforms was to bring equity, efficiency, reliability and user satisfaction in water distribution and improve water charges collection to reduce subsidies for operation and maintenance of the system.

The reform process in Pakistan is facing lot of challenges in achieving the objectives of the reforms and winning support and will from government and policy makers. The reforms are also facing resistance from the provincial public sector irrigation departments, which are to be transformed. Farmers Organizations (FOs), among the other reform institutions, are the key to success and sustainability of the reforms.

With this back drop, the paper analyzes performance of FOs in various categories including organizational development, service delivery, maintenance of channels, conflict resolution and water charges collection. The paper also analyzes the status of reforms and the legal and institutional aspects of these reform institutions.

REVIEW OF LITERATURE

Though literature on water management efficiency in Pakistan is sparse, various aspects of water management and use efficiency have been studied by different researchers and authors. The Planning Commission, Government of Pakistan, has attempted to study supply side aspects of water use efficiency by assessing the scale of current water charges for canal irrigation and its impacts on cost recovery, financial sustainability and water use efficiency in Pakistan (Government of Pakistan 2012). Policy aspects of efficiency of water use and management have also been studied by Raza *et al.* (2012), but the aspects of water management capacity and performance of institutions have not yet been studied in detail. Latif and Pomee (2003) in their study of some pilot projects have concluded that Farmer Organizations (FOs) in Pakistan increased the cost recovery (water charges collection) by 4 to 23 percent. However, the overall progress in the country remained slow. Bandaragoda (2006) observed that the reason for slow (or stalled reforms) could be that the reforms initiated in Pakistan have not been generated by local demand but were dictated by the donor agencies. Hassan (2010) pointed out that similar irrigation management reforms in Turkey, which also were dictated by the World Bank, were successful mainly because the "demand for change emerged internally from the irrigation agency due to fiscal crises".

Briscoe (1996) in his study on Egypt considered the aspects of willingness to pay and suggested that the value of water is the maximum amount a water user would be willing to pay for its use. ICID (2004) defined the value of irrigation water as equal to opportunity cost of water. However, the water markets are imperfect, so determining the real value of water for different uses is quite difficult. Besides, value of water varies among users, sectors, locations, seasons, times, quality and reliability (Briscoe, 1996). Theoretical debates and empirical studies starting from tragedy of commons, managing commons and governing commons and extending to game theory and collective actions have highlighted various aspects of the subject. The community level cooperation in irrigation management in South Asia has a mixed history witn many successes and failures, both mentioned by Bardhan based on various other studies. Similarly, Rasmussen and Meinzen-Dick (1995), Bardhan (1993; 2000), Heltberg (2001; 2002),Gebremedhin *et al.* (2003), and McCarthy *et al*. (2004) have also tried to define the types of local collective action responsible for successful management of natural resources challenges. Ul-Haq and Shahid (1997) summarized the strategies and models proposed by various agencies for PIM (Participatory Irrigation Management) and evaluated their strengths and weaknesses.

Sagardoy (1997) reviewed the concepts and strategies of farmers' participation in irrigation management and concluded that the results were quite satisfactory in some cases and disappointing in others. Smith and Sohani (1997) found that participatory management of irrigation was financially feasible whereas the equity of water distribution and criteria for sharing of drainage costs were the critical issues requiring urgent attention. Tekynel and Aksu (1997) in their study on participatory irrigation management in Turkey found that the reforms achieved remarkable success beyond all expectations. Ul-Haq and Shahid (1997) undertook an assessment of irrigation reforms

focusing on strengths and weaknesses of the systems and discussed the issues and outlined options and prospects for better service delivery in irrigation management. The study recommended a holistic approach and integrated strategies based on sociopolitical setting for better service delivery and sustainable irrigation management.
Jahangir *et al*., (1999) in his study focused on the institutional and financial aspects of participatory irrigation management and found collection of revenue to be higher than operation and maintenance expenditure from 1971/72 to 1995/96 except for the year 1986/87 which is the pre reforms period. . The author found that empowered Farmers Organizations can bring improvement. Thus, FOs may undertake activities of fee assessment and collection as well as managing operation and maintenance expenditures. Waqar (1999) conducted a case study on financial viability of participatory irrigation management and found that the government of Pakistan was spending heavily on the operation and maintenance of irrigation systems, yet shortages of funds were a major reason for deferred maintenance, which threatened the operational efficiency of the system. The shortfall in O&M budget was estimated to be more than 24 percent in 1993.

Wijayaratna (2002) summarized resource papers and selected country papers from the Asian Productivity Organization seminar on organizational change for participatory irrigation management held in Philippines. The country papers indicated the efforts of the participating countries for strengthening and deepening the participatory approaches to irrigation management had realized some successes, yet the results varied across countries because of country-specific social and political settings. The other causes of varied results included the level of irrigation development, characteristics of the organizational structures established for irrigation management and the degree of external assistance. Parthasarathy (2004) studied results of institutional reforms and participatory approach to irrigation management in India. The study noted that institutional reforms in irrigation with farmers' participation were initiated by many Indian states aiming at decentralization of irrigation management and revitalization of agricultural development and rural uplift at the grass roots level. The paper highlighted tardy progress and performance of irrigation management in almost all the states. Rational and equitable water supplies, water rights and just regulation, policy and political will or participation were identified as necessary conditions for success of the participatory approach in irrigation management.

METHODOLOGY

The following methodology was used to undertake the research and collect primary as well as secondary data. The methodology includes a field survey using questionnaires. The data were collected from a sample of 50 distributaries across three canals (Nara Canal, Ghotki Feeder Canal and Left Bank Canals). A total of 50 FO chairmen and 150 farmers were interviewed, selected evenly from head, middle and tail canal reaches. The survey was further enhanced by conducting focused group discussions at 9 locations across the selected three canals to collect qualitative information.

Field Surveys

A Field Survey was conducted during 2012 to collect data on FO performance and the collected data were also verified with the other sources (AWBs/SIDA) for consistency and authenticity. Two different questionnaires were prepared and tested in the field for data collection.

Questionnaire for FOs. This questionnaire included questions related to internal structures and activities of FOs, water charges (*abiana*) assessment and collection as well as general conditions of the operational area of FO in terms of land use, cropping pattern, I&D infrastructure, etc. Mostly, chairpersons of FOs were interviewed but other members of management were also present in some cases.

Questionnaire for Common Farmers. This questionnaire was designed to record the perception of farmers about the equitable distribution of water, services of FOs, and satisfaction with the water management and water distribution practices. From each selected distributary or minor canal, five (5) farmers were selected as a sample for face to face personal interviews.

The purpose of these questionnaires was to cross-check the performance of FOs and to assess the satisfaction level of water users. Group discussions were also held in order to get information about the functioning of FOs and benefits of institutional reforms in the irrigation sector.

ANALYTICAL FRAMEWORK

Performance Monitoring Indicators

The FO performance was developed using the following weights and scores.

A. Organizational Development (20)
1. General Body and board of management committee meetings (3)
2. Participation of members in the meeting (2)
3. Formation and functioning of standing committees like advisory and water allocation committee (2)
4. Staff of FOs (2)
5. Status of progress reports submission by FOs (1)
6. FOs records management/maintenance status (2)
7. Accounting and auditing (2)
8. Representation of small farmers in board of management (1)
9. Representation of tail end users in board of management (1)
10. Participation level of General body members in the FO formation/ Re-election process (2)
11. Participation level of FOs in training activities (1)
12. Existence of FOs offices (1)

B. Irrigation Service Delivery (30)
1. Dissemination of information about the rotational schedule to water users (2)
2. Perception of FOs regarding equitable water distribution among the watercourses (field channels) (3)
3. Perception of farmers regarding equitable water distribution among the watercourses (5)
4. Perception of farmers regarding equitable water distribution within the watercourses (5)
5. Satisfaction level of farmers regarding water distribution (5)
6. Monitoring of water supply along the channel through patrolling (4)
7. Maintaining good gauge register records (4)

C. Management of Physical Condition of Channels (20)
1. FOs participation in walkthrough surveys, contract agreements, consultation meetings and monitoring project implementation in participatory way (5)
2. Maintaining condition of channels & structures (3)
3. Budget allocation for repair and maintenance works (4)
4. Support for maintenance activities conducted on a self-help basis by farmers (4)
5. FOs response in case of emergency due to breach/cuts/erosion etc (4)

D. Dispute Resolution (10)
1. Farmers approach to FOs for internal dispute resolution (2)
2. Frequency of conflicts resolved (4)
3. Level of acceptance of decisions by farmers (4)

E. Water Charges (Abiana) Assessment and Collection: (20)
1. Assessment of water charges as per schedule (7)
2. Collection of water charges (7)
3. Application of penalties for late or non-payment of water charges (2)
4. FOs deposited 60% share of AWB (4)

The above scores have been used to calculate FO maturity indexes by allocating 20 percent weight to organizational development, 30 percent to irrigation service delivery, 20 percent to management of the canal and structures, 10 percent to dispute resolution and 20 percent to collection of water charges.

Performance Measurement Grids

An evaluation criterion was developed to judge the performance of FOs against the prescribed indicators in view of their importance as presented in Table 1. FOs securing scores over 70 percent were considered to be satisfactory, between 55 percent to 70 percent as unstable and less than 55 percent as fragile.

Table 1. Evaluation Grid

Category	Range in percent
Satisfactory	≥70
Unstable	<70 to ≥55
Fragile	< 55

RESULTS AND DISCUSSIONS

Legal and Institutional Framework

Sindh Irrigation and Drainage Authority Act of 1997 (SIDA ACT, 1997) was enacted to revitalize the system by replacing the traditional system with a participatory irrigation management system. The SIDA is responsible for taking decisions on key issues such as: (a) annual business plan and the annual budget, (b) the annual report and the annual accounts, (c) guidelines and regulations concerning finances and accounting, human resources management, internal business procedures etc., (d) major investments and (e) entering into new obligations, such as new tasks.

The new system was further defined/refined in the Sindh Water Management Ordinance (SWMO), 2002. According to the 2002 Ordinance, the irrigation and drainage management system rests on three pillars. Firstly, Sindh Irrigation and Drainage Authority (SIDA) has the responsibility of managing and operating the main structures such as barrages and overall monitoring and supporting the reform process. Secondly, Area Water Boards (AWBs) are responsible for the operation and maintenance of main canals, and supporting and coordinating the FOs established or to be established under their canal command areas. Thirdly, Farmer Organizations (FOs) are water users' organizations that are formed by themselves at the bottom, and are responsible for management, operation and maintenance of distributary and minor canals. In the 2002 Ordinance, a fourth organization is defined at water course level as Water Course Associations, but these are small irrigation groups formed at water course level.

Institutional Status under SWMO, 2002

Sindh Irrigation & Drainage Authority (SIDA), a corporate body was established in 1997 and delineated by Government of Sindh; 3 Canal Area Water Boards were established out of 14 canals; SIDA itself is performing as the Regulatory authority, as the target of Regulatory Authority has not been achieved; SIDA has to establish Water Allocation Committees; till now 3 Water Allocation Committees have been formed on three canals and same on 11 Canals are yet to be formed. For the purpose of smooth interaction with the local communities, Community Advisory Committees (CACs) were also to be established at SIDA & AWBs level, but to date no CACs have been formed. A total of 354 FOs have been formed on 354 distributaries out of 1400 distributaries.The status or progress of the institutions to be established under the law is presented in the Table 2.

Table 2. Status of Institution under SWMO 2002

Institution	Target	Achievement
Sindh Irrigation and Drainage Authority	Establishment of Sindh Irrigation and Drainage Authority	Done in 1997
Canal Area Water Boards	Establishment of 14 Canal Area Water Boards	Established 3 Canal Area Water Boards
Regulatory Authority (RA)	Establishment of Regulatory Authority	Not established yet. SIDA itself is performing as RA
Water Allocation Committees (WAC)	Formation of Water Allocation Committees at SIDA and AWBs levels	Formed WACs at 3 canals. WACs at SIDA and 11 Canals are yet to be formed
Community Advisory Committees	Formation of Community Advisory Committees at SIDA and AWBs levels	Not formed yet
Farmers Organizations	Formation of FOs at 1400 distributaries and Minors	Formed 354 FOs out of 1400

Gaps between 2002 Ordinance and Implementation

Participatory irrigation management was the objective of 2002 Ordinance, but there are several gaps between the objective of the Ordinance and the way it has been implemented to date. First of all, elected members of the SIDA Board have not been elected democratically by the water users'; instead, they are selected by the relevant Government bodies. Consequently, the so-called elected members of the Board do not feel themselves responsible and accountable to the water users. Instead, they are inclined to the political leaders who selected them. Additionally, a majority of the members of the Board of Management are selected by relevant governmental bodies, not by decision of the SIDA Board. They are employed on contract basis and paid by the Project, meaning that SIDA is dependent on the external fund which, in turn, may bring some risks in terms of institutionalization and sustainability of SIDA. It may also create some risks for ownership of SIDA. A third issue is that no barrage has been transferred to SIDA so far and, therefore, no Water Allocation Committee at barrage level has been established closing the administrative structure of system to farmers/stakeholders. Finally, according to the 2002 Ordinance, SIDA may establish a community advisory committee (CAC) for the purpose of a smooth interaction with the local community; however, SIDA has not established any CAC so far. Even though it is not compulsory, the establishment of a CAC would support the spirit of a participatory system considerably.

Formation of Farmers Organizations

Farmers organizations are key reform institutions in the process of participatory irrigation management. Their functions include: (a) operate and maintain the parts of the irrigation system conferred on it to ensure equitable/judicious distribution of water including small and tail end farmers, to supply non-agricultural users and to guarantee minimum drinking water; (b) operate and maintain the parts of the drainage and sewerage system conferred

on it; (c) carry out flood protection and maintain infrastructure within its command area; (d) advise Local Councils on any matter strategic or tactical, related to its role and functions and (e) fulfill any other function conferred on it by this Ordinance or by any subsequent enactment. Fulfilling the functions set out in the law, the FO shall promote and facilitate as much as possible its member Water Course Associations.

The status of FOs formation is presented in Table 3 which depicts that almost all FOs have been formed on distributaries falling in the command areas of the canals where Area Water Boards (AWBs) have been established. These FOs are taking on the management function through transfer from AWBs, and are continuing their democratic process of elections. The process of collection of water charges from farmers is not yet satisfactory, though the rate of deterioration has been arrested.

Table 3. Summary of FO Related Activities

Sub-Components	**Target**	**Achievement**
Formation of FO	383	354
Signing of Irrigation and Drainage Management Transfer Agreement (IDMTA)	383	318
Re-elections	262	121
Abiana (Water Charges) collection (Million Rs.)	1034.69	488.41
Percentage of Target	100	47.2

Performance of Farmers Organizations

Considering various categories including organizational development, service delivery, maintenance of channels, conflict resolution and water charges collection, the performance of FOs have been gauged. Maturity index values calculated from the field data based on ranking and scoring indicate that 20 percent of the FOs on average are performing satisfactorily, 49 percent are unstable, and remaining FOs (31%) are falling in fragile category (Table 4). The results further show that the Nara Canal has the highest percentage of FOs; this canal also has the longest active Area Water Board. This shows that the FOs move towards the category of satisfactory performance with age and experience. The maximum number of fragile FOs is in the canal command of Left Bank Canal Area Water Board, indicating the need for urgent attention.

Table 4. Overall Performance of FOs

Area Water Board	**Unit**	**Satisfactory**	**Unstable**	**Fragile**
Ghotki	Number	2	4	5
	Percent	22.2%	18.18%	35.71%
NARA	Number	5	12	3
	Percent	55.5%	54.54%	21.4%
Left Bank	Number	2	6	6
	Percent	22.2%	27.27%	42.85%
Overall	Number	9	22	14
	Percent	20%	49%	31%

ISSUES

Sindh Irrigation & Drainage Authority (SIDA) has been established but that has not established in letter and spirit as was conceived in the beginning. Secondary management bodies have not been established in a timely manner, which shows poor support for the participatory irrigation management concept. For example, so far, only three Canal Area Water Boards out of fourteen have been established, and Regulatory Authority (legal body that will resolve disputed matters) has not yet been formed. The lack of action on these important matters shows lack of political will and interest to make these reforms successful. Water Allocation Committees are not yet formed on any of the fourteen canals, and Community Advisory Committees (CACs) at the barrage level still need to be formed; they have not yet been established on any single barrage because full command of any barrage has not been given to SIDA. All of these organizational units are needed to support SIDA to function smoothly and achieve the objectives of participatory irrigation management. The bits and pieces of administrative units that have been established within and under SIDA are insufficient to make the reforms successful.

CONCLUSIONS AND SUGGESTIONS

SIDA and Canal AWBs are facing many challenges including functioning of regulatory authority, completion of transition processes for other canals and service oriented performance of the canal area water boards. Because two parallel organizations are undertaking irrigation management (SIDA/CAWBs and IPD), the result is duplication, institutional conflicts and inefficient service delivery.

The SIDA Board is not effective because all members have not been elected, but are instead selected by the government, and thus they are not primarily responsible and accountable to the water users and the SIDA. The SIDA is still dependent on project money and has not generated sufficient funds to make SIDA sustainable.

The management structure of the SIDA is incomplete, as there are several other units which need to be formed in SIDA that are still either missing or in the process of formation. Absence of these bodies/units hinders the smooth functioning of SIDA and creates issues that delay decision making and proper achievement of objectives.

Analysis of survey results has concluded that the FOs which are the third tier in the participation irrigation management reforms are only partially established, and those that are in existence are still not functioning effectively. Results have shown that 20 percent of FOs are performing satisfactorily, 49 percent are unstable, and 31 percent are fragile.

The paper also concludes and suggests that the unstable FOs can be moved to a stable level by providing capacity building and social mobilization. Further, in the short term, initiating policy and institutional changes that provide political support to the reform institutions (SIDA, AWBs and FOs) is needed. It is also recommended that the government develop user-directed, autonomous, commercially-oriented public utilities

(PUs) that will ensure operational transparency and cost recovery for the sustainability of the system in the long term.

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COMPARING THE GLOVER-BALMER METHOD WITH A CALIBRATED GROUNDWATER MODEL TO ESTIMATE AQUIFER-STREAM IMPACTS DUE TO ALTERED FIELD WATER MANAGEMENT

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ABSTRACT

In recent decades, demand for water has risen to levels that necessitate accurate water resource management tools in many arid regions, including Colorado. Recognition of the interconnectedness between groundwater and streams has brought about specific regulations of management practices for Colorado's water resources. Additionally, increasing water demands of municipalities and industries are creating an impetus for water rights transactions and leases, under the constraint of adhering to interstate river compacts. Historically, simple analytical models, such as the Glover-Balmer method, have been used to assess the impact of field-based water management on groundwater-surface water interactions within Colorado. In this study, we investigate the applicability of such a method by comparison with results from a MODFLOW-UZF groundwater flow model that has been calibrated and tested for the Lower Arkansas River Valley (LARV) in southeastern Colorado. Comparisons are performed by stressing the numerical model to simulate addition or extraction of water to specific cultivated fields, and determining accretion or depletion to the river due to the system stress. The Glover-Balmer model is provided inputs based on the aquifer parameter values developed by the MODFLOW-UZF model, and stream accretions/depletions from the analytical model are compared to those simulated by the numerical model. Preliminary results from first stages of the study show that the Glover-Balmer model typically overestimates streamflow accretion from recharge basins as compared to the numerical model, and the overestimation generally increases with distance from the river. Similar overestimation is revealed upon initial examination of irrigation water removal from fields selected for fallowing to allow water leasing to municipalities.

INTRODUCTION

Water shortages caused by drought have increased the desire of urban water facilities in the West to have reliable water supplies. In response, an interest in lease-fallowing programs, in which producers temporarily lease water rights and then fallow certain agricultural fields, has arisen in recent years. To comply with state and interstate water laws, accurate methods are required to manage water systems and water rights. This includes consideration of the interconnectedness between groundwater and surface water. A common practice in estimating depletion from streams due to well pumping is the use

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of the Glover-Balmer method (1954), an analytical solution that multiplies the well pumping rate by a factor based on the complimentary error function:

$$Q_r = Q\left[erfc\sqrt{\frac{S}{4T}\frac{x^2}{t}}\right] \quad (1)$$

Where S is aquifer storativity, T is transmissivity, x is the straight-line distance to the stream, t is time, Q is the pumping rate, and Q_r is river depletion. In order to derive the analytical solution of Eq. (1), several assumptions were imposed, including a semi-infinite, homogeneous, and isotropic aquifer, a perfectly straight stream, an aquifer that is fully penetrated by the stream, perfect connection of the stream to the aquifer, and constant stream stage over time. These assumptions can lead to significant errors in predicting stream depletion due to well pumping. When compared with numerical groundwater flow models, the Glover-Balmer solution has been found to over-estimate the impact of system stresses on the stream (Spalding and Khaleel 1991, Sophocleous et al. 1995). The numerical models used for the comparison, however, have been applied to hypothetical aquifers and streams, with simplistic representation of boundary conditions, aquifer geometry, and spatial patterns of aquifer properties (e.g., hydraulic conductivity, specific yield).

Though initially developed to assess the impacts of well pumping on depletion of groundwater return flows to streams, the Glover-Balmer method also has been used to estimate depletion due to a reduction in recharge originating from the ground surface, accompanied by a lowering of the groundwater table. Moreover, accretion to streams arising from injection wells or from an increase in recharge, with associated mounding of the groundwater table, can be evaluated by interpreting the stress Q in Eq. (1) as a negative quantity.

The objective of this study is to evaluate the Glover-Balmer solution by comparing it to a calibrated numerical groundwater flow model, developed for the Lower Arkansas River Valley (LARV) in southeastern Colorado. The study is motivated by the need to develop a water accounting tool for assessing the likely impacts of lease-fallowing scenarios on the quantity and pattern of Arkansas River flow. In such scenarios, both depletion of groundwater return flow due to removal of irrigation water from fallowed fields and accretion of return flow due to recharge basins used to augment depleted return flows must be evaluated to insure that Colorado water rights are kept whole and that river flows comply with Colorado's compact with Kansas.

The MODFLOW-UZF model application to the LARV (Morway et al., 2013) is based on extensive data collected from 1999 to 2009, and includes spatial- and temporal-varying sources and sinks (e.g., applied irrigation water, groundwater pumping), spatial- and temporal-varying boundary conditions (e.g., river, canal stage), and spatially-variable aquifer parameter values. In the first stages of this study, scenarios are established to simulate with the model groundwater recharge on a number of different fields associated with installation of recharge basins, and resulting predictions of time-varying return flows (accretion) to the river are assessed. Results are compared to return flows predicted by application of the Glover-Balmer method. Similar comparisons are presented here for a

few scenarios representing reduction of groundwater recharge due to removal of irrigation water associated with fallowing of selected fields.

METHODOLOGY

The MODFLOW-UZF application for the LARV (Morway et al 2013) employs the MODFLOW-NWT (Niswonger et al., 2011) version of MODFLOW (Harbaugh, 2005) for simulating three-dimensional flows in unconfined alluvial aquifers based on a finite-difference formulation and the Newton solution method. Flow in the unsaturated zone above the water table is approximated using the UZF1 package for MODFLOW (Niswonger et al., 2006).

Model development used extensive observations of groundwater hydraulic head, groundwater return flow to streams, aquifer stratigraphy, canal seepage, total evapotranspiration (ET), the portion of ET supplied by upflux from the shallow water table, and irrigation flows. Data were collected for over 9 years and are broadly representative of the LARV. Two MODFLOW-UZF models were developed for regions upstream and downstream of John Martin Reservoir. For this study, the model for the Upstream Study Region (USR), comprised of about 50,600 ha (of which 26,400 ha are irrigated), is applied. The model boundary begins just west of Manzanola and continues eastward to Adobe Creek, which is near Las Animas, highlighted in Figure 1.

The finite-difference computational grid is defined by dividing the alluvial aquifer into 250 m × 250 m cells, as shown in Figure 2. The model has 15, 600 active nodes and 2 layers. The top layer has a thickness approximately 5 m, encompassing deeply-rooted crops. The lower layer extends from the bottom of the upper layer to the impervious bedrock. The simulation period for calibration and testing is 1999 – 2009 with 552 weekly time steps. Additional information can be found by referring to Morway et al. (2013).

To conduct a comparison between the MODFLOW-UZF model and the Glover-Balmer model, simulations initially are developed where a particular field receives stress in the form of a water addition due to recharge from the ground surface. This stress is in addition to irrigation and precipitation events that occur within the model; all other calibrated parameters and data remain unchanged. To adequately encompass the groundwater processes, the MODFLOW-UZF simulations are extended to a 32 year period by repeating the 10.5-yr 1999-2009 simulation three times, with the end of each 10.5-year simulation used as the initial conditions for the next 10.5-year simulation. To determine the effects from the stress, MODFLOW-UZF outputs for the stressed scenario are compared with MODFLOW-UZF outputs from the baseline (unstressed) scenario, with results subtracted from the baseline scenario to estimate the impacts of the water addition on the hydrologic system. These outputs include accretion to the river main stem, individual canal seepage, tributary accretion, changes in water table depth, and changes in ET and upflux. Infiltration and surface runoff components are also observed to insure that the added volume infiltrates entirely. In using the Glover-Blamer model, only stream accretion is estimated.

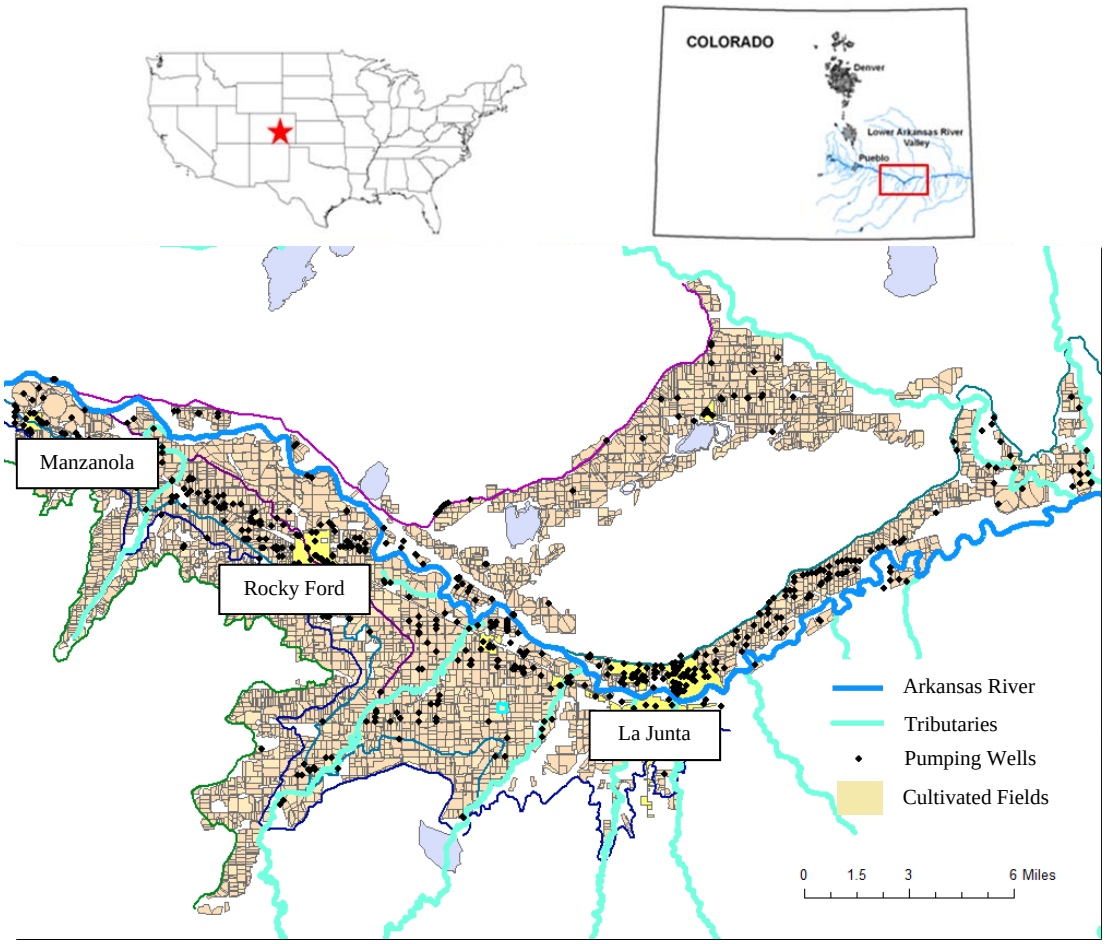

Figure 1. The USR within Colorado's LARV.

Results using the Glover-Balmer model are obtained by applying the volume of recharge water as a negative pumping rate, Q. The Glover-Balmer model utilizes a single value for transmissivity (T) and storativity (S) to estimate stream depletion. It is assumed that the specific elastic storage is relatively small (Theis, 1952), so that storativity can be assumed equal to specific yield (S_y), which is the value used for analyses. Usually, estimations of stream depletion using the Glover-Balmer model are in a data-limited setting where aquifer parameters are not well defined. For this study, it is possible to use aquifer parameters determined by the calibrated MODFLOW-UZF model as inputs for the Glover-Balmer solution. As is common in real-world applications of the Glover-Balmer solution, the flow path to the river is chosen as the minimum-distance straight line from the field to the river. The aquifer parameters of the numerical model cells intersected by the straight line are then selected, as shown in Figure 3, for use in estimating the values of T and S_y to be used in the Glover-Balmer solution.

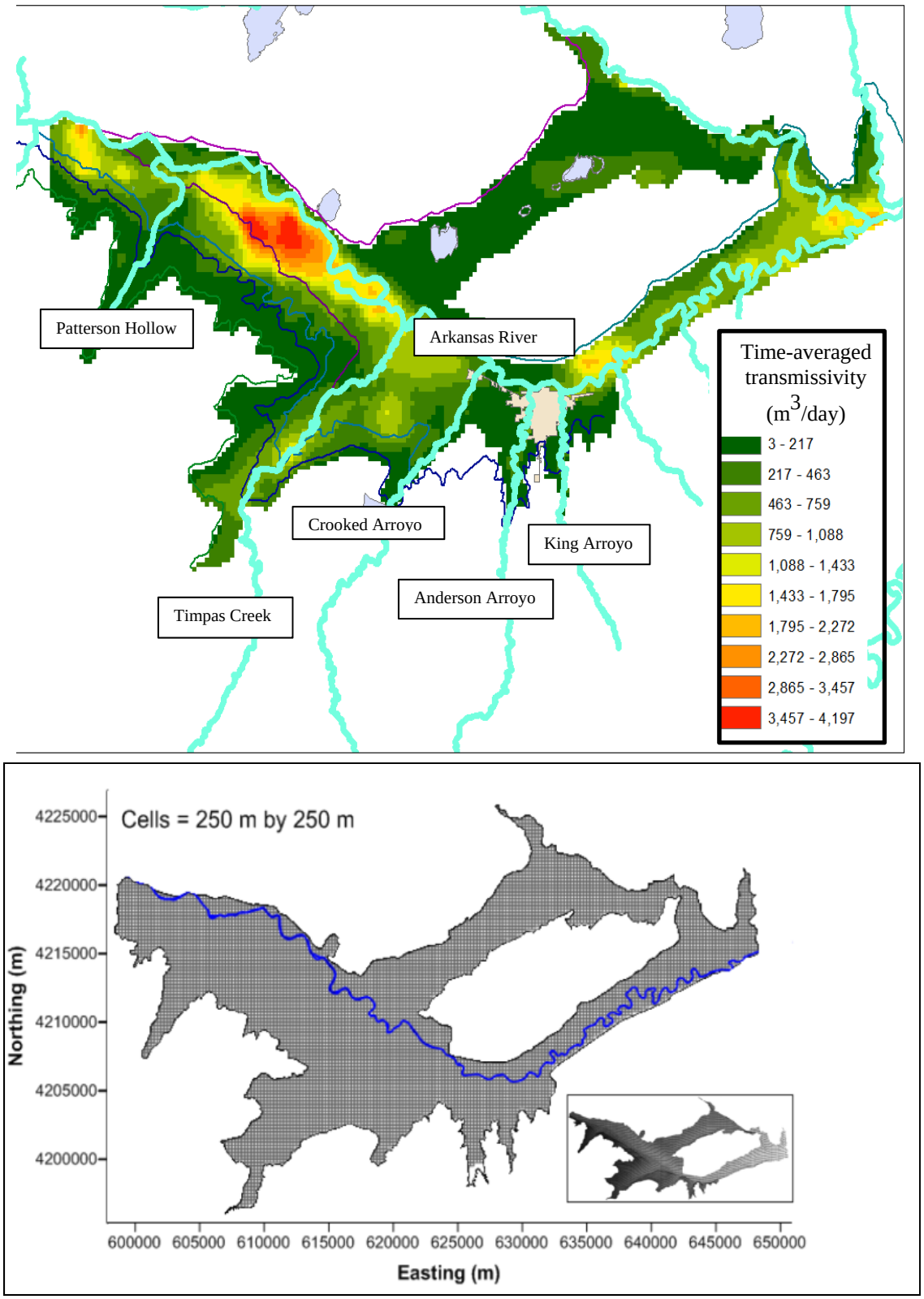

Figure 2. Time-averaged *T* map and model boundary with cell discretization.

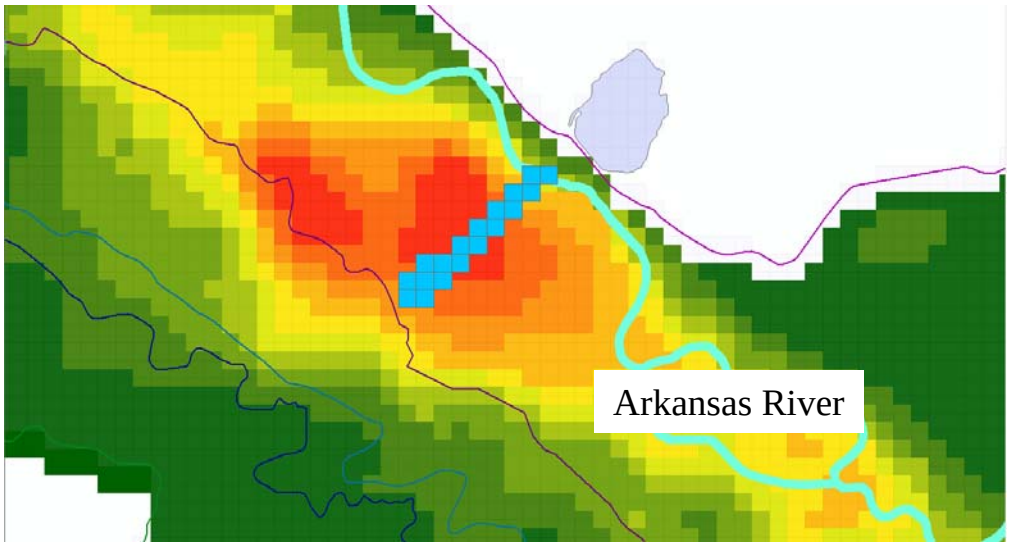

Figure 3. Map of *T* with an example of MODFLOW-UZF cells selected as a minimum-distance straight line flow path (in blue) to obtain *T* and S_y values for use by the Glover-Balmer solution.

Due to fluctuating water table elevation during the simulation, each MODFLOW-UZF model cell has a unique *T* value (computed as the layer-averaged product of hydraulic conductivity and saturated aquifer thickness) for each of the weeks of the simulation. In order to obtain a single value for each cell, an arithmetic average over the simulation period is taken over all the grid cells along the flow path between the field and the river. A single *T* value and S_y value then are obtained from the selected cells by taking a harmonic average along the flow path (Aral and Taylor, 2011). This scheme allows for the use of parameters that represent the natural system more closely than would be expected in many applications of the Glover-Balmer solution.

The rate of water addition, aquifer parameter values, and straight-line distance to the stream are used to estimate river accretion in the Glover-Balmer solution. To account for the long-term effects of the stress event, the principle of superposition is applied over the same weekly time steps as the numerical model. Superposition is possible by incorporating imaginary recharge basins in order to simulate gradual changes in water table elevation and groundwater flow following a recharge event. For each scenario, the water addition occurs in the second year of the simulation at the beginning of the irrigation season. The amount of water added to the field equals 25 acre-feet, divided over four consecutive weeks.

RESULTS AND DISCUSSION

To obtain results that are representative of the complexity and variability of the LARV, an array of fields is selected to individually receive water stress as recharge. Each field that receives recharge is considered as a possible recharge scenario. Results presented here depict general trends and some special cases that merit discussion. A summary of all recharge scenarios is also presented to further depict general trends.

Scenario 1 is selected to highlight cases where MODFLOW-UZF predictions match those of the Glover-Balmer model quite closely. The straight line distance from the field to the Arkansas River is about 400 m. The field's location within the LARV is depicted in Figure 4. Results in Figures 5 and 6 show stream accretion (negative depletion) with respect to time and the percentage of the water recharge volume that has accreted to the Arkansas River and tributaries, respectively. It is seen from Figure 5 that the MODFLOW-UZF model river accretion closely resembles that of the Glover-Balmer model.

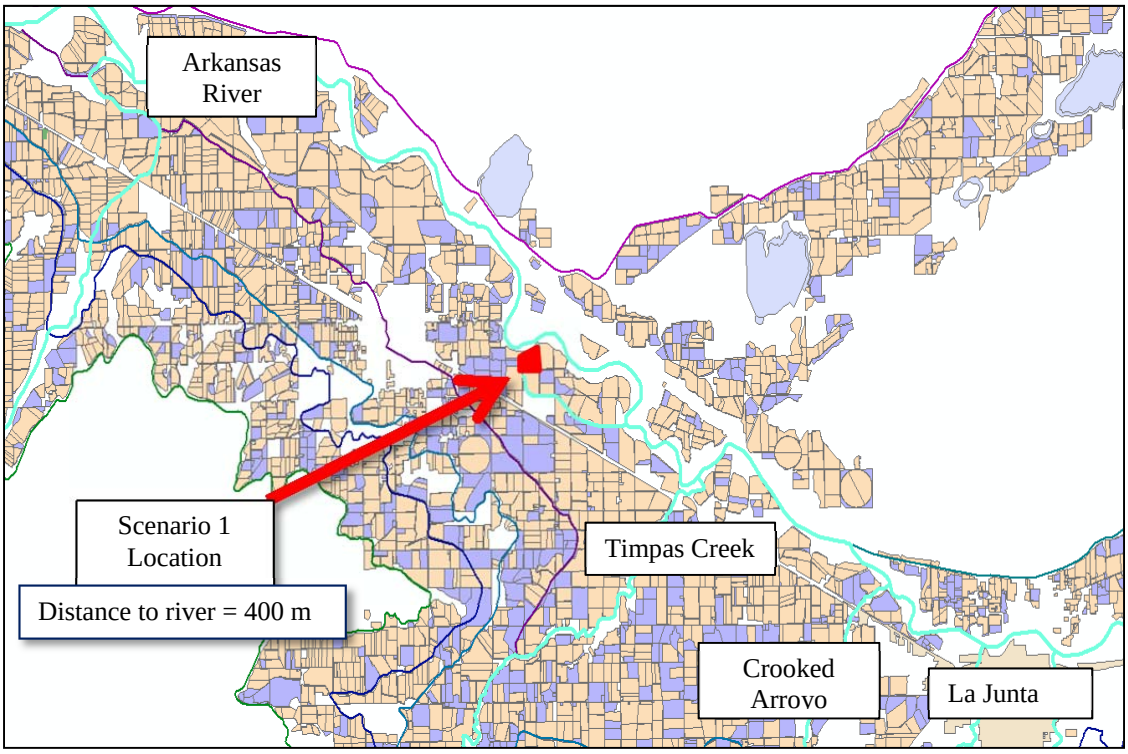

Figure 4. Location of Scenario 1 field within the LARV.

It is worth noting that tributary accretions are included in the MODFLOW-UZF model output for Figures 5 and 6 and all similar subsequent plots. Due to the existence of a shallow water table in many parts of the LARV, changes in water table depth and aquifer saturated thickness are monitored to account for potential changes in ET and upflux from the water table, which have been shown to be potentially significant (Niemann et al., 2011). Figure 7 is an example for Scenario 1 and is relatively representative of all changes in saturated aquifer thickness for water addition scenarios. For the cell receiving the water addition, the water table rises during the month of the addition and returns to the baseline level within a year. As mentioned previously, the UZF1 package accounts for water in the unsaturated zone, and can be used to account for changes on an individual cell basis. A summary of UZF1 outputs is shown in Table 1.

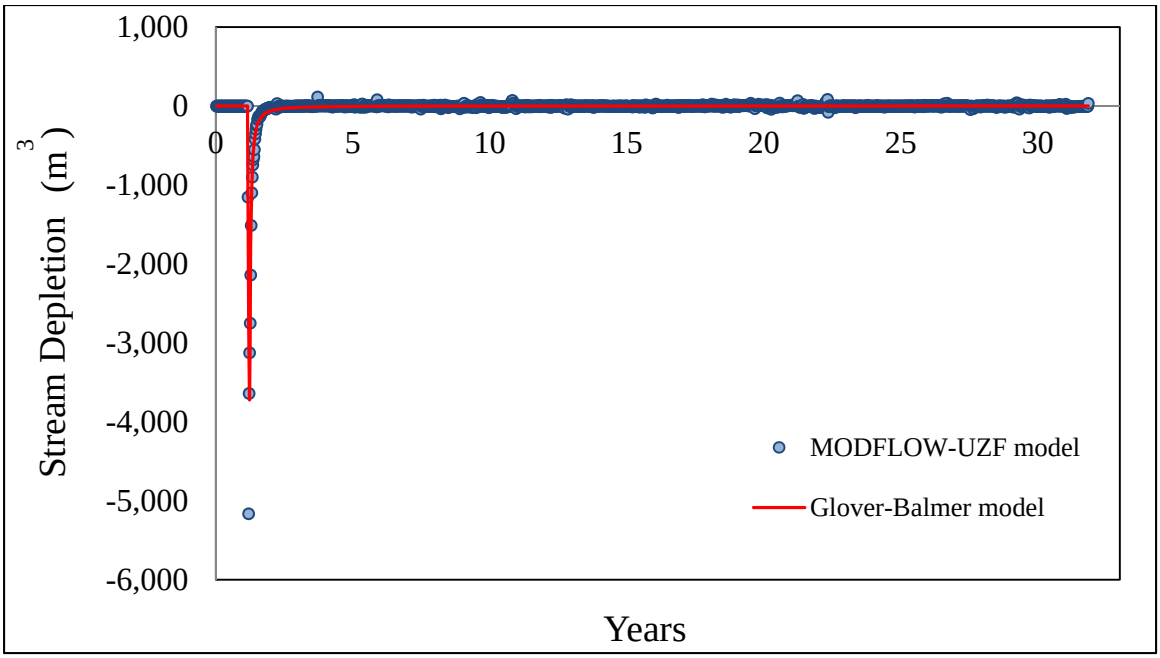

Figure 5. Stream accretion (negative depletion) estimates from MODFLOW-UZF the numerical model and from the Glover-Balmer analytical model for the 32 year simulation period.

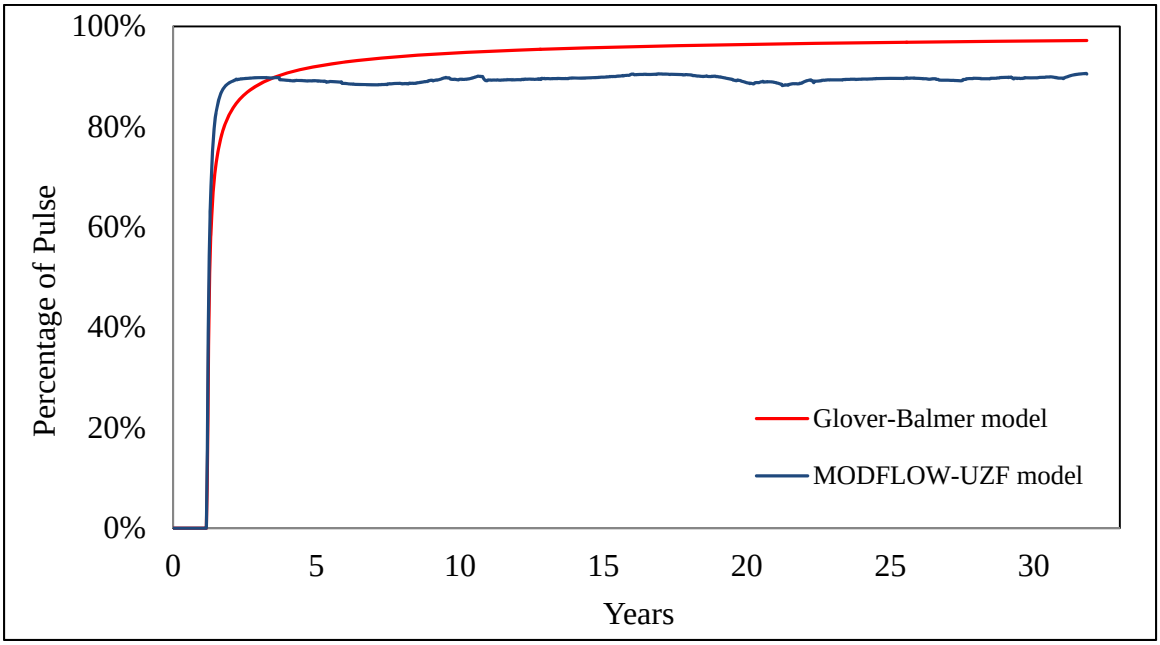

Figure 6. Cumulative recharge volume accreted to the Arkansas River, as predicted by the numerical and analytical models.

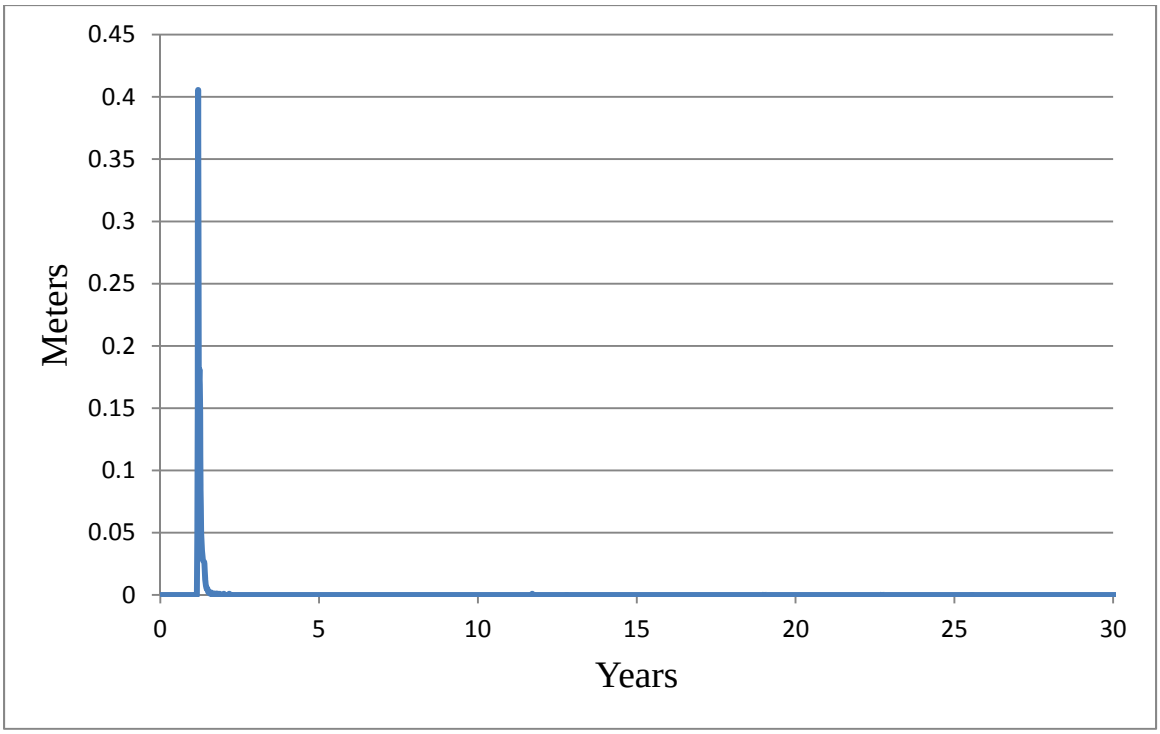

Figure 8. MODFLOW-UZF model output depicting the change in saturated thickness in the cell that receives the water addition.

Table 1. UZF1 summary for the stressed cell in Scenario 1.

Volumes expressed as a percentage of the stress volume	
Infiltrated volume	100%
Volume as aquifer recharge	88%
Total change in ET	+9%

Scenario 2 considers a field between Timpas Creek and Crooked Arroyo, at a straight line distance of about 1050 m. This example is generally representative of fields at similar distances from the river. Figure 8 depicts the location within the LARV. Figure 9 shows the deviation between the estimations by the numerical and analytical models. Table 2 highlights activity in the unsaturated zone and a significant accretion to the Arkansas River tributaries.

Due to hydraulic conductivity patterns, a significant amount of the added recharge water reaches Timpas Creek as groundwater flow, instead of the Arkansas River. This volume is accounted for in the MODFLOW-UZF model, but the Glover-Balmer model assumes there is only one stream and that all accretions end up directly there.

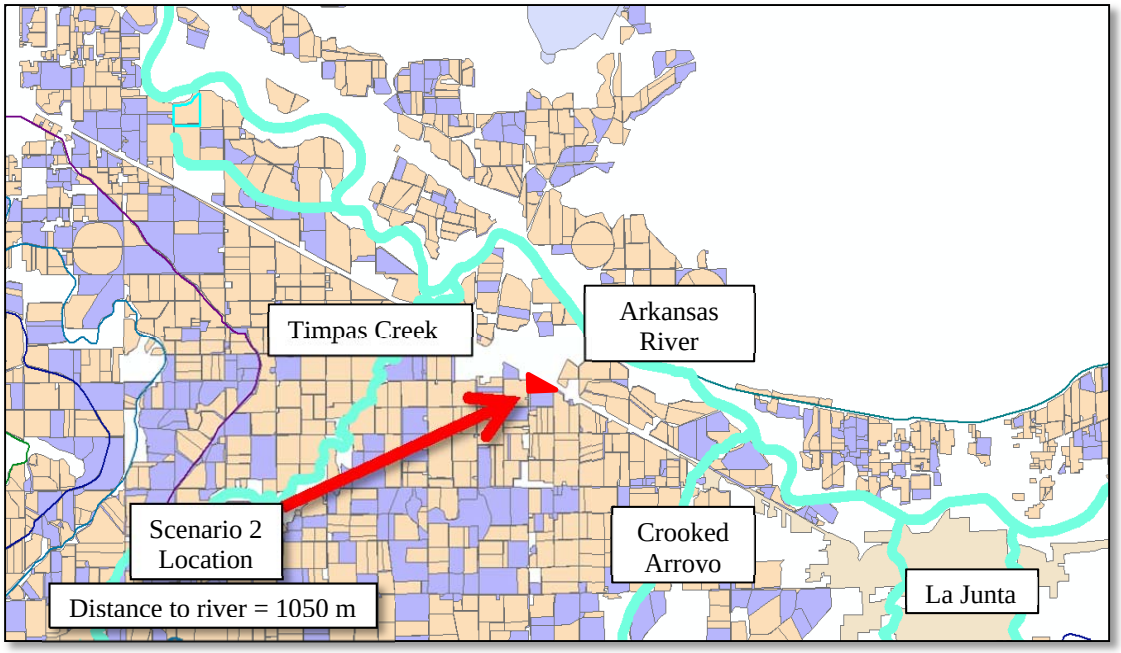

Figure 8. Location of Scenario 2 field in the LARV.

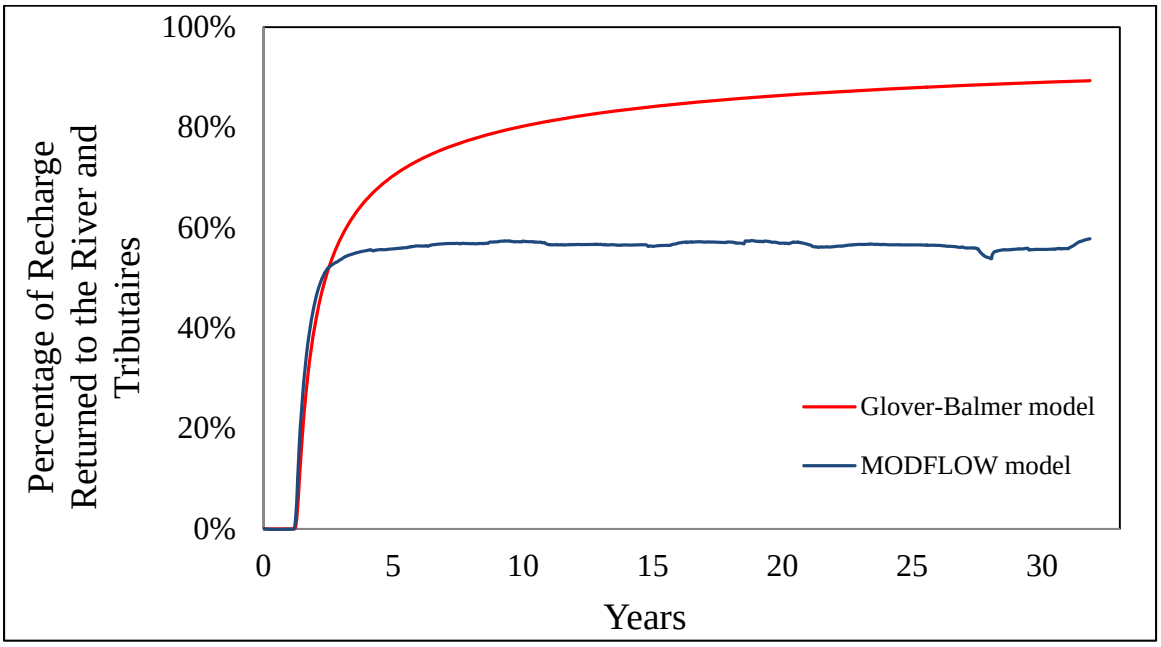

Figure 9. Cumulative recharge volume accreted to the Arkansas River, as predicted by the numerical and analytical models.

Table 2. UZF1 summary and Arkansas River tributary accretion for the Scenario 2 stressed cell.

Volumes expressed as a percentage of the stress volume	
Infiltrated volume	98%
Volume as aquifer recharge	95%
Total change in ET	+4%
Volume as tributary accretion	17%

A third scenario depicts the general result when a field is over 2 km from the Arkansas River. The field's location adjacent to the Otero Canal is shown in Figure 10. Figure 11 depicts the difference in timing of accretions to the Arkansas River between the numerical and analytical models as the distance from the river becomes relatively large. As distance is increased beyond about 2 km, accretions estimated by the Glover-Balmer solution begin to lag by a few years. The analytical solution also generates a significantly longer residual effect that tends to extend well beyond the effects seen using the numerical model. The cumulative accretions predicted over time highlight the deviation between the two models, as shown in Figure 12. Table 3 displays results specific to the numerical model. Results include a summary of activity in the unsaturated zone and significant impacts to the irrigation canals.

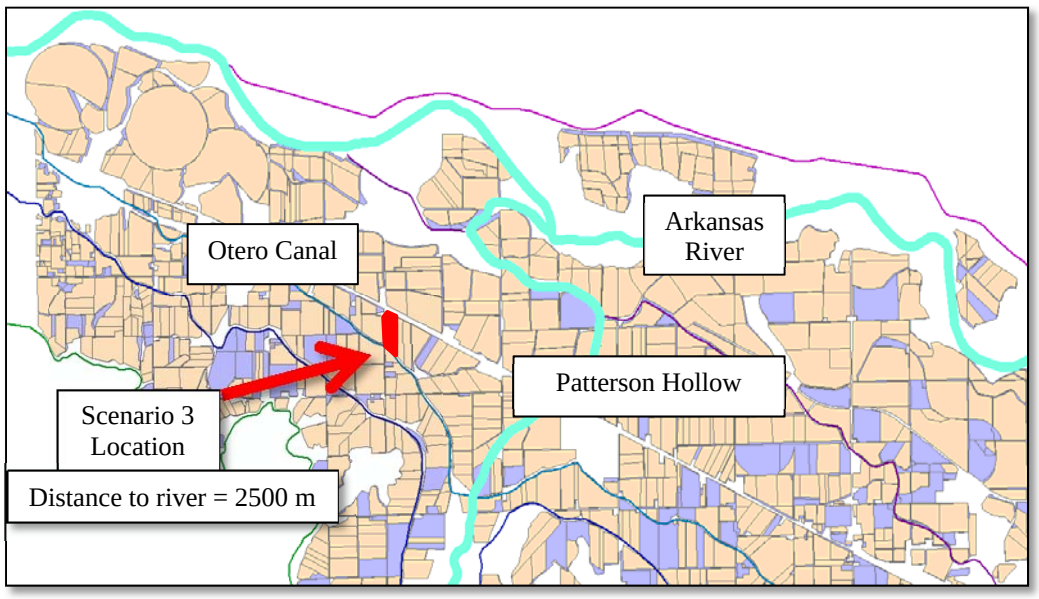

Figure 10. Location of Scenario 3 in the LARV.

In Scenario 3, total canal seepage drops significantly due to the close proximity of two irrigation canals. As water is added to the stressed cell, the water table rises which decreases the hydraulic head gradient from the irrigation canals to the adjacent groundwater table, significantly decreasing canal seepage.

For all scenarios, a ratio between the percentages of the added water in recharge basins estimated as accretion to the stream network using the MODFLOW-UZF model and using the Glover-Balmer model can be produced. This ratio, expressed as the MODFLOW-UZF model percentage divided by the Glover-Balmer model percentage, shows how closely the analytical model estimates are to a more realistic, complex scenario that is considered by the calibrated numerical model. If the two solutions predict that the same volume is accreted to the stream network, the ratio will equal 1. If the Glover-Balmer solution predicts a higher percentage accreting to the river than the MODFLOW model, the value will be less than one. Figure 13 is a plot of the ratio for each water addition scenario.

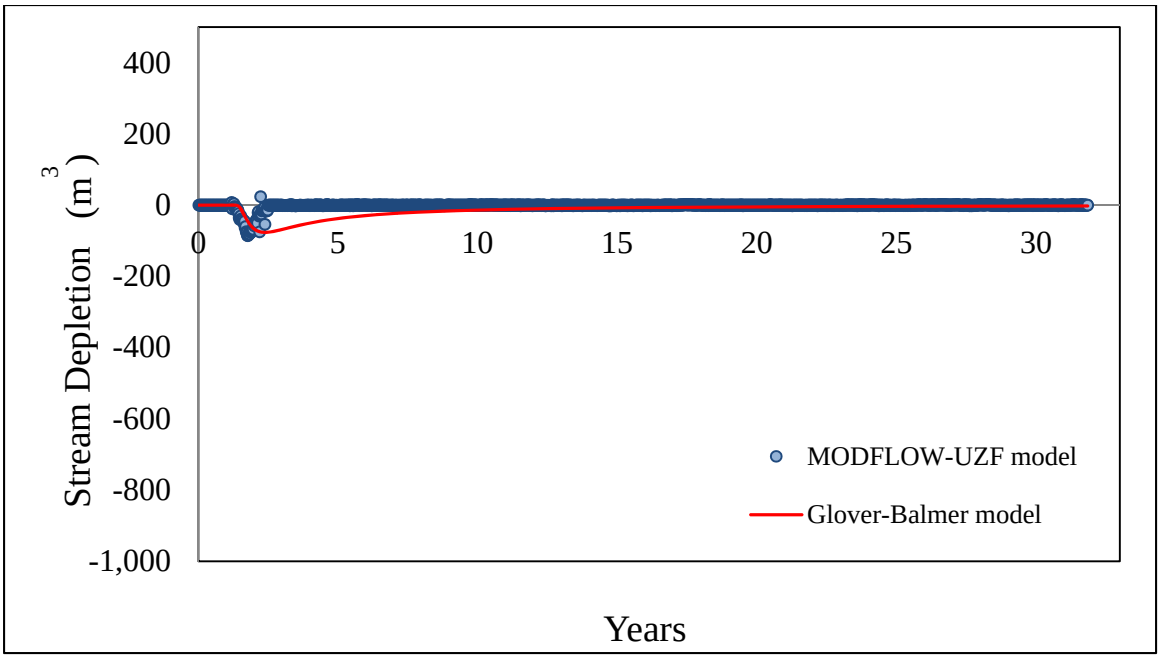

Figure 11. Stream accretion estimated by the MODFLOW-UZF model and the Glover-Balmer model over time for Scenario 3.

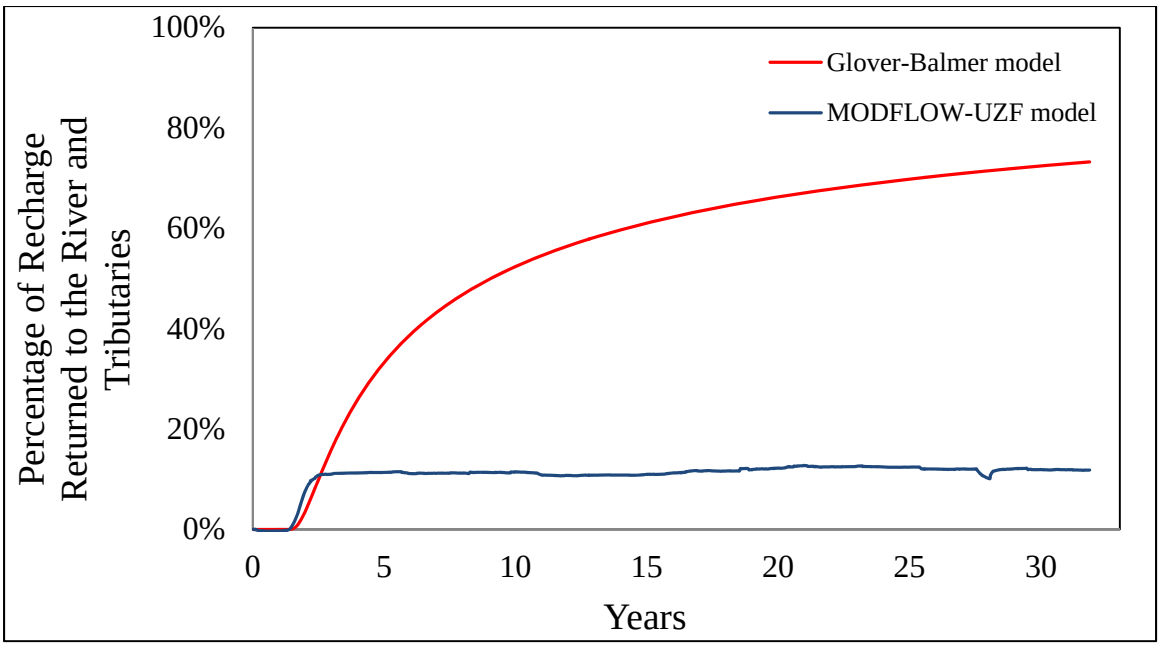

Figure 12. Cumulative recharge volume accreted to the Arkansas River for Scenario 3, as predicted by the numerical and analytical models.

Table 3. UZF1 activity for the stressed cell in Scenario 3 and canal seepage impacts.

Volumes expressed as a percentage of the stress volume	
Infiltrated volume	100%
Volume as aquifer recharge	90%
Total change in ET	+10%
Volume as a change in canal seepage	-45%

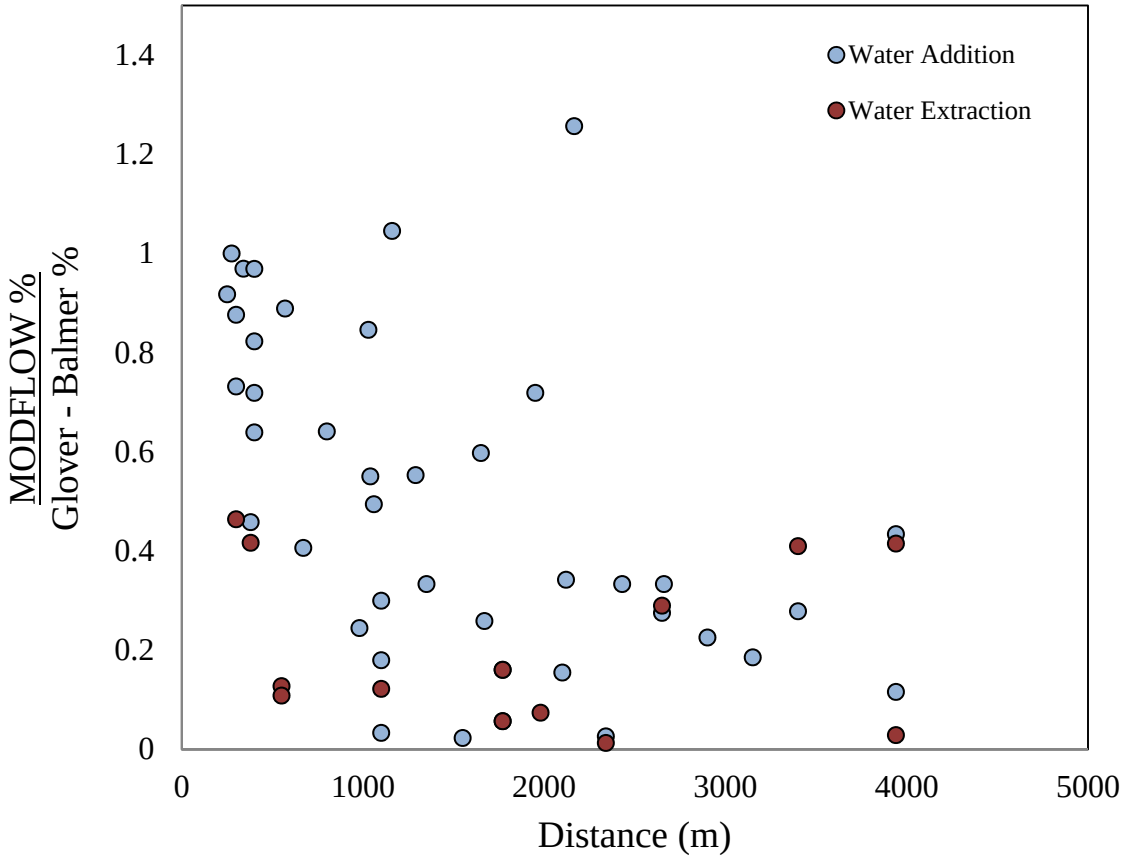

Figure 14. Summary plot for all recharge scenarios and for a few preliminary fallowing scenarios.

Scenarios 2 and 3 show that as fields increase in distance from the river, the accretion pattern predicted by the analytical solution increasingly deviates from what can be expected in reality. Scenarios 2 and 3 also demonstrate that, in certain cases, canals and tributaries can significantly impact accretion timing. This is due to their ability to intercept groundwater flows that are unaccounted for when using the single stream Glover-Balmer solution. Simplifying assumptions made by the Glover-Balmer solution do not account for complex flow patterns, changes in hydraulic head gradient, and water lost due to upflux and ET. In a typical recharge or lease-fallowing application, water will be applied or removed over an area that could span tens of acres. Since the Glover-Balmer solution was designed to model a pump, or point source, it is unable to adequately account for this distributed areal effect. Some effort has been made by Knight et al. (2004) to modify the Glover-Balmer solution to account for areal effects.

An examination of the performance of the Glover-Balmer solution for depletion of return flows due to water extraction has been initiated by considering thirteen scenarios of the removal of irrigation water from fallowed fields. For comparative purposes, the volume and temporal pattern of extraction in these cases were the same as the volume and temporal pattern of recharge for the previous scenarios. Future refinements will consider water-removal patterns that represent lease-fallowing conditions more realistically. Results for these scenarios also are plotted in Figure 14. Significant deviations between the two models are prevalent for these scenarios, and the deviation between the two are of a similar magnitude as those for the water addition scenarios. However, the addition and extraction scenario types create a mounding or lowering of the groundwater table, respectively, and this creates differences in ET, upflux, and canal seepage. More extraction scenarios will be considered in the future.

This study demonstrates that, due to the simplifying assumptions, the Glover-Balmer solution typically tends to overestimate impacts at the stream network due to field water accretions and depletions when compared to a data-calibrated model of a real-world system. This is consistent with the comparisons to simplified hypothetical aquifers of Spalding and Khaleel (1991) and Sophocleous et al. (1995) which considered only cases of water extraction.

CONCLUSION

The results of this study reveal general trends regarding the accuracy of the Glover-Balmer analytical solution in estimating stream accretions and depletions due to alterations in field water management in a real-world setting. In general, if the field in consideration is adequately close to the river (within about 500 m), the Glover-Balmer solution is relatively accurate in estimating impact on groundwater return flow. Accuracy of the Glover-Balmer solution tends to decrease with distance from the stream, although a high degree of scatter exists in the trend due to the complexities of the natural environment and engineered hydrologic system of the LARV. Examples of processes that can have a significant impact on the hydrologic system, and that are not accounted for in the Glover-Balmer solution, include upflux from a shallow water table, ET in the unsaturated zone, a stream network that does not fully penetrate the aquifer, canals and tributaries that intercept groundwater flow, and heterogeneity in aquifer materials, among others.

This study shows that the Glover-Balmer method can yield estimates that deviate significantly from reality, which is consistent with the findings of others studies. Results also show that the solution tends to increasingly deviate from the real system as it is applied increasingly far from the river. This can begin to serve as a guideline for water rights administration and for those who apply the solution in real-world settings, such as in the determination of effects on streamflow from lease-fallowing arrangements.

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GRASS REFERENCED BASED VEGETATION COEFFICIENTS FOR ESTIMATING EVAPOTRANSPIRATION FOR A VARIETY OF NATURAL VEGETATION

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ABSTRACT

In arid and semi-arid regions, evapotranspiration from vegetation results in the significant utilization of available water. Accurate estimates of evapotranspiration are required for surface and subsurface hydrologic evaluations as well as irrigation district water balance studies. A significant amount of transferable information exists for irrigated agricultural crops through past and current research in the form of grass or alfalfa reference based crop coefficients (Kc) and basal crop coefficients (Kcb). However, transferable evapotranspiration information on natural vegetation is limited. Much of the work was conducted in the early to mid-1900's and is presented as actual evapotranspiration from the vegetation at the research site either as annual or monthly values. In some cases, the data may have been referenced to evaporation pan measurements (typically Class A type pans) with unknown site conditions. An intensive literature review was conducted to extract monthly measured evapotranspiration information for natural vegetation types under various conditions. Monthly vegetation coefficients (Kv) for standardized grass reference based evapotranspiration (ETo) were computed using long-term average grass reference evapotranspiration information computed with data from nearby weather stations. Comparisons of the Kv values for similar vegetation indicate higher variability during the non-summer months but results from most of the studies examined are in good agreement. These Kv values provide some level of transferability so that it is possible to compute an accurate estimate of vegetative evapotranspiration with daily or monthly standardized grass reference evapotranspiration values in areas away from the original study.

INTRODUCTION

Estimating plant evapotranspiration accurately for planning and management has long been a challenge. Since the early 1900's, if not earlier, researchers have used an array of methodologies to attempt to measure plant evapotranspiration. While most of the early work on evapotranspiration was problematic because of poor experimental setup (Young and Blaney 1942), much was learned about proper ET measurement.

There has been significant research regarding ET from agricultural crops as well as natural vegetation. However, there is often a major difference in the way the data is

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presented for agricultural crops versus natural vegetation. For agricultural crops, information is generally presented so that evapotranspiration measurements made during specific times and at specific locations can be used in the future in different locations. In fact, over the past several decades there has been a focus on standardizing the way evapotranspiration predictions are made. Through this work, the American Society of Civil Engineers and the Food and Agricultural Organization have standardized the reference crop evapotranspiration computations that are used throughout the agricultural community worldwide (Allen et al. 1998; Allen et al. 2005).

Presenting results that can be used towards prediction of ET from similar plant types and growing conditions has not been a focus of much of the work with natural vegetation. In cases where a relationship between some reference and the measured ET has been presented, there has been no standardization of the reference. Historically, Weather Bureau Class A Pan evaporation was used as the reference. Starting in the early 1970's the Priestley-Taylor method became popular for natural vegetation ET estimation because of the limited amount of input data needed. The Jensen-Haise and Blaney-Criddle Methods have also been used as references (Jensen et al. 1990).

Without some standard reference for computing evapotranspiration from natural vegetation, it is difficult to utilize existing and past research to estimate historical or predict future evapotranspiration from similar vegetation. The goal of this paper is to present grass (short crop) reference evapotranspiration based vegetation coefficients (Kv) for a variety of natural vegetation types from other researchers. Most of the data included here was originally presented as monthly evapotranspiration depths without any reference. A major challenge was to estimate the grass reference evapotranspiration during the time frame and at the location the studies were conducted.

METHODS

An intensive review of natural vegetation evapotranspiration literature was conducted as part of this work. There have been several reviews conducted on this subject (Johns 1989; Drexler et al. 2004; Moore et al. 2004) and it was not the intent to repeat this information here. Within the literature, specific information was sought to develop useful, reliable vegetation coefficients. One of the main criteria for selection was that at least monthly data had to be provided. Interestingly, this was one of the most limiting factors.

The investigators must have measured evapotranspiration from vegetation surrounded by similar vegetation on all sides using a lysimeter/tank, Bowen ratio, eddy correlation, or remote sensing of actual evapotranspiration using an energy balance. Estimates of ET using a larger scale water balance were avoided because of the inaccuracies associated with measurements of inflow and outflow and the change in storage. A number of studies investigated evapotranspiration of vegetation that was not surrounded by vegetation of similar height and density. This was not uncommon in early ET measurements and will lead to significant overestimation of ET due to the clothesline effect (Young and Blaney 1942; Allen et al. 2011). The data gathered from the literature review focused on ET investigation after 1945 unless the site conditions and experimental

methods were explained in sufficient detail and the researcher had significant amount of experience to provide confidence in the measurements. A majority of the studies utilized in this paper were conducted in the western U.S., although some information from Florida was used.

The ability to transfer and adjust evapotranspiration estimates made during a specific time frame in one location to a different location during a different time frame has been a challenge. Transferability is commonly attained by using a reference based on local weather conditions and an adjustment coefficient based on the vegetation and growth stage. The standard approach for agricultural crops is to use a reference crop evapotranspiration (ETo) computed from specialized weather station networks along with a crop coefficient (Kc) that was developed through research for specific stages of the crop cycle. Crop evapotranspiration (ETc) can be computed as:

$$ETc = ETo*Kc \tag{1}$$

The reference crop used is generally grass (short crop) or alfalfa (tall crop). Generally, ETo is used to identify grass and ETr is used to identify alfalfa reference. The 2005 ASCE Standardized Penman-Monteith (ASCE ETo) equation is the current standard for computation for either a grass or alfalfa reference evapotranspiration (Allen et al. 2005). Knowing the reference crop is critical since the crop/vegetation coefficients are different for each reference crop. In this paper all vegetation coefficients are based on a **grass reference crop**.

Using Equation 1, the monthly vegetation coefficients (Kv) were developed from the monthly ETc measurements obtained from the literature review as:

$$Kv = ETc/ETo \tag{2}$$

The grass reference evapotranspiration had to be estimated on a monthly basis for the time frame and the location that the study was conducted. Since most ETo weather stations were not installed in the western U.S. until the 1980's, it was not possible to use the standardized reference evapotranspiration equation for many of the datasets. Alternatively, the Hargreaves ETo equation was used in many cases where the full set of weather parameters was not available. The Hargreaves equation has been shown to provide relatively accurate ETo estimates with limited data (maximum and minimum temperature only) in arid regions (Jensen et al. 1990; Allen et al. 1998). Hargreaves ETo is computed based on temperature and extraterrestrial radiation (R_a) as:

$$\text{Hargreaves ETo} = 0.0023(T_{mean} + 17.8)(T_{max} - T_{min})^{0.5}\, R_a \tag{3}$$

where temperatures are in degrees Celsius and R_a and ETo are in millimeters per unit time. The Hargreaves equation does not include input information for wind or relative humidity. The lack of this information can lead to inaccuracies associated with the Hargreaves ETo. Allen et al. (1998) discusses a calibration method to improve the

accuracy of the Hargreaves ETo estimate on a monthly or annual basis by comparing it to the ASCE ETo for years with overlapping data.

ETo was determined for each site depending on the data availability. The list below is used to identify the method used to compute ETo for each study in Tables 1 and 2. The priority for determining ETo was:

1. In cases where the vegetation coefficient was provided and ETo was not needed, if the Kv provided was based on an alfalfa reference crop, these Kv values were multiplied by 1.15 to estimate Kv based on a grass reference. When possible, a conversion factor was computed on a monthly basis by dividing ETr/ETo over a period of two or more years to increase the accuracy of the grass reference based Kv.
2. If an ETo weather station existed near the study location during the study period, ASCE ETo was used.
3. If an ETo weather station was placed near the location (within 10-20 miles depending on the climate variability and terrain) of the study site after the study was conducted, a monthly calibrated Hargreaves ETo was used. Calibration was conducted based on years when weather station ETo was available.
4. If no ETo weather station was near the weather station but monthly temperature data was provided with the study data, Hargreaves ETo was used based on this temperature data.
5. If no ETo weather station was near the weather station and monthly temperature data for the study period was not provided, Hargreaves ETo was used based on PRISM data for the location and time frame of the study.

If (4) or (5) were used to estimate ETo, a check on these ETo values was made by checking against long-term average ASCE ETo, on an annual basis. The long-term average ASCE ETo used for the check was either from weather stations within 20-40 miles with similar climate conditions or, for studies in California, from Spatial CIMIS for the location of the study site (http://wwwcimis.water.ca.gov/cimis/cimiSatSpatialCimis.jsp). The difference between the annual ETo values was set at a threshold of a +/-15%. This reality check ensured that gross errors in the ETo were avoided. If the Hargreaves ETo was outside of this threshold, alternative means of computing ETo was attempted or the dataset was abandoned. The alternative method was to find a nearby NCDC weather station with temperature data for the time frame and use the Hargreaves equation to compute the ETo based on this data.

The PRISM (Parameter-elevation Regressions on Independent Slopes Model) system maintained by Oregon State University provides a grid of monthly temperatures (minimum and maximum) from 1895 to present (Daly et al. 2002; Daly et al. 2008). PRISM temperature data is computed based on surface weather station data and is interpolated based on factors such as location, coastal proximity, elevation and topography (Daly et al. 2008).

RESULTS

The following tables show the vegetation coefficient computed using Equation 2. Table 1 shows monthly Kv values for vegetation that is not lacking for water. The vegetation types include wetland tules and cattails in standing water, riparian habitat with access to the shallow groundwater table year-round, and native pasture grass with access to shallow groundwater. For the wetland and riparian vegetation (willows, cottonwoods, etc.), the tables have been split into two categories for each of these types of vegetation to differentiate between large and small stand (isolated patches) of vegetation.

Native pasture grass and irrigated pasture are shown following the riparian vegetation. Studies were selected for the pasture where the water tables were shallow. Native perennial grasses often have access to water through a shallow groundwater aquifer. The Kv values shown would represent relatively large meadow/grassland areas under these conditions.

Basic statistics are shown by month for vegetation with more than one value. The average, sample standard deviation (SD), and the coefficient of variation (CV) of monthly Kv's are shown. The CV is computed as the SD divided by the average monthly Kv.

For the large stand wetland vegetation, several of the published ETc values resulted in an relatively high Kv value for certain months. The amount of evapotranspiration is limited by the available energy to convert water as a liquid into a gas. For large stands of vegetation (>200 m of similar vegetation), the maximum potential Kv is 1.2-1.4 (Allen et al. 2011). Allen et al. (2011) recommends that values that exceed 1.4 should be excluded for large stands of vegetation. The highest Kv value for any single month for a study was 1.37 which is less than 1.4 so no values were excluded from these tables.

The limitation of Kv to 1.2-1.4 based on a grass reference ETo does not apply for small stands of vegetation. Very high ETc rates can occur in situations where a small, taller, stand of vegetation is surrounded by shorter vegetation. This is termed the "clothesline effect", whereby air can more efficiently move between the vegetation, lowering the humidity outside of the leaf and creating a greater potential for higher ETc (Allen et al. 2011). For this reason Kv values were differentiated between large and small stands in Table 1 for wetland and riparian vegetation.

Monthly grass reference based Kv values for other types of vegetation are shown in Table 2. These vegetation types may not have access to the shallow groundwater, and are therefore reliant on precipitation. In arid climates where vegetation will undergo water stress, Kv values will be dependent on the amount of available water. The Kv values shown in Table 2 should be used with caution.

Table 1. Grass reference based vegetation coefficient (Kv) for vegetation types that had continuous access to water (no water stress).

Large Stand Wetland Vegetation	Long-Term Freeze	ETo Est.	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Cattails	No	1	Fort Drum, Florida	0.51	0.61	0.64	0.73	0.87	0.87	0.78	0.76	0.86	0.78	0.65	0.56	lysimeter	(Mao et al. 2002)
Cattails	No	1	Southern Florida		0.69	0.73	1.00	1.15	1.15	1.15	1.15	1.09				lysimeter	(Abtew and Obeysekera 1995) Reported in (Allen 1998)
Tules and Cattails	No	1	Twitchell Island, CA					0.80	0.92	1.02	1.09	1.01	0.90			surface renewal	(Drexler et al. 2008)
Tules/Bulrush	No	5	Bonsall, Ca	0.36	0.61	0.76	1.09	1.21	1.20	1.21	1.16	1.15	1.33	0.98	0.78	tank within vegetation	(Muckel and Blaney 1945)
Tules/Bulrush	No	5	Bonsall, Ca	0.83	0.61	0.94	1.11	1.24	1.24	1.14	1.14	1.12	1.06	0.78	0.97	tank within vegetation	(Muckel and Blaney 1945)
Tules/Bulrush	No	5	Bonsall, Ca	0.98	0.77	0.66	0.83	0.99	1.22	1.27	1.37	1.25	1.23	1.20	0.70	tank within vegetation	(Muckel and Blaney 1945)
Cattails	Yes	1	Logan, Utah				0.35	0.75	1.27	1.30	1.30	0.73				Bowen ratio	(Allen 1998)
Tules, cattails, wocus lily	Yes	1	Upper Klamath NWR	0.91	0.91	0.91	0.91	0.92	1.06	1.10	1.12	0.72	0.91	0.91	0.91	eddy covariance	(Stannard 2013)
Tules/Bulrush	Yes	1	Upper Klamath NWR	1.01	1.01	1.01	1.01	0.97	1.08	1.09	1.20	0.83	1.01	1.01	1.01	eddy covariance	(Stannard 2013)
			Average	**0.77**	**0.74**	**0.81**	**0.88**	**0.99**	**1.11**	**1.12**	**1.14**	**0.97**	**1.03**	**0.92**	**0.82**		
			SD	**0.27**	**0.16**	**0.15**	**0.25**	**0.18**	**0.14**	**0.15**	**0.17**	**0.19**	**0.19**	**0.19**	**0.17**		
			CV	**0.35**	**0.22**	**0.18**	**0.28**	**0.18**	**0.13**	**0.14**	**0.15**	**0.20**	**0.19**	**0.21**	**0.21**		

Italicized values are likely measurement errors and were not included in the statistics

Table 1. (continued)

Small Stand Wetland Vegetation	Long-Term Freeze	ETo Est.	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Cattails	No	5	King Island, CA	1.28	1.47	1.61	1.18	1.79	1.46	1.87	1.43	1.52	1.70	1.97	1.36	tank within vegetation	(Young and Blaney 1942)
Cattails	No	5	King Island, CA	0.75	1.09	1.80	1.96	2.64	1.85	1.88	1.40	1.59	2.38	1.97	0.60	tank within vegetation	(Young and Blaney 1942)
Tules/Bulrush	No	3	Victorville, CA	0.46	0.56	0.75	0.81	1.08	1.33	1.39	1.42	1.58	1.26	0.89	0.73	tank within vegetation	(Young and Blaney 1942)
Tules/Bulrush	Yes	1	Logan, Utah				0.35	0.96	1.76	1.81	1.81	0.97				Bowen ratio	(Allen 1998)
Cattails	Yes	1	Logan, Utah				0.35	0.82	1.60	2.03	1.54	0.52				Bowen ratio	(Allen 1998)
			Average	**0.83**	**1.04**	**1.39**	**0.93**	**1.46**	**1.60**	**1.80**	**1.52**	**1.24**	**1.78**	**1.61**	**0.90**		
			SD	**0.42**	**0.46**	**0.56**	**0.67**	**0.76**	**0.21**	**0.24**	**0.17**	**0.47**	**0.56**	**0.63**	**0.40**		
			CV	**0.50**	**0.44**	**0.40**	**0.73**	**0.52**	**0.13**	**0.14**	**0.11**	**0.38**	**0.32**	**0.39**	**0.45**		

Large Stand Riparian Vegetation	Long-Term Freeze	ETo Est.	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Measurement Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Willows	No	4	Santa Ana, CA		0.68	0.78	1.05	0.82	0.90	1.13	1.20	1.43	1.21	1.09	0.80	tank within vegetation	(Young and Blaney 1942)
Cottonwood	Yes	1	Middle Rio Grande, NM	0.81	0.72	0.61	0.66	0.82	0.94	1.02	1.02	1.07	1.08	0.88	0.89	tank within vegetation	(Allen et al. 2005)
R.Olive	Yes	1	Middle Rio Grande, NM	0.83	0.74	0.64	0.70	0.86	0.99	1.06	1.06	1.12	1.12	0.92	0.92	remote sensing (METRIC)	(Allen et al. 2005)
Willow	Yes	1	Middle Rio Grande, NM	0.81	0.67	0.55	0.59	0.74	0.86	0.93	0.95	1.07	1.05	0.86	0.89	remote sensing (METRIC)	(Allen et al. 2005)
			Average	**0.82**	**0.70**	**0.65**	**0.75**	**0.81**	**0.93**	**1.03**	**1.06**	**1.17**	**1.11**	**0.94**	**0.87**		
			SD	**0.02**	**0.03**	**0.10**	**0.21**	**0.05**	**0.06**	**0.09**	**0.11**	**0.18**	**0.07**	**0.10**	**0.05**		
			CV	**0.02**	**0.05**	**0.15**	**0.28**	**0.06**	**0.06**	**0.08**	**0.10**	**0.15**	**0.06**	**0.11**	**0.06**		

Table 1. (continued)

Small Stand Riparian Vegetation	Long-Term Freeze	ETo Est.	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Measurement Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Common reed, willow, cottonwood	Yes	1	Platte River Basin, Central City, Nebraska					0.80	1.24	1.40	1.50	1.13	0.91			Bowen ratio	(Irmak et al. 2013)
Common reed, willow, cottonwood	Yes	1	Platte River Basin, Central City, Nebraska					1.00	1.69	1.75	1.79	1.97	1.66			Bowen ratio	(Irmak et al. 2013)
			Average					**0.90**	**1.46**	**1.57**	**1.64**	**1.55**	**1.28**				
			SD					**0.14**	**0.32**	**0.25**	**0.21**	**0.60**	**0.53**				
			CV					**0.16**	**0.22**	**0.16**	**0.13**	**0.39**	**0.42**				

Large Stand Saltcedar	Long-Term Freeze	High WT	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Measurement Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Saltcedar	No	8 ft	Havasu NWR, AZ	0.22	0.22	0.28	0.50	0.72	0.76	0.76	0.76	0.75	0.59	0.34	0.22	Bowen ratio	(Westenburg et al. 2006)
Saltcedar	Yes	>10 ft	Bosque del Apache, NM		0.44	0.23	0.30	0.53	0.85	0.96		0.78				eddy covariance	(Bawazir et al. 2009)*
Saltcedar	Yes	3ft to 15ft	Bosque del Apache, NM				0.50	0.76	0.87	1.05	1.21	1.14	0.97	0.43		eddy covariance	(Bawazir et al. 2006)
Saltcedar	Yes		Middle Rio Grande, NM	0.75	0.61	0.47	0.49	0.58	0.68	0.79	0.85	1.00	1.02	0.82	0.83	remote sensing (METRIC)	(Allen et al. 2005)
			Average	**0.49**	**0.42**	**0.33**	**0.45**	**0.65**	**0.79**	**0.89**	**0.94**	**0.92**	**0.86**	**0.53**	**0.53**		
			SD	**0.37**	**0.19**	**0.13**	**0.10**	**0.11**	**0.09**	**0.14**	**0.24**	**0.19**	**0.24**	**0.26**	**0.43**		
			CV	**0.77**	**0.45**	**0.40**	**0.22**	**0.17**	**0.11**	**0.16**	**0.25**	**0.20**	**0.28**	**0.48**	**0.81**		

*Values are single day measurements within the months shown, not monthly averages

Table 1. (continued)

Large Stand Non-stressed Pasture	Long-Term Freeze	ETo Est.	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Measurement Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Native Pasture	Yes	5	Alturas, CA	0.46	0.43	0.51	0.97	0.97	1.13	1.21	1.28	1.20	1.07	0.69		tank within vegetation	(MacGillivray 1975)
Native Pasture	Yes	5	Shasta County, CA	0.29	0.29	0.38	0.90	0.95	1.02	1.09	1.12	1.10	0.99	0.93	0.86	tank within vegetation	(MacGillivray 1975)
Irrig. Pasture (WT 0-2 ft)	Yes	5	Carson Valley, NV	0.82	0.82	0.90	1.23	1.17	0.93	0.99	0.98	1.09	0.86			eddy covariance	(Maurer et al. 2006)
Irrig. Pasture (WT 2-5ft)	Yes	5	Carson Valley, NV	0.75	0.70	0.63	0.76	1.00	0.84	0.77	0.56	0.50	0.48			Bowen ratio	(Maurer et al. 2006)
			Average	**0.58**	**0.56**	**0.60**	**0.96**	**1.02**	**0.98**	**1.02**	**0.98**	**0.97**	**0.85**	**0.81**	**0.86**		
			SD	**0.25**	**0.24**	**0.22**	**0.20**	**0.10**	**0.13**	**0.18**	**0.31**	**0.32**	**0.26**	**0.17**			
			CV	**0.43**	**0.43**	**0.37**	**0.20**	**0.10**	**0.13**	**0.18**	**0.31**	**0.33**	**0.31**	**0.21**			
			CV	**0.77**	**0.45**	**0.40**	**0.22**	**0.17**	**0.11**	**0.16**	**0.25**	**0.20**	**0.28**	**0.48**	**0.81**		

Table 2. Grass reference based vegetation coefficient (Kv) for other types of vegetation types that were primarily rainfed.

Other Vegetation	ETo Est.	WT Depth	Location	Vegetation Coefficient (ETc/ETo) based on Grass Reference												Method	Source
				Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec		
Oak-Grass Savanna	2	None	Near Iona, CA	0.54	0.39	0.49	0.59	0.55	0.30	0.18	0.11	0.07	0.04	0.36	1.06	eddy covariance	(Baldocchi et al. 2004)
Bermuda grass	4	2 ft	San Bernardino, CA	0.58	0.35	0.43	0.59	0.61	0.89	1.02	0.87	0.74	0.75	0.88	0.84	tank within vegetation	(Taylor 1934)
Bermuda grass	4	3 ft	San Bernardino, CA	0.55	0.33	0.30	0.48	0.45	0.76	0.84	0.71	0.61	0.66	0.82	0.60	tank within vegetation	(Taylor 1934)
Arrowweed	1	8 ft	Havasu NWR, AZ	0.21	0.21	0.32	0.46	0.56	0.56	0.56	0.55	0.45	0.34	0.23	0.21	Bowen ratio	(Westenburg et al. 2006)
Mesquite, saltcedar, salt grass	1	4-8 ft	Havasu NWR, AZ	0.30	0.30	0.37	0.46	0.53	0.53	0.53	0.47	0.40	0.33	0.30	0.30	Bowen ratio	(Westenburg et al. 2006)
chaparral	5	No	Sierra Ancha Forest, AZ	0.30	0.32	0.26	0.34	0.35	0.04	0.21	0.33	0.30	0.21	0.34	0.40	lysimeter	(Rich 1951)
Sacaton	5	25 in	Carlsbad, NM			0.47	0.72	0.84	0.74	0.96	0.92	0.49	0.78	0.72	0.16	tank within vegetation	(Blaney 1954)
Saltgrass	4	12 in	Santa Ana, California	0.51	0.54	0.44	0.67	0.64	0.89	1.15	1.00	0.81	0.72	0.75	0.66	tank within vegetation	(Young and Blaney 1942)
Saltgrass	4	24 in	Santa Ana, California	0.19	0.20	0.17	0.26	0.27	0.35	0.45	0.40	0.33	0.29	0.29	0.25	tank within vegetation	(Young and Blaney 1942)
Saltgrass	4	36 in	Santa Ana, California	0.79	0.63	0.64	0.61	0.00	0.27	0.38	0.33	0.33	0.35	0.70	0.89	tank within vegetation	(Young and Blaney 1942)
Saltgrass	4	48 in	Santa Ana, California	0.12	0.11	0.09	0.19	0.06	0.12	0.37	0.49	0.31	0.39	0.29	0.29	tank within vegetation	(Young and Blaney 1942)

DISCUSSION

The Kv values show relatively good agreement between studies from Table 1. As one might expect, fall and winter Kv values have higher coefficients of variation than those from late spring through summer in most cases. This is likely due to variable precipitation amounts resulting in different amounts of evaporation. Additionally, this variability can be attributed to data from studies where the vegetation that may have been dormant (long-term winter freeze) was grouped with studies that had lower levels of dormancy.

Since the higher variability in Kv's occurs during the portion of the year where ETo is lower, the potential resulting inaccuracy toward the annual ETc estimate will be less significant. This is illustrated in Table 3. Long-term ETo for an area near Stockton, CA was used to compute the ETc for large stands of wetland vegetation. Kv values from the study by Muckel and Blaney (1945) for an area near San Diego were compared with the average monthly Kv values shown in Table 1. The resulting difference in annual ET estimates was approximately -1.7%. On a monthly basis the differences were higher during the fall, winter and spring because of the variability in Kv during these months. However, during the highest ETo months, the differences were smaller.

Table 3. Comparison of ETc computed using Kv values estimated from Muckel and Blaney (1945) for Bonsall, CA and the overall average Kv for large stand wetlands.

Large Stand Wetlands	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
ETo (inches)	1.59	2.2	3.66	5.08	6.83	7.80	8.67	7.81	5.67	4.03	2.13	1.59	57.06
Bonsall, CA													
Kv	0.36	0.61	0.76	1.09	1.21	1.20	1.21	1.16	1.15	1.33	0.98	0.78	
ETc (inches)	**0.58**	**1.33**	**2.78**	**5.53**	**8.27**	**9.36**	**10.51**	**9.10**	**6.54**	**5.37**	**2.09**	**1.25**	**62.70**
Average													
Kv	0.91	0.88	0.92	0.96	1.10	1.14	1.16	1.17	1.14	1.11	0.92	0.87	
ETc (inches)	**1.44**	**1.94**	**3.37**	**4.89**	**7.54**	**8.89**	**10.10**	**9.17**	**6.47**	**4.49**	**1.96**	**1.38**	**61.63**
ETc Difference (inches)	**0.86**	**0.61**	**0.59**	**-0.64**	**-0.74**	**-0.47**	**-0.41**	**0.07**	**-0.07**	**-0.89**	**-0.13**	**0.14**	**-1.07**

Most of the studies examined were conducted in arid environments. Kv values can be different in areas with higher relative humidity. Transferability of Kv values should be limited to similar general climate conditions.

The values in Table 2 should be examined with caution. Computing ETc using a single vegetation coefficient method can result in significant error since the ETc rate will depend on water availability to the plant. A more appropriate method would be to use the dual crop coefficient method (Allen et al. 1998) using a daily, or more frequent, root zone soil water balance to account for potential water stress with limited soil moisture. The

values in Table 2 along with information from the studies themselves may be useful for model calibration or as a reality check to a root zone soil water balance model.

CONCLUSION

A list of grass reference based vegetation coefficients estimated from previous research on natural vegetation is presented. While the list is not exhaustive, there is good agreement between studies for similar vegetation types and site conditions especially during the high evapotranspiration months. During the winter, Kv values showed more variability due to dormancy and precipitation. The Kv values presented in this study will hopefully assist water managers and planners more accurately estimate natural vegetation ETc.

ACKNOWLEDGEMENTS

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EVALUATION OF APPROPRIATE URBAN WATER CONSERVATION PRACTICES TO ENSURE WATER AVAILABILITY FOR AGRICULTURAL PRODUCTION

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Larry Roesner[2]

ABSTRACT

Urban water conservation is a way to enable economic growth in urban areas while ensuring national food security. The Integrated Urban Water Model (IUWM) has been developed by Colorado State University to evaluate alternative water conservation practices such as reclaimed water reuse, rainwater catchment, indoor conservation fixtures and graywater reuse. Results will be presented for scenarios in Colorado where water supply savings, wastewater treatment reduction and cost savings are demonstrated for these conservation practices. Reclaimed water reuse results in the highest reduction of urban water demand, but can be costly due to infrastructure requirements. Graywater (water from laundry, showers, baths and handwash sinks) reuse is an attractive alternative for water conservation in the urban environment due to low infrastructure requirements and its ability to achieve substantial reductions in demand for potable water while also reducing wastewater generated.

INTRODUCTION

Many Colorado municipalities are looking for approaches to reduce urban water demand to ensure that agricultural demands can be met. Urban water conservation is a way to enable economic growth in urban areas while ensuring water availability for crop production. Given the large population growth projects in the Front Range area of Colorado and a decreasing water supply, urban water conservation is imperative to ensure long term food security. The Integrated Urban Water Model (IUWM) has been developed by Colorado State University to evaluate alternative water conservation practices such as reclaimed water reuse, rainwater catchment, indoor conservation fixtures and graywater reuse. The objective of this work was to apply IUWM to a simulated new development area in Fort Collins, CO to evaluate demand reduction, wastewater reduction and cost savings associated with these practices.

METHODS

IUWM was developed in the Visual Basic .NET programming language and serves as a front end interface for a collection of database files. The application can be installed on any Windows operating system that has the .NET framework installed. The user starts by selecting a water user area and water user group (i.e. single family residential, multi-

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family residential, commercial/institutional, or industrial) and inputting data on the water user area (Figure 1). The user then selects water management practices they would like to explore and is finally provided with information on potable water use, wastewater generated, and potential cost savings associated with these management practices. Water management practices included in IUWM are installation of indoor conservation fixtures, irrigation conservation, graywater reuse for toilet flushing and irrigation, reclaimed water use for irrigation, and rainwater collection for irrigation. Indoor water demands were estimated based on the Residential End-uses of Water Study (REUWS; Mayer, P.; DeOreo, W., 1999) study. Irrigation demand was estimated based on monthly evapotranspiration for a given hydrologic region, while precipitation data were collected from the National Climatic Data Center (NCDC). Of note is that hydrologic data was considered on a daily basis such that appropriate storage and use of rainwater could be calculated. All model outputs were calculated on a daily time step. Future versions of IUWM will consider daily evapotranspiration and include specific savings associated with different irrigation systems (i.e. automated precipitation based irrigation, soil moisture irrigation and drip irrigation). A simplified estimate of cost is supplied as output for of treated water and wastewater treatment in expressed in terms of millions of gallons per year. These estimates are based on a flat rate, per volume cost input by the user for each potable water supply, wastewater treatment, and delivery of treated effluent. At this time, IUWM is not capable of estimating whole life costs associated with various management practices.

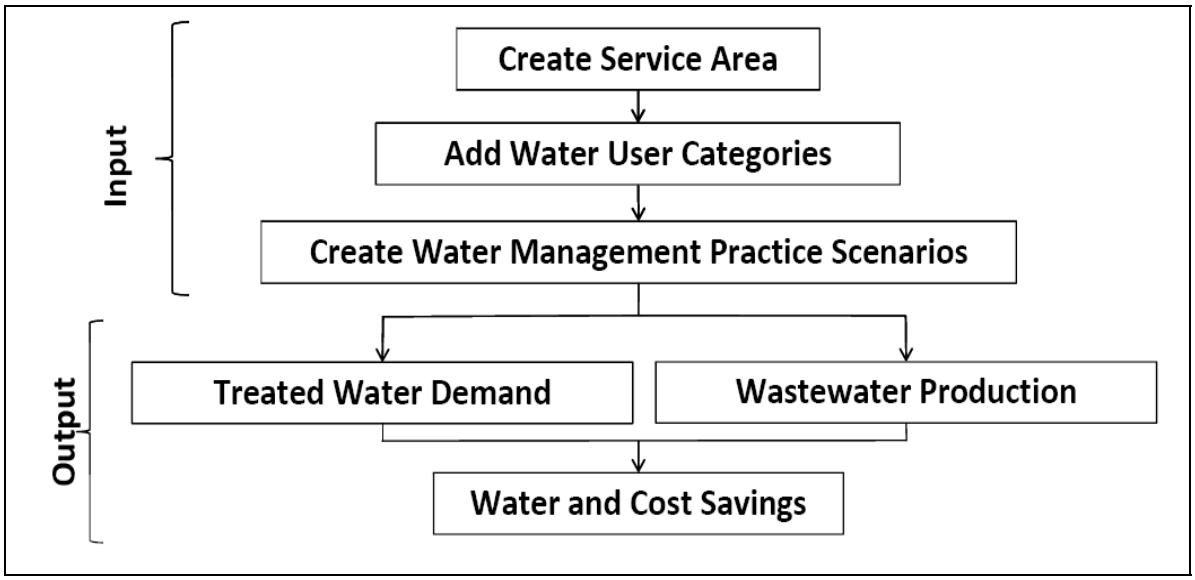

Figure 1. Schematic of Input and Output for the IUWM

An example application of IUWM was conducted for a simulated new development residential area in Fort Collins. While IUWM is capable of modeling several water user categories (residential, multi-residential and commercial), the focus of this work was residential water users. The application serves as an approach to explore the effectiveness

of combinations of water management practices based on the hydrology of the area. Since the application was assumed to be a new development area, no limitations were assumed in terms of ability to implement water conservation practices due to retrofitting requirements. Population characteristics, land use, and cost inputs were set at values reported in Table 1 for all simulations. Therefore, results reported are for an 800 acre residential area in Fort Collins, CO.

Table 1. Input Values for Application to Fort Collins Area.

Average Indoor Demand	69.3	*gpcd*
Water User Area Size	800	*acres*
Average Lot Size	0.25	*acres*
Average Percent of Area Irrigated	50	*%*
Average Persons per Lot	3	
Potable Water Delivery Cost	2.50	*$/1000 gal*
Municipal WWTP Cost	3.25	*$/1000 gal*

The following assumptions were made for urban water conservation practices:

1.) Indoor conservation – reduction of 35% demand through leak reduction and use of water conservation fixtures including high efficiency laundry machines (33% reduction), low flow shower heads (24% reduction), low flow toilets (55% reduction) and high efficiency dishwasher machines (30% reduction).

2.) Irrigation conservation – Irrigation demand reduction of 25% through installation of efficient irrigation systems, management, and xeriscape.

3.) Graywater reuse for toilet flushing and irrigation – Graywater is defined as wastewater from laundry, handwash, baths and showers. An adoption rate of 100% was assumed for combined toilet flushing and irrigation. The storage tanks were assumed to be 200 gallons on average.

4.) Treated Wastewater Reuse for Irrigation –90% of wastewater is available for reuse at a cost of $1.90 per 1000 gallons.

5.) Stormwater Capture for Irrigation – Each home has storage tanks with a volume of 1000 gallons and 15% of the rainfall can be collected (i.e. 15% of the area is roofs where runoff can be collected).

RESULTS

Urban water demand reduction was evaluated for each of the studied conservation practices individually (Figure 2). Based on the hydrologic conditions in Fort Collins, CO, the largest reduction in urban demand can be achieved through reuse of reclaimed water for irrigation (near 27% reduction in demand). Irrigation conservation and graywater reuse can also achieve substantial reduction in water demand. Of note is that rainwater capture, even when 1000 gallon storage tanks are adopted, does very little to reduce

urban water demand. Much concern exists over the impacts to water rights when rainwater is captured and used in the urban environment and this is a practice that the public is generally enthusiastic about. However, it offers very little benefit in terms of water savings.

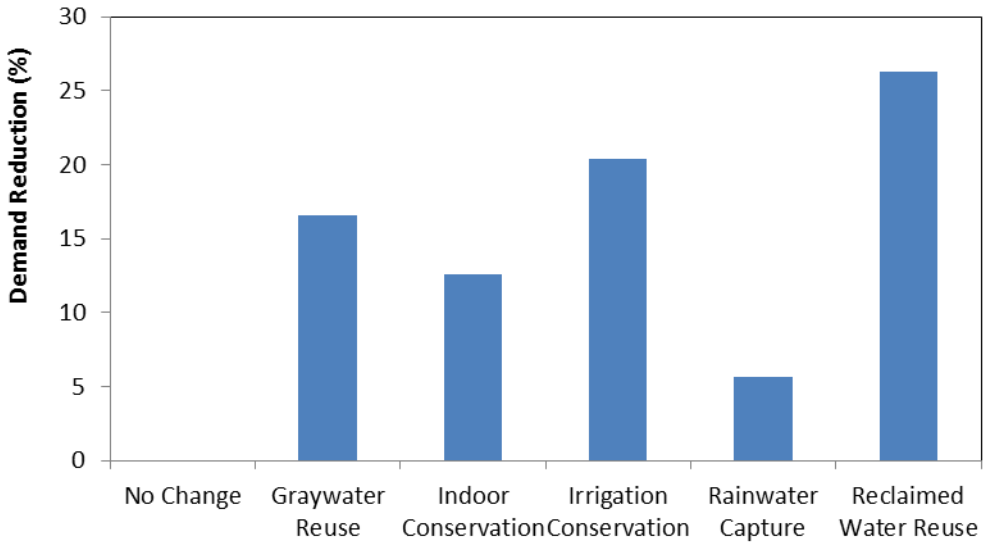

Figure 2. Water Demand Reduction Associated with Urban Water Conservation Practices

It is critical to consider cost savings associated with water conservation practices. For this project, costs savings were determined based on water demand reduction and wastewater reduction and their associated costs (Figure 3). Of note is that while reclaimed water reuse resulted in the highest reduction of water demand (Figure 2), the practice offers little cost savings (Figure 3). Cost savings are calculated for an 800 acre residential service area in the City of Fort Collins as defined in Table 1. This due to the cost associated with delivering treated wastewater back to customers. Here, a cost of $1.90 per thousand gallons was assumed, slightly lower than the cost for delivering potable water. In some U.S. cities, such as Raleigh, North Carolina, the cost for reclaimed water is much higher ($3.25 per thousand gallons) than this. Graywater reuse and indoor conservation practices offer the most cost savings. This is because both of these practices reduce the flow of wastewater delivered to a wastewater treatment facility.

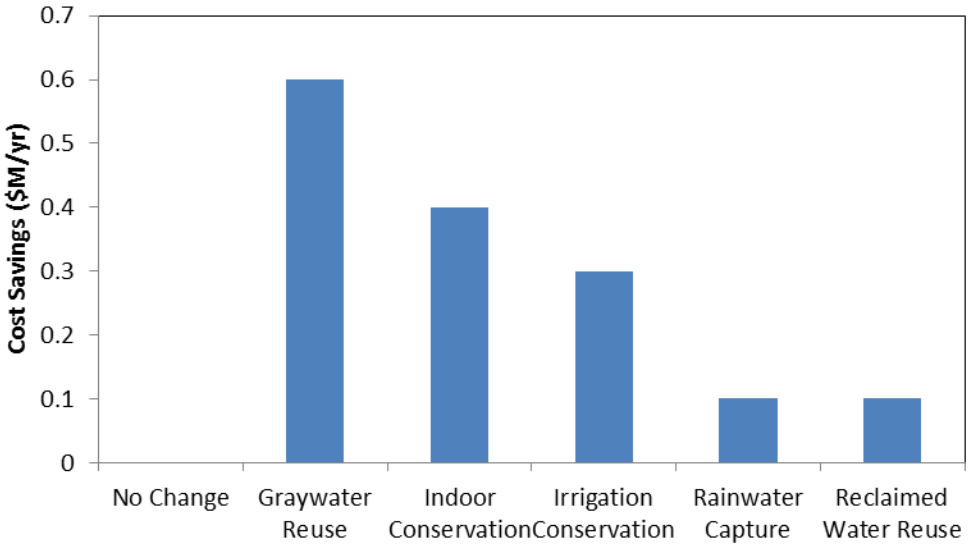

Figure 3. Cost Savings in Million Dollars per Year ($M/yr) Associated with Evaluated Water Management Practices. Cost savings include costs associated with reduced water demand and wastewater production.

SUMMARY

While reclaimed water reuse has the potential to result in very large reduction on urban water demand, it is not necessarily the most cost effective practice to achieve such savings. Graywater reuse, indoor conservation and irrigation conservation are the most cost effective practices to achieve water savings in areas with similar climatic and hydrologic conditions to Fort Collins, CO. Onsite graywater reuse can be implemented with very simple and low cost systems. When cost is not a consideration, reclaimed water is a very effective means to reduce water demands in urban areas. An improved version of IUWM is currently under development which includes infrastructure costs and also more sophisticated modeling of irrigation conservation options.

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LESSONS FROM SMALL IRRIGATION DAM FAILURES

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ABSTRACT

The Upper West Region of Ghana provides a valuable case study on small earth dam failures. Numerous government, international, and community sponsored programs have constructed 229 small earth dams in the region, with mixed results on dam safety. Small dam failures were investigated through visits to fifteen breached dams and a review of historical records on over 80 other dam failures. Ninety seven dam failures were documented between 1920 and 2002 and 24 percent of the dams in the region were breached in 2002. Dam failures occur almost every year, requiring resources for repair of these dams rather than construction of new dams. The primary cause of failures is inadequate spillway capacity, and 40 percent of the dams lack a clearly defined spillway. Other causes of failure include poor embankment compaction, poor fill materials, lack of key trench, incomplete construction, poor maintenance, and crocodile burrows. Several trends were identified in failed dams and recommendations are provided for preventing future failures. Lessons learned from these dam failures are relevant for low-head earth dams, non-jurisdictional dams, farmer and ranch constructed dams, and small dams constructed by government programs in the United States.

INTRODUCTION

This paper provides recommendations for reducing small irrigation dam failures based on data collected and lessons learned in the Upper West Region of Ghana in West Africa. The Upper West Region experiences an annual ritual of dam failures almost every rainy season. Ninety-seven dam failures were documented between 1920 and 2002, and 24 percent of the dams in the region were breached in 2002. This paper provides a critical review of historical data, current practices, and the status of 229 existing dams in the Upper West Region. The paper has the following objectives:

- Identify the causes and modes of dam failures
- Characterize trends in failed dams
- Recommend measures to reduce failure rates

WATER RESOURCES IN GHANA

Ghana is situated on the west coast of Africa with a total area of 238,540 square kilometers. It shares borders with Côte d'Ivoire to the west, Burkina Faso to the north, Togo to the east, and the Atlantic Ocean to the south. Ghana has a warm, humid climate. Mean annual rainfall is estimated to be 1.2 meters, but varies throughout the country. Irrigation is required in most parts of the country to provide year round cropping. Some areas, such as the southern portion of the country, only require supplemental irrigation for a few months, while northern Ghana has a distinct dry season with little rainfall. This

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paper focuses on the Upper West Region of Ghana which covers about 18,500 square kilometers in the northwest part of the country.

Ghana is not self-sufficient in food production, and it has been difficult to ensure food availability in sufficient quantities year round. Development of small dams and irrigation schemes has and will continue to expand food production. However, the development of formal irrigation is comparatively recent in Ghana. The first scheme was initiated in the early 1960's and additional projects have been added through the present. Numerous small earth dams have been constructed to support these projects, but many of the dams have failed and numerous lessons can be learned from these failures. Lessons learned from these dam failures are relevant to low-head earth dams, non-jurisdictional dams, farmer and ranch constructed dams, and small dams constructed by government programs in the United States.

IMPORTANCE OF SMALL DAMS

Small irrigation dams are an important economic asset to many rural communities. They can provide numerous benefits including:

- Irrigation water supply
- Livestock watering
- Domestic water supply
- Industrial water supply
- Fishing
- Flood control
- Groundwater recharge
- Access across river valleys
- Recreation
- Small scale-hydropower

Communities have historically settled near valley bottoms where water could be obtained from streams in the wet season, or groundwater and ponds in the dry season. In Ghana, as populations expanded and new villages were established, the groundwater and ponds provided insufficient water sources. Many ponds were constructed before 1940 and later upgraded to dams or dugouts. Hence, dams have become an important alternative for meeting water needs.

When a dam fails the aforementioned benefits are lost, crops and property are damaged, reconstruction costs are incurred, and lives can be lost. Most small communities in Ghana believe that organizations governing dams have a general fund to promptly repair the failed dam. In reality, dams are rehabilitated as part of large regional or national programs that extend for five or more years. Consequently, rehabilitation is often delayed ten years or longer, until funding becomes available. Furthermore, the high number of dams presently breached has created a long waiting list. Poverty and malnutrition can increase in communities after a local dam failure. This emphasizes the

need to prevent dam failures and help ensure their benefits are realized without interruption.

DAMS IN THE UPPER WEST REGION OF GHANA

Most dams in the Upper West Region are homogenous earth-fill embankments with heights from 1.5 to 7.5 meters. Some dams may include zoned earthfill. The number of dams, dugouts, and documented failures in the region are shown in (Table 1).

Table 1. Dams and Dugouts in Upper West Region

		Historical Failures Documented**		**Breached in 2002**	
Structure	**No.**	**No.**	**%**	**No.**	%
Dams	62	26	42	12	19
Dugouts*	167	57	34	43	26
Total	229	83	36	55	24

* A dugout is defined as a small dam with a height less than five meters or reservoir smaller than three hectares

** 97 dam failures have occurred involving 83 dams

Records show that five percent of dams have failed more than once, and one dam has failed at least four times. The total number of failures is likely greater than shown in (Table 1) since historical information is limited for many dams and dugouts.

The life expectancy of dams and dugouts in the region determined from failure rates and average age at failure is about 30 years. Thirty years is generally considered a short life for an earth dam. Earth dams should be constructed with design lives of at least 50 years. Moreover, the design life should designate the period between each major *rehabilitation*, and not between each failure. Some dams constructed in the region in the 1930's have never failed and are in good condition. These projects illustrate the potential lifespan for dams that are properly constructed and maintained.

Typically, five or six dams breach each year in the region. Repairing the breached dams diverts resources from constructing new dams or rehabilitating other dams to prevent new failures. Therefore, constructing new dams has been constrained until the failure rates at existing dams are reduced.

TRENDS IN FAILED DAMS

The information in *An Inventory of Dams for the Upper West Region* (Ministry of Food and Agriculture, 1999) was reviewed to identify trends in failed dams. This was supplemented with information collected on site visits to 15 failed dams in the region. Following are discussions on trends related to the funding source and construction method.

Funding Source

Donors for dam construction include Non-Governmental Organizations (NGOs), Government of Ghana, foreign governments, local communities, and private donors. The dam failure rate is approximately the same for all donor categories, except NGOs, which have a slightly higher failure rate.

Dams sponsored by NGOs often lack the structured guidelines for government or international lending institution. As a result, the projects are sometimes supervised by personnel with little to no technical expertise, and thus have higher failure rates. Funding from NGOs is always welcome by communities, but there is a need for NGOs to solicit qualified consultants or government engineers when the required expertise is lacking in their organization.

Intervention by domestic or foreign governments does not appear to provide dams with lower failure potential. Dams constructed under community control have comparable failure rates. Although these dams are often lacking in sophistication, the community's interest clearly plays a role in sustaining quality. Better construction monitoring and maintenance have been observed on community sponsored projects. Conversely, communities show limited interest in government sponsored projects because they perceive the government as owner of the dam and consider the government responsible for future maintenance.

A combination of government intervention and community participation is the preferable alternative. Some contemporary projects require that communities file an application (to demonstrate initiative and interest) and contribute some of the construction cost (5%-10%). Often the cost share can be in the form of in-kind labor if the community lacks financial resources. The government provides technical assistance and the remainder of funding, which would otherwise be beyond the financial capacity of the community.

Construction Method

Many dams were constructed with manual labor in the 1930's to 1960's. Statistics show that manually constructed dams have higher failure rates than dams constructed with machinery. This is likely due to the lower compactive effort applied to embankment materials. Manually constructed dams are compacted by human or livestock treading, while machinery constructed dams are compacted with heavy compactors, bulldozers, or trucks. In Ghana few dams with the possible exception of small dugouts, are currently constructed with manual labor. Manually constructed dams were built on average about 60 years ago, compared to about 25 years ago for dams constructed by machinery. Therefore, manually constructed dams present a higher risk, both due to their age and poor compaction. Subsequently, these dams should receive priority in dam rehabilitation programs.

CAUSES OF DAM FAILURES

Dams can fail for numerous reasons. The common causes in the Upper West Region are shown in (Table 2).

Table 2. Causes for Dam Failures in Upper West Region (1920-2002)

Cause	No.	%
Inadequate Spillway Capacity	15	43%
Poor Embankment Compaction (piping)	6	17%
Incomplete Construction	5	14%
No Key Trench (piping)	3	9%
Poor Fill Material (piping)	3	9%
Crocodile Hole	1	3%
Other	2	6%
Total	35	100%

The precise failure mode is only known for 35 of 97 documented dam failures. However, observations made during recent inspections and discussions with communities at over 25 dam sites tend to confirm the information in (Table 2). Namely, spillways are often undersized and embankments frequently overtop, key trenches are lacking in most designs, and embankment construction often does not include proper fill or compaction.

These results are compatible with other studies on earth dam failures. Jia et al. (2007) collected data on 593 earth dam failures in over fifty countries, including the United States. They concluded that dam failures usually occur within five years of construction. Inadequate spillway capacity was identified as the largest problem and accounted for 36% of failures (versus 43% in Ghana). Erosion and piping failures caused by poor quality construction were the cause of 43% of the failures (versus 35% in Ghana).

The National Dam Performance Program (http://npdp.stanford.edu/) collected data on the causes of dam failures in the United State between 1975 and 2001. The results are shown in Figure 1.

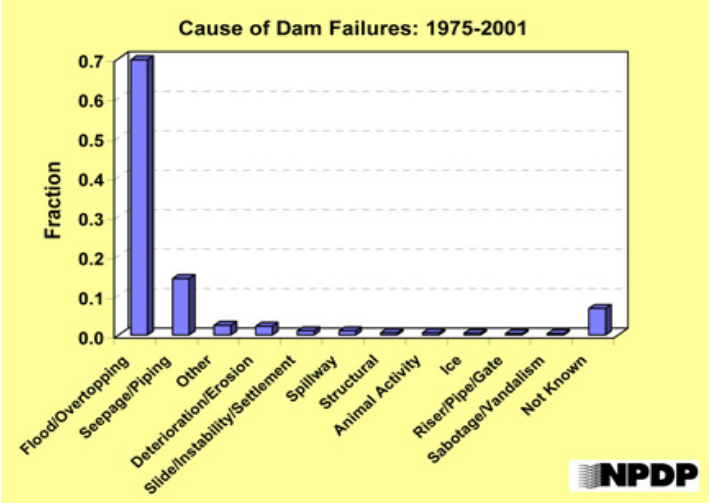

Figure 1. Dam Failures in United States from 1975-2001
Source: National Performance of Dams Program (2013)

This graph shows that overtopping is the primary cause and seepage/piping is the second most common cause of failure. These results also agree closely with the data collected in Ghana and the aforementioned paper by Jia et al.

MULTIPLE DAM FAILURES

The sudden surge of water generated by a dam failure can often exceed the maximum flood expected along a river or stream each year. When one dam fails, the surge may be powerful enough to destroy another downstream dam. A chain reaction can cause several dams to fail in a single storm.

In one instance, a new dam was constructed only 500 meters downstream of an older dam. The older dam was in poor condition, had no clearly defined spillway, and frequently overtopped in the rainy season. Failure of the old dam presented a considerable risk to the new dam. The new dam project should have incorporated the rehabilitation or removal of the older dam. Prior to constructing a new dam or raising an existing dam, engineers must consider the influences from upstream or downstream dams. This illustrates the importance of an updated and accurate inventory of dams.

MEASURES TO REDUCE DAM FAILURES

The high incidence of dam failures in Ghana required prompt action to alleviate the problem. A discussion of methods identified for reducing the frequency of dam failures follows.

Increase Spillway Capacity

Inadequate spillway capacity is the leading cause of dam failures worldwide (Moffat et al. 1996). It is also the primary cause of dam failures in the Upper West Region of

Ghana. An undersized spillway will compel floodwater to rise above the dam crest, overtop and washout the embankment. Spillways are invariably needed on any dam or dugout. Over 95% of dams in the region have spillways, but 55% of the dugouts lack a clearly defined spillway. In the United States small dam spillways are designed for the 100 to 500-year storm, or even larger events. Formerly, flood return periods as low as 10 years were used on dams in Ghana. Usually, the incremental cost for spillway expansion is not directly proportional to the increase in capacity. Significant capacity increases can often be realized with only moderate expenditure increases.

Proper Embankment Compaction

Proper embankment compaction is required to reduce permeability and increase erosion resistance. The importance of compaction is accentuated in Design of Small Dams (2):

> "*The consequences of ignoring construction control are exemplified by the large number of earthfill dams built in the United States during the .first quarter of this century that did not survive the first filling of the reservoir. Records show that most of the dams were constructed without moistening the soil and without applying special compactive effort*." (page 641)

This statement shows that dam construction in the United States was once a trial and error process. Ghana still appears to be in this phase. However, some small dam failures still occur in the United States, and the lessons learned from Ghanaian dam failures can help engineers in both countries to improve dam design and construction.

Proper compaction requires placing, watering, and compacting fill in layers with vibratory or sheepsfoot rollers. If water is not locally available it must be imported. Performing construction during or right after a wet season can help ensure that the soil has some moisture. However, construction during the wet season can have detrimental consequences. One dam failed from poor compaction after rainwater over-saturated the fill material.

Fill materials should be clay or clay loam. These materials can be compacted to provide a lower permeability and offer good resistance to erosion. Figure 2 shows a dam constructed with sandy soils that began to erode within one week of construction.

Figure 2. Erosion on New Dam Constructed with Sandy Soils

Bulldozers alone should not be used to construct dams because they fail to compact soil effectively. Many dams in the region were constructed with bulldozers, but retain little water and have high failure rates. Dam contracts need to be awarded to qualified contractors with tipper trucks and compactors. There have been cases where bulldozer operators, pretending to be qualified contractors, have secretly won dam construction contracts. In addition, in the early 1990's a Ghanaian government agency rented bulldozers to communities and individuals at subsidized rates to construct dams. In the process the agency was promoting an affordable, but inferior, method of dam construction.

Several cases are also documented for dams that were constructed in less than one week. Based on the size of the dams the contractors could not have properly compacted layers of fill properly in this time period. These dams have high seepage, high failure rates, and experience significant settlement, because the foundation was not properly constructed.

Provide Key Trench

A key trench is excavated in the dam foundation and backfilled with impervious material. Key trenches reduce seepage and migration of foundation materials, which can cause failures. Few communities recall seeing a key trench constructed under their dams, and even recent designs in the region have lacked key trenches. In one case, a key trench was excavated in a sandy foundation and later backfilled with sand. The key trench did nothing to reduce seepage and the dam leaked heavily after construction was completed.

Figure 3 shows a breached dam in the Upper West Region. The original construction did not include a key trench. Like many dam failures in the region, it failed along the natural streambed where sandy soils are present. The dam was later reconstructed with a clay key trench.

Figure 3. Breach in Earth Dam

Use Proper Fill Materials

Proper fill materials are needed to achieve optimal density, impermeability, and erosion resistance. Most dams in the region are homogenous embankments, which require fill with high clay content. Contractors typically excavate materials adjacent to the dam and rarely haul material more than one kilometer. This can greatly limit the potential quality of fill materials. The characteristic fill materials at 23 dam sites are shown on Figure 4. Few of the dams contained fill material desired for homogenous dams (clay and clay loam). Figure 4 also indicates that dams with lower clay contents have higher failure rates.

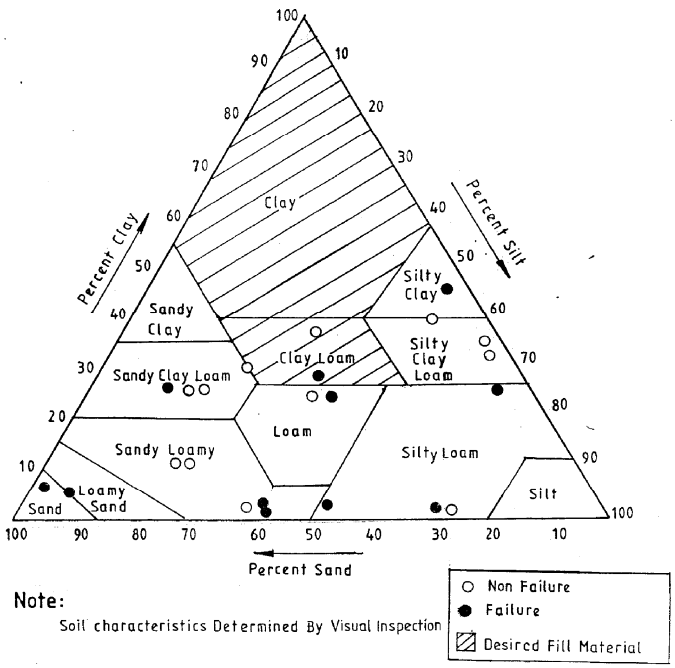

Figure 4. Dam Fill Materials on 23 Projects in Ghana

The problem can be rectified by performing geotechnical investigations prior to construction to identify the location and quantity of suitable materials. Many areas in the Upper West Region have a thin surficial clay layer overlying sand. Inspection of the surface gives the illusion that clay is plentiful, but sand is eventually used to construct the majority of the dam. Many dams have been improperly sited in areas without suitable clay. These sites not only provide poor dam construction materials, but also have reservoir soils with high infiltration rates. Hence, the dams store little to no water.

Maximum haul distances for fill are sometimes stipulated in the bill of quantities, but are usually less than one kilometer. Several cases have occurred where proper fill materials were not available within the maximum haul distance. In these cases, the contractor cannot be blamed for inferior fill materials or resulting failures . In conclusion, dams must be located only after subsurface investigations confirm the presence of suitable fill materials, or sufficient funding is available to haul material to the site.

Maintenance

Maintenance is particularly important for earth dams because they are constantly subject to erosion. However, poor maintenance has been observed at most dams in the Upper West Region. Routine maintenance is lacking just as the country is lacking a maintenance culture. In most cases the community has inherited responsibility for maintaining the dam, but they often expect the government to fulfill that responsibility. Proper maintenance also requires adequate training, which was also lacking in the Upper West Region.

Trees and large shrubs should not be planted or allowed to grow on earth dams. Roots can cause seepage paths and lead to piping failures. Fallen trees that are uprooted can create a low point and reduce freeboard on a dam. The number of dam failures from trees is not large in the Upper West Region or in the United States, but they still present a risk to dam safety.

Burrowing Animals

Crocodiles are found in most water bodies in the Upper West Region. Crocodiles can dig deep burrows in dam embankments for shelter and to lay eggs. These burrows can be large, can extend through embankments, and have caused at least one dam failure in the Upper West Region. Crocodiles can be removed through hunting, but this is not acceptable to some Ghanaian communities who consider crocodiles sacred and part of their heritage. These communities need to be educated on the potential risks of crocodiles living in their reservoirs. Monitor lizards also dig burrows, but they are smaller than crocodile burrows.

In the United States alligators and burrowing rodents pose problems at earth dams. Alligator burrows are well documented at alligator farms. Borrowing animals such as squirrels and gophers are a problem at many earth dams. Their burrows are narrow but can be deep and extend through a dam, creating a pathway for water and cause a piping failure.

Burrowing animals can be controlled by riprap, fencing placed flat on the dam surface, backfilling burrows, hunting, baiting and relocation. Riprap should include large angular rocks that interlock to improve stability and reduce movement. Rounded boulders or flat foliated boulders should not be used.

Ensure Completeness of Construction

Some dam failures are attributed to incomplete construction. In these cases the dam was not completed before the rainy season, or more often it was abandoned before all of its features were properly installed. Many of these dams do not reach their design crest level and overtopped during small storms. The following circumstances have led to incomplete construction:

- Poor enforcement of contract deadlines
- Poor quality control monitoring
- Failure to measure levels with proper geodetic equipment
- Contractors credit problems
- Political instability

Solicit Expertise from Qualified Dam Engineers

Qualified engineers should be contacted to address any dam failure or major repair. Several cases are documented of communities repairing breached dams that failed again

in less than a year. The community's interest in repairing the dams is commendable. However, a dam failure indicates a serious problem and a qualified engineer should be consulted. In another case, a community removed a spillway beam to lower the spillway crest and increase spillway capacity. Without the beam the spillway eroded to the base of the dam, which is now essentially breached. Proper training in dam inspection and maintenance and knowledge on when to consult a qualified engineer can prevent similar occurrences. The author met many qualified Ghanaian engineers with expertise in small dams. However, their services are often not used or their designs are not followed during construction.

CONCLUSIONS AND RECOMMENDATIONS

Dam failures in the Upper West Region of Ghana have reached an unacceptably high level. Dams are an important economic asset to the region. The impacts from dam failures in terms of crop damage, property damage, lost benefits, and reconstruction costs are significant. Measures are needed to prevent further failures and construct new dams with lower failure potential. Emphasis on short-term savings has resulted in dam construction with high failure potential. Many dam failures occur when standard guidelines for dam design and construction are not followed.

The primary causes of failure are inadequate spillway capacity and piping due to poor fill material or construction. These conclusions are similar to the findings in other dam failure studies. Recommended measures for reducing dam failures include spillway expansion, proper embankment compaction, key trenches, use of proper fill materials, dam maintenance, burrowing animal management, ensuring completeness of construction, and using qualified technical personnel.

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FLOOD 2010 AND ITS DISASTROUS EFFECTS IN SINDH, PAKISTAN

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Ehsan-ul-Haque Leghari[2]
Fateh M Marri[3]

ABSTRACT

Sindh is the second largest province of Pakistan. The peaks of Upper Indus Basin (UIB) and synchronization of western and eastern rivers flow peaks were found one of the major causes of super flood 2010 in Lower Indus Basin (LIB) from Guddu Barrage to Sea. The peaks at UIB were observed 32 percent from glacier melt, 40 percent snowmelt and 28 percent rainwater.

Flow data analysis of LIB from last eight decades (1932-2012) categorize flood patterns as 15 percent super floods, 16 percent very high floods, and 69 percent high to normal floods. But, the distinct feature of flood 2010 found was the peak flow which was generated in 4 days, while the peak flows of other super floods of last 80 years were taking averagely 25-30 days. Thus, in flood 2010, two major breaches occurred one at upstream of Sukkur Barrage and another downstream of Kotri Barrage. Both breaches left remarkable distinctiveness for flood management managers/experts.

The continued 96 hours peak flows, rapid runoff to the rivers and human intervention and encroachments along the river catchment and absence of any flood management plan became the major causes of devastating effect in the province of Sindh. As a result about 199 people lost their lives, 2.55 million acres of irrigated agriculture damaged, 2,444 villages destroyed, about 1.07 million houses damaged and 0.825 million people affected

The detailed investigation has concluded that to manage such unprecedented floods and minimize the losses a thorough investigation is required to determine the blockage of natural flood waterways and human settlements which are common in river catchment and to develop an integrated flood management plan, which should have close relationship between: flood plain management, water resource management, river management, land use management, agriculture and Environment within a basin.

INTRODUCTION

Floods are blessing and disguise. The catastrophic floods 2010 demonstrate, the Indus is both friend and foe. It recharged groundwater, replenished the wetlands, and rehabilitated river flood plain; supported agriculture and fisheries thereby helped livelihoods. But at the same time it produced severe adverse impacts on the economy and safety of people where around 199 people died, 2.55 million acres agriculture crops damaged, around

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2,444 villages destroyed and more than 1.07 million houses partially and completely damaged, and also more than 0.825 million people and lot livestock suffered in the province of Sindh.

Sindh is bounded to the west by the Indus River and Balochistan province to the north by Punjab province and to the south by the Arabian Sea. Sindh province of Pakistan comprises of 23 administrative units "called districts" (Figure 1.) The population of the province is now more than 30 million people which is more than double as compared to a few decades back. The natural drainage and manmade infrastructure were standards and requirements applicable at the time. But from last few decades intense developments for human settlement and blockage of natural waterways by constructing the communication network in the province were become the major causes of flood disaster.

In flood 2010, on the right side of upper part of Sindh including Kashmore-Kandhkot, Jacobabad, Shikarpur, Kambar-Shahdadkot and Dadu districts and on the left side of lower Sindh including Tando Mohammad Khan, Badin and Thatta districts were severely affected. These districts were affected due to breaches occurred at upper part of Sindh upstream of Sukkur Barrage at Kashmore and another breach occurred at lower part of Sindh below Kotri Barrage at Tando Moammad Khan.

Figure 1. Map of Sindh Province along with Districts.

The floods 2010 began in late July 2010, resulting from heavy monsoon rains in Pakistan which affected the Indus River basin and was described as the worst in the last eight

decades. Approximately one-fifth of Pakistan's total land area was underwater (Figure 2).

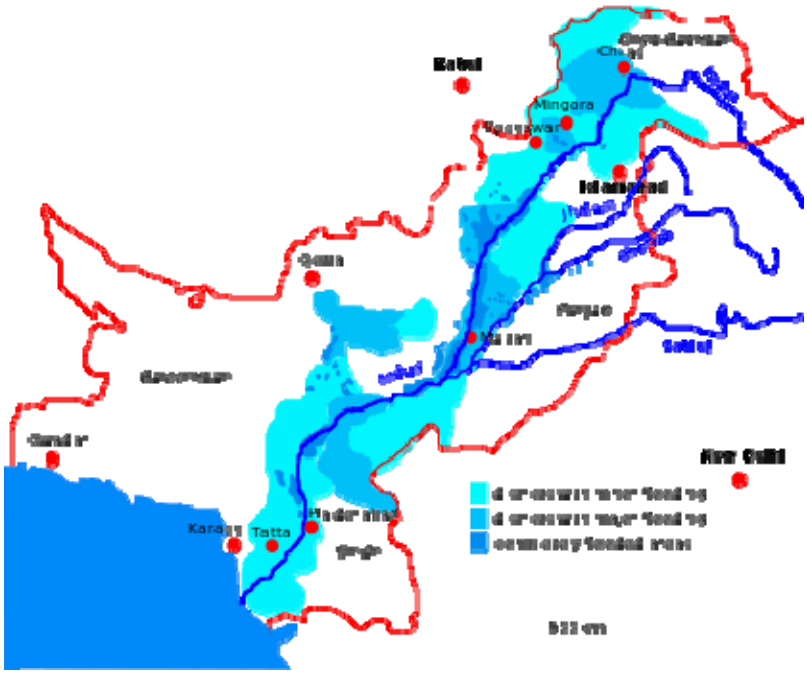

Figure 2. Flood Affected Areas of Pakistan.

The Indus System is largely fed by the snow and glaciers of the Himalayas, Karakoram and the Hindokush ranges of Tibet, Jammu, Kashmir and the northern areas of Pakistan. The flow of the river is determined by the seasons. It diminishes greatly in winter, while flooding its banks in the monsoon months from July to September. The peak stream flows start generating in July in Upper Indus Basin (UIB) constitutes 32 percent glacier melt 40 percent snow melt and 28 percent rain water which causes major floods in the Basin. The combination of UIB flow and the synchronization of peaks from Western and Eastern rivers cause extremely high floods in the Lower Indus Basin (Khero *et al* 2013).

Literature review mentions that in recent years there has been a worldwide increasing frequency of major flood events and it is predicted that these trends will continue. The forecasts estimate that by 2100, climate change could increase river flows by 20 percent. Due to extreme weather events it is found that between 1990 and 2006 there was an annual average of over 7500 deaths worldwide due to flooding [Loster (1999), Werritty *et al., (*2002), Goklany (2007)]. It is known fact that increased flooding is not just a consequence of climate change, but there are established links to human mismanagement of rivers and their catchments. Drainage and land management have reduced the capacity for catchments to hold stormwater and release runoff slowly, producing flashy flow regimes in the river systems (Schneidergruber *et al*., 2004). Historically, most developed countries have adopted heavily engineered solutions to flooding problems whereas developing countries tend to lack the economic resource needed to create and maintain

them. Natural flood regimes are almost absent in the rivers of industrialized countries as a result of the re-engineering of waterways. Developing countries tend to have rivers with natural flow regimes and they take a more basic approach looking at the source of the problem and how to live with rivers. This fundamental dichotomy in approaches to flood management is described at length [Bayley, (1995) Ogtrop *et al.*(2005)].

KEY FINDINGS OF FLOOD 2010

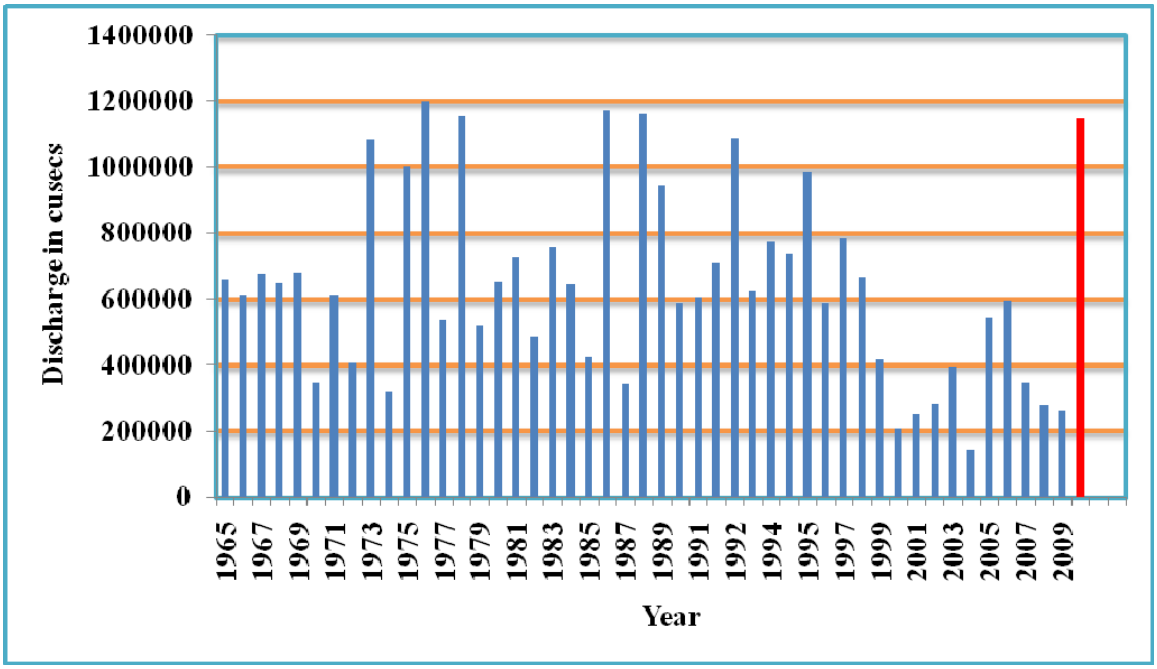

Figure 3. Flood Trends of Last Five Decades at Guddu Barrage of Sindh, Pakistan.

Flood Categories

The flow of water was recorded by the provincial irrigation department at Guddu Barrage, which is the first barrage of Sindh province of Pakistan. The flow data depicted in (Figure 3) has categorized flood patterns of last eighty years and found the flood trend as 15 percent super floods, 16 percent very high floods, and 69 percent high to normal floods so as the flood 2010 lies in super category. Further, data indicates that from 1965 to 1972 no super flood occurred. From 1973 to 1978, within six years four times super floods were recorded in the province of Sindh. Again there has been an era (1986 to 1997) of super floods where 5-6 times super floods were occurred. Then after thirteen years a super flood of 2010 occurred which left lot of miseries. During the period of 1998 to 2010, population has reached to about 30 million therefore, lot of development of road infrastructure projects were undertaken in the province. But no due consideration was given to the design where many natural waterways were not kept free to drain out the storm water and at the same time people have tried to occupy the waterways areas. Consequently, runoff of storm water significantly stopped for several days and destroyed the livelihood of the community and paralyzed the population of the province.

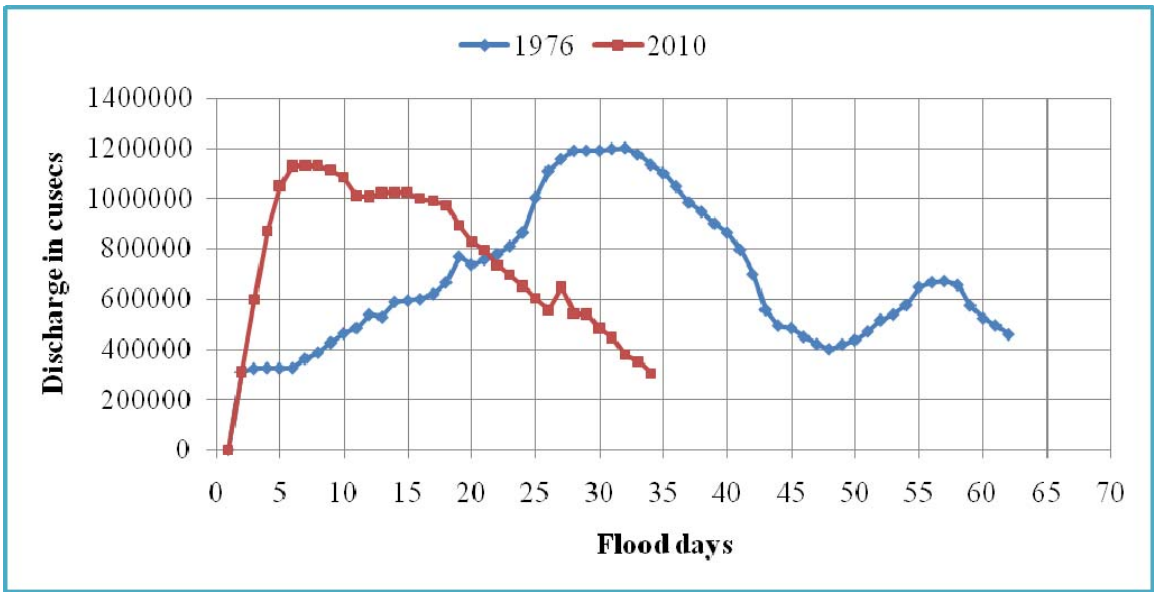

Figure 4. Distinctive Feature of Two Super Floods at Guddu Barrage in Sindh, Pakistan.

Flood Distinction

The trend of flood rise at Guddu barrage was recorded from last eight decades by the Irrigation Department, Government of Sindh. From the data it was found that in all super floods in Sindh, the rise of peak flows was almost following the gradual pattern of rise in water flows as of flood 1976 except flood 2010 (Figure 4). It can be seen from the figure that the flood 2010 has distinct feature of rise of peak flows which was generated in only 4 days, while the peak flows of 1976 took 25-30 days. Thus, in flood 2010, two major breaches were occurred one at upstream of Sukkur Barrage and another at downstream of Kotri Barrage. Both breaches left remarkable distinctiveness for flood management managers and experts.

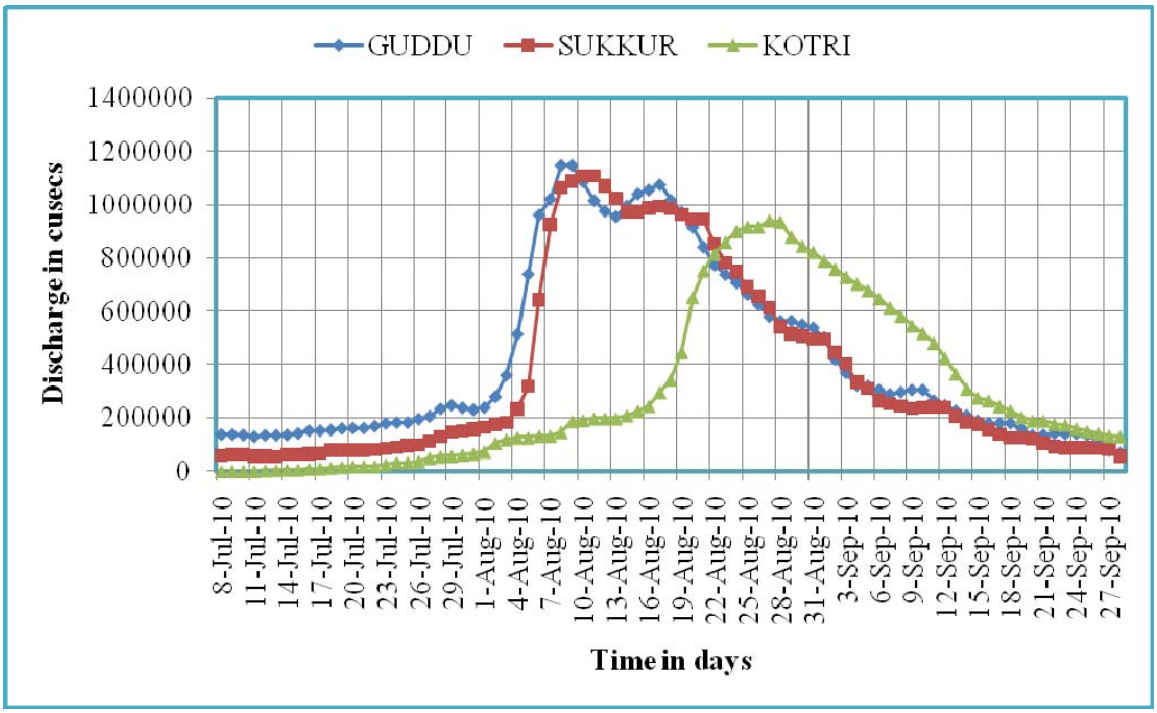

Figure 5. Floods Travel Time from Guddu to Sukkur to Kotri Barrages of Sindh.

Guddu to Sukkur barrages the distance is about 120 kilometers and the distance from Sukkur to Kotri barrages is about 300 kilometers. Generally, in the past, the travel time of super floods between Guddu and Sukkur barrages was 4-5 days and from Sukkur to Kotri barrages was 10-12 days. But in flood 2010, the travel time from Guddu to Sukkur was less than three days and from Sukkur to Kotri barrages was more than 25 days (Figure 5). The situation analysis have indicated that there has been flow contribution from the rain fall between Guddu and Sukkur barrages and therefore the travel time decreased and peak flows at both barrages were almost of same quantity, whereas increase of travel time between Sukkur and Kotri barrages was due to obstruction in natural waterways. Because more than four crossing bridges have been constructed and huge road network has been built in the floodplain between Sukkur and Kotri barrages.

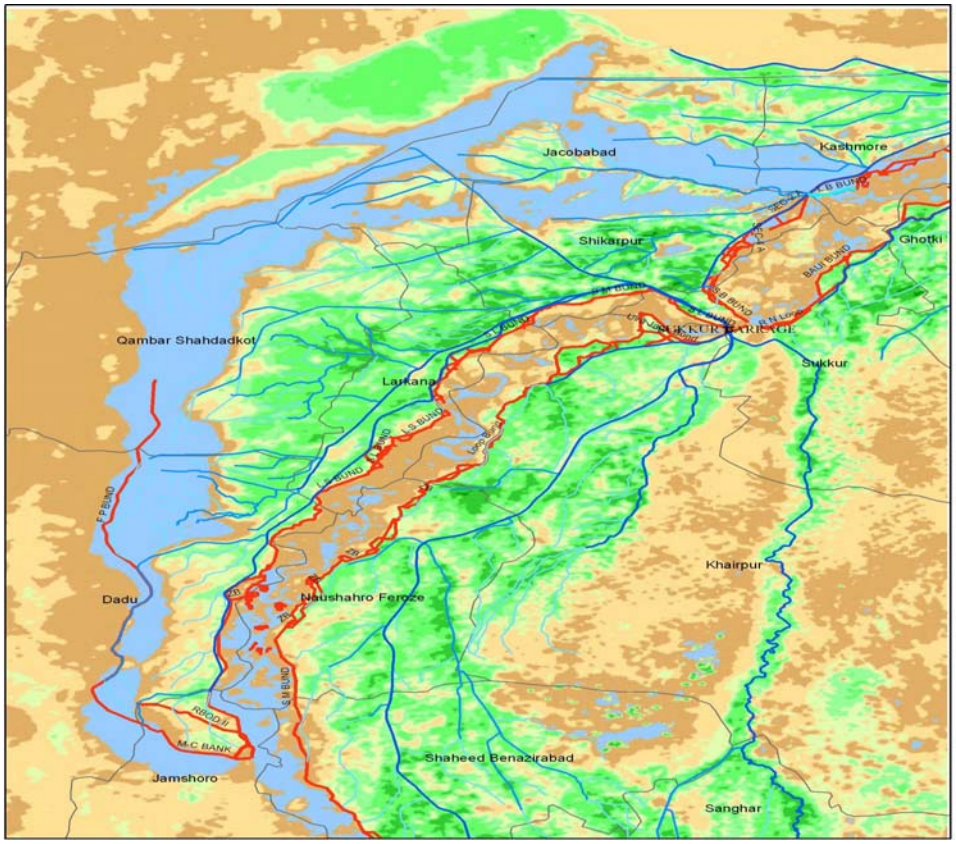

Figure 6. Movement of Flood Water from Breach at Upstream of Sukkur Barrage.

(Figure 6) The satellite images were collected which show the flood water was flowing from the breach at upstream of Sukkur Barrage and moving towards south-west and then and after traveling some kilometers then moved back to south-east direction and then discharged into the river Indus at Sehwan city upstream of Kotri Barrage. This was because of upslope (east to west) of the country. After merging it into river Indus it put pressure on right side below Kotri Barrage and thereby the second breach took place.

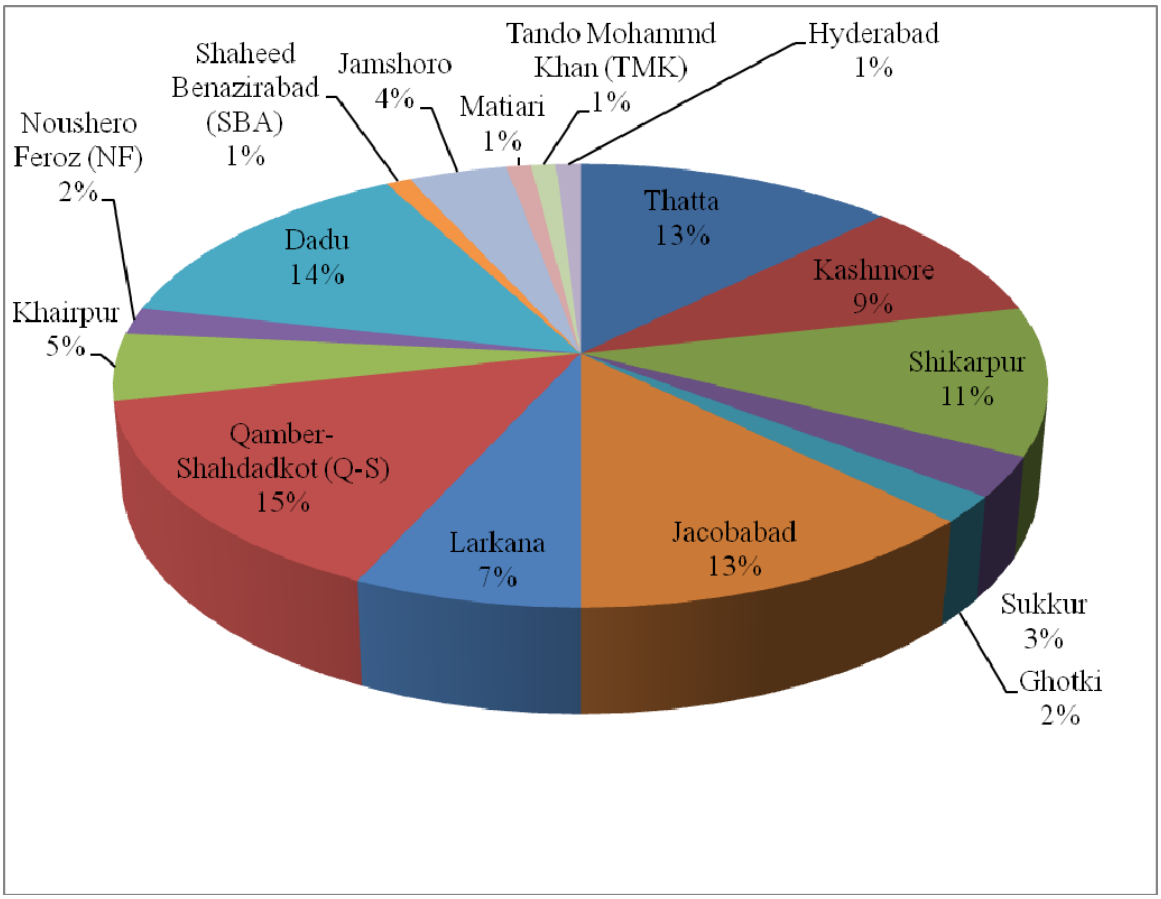

Figure 7. People Affected in each District of Sindh in Flood 2010.

(Figure 7) Due to these two breaches, mentioned above, the five districts on right side including Kashmore, Shikarpur, Jacoband, Qamber-Shahdad Kot and Dadu and one district on left side i.e., Thatta downstream of Kotri Barrage were more affected. Almost three-fourth population of the province was suffered only in these six districts. This all effect was due to water standing for several days not only because of water moving forward and then back but also due to blockage of natural water ways due to construction of road net work and illegal occupation of natural waterways areas by the people.

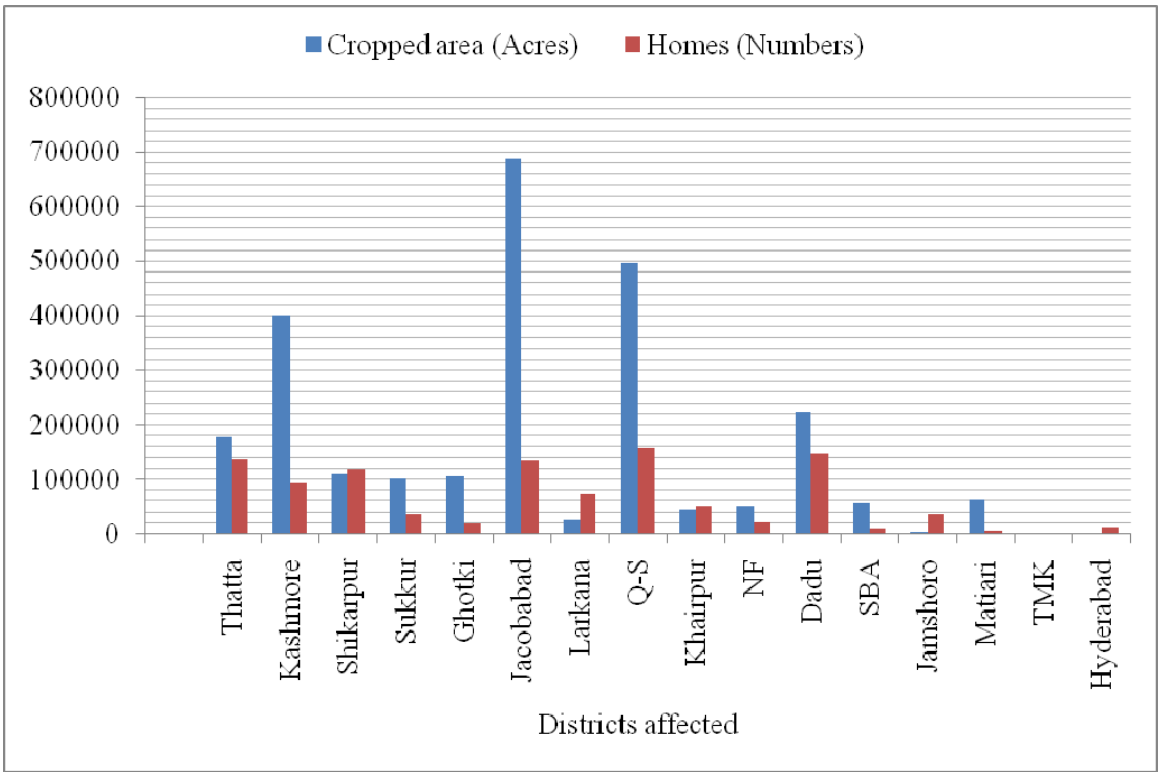

Figure 8. Agriculture Loss and House Damages in flood 2010 in Sindh Province of Pakistan.

(Figure 8) Similarly, damages to cropped area and houses were recorded and found that the districts Jacobabad, QS, Kashmore Dadu and Thatta were affected more . However, the major crop losses took place in jacoabad, kashmore, QS and dadu districts. District Thatta was also affected district. Again these losses are attributed to mismanagement, absence of flood management plan, insufficient drainage network and illegal occupation of waterways areas. One of the major reason of more losses observed was poor and lack of administrative coordination. Also it was noted that there were no coordinated and collaborative approaches to manage the flood.

CONCLUSIONS

Floods 2010 was unprecedented and flash flood. The peak flood flows generated in very short period of time of 4-5 days and continued up to 96 hours. Thus, rapid runoff to the river system and prolonged rainfall in the catchment areas lead to longer and high peaks at Guddu Barrage. The encroachment of river flood plain, blockage of waterways, human settlements and absence of comprehensive floodplain management plan were become the major causes of disasters in the province of Sindh. It is concluded that if, similar type of floods comes in future – and that is uncertain –the flood misery for the people of Sindh and Pakistan may be more.

SUGGESTIONS

In order to cope up the flood disasters and flood miseries there is a need of comprehensive flood management plan which should include three dimensions; ecology, economy and technical. Therefore, flood management plan requires flood mapping and social understanding; climate change impact, regional flood problems, community education, flood awareness and preparation and integrated flood management framework .This plan should have a close relation between: water resource management, river management, land use management, forest management, agriculture, environment and ecology. Flood 2010 experiences have recognized that no single organization and no single approach can deliver an effective response to flood management issues, thus a coordinated and collaborative approaches are required.

ACKNOWLEDGMENTS

Authors are highly thankful for the Irrigation Department Government of Sindh, Provincial Disaster Management Authority, Sindh Irrigation and Drainage Authority (SIDA) and local NGOs for providing the data and allowed to use for scientific purposes.

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WILDFIRES AND FOREST HEALTH — COLORADO-BIG THOMPSON PROJECT

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ABSTRACT

The Colorado-Big Thompson Project (C-BT) and the associated Windy Gap Project deliver over 230,000 acre-feet of water annually to supplement municipal, agricultural and industrial water supplies for 860,000 people and 640,000 acres of irrigated land in Northern Colorado. Water is diverted from the Colorado River Basin to the South Platte Basin through a system of 12 reservoirs with a total storage capacity of nearly 1 million acre-feet, 35 miles of tunnels, 95 miles of canals, three pumping plants and six hydroelectric power plants with an installed capacity of 216 megawatts. The Northern Colorado Water Conservancy District (Northern Water) was created in 1937 to contract with the U.S. Bureau of Reclamation (Reclamation) for construction and repayment of project facilities, and jointly operates and maintains C-BT with Reclamation.

C-BT water supplies are nearly entirely dependent upon snowmelt from high elevation watersheds along the Continental Divide in Northern Colorado. Forest health and fires within these watersheds can have dramatic effects on the quality of watershed runoff and the ability of C-BT water supplies to meet municipal, industrial and agricultural water uses. Catastrophic wildfires that occurred in Northern Colorado during 2012-2013 drought conditions highlighted the risk that C-BT water supplies face given deteriorated forest health conditions, drought, and urbanization at the wildland-urban interface. Northern Water, in conjunction with its partner local, State and Federal agencies are taking a pro-active approach to addressing these conditions, including actions to protect water supplies from recent wildfires, as well as initiating the C-BT Headwaters Partnership, which will develop a plan and program to address forest health conditions in C-BT watersheds and pre-plan post-wildfire response in preparation for potential future wildfires.

INTRODUCTION AND BACKGROUND

C-BT is Colorado's largest transmountain diversion project. The project was authorized as a federal reclamation project by Congress on June 24, 1937 with the approval of Senate Document 80 (Reclamation, 1937). The project was built from 1938 to 1957 with a final construction cost of $164 million, and provides supplemental water to 32 cities and towns and more than 100 ditch and reservoir systems. C-BT is designed to collect and deliver up to 310,000 acre-feet of water annually from the Upper Colorado River Basin. It transports up to

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550 cubic feet per second (cfs) of water to the Front Range via the 13.1-mile Alva B. Adams tunnel beneath the Continental Divide. Total storage of the C-BT system (not including Green Mountain Reservoir, which is used for water and power obligations on the West Slope) is approximately 845,000 acre-feet. The largest reservoirs are Lake Granby (540,000 acre-feet), Horsetooth Reservoir (157,000 acre-feet) and Carter Lake (112,000 acre-feet).

The Windy Gap Project consists of a diversion dam on the Colorado River, a 445-acre-foot reservoir, a pumping plant, and a six-mile pipeline to Lake Granby. Windy Gap water is pumped and stored in Lake Granby before it is delivered to water users via the Colorado-Big Thompson Project's East Slope distribution system.

Northern Water is a public agency created in 1937 under Colorado's Water Conservancy Act (Colorado Revised Statutes § 37-45-101 et seq.). The boundaries encompass nearly 1.5 million acres in portions of Boulder, Broomfield, Larimer, Logan, Morgan, Sedgwick, Washington and Weld counties (Figure 1). Water assessments and ad valorem taxes form Northern Water's primary revenue base. The Municipal Subdistrict is a separate and independent conservancy district that constructed and operates the Windy Gap Project. The Municipal Subdistrict Board directors are the same as the Northern Water Board.

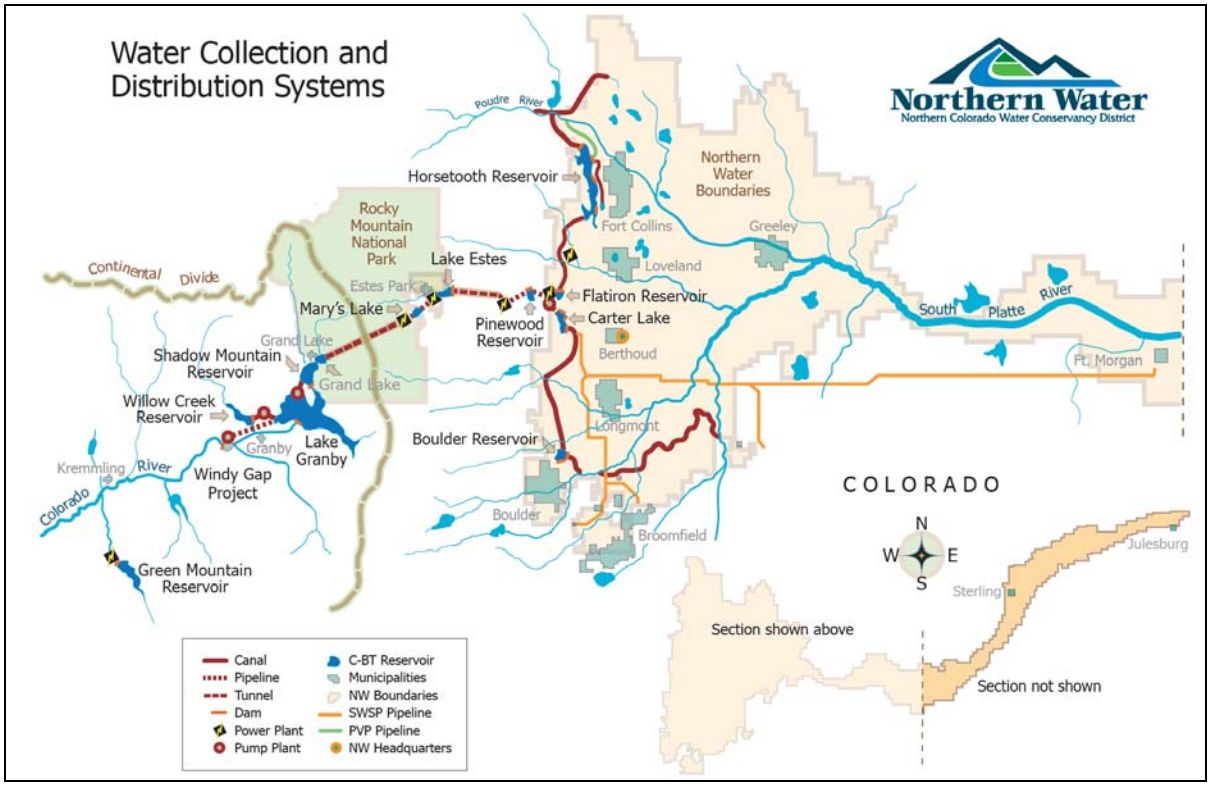

Figure 1. C-BT and Northern Water Location Map

West Slope Watersheds

Snowmelt from the watersheds of the West Slope Collection System comprises more than 80 percent of the water supply yield for the C-BT and Windy Gap projects. Watersheds range in elevation from nearly 13,600 feet along the continental divide to the east and north, to 8,100

feet at Willow Creek Reservoir and 7,800 feet at Windy Gap Reservoir. Unlike many large water supply agencies, neither Northern Water nor Reclamation own significant portions of the C-BT watersheds. Approximately ¼ of the West Slope watersheds are within Rocky Mountain National Park, and ½ are within the Arapaho National Forest (Figure 2). Northern Water and the Municipal Sub-District own approximately 3,300 acres (less than one percent of the West Slope watersheds) around Willow Creek Reservoir and Windy Gap facilities, while Reclamation owns land immediately surrounding its facilities.

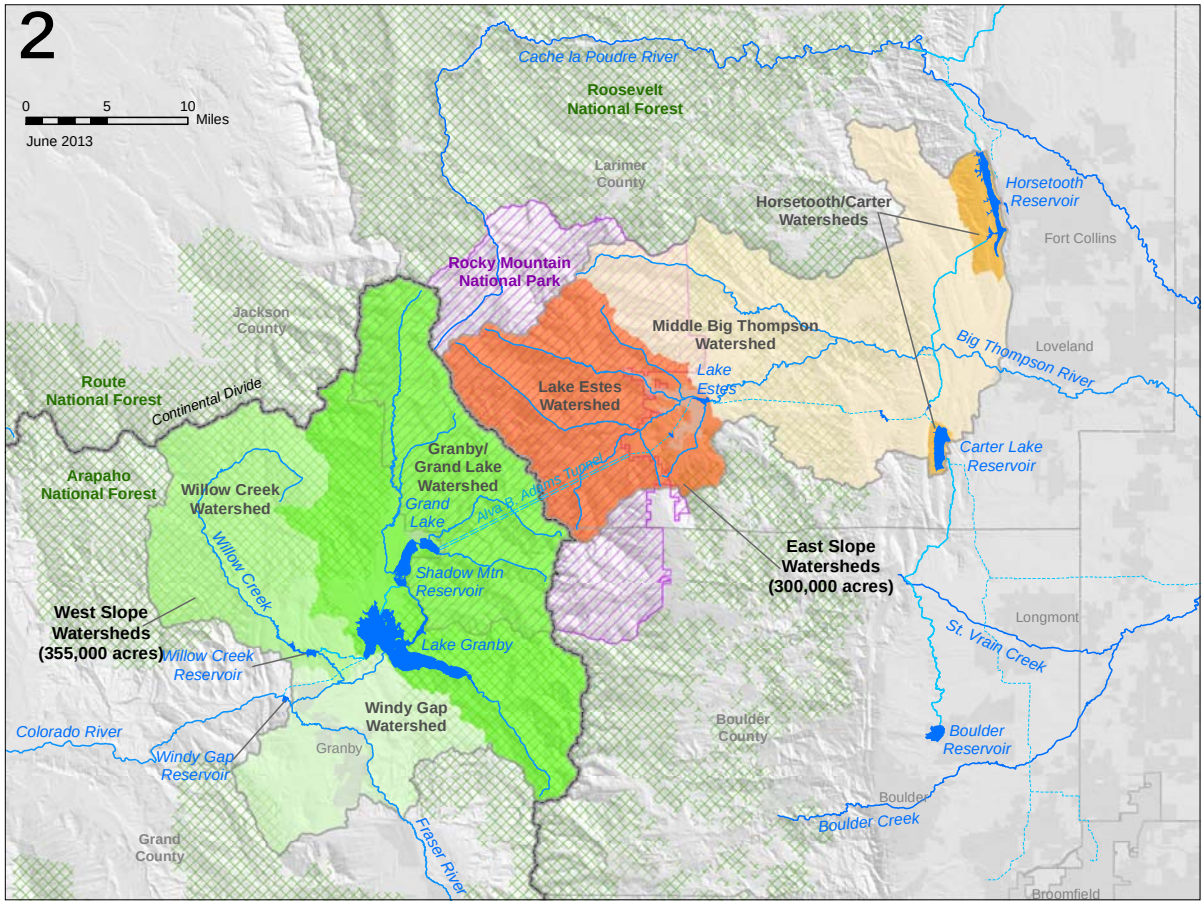

Figure 2. C-BT West Slope and East Slope Watersheds

West Slope watersheds are characterized by alpine and sub-alpine vegetation zones (Lugo et al. 1999), with large expanses of even-aged lodgepole pine at the lower elevations of the sub-alpine zone surrounding and immediately upstream of C-BT facilities (Table 1). Forest health has been severely affected by the mountain pine beetle epidemic, resulting in mortality of approximately 90 percent of mature lodgepole pine (Grand County, 2008). The epidemic began on the West Slope in the late 1990's, with most tree mortality occurring between 2000 and 2009 (U.S. Forest Service 1997-2012). Most of the lodgepole pine forests on the West Slope are in the gray stage, which occurs about 2 to 3 years after initial infestation once needles fall off the trees, but before the time when trees begin falling to the forest floor. The grey stage is expected to last 4 to 10 years (Schoennagel 2012), and presents the optimum time to perform forest health treatments as there can be some value left in the lumber, and access and logging activities have not been affected by downed timber.

Table 1. Vegetation Types in C-BT Watersheds

Vegetation Type	West Slope		East Slope		Total	
	(acres)	(%)	(acres)	(%)	(acres)	(%)
Lodgepole	128,000	36%	51,000	17%	179,000	27%
Ponderosa	9,000	3%	90,000	30%	99,000	15%
Spruce-Fir	90,000	25%	28,000	9%	118,000	18%
Other Forested	12,000	3%	21,000	7%	33,000	5%
Alpine	42,000	12%	29,000	10%	71,000	11%
Other	74,000	21%	81,000	27%	155,000	24%
Total	355,000		300,000		655,001	

East Slope Watersheds

Although East Slope water supplies only constitute a small amount of the overall C-BT yield, these watersheds are extremely important to C-BT because these streams intermingle with C-BT water, primarily at Lake Estes, Dille Tunnel diversions in the lower Big Thompson Basin, and from watersheds that surround terminal reservoir facilities. Watersheds range in elevation from 14,250 feet on Long's Peak near Estes Park to about 5,400 feet at Horsetooth Reservoir. As on the West Slope, Northern Water and Reclamation do not own appreciable land in the East Slope watersheds. Approximately ⅓ of East Slope watersheds are within Rocky Mountain National Park, and ¼ are within the Roosevelt National Forest (**Error! Reference source not found.**).

Because of larger variability in elevation and precipitation, East Slope watersheds are characterized by alpine, sub-alpine, upper montane, lower montane, and lower ecotone vegetation zones, with associated variability in vegetation types (Kaufmann et al. 2006). The dominant forest type is ponderosa pine (**Error! Reference source not found.**) and occurs throughout the elevation range surrounding and immediately upstream of C-BT facilities. Forest health has also been severely affected by the mountain pine beetle epidemic. However, since the epidemic generally proceeded from west to east through Northern Colorado forests, the epidemic is younger and still on-going within the East Slope watersheds (Colorado State Forest Service 2012). A similar level of tree mortality in East Slope lodgepole pine forests is expected as on the West Slope; it is possible that ponderosa pine mortality may be less severe. Beetle-impacted forests on the East Slope are a mix between the initial, red stage within the foothills areas to the early grey stages at higher elevations.

IMMEDIATE CONCERNS: WILDFIRE IN C-BT WATERSHEDS

As a result of extreme weather conditions and declining forest health, 2012 produced a devastating wildfire season throughout Colorado. In Northern Colorado, the 2012 fire season included the second largest wildfire in Colorado's recorded history (at the time), as well as a rare late-season high-elevation wildfire in Rocky Mountain National Park. The 2013 wildfire season didn't start much better, with a quick-moving wildfire during March and another high-elevation early-season fire in Rocky Mountain National Park. The lessons learned about the landscape's physical response to these fires as well as the actions and coordination

needed during and after these fires has shaped development of Northern Water and the C-BT Headwaters Partnership wildfire plan and program.

Recent Fires

Fires have always been a part of the landscape within the low and high-elevation watersheds of the C-BT system. However, as with the rest of Colorado, there has been an increase in both the number and size of fires over the last two decades. **Error! Reference source not found.** presents a summary of the number of fires and acreage burned on state and private land in Colorado since 1960 (Colorado State Forest Service 2013, Edwards 2013). Although this data does not include federal land, the trends are indicative of overall fire behavior within the state.

The reasons for increases in wildfire starts and size are reflected by changes in the three dominant factors that influence fire (vegetation characteristics, fire weather climatology, and ignition patterns), all of which vary in importance spatially and temporally (Moritz et al. 2012). The single most important factor affecting wildfire, especially at higher elevations, is weather and climate (Veblen et al. 2012). Colorado has been in an extended drought since the early 2000's, and indeed, the most destructive statewide fires occurred in the severe drought years of 2002 and 2012. Increased tree densities (resulting not only from fire exclusion policies, but also early 20th century logging, grazing and timber practices), especially in lower elevation ponderosa pine forests where many of the larger recent fires occurred, have increased fuel loads beyond natural levels (Kaufmann et al. 2006). Ignitions include both natural and man-made causes, and the number of fires in Figure 3 is well correlated with increases in Colorado population and associated wildland urban interface population.

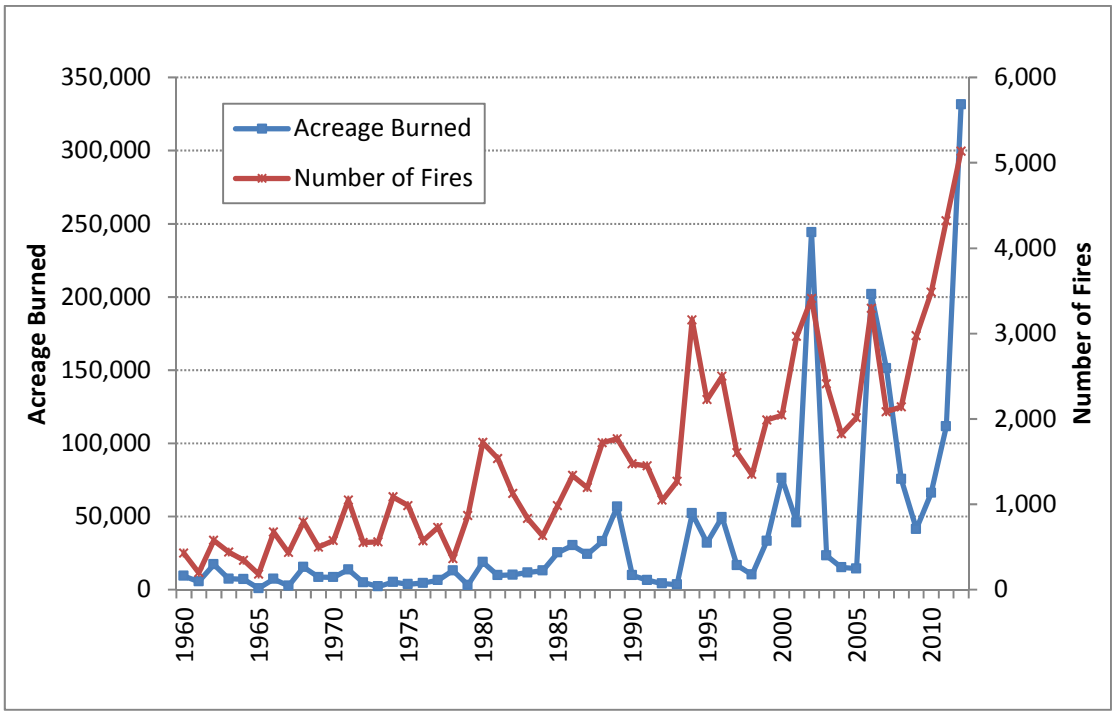

Figure 3. Colorado Wildfires on State and Private Lands (Colorado State Forest Service 2013, Edwards 2013)

Several fires have burned within or adjacent to critical C-BT watersheds (i.e. watersheds that are direct water supplies for the project, or watersheds immediately upstream of key reservoirs) within recent years (Figure 4). The following describes the major fires that have affected C-BT watersheds:

- High Park Fire. The High Park Fire burned more than 87,200 acres in Larimer County during summer 2012, killing one person and destroying 259 homes. The fire burned approximately 410 acres within the Horsetooth Reservoir watershed – the remainder of the fire was adjacent to main C-BT water supply watersheds (approximately 8,465 acres burned in the Redstone Creek watershed, which is considered a C-BT watershed but is not a main water supply watershed). The fire burned primarily in upper and lower montane forests, with predominantly mature lodgepole pine and a relatively closed canopy at mid and higher elevations, and mixed-conifer forests containing lodgepole pine, ponderosa pine, and Douglas-fir at lower elevations. Nearly 50 percent of the area burned at moderate or high intensity (U.S. Forest Service 2012).

- Fern Lake Fire. The Fern Lake Fire began in October 2012 in the higher elevations of Rocky Mountain National Park in the Big Thompson watershed. Under a full-suppression strategy since it began, the fire remained in inaccessible terrain until late November, when 70-mile-per-hour winds pushed the fire east into Moraine Park, more than doubling its size. The fire burned about 3,498 acres, and is located in the sub-alpine lodgepole and mixed conifer forests of the Lake Estes watershed.

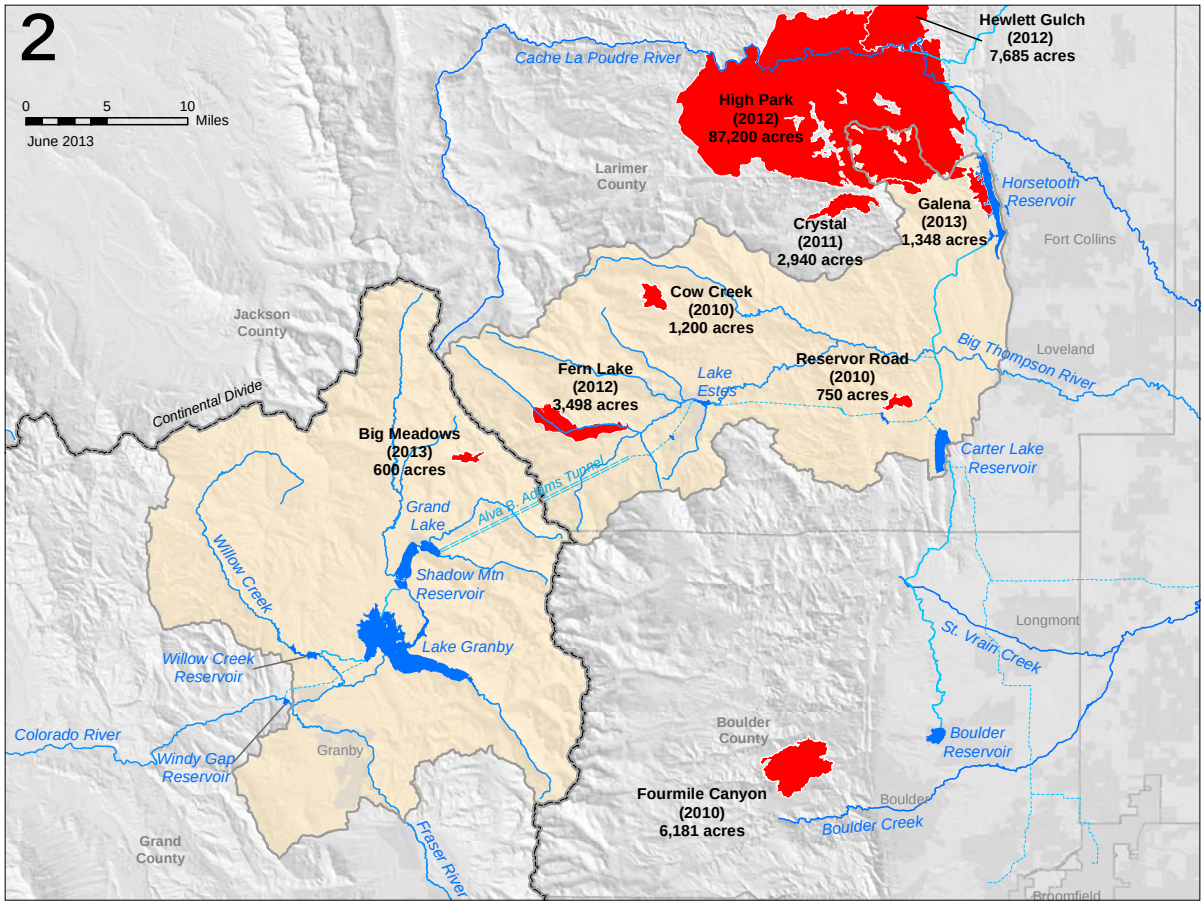

Figure 4. Recent Major Fires In and Near C-BT Watersheds

- Galena Fire. In March 2013, the Galena Fire burned about 1,348 acres directly west of Horsetooth Reservoir in Lory State Park and Horsetooth Mountain Park. This fire was immediately south of the southeast corner of the High Park Fire, and located in the lower montane ponderosa pine forests and grasslands of the Horsetooth Reservoir watershed. The fire burned down to the reservoir high water line in several places.

- Big Meadows Fire. The Big Meadows fire is currently burning in Rocky Mountain National Park. The fire began in June 2013 and has burned slightly more than 600 acres at the time of this paper. The fire is located in the Grand Lake watershed on the West Slope.

The Hewlett Gulch fire, located at the northeast boundary of the High Park fire, burned approximately 7,685 acres during the spring of 2012. Although this fire had significant effects on other municipal water supplies in the Poudre River Basin, including those for the cities of Fort Collins and Greeley, it did not have a direct impact on C-BT water supplies.

Fire Response

Northern Water's responses to the recent fires were focused in three main areas: direct mitigation of land and facilities affected by the fires, water quality monitoring, and evaluating opportunities for operational changes to address potential poor quality runoff.

- Direct Mitigation: funded seeding and mulching efforts to revegetate and reduce erosion; installed and maintain temporary wattles in tributary channels, and constructed sediment basins to capture sediment and debris in drainages upstream of reservoirs; cleaned sediment from facilities; and, planned for installation of floating debris booms in reservoirs and tributary channels to capture and remove larger floating debris from reservoirs.

- Water Quality Monitoring: Coordinated and installed water quality monitoring to provide real-time information on water quality conditions, primarily turbidity, to water users through a web-based application; and, developed baseline water quality sampling in conjunction with the USGS and municipalities to study rainfall and runoff processes as a result of the fires.

- Operational Investigations: Considered options to reduce or restrict imports of water from the West Slope to the East Slope for short durations following events that cause short-term adverse water quality conditions.

Because of its limited impact on C-BT water supplies, the mitigation efforts by Northern Water were only a small part of the overall mitigation efforts for the High Park and Hewlett Gulch fires. Numerous federal, state and local public agencies, non-profit organizations and land owners are investing millions of dollars and volunteer hours to address the long-term recovery of the High Park burn area. Mitigation efforts include reseeding, aerial and hand placement of straw and wood mulch, replanting of trees, directional felling of logs in drainages, sedimentation basins, culvert replacement, increased water quality monitoring and general debris cleanup.

Effects of Fires

The primary effects of fires on water supply systems are changes in post-fire hydrology and water quality. Post-fire hydrology varies by fire severity, and is characterized by loss of protective surface cover and surface roughness; hydrophobic (water repellent) soil conditions; a shift from sub-surface to overland flow, and increases in runoff and erosion rates. Post-fire water quality is characterized by increased stream turbidity and suspended sediments, increased total organic carbon, volatilization of nitrogen, large phosphorous flux from soil erosion and sediment transport, and oxidation of heavy metals in soils and plants (Stednick 2013).

Of the numerous fires, the High Park Fire has had the most significant effects on regional water supplies. Normal-intensity rainfall events and spring runoff conditions have flushed sediment and fire debris into the Poudre River, a major source of drinking water for the cities of Fort Collins and Greeley and surrounding areas. The water suppliers become nearly

entirely dependent on Horsetooth Reservoir water supplies immediately following these types of events, including for several months following the fire. It is expected that over the next several years until the watershed fully recovers, these types of events will continue to result in periods of highly turbid runoff resulting in increased reliance on Horsetooth Reservoir and C-BT water supplies.

Irrigators were also affected by the fire, primarily from ash, sediment and debris at diversions and in canals. The Munroe Gravity Canal diversion, a part of the C-BT project that delivers irrigation water to the North Poudre Irrigation Company, was completely clogged with sediment and debris following a rainstorm over parts of the High Park Burn area in 2013 (one year following the fire; Figure 5). This event interrupted deliveries through the Munroe Gravity Canal for approximately 10 days. Fortunately, in-system reservoir storage was available to supply most irrigators canal during the outage. Northern Water assisted the North Poudre Irrigation Company in the excavation of about 1,000 cubic yards of debris from the diversion structure and downstream tunnel, and further structural repairs may be required.

Additionally, reports were received regarding clogging of center-pivot sprinkler nozzles and clogging/interruption of irrigation siphon tubes (Gertig 2013), and there are many locations where tributary drainages flowing are passed over or under irrigation ditches and canals that could be adversely impacted by increases in runoff (U.S. Forest Service 2012).

The most significant regional effect of the High Park Fire on irrigators may be changes in temporary water supply availability from municipalities. Typically, municipalities execute annual rentals with irrigators for C-BT and other water supplies that are in excess of what they can use in a given year. However, partially due to uncertainties regarding Poudre River runoff as a result of the High Park Fire and partially due to current drought conditions, most cities will not have excess water available to rent in 2013. Coupled with the drought conditions, late season water supplies are expected to be lower than normal in 2013, and have affected cropping and irrigation patterns throughout Northern Water boundaries.

Figure 5. Before and After Photos of Debris Flow at Munroe Gravity Canal Diversion

Initial assessments by the National Park Service indicate that both the Fern Lake Fire and the Big Meadows fire burned at low intensity, and that natural filtering of runoff through large meadows downstream of each fire will help reduce sediment and debris in the water. Further evaluation is currently being completed by the National Park Service.

The Galena Fire occurred during a time of year that allowed substantial vegetative recovery prior to summer thunderstorm events. Wattles and sediment basins installed by Northern Water were effective in reducing ash and sediment runoff into Horsetooth Reservoir during spring snowmelt runoff and lighter precipitation events in the early summer.

LOOKING AHEAD: C-BT HEADWATERS PARTNERSHIP

The 2012 fire season was a wakeup call to water supply entities in Northern Colorado that watershed health is paramount to delivering clean usable water supplies to water users in Northern Colorado. This, along with other Colorado water suppliers' and agency experiences in responding to wildfire and forest health planning, led to establishment of the C-BT Headwaters Partnership. The partnership was created in 2012 through a Memorandum of Understanding between the U.S. Forest Service, Colorado State Forest Service, Reclamation and Northern Water. The goal of the partnership is to proactively restore forest and watershed health, and to pre-plan post-wildfire response to protect C-BT infrastructure and water supplies through the following efforts to be conducted by the partnership:

- Conduct forest and watershed health treatments, and pre-plan post-wildfire response
- Develop a 5-year operating plan specifying treatment zones and activities

- Support creation and refinement of watershed assessments
- Coordinate to provide education, technical and financial incentives
- Engage other partners
- Develop a shared communications and media campaign

As the C-BT Headwaters Partnership is in its infancy, the remainder of this section will discuss on-going planning activities and targeted treatment areas for the early stages of the program, including watershed assessments, opportunities and constraints analysis, and funding. In addition to the initial planning-level work, a monitoring plan will be developed for both forest health treatments and post-wildfire activities to ensure that all activities will have a meaningful effect on forest health, water quality, and wildfire mitigation.

Watershed Assessment

The primary planning document being developed under the partnership is a wildfire watershed assessment and post-wildfire plan. The plan will update and build upon the Upper Colorado Headwaters Watershed Assessment (JW Associates 2013) and the Big Thompson Wildfire/Watershed Assessment (JW Associates 2011). The watershed assessments are "designed to identify and prioritize sixth level [USGS] watersheds based upon their hazards of generating flooding, debris flows and increased sediment yields following wildfires that could have impacts on water supplies" (JW Associates 2011). The wildfire watershed assessment and post-wildfire plan follow procedures outlined in the Colorado Watershed Protection Data Refinement Work Group (2009). The watershed assessments include a stakeholder process that includes representatives from water providers; federal, state and local land management agencies; counties; towns and other interested groups.

The C-BT wildfire watershed assessment considers four components in evaluating watershed conditions, including wildfire hazard, flooding or debris flow hazard, soil erodibility and water supply. The following describes the general ranking process for these components. Some variance from these procedures was necessary to account for special situations that fall outside of the general bounds used to develop the methods.

- Wildfire Hazard. Wildfire hazard was developed using the Fire Behavior Assessment Tool, which is an interface between ArcMap and FlamMap (U.S. Forest Service, 2013). Certain input spatial data sets were updated and adjusted based on mountain pine beetle mortality conditions as described in 2002-2012 aerial detection surveys (USGS, 1997-2012). The flame length results were divided into five categories of wildfire hazard, with a formula based on the percent of watershed in the highest flame length categories used to develop the final wildfire hazard ranking.

- Flooding or Debris Flow Hazard. Flooding and debris flow hazard was calculated as the road density ranking plus twice the ruggedness ranking. Watershed ruggedness, calculated as the product of basin height and the inverse square root of basin area, is an indicator of the relative sensitivity to debris flow (Cannon and Reneau 2000), with higher more rugged watersheds having a higher sensitivity to debris flow (Melton 1957). Road density, in miles of road per square mile of watershed area, was used as

an indicator of the tendency for roads to convert subsurface runoff to surface runoff and route the surface to stream channels, increasing peak flows (Megan and Kidd 1972, Ice 1985 and Swanson et al. 1987).

- Soil Erodibility: High severity wildfires can dramatically change runoff and erosion processes in watersheds. Water and sediment yields may increase as more of the forest floor is consumed (Wells et al. 1979, Robichaud and Waldrop 1994, Soto et al. 1994, Neary et al. 2005, and Moody et al. 2008) and soil properties are altered by soil heating (Hungerford et al. 1991). As a measure of soil susceptibility to erosion, standard K factors from U.S. Department of Agriculture STATSGO and SSURGCO data sets and land slope derived from USGS 30-meter digital elevation models were combined with a slope grid using Natural Resources Conservation Service (1997) slope-soil relationships to create erosion hazard ranking.

- Water Supply: For purposes of C-BT watersheds, the location and importance of water supply and power infrastructure is an important aspect in the overall watershed ranking. Water supply and power infrastructure was prioritized based on its operational importance and flexibility. For instance, systems such as the Windy Gap Project and Willow Creek Reservoir and Pump Canal were ranked lower because these systems can be temporarily shut down with little effect on water supply yield. In-line reservoirs and terminal reservoirs were ranked higher, as there is little ability to temporarily disrupt operations without having effects on yield. A similar approach was used for power generation and transmission facilities.

The final step in watershed prioritization was developing zones-of-concern for the C-BT and Windy Gap systems. The zones-of-concern recognize that the immediate watersheds above important surface water intakes, upstream diversion points and reservoirs have higher potential to contribute significant sediment or debris following wildfire events. Based on Colorado Revised Statute § 31-15-707 et seq., which allows municipal water providers to enact ordinances within five miles upstream of water intakes to protect water supplies, 14 primary zones-of-concern were developed 5 miles upstream of water sources total nearly 156,000 acres. Based on experiences with previous fires and subsequent flooding events in the Upper South Platte, several zones-of-concern were extended to 11 miles upstream of water sources (or where they encounter watershed divides) for an additional 159,000 acres.

The C-BT Headwaters Partnership is now developing a small-scale watershed prioritization that will guide watershed protection projects. The highest priority watersheds are anticipated to generally lie upstream of Grand Lake, Shadow Mountain Reservoir and Lake Granby on the West Slope, and upstream of Lake Estes on the East Slope. A map showing the C-BT watershed zones-of-concern and preliminary watershed prioritization is presented in Figure 6. Preliminary prioritizations are subject to change as the study continues moving forward.

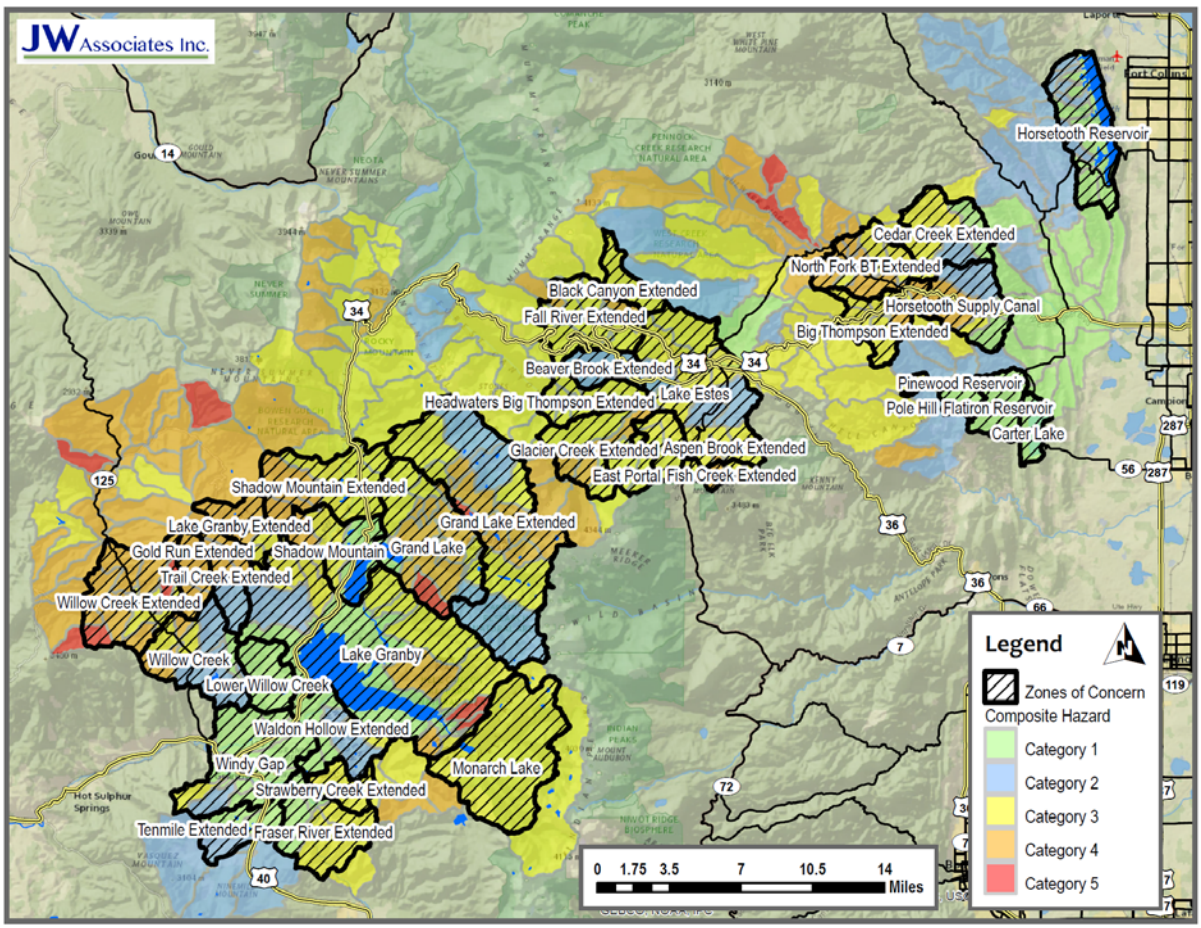

Figure 6. C-BT Watershed Zones of Concern and Preliminary Watershed Prioritization

Opportunities and Constraints

The opportunities and constraints analysis identifies potential opportunities for specific watershed treatments in all of the zones-of-concern developed in the watershed assessment phase. Four major constraints limit the ability to reasonably conduct forest treatments, including land ownership, land management, access and slopes.

- Land Ownership: Much of the high priority watershed areas lie within Rocky Mountain National Park, and will be treated according to the Long-Term Fuels Treatment Plan for Rocky Mountain National Park (National Park Service 2012). This plan generally identifies fuels treatments projects "due to their proximity to values at risk". The plan identifies approximately 28,000 acres of treatments primarily located along the eastern park boundary surrounding Estes Park, and the western park boundary near Grand Lake and the North Fork of the Colorado River.

- Land Management: Treatment activities are limited within wilderness areas, Upper Tier roadless areas and U.S. Forest Service special protection areas due to law and agency policies.

- Access: Access to specific areas within a watershed is a key factor in determining the opportunities for mitigation wildfire hazards or the ability to install, operate and maintain erosion and sediment control structures. Traditional logging and thinning operations are best suited within ¼ to ½ miles of roads. Specialized logging equipment can be used to move logs and other products as far as 2 miles or more depending upon terrain. If products do not require removal to meet treatment requirements and can be masticated, equipment can be moved from further distances.

- Slopes: Land slopes are a major constraint when considering the locations and types of treatments that can be conducted. Although some equipment exists to perform treatments on nearly all slopes (including tracked mastication equipment and helicopters), a slope of 40 percent was used to identify reasonable maximum slope for which treatments would be targeted.

As previously discussed, several factors influence the potential for wildfires to occur and have adverse effects on water supplies, including vegetation type, physical characteristics and health; ground slope, aspect and soil types and the potential for erosion; and weather conditions. The primary means for influencing fire behavior is modifying the vegetation conditions that fuel wildfires. It is not the intention of this influence to completely prevent wildfires, but rather to influence the size, intensity and movement of these wildfires, especially in areas that have physical characteristics that could have detrimental effects to water quality should they burn intensely.

To guide opportunities for vegetation modification, it is important to understand the nature of the different forest types within the watersheds.

- Ponderosa Pine - Lower elevation ponderosa pine forests, the dominant forest type in the lower montane zones of the East Slope watersheds, are the most vulnerable parts of the forest due to their naturally short fire return interval (sometimes less than 30 years), greater impacts from human use, settlement and historical fire exclusion. These types of forests provide the best opportunity to improve forest sustainability by performing treatments that return and emphasize characteristics of pre-settlement conditions.

- Aspen – Aspen occurs throughout the montane and sub-alpine zones. It is an aggressive invader to disturbed areas, and is more resistant to fire because crown fires will seldom carry through this forest type except for extreme drought combined with windy conditions, primarily during fall periods. For these reasons, Aspen is considered a good species to maintain or promote within the landscape (JWA 2011).

- Lodgepole Pine - Lodgepole pine, the dominant forest type in the sub-alpine zones in the West Slope watersheds, are found in dense, continuous stands. Lodgepole pine has a longer natural fire interval (150-300 years), and fires that do occur are typically large scale stand replacing fires. The dense forest types are not a substantial departure from pre-settlement conditions (Veblen et al., 2012). However, as the mature trees begin to fall as a result of mountain pine beetle or other diseases, they create large

fuel loads on the forest floor, which could result in hot, fast moving crown fires that are difficult to contain. Treatments in these areas do not target broad-scale fuel treatments, but rather a more focused response of hazard tree removal, fuels reduction near the wildland-urban interface and infrastructure, and creation of diversity at the stand and landscape levels by clear-cutting, patch cutting, permanent openings, or converting areas to aspen (JWA 2011).

- Spruce-Fir – Spruce-fir, a mixture of Engelmann Spruce, Colorado blue spruce, subalpine fire, and other minor species, occurs in the sub-alpine zones, especially within the West Slope watersheds. Spruce-fir has a very long natural fire interval (up to 500 to 700 years). However, when it does burn, fires can be intense and cause severe erosion and sedimentation problems. Spruce-fir is difficult to thin within a short period sufficiently to develop enough diversity to reduce wildfire hazards.

Given these opportunities and constraints, approximately 112,000 acres were identified as potential targets for long-term forest treatments as part of the C-BT Headwaters Partnership. Although the ponderosa pine in the montane zones present a higher risk of wildfire during any given year, the effects on C-BT infrastructure from these watersheds is substantially less than the higher subalpine watersheds on the West Slope and East Slope above Lake Estes. Given the vulnerability of West Slope infrastructure and the limited opportunities due to land ownership above Lake Estes, treatments are focused primarily on West Slope watersheds within lodgepole pine forests. Specific treatment locations and types have yet to be defined, and will vary based on land-owner willingness to participate, environmental considerations, economic considerations, and other factors.

Treatments conducted as part of the C-BT Headwaters Partnership will complement thousands of acres already treated within the C-BT zones-of-concern by the U.S. Forest Service, Colorado State Forest Service, National Park Service and private land-owners, and those being conducted by the National Park Service. The partnership is currently developing an overall treatment targets for the forested area within the C-BT zones-of-concern. The amount of treatments actually conducted will primarily be a function of funding available from each partner on an annual basis.

Initial treatments are commencing immediately, with over 100 acres of treatments targeted on lands owned by Northern Water south of Willow Creek Reservoir, and approximately 15-30 acres surrounding critical infrastructure at the West Portal of the Alva B. Adams tunnel along the eastern shores of Grand Lake. The anticipated Willow Creek treatment will consist of the removal of beetle-kill lodgepole pine, and regeneration of aspen. The West Portal treatment will primarily consist of removal of hazardous beetle-kill trees.

Pre-Planning for Post-Wildfire Response

Spurned to action by lessons learned in multiple fires within and near C-BT watersheds, pre-planning post-wildfire responses is a key part of the C-BT Headwaters Partnership. Using results of the watershed assessment analysis, key locations for implementation of post-wildfire mitigation are being identified, and discussed with permitting agencies. Post-wildfire activities may include but are not limited to similar activities described as mitigation efforts

for previous fires. The group is also identifying specifications and suppliers of key materials such as wattles, seed, mulch, floating debris booms, and barge mounted equipment to perform work along reservoirs where land access is not available. Northern Water is procuring some critical materials to have on-hand for emergency situations.

Another key aspect of post-wildfire response is developing communication chains that can be implemented during and after wildfires. The C-BT Headwaters Partnership partners have been able to leverage Northern Water and the Colorado State Forest Service's extensive connections with local water providers and agencies with federal-level communications through the U.S. Forest Service and Reclamation. Sharing up-to-date and accurate information is critical during and following wildfires, especially given social media, which is a normally helpful tool, but can be problematic if inaccurate information is conveyed.

Funding

As with any project of this magnitude carried out over multiple years and multiple agencies, funding becomes a major constraint to implementation of the planning document. At the federal level, this is further complicated by the uncertainties of the annual federal budgeting process. No annual budgets for the implementation program have been developed at this time. Northern Water is currently undertaking a rate study to provide more information on what types of future revenue may be available from its existing revenue sources and assessments, and may be reasonably expected from potential future rate adjustments.

The C-BT Headwaters Partnership has also received a grant through the state-funded Wildfire Risk Reduction Grant program. This program was established by the legislature in Senate Bill 269, and will be administered through the Department of Natural Resources. The program was funded with $9.8 million of general fund dollars, and is focused on projects that reduce the risk for damage to property, infrastructure, and water supplies, and those that limit the likelihood of wildfires spreading into populated areas. The grant includes a monitoring component, which will demonstrate the effectiveness of the treatment projects and the utility of grant resources (Colorado Department of Natural Resources 2013). The C-BT Headwaters Partnership grant award will leverage Northern Water revenue and funding with local treatments on private lands (through hard matches or soft in-kind contributions) to maximize treated acreage within and benefits to C-BT watersheds.

SUMMARY

As with most irrigation and municipal water supply systems in the Western United States, protection of Colorado-Big Thompson Project watersheds from the devastating effects of wildfire has gained increased importance over the last decade as the incidence of wildfire has increased. Both municipal and agricultural water users have expressed an interest in taking a pro-active approach to addressing forest health within the watersheds and pre-planning for post-wildfire response. The C-BT Headwaters Partnership was established with these goals in mind, and provides a coordinated program between local, state and federal land managers and water agencies to address plan, fund and execute these activities. Forest treatments within C-BT watersheds, which were heavily affected by the recent mountain pine beetle epidemic, will be implemented over the next 5 to 10 years.

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WATER QUALITY RELATIONSHIPS AMONG CDOM, PHYSICOCHEMICAL PARAMETERS, AND NUTRIENTS IN AN IRRIGATION CANAL SYSTEM, HIDALGO COUNTY, TEXAS

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ABSTRACT

The water quality of irrigation canals in South Texas has received little attention despite: (1) canals being the only other source of freshwater besides the Rio Grande, and (2) use of the canals by wildlife and fishes. An irrigation canal system in Hidalgo County was investigated to determine whether water quality gradients of chromophoric dissolved organic matter (CDOM), physicochemical parameters, nutrients, and plankton were present. Over a several month period, field measurements included water temperature pH, salinity, electrical conductivity, total dissolved solids, and dissolved oxygen. Laboratory analysis included ammonia, nitrite, nitrate, phosphorus, and turbidity. These measures were correlated, and then analyzed statistically to determine the influence of site location and season. The results of this study are presented and implications for understanding the water quality along an irrigation canal system are discussed.

INTRODUCTION

Dissolved organic matter (DOM) is a major contributor to the carbon cycle of fresh and saltwater ecosystems. When measured as chromophoric or colored dissolved organic matter (CDOM), it consists of organic substances that absorb light in the blue region of the spectrum. CDOM concentrations, produce the yellow, green, and brown colors in river, lake, and ocean waters from its absorption of light and photoinduced chemical reactions (e.g., photo-oxidation) (Shank et al. 2009; Vodacek 1992). CDOM not only contributes to the color of water, but also to the physical and biological processes taking place in waterbodies. More commonly, studied in estuaries, coastal areas, and seas, CDOM relates to the physicochemical measures of electrical conductivity, salinity, pH, and temperature (Bowers and Brett 2008; Ferrari and Dowell 1998; Kostoglidis et al. 2005; Mostofa et al. 2007; Vodacek 1992; Watras et al. 2011).

Relationships among CDOM and physicochemical and nutrient concentrations in shallow waterbodies (e.g., esturaries) are particularly sensitive to the influence of these factors (Gallegos et al. 2005). The color of shallow freshwater streams, rivers, and canals can be a product of phytoplankton (e.g., Chlorophyll *a*), tannin and lignin concentrations from

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decomposing leaf litter and detritus, effluent discharge, suspended particles and solids from watershed runoff (Gallegos et al. 2005; Hernes et al. 2008; Keith et al. 2002; Nelson and Guarda 1995; Nishimura et al. 2012; Zhang et al. 2009). This applies to irrigated agricultural watersheds that can impact DOC concentration and composition as soil organic matter is mobilized (Hernes et al. 2008).

Hydrological events that mobilize soils and water affect CDOM in agricultural watersheds. Examples include groundwater input, overland flow, flushing events, shallow lateral transport of water through soil, and irrigation (Hernes et al. 2008). Moreover, CDOM is affected by agricultural practices that mobilize soils. For example, poorly managed irrigation can mimic the effects of a storm event causing water to move over the upper-most soil horizon and resulting in soil erosion into a receiving watershed (Hernes et al. 2008). Huang and Chen (2009) also found land-use to influence organic matter and CDOM and its transportation to receiving waters. Hence, CDOM not only varies among groundwater, river and lake waters (Mostofa et al. 2007) in a watershed, but also potentially in agricultural lands and irrigation fields (Henes et al. 2008).

This study examines water quality in an irrigation canal system located in the Lower Rio Grande Valley, Texas. The goal was to determine the factors that influenced CDOM, and therefore water clarity, along an irrigation pathway. The findings have implications for understanding the optical properties of CDOM in shallow waters and the application of remote sensing algorithms to monitor salinity and water quality in irrigation systems (Ahn et al. 2008; Bastiaanssen et al. 2000). This study is the first comprehensive evaluation of irrigation water quality in south Texas.

METHODS

The Lower Rio Grande Valley (Starr, Hidalgo, Cameron, and Willacy Counties, Texas) consists of 353,000 acres of irrigated agricultural land and over 4,830 km of irrigation canals, pipelines, and resacas (i.e., oxbow lakes) (Huang et al. 2009; Wilkins et al. 2009). Surface water was sampled for nine months from 2012-2013 at five locations along an irrigation pathway in Hidalgo County beginning at Donna Reservoir (Donna, Texas) and terminating at a citrus field (Figure 1). The 400 acre Donna Reservoir stores drinking and irrigation waters for the Donna Irrigation District-Hidalgo County #1. This section of the district includes an approximately seven mile main canal and 168 miles of lateral canals and pipelines. Donna Canal and Reservoir System is a US Environmental Protection Agency (US EPA) superfund site and contains contaminated sediments along a five mile stretch of the system.

In the field, CDOM was measured as relative fluorescence (RFU) using an optical UV sensor (Cyclops-7, TurnerDesigns, Inc., Sunnyvale, California) and Databank handheld datalogger. Physicochemical parameters for water temperature, dissolved oxygen (DO), electrical conductivity (EC), pH, salinity, and total dissolved solids (TDS) were measured using a HI 9828 multimeter (Hanna Instruments, Smithfield, Rhode Island).

Figure 1. Irrigation canal pathway and five sampling sites, Donna (Hidalgo County), Texas

Water samples were collected in polycarbonate bottles, transported on ice, and filtered (0.45µm, Millipore, Billerica, Massachusetts) immediately on accession to the laboratory. Water quality testing took place within 48 hrs of sample collection and at room temperature (22.5˚C). Nutrient concentrations for ammonium (NH3-N), nitrite (NO_3), nitrate (NO_2), and phosphorus (PO_4) were measured using a DR3800 spectrophotometer and HACH (Loveland, Colorado) chemistries. Turbidity was measured using a HACH 2100Q turbidmeter. Plankton biomass was measured as the difference between incubated (25˚C) dried matter and 0.45µm filter weight.

To determine the effects of season and site, an ANOVA was determined using SPSS version 19 (IBM, Company). The Pearson correlation coefficient test determined the individual effects of CDOM, physicochemical parameters, nutrients, and plankton on each other.

RESULTS

Sample Sites

The Donna Reservoir is the origin of irrigation water (Donna, Hidalgo County) (Site 1), which flows northward and terminates at an agricultural field growing citrus (Site 5) (Figure 2). Except for Site 1, all the irrigation canals are concrete lined to prevent water

loss. The width of the canal system decreased northward from Donna Reservoir from 11.07 m (Site 2), 2.39 m (Site 3), 1.89 m (Site 4), to 1.5 m (Site 5).

Figure 2. Sites 4 and 5 along the irrigation canal pathway, Donna (Hidalgo County), Texas. Site 4 is the canal in the foreground, and Site 5 is the east branch terminating at the citrus field

Locations of the sample sites are as follows: Site 1 (26.153839,-98.082989), Site 2 (26.154427,-98.082903), Site 3 (26.175308,-98.046912), Site 4 (26.199892,-98.033957), and Site 5 (26.200027,-98.033828).

CDOM

CDOM was detected in the irrigation waters and ranged from 481 MHz (May, Site 1) to 1302 MHz (December, Site 5) (mean=644.90, 179.25). CDOM varied the greatest in the summer and increased northward from the source water at Donna Reservoir (Site 1) to the terminating irrigation field (Site 5) (Table 1). The difference between the means of CDOM through the season was significant ($p < 0.05$) (Table 2), whereas it was insignificant among the five sample sites (Figure 3). Positive correlations existed for CDOM and conductivity, total dissolved solids, salinity, ammonium, and nitrite (Table 3).

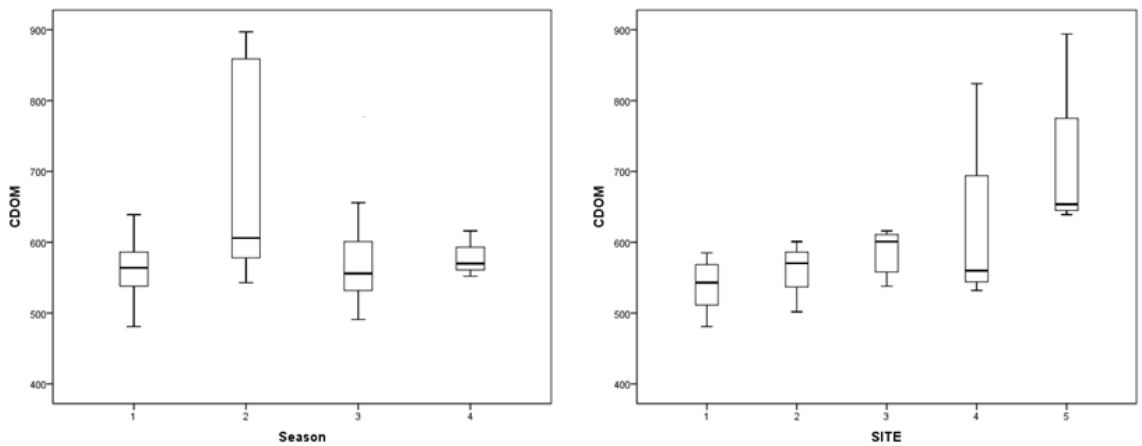

Figure 3. Box-plots of CDOM by season and sample site. Note for season plot: 1 (spring), 2 (summer), 3 (fall), and 4 (winter). CDOM is measured as MHz

Physicochemical Parameters

Temperature. Water temperature of the irrigation canal ranged from a December low of 15.5 ˚C at Site 3 to a May high of 33.3 ˚C at Site 4 (mean=25.5 ˚C, sd=5.33). No northward gradient existed among the sites. Not surprisingly, temperature varied by season and not by sample sites (Table 2). Negative correlations existed between temperature and nitrite and phosphorus (Table 3).

Electrical Conductivity. Electrical conductivity ranged from 1104 µS/cm (January, Site 2) to 1626 µS/cm (December, Site 5) (mean=1280.8, sd=111.85). Increases in electrical conductivity occurred northward in Sites 4 and 5 (Table 1). Electrical conductivity did not vary by season or sample site (Table 2). Positive correlations resulted among electrical conductivity and CDOM, total dissolved solids, salinity, ammonium, nitrite, and phosphorus (Table 3).

Salinity. Salinity of the irrigation water ranged from 0.55 PSU in July at Site 3 to 0.82 PSU in December at Site 5 (mean=0.63, sd=0.06). No recognizable northward gradient for salinity was found (Table 1). Salinity did not vary by season or sample site (Table 2). Positive correlations were found among salinity and CDOM, electrical conductivity, total dissolved solids, ammonium, nitrite, and phosphorus (Table 3).

Total Dissolved Solids. Total dissolved solids occurring in the water ranged from 555 ppm (November, Site 2) to 808 ppm (December, Site 5) (mean=640, sd=54.5). No recognizable northward gradient for total dissolved solids was found (Table 1). Total dissolved solids did not vary by season or sample site (Table 2). Positive correlations existed among total dissolved solids and CDOM, electrical conductivity, salinity, ammonium, nitrite, and phosphorus (Table 3).

Table 1. Annual water quality parameters (mean, range) of sites along the Donna Reservoir Irrigation Canal System. Only values above detection limits considered

Water Quality Parameter (Detection Limit)	Site 1	Site 2	Site 3	Site 4	Site 5
Temperature (ºC) (na)	25.8 (15.5-30.5)	25.3 (16.0-30.2)	25.3 (15.5-30.8)	25.9 (16.5-33.3)	25.5 (15.7-31.3)
pH (0.01)	8.16 (6.48-9.41)	7.82 (6.72-8.91)	7.15 (6.67-7.92)	7.43 (6.38-8.75)	7.06 (6.34-8.19)
Turbidity (NTU) (na)	60.20 (10.00-120.00)	73.88 (50.60-119.00)	69.22 (34.20-121.00)	58.63 (6.08-214)	81.65 (7.62-225)
DO (ppm) (na)	7.72 (2.97-13.61)	7.00 (2.81-11.52)	5.14 (1.55-7.89)	4.77 (1.03-10.05)	6.02 (1.15-13.62)
Salinity (PSU) (na)	0.62 (0.57-0.70)	0.62 (0.55-0.70)	0.62 (0.55-0.68)	0.66 (0.59-0.76	0.67 (0.59-0.82)
TDS (ppm) (na)	626 (575-706)	625 (555-705)	628 (557-688)	661 (591-752)	669 (606-808)
EC (µS/cm) (na)	1254 (1150-1413)	1242 (1104-1409)	1256 (1114-1375)	1322 (1182-1503)	1338 (1183-1448)
Ammonium (ppm) (0.02)	0.084 (0.055-0.113)	0.107 (0.048-0.230)	0.131 (0.051-0.444)	0.181 (0.040-0.630)	0.292 (0.05-0.854)
Nitrite (ppm) (0.002)	0.008 (0.002-0.018)	0.111 (0.002-0.021)	0.111 (0.004-0.018)	0.015 (0.003-0.033)	0.035 (0.006-0.159)
Nitrate (ppm) (0.23)	0.208 (0.119-0.230)	0.207 (0.115-0.230)	0.159 (0.105-0.230)	0.244 (0.133-0.339)	0.162 (0.108-0.230)
Phosphorus (ppm) (0.15)	0.200 (0.200-0.200)	0.230 (0.230-0.230)	0.226 (0.226-0.226)	0.258 (0.164-0.327)	0.353 (0.012-0.027)
CDOM (MHz) (na)	563 (481-758)	561 (502-601)	629 (538-897)	698 (532-1016)	870 (639-1302)
Plankton (g) (na)	0.000142 (0.000020-0.000397)	0.000159 (0.000001-0.000420)	0.000170 (0.000003-0.000276)	0.000161 (0.000010-0.000400)	0.000131 (0.000009-0.000396)

Dissolved Oxygen. Dissolved oxygen levels ranged from an August low of 1.03 ppm at Site 4) to a July high of 13.62 ppm at Site 5 (mean=6.22, sd=3.7). Dissolved oxygen levels decreased after Sites 1 and 2 (Table 1). Dissolved oxygen did not vary by season or sample site (Table 2). Dissolved oxygen correlated positively with pH (Table 3).

pH. pH levels ranged from acidic 6.34 (September, Site 5) to alkaline 9.41 (November, Site 1) (mean 7.53, sd=0.79). pH displayed a decreasing northward gradient (Table 1). pH did not vary with season; however, a significant variation occurred among sample sites (Table 2). Positive correlations were found among pH and dissolved oxygen and ammonium (Table 3).

Turbidity. Turbidity of the irrigation water ranged from 6.08 NTU in July at Site 4 to 225 NTU in August at Site 5 (mean 68.71, sd=55.55). Turbidity had no recognizable northward gradient (Table 1). Turbidity did not vary with season or sample site (Table 2). Turbidity correlated positively with nitrite, nitrate, and plankton (Table 3).

Nutrients

Ammonium. Ninety-percent of the ammonium values were above detection limit (0.02 ppm) with a range of 0.040 ppm (May, Site 4) to 0.854 ppm (September, Site 5) (mean=0.162, sd=0.19). A northward gradient in ammonium was present in the irrigation system (Table 1). Ammonium did not vary by season or sample site (Table 2). Positive correlations existed with CDOM, electrical conductivity, total dissolved solids, salinity, and nitrite (Table 3). A negative correlation was found between ammonium and pH.

Nitrite. All nitrite values were above the detection limit (0.002 ppm) with a range of 0.002 ppm (Sites 1 and 2, April and May) to 0.159 ppm (Site 5, December) (mean=0.016, sd=0.23). No recognizable northward gradient in nitrites occurred (Table 1). Nitrite varied by season and did not vary with sample site (Table 2). Nitrite correlated positively with CDOM, electrical conductivity, total dissolved solids, salinity, turbidity, ammonium, and phosphorus (Table 3). A negative correlation was found with temperature.

Nitrate. Forty-percent of the nitrate values were below detection limit (0.1 ppm), resulting in a range of 0.105 ppm in December at Site 3 to 0.339 ppm in December at Site 4 (mean=0.201, sd=0.06). No recognizable northward gradient in nitrates occurred (Table 1). Nitrate varied significantly by season and did not vary with sample site (Table 2; Figure 4). Positive correlations existed between nitrate and phosphorus (Table 3). A negative correlation was found with turbidity.

Table 2. Analysis of Variance (ANOVA) of CDOM, physicochemicals, nutrients, and plankton by season and sample site. * Significant at α-0.05

Dependent Variable	Factor	DF	Mean Square	F	p
Temperature	Season	3	233.623	19.03	0.00*
	Sample Site	4	0.673	0.02	0.99
pH	Season	3	0.835	1.35	0.27
	Sample Site	4	1.719	3.36	0.02*
Turbidity	Season	2	4268.044	1.42	0.26
	Sample Site	4	552.864	0.16	0.96
Dissolved Oxygen	Season	3	5.015	0.34	0.79
	Sample Site	4	11.181	0.79	0.54
Salinity	Season	3	0.996	1.16	0.34
	Sample Site	4	0.748	0.85	0.50
Total Dissolved Solids	Season	3	6394.629	2.36	0.09
	Sample Site	4	3777.261	1.31	0.28
Conductivity	Season	3	23929.721	2.06	0.12
	Sample Site	4	16428.261	1.36	0.27
Ammonia	Season	3	0.039	1.11	0.35
	Sample Site	4	0.068	2.17	0.09
Nitrite	Season	3	0.001	2.96	0.04*
	Sample Site	4	0.001	2.05	0.11
Nitrate	Season	3	0.027	3.55	0.02*
	Sample Site	4	0.014	1.57	0.20
Phosphorus	Season	3	0.009	0.67	0.57
	Sample Site	4	0.015	1.27	0.30
CDOM	Season	3	87437.537	3.40	0.03*
	Sample Site	4	68316.938	2.59	0.06
Plankton	Season	2	0.000	6.42	0.00*
	Sample Site	4	0.000	0.07	0.99

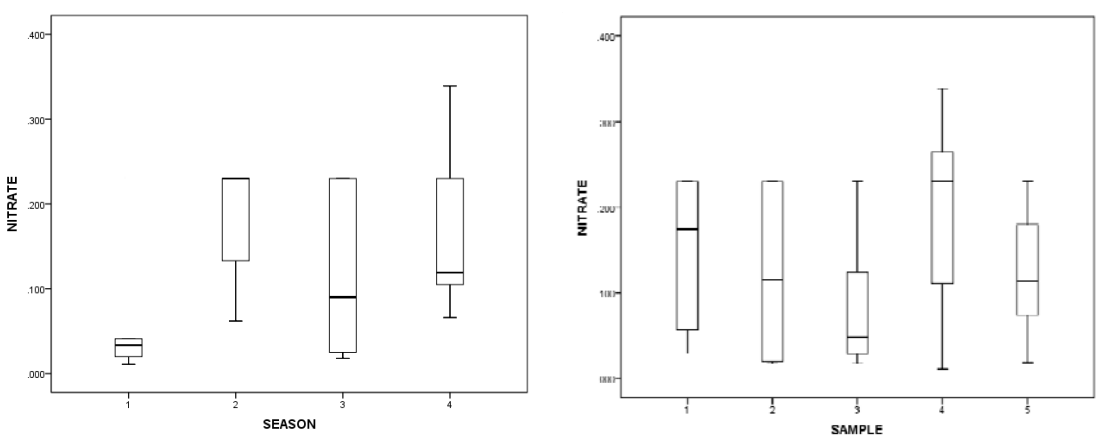

Figure 4. Box-plots of Nitrate by season and sample site. Note for season plot: 1 (spring), 2 (summer), 3 (fall), and 4 (winter). Nitrate is measured as ppm

Phosphorus. The majority (80%) of the phosphorus values were below detection limit (0.15 ppm), resulting in an August low range of 0.164 ppm at Site 4 to a December high of 0.622 ppm at Site 5 (mean=0.274, sd=0.13). A northward gradient in phosphorus occurred (Table 1). Phosphorus did not vary with season or sample sites (Table 2). Phosphorus correlated positively with temperature, electrical conductivity, total dissolved solids, salinity, nitrate, and nitrite (Table 3).

Plankton

Plankton values ranged from a November low of 0.000001 g at Site 2 to an August high of 0.000420 g at Site 2 (mean=0.000154, sd=0.0001). Plankton displayed no recognizable northward gradient (Table 1). Plankton varied by season ($p < 0.05$), but not among the sample sites (Table 2; Figure 5). Whereas no significant positive correlations occurred, a negative correlation existed with temperature (Table 3).

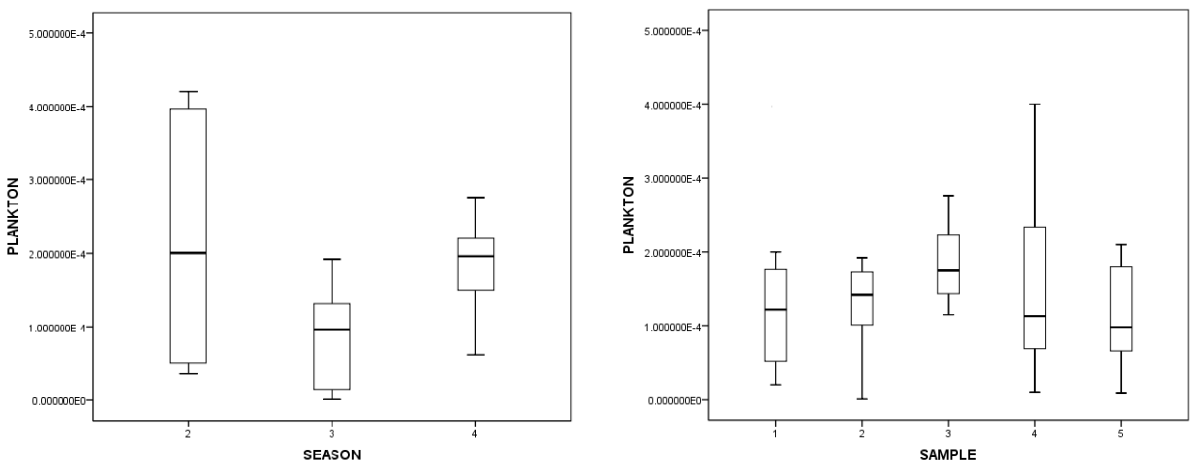

Figure 5. Box-plots of total plankton by season and sample site. Note for season plot: 2 (summer), 3 (fall), and 4 (winter). Plankton is measured as grams

Table 3. Pearson Correlation Coefficient for CDOM, physicochemicals, nutrients, and plankton. Values below detection limit not considered. * Significant at α-0.05

	Nitrate	Nitrite	Phosphorus	Ammonium	CDOM	Temperature	Conductivity	TDS	Salinity	DO	pH	Turbidity	Plankton
Nitrate	1.0												
Nitrite	-.229	1.0											
Phosphorus	.768*	.882*	1.0										
Ammonium	.074	.619*	.586	1.0									
CDOM	.337	.753*	.995	.867*	1.0								
Temperature	.035	-.334*	-.779*	-.239	-.311	1.0							
Conductivity	.125	.425*	.767*	.500*	.508*	-.042	1.0						
TDS	.078	.428*	.758*	.503*	.503*	-.056	.994*	1.0					
Salinity	.067	.464*	.771*	.533*	.547*	-.107	.989*	.996*	1.0				
DO	.219	-.205	.087	-.248	-.147	-.131	-.023	-.027	-.033	1.0			
pH	.259	-.205	.013	-.418*	-.275	-.154	-.142	-.145	-.129	.571*	1.0		
Turbidity	-.575*	.364*	.040	.065	.183	-.313	.083	.083	.104	-.190	-.049	1.0	
Plankton	.207	.021	.179	.005	.251	.354	.171	.184	.129	-.102	-.213	-.384*	1.0

DISCUSSION

Gradients of water quality parameters existed for the irrigation pathway (Table 1). The sample sites along the irrigation pathway exhibited northward increasing gradients of annual mean ammonium, phosphorus, and CDOM and a decreasing gradient in pH. Although annual mean electrical conductivity increased at the terminus (Sites 4 and 5) of the pathway, no clear pattern occurred. Likewise, dissolved oxygen was greatest at Sites 1 and 2 and fluctuated at the remaining sample sites. These high dissolved oxygen values at the origin of the irrigation water (Donna Reservoir, Site 1) would be expected with the wave action and mixing occurring at the reservoir. Similarly, the high values at Site 2, which is immediately adjacent to the reservoir, most likely results from the pump house supplying the water into this system.

Seasonal influences did exist for several water quality parameters (Table 2). The results suggest significant seasonal differences for water temperature, nitrite, nitrate, CDOM, and plankton. Not surprisingly, water temperature was coldest in December and warmest in May. The lowest nitrite values occurred in April and May and highest values in December. Both lowest and highest values of nitrate were in December. CDOM exhibited the lowest value in May and highest value in December. Plankton values were low in November and high in August at the same site (Site 2). However, the interpretations of these seasonal differences also must consider the character of the canal segment. The high values of temperature, nitrites, nitrates, and CDOM are associated with Sites 4 and 5, which are the shallowest and narrowest segments in the canal system. The high plankton values may result from regular water circulation; the water is pumped from the reservoir (Site 1) to the first receiving canal (Site 2). Plankton productivity is

known to be influenced by high rates of water discharge and nutrient input (Kitheka et al. 1996; Malone 1992).

Few differences in water quality parameters existed along the irrigation pathway (Table 2). The only water quality parameter found to vary significantly with the different irrigation canal segments was pH. Mean pH values were lowest (acidic) at Site 5 and highest (alkaline) at Site 1. The results suggest that pH did not correlate with expected dependent changes in water temperature (Table 3). Thus, the differences could result from water level (lowest at the citrus field, Site 5 and highest in the reservoir, Site 1) and evaporation rates (highest at the citrus field, Site 5 and lowest in the reservoir, Site 1).

Numerous correlations existed among the water quality parameters (Table 2). CDOM changed positively with conductivity, total dissolved solids, salinity, ammonium, and nitrite. The physicochemical measures of temperature, electrical conductivity, total dissolved solids, salinity, dissolved oxygen, pH, and turbidity all exhibited negative and positive correlations. The correlation between dissolved oxygen and pH in the irrigation canal could be related to the amount of decomposing organic matter (nitrification-denitrification) accumulating in the canal bottom. pH levels conducive to bacterial growth would result in lower dissolved oxygen levels. The relationships between electrical conductivity-total dissolved solids and electrical conductivity-salinity are expected, because the metered calculation of these parameters is dependent on each other. Furthermore, electrical conductivity is related to the amount of dissolved solids and salt concentration that conduct electricity. Significant correlations also existed for nutrients (ammonium, nitrate, nitrite, and phosphorus). Correlations among ammonium, nitrite, and nitrate are expected with nitrogen cycle processes taking place in the irrigation canal. Phosphorus correlated positively with nitrite and nitrate. These relationships are known to exist, be dependent on water flow, and the ratios are important to supporting aquatic organisms (Jarvie et al 1998). Plankton only correlated negatively with turbidity. Because high turbidity decreases light penetration, it is expected that primary productivity and therefore plankton abundance would also decrease (Lloyd et al. 1987).

CONCLUSION

The water quality of irrigation canals in South Texas has received little attention despite the: (1) canals being the only other freshwater watercourses besides the Rio Grande, and (2) use of the canals by wildlife and fishes. The water quality of south Texan irrigation canals is seldom studied (Wells et al. 1998), although it has been the focus of environmental exposure and contamination (Garcia et al. 2001; Robertson and Gamble 1991). Irrigation districts in south Texas tend to assume that water pumped from the Rio Grande to the canal system does not change in quality along the pathway, or that only testing for salinity is important.

This study of five segments along a canal pathway presents the first comprehensive study of irrigation water quality in south Texas. CDOM, physicochemical, and nutrient parameters were examined to determine the presence of gradients and the effects of season and sample sites. The analysis supported increasing gradients with the direction

of water flow (i.e., northward) for annual mean ammonium, phosphorus, and CDOM and a decreasing gradient in pH. Seasonality affected measures of water temperature, nitrite, nitrate, CDOM, and plankton. Only pH was found to vary with the different sample sites. Correlations were found among each parameter considered in this study.

CDOM displayed a gradient along the irrigation canal pathway and varied significantly by season. Correlations were found for CDOM and conductivity, total dissolved solids, salinity, ammonium, and nitrite. Our findings are similar to other studies of CDOM that identified correlations among physicochemical variables (Bowers and Brett 2008; Ferrari and Dowell 1998; Gallegos et al. 2005; Kostoglidis et al. 2005; Mostofa et al. 2007; Vodacek 1992; Watras et al. 2011). Therefore, CDOM could prove to be an additional useful tool in rapid assessments of irrigation canal systems and water quality, complimenting physical inspections (Leigh and Fipps 2003).

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SELF-CLEANING TRASH RACK FOR IRRIGATION CANALS

Tom Gill[1]

ABSTRACT

Engineers at the US Bureau of Reclamation Hydraulics Laboratory in Denver have developed a prototype self-cleaning trash rack concept for irrigation canals that is operated on solar-charged power. The system is designed to trigger a cleaning cycle when the water level differential across the trash rack exceeds a threshold value. Laboratory tests were conducted during 2012 using three debris types including synthetic aquatic plants, sago pondweed, and filamentous algae. The lab tests yielded promising results. The prototype device has been installed at the field site in Northeastern Colorado where it is in operation during the 2013 irrigation season. The project team believes this system can be a highly cost competitive alternative for removing debris from canals compared with systems currently available that offer similar capabilities.

INTRODUCTION

Removal of debris from water flowing in canals is an operational issue for virtually any open-channel conveyance system. Commonly encountered debris would include wind-blown materials including weeds and trash generated by human activities, aquatic plant tissue, dead fish and/or other dead animals. Stationary trash racks are commonly installed to serve as a debris collection point. Accumulated debris must be periodically removed – either manually or mechanically to keep the delivery system operating properly.

This self-cleaning trash concept is largely a result of seeking to address a problem site at a remote location in southwestern South Dakota. In July of 2003, the author accompanied Gary Velder of Reclamation's Dakota Area Office (DKAO) in a site visit to Angostura Irrigation District (AID) near Oral SD. A trash rack at the Cheyenne River invert siphon entrance on the AID main canal was pointed out by AID staff as a key operational issue for the district. Whenever the wind was of sufficient velocity to move tumbleweeds this rack would need to be cleaned every few hours. In subsequent years, AID has reported problems with debris accumulations from sago pondweed and from filamentous algae at the site. Checking this trash rack represents approximately a 20 mile round trip from the District office in Oral. Figure 1 is a photograph of the Cheyenne River siphon entrance.

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Figure 1. Trash rack at the AID Cheyenne River siphon entrance

SYSTEM CONCEPT

The objective of this project was to develop a system that could address the issues at the AID Cheyenne River invert siphon entrance. Site parameters considered include:

- Head availability at this site is the limiting factor in the flow rate that may be delivered through the invert siphon. A replacement trash collection system could not cause additional head loss without impacting deliveries through the invert siphon.
- The "coarse" degree of cleaning afforded by the existing trash rack has been meeting operational needs. Improved capability for screening trash from the flow was not considered an objective.
- Given the remote location of the site, an automated cleaning system (in lieu of a manually controlled mechanical cleaning system) would represent a significant increase in value (staff time and travel cost savings) for AID.
- An electric power service line happens to be in place adjacent to the site. In order to configure a system that can be readily applicable for remote sites where electric service would not be readily available, the prototype was developed to operate using solar charged power.

PREVIOUSLY EXISTING TECHNOLOGIES

Various mechanical trash rack cleaning systems have been developed that utilize a rake mechanism to clean trash from stationary bar racks. These are generally systems that represent a level of capital investment that is often not affordable for installation on moderate to small sized agricultural canals and commonly have significant power requirements. Examples would include the automated cleaning rake system at the head of the Gila Gravity Main Canal in southwestern Arizona shown in Figure 2 and the manually operated power rake on the Orchard-Mesa Power Canal in western Colorado shown is Figure 3.

Figure 2. Gila Gravity Main Canal automated trash rack rake

Figure 3. Orchard-Mesa manually-operated mechanical trash rake

An assortment of debris removal technologies have been utilized successfully on moderate to small sized canals. These include water driven devices such as drum screens, traveling screens and an endless chain with attached brushes. Water driven devices were not deemed viable for the Cheyenne River invert siphon entrance site where head availability is already a controlling factor in delivery capacity.

A screen system versus a bar rack trash barrier would be expected to present an increase in head loss due to the smaller open spaces through which flow passes. Thus even motorized screens options – which would not be dependent on energy from the flowing water to produce mechanical motion – did not appear able to meet the objective of posing no additional head loss compared with the existing bar rack at the Cheyenne River invert siphon entrance.

PROTOTYPE CONFIGURATION

The concept that emerged from the site considerations plus a review of existing technologies was a trash rack designed such that the rack bars themselves would serve as the cleaning mechanism. A three foot wide prototype system was assembled consisting of one half inch wide by approximately three inches deep steel bars spaced three inches on center. The bars are installed with a 3:1 (H:V) slope – similar to the slope of the existing bar rack at the Cheyenne River invert siphon entrance.

The upper edge of each bar is cut in a saw-tooth pattern. Each bar is able to move approximately 1 foot in the plane of the rack slope. Slotted holes cut near each end of the bars define the mechanical limits of travel distance in the plane of the slope. Bar motion is powered by three 12 volt DC electric motors. The bars are organized in three groups with motion of every third bar powered by a given motor.

For a cleaning sequence, all bars are advanced in the direction of the upper end of the rack in unison. Following the advance, one group of “every third” bar is retracted to the original position. This is followed by the retraction of the second bar group and then by the retraction of the third bar group. The orientation of the saw teeth on the upper edge of the bars is such that the teeth bite into a debris mat during the advance cycle. During each phase of the retraction, two of every three bars remain stationary while motion of the retracting bar group is such that the sloping edge of the saw teeth slide under the debris mat.

A segment of roller chain is welded to the underside of each bar at the appropriate location for alignment with the drive motor for each respective “every third” bar group. Each motor is linked to a shaft that passes under the bar rack. Sprockets are installed on the respective shafts in the appropriate position to engage the roller chain on the bars to be driven by each motor. Figure 4 shows the prototype trash rack installed in a laboratory flume.

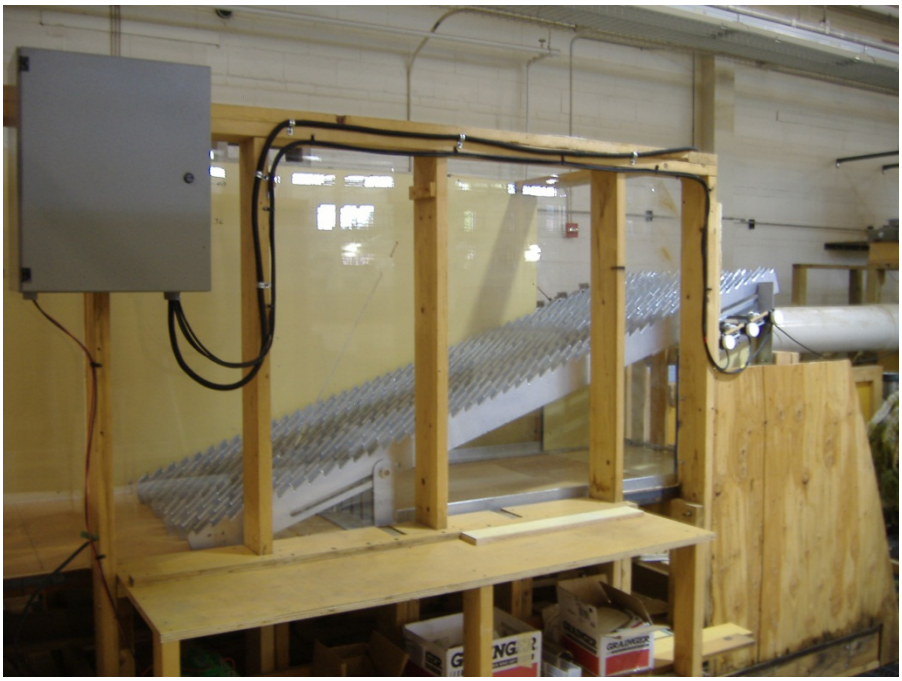

Figure 4. Prototype self-cleaning trash rack in a laboratory flume

LABORATORY TESTS

Test runs in the laboratory flume were performed with three types of debris. The system was first tested using synthetic aquatic plants that were available in the lab. After a mat of this material had collected on the rack, the system performed efficiently at transporting the debris up out of the flow and off the upper end of the rack.

Debris was then collected from canal trash rack sites in the field for further testing. Material from a debris pile consisting primarily of sago pondweed was acquired at a site on the South Platte Ditch near Merino CO. Material from a second debris pile consisting primarily of filamentous algae was acquired at a site on the Tetsel Ditch in Messex CO.

In a laboratory test run using the pondweed material, an accumulated debris mat was transported up from the flow and off the upper end of the rack a performance level similar to that observed with the synthetic plant material. In a laboratory test run using the algae material the system was effective at transporting a debris mat that covered approximately 75% of the trash rack width up and off the rack.

The field debris tests were limited by the volume material that been collected in the field. For the algae test in the area of the bar rack not covered by a dense mat of material, individual clumps (or streamers) of the algae became wrapped around a single bar. For this case as the bars moved during a cleaning cycle, the clump of algae would travel back with the bar. It was reasoned that for a field situation with algae laden flow the continuing stream of debris would likely result in formation of a debris mat extending the full width of the rack. The laboratory algae test was therefore considered successful.

Figures 5 and 6 show the positions of the synthetic plant material debris mat of the trash mat in the position of initial accumulation and after transport to the top of the rack.

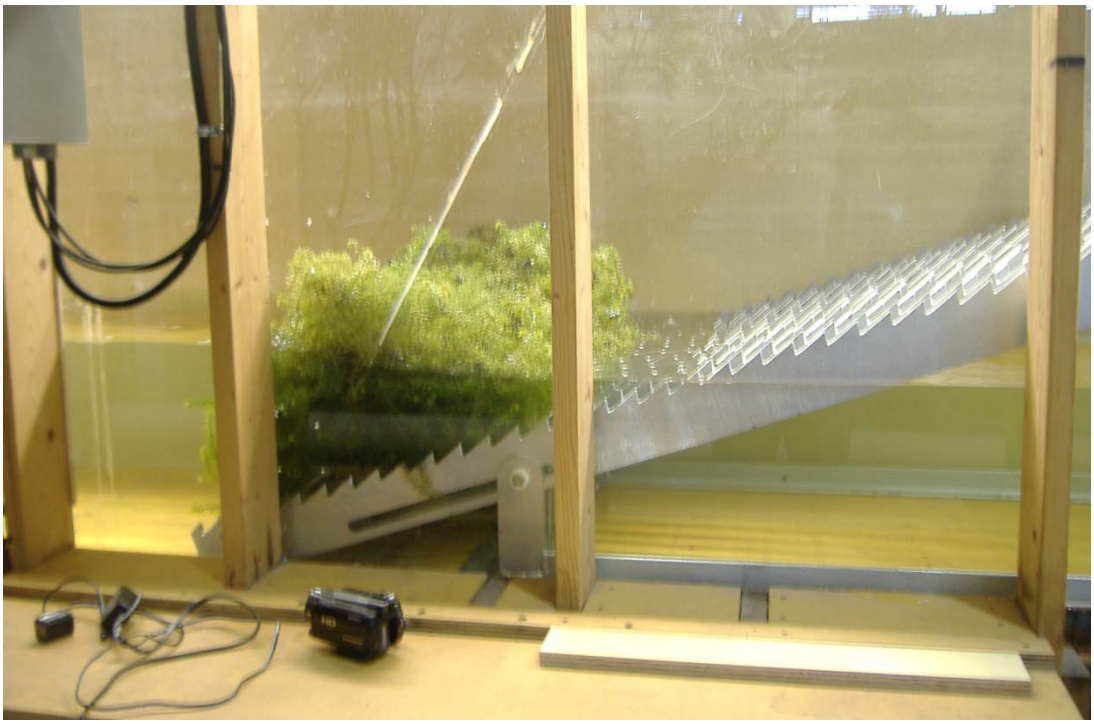

Figure 5. Debris mat accumulated within flow zone prior to cleaning cycle

Figure 6. Debris mat that has been transported to the upper end of the trash rack

FIELD TESTING

Following laboratory testing during 2012, the prototype unit was installed in the field prior to the 2013 irrigation season. The field installation site is the Tetsel Ditch site where the filamentous algae material was collected that was used in the laboratory tests. Figures 7 and 8 show the Tetsel Ditch installation.

Figure 7. The prototype self-cleaning trash rack installed at the Tetsel Ditch

Figure 8. The debris collection deck at the downstream end of the rack

The Tetsel Ditch site provides multiple attractive aspects for a field test site. The small canal was well suited for installation of the three-foot wide prototype unit tested in the laboratory. The site can be expected in service most if not all of the irrigation season. [The Tetsel Ditch has a senior water right on the South Platte. Water deliveries have rarely been interrupted by a senior priority call]. This site can be expected to have significant debris loads – the including the issue with algae seen during 2012. The site is also in relatively close proximity to Reclamation's Denver Office.

For the field installation, a steel grating deck is installed to catch debris as it is transported off the upper end of the trash rack. The controller is programmed with menus that provide on-screen prompts for keypad input to enable an operator to select "auto" or "off" mode for the system. With the system in either mode a menu option will allow the operator to run a cleaning cycle.

While operating in "auto" mode the unit is programmed to take water level readings at pre-set time intervals. When water level reaches a pre-set threshold indicating the rack is becoming clogged, a cleaning cycle is triggered. For each cleaning cycle the rack will perform the advance and retract sequence a pre-set number of times. The control unit being used at the prototype field test site is equipped with an integral data radio. Both the

programmable and the radio are set up to function with the Modbus RTU communication protocol. The “pre-set” values for level reading intervals, water level threshold, for advance/retract sequence repeats per cleaning cycle (as well as other programming constants) are numbers stored in Modbus registers. These values may be edited without stopping the program or modifying the program code using a Modbus instrument linked directly to the controller or may be edited remotely by radio from a compatible Modbus instrument.

For the field test setup, debris is deposited on the grating deck from which it is periodically being removed using a pitchfork. Figure 9 shows a debris mat being transported off the end upper end of the thrash rack where it is deposited on the grating deck.

Figure 9. A debris pile that has been transported off the trash rack onto the deck.

FIELD PERFORMANCE

The field demonstration site has been operating in “auto” mode since early June, 2013. As a debris mat accumulates it requires 8 to 9 repeats of the advance/retract sequence to fully clear debris from the trash rack bars. However no more than three repeats of the advance/retract sequence are needed to fully clear the area of the rack through which flow is passing.

From these observations it was reasoned that continued repetition of the advance/retract sequence in order to fully clear the debris mat from the rack each cleaning cycle represented an unnecessary power demand on the solar-charged battery. The system has been set to perform five repeats of the advance/retract sequence for each cleaning cycle. A debris mat transported up and out of the flow zone will remain on the upper section of the rack until it is eventually moved to the top and off of the rack during a subsequent cleaning cycle. This serves both to limit power consumption and to increase storage capacity for screened debris.

Through late August, 2013 the field demonstration site has been in continuous operation with only a minor need for adjustment. During this initial segment of field evaluation one mechanical issue has occurred. A nylon shaft collar which held in place a rod passing through the alignment slot near the upper end of each of the rack bars came off the rod and the rod worked out of its mounting hole on the right side of the rack. This allowed one of the bars to become out of alignment with its drive sprocket. Five other bars became one also became one or two teeth "out of time" with respect to the roller chain/sprocket positioning that keeps advance and retract stop positions of all bars approximately even.

There was no damage to equipment as a result of the alignment rod getting loose. The failed nylon collar attachment was upgraded by drilling a hole through the rod and inserting a steel spring clip to keep the rod in the mounting hole. The alignment and timing problems with the affected bars was corrected. These repair tasks were accomplished in approximately a half hour.

SUMMARY

The objective of devising a self-cleaning trash rack that could be functional and cost effective for applications similar to the Cheyenne River invert siphon entrance at Angostura Irrigation District appear to have been achieved by this system. Utilizing the trash rack bars as the rack cleaning mechanism represents a mechanically simplistic design compared with existing trash rack cleaning systems.

For the small canal selected for the field test site, manual removal of accumulated debris from the grating deck on a periodic (i.e. no more than daily) basis will be suitable in most cases. For higher capacity sites, some mechanical means of periodic removal of accumulated debris would likely be desirable. This system could be readily coupled with a conveyor for this purpose. If this system is installed with a debris accumulation deck that is of suitable weight carrying capacity use of equipment like the skid steer loader seen in Figure 3 at Orchard Mesa Irrigation District would also be an alternative. The secondary transport of accumulated debris is seen as a function that is independent from the task of collection and removal of debris from the flowing water which was the focus of this project.

An interesting aspect to the development of this system – in contrast with working with other canal modernization technologies – is the relatively modest degree of impact that

could result from a shut down or failure. At any time the automated cleaning function is out of service the system remains 100% functional as a stationary trash rack.

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WATER LEVEL SENSORS, WHAT WORKS?

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ABSTRACT

Water level sensors come in all shapes, sizes and types and can range in price from a couple hundred to several thousand dollars. This project utilizes real world examples from around the western United States to determine what level sensors work, and document what characteristics may prevent them from providing accurate water level measurements in differing climates. Researchers developed a calibration procedure that can be used in field or laboratory situations to obtain accurate calibrations of multiple types of water level sensors. Sensors that have been calibrated and installed are being monitored to determine their accuracy and reliability over several irrigation seasons.

INTRODUCTION

As water delivery entities are incorporating increasing levels of remote monitoring and automated control into their operations there is increasing need for water level sensors that function reliably. In today's age a wide variety of sensors are available to measure and monitor water levels. These sensors can range in price from a couple hundred to a several thousand dollars depending on what type and configuration of sensors are selected. This research discusses the need for water level sensors and answers a question that many hydraulic structure operators and managers pose to the U.S. Bureau of Reclamation engineers: What water level sensors provide accurate measurements over a sustained period of time and in a range of field conditions?

RESEARCH METHODOLOGY

This research is being conducted in cooperation with irrigation districts and Area Offices throughout Reclamation. To better represent realistic water level sensor usage, actual projects requiring water level sensors have been selected for this study. Researchers have installed and are monitoring the field performance of several water level sensing technologies at Reclamation related field sites. The work can be summarized with the following objectives:

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1. Identification of the water level sensors that will be tested. Equipment representing a full range of level sensing technologies commonly employed on canal systems and in conjunction with hydraulic structures will be included in the study.
2. Develop a protocol for periodic water level sensor equipment calibrations and checks. Configure portable calibration equipment that can provide accuracy on par with Reclamation's laboratory calibration equipment. This equipment will be used for regular on-site sensor calibrations.
3. Develop a standardized data collection process. Identify frequency of collection, variables (water level, temperature, elevation, time), and storage.
4. Identify sites within cooperating districts where existing water level monitoring stations can have additional (redundant and different) water level sensors added without extensive costs.
5. Implement calibrations and monitor each water level sensor's field performance over time using the protocols and standards identified in steps 2 & 3.
6. Document each water level sensor's performance and note constraints and/or ancillary capabilities offered by each type of instrument.

TYPES OF WATER LEVEL SENSORS TESTED

General

All sensors included in this study provide an analog signal output – either 4-20 milliamp or 0-5 volt – that may be readily linked with a broad range of programmable control units. Included sensors are also available within what was deemed a "moderate" price range (<$1,000). Most of the selected sensors are either units the researchers have either encountered first hand at field installations or are units that were recommended for inclusion by irrigation district contacts. Selected sensors that had not been identified by recommendation or by encounter were identified through a product search as highly cost competitive instruments that appeared to meet all capability needs. All of the selected sensors fall with one of the four level sensing technologies discussed below.

Submersible Pressure Transducer

Submersible pressure transducers are installed in the water at the location where the level is to be measured. The transducers convert fluid pressure into a proportional electronic signal over a specified range of water levels. Sensor outputs available from various instruments include 4-20 mA, 0-2.5 or 0-5 volt, SDI12 and Modbus. Figure 1 shows a submersible pressure transducer installed in a stilling well on a laboratory flume.

Ultrasonic Downlooker

Ultrasonic downlookers are installed suspended above the water at the location where the level is to be measured. Sensors are mounted normal to the water surface such that an acoustic signal can be sent and the return signal off the surface of the water can be received. Sensor intelligence determines the distance to the reflective surface using the

speed of the signal in the surrounding air. Sensor outputs can include 4-20 mA, 0-2.5 or 0-5 volt, SDI12 and Modbus. Figure 2 shows an ultrasonic downlooker sensor installed at the Buford-Trenton Irrigation District in North Dakota.

Float, Pulley, and Potentiometer

Float and pulley instruments feature a float and a weight attached to opposite ends of a cable or thin metal tape. The cable (or tape) is wrapped over a pulley such that any movement of the float/cable/weight/ apparatus will cause a shaft that the pulley is mounted on to rotate. These instruments are almost always installed in stilling wells to eliminate impact of surface waves. Float and pulley mechanisms have long been utilized as level sensors for mechanical level recording devices. For an economical electronic senor a multi-turn potentiometer may be attached to the shaft rotated by the pulley. As the float raises and lowers the potentiometer input shaft is rotated which increases or decreases the electrical resistance of the potentiometer which in turn increases or decreases the voltage of the output signal circuit. The float and pulley units that are being used in this study have been custom fabricated by Reclamation using an inexpensive potentiometer with a 10K ohm resistance and other parts commonly found in a hardware store. When operated using a 5 volts excitation this instrument provides a 0-5 volt output. Figure 3 shows two float, pulley and potentiometer sensors installed on a laboratory model.

Bubbler

For a bubbler sensor a small tube is installed with outlet end submerged at the location where the water level measurement is desired. The other end of the tube is connected to the bubbler sensing apparatus which includes an air source (air pump) and an electronic pressure transducer. Water level is determined by measuring the required pressure to push a bubble out the end of the tube. An electric air pump is operated for a short interval to propel a small burst of air into the tube. After a short pause to allow the system to become static, the pressure in the tube is measured.

This initial pressure value is retained while a second air pump operation pressure reading cycle is performed. These consecutive pressure readings are then compared. This process is repeated until consecutive readings reflect similar pressure values. If the tube is partially filled with water, the pressure measured after each pump operation cycle will increase and a portion of the water will be forced out of the tube. Once all the water is out of the tube, air bubbles will be pushed out the end of the tube during an air pump operation cycle and pressure in the tube will be nearly constant for consecutive cycles.

When the electronic controller recognizes that consecutive pressure readings are constant (within a tolerance range) the pressure will correspond to the current water level and a water level reading is completed. Sensor outputs can include 4-20 mA, 0-2.5 or 0-5 volt, SDI12 or Modbus. Figure 4 shows a bubbler sensor along with a programmable controller installed at Mohave Valley Irrigation and Drainage District in Arizona.

Figure 1. Submersible Pressure Transducer

Figure 2. Acoustic Downlooker

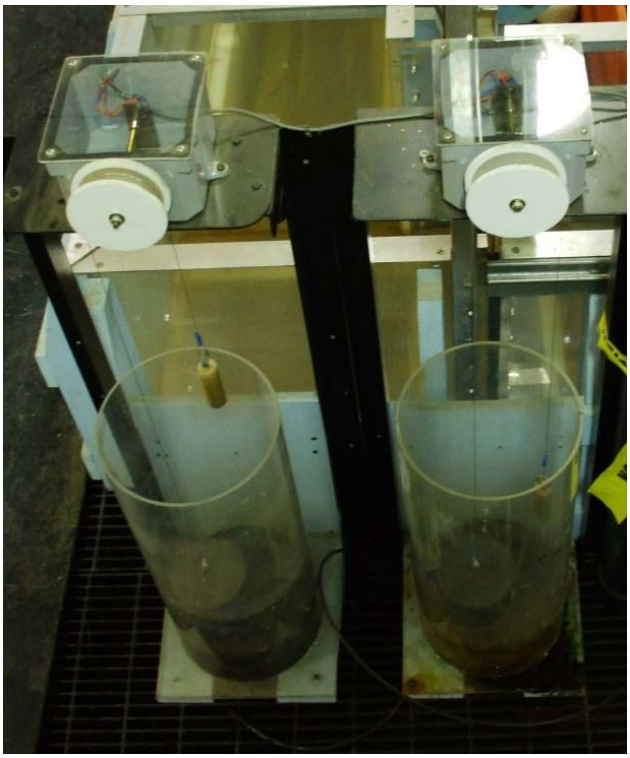

Figure 3. Float, Pulley & Potentiometer Sensors

Figure 4. Bubbler Sensor

Table 1 contains a list of the level sensors that have been included in the study so far. As noted above sensors were selected from literature review and from recommendations by end users and other interested parties. Often when sensors are selected for field deployment price becomes a limiting factor either because of budget constraints or the number of sensors required to accomplish the desired operation. For this reason an approximate price has been included for each of the sensors. Please note that sensor prices can fluctuate and the authors recommend requesting updated cost information from the manufacturers.

Table 1. List of sensors and their approximate cost

	Manufacturer	Model	Approx. Cost ($/Unit)	Quantity
Pressure Transducer	AGP	PT-500	460	3
	AutoMata	Level-Watch	280	3
	Endress Hauser	FMX21	955	2
	Endress Hauser	FMX167	1045	1
	GE Druck	PTX 1730	525	2
	Global Water	WL400	590	2
	Instrumentation Northwest	98i	540	4
	Keller	Acculevel	480	2
	Keller	Levelgage	315	4
	Stevens	SDX	355	4
Ultrasonic Downlooker	Judd Communications	-	655	4
	AGP	IRU-2005	495	3
	AutoMata	Ultra-Ultra	720	2
	EMS	SR6	250	5
	Flowline	EchoPod DL10-00	255	2
	Flowline	EchoPod DX10-00	235	2
	Global Water (EMS)	WL700	665	2
	Nova Lynx (APG)	IRU 9423	475	2
	Siemens "The Probe"	7ML12011EF00	860	3
Other	Float, Pulley and Potentiometer	USBR Design	150	3
	Bubbler - Control Design	CD 103-1	595	4
	Bubbler - OTT	CBS – Std.	1690	1
	Dwyer Temperature & Humidity	RHP-2R11	200	3

CALIBRATION PROCEDURE

Researchers developed a portable calibration stand and procedure which provides the ability to accurately calibrate sensors in the laboratory or field. The calibration stand consists of a 10-ft piece of heavy duty Unistrut attached to a 5-ft piece of light duty Unistrut. The light duty Unistrut attaches to a standard surveying tripod (Figures 5 and 6) and is staked to the ground at the base. Pressure transducers and bubblers are tested in a 2-inch clear PVC pipe that is attached to the heavy duty Unistrut with hose clamps. The pipe is filled using a 12 volt pump and 5 gallon storage reservoir filled with water. Ultrasonic downlookers are tested using a control arm that attaches to the heavy duty Unistrut and can be adjusted up and down the calibration stand using a thumb nob set screw (Figure 7). Acoustic signals are reflected off a level surface mounted at the bottom of the test stand (Figure 8).

Depth and distance measurements used to calibrate each sensor are taken using a tape measure that is attached to the outside of the calibration stand and can be read in 0.005-ft or 0.0625-inch increments depending on the user's preference. To remain consistent throughout the research, calibrations are conducted the same for all level sensors and can be summarized in the following steps:

1. Setup and level the calibration stand.
2. Determine the range of the sensor/s that will be calibrated.
3. Setup the sensors with the same equipment that will be used to obtain the level measurements when installed in the field.
4. Collect 10 data points with the water surface or distance increasing.
5. Collect 10 different data points with the water surface or distance decreasing.
6. Determine the linear regression lines for each:
    a. The rising water surface or distance calibration
    b. The decreasing water surface or distance calibration
7. Compare the slopes and coefficient of determinations for both linear regressions.
8. Determine the average slope and print a tag with the sensors serial number, slope and date of calibration and attach it to the sensor cable.

Figure 5. Calibration Stand

Figure 6. Calibration stand mount on tripod

Figure 7. Ultrasonic Mount

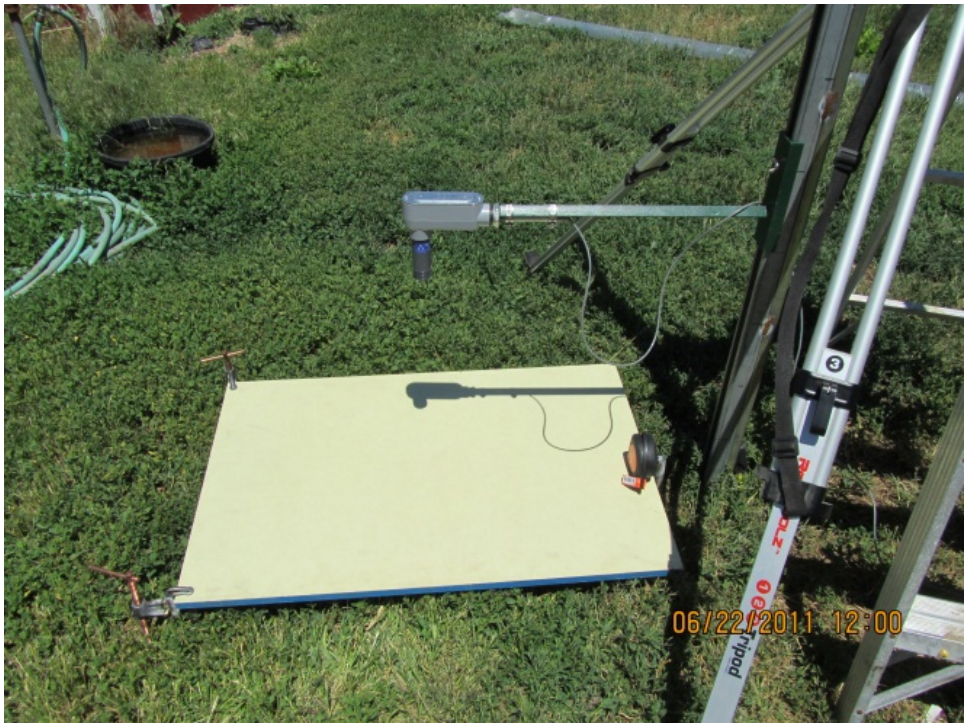

Figure 8. Reflective surface for ultrasonic sensors

SITE SELECTION

Geographic

Funding for this project was provided by Reclamation Science and Technology Research Program with additional funds from Reclamation Area Offices and support from irrigation districts. To keep costs at a reasonable level and ensure that the project would be useful to contributing partners, sites were selected where water level measurements were needed and existing infrastructure could support multiple measurement devices at one location.

Sites located near Yuma Arizona (extreme temperature), Grand Junction Colorado (moderate temperatures) and Sterling Colorado (moderate temperatures) have already been identified and have instrumentation either installed and operational or in the process of being installed. Sites in Montana and North Dakota are both being investigated to represent a much colder environment. Figure 9 is a location map showing the current installation sites in Colorado and Arizona (blue dots).

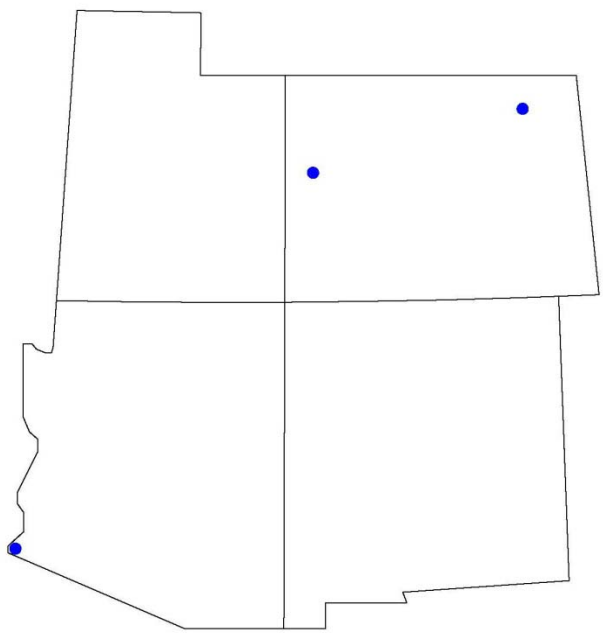

Figure 9. Project Site Installation Map

Sensor location

Once a geographical location has been established, locating a site to install sensors that will provide stable water surface measurements is essential. This can be done by locating the instrument in a stilling well with a port of around 1/20-1/30 the stilling well diameter connecting the well to the hydraulic structure (RECLAMATION 2001). Another method is to select a location free from drawdown influences, waves or other disturbances that could influence water level readings. When installing a submerged sensor is it important to make sure the sensor will not be damaged by debris or sediment in the flowing water.

SENSOR INSTALLATION PROCEDURE

The following procedure is followed when installing a new sensor at a site:

1. Determine the min and max water levels to be measured.
2. Determine the type of controller or base unit that will be used to record/transmit the data and what type of inputs it will accept.
3. Select multiple sensors that will accommodate the range and controller inputs.
4. Install a staff gauge and determine a reference to a readily identifiable datum or bench mark.
5. Install infrastructure which will allow multiple sensors to be installed in acceptable conditions (away from drawdown, waves, damage from debris, disturbances)
6. Install controller (base unit) in a secure location (preferably a lockable enclosure)
7. If applicable, install solar panel, ground-rod, antenna, and battery box.
8. Install sensors with the correct calibration slopes.
9. Trouble shoot sensors and base unit to ensure everything is working.
    a. Simulate different level measurements if possible to make sure sensor is working.
    b. Determine if the correct sign is used on the calibration slope by raising and lowering sensor ensuring that the output corresponds appropriately.
    c. Simulate radio communications (if necessary).
    d. Use controller menus to adjust offsets and shifts to check functionality.
10. Determine the sensor offset using a known datum and program it into controller.
11. Record necessary information in the project book. Include: Installation date, sensor types and serial numbers, slopes, offsets, datum reference used and where it came from, controller ID and any other important information.

DATA COLLECTION

Data collected for each site become a unique challenge because each site will have unique sensors and datum. To help manage the data, water levels are recorded at least fifteen minutes apart. All data is arranged with a date and time stamp. In addition, all sites should log battery and charge voltage, internal base unit temperature, and one site in each geographical location should log the ambient air temperature. Where possible manual data collection of the installed staff gauge will be taken periodically to compare to the logged data and determine what sensors, if any, are drifting or having any other issues. Data collection is in beginning stages, any suggestions on how to improve the data management portion of the research are welcome.

KNOWN ISSUES

To date there a several known issues that have arisen when it comes to installing and using water level sensors that may prevent accurate level measurements. Ultrasonic downlookers are sensitive to spider webs; most often the webs will concentrate near the sensor face and prevent accurate measurements. Although many of the manufacturers claim to be compensating for fluctuations in the temperature, the authors noticed that

during rapid changes in temperature some of the sensors that "compensate" for temperature fluctuations would not provide repeatable calibrations. It seems that pressure transducers seem to be sensitive to calibration shifts if they are mishandled, dropped or banged. To date we have had two sensors stop working, we have not determined why but both manufacturers replaced the sensors under warranty. Although many of the sensors claim to work with as little as 12 volts power the authors have found that in most cases using a 12 to 24 volt converter to increase the supply voltage has provided more consistent measurements and calibrations. As more issues arise the authors will be documenting the problems and any solutions that are found.

CONCLUSIONS

To better understand water level sensors and to evaluate performance in a range of operating environments, the U.S. Bureau of Reclamation has been installing level sensors in varying climates and documenting any issues and successes that are observed. Sensors from multiple manufacturers and of multiple types have been purchased and installed in both hot and mild climates including Yuma Arizona, Grand Junction Colorado, and near Sterling Colorado. Additional sites located in a colder environment are being investigated in Montana and North Dakota.

Once geographic locations are determined sensor sites can be identified, equipment can be deployed in the field. Researchers developed a portable calibration procedure which allows all sensors to be calibrated before they are deployed. Sensors are calibrated in both a rising and decreasing water surface or distance with 10 points in each direction. Calibrations are performed periodically when needed but at least once per year. Data is collected and compared against periodic manually recorded data to determine if sensors are drifting or calibrations have shifted. Data collection is just beginning and will be on-going for multiple years.

Several issues have already been documented that prevent accurate water level measurements including by not limited to:

- Spider webs preventing ultrasonic downlookers from working
- Lagging temperature compensation in response to rapid temperature changes
- Sensors failing for no apparent reason
- 12 volt supply power inadequate for consistent measurements

As more issues and successes are found the list will continue to grow.

ACKNOWLEDGMENTS

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