Energy, Climate, Environment and Water — Issues and Opportunities for Irrigation and Drainage

San Luis Obispo, California
July 9-12, 2002

Sponsored by

U.S. Committee on Irrigation and Drainage
Environmental & Water Resources Institute, ASCE

Co-Sponsored by

Irrigation Training and Research Center
California Polytechnic State University

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Published by

U.S. Committee on Irrigation and Drainage
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Proceedings Sponsored by the Bureau of Reclamation

The U.S. Committee on Irrigation and Drainage accepts no responsibility for the statements made or the opinions expressed in this publication.

Printed in the United States of America

Library of Congress Number 2002108153

ISBN 1-887903-14-3

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Preface

The papers included in these Proceedings were presented during a Conference held July 9-12, 2002, in San Luis Obispo, California. **Energy, Climate, Environment and Water — Issues and Opportunities for Irrigation and Drainage** was sponsored by the U.S. Committee on Irrigation and Drainage and the Environmental & Water Resources Institute of the American Society of Civil Engineers. The Irrigation Training and Research Center, California Polytechnic State University, co-sponsored the Conference.

Irrigators are facing new challenges as competition for water supplies, coupled with significant increases in energy costs and environmental considerations, threaten the economic viability of irrigation. Climate changes, whether natural or a result of human activity, are providing additional concerns. The Conference provided a forum to discuss and evaluate these issues, with a focus on the technology being applied to meet the challenges.

Papers included in the Proceedings were accepted in response to a call for papers and were peer-reviewed prior to preparation of the final papers by the authors. The authors are professionals from academia; federal, state and local government agencies; water districts and the private sector.

The U.S. Committee on Irrigation and Drainage, the Environmental & Water Resources Institute and the Conference officers express gratitude to the authors, session moderators and participants for their contributions.

The Office of International Affairs, Bureau of Reclamation, sponsored the Conference Proceedings and this support is acknowledged with appreciation.

Charles M. Burt
Conference Chairman
San Luis Obispo, California

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ENERGY EFFICIENCY AS A NON-POINT SOURCE PROBLEM

Peter Canessa[1]

ABSTRACT

This paper argues that improving energy efficiency in agriculture is a long-term, non-point source type of problem. "Fixing" non-point source problems generally involves changing the behavior of resource managers. These types of problems are generally addressed by programs involving three components: 1) problem awareness, 2) solution awareness, and 3) targeted resources. The concept of market transformation as applied to energy efficiency is discussed in relation to non-point source programs. Market transformation also involves changing the way end-users (managers) think and act, but also recognizes supply-side issues in terms of changing the services and hardware being offered to the end-user.

"A problem well-stated is a problem half-solved"

INTRODUCTION

Business, government agency, and political managers are often charged with correcting problems in society. Generally, resources (time and money) to address these problems are limited. It is essential that problems be correctly characterized as to type so that the correct form (as opposed to the details) of response is utilized.

Two obvious examples of problem type are point source versus non-point source. The defining feature of the point source problem is that it can be "fixed" by addressing relatively few and/or well-defined situations. In contrast, non-point source problems require a response to a large number of (possibly ill-defined) situations. Point source problems may take a relatively short amount of time to correct while non-point source problems usually require a long-term effort.

The argument of this paper is that energy efficiency in irrigated agriculture is a long term, non-point source problem (NPS). Characterizing energy efficiency as this type of problem leads to an efficient strategy for achieving solutions.

CHARACTERISTICS OF NON-POINT SOURCE PROBLEMS

NPS have several defining characteristics:

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- The source of the problem is diffuse. That is, there are multiple sources of the problem.
- Each individual source may be operating "legally". That is, the activity is legal and is also being conducted to prevailing community and business standards. Importantly, the activity may be (or may have been) encouraged by society (e.g. use of fertilizer to increase crop yields).
- Few, if any, of the individual sources on their own are causing a problem as a legal or practical matter.
- The problem is caused by the cumulative effect of the diffuse sources.
- Because NPS are generally slow to appear, the activities causing the problem are many times "entrenched". That is, they are the result of long-term investments both in money to purchase and install the activity, but also in terms of management education in how to actually conduct the activity. There may be a cultural environment built up around the activity.

The characteristics of NPS add considerable political, economic, and engineering complexity to solving these types of problems.

Does energy efficiency (or lack thereof) in agriculture fit this description? Consider these points concerning energy used for pumping irrigation water:

- There were over 86,000 agricultural accounts in 1997 in just one of the three major California utilities. About 80,000 of these were pump users.
- Although there may be restrictions of one sort or another placed on the use of water in agriculture, there are no laws governing the use of energy (except for those governing the efficiency of electrical motors or laws governing fuel efficiency and emissions of internal combustion engines).
- Pumping plants are a major investment and may be in operation for 40 years or more.
- No one pumping plant (not even a thousand pumping plants) on its own is causing a problem (i.e. lack of electricity in California)
- Pumping water for irrigation can obviously be considered a benefit to society considering improved yields and is encouraged where applicable.
- The problem is the cumulative effect of all pumping plants. Electricity use for water pumping for one California utility in 1997 was in the range of 3 billion kWh/year, representing 81% of all energy use in the

agricultural sector for that utility. Current data from one large-scale energy efficiency program indicates the following average pumping plant efficiencies for various sizes of electric-powered water pumps (all types).

Table 1. Average Overall Pumping Plant Efficiency in California from Pump Tests Performed During the Agricultural Peak Load Reduction Program - June 1, 2001 - February 5, 2002 (all pump types and uses represented)

Horsepower Range	Number of Tests	Average Overall Pumping Plant Efficiency
5 - 20	386	43
21 - 50	701	53
51 - 200	926	59

Improving the efficiency of pumping plants by three percentage points could save in the range of 90 million kWh/year for that utility alone. More importantly, even ignoring any desirable improvement in pumping plant efficiency, moving a significant percentage of the agricultural pumping demand away from the peak periods of overall statewide demand might forestall the need to construct power plants or avoid blackouts.

- Obviously production agriculture has created an economic, political, and cultural environment in California.

It seems clear that improving energy efficiency in agriculture can be characterized as a non-point source problem.

IMPROVE MANAGEMENT OR HARDWARE?

Given that energy is used by hardware, another aspect of the problem statement is whether to address the hardware itself or the management of hardware. The question can be framed analytically for production agriculture by equations [1], [2], and [3]. They represent a simplified method for estimating energy use for an agricultural pumping plant used for irrigation.

$$\text{kWh/yr} = \text{kWh/AF} * \text{AF/yr} \qquad [1]$$

where:

kWh/yr = kiloWatt-hours of electricity used annually

kWh/AF = kiloWatt-hours required to pump an acre-foot of water through the pumping plant

AF/year = acre-feet of water pumped per year

$$kWh/AF = 1.0241 * TDH / OPE \qquad [2]$$

where:

kWh/AF = kiloWatt-hours required to pump an acre-foot of water through the pumping plant

TDH = total dynamic head developed by the pumping plant in feet

OPE = overall pumping plant efficiency as a decimal (0 - 1.0)

1.0241 is the conversion constant for water at standard conditions

$$AF/yr = CL + (Ac * (ETc - PPTeff) / ((1 - LR) * IE)) \qquad [3]$$

where:

AF/yr = acre-feet pumped annually for irrigation of a crop

CL = conveyance losses in the irrigation system in acre-feet

Ac = acres irrigated

ETc = annual net crop water use as acre-feet/acre

PPTeff = annual effective rainfall (rainfall used for crop ET)

LR = required leaching ratio as a decimal (0 - 1.0)

IE = irrigation efficiency as a decimal (0 - 1.0)

Using a superficial analysis of equation [2] one can state that reducing energy use is partially a problem of <u>hardware</u>. That is, equation [2] implies that overall energy use is governed both by the design of the system (which leads to the required TDH) and by the design of the pumping plant (which leads to the OPE).

In addition, by equation [3] it is seen that <u>management</u> of the overall activity, as represented by IE, is equally as responsible for overall energy use. As IE decreases (i.e. poor management), AF/yr increases and thus, the more energy required. Note also that OPE in equation [2] is partially governed by maintenance of the pumping plant. Thus, poor management can lower OPE, which leads to higher overall energy use.

However, it is also true that the design of the pumping plant affects OPE. Management is involved in the design (i.e. choice of hardware) of a system. Thus, some might argue that the distinction between management and hardware is somewhat artificial.

However, where the distinction is not artificial is when the decision is made to address what type of hardware is <u>available</u> to the manager. This is the difference between addressing supply-side issues (e.g. available hardware, design expertise) versus demand-side issues (e.g. choice, use, and maintenance of hardware). Examples of this are the implementation of Corporate Average Fuel Efficiency standards that the automotive industry must meet, or minimum electric motor efficiencies for motor manufacturers set in the Energy Policy Act of 1992.

PROGRAM FORM TO ADDRESS NPS

Having established that energy efficiency is a non-point source problem, the question is how should this type of problem be addressed? As just pointed out, NPS are substantially the result of management actions, either design, maintenance, or operational. Thus, if a non-point source problem is to be fixed, there must be a change in management action - people have to change the way they think and act.

Three things have to happen to make someone change in the context of solving some problem of behavior:

1. He/she must see that there is a problem and that that problem is their responsibility.

2. He/she must see that there are solutions available for the problem.

3. He/she must have resources to implement the solution.

Thus, programs that address non-point problems at the end-user level generally have three components. These three components exist at both the program design and the program implementation level. They are:

1. Problem awareness - At the implementation level the actors need to see that there is a genuine problem and that they are (wholly or partially) responsible for solving that problem. "Seeing" implies that the actor not only takes responsibility but also has tools by which specific problems can be identified. At the program design level it is essential managers a) identify the real (or priority) problem(s), b) implement sufficient education and public outreach, and c) ensure that engineering/analysis tools are available for individual problem identification.

2. Solution awareness - At the implementation level the actors need to see that there are solutions to the problem(s)- that is, something can be done. At the program design level it is essential that managers identify viable solutions ("targeted technologies"). Viability means more than just the ability to improve energy efficiency. It must be economical, reliable, practical, widely adaptable in the field, and understandable.

3. Resources - At the implementation level the actors need the time, money, and expertise to a) identify their problem(s), b) identify the most applicable solution(s), and c) install the solution(s). It is essential that program designers recognize when aide in the form of engineering services, low interest loans, and outright grants are required.

To paraphrase the above discussion: if a manager doesn't see a problem, or doesn't believe it is his/her problem, nothing will change – we need problem awareness. If the manager sees the problem but doesn't see that there is anything that can be done, nothing will change – we need solution awareness. However, even if the manager sees the problem and has a solution, nothing will be done unless resources are available to implement the solution.

As a practical example of how the above three components interact consider low overall pumping plant efficiency for a pump in place. The first concern is making the pump operator aware of the situation. Thus, the program will advertise the availability and encourage the operator to have the pumping plant tested for efficiency. The pump test identifies the current overall pumping plant efficiency. Thus, the operator is given objective data with which to make a decision.

The program should also develop awareness of solutions. In this situation most every pump operator will know that a pump can be repaired. However, it is possible that part of the problem is excessive drawdown in a well. The owner may not be aware of actions that reduce encrustation or otherwise improve specific yields.

Economics of the solution are also important. Equations [1], [2], and [3] can be packaged in different forms so that operators can develop valid benefit/cost ratios for pump repairs. Note that a necessary part of the package is information concerning achievable levels of efficiency.

Finally, the program may have to provide resources. These may be in the form of direct cash rebates, either to defer some of the cost of the pump test or a subsequent pump repair. Time can also be considered as a provided resource as the tools developed for economic analyses, including developed data, save the operator time in making a decision.

Programs that address NPS may also include development of new solutions if current solutions are not satisfactory. This could include development of new management techniques (e.g. improved irrigation event management leading to increased irrigation efficiency) or research to develop new hardware.

"MARKET TRANSFORMATION" AS A SOLUTION TO THE ENERGY EFFICIENCY PROBLEM

"Market transformation" has been a popular term with the California Public Utilities Commission (CPUC) in the recent past while California was moving towards deregulation. CPUC, in light of the advent of a "fully competitive" market, was trying to move away from energy efficiency programs that were mainly "buying the resource". That is, they wanted to reduce the use of programs that primarily were designed to provide cash incentives to install energy efficient equipment. In terms of the preceeding discussion, it was an indication that existing programs provided resources, but possibly were deficient in improving problem and solution awareness.

The major problems of incentive-based programs were seen to be:

1. Substantial amount of "free riders". That is, people were participating in the program that would have installed the measure without the incentive. Thus, the cost of the energy efficiency program was artificially inflated.

2. Questions of persistence- that is, would energy-efficiency behavior persist in the absence of the incentive? For example, if an incentive grant for pump repairs is discontinued, will the rate of pump repairs stay steady or decline?

Market transformation programs would be specifically intended to change the way people think and act. Thus, after market transformation, people would buy and act in an energy efficient manner without being paid to do so. The goals of moving towards market transformation programs were:

1. More efficient programs in terms of impact for dollars spent.

2. Creation of lasting energy efficient behavior- that is, increased persistence.

In current terminology, market transformation activities include identifying barriers to technology adoption and implementing programs that will serve to break down those barriers. That is, if there is technology available that will improve energy efficiency and it is not being adopted (or at least not being adopted without some form of "bribe"), what can be done to increase adoption? Further, can this adoptive behavior be made to persist without the program?

Commonly identified market barriers include:

1. Information and search costs - the costs of searching out new technologies and learning enough about them to make an informed decision.

2. Asymmetric information - this barrier surfaces when information concerning a technology cannot be verified by disinterested third parties. New technology is most often marketed by those wishing to profit from it. The marketers may be the only ones with information and test data regarding the product. The end-user may not trust this information.

3. Performance uncertainty - can the true benefits and costs be identified?

4. Hidden costs - this is different than performance uncertainty in that performance uncertainty relates to the known factors while hidden costs relate to the unknown, and thus unevaluated, factors.

5. Unstable investment environment – this may involve overall business economics, overall energy costs, or the relation between costs of alternative energy sources.

Another way to state the problems identified in points 3 through 5 above is "risk". There is a risk to changing the hardware and/or management of an activity. A manager must not only determine the benefit/cost ratio of known variables but also try to evaluate the unknown.

Also, note the repeated theme of the need for complete and valid information in points 1 – 4 above. Thus, desirable characteristics of market transformation programs would seem to be similar to those programs designed to address standard non-point source problem. In fact, programs that address non-point source problems are very much "market transformation" programs. Both are trying to change the way people think and act.

However, the concept of market transformation, in the context of energy efficiency, extends and clarifies some of the desirable aspects of NPS program design and implementation. The market transformation concept specifically identifies both supply side and demand side actors. The supply side of the equation includes both those designing and manufacturing the individual components of a solution as well as those who package and sell the components to the end-user (the demand side). Thus, within the basic components of problem awareness, solution awareness, and appropriate resources, the following factors must also be addressed:

- Educational programs concerning the problem and the solution need to involve all "actors" including end-users, consultants, suppliers, and lending institutions. At the very least this will promote efficiency in program implementation as suppliers, consultants, and lending institutions become involved in encouraging energy conservation. To the extent that these actors become involved and existing communication links utilized, the efforts (and the costs) of the Program Manager can be reduced. It should be obvious that different methods/levels of education are appropriate for different audiences.

- The individual program components should not become a necessary part of the adoption process. The intent is to transform the market so that it acts on its own. Thus, if the intent is for supply side actors (i.e. a pump repair company) to help sustain the solution adoption (i.e. provide pump tests to identify low efficiency pumps) after the program stops, the supply side actor (who acts independently of the Program) must be part of the program design. For example, while the end-user should see (and use) the pump test as a source of information, the supply side should see (and offer) the pump test as a (continuing) source of revenue- the pump test can lead to a pump repair.

- There must be a monitoring component. That is, there must be some measure of success of the Program and some control function to indicate when and where changes have to be made in the Program. Also, there should be some level of the monitored measure that indicates Program objectives have been reached. If the goal of a market transformation program is the market acting on its own, then it is logical that sooner or later the Program can be (should be) discontinued.

CONCLUSION

In summary, it is argued that energy efficiency in agriculture is a long-term, non-point source type of problem. "Fixing" non-point source problems generally involves changing the behavior of resource managers. These types of problems are generally addressed by programs involving three components: 1) problem awareness, 2) solution awareness, and 3) targeted resources.

Market transformation as a concept is directly related to non-point source programs. Market transformation also involves changing the way end-users think and act, just as the traditional NPS program. However, the concept explicitly recognizes that supply side actors must be involved in terms of changing the services and hardware being offered to the end-user. This is especially true if the long term objective is the market to act on its own to ensure high-energy efficiency.

Involving the supply side also takes advantage of existing communication links and hopefully can lead to lower program costs.

SUGGESTED RESEARCH ON THE EFFECT OF CLIMATE CHANGE ON CALIFORNIA WATER RESOURCES

Maurice Roos[1]

ABSTRACT

Quite significant changes in climate are being predicted for the latter part of this century due to global warming. The changes would be the result of increases in greenhouse gases from human activities, such as carbon dioxide, methane, and other trace gases. These potential changes are expected to affect many of our water resources systems. Some of the more important changes would be temperature increases which would raise temperate zone snow levels and change the pattern of runoff from mountain watersheds, thereby affecting reservoir operation. Other consequences would be sea level rise which could adversely affect the Sacramento San Joaquin River Delta, source of major water exports for the State; possibly larger floods and more extreme precipitation events; and changes in the water requirements of crops.

By and large, reservoirs and water delivery systems and operating rules have been developed from historical hydrology on the assumption that the past is a good guide to the future. With global warming, that assumption may not be valid. This paper will briefly look at the major factors affecting water resources systems and go on to suggest eleven priority items of research. The emphasis will be on items important in California and other western states.

In view of these forecasts of a significant change in future climate, with the author's knowledge of the existing water resources system in California, an analysis of potential effects and a list of higher priority research items has been developed. In summary, the list is as follows, and will be explained in more detail subsequently in the paper:

- --Monitoring of hydrologically important variables
- --Test operation of the Central Valley Project and State Water Project system with modified runoff
- --Modeling of future precipitation
- --Update depth-duration-frequency rainfall data
- --Evaluate Golden Gate Tide Gage datum
- --Catalog sea level trends along the coast, in San Francisco Bay and the Delta
- --Check for recent changes in evapotranspiration
- --Estimate future changes in evapotranspiration and crop water use.

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--Evaluate effect on major multipurpose flood control reservoirs
--Water temperature modeling in major reservoir/river systems
--Effect of climate change on regions adjoining California, such as the Colorado River and the Pacific Northwest

POSSIBLE CHANGES AFFECTING WATER RESOURCES SYSTEMS

Most of the forecasted climate changes by year 2100, the end of the century, due to the increase in greenhouse gases have been developed by the Intergovernmental Panel on Climate Change (IPCC). The IPCC was jointly established in 1988 by the World Meteorological Organization and the United Nations Environment Programme to study climate change. The IPCC has issued several reports since 1990 outlining possible global warming and the effects as a result of increased amounts of carbon dioxide, methane, and other trace gases originating from human activities.

A good assessment of the state of research on the potential consequences of climate change on water resources in the United States, including what is known and what is not known, is the report of the National Water Assessment Group for the U. S. Global Change Research Program (Gleick and Adams, 2000).

The most recent IPCC Working Group I Summary Report, in its third assessment (IPCC, 2001), projects a 1990 to 2100 average surface temperature increase of around 3 degrees C, with a range of 1.4 to 5.8 degrees. The different scenarios cover a wide range of assumptions about the rate of future increases in greenhouse gases and the amount of temperature forcing in the climate system. The increase in global temperature during the 20th century was estimated to be about 0.6 degrees C. much of which occurred by 1940, and a recent significant increase after 1980 which is believed to be primarily of human origin. Because of warmer temperatures, some increase in global evaporation and therefore precipitation is projected for the 21st century, more likely at higher latitudes. For hydrology and water resources, precipitation is the most important variable; rainfall changes in specific California regions cannot be well defined by the current general circulation climate models of the atmosphere.

Sea level (IPCC,2001) is projected to rise around 0.5 meter (1.6 feet) by 2100, with a range of 0.1 to 0.9 meters (0.3 to 2.9 feet). The rate during the 20th century appears to have been around 0.2 meters (0.7 feet) with a range of 0.1 to 0.25 meters (0.3 to 0.8 feet). The 0.2 meter figure is consistent the historical trend at the Golden Gate tide station, although it is possible that tectonic movement, or settlement, has influenced the stages there.

There is a general expectation that a warmer climate would lead to more intense precipitation events, thereby causing somewhat bigger floods and more intense convective storms, thereby affecting the rainfall statistics used for storm drainage design. The IPCC report rates prediction confidence in more intense precipitation events as "very likely, over many areas".

The increase in carbon dioxide, from the current 370 ppm to perhaps 600 or 700 ppm, is expected to be beneficial to plant growth on many food crops, provided the water supply is adequate. To some extent, higher carbon dioxide concentrations in the air could partly offset the higher water use (evapotranspiration) resulting from warmer temperatures.

All of these projected changes, as well as some not yet identified, are likely to affect the hydrologic cycle and the water resources of California.

WATER SUPPLY RELATED RESEARCH

Probably the most significant change, which is judged fairly certain in the next 100 years, is a change in temperate zone mountain runoff patterns. Even if precipitation remains the same, the rise in temperatures means less snow, less snow covered area, and winter rain to higher elevations. The higher snow levels during storms, about 450 meters (1500 feet) under the 3 degree median projection, would produce more winter runoff. Less spring snowmelt would make it more difficult to refill winter reservoir flood control space during late spring and early summer of many years, thus reducing the amount of water deliverable during the dry season. Lower early summer reservoir levels also would adversely affect lake recreation and hydroelectric power production, with possible late season temperature problems for downstream fisheries.

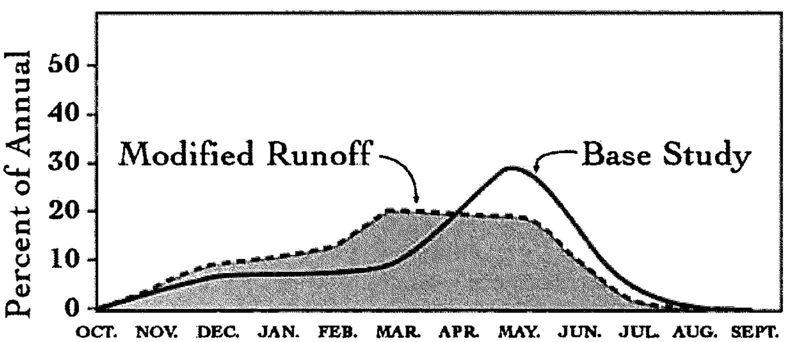

Figure 1. Comparison of historical and modified runoff from a mid-Sierra snow runoff watershed, assuming 3 degrees C warming

It is possible, if precipitation increases and the mountains are high enough, for the volume of April through July snowmelt to increase. In California, this could happen in the higher elevation southern Sierra, where about 70 percent of the historical April 1 snow zone area would remain with a 450 meter (1500 feet) rise. In the northern Sierra, however, the same rise in snow levels would reduce estimated snow covered area by over 70 percent, so a reduction in spring snowmelt is virtually certain. In fact, there has been such a trend during the last 50 years.

Monitoring of Hydrologically Important Variables

The first research need is good hydrometeorologic monitoring. Regular, consistent and sustained measurements of hydrologically important variables are essential to track what is happening and to verify model predictions. This means continuing measurements of variables such as precipitation and other climate data, snowpack, streamflow, and ocean and Delta tide levels. Emphasis should be in the locations where significant change is expected, for example the mountain snow zone. The National Weather Service, in its reorganized Climate Services Division, is developing a climate reference network of 250 high grade weather stations to be a national benchmark for long term climate monitoring. That would be a good start, but only 5 of these are likely to be located in California. Scripps Institution of Oceanography, in cooperation with the Yosemite National Park and others, has recently installed a number of new snow measurement and meteorological instruments in the Park, with high hopes that these will be operating for the long term. But more thought should be given to several networks along gradients from east to west and north to south, and across climate zones.

Test Operation of the Central Valley Project – State Water Project System

These two big water projects furnish about 30 percent of California net water demand for agricultural and urban use. The major reservoirs of these two projects are located on watersheds (Trinity, Sacramento, Feather and American) likely to see large shifts in runoff patterns as a result of rising snow levels. At least 50 years of monthly hydrology are suggested as a minimum length for comparisons. Currently many studies are made with the 1922 – 1994 historical period of 72 years. This longer period includes simulation during the two major 6-year historical droughts , 1928-1934 and 1987-1992. The operation model of choice would be the CALSIM model jointly developed by the Department of Water Resources and the Bureau of Reclamation.

The test evaluations would logically proceed in two stages: (1) a simplified run involving approximate adjustments to major project reservoir inflows with the changes anticipated with global warming, and (2) more detailed studies involving all major facilities, including local and upstream power reservoirs. Initial studies should focus on the assumption of precipitation similar to the historical amounts, except warmer. Later studies could try some of the projected precipitation scenarios derived from a new generation of atmospheric general circulation models (GCMs).

Modeling Future Precipitation

Future precipitation is probably the most important variable influencing water resources and water supply. It is also the most difficult to predict at the regional and watershed levels. It should be no surprise that research should continue into modeling likely future precipitation in enough detail in time and space to feed into individual watershed runoff models. We need to support the University of California and National Laboratory experts in analyzing results of newer GCM modeling by the modeling centers of the world as they apply to California and other western states, especially in simulating historical precipitation and predicting future precipitation. And feedback from these expert researchers to the GCM modelers should be encouraged.

Depth-Duration-Frequency Rainfall Data

Support of the processing and dissemination of up-to-date rainfall depth-duration-frequency data is important to incorporate extreme events of recent years and as more of these extreme events are expected in future years. These data are some of the most valuable rainfall statistics, widely used by engineers and designers for storm drains, culverts, roofs and host of works. These are the curves, for example, which show that a 1 in 25 year storm can produce 2 inches (50 mm) in 2 hours at a particular place. Continual updating will gradually incorporate expected storm intensity increases as a result of climate change so that structures built with their guidance will not be out of date with less protection than intended.

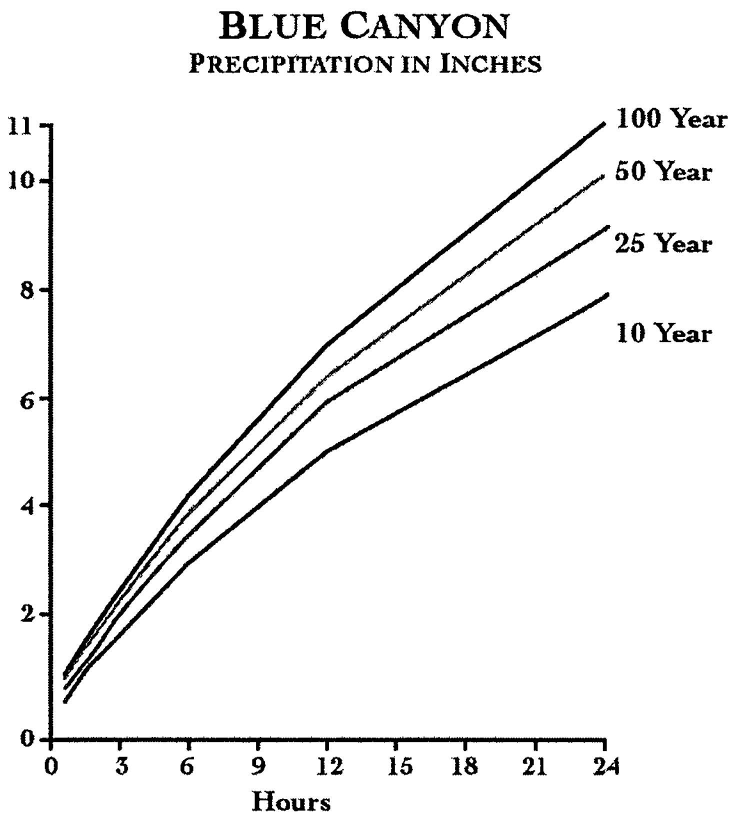

Figure 2. Chart of depth-duration-frequency statistics for Blue Canyon

SEA LEVEL RISE

California is not generally as vulnerable to sea level rise as some of the eastern and southern states, which have lower shorelines. Since California is tectonically active, it is the combined effect of geological change, rising or falling land, and global sea level rise that matters. But there are areas, such as San Francisco Bay and the Sacramento San Joaquin River Delta, which are vulnerable. Major water projects have been built to convey water from areas of plenty to the drier central and southern portions of the State. The tidewater region of the Delta is the hub of major water transfer; water quality there can be affected by salinity intrusion from the Pacific Ocean via San Francisco Bay.

Reservoirs in the north provide water storage and flood control. Water is then released downstream to meet Delta and fishery needs and to be exported by large pumps in the southwestern corner of the Delta. During low flow months additional water is released to repel ocean salinity incursion in the western Delta as a hydraulic barrier to preserve suitable fresh water quality within the estuary and for export. Much of the land within the Delta is below sea level protected by levees, many of which are on weak peat soil foundation.

Evaluate the Golden Gate Tide Gage Datum

Rising sea level is not only a concern along the coast but to the hub of California's water transfer system in the Sacramento San Joaquin Delta. Rising ocean levels could affect water transfer across the Delta from north to south by decreasing the stability of Delta levees. The second factor is more salinity intrusion, which would gradually degrade the quality of export supplies or require more precious fresh water releases from upstream reservoirs to repel ocean salinity.

Since the Golden Gate tide gage is the key reference for so many sea level determinations (i.e., the central California coast, the Bay, and the Delta), it is essential that an accurate determination of its vertical stability be made, checking for long term vertical movement of the datum. Tools may now be available by use of highly precise space geodetic techniques, which can measure very small changes in vertical elevation. The gage has shown an apparent rising sea level trend of about 0.2 meters (0.7 feet) per century over the past 80 years, but we can't be sure if this is real or due to settlement of the pier or tectonic movements in the underlying rock. The measurements should proceed in three stages: (1) compare previous precise leveling where the Golden Gate gage can be compared to nearby benchmarks on solid rock; (2) investigate whether the GPS system can be used for a precise determination of the Golden Gate tide gage datum and its changes, if any, over time and how long a record would be needed to give confidence of a measurement which could be less than 2 mm (.007 feet) per year; and (3) perform the measurements of actual tide gage datum over a period of time, probably years. The National Geodetic Survey does this kind of work.

Catalog Sea Level Trends Along the Coast

The objective of this proposed study would be to catalog all available tide station data along the California coast, in San Francisco Bay and in the Delta. Long term data is needed, at least 20 years and preferably 50 years, to look for apparent trends in annual sea level. Presumably an average of San Francisco at the Golden Gate and San Diego can be used to represent global sea level rise. Departures from that could be due to tectonic activity or possibly some other effect such as oil and gas extraction, or perhaps consolidation of deep sediments (as in the Delta, for example). The catalog would establish a useful base to guide government and developers in the coastal zone. The San Francisco Bay Conservation and Development Commission (1988) and the California Coastal Commission (2001) have both made some studies of this matter.

CHANGES IN WATER REQUIREMENTS

There are at least two factors which could change water use by vegetation. As a general rule, warmer temperatures mean more evapotranspiration. But higher carbon dioxide levels tend to reduce water consumption, at least in laboratory tests. Most observers expect the net change to be somewhat higher water requirements but not as high as would otherwise be expected from temperature change considerations.

Recent Changes in Evapotranspiration

Since carbon dioxide concentrations have increased about 17 percent over the past 40 years (Keeling and Whorf, 2001), one wonders whether there is any noticeably effect on evapotranspiration, especially the measured reference ET of grass. During the 1960s there were a number of well measured grass lysimeter plots in various locations in California, measuring directly, by weight changes, the water consumption of grass. The University of California, in cooperation with the Department of Water Resources, should reinstall or reoperate former lysimeters at Davis and Five Points (on the San Joaquin Valley west side). Since there is variation from year to year and day to day, depending on weather conditions, it will probably take 3 to 5 years, perhaps 10 years, to see if there is a noticeably change from the measurements 40 years ago. It is possible that higher carbon dioxide is a factor in the continuing improvement in crop yields.

Estimate Change in ET and Crop Water Requirements

Knowledgeable experts in plant water consumption at the University of California and the Agriculture Extension Service and land and water use analysts in State and federal government should estimate likely future ET rates for major crops. To do this, they will need to obtain reasonable median projections of weather in 2050 and 2100 from the GCM climate modelers. This would be primarily temperatures, both average and maximum and minimum, in our dry summer climate, but could include projections of monthly rainfall changes if significant, and should include projected carbon dioxide levels. One would expect that higher water consumption because of increased temperatures will only be partly offset by carbon dioxide based reductions. The result would be slightly higher water requirements, probably varying somewhat by crop type. A complicating factor is possible shifts in the growing season of annual crops. For example, because of less frost risk, tomatoes might be planted earlier when the sun angle is not as high.

OTHER ITEMS OF RESEARCH

There are several other item of potential research which don't fit into the previous headings. These are discussed below.

Conduct a Systematic Review and Evaluation of the Effect of Global Warming on Major Multipurpose Flood Control Reservoirs

In a warmer world, some increase in the intensity of major precipitation events could be expected because saturated warmer air can hold more water vapor than cooler air. More intense precipitation would generate larger floods. Another factor on streams draining high mountain areas is that many storms now produce a mix of rain with snow on the higher parts of the watershed. A warmer climate means a greater proportion of storm precipitation is likely to be rain, producing more direct rain runoff.

Currently reserved flood control space in major multipurpose reservoirs during the winter may not be adequate for the larger storms. As a result, the degree of protection would gradually shrink. Additional flood control space or downstream channel capacity is likely to be expensive on our major rivers.

As GCM models are developed and improved, their precipitation results should be analyzed to see if there is a consistent trend for more intense storms and precipitation events in California. The model precipitation can then be entered into watershed runoff models to assess the higher risk. A careful analysis of historical trends during the last 30 years or so may be useful too.

Water Temperature Modeling in Rivers

Warmer air and less snowmelt will make it more difficult to maintain rivers cold enough for cold-water fish, including anadromous fish like salmon. This could create difficult problems for some salmon, such as the winter run, where fish spend the warm season in the streams. There may be similar problems for juvenile steelhead too. On most California rivers large foothill reservoirs provide some temperature control for downstream reaches.

There are some existing models of water temperature in reservoirs and downstream rivers. These models may need improvement as the job of maintaining suitable temperatures becomes more difficult. Analysis of selected foothill reservoirs and rivers is suggested to see what a different pattern of inflow and higher temperatures would do. Some new temperature modeling is anticipated as part of the Oroville power plant relicensing during the next several years. A logical extension would be to apply the new Lake Oroville and Feather River temperature models under a changed climate and runoff regime.

Effect of Climate Change in Other Regions

The Colorado River, which drains a huge area of the American Southwest, is a very important component of California water supply, especially in the south. Earlier studies (Nash and Gleick, 1993) have indicated a high probability of a change to less runoff. Since the Colorado River is already fully subscribed, if not oversubscribed, a reduction in average runoff could affect California's water supply as well as that of the other Colorado River Basin states. Hydroelectric power too could be affected, especially generation at Glen Canyon and Hoover dams due to change in reservoir levels.

California's long-term entitlements to Colorado River water are 4.4 million acre-feet (5.4 billion cubic meters) per year. In recent years, California's net diversions have been as much as 5.2 million acre-feet and the State is being forced to reduce its diversions as water demands in the other states build toward use of their entitlements. Climate change in the watershed could acerbate the water supply situation. It is also possible that a wetter scenario could improve Colorado River runoff, even to the point of generating more flood problems on the lower river.

California depends on the Pacific Northwest, including the Columbia River system, for about 10 percent of its electric energy supply. As we saw during 2001, when Columbia River runoff was down, there is an impact on California's electric power supply and reliability. In conclusion, the effects of climate change, especially precipitation, in the adjoining Pacific Northwest and Colorado River watershed would have an impact on California—on electricity for both regions and on water supply for the Colorado.

It is anticipated that new research and studies on runoff and water supply in both of these regions will be forthcoming by interested regional parties. It is recommended that the California Energy Commission and the Department of Water Resources monitor results of these studies as they are completed and try to assess what they might mean for California water supply and electric energy imports.

SUMMARY

The preceding are 11 areas of research on the effect of global warming on water resources systems in California that the author feels should have priority. There are many more items which could be added. In a report on potential water resources research ideas for a globally warmer world now being prepared by for the California Energy Commission Public Interest Energy Research (PIER) program, some 35 items were identified. However, completion of the work suggested herein would be an excellent start in adapting to a potentially different climate.

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A CONCEPTUAL SALT BUDGET FOR CHARACTERISTIC WATER DISTRICTS IN THE WESTERN SAN JOAQUIN VALLEY, CALIFORNIA

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Manucher Alemi[1] Nigel Quinn[2]

ABSTRACT

When considering salinity management in the San Joaquin Valley there is a tendency to take a broad approach and assume that all water districts have similar hydrologic characteristics, employ the same suite of technologies to address the soil management to sustain crop productivity, and maintain water quality. However, this article shows that water districts with different characteristics face different salt management challenges within the Valley. The three water districts chosen in this study are Westlands Water District, Panoche Water District, and West Stanislaus Irrigation District. They each have different histories, differ in the source of their irrigation water supply and differ in the physical and institutional constraints on their drainage return flows. These factors can have an impact on salt budget within each district. Approximate values of salt budget components for the three irrigation districts are presented.

INTRODUCTION

The westside of the San Joaquin Valley includes the San Joaquin River Basin and the Tulare Lake Basin with 2.4 million acres of mostly irrigated agricultural land. Historical drainage discharged to the San Joaquin River was about 55 thousand acre-feet (TAF) per year from an estimated 50 000 acres of land with installed subsurface drain. The Tulare Lake basin at the southern end of the Valley has no natural drainage outlet and annually discharges 15 TAF of drainage water to evaporation ponds. In areas without installed subsurface drains and no or inadequate natural drainage, salts accumulate in the groundwater aquifer and the water table may rise over time.

Farmers and water districts within the Valley have adopted various irrigation improvements and drainage reduction measures to manage salts and trace elements in response to regulatory requirements to protect environmental

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resources. These actions have resulted in significant reduction in the volume of drainage water discharged to the San Joaquin River (from 57 TAF in 1990 to about 30 TAF in 2000). Acreage of and discharge rates to evaporation ponds have also been reduced, since selenium induced teratogenesis in wildfowl embryos was found. For example, the evaporation pond acreage reduced from about 6 000 acres in 1990 to about 4 000 acres in 2000. The irrigation and drainage management measures and some fallowing have been the primary mechanisms for water conservation and reduction of contaminant loads to water bodies in the region. These measures coupled with separation and safe disposition of salts from the root zone or groundwater aquifer could result in sustainable soil and water quality. Drainage reduction and separation of salts from drainage help maintain agricultural productivity and water quality, and have the benefit of creating new sources of water supply.

BACKGROUND

One of the principal subsurface geological features of the San Joaquin Valley is the Corcoran clay formation. Formed as a lakebed about 600 000 years ago, this clay layer ranges in thickness from 20 to 200 feet and underlies most of the westside of the Valley. The Corcoran clay divides the groundwater system into two major aquifers – a confined aquifer below and a semi-confined aquifer above. The Diablo Range to the west is comprised of complex, uplifted sediments, which are composed predominantly of sandstones and shales of marine origin. These sandstones and shales contain salts, as well as trace elements such as selenium. With decreasing elevation from the west to east, soil textures become finer. These fine textured soils are characterized by low permeability and increased concentrations of water-soluble solids, primarily salts and trace elements. Furthermore, there are discontinuous clay lenses at different depths within the semi-confined aquifer, resulting in localized shallow groundwater. Three water districts are addressed in this paper, chosen for their distinct applied water and drainage characteristics. For example, West Stanislaus Irrigation District (WSID) can release drainage water to the San Joaquin River, Panoche Water District (PWD) has limited access to release drainage water to the San Joaquin River, and Westlands Water District (WWD) has no means for drainage water removal. Additionally, the districts differ in the quality of their irrigation water supply. WSID receives medium-quality water from a combination of Central Valley Project (CVP) and San Joaquin River water, PWD receives high-quality water from CVP water, and WWD receives some high-quality CVP water as well as lower-quality groundwater (depending on the availability of CVP water).

Panoche Water District

PWD is located in the northern part of the Valley. Soils in the PWD can be characterized as very deep, nearly level to gently sloping, and moderately to poorly drained soils that are drained to some extent on low alluvial fans. PWD began receiving its first water from the San Joaquin River in 1947 under interim contracts. In August 1949, contract negotiations began with United States Bureau of Reclamation (USBR) for the CVP water, and a long-term contract was established in 1955. Currently PWD has a contract for the delivery of 99 TAF of water per year.

PWD receives a yearly average of nine inches of precipitation, most of which falls during the months of November through March. Reference evapotranspiration measured in PWD averages 55 inches per year.

PWD's service area is approximately 38 000 acres. Actual irrigated land is approximately 36 000 acres. The five major irrigation methods in PWD (in order by highest to lowest acreage, in 1997) are (1) sprinkler (hand move) and graded (surface ¼ mile: siphon tube); (2) graded (surface ¼ mile: siphon tube); (3) trickle (surface and subsurface); (4) graded (surface ¼ mile: gated pipe); and (5) sprinklers (hand move).

Historically, drainage water discharged from the PWD service area flowed through the wildlife refuges and then into the San Joaquin River. As a result of environmental limitations, it became necessary to remove the drainage water from the wildlife areas. Therefore, in 1996 the drainage water (from PWD and other districts in the area) was rerouted through a newly constructed channel that bypasses the refuges. The water now travels through the bypass, and portions of the San Luis Drain and is then discharged to the River. Due to selenium load limitations imposed by the Central Valley Regional Water Quality Control Board, a large portion of the subsurface drainage water (about 4.5 TAF annually) collected by PWD is blended back into the delivery system at several different locations. Additionally, as of 1997, PWD does not allow tailwater (surface drainage water) into its drainage or distribution system. The water is to be retained on farm by the individual water users where it is pumped back into the farmer's delivery system and redelivered to the field. This promotes more efficient water use and reduces the volume of drainage water conveyed to PWD's drainage works.

West Stanislaus Irrigation District

WSID is located in the northern part of the Valley. WSID was formed in 1920 and began diverting water from the River in 1929. Diversions increased from 12 TAF to a maximum of 113 TAF in 1984. After construction of the Friant Dam in 1942, the quantity of water available to WSID users decreased, and the quality of water became increasingly saline. WSID signed a contract with USBR in 1953 to receive 20 TAF of water from the CVP, which was increased to 50 TAF in 1976.

WSID receives a yearly average of ten inches of precipitation, most of which falls during the months of November through March. Temperatures range from an average monthly low of 17°F in winter months, to a high of 111°F in summer months. Reference evapotranspiration is 57.0 inches/year.

WSID total acreage and irrigable area is 24 500 and 21 500 acres, respectively. White Lake Mutual Water Company also irrigates approximately 5 000 acres under an agreement. The soils are moderately to well drained, and permeability is low to medium. The majority of agricultural acreage is used to produce almonds, apricots, dry beans, green beans, melons, peas, walnuts, wheat, and can tomatoes. The five major irrigation methods (in order by highest to lowest acreage) are furrow (gated pipe), border (gated pipe), sprinkler, and micro/drip. Drainage water flows back into the River, either directly into natural channels or onto riparian land adjacent to the River.

Westlands Water District

WWD is located in the southern half of the Valley. It is approximately 15 miles wide and 70 miles long. Formed in 1952, it included approximately 376 000 acres. WWD later merged with its western neighbor to form the current 604 000-acre district with an irrigable acreage of 567 800 acres. In 1968, USBR created a 40-year contract with WWD, providing 900 TAF of CVP water per year. Then in 1986, a new agreement added an additional 250 TAF of water per year[3].

WWD receives a yearly average of seven inches of precipitation, most of which falls during the months of December through March. The mean annual temperature is 62°F. Reference evapotranspiration is 58 inches/year.

Approximately 60 different crops are grown in WWD. In 2000, cotton, tomatoes, wheat, and almonds comprised almost 64% of the 564 200 cropped acres. WWD crops are irrigated using groundwater and CVP water. In the 1990's, approximately 40% of all crop acreage was irrigated by surface irrigation systems (furrow and/or border irrigation), approximately 20% was irrigated by pressurized systems (sprinklers and/or drip irrigation), and approximately 40% was irrigated by a combination of surface and pressurized systems.

Currently, there is no outlet for drainage water. Some farms reuse subsurface drainage water. Some lands are fallowed annually due to inadequate water supplies and drainage-related problems. Shallow groundwater levels are typically highest in April after preirrigation and lowest following the cropping season in October after crops have extracted a portion of the shallow groundwater.

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[3] Water Management Plan, 1999.

FLOW AND SALT MODELING

Numeric models have been developed to understand and predict groundwater levels and flow on the West Side of the San Joaquin Valley. These models provide a tool by which the effects of future changes in water use and management might affect groundwater levels. However, current models don't integrate water salinity data with flow to determine components of a salt budget for the west side of the Valley. In 1990, the United States Geological Survey created a three-dimensional groundwater flow model that was calibrated for a portion of the Valley[4]. This model was used to evaluate alternative management approaches over a 50-year period. The model evaluated groundwater responses to: land retirement; reduced recharge; increased groundwater pumping; and simultaneous reductions in recharge and increases in groundwater pumping. Results from the model showed that land retirement was an effective management tool on a local scale but not on a regional scale and that management of the semi-confined aquifer as a salt sink could be maintained from 40 to 275 years. This model represented one of the first comprehensive, calibrated attempt to characterize the hydrology of the Valley.

Following the Belitz and Phillips model, the USBR completed the Central Valley Groundwater and Surface Water Model (CVGSM). CVGSM is a specific application of the Integrated Groundwater and Surface Water Model (IGSM)[5]. The CVGSM model simulates water inflow and outflow using various specific parameters including crop type, soil type, rainfall, applied irrigation water, potential ET, and irrigation efficiency. Additionally, stream-flow, stream-aquifer interactions, and unsaturated flow are also simulated. However, the model does not consider salt transport or loading. Presently CVGSM is being updated and improved as the WESTSIM model and will include a land use module. However, neither CVGSM nor WESTSIM have been calibrated for large-scale use. WESTSIM is capable of incorporating a salt budget module, but this is not in the current plans. Further development is needed to include salt budgets for future management planning. A number of other hydrologic models have been developed for use in the Valley but none of them have been sufficiently calibrated or include a salt budget module to allow for salt transport characterization. Most models suffer from a general lack of measured data that is needed for rigorous calibration.

Various attempts at modeling salt budgets in the Valley have been attempted. In 1990 a course salt budget estimate was published by SJVDP[6]. These values were

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[4] Belitz and Phillips, 1992.
[5] Montgomery Watson, 1993.
[6] Technical Information Record by Swain and Associates, 1990.

obtained using a mass balance approach[7] from published and estimated data. The salt budgets obtained were derived from data collected from 1980 to1985 and will be referred to as the 1985 estimates. These values represent a rough estimate of salt in the Valley and were never validated with measured observations.

WATER AND SALT BUDGETS

The principal components of water flow and salts (dissolved solids) into and out of the planning subareas (water and salt budget) in the Valley were estimated and reported in the San Joaquin Valley Drainage Program Technical Information Record mentioned above. The system is comprised of two subsystems: The root-zone subsystem, which includes all surface water and subsurface water to the free water table, and the semi-confined aquifer system which includes all subsurface water below the free water table to the Corcoran clay. Because shallow groundwater is the focus of drainage problems, the base of the semi-confined aquifer was chosen as the lower boundary for salt budget calculations. The water and salt budgets reported in the 1990 Technical Information Record were developed using the Department of Water Resources' Hydrologic and Economic Model database, the U.S. Geological Survey's Regional Aquifer System Analysis model database, and USBR and local water district data. The average annual salt budget for each of the three districts was taken from the 1985 estimates and is shown in Figures 1, 2 and 3.

Using 1990-1999 water supply and drainage discharge data (flow and quality) from the three districts, the net annual salt gain or loss (Figure 4) was calculated for the three districts (the gain/loss calculation doesn't include other salt inflow and outflow such as salt dissolution, lateral inflow or leakage through Corcoran clay and lateral outflow). PWD receives water from groundwater and CVP; WSID receives water from the San Joaquin River and CVP; and WWD receives water from the CVP and groundwater with no drainage discharges out of the district. Both WSID and PWD discharge drainage water to the River.

RESULTS AND DISCUSSION

Panoche Water District

The 1985 estimate surface water inflow was 132 TAF (including 87 TAF from surface deliveries and 35 TAF from precipitation), including a salt inflow of 43 000 tons (see Figure 1); 12 TAF from groundwater, and 700 acre-feet from aquifer inflow, including a salt inflow of 19 000 tons (with 18 000 tons due to salt dissolution), for a total of 145 TAF of water and 75 000 tons of salt. Surface water outflow was 16 TAF, which removed 12 000 tons of salt; consumptive use was 101 TAF, which removed no salt; and semiconfined aquifer outflow was 29 TAF

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[7] CH2M Hill, 1988.

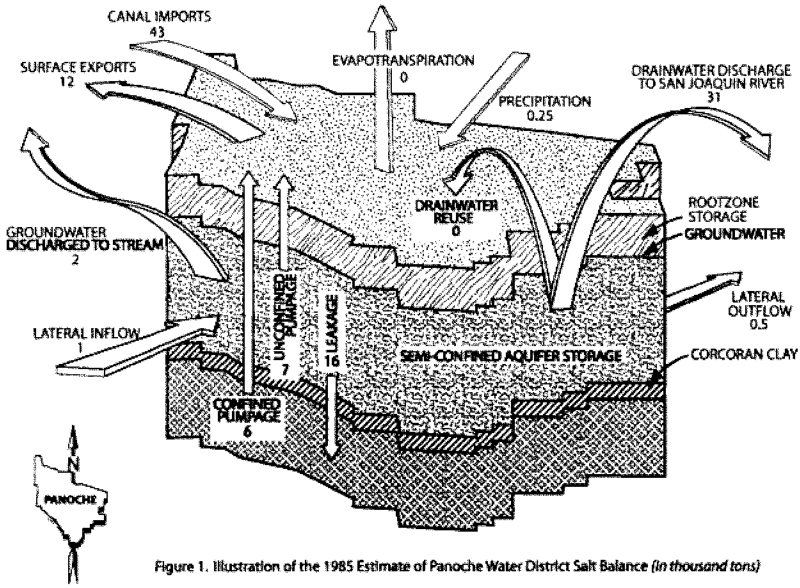

Figure 1. Illustration of the 1985 Estimate of Panoche Water District Salt Balance *(in thousand tons)*

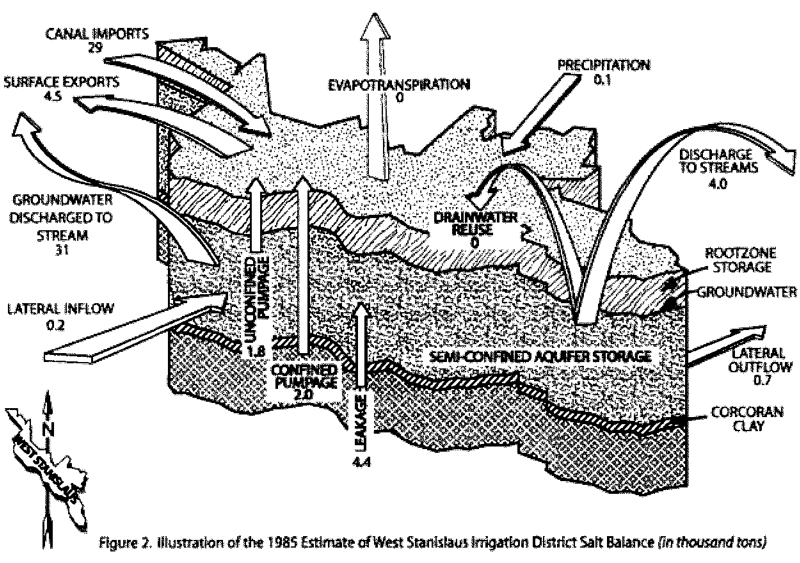

Figure 2. Illustration of the 1985 Estimate of West Stanislaus Irrigation District Salt Balance *(in thousand tons)*

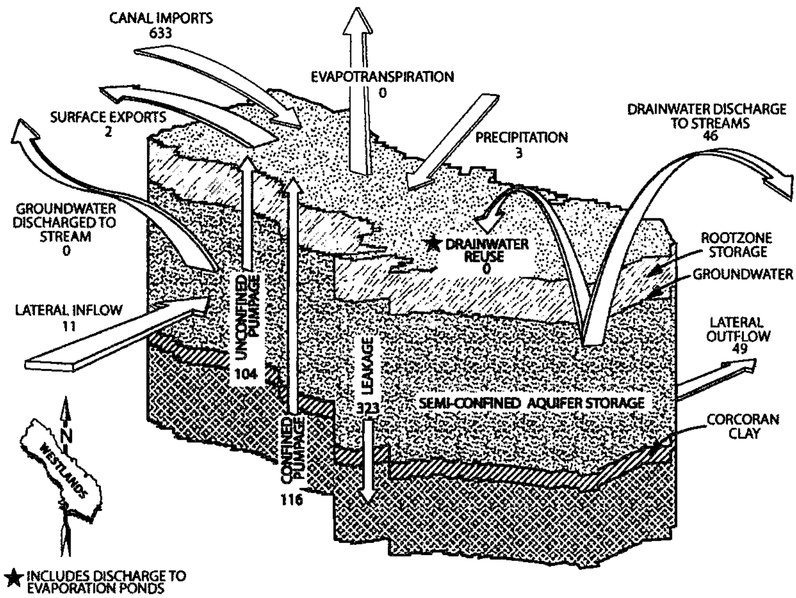

Figure 3. Illustration of the 1985 Estimate of Westlands Water District Salt Balance *(in thousand tons)*

(including 4 TAF through subsurface drainage discharges and 11 TAF through Corcoran clay), which removed 56 000 tons of salt (31 000 tons through drainage discharges and 16 000 tons by leakage through the Corcoran clay to the confined aquifer), for a total outflow of 146 TAF of water and 68 000 tons of salt . The 1985 estimate of net salt accumulation was 7 000 tons. The major source of salt is applied water and salt dissolution. The major salt outflow is leakage through the Corcoran clay layer and drainage discharge. PWD had a discharge to the San Joaquin River during 1990 to 1996, but beginning in 1997 it was reduced through district-wide recycling of the drainage water to comply with selenium load limits imposed by the waste discharge requirements.

During 1990-1999 the net annual salt loss (salt from surface water delivery plus salt from groundwater delivery less salt outflow due to drainage discharges) ranged from 40 000 to 100 000 tons per year (Figure 4). However, after PWD started recycling its drainage water and reduced its drainage volume in 1996, the salt loss from the district declined to 35 000 tons in 1999. During the period from 1990 to 1999, approximately 600 000 tons of salt were removed from PWD.

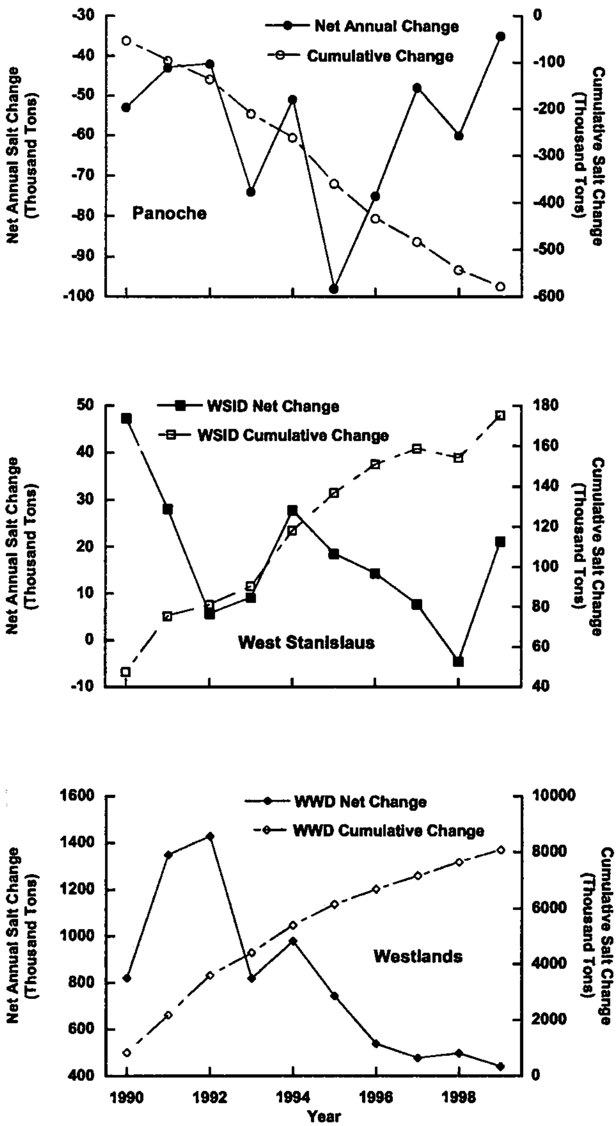

Figure 4. 1990 to 1999 District Salt Gain/Loss

PWD also provided water flow and quality data[8] from 1985 which was used for rough validation of the 1985 estimates. According to measured data, salt inflow to PWD was 29,000 tons (compared to 43,000 tons from the 1985 estimate) and 50,000 tons of salt was removed (compared to 31,000 tons from the 1985 estimate). In both cases these measured values resulted in a larger net removal of salt from PWD than the 1985 estimate. This confirms that better salt budget models are needed if they are to be used as a management tool in the San Joaquin Valley.

West Stanislaus Irrigation District

The 1985 estimate of surface water inflow was 70 TAF (including 32 TAF from canal water deliveries, 8 TAF from precipitation, and 18 TAF from San Joaquin River), including a salt inflow of 29 000 tons (see Figure 2); groundwater inflow was 3 TAF, including a salt inflow of 3 800 tons (with 3 600 tons from salt dissolution); and aquifer inflow was 100 acre-feet, including a salt Inflow of 3 800 tons, for a total of 73 TAF of water and 37 000 tons for salt. The surface water outflow was 10 TAF, which removed 4 500 tons of salt; consumptive use was 44 TAF, which removed no salt; and semiconfined aquifer outflow was 19 TAF, which removed 33 000 tons of salt, for a total outflow of 73 TAF of water and 38 000 tons of salt. The 1985 estimate shows a net loss of 1 000 tons of salt from the district.

During 1990-1999, Figure 4 shows that the net annual salt load ranged from a maximum gain of 50 000 tons in 1990 to a loss of 4 500 tons in 1998 (from surface water delivery plus salt from groundwater delivery less salt outflow due to drainage discharges). Additionally, during the period from 1990 to 1999, WSID accumulated approximately 180 000 tons of salt.

Westlands Water District

The 1985 estimate of surface water inflow was 1 809 TAF (including 1 407 TAF from surface deliveries and 378 TAF from precipitation), including a salt inflow of 633 000 tons (see Figure 3); 170 TAF from groundwater, including a salt inflow of 220 000 tons; and 14 TAF from aquifer inflow, including a salt inflow of 844 000 tons (with 780 000 tons due to salt dissolution and 53 000 tons from stream inflows), for a total of 1 993 TAF . Surface water outflow was 4 TAF, which removed 2 000 tons of salt; consumptive use was 1 472 TAF, which removed no salt; and semiconfined aquifer outflow was 336 TAF, which removed 522 000 tons of salt (including 236 TAF of water leaking through Corcoran clay, removing 323 000 tons of salt) for a total of 1 812 TAF of water flowing out of the system. The 1985 estimate of net salt accumulation was 1 175 000 tons or 1.5

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[8] Personal communication with Chris Linneman, Summers Engineering.

tons per acre. The major source of salt is applied water and salt dissolution. The major salt outflow is leakage through Corcoran clay.

During 1990-1999 the net annual salt gained (salt from surface water delivery plus salt from groundwater delivery with no salt outflow) ranged from 1.5 million tons to about 440 000 tons per year (Figure 4). The 1990-99 estimates assume no drainage water and no other sources and sinks for salts. During the period from 1990 to 1999, WWD accumulated approximately 8 million tons of salt.

CONCLUSIONS

The 1985 estimate for the PWD shows a negligible rate of salt accumulation. During 1990-1999 there was a loss of salt from the district, but it declined as a result of drainage water recycling (Figure 4). The 1985 estimate shows that WSID was in a state of hydrologic balance with surface and semi-confined aquifer inflows mostly balanced by outflow. A small net loss of salt of less than 0.1 tons/acre-yr was estimated. The 1990-1999 estimate shows that the district is gaining salt at a small rate. Approximate salt gain/loss results from the 1985 estimate and during the period from 1990 to 1999 show that WWD continued to gain salt from irrigation water sources.

The 1990-99 salt calculation is an approximation of salt gain and loss. A precise calculation of salt budget components requires salinity and flow data, and a salinity model which incorporates flow and salt transport within the complex groundwater system of the Valley. Specific data relating to water quality and flow for the various components of the conceptual salt budget model are necessary for calibration and validation of any future comprehensive salinity flow model. Evaluating root zone salinity is critical in determining management practices and maintaining agricultural productivity.

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RELATIONSHIP OF SELENIUM IN DRAINAGE WATER TO THE SELENIUM CONTENT OF IRRIGATED SOILS

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ABSTRACT

Predicting probable selenium concentrations in drainflow is now a critical step of project planning for Reclamation. Reclamation scientists tend to be very cautious in planning projects in areas with above average selenium concentrations. A better understanding of relationships between selenium concentrations in field soils and the eventual concentrations in drainwater will permit more realistic planning in areas with significant selenium concentrations in soils

This research was conducted to determine the relationship between the selenium concentrations in soil profiles and the selenium concentrations in the drainage water originating from the same area. In order to overcome selenium variability in soils, a large number of sample increments were collected and composited at each of three depth intervals for each study site.

A total of eighteen sites were examined from six areas of the western United States. Most of these areas were selected because of known high selenium soils. Both natural and artificially drained areas were included in the study. Soil saturation extracts as well as total soil selenium concentrations were compared to drainflow concentrations in order to evaluate their relationships for use in predictive modeling.

The findings of this research generally indicate that the total selenium concentration in soil is the most reliable indicator of selenium drainwater concentrations. Random variability of readily soluble selenium concentrations even in individual irrigated fields of the same soil type makes interpretation of saturation extract data difficult.

INTRODUCTION

The purpose of this research is to provide data that could be used to develop, test, and validate an improved selenium fate and transport prediction procedure. This

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model would be used to predict selenium concentrations in planned drains based in part on soil selenium concentrations existing before drain construction. Soil and drainwater data from irrigated fields will be used to establish relationships and associations used in the transport model. These data will be used in conjunction with other information such as soil column testing data, solute flow models, and geochemical models in order to improve Reclamation's selenium fate and transport model. This research found that use of a salinity transport model would usually over predict selenium concentrations in drainwater.

STUDY AREA LOCATIONS

The following areas were selected for study. The study areas represent a wide range of climatic and physical conditions. Most of the areas are known to have moderate to high levels of selenium in soil and drainwaters. High levels of selenium were considered desirable in order to insure the accuracy and reliability of laboratory data since laboratories may have higher error allowances at levels near the detection limits. Table 1 indicates the approximate location of the study areas. Four of the areas represent fields drained by artificial drains while natural streams and springs drain the remaining 2 areas.

Table 1. Location of study areas

Area	State	# of sites	Drainage Status
Navajo irrigation project (NIP)	New Mexico	3	artificial
Oxford tract (OX)	Colorado	3	natural
Spokane bench (SPOB)	Montana	1	natural
Imperial Valley (IMP)	California	2	artificial
Broadview ID (BV)	California	2	artificial
Uncompahgre Valley (UNC)	Colorado	7	artificial

REVIEW OF EXISTING DATA

A search of existing data was conducted in order determine if findings from related studies could be used to better design and frame this study.

The Department of the Interior (DOI) has conducted many selenium investigations over the years in support of the National Irrigation Water Quality program. A partial review of these data indicates that drainwater selenium is sometimes lower than saturation extract concentrations of soils within the drainfield, while drainwater salinity is often 2-10 times higher. Based on the soil and drainwater data it appears that the use of steady state salinity models to predict drainwater selenium concentrations would tend to overestimate drainwater selenium concentrations. Selenium studies at Stewart Lake (Stephens & others 1992) in Utah indicated the average selenium concentration of 16 soil profiles sampled to a depth of 6 feet had an average saturation extract selenium concentration of 74ug/l while the drainwater concentration from the same areas averaged 53 ug/l. Several other previous studies show a similar relationship based on limited soil data and usually a single drainwater sample.

Existing data also indicate that the coefficient of variation (CV) values for total selenium in soils is high and that the CV for soluble selenium in soils is very high.

ANALYSIS OF VARIANCE

A summary of CV data for soils sampled in the western states (Brummer & Yahnke), (Deboer and others), (Reclamation 1996) including sites sampled in this study is presented below.

Table 2. Coefficient of variation data soils

Natural land body	Type of Se test	CV% range	CV% average
landform	soluble	59-219	126
Field of same soil series	soluble	30-110	78
Landform	total	18-95	53
Field of same soil series	total	14-48	29

In order to determine if the soil/ drainwater relationships discussed above are valid it was apparent that more extensive soil sampling in the fields along with more drainwater sampling over time was needed to avoid a poor estimation of soluble selenium concentrations.

METHODS

Soils

The case study tracts were selected based on soil survey information, exploratory soil borings and Electromagnetic Meter (EM38) surveys. Table 1 describes the location of case study areas. Study areas consisted of 16 separate irrigated field sized tracts, and two large landforms. The soils within each tract are very similar, usually classed by the NRCS as a single mapping unit. Examination of previous data, as well as, independent selenium data from this investigation reveal that random variation of selenium in irrigated field size tracts of the same soil series is high. Coefficient of variation (CV) values for total selenium in field size tracts of the same soil series are estimated at 29 percent and soluble selenium about 78 percent.

Random variation appears to be similar at all depth increments to at least 30 inches. Coefficient of variation tends to decrease somewhat with depth in most fields but is still high throughout the soil profile. Based on the coefficient of variation estimates and professional judgment it was decided to sample each irrigated field by a stratified random sampling procedure since systematic, as well as, random variation in the soil exists in most fields. In order to accommodate this micro systematic variation, nearly equal numbers of sample increments were collected in furrows, on top of beds, and on the sides of beds. Approximately equal numbers of samples were collected in the top, mid, and lower parts of irrigated fields

Table 3. soil sampling goals

Depth	Soil Zone	Size of Composite Sample
0-12"	plow layer	30 increments
12-30"	active root zone	at least 10 increments
30-60'"	substrata	at least 10 increments

Based on the above sampling goals, a target accuracy estimate, at 95 percent confidence levels, for 8-30 inches and 30-60 inch zones is 48 percent for soluble and 18 percent for totals. An accuracy of 18 percent indicates that the data interpreter can be 95 percent confident that the selenium value determined by the sampling is within eighteen percent of the true mean selenium value for the field. Accuracy should be considerably higher for plow layer samples since a 30 increment composite should equate to an accuracy of about 12 percent for total selenium and about 28 percent for soluble selenium. It should be stressed that these sampling goals were met or exceeded on most fields; however, they were not met on all fields. Accuracy values are considerably greater than 20 percent on a few fields where less than 10 increments were collected to represent the field, or where samples were collected in different fields within the same landform. The Spokane Bench and Oxford sites were sampled based on large landforms rather than irrigated fields since subsurface drainage selenium data were obtained from seeps that collected drainage from a large area, and therefore accuracy values of soil selenium mean estimations may be greater on these case study areas.

A representative boring was logged at each of the 18 field sites. Borings were also examined and logged at a few dryland control sites. The soil boring logs contain site-specific information for the representative site, as well as, general information for the entire field represented. Information included on the boring logs include: ECe; pH paste; qualitative CaC03 content; soil texture; number of increments in composite samples; EM 38 measurements near boring; boring location.

Soil samples were collected with oakfield probes, hydraulic probes and hand augers.

Soil samples remained cool on the way to the Bureau of Reclamation laboratory in order to reduce the chances of selenium losses by volatilization. The samples were air dried, ground, passed through a number 10 screen and mixed . Sample splitting and size reduction was accomplished by the fractional shoveling method.

A 700-800 gram subsample was used to prepare a saturated soil paste. The pH and saturation percentage of the paste were measured. The paste was extracted under vacuum pressure until a 30-60 ml volume of saturation extract was obtained. The EC_e of the saturation extract was measured and the extract was sent to the USGS Branch of the Geochemistry laboratory in Denver for soluble selenium analysis. A separate 50-gram soil subsample was ground to a 100 mesh size (#100 standard sieve) and sent to the same USGS laboratory for the total selenium analysis.

The USGS Geochemistry laboratory determined selenium using the hydride generation method on both the saturation extracts and the digested solid samples (Hageman and Welsch,1996).

Each batch of soil and water samples had at least 5 percent QA/QC samples. Data validation results indicated the USGS was generally well within allowable sample accuracy ranges. Field sampling error was determined by independently sampling fields in a stratified random manner.

The number of drainwater samples varied between sites, ranging from 1-8. In some cases previous data or data collected by others was used to determine selenium drainwater concentration.

Drainwater samples were collected at drain outlets, or in some cases drain sumps. Streams and springs were also sampled. Drain flow estimates were made at sample collection events. In some cases district records were used to estimate drain flow at sumps. Where possible a stopwatch and a bucket were used to estimate drain flow at subsurface drain outlets.

QUALITY CONTROL (QC) SUMMARY

The USGS Geochemistry Laboratory in Lakewood, Colorado analyzed nearly all samples collected during this study. All sample batches contained at least 1 blind QC sample, approximately 1 out of 20 samples were QC samples in large batches. Most QC samples were reference material of known Se concentration, however, a few split and duplicate samples were also evaluated. All data met the following Reclamation acceptance criteria. Laboratory splits of soils RPD of 35 percent; Soil reference materials, +/- 20 percent of most probable value; Water reference materials +/- 15 percent of the most probable value; Saturation extract reference materials +/- 35 percent of most probable value.

Four field replicate samples were also collected in order to evaluate the soil sampling procedures. The results of these samples are presented below:

Table 4. Field replicate samples

Sample	Depth	Total Se mg/kg	Se_e ug/l	# of increments
T10-2	0-12	1.1	8.7	32
T10-2-fr	0-12	1.0	8.5	32
OX5-1	0-12	0.82	16.0	30
OX5-1 fr	0-12	0.86	9.8	10
OX2-1	0-12	0.49	11.0	11
OX2-rep	0-12	0.56	13.0	30
UC-1	0-8	4.6	330	60
UC-1rep	0-8	4.6	270	12

DATA SUMMARIES

The following tables and graphs summarize the data collected in this study and relationships between the soil and drainwater data. One study area was removed from the graphs because only one drainwater sample was collected at the site (UC1). No flow was ever observed at the drain outlet. The only sample collected was obtained from a water trap placed at the drain outlet. It is possible that some evaporation of the water sample occurred prior to collection of the sample.

Table 5. Site Selenium Summary

Sample I.D.	$WASe_T$ mg/Kg	$BRZSe_e$ Ug/L	$WASe_e$ Ug/L	Se_{dw} UgL
BU 3-2	.92	75	42.9	79.6
BV 10-2	.93	44	27.4	61.4
NIP 1	.10	1.9	1.7	1.0
NIP 2	.05	1.9	2.6	1.0
NIP 3	.05	1.3	1.7	.5
IMP 1	.20	5.2	6.3	18
IMP 2	.14	6.6	5.4	3.3
SPO B	.12	1.6	1.8	1.0
OX 20	1.47	170	88	10.4
OX 12	.64	54	32	48
OX ALL	.77	384	192	36.5
UNC 1	5.60	1200	886	260
UNC 2	4.15	81	54	55.4
UNC 3	4.7	140	105.9	106.6
UNC 4	2.05	11	12.4	34.1
UNC 5	5.31	60	26.7	121.2
UNC 6	8.6	3000	2116	154
UNC 7	4.84	32	28.9	96

A = Artificial; N = Natural WA = weighted average BRZ = bottom of rootzone
$WASe_T$ - Weighted average of total selenium in soil
$BRZSe_e$ - Selenium content of the saturation extract near the bottom of the root zone
$WASe_e$ – Weighted average of selenium content of the saturation extract from soil 0-60"
Se_{dw} – flow weighted average of selenium for all drainwater samples collected at the site

Figures 1 and 2 indicate a good correlation between total selenium and drainwater selenium content. The correlation between the two appears to be highly significant when only artificial drains are considered. The relationships between soluble selenium in saturation extracts and selenium in drainwater is still significant at the 5 percent level when artificial drains only are considered, however the correlation deteriorates when natural drained sites are also considered. A listing of best fit equations and coefficient of determinations for data collected in this study are presented in table 7.

Table 6. Soil – Drainwater Selenium Relationship & Summary Table

TABLE SITE	Drain Status	$WASe_{dw}$ Ug/L	$WASe_e$ Ug/L	$BRZSe_e$ Ug/L	$WAEC_e$ ds/M	$BRZEC_e$ ds/M	$WAEC_{dw}$ ds/M	EC_{dw}/ $WAEC_e$ ds/M	EC_{dw}/ $BRZEC_e$ ds/M	Se_{dw}/ $WASe_e$	Se_{dw}/ $BRZSe_e$
BV 2	A	79.6	42.9	75	3.52	5.59	4.68	1.33	0.84	1.86	1.06
BV 2	A	61.4	27.4	44	3.61	5.12	6.06	1.68	1.18	2.24	1.40
NIP 1	A	1.0	1.7	1.9	0.82	0.64	1.35	1.65	2.10	0.59	0.53
NIP 2	A	1.0	0.5	1.9	0.68	0.5	1.53	2.25	3.06	2	0.53
NIP 3	A	0.5	0.5	1.3	0.53	0.51	1.04	2.04	2.04	1	0.38
IMP 1	A	18.0	6.3	5.2	2.44	2.55	10.4	4.26	4.07	2.86	3.46
IMP 2	A	3.3	5.4	6.6	2.49	3.02	2.17	0.87	0.72	0.61	0.50
SPO B	N	1.0	1.8	1.6	0.46	0.43	0.61	1.42	1.33	0.56	0.63
OX 20	N	10.4	88	170	1.93	3.17	0.60	0.31	0.19	0.12	0.06
OX 12	N	48	32	54	1.24	1.89	1	0.81	0.53	1.5	0.89
OX ALL	N	36.5	192	384	1.5	2.3	0.78	0.52	0.34	0.19	0.1
UNC 1	A	260	886	1200	20.2	25.5	7.0	0.35	0.27	0.29	0.22
UNC 2	A	55.4	54	81	2.56	2.36	2.96	1.16	1.25	1.03	0.68
UNC 3	A	106.6	106	140	2.50	2.74	3.08	1.23	1.12	1.01	0.76
UNC 4	A	34.1	12.4	11	2.57	2.55	2.57	1.0	1.01	2.75	3.1
UNC 5	A	121.2	39.5	60	2.47	2.47	3.11	1.26	1.26	3.06	2.02
UNC 6	A	154	2116	3000	19.6	18.0	3.28	0.17	0.18	0.07	0.05
UNC 7	A	96	28.9	32	1.65	2.59	3.25	1.97	1.25	3.32	3.0
Average								**1.35**	**1.26**	**1.39**	**1.08**

A = Artificial; N = Natural   WA = weighted average EC_{dw} = EC of drainwater   BRZ = bottom of rootzone
$WASe_T$ - Weighted average of total selenium in soil $BRZSe_e$ - Selenium content of the saturation extract near the bottom of the root zone
$WASe_e$ – Weighted average of selenium content of the saturation extract from soil 0-60"
Se_{dw} – flow weighted average of selenium for all drainwater samples collected at the site

Fig. 1 - Relationship of Total Selenium in Soils vs. Drainwater Selenium

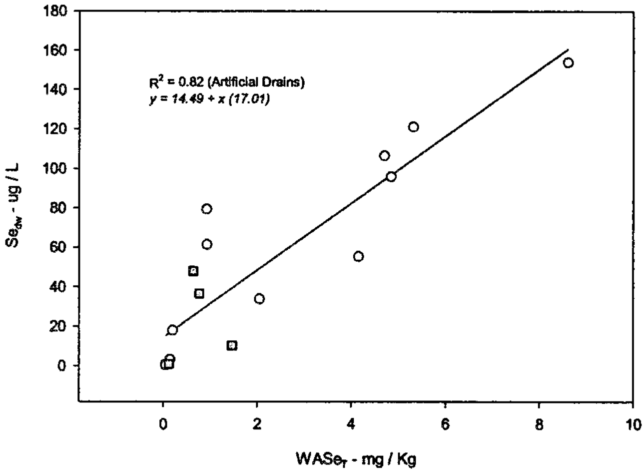


Fig. 2 - Relationship of Total Selenium in Soils vs. Drainwater Selenium
(Lower Values)

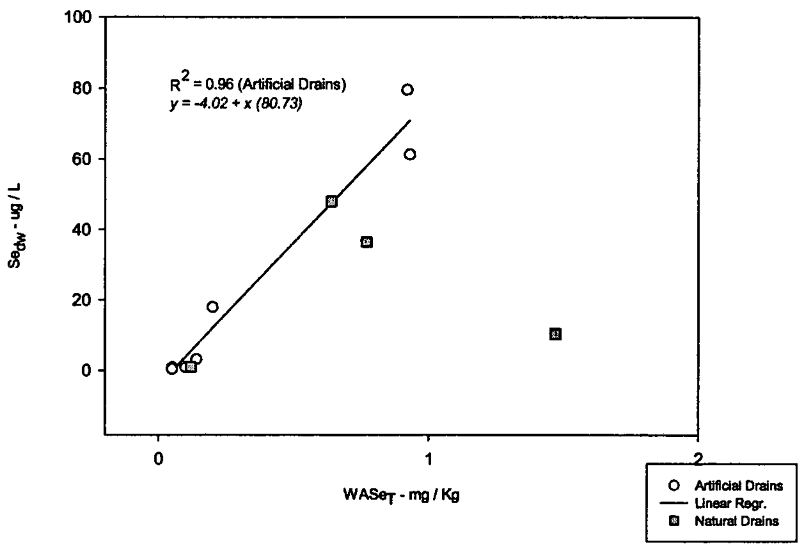


Fig. 3 - Relationship of Selenium from Saturation Extracts in Lower Root Zone Soils vs. Selenium in Drainwater

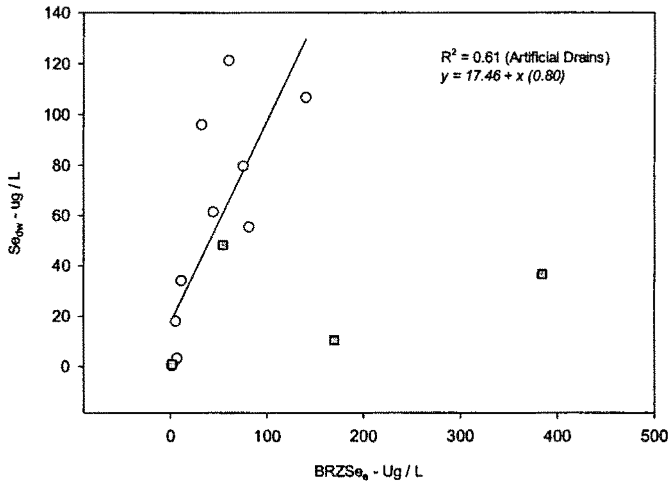
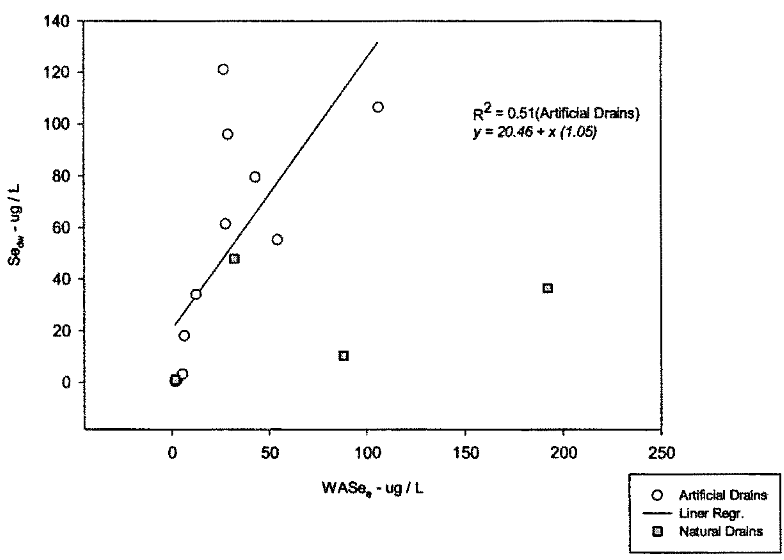


Fig. 4 - Relationship of Selenium from Weighted Average Root Zone Soil Saturation Extracts vs. Selenium in Drainwater



The following table presents a data summary of soil and drainwater selenium relationships found in this study. The data are sorted to separate artificially drained areas, (ART) , Uncompahgre project drains only (UNC) and all sites covered in the study (ALL)

Table 7 Soil - Drainwater Selenium Relationships

Data Sort	Parameter X	# of Observations	Relationship to Se_{dw} (y)	R^2	Significance (5%)
All	WAECe	17	y = 7.12x + 27.5	0.42	HS
All	BRZECe	17	y = 8.03x + 22.0	0.46	HS
All	$WASe_e$	17	y = 0.08x + 44.1	0.38	HS
All	$BRZSe_e$	17	y = 0.06x + 43.98	0.36	HS
All	$WASe_T$	17	y = 21.48x + 11.94	0.66	HS
UNC	WAECe	6	y = 4.05x + 73.4	0.43	NS
UNC	$WASe_e$	6	y = 0.03x + 80.91	0.46	NS
UNC	BRZECe	6	y = 4.65x + 70.7	0.45	NS
UNC	$BRZSe_e$	6	y = 0.02x + 80.85	0.45	NS
UNC	$WASe_T$	6	y = 19.05x + 0.42	0.86	S
ART	WAECe	13	y = 6.73x +32.8	0.41	S
ART	BRZECe	13	y = 7.68x +27.6	0.45	HS
ART	$WASe_e$	13	y = 1.05x + 20.46	0.51	HS
ART	$BRZSe_e$	13	y = 0.8x + 17.46	0.61	HS
ART	$WASe_T$	13	y = 17.01x + 14.49	0.82	HS

NS = not significant
S = significant 5% HS = highly significant 1%

FINDINGS

Comparison of the figures and tables presented above indicate that total selenium in soil correlates better with drain water concentration than readily soluble selenium in soil. Total selenium concentrations in soil exhibit much less random variability than soluble selenium, therefore considerably fewer soil samples are needed to adequately characterize total selenium than saturation extract soluble selenium. When collecting multi increment composite samples in order to estimate the mean selenium content of a soil body or a land form, total selenium has provided much more reliable data than soluble selenium. Soluble selenium in soils can also change back and forth from soluble to insoluble forms with changes in soil reaction, temperature, and redox conditions.

Readily soluble selenium concentrations in soil saturation extracts can be used to predict drain water concentrations but the variable nature of the relationship between saturation extract selenium concentrations and drain water concentrations makes predictions on a west wide basis somewhat questionable. The use of steady state soil salinity models to predict drain water selenium concentrations from saturation extract selenium concentrations will generally over predict drain water selenium concentrations since soluble selenium appears to be less mobile than most soluble salts in reduced saturated substrata characteristic of drain field conditions.

Coefficient of variation data presented in this report clearly states the case for using pre-construction total soil selenium data as a first evaluation technique to predict possible selenium concentrations in drainwater following drain installations. Samples or sample increments required to predict field (same soil series) and landform(same soil association) selenium within 15 percent of the true mean at the 95 percent confidence level are listed below:

Soluble selenium field	104	Soluble selenium landform	270
Total soil selenium field	15	Total selenium landform	59

This research and other research being conducted have demonstrated that average total selenium concentrations in soils can accurately be determined for areas with the use of composite soil samples. We could not conclusively prove however that compositing soil samples for soluble selenium was a valid way to accurately estimate readily soluble soil selenium content. In order for a multi increment composite soil sample to be meaningful it should represent a single soil population and each increment included in the composite sample should not be capable of changing the concentration of each other increment in the remainder of the sample. Both of these factors can be accurately evaluated and controlled for total selenium, however the latter factor may have the potential to compromise readily soluble selenium concentrations from composite samples. For example: If 20 increments of neutral soil are composited and the 21^{st} increment contains a small amount of residual calcium carbonate, the soil reaction (pH) of the sample could be raised from about 7.0 to 7.6. This pH change in the entire sample could change the solubility of selenium in the entire sample. For this reason great care must be taken when collecting composite samples for soluble constituents so that only a single well defined population is included in the composite sample. A separate paper is currently being prepared to further evaluate the use of multi increment composite samples on various constituents including selenium.

It should be noted that saturation extract selenium determinations are also subject to other errors besides the analytical error. Soil preparation steps such as grinding, mixing, preparing saturation pastes, and vacuum extraction add additional potential error to the saturation extract values when compared to water samples. Saturation extracts may also contain other substances that cause interference in analytical procedures.

It appears that the naturally drained areas examined in this study have a lower drain water concentration at any given soil selenium level than artificially drained areas. This may be due to slower drain out in naturally drained areas leading to more chemically reduced conditions in the subsurface flow systems. Soluble selenium may encounter reduced conditions in the substrata flow systems and be precipitated out of the drainage waters as elemental selenium and or selenide. It is also possible that dilution of water from other sources also lowered selenium concentrations. In contrast well-designed artificial drains can rapidly lower the water table. The soils in the drainage zone are rapidly oxidized and reduced with successive irrigation and drain out events. Under these circumstances, a larger portion of the soil inventory appears to remain in more soluble and mobile forms such as selenate and selenite.

Based on figures 1 and 2 it appears total soil selenium values have potential to be used as a possible damage threshold. A value of 0.5 mg/kg is suggested. If total selenium values exceed 0.5 mg/kg of total selenium then soluble selenium analysis and possibly leaching column tests should be considered to better evaluate hazard potentials.

Recommendations for further research.

Due to the high variation associated with soluble selenium in soils it is recommended that further research of this type be conducted on small irrigated fields of the same soil type. Ideally soils would be intensively sampled before drain installation and drain water samples and flow measurements would be collected periodically for at least one year following drain installation. It would be desirable to sample a number of individual sites in order to determine the coefficient of variation for the field. Those samples could also be used to test the validity of compositing samples for soluble selenium on the field. This type of study would probably be best carried out at an agricultural research facility.

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A DECISION SUPPORT SYSTEM FOR FIELD DRAINAGE MANAGEMENT

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ABSTRACT

The Colorado State University Irrigation and Drainage model (CSUID) is a decision support system (DSS) that helps design and/or manage irrigation and drainage systems, which maintain crop productivity while controlling drainage return flows. CSUID includes components for irrigation scheduling; root growth calculations; flow and transport in unsaturated and saturated zones; drain discharge; and crop yield estimates. The DSS runs on a PC with Windows 95/NT. Data for the model is currently being collected from four fields in Colorado's Arkansas River Basin.

INTRODUCTION

About 30 percent of the land in the western United States has the potential for moderate to severe salinity problems (Nation Research Council, 1996). In communities where salinity problems occur, sustainable agriculture is threatened (Western Water Policy Review Commission, 1997). Sustainable agriculture is defined as being productive and profitable while also conserving resources, protecting the environment and enhancing the health and safety of the public (O'Connell, 1991).

While the agricultural community has addressed salinity problems for many years, the inability to achieve sustainability in areas prone to salinity reflects a lack of an integrated/holistic approach to solving the problem (Water Environment Federation, 1992). As concerns about possible long-term environmental damage from downward percolating waters and the disposal of saline drainage water have increased, it has become more difficult to reach the goal of maintaining sustainable crop production in areas of high salinity (Wiley,

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1986; Gates and Grismer, 1989). Furthermore, the discharge of highly saline drainage effluents with high loads of trace elements from the irrigation-drainage system can induce water quality problems in downstream surface water systems (NRC, 1989).

Consumptive use from human activities significantly increases salt concentration. Consumptive use alone causes a seven-fold increase in the salt concentration in the Arkansas River (USGS, 1997). Evaporation from reservoirs, canals, high water table areas and from cropland receiving excessive or poorly timed irrigation are important consumptive uses. Evapotranspiration from crops and weeds and from phreatophytes in wastewater areas also occurs.

Crop yields are being reduced, and in some areas, cropland is being lost because of waterlogging and high salinity levels. To combat waterlogging problems, agricultural producers need a complete management package that blends information about irrigation practices, crop types, capabilities for improving yield, economic returns, and water quantity and quality.

Traditional irrigation systems have been designed based on a purely agronomic objective: sustain crop productivity on lands subject to saline high water tables. Growing concerns about the impacts of drainage return flows on water quality in downstream systems require the formulation of designs and innovative management strategies for both irrigation and drainage

Irrigation and drainage systems often have been designed separately. Frequently, the drainage system is designed only after the impacts of waterlogging and salinity are observed. The practice of designing each component separately has generated extensive reference work in each discipline but produced only weak links between irrigation and drainage design. In reality, the two disciplines are inextricably linked, especially in arid and semi-arid irrigated areas. First, irrigation inputs are controllable, as opposed to the random inputs due to rainfall in humid regions. Thus, the quantity of drainage water is controllable. Second, drainage requirements often change after a system has been designed because irrigation practices change. Third, disposal options for drainage effluents are directly affected by the on-farm design. The quality, quantity and timing of these discharges impact downstream users of the receiving stream.

This paper describes CSUID, a comprehensive irrigation and drainage system design and management DSS.

SYSTEM DESCRIPTION

CSUID (IDS, 1994) is a DSS that will be used in the Arkansas River Basin in Colorado to help in the design and/or management of irrigation and drainage systems that maintain crop productivity while controlling contaminant loads in drainage return flows. It includes irrigation scheduling, root growth calculations, flow and transport in unsaturated and saturated zones, drain discharge and salinity of the effluent, and crop yield estimates (Garcia et al., 1995). The components of the DSS have been verified using field data, and the results are presented in Manguerra and Garcia (1995 and 1997).

The drainage modules are based on the numerical implementation of a quasi three-dimensional finite difference model that solves the Richards' equation. The advective-dispersive transport equation is solved for one-dimensional vertical flow and salt transport in the variably saturated zone. The saturated zone is modeled using three dimensional flow and salt transport equations. The Newton-Raphson method is used to handle non-linearities in the numerical solution. The resulting tridiagonal matrices for one-dimensional flow and transport are then solved by the Thomas algorithm. A strongly implicit procedure is used to solve the pentadiagonal matrices that result from the finite difference formulation of the governing equations in the fully saturated zone.

CSUID allows users to manipulate the large amounts of spatial information that are required to manage irrigation and drainage systems. The user can study the spatial variability of data and the impacts of design and management decisions on irrigation and drainage systems. CSUID significantly reduces the amount of effort involved in the creation and/or debugging of input data and improves the understanding of the output.

The Colorado State University Irrigation and Drainage DSS: CSUID

Design and analysis of irrigation and drainage systems using decision support tools is not yet common practice. However, computer models allow consideration of more complicated scenarios, reflecting the complex concerns surrounding both on-farm and regional management. While models are numerous, few include both irrigation and drainage components and even fewer have the capabilities needed to make them useful decision support tools for arid and semi-arid irrigated regions. To date, most models have not been sophisticated enough to simulate water and salt transport in both the unsaturated and saturated zones, taking into account three-dimensional mixing in the saturated zone. This level of sophistication is needed to adequately study the effects of irrigation and drainage design on the quantity and quality of the drain effluent.

The numerical formulation of the computer simulation model is based on a modified linked finite difference implementation of the governing flow and salt transport equations. The modified linked approach evolved from the standard formulations because of the shift in the design and management of irrigation-drainage systems from a primarily agricultural focus to an approach that takes into account both agricultural and environmental concerns. Because of its computational economy, the standard linked approach is normally used for modeling traditional systems that have the main objective of maintaining crop yield despite the presence of a highly saline shallow water table. However, more rigorous models that adopt the continuous approach are required to handle adequately both the agricultural and environmental components of the system being modeled. The modified linked approach is an attractive alternative to the continuous formulation because it couples computational economy with accuracy.

The current CSUID DSS allows users to simulate such complex systems. The model is supported by an interactive Graphical User Interface (GUI).

Graphical User Interface

The GUI for CSUID is a combination of window, menu, and icon selections designed to allow quick and easy movement through the model. The GUI makes the tasks of data entry, editing, and viewing easier by providing editing tools that allow the user to graphically specify the data. Different irrigation and drainage scenarios (drain spacing, depth from the ground surface, irrigation rate, irrigation duration, and irrigation frequency) can be formulated for sensitivity analysis.

The model currently works on a PC running Windows 95/NT. The GUI was developed in C^{++} using OpenGL for the graphics.

The GUI provides the capability to discretize the system being modeled into a row-column-layer finite difference grid network. The location of drains, collectors, irrigation canals and no flow boundaries are graphically specified. Any number of basins can be included, and a different irrigation schedule can be specified for each basin. Some of the input parameters required to perform a simulation are bedrock elevation, vertical hydraulic conductivity, horizontal hydraulic conductivity, specific yield, initial root depth, maximum root depth, date of planting, and various crop coefficients.

Spatial crop and aquifer properties can be specified using the drawing area. Space-invariant parameters such as simulation control flags and coefficients of equations can be edited through a series of pop-up windows. These parameters include simulation parameters, pressure-saturation functions, root growth functions, root extraction functions, crop yield functions, ET functions, K-weighting method, equation type, and initial conditions.

The behavior of the hydraulic conductivity and saturation with pressure head can be defined by empirically-based nonlinear functions. The user can select from the following relationships: Brooks and Corey (1964), van Genuchten (1980), and Haverkamp et al. (1977).

In addition, different plant water uptake functions developed by van Genuchten (1987), Molz and Remson (1970), Hillel et al. (1976), Feddes et al. (1974), Gardner (1964), Whisler et al. (1968), and Herkelrath et al. (1977) are provided in the DSS. Root growth functions included are Rasmussen and Hanks (1978), Ferreres et al. (1981), Hank and Hill (1980), Borg and Grimes (1987) and Schouwenaars (1988). At present, the user can enter a crop yield model or use the crop yield model by Doorenbos and Kassam (1979). The model also provides different methods for calculating interblock hydraulic conductivities, including upstream weighting, and geometric, arithmetic and harmonic means.

The results can be displayed both spatially and temporally. Maps of soil moisture, salt concentration, flow vectors, and relative yield can be displayed. Cross-sectional profiles of parameters varying with depth can also be generated.

Field Application

The DSS will be validated at and applied to four field sites in the Arkansas River Valley, a semi-arid irrigated area where waterlogging and high salinity levels threaten both crop production and ground water, surface water and soil water quality. Intensive sampling at each field site will provide input data for the simulations and show the ability of the DSS to model existing conditions as well as changes in the system under different irrigation and drainage options. The DSS will help determine how well the proposed management options meet water quality objectives, while maintaining crop productivity and production.

Irrigation and Drainage Strategies

Traditionally, irrigation is scheduled based on soil moisture depletion to prevent or minimize matric stress. However, in areas with shallow water tables, water is continuously available for crop water use. Irrigation, however, needs to be scheduled whenever the average root zone salinity has exceeded a selected threshold level to prevent or minimize osmotic stress. Thus, for an extended period of time, a favorable crop root zone environment can be maintained without the use of artificial drains through the adoption of controlled irrigation using low salinity water combined with the use of the shallow water table as a supplemental source of water (Wallender et al. 1979; Campbell et al., 1960). When the groundwater contribution is included in the soil moisture budget, the estimated rate at which water is depleted from stored soil water is reduced, and the interval between irrigation events is increased, reducing the total number of irrigations (Ayars and Hutmacher, 1994).

Current State of Salinity in the Arkansas River Basin

Salinity levels in the Arkansas River in Colorado increase from 300 mg/l to 4000 mg/l over the 150-mile stretch from Pueblo to the Kansas Border. It is estimated that consumptive use alone causes a sevenfold increase in salinity in the river (USGS, 1997; Malinski, 1990). Crop yields are being reduced, and in some areas, cropland is being lost due to irrigation with highly saline water. In a recent study by Timothy Gates and John Labadie at Colorado State University, salinity levels were measured at 50 points (on average) in each of 30 fields, in the Arkansas River Basin, using an EM-38 probe. In the late summer, median salinity levels for 27 of these 30 fields were found to be at least 1,200 mg/L. Corn yield reductions occur when salinity is above this level. These results are consistent with the results obtained by Luis Garcia in four field studies presently being conducted in the same area. The combination of high on-farm salinity levels, high river salinity levels and highly variable seasonal river flows makes the Arkansas River Valley an ideal location for this project.

With time, the evapotranspiration processes will eventually cause the salinity of the shallower layers to increase so that irrigation becomes more frequent, and waterlogging becomes inevitable. As this stage approaches, the flows in the river should be evaluated to determine when the next drainage cycle should occur. Both on-farm crop requirements and flow and water quality requirements in the river determine the end of the no-drainage cycle.

Field Investigation and Data Collection Activities

Currently four experimental sites with different depths to water table and/or salinity levels have been selected for data collection. For each of the fields the following data are being collected:

- Weekly water table depth and salinity measurements from wells. In each of the fields between seven and eleven observation wells have been installed using a Giddings Rig, and their position and elevation has been recorded with a differential GPS receiver. In 10 of these wells continuous water table recorders have been installed measuring water table every hour. Figure 1 shows the hourly and weekly values of depth to water table for one of the observation wells.

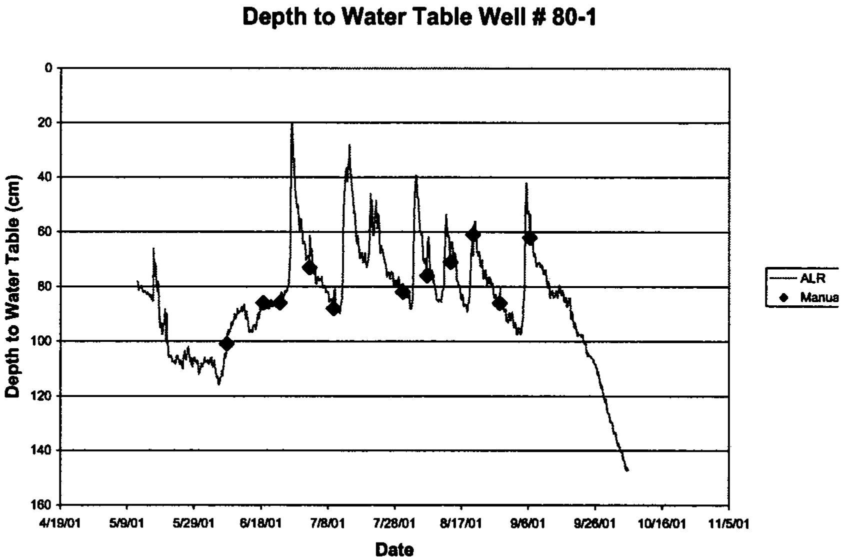

Figure 1. Hourly and Weekly Values of Depth to Water Table for Well 17-7.

- Average root zone soil salinity (as electrical conductivity of saturated extract, EC_e measured using an EM-38 probe). The sampling was done using a grid approach and determining the location of each sample using a differential GPS unit. Figure 2 shows the spatial variation in soil salinity in one of the fields being studied.

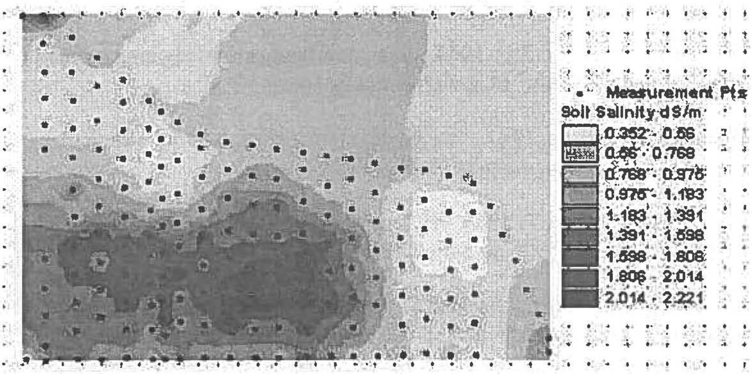

Figure 2. Soil Salinity for One of the Fields

- Survey of surface elevations, boundaries, and water table levels.
- Soil samples are being collected using a grid approach. A Giddings rig has been used, and four samples at one-foot depth intervals are collected at each sampling point. A differential GPS receiver will be used to record the position of each sample. Each soil sample is analyzed in the laboratory for particle size distribution, electrical conductivity of the soil solution, hydraulic conductivity and capillary-pressure saturation curves.
- Crop ET. ET gages have been installed at each of the four fields to estimate crop evapotranspiration.

Computer Modeling and Expected Results

The field data for the first year will be used for model calibration. A data set will be generated based on the best estimate of the parameters, and the model will be run. The input values of the model will be changed by trial and error until the modeled results match the measured ones. The criteria for success of the calibration process will be the minimum root mean square error (RMSE) in output estimates. After the model has been calibrated, it will be run for other scenarios using the data gathered during the remainder of the project time. The final evaluation of the model will be conducted by looking at the model's performance over the entire length of the project.

The DSS will be used to test suitable, alternative drain designs and management schemes, and if possible adjustments will be made in the current field management to verify the model's predicted results.

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STRATEGIES FOR REGIONAL-SCALE RECOVERY OF A SALINITY-THREATENED IRRIGATED RIVER VALLEY

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ABSTRACT

Two major problems that inherently stem from irrigation practice threaten the vitality of many of today's agricultural regions. Salinity of soils and irrigation water and waterlogging of fields due to high water tables have caused significant adverse socio-economic and large-scale environmental problems worldwide. Currently, at least one-fifth of the total irrigated land in the world is damaged by salinity build-up, and this damage translates to an estimated US$11 billion in reduced farmer income (Postel 1999). In the study presented, an area comprising 26,400 irrigated hectares (65,300 ac) located within the Lower Arkansas River Valley of Colorado, was investigated through intensive data collection over a period of four years to quantify the current salinity and waterlogging crisis in the region. Additionally, utilizing the collected data, a three-dimensional, transient, finite-difference groundwater model was developed, calibrated, and applied to evaluate alternative solution strategies. Considered strategies include improvements in on-farm irrigation practices, upgrading of the irrigation-water-delivery infrastructure (e.g. canal lining), and investment in new surface and sub-surface drainage facilities (e.g. use of pumping wells as vertical drains, installation of horizontal "tile" drains). Predicted effects on water table depth and salinity are presented and discussed.

INTRODUCTION

Early explorers of the American plains noted that the waters of the Arkansas River are affected by a natural salinity source. The Long Expedition noted in 1820 that, "At the mountains the water was transparent and pure, but soon after entering the plains it becomes turbid and brackish" (Long's Journal July 18, 1820). And, in 1845, as part of the Frémont Third Expedition, Lieutenant James William Abert commented on a pool of water found near the present day city of La Junta, Colorado, as being, "...so highly impregnated with common salt and

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sulphate of soda as to be nauseous and bitter to the taste" (Carroll 1941). After irrigation was introduced within the region in the 1870's, saline high water tables began to develop in the early part of the twentieth century (Miles 1977). Since that time, the problems have fluctuated in severity in response to a variety of changes within the river basin. In the 1930's, a large-scale effort to install subsurface clay tile drains achieved some success in easing high water table problems. During the 1950's, installation and operation of a large number of pumping wells penetrating the alluvial aquifer had the indirect effect of maintaining lower groundwater levels.

Recently, however, changes within the Lower Arkansas basin have caused the waterlogging and salinity problems to seemingly worsen. Construction of two major reservoirs (Pueblo and John Martin) has not only allowed a much larger and more consistent supply of irrigation water (and, therefore, caused the overall application amount to increase), but also has changed the sediment transport and peak flow characteristics of the river. Reduced sediment load is suspected of causing a reduction in canal "sealing" and an associated increase in seepage. Additionally, the reduction in peak river flow has caused a gradual aggradation of the river bed which has elevated river levels and reduced the potential return flow drainage gradient from irrigated lands. Also, recent court decisions concerning the Arkansas River Compact between Colorado and Kansas have required that the utilization of existing pumping wells be significantly curtailed. Evidence also exists indicating that much of the subsurface drainage installed in the 1930's is no longer functional. Likely, all of these factors have played some role in the recent intensification of problems in this region.

Description of Study Area

The study area is comprised of approximately 26,400 ha (65,300 ac) of irrigated land within Otero and Bent Counties, Colorado, and stretches along a 62 km (38.5 mi) reach of the Arkansas River (see Figure 1). The western boundary is marked by the town of Manzanola, and the eastern boundary is defined by Adobe Creek. The cultivated crops consist of alfalfa, corn, grass, sorghum, wheat, melons, onions, and various other vegetables (FSA 2001). The prominent irrigation system employed is open-ditch furrow irrigation, although there are a significant number of farms now using gated pipe in lieu of the traditional open-ditch and siphon tube technique. The number of center pivot sprinkler and drip irrigation systems is still very small. Soils in the area are principally alluvial deposits dominated by silty-clay-loam surface layers and loam-to-sandy loam substrata (USDA 1972a, 1972b).

DATA COLLECTION AND ANALYSIS

Data collection began in 1998 in a limited fashion, with soil salinity being the only property investigated with intensity. By April, 1999, however, a rigorous data collection program was developed and initiated. Included in this program is the monitoring of water table depth and salinity, soil salinity and surface water salinity, as well as estimation of hydraulic conductivity through slug tests and

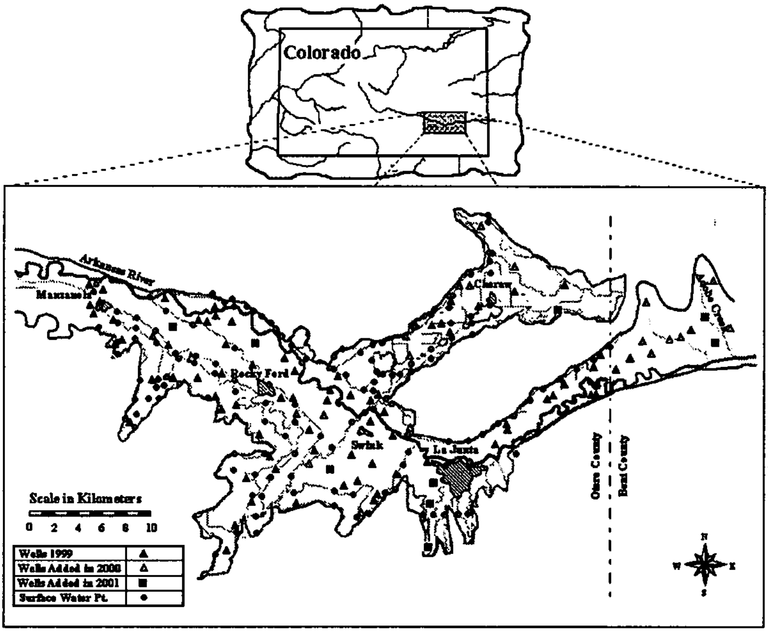


Figure 1. Study subregion located in the Lower Arkansas Valley, Colorado.

topographic surveys of land and water features using GPS technology. This program is on-going; however, only data collected through 2001 are presented.

Data Collection Program

Initially, a total of 74 monitoring wells were utilized for collection of water table depth and salinity data (see Figure 1, "Wells 1999"). Of these 74 wells, 69 were installed as part of the program, and 5 were adopted from previous studies. Observation sites were selected using a stratified random sampling technique (Cressie 1991) to minimize any bias in well placement; although, a few wells were specifically placed near the eastern and western boundaries, and a few randomly selected sites were moved slightly to accommodate farmer preferences. Wells were cased using screened PVC pipe with an inside diameter of 6.4 cm (2.5 in), and, initially, were drilled to a depth of 3.05 m (10.0 ft). In 2000, 21 of the wells were deepened to a depth of 7.0 m (23.0 ft), and 23 additional wells ranging in depth from 3.05 to 7.0 m (10.0 to 23.0 ft) were installed (see Figure 1, "Wells added in 2000"). In 2001, nine more wells were added in locations specifically targeted to increase data density where needed (see Figure 1, "Wells added in 2001").

Measurements of water table depth and salinity are taken at each monitoring site weekly during the peak irrigation season (May through September) and biweekly to monthly during non-peak times. Depths are measured manually using a metal tape and float, and salinity is measured indirectly as electrical conductivity (EC) using a calibrated, temperature-compensating specific conductance meter. At each monitoring site, three measurements of EC (one just below the water table level, one at an intermediate depth, and one near the bottom of the well) are recorded and averaged to obtain a representative value. These measurements are converted to total dissolved solids (TDS), i.e. salinity, using the relationship described in equation (1). This relationship was derived from analysis of groundwater samples collected at 17 of the monitoring wells in 1999 and reflects an r^2 value of 0.98.

$$TDS = 882.2EC \tag{1}$$

In addition to the weekly monitoring of water table depth and salinity, a key component of the data collection program is the monitoring of soil salinity twice per year. Soil salinity to a depth of near 1.0 m (3.28 ft) is estimated at selected fields (corresponding to groundwater monitoring sites) utilizing electromagnetic induction techniques (Rhoades et al. 1999). Four Geonics™ EM38 instruments are used to measure 30 – 90 locations per monitoring site, depending on field size. Readings are taken once near the beginning of the irrigation season (late May) and once near the end of the irrigation season (late August). The approximate data density within a field is one point (including both vertical and horizontal instrument positioning) per 0.10 ha (0.25 ac). Measured values are converted to bulk soil salinity and soil extract salinity for modeling and analysis purposes using relationships developed by Rhoades et al. (1989). These relationships have been generally confirmed through the collection and analysis of soil samples from numerous monitoring sites (Cardon 2002).

Another component of the data collection program is the monitoring of surface water salinity. Like the groundwater measurements, salinity is measured indirectly by the measurement of EC using a conductance meter at 163 monitoring points (see Figure 1, "Surface Water Pt."). A relationship between EC and TDS with an r^2 value of 0.97 was derived from the analysis of 28 surface water samples. This relationship is shown below in Equation (2) and was used in all surface water EC-TDS conversions.

$$TDS = 1479.2EC^{0.668} - 617.8 \tag{2}$$

Additional data collection program activities include performing slug tests (Chin 2000) at each monitoring well to estimate hydraulic conductivity, as well as the surveying of land and water surface elevations using multiple GPS receivers

(Trimble 4600LS and Ashtech Locus systems) and differential correction techniques accurate up to ±0.03 m (0.1 ft). Also, numerous river, tributary, and canal cross-sections have been surveyed from bridges using a measuring tape and depth probe, and a separately funded study on seepage from the Fort Lyon Canal was performed in 2001.

Current Findings

Data obtained through the data collection program, as well as data gathered from outside sources, were used to describe and assess the current conditions within the study subregion. The following is a brief summary of the findings.

Water Table Depth and Salinity: A few important statistics which have been compiled from the collected water table depth and salinity data are shown in Table 1. These particular statistics are shown because they serve nicely as indicators of the general condition of the study subregion. A more detailed statistical analysis has been performed, but is not presented here due to space constraints (see Gates et al. 2002).

Table 1. Summary of Collected Water Table (WT) Depth and Salinity Data

Year	*Avg. # of Wells Read per Week*	*Seasonal Avg. WT Depth (m)*	*Est. % of Area with WT Depth < 1.5 m*	*Seasonal Avg. WT Salinity (mg/l)*	*Est. % of Area with WT Salinity > 2000*
1999	69	2.14	25	3117	27
2000	90	2.48	19	2850	33
2001	96	2.69	18	2706	48

The values presented above represent both spatial and temporal averaging. Spatial averaging was achieved by interpolating across the study area between data points using the inverse distance weighted (IDW) method (Shepard 1968). The values shown reflect "seasonal" conditions, i.e. the conditions which occur during the main growing season (April through October). Data collected between November through March was not included in the calculation of the statistics shown since this time period is less critical from an agricultural standpoint. These off-season months are, however, included in the numerical modeling. It should also be noted that the salinity data presented represents only the upper layer of the aquifer which is penetrated by the monitoring wells – the deep aquifer characteristics are likely quite different.

The data conclusively reveal that large portions of the study region are subjected to waterlogged conditions, with some areas exposed to very high groundwater salinity. Specific areas identified from the data collection as having particularly acute waterlogging problems are the Patterson Hollow area west of Rocky Ford, an area directly south of the town of Swink, the area surrounding the town of Cheraw, and an area just east of North La Junta along the Fort Lyon canal. Areas showing high levels of groundwater salinity are an area directly west of Rocky Ford, the Holbrook Reservoir area, the Cheraw Lake area, and the La Junta area.

Interestingly, the seasonal average water table depth has increased (i.e. the water table has lowered) in each of the study years. This trend is likely a result of water table response to reduced aquifer recharge stemming from decreased irrigation water supply and diversions. Over the course of the study, the Rocky Ford Weather Station (#057167) has reported seasonal (April – Oct.) total precipitation amounts of 30.38 cm (11.96 in) in 1998, 46.63 cm (18.36 in) in 1999, 17.04 cm (6.71 in) in 2000, and 24.41 cm (9.61 in) in 2001 (NCDC 2002). State engineer diversion records support this theory. Seasonal average water table salinity also decreased each study year. A possible explanation of this observation is, because of the decrease in overall aquifer volume, there was a lowering of overall salinity due to a decrease in dissolution of native salts from salt-bearing soil layers (which are derived from marine shales) (Zielinski et al. 1995).

Soil Salinity: Results of the soil salinity monitoring are shown in Table 2. Values are shown in terms of soil saturation extract electrical conductivity (EC_e) and represent spatial averages using IDW interpolation.

Table 2. Summary of Soil Salinity Monitoring Data

Year	*# of Fields Monitored*	*Early Season Avg. EC_e (dS/m)*	*Late Season Avg. EC_e (dS/m)*	*Est. % of Study Area with Avg. $EC_e > 2.0$*
1998	30	2.3	3.1	NA
1999	68	2.6	2.9	80
2000	77	2.4	2.0	69
2001	80	2.8	2.5	69

The data show a seasonal increase in average soil salinity during the "wet" study years (1998 and 1999); conversely, a decrease occurs in the "dry" years (2000 and 2001). Likely, this pattern is due to a greater upflux of salts through high water tables and less potential for leaching during 1998 and 1999, with less salt upflux and greater leaching occurring in 2000 and 2001.

Surface Water Salinity: Table 3 summarizes the average salinity (shown in terms of EC) for the Arkansas River and the six main canals for each study year. The values are reflective of the dilution that takes place in higher water supply years (such as 1999). Although only overall averages (spatial and temporal) are shown, it should be noted that the spatial variability in each watercourse was significant, with salinity levels increasing downstream in all cases. Major surface

Table 3. Summary of Surface Water Salinity Data in dS/m

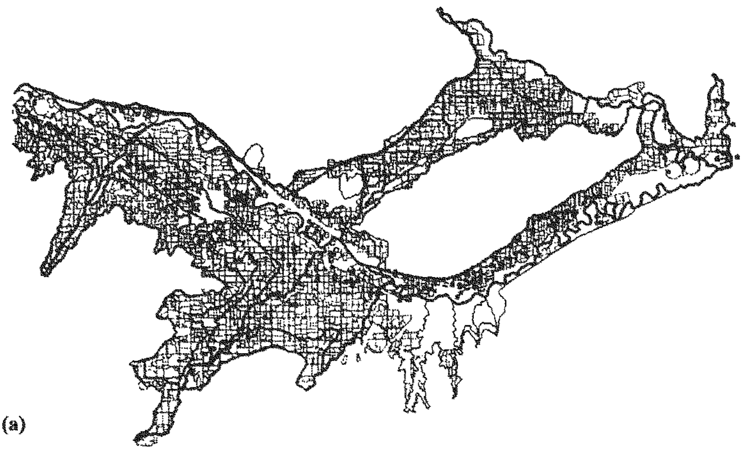
Year	*Arkansas River*	*Rocky Ford Canal*	*Catlin Canal*	*Otero Canal*	*Rocky Ford Highline Canal*	*Holbrook Canal*	*Fort Lyon Canal*
1999	0.97	1.05	0.88	1.35	0.70	0.80	1.00
2000	1.33	1.06	0.93	1.48	0.84	1.04	1.35
2001	1.19	1.00	0.91	1.35	0.77	0.97	1.18

drains were also monitored and yielded an overall seasonal average EC of 2.61 dS/m in 1999, 3.19 dS/m in 2000, and 3.10 dS/m in 2001. Additionally, three major storage facilities were monitored. The Fort Lyon Storage Canal had a seasonal average EC of 1.89 dS/m in 1999, 2.03 dS/m in 2000, and 1.91 dS/m in 2001. Holbrook Reservoir was found to have a season average EC of 1.31 dS/m in 1999, 1.66 dS/m in 2000, and 1.32 dS/m in 2001, and Cheraw Lake, which receives drainage from the northern portion of the study area, had a seasonal average EC of 13.87 dS/m in 1999, 13.27 dS/m in 2000, and 15.06 dS/m in 2001.

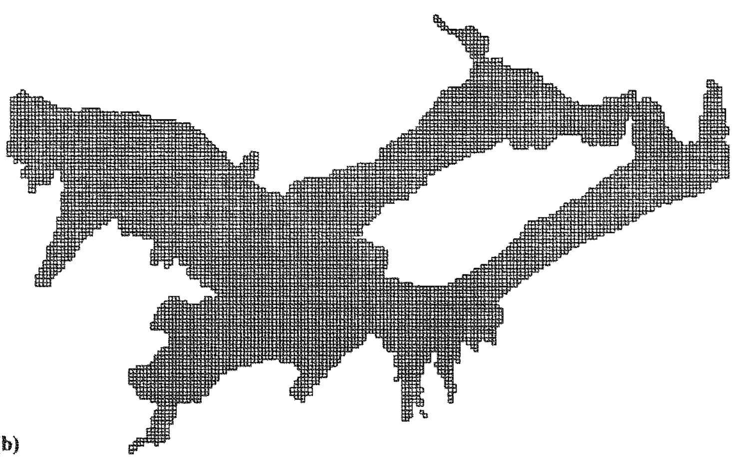
<u>Additional Items</u>: Analysis of collected data, as well as diversion records obtained from the State Engineer's Office and data from the Colorado Climate Center, has indicated that existing irrigation efficiencies range from 30 to 50% over the study subregion. Slug tests were performed at 95 of the monitoring well sites, and analysis has yielded estimates of hydraulic conductivity from 0.003 m/day (0.01 ft/day) to 10.24 m/day (33.60 ft/day) in the upper aquifer layer. Seepage tests have indicated that conveyance losses are approximately 0.25 % (of total flow diversion) per km (0.40 % per mi) to 0.33 % per km (0.53 % per mi); however, to date, only the Fort Lyon Canal has been tested.

MODELING

Numerical modeling of potential solution strategies was performed utilizing MODFLOW (McDonald and Harbough 1988) to simulate groundwater flow and MT3DMS (Zheng and Wang 1999) to simulate salinity transport. A graphical user interface known as the Groundwater Modeling System (GMS), version 3.1 (BYU 1999), was used in initial model development and in the analysis of model output. The applied models use finite-difference techniques to approximate the governing non-linear flow and mass-transport partial differential equations. The study area is represented by a three-dimensional grid containing 16,188 active cells. These cells have a uniform X-Y dimension of 250 m (820 ft) and vary in cell height depending upon aquifer thickness. The aquifer is represented by two layers of cells: the top layer represents the shallow, low-permeability portion of the aquifer which contains the root zone, and the bottom layer represents the deeper, higher-permeability portion of the aquifer which lies between the root zone and the confining bedrock. Surface features including field boundaries, the river, tributaries, irrigation canals, pumping wells, and reservoirs were digitized within the GMS interface (or digital representations were imported into the interface) which translated the associated input data into the correct MODFLOW and MT3DMS format. This conceptual model, along with the finite-difference grid, is shown in Figure 2.



(a)



(b)

Figure 2. (a) Conceptual model of study area; (b) Finite-difference grid

Both the MODFLOW and MT3DMS models simulate time-varied changes using a weekly time step. The modeled time period (April 1999 – Oct. 2001) includes a total of 133 time steps. The source-sink modeling packages utilized in the MODFLOW model include the *General Head*, *River*, *Well*, *Evapotranspiration (ET)*, *Recharge*, *Time-Variant Specified Head*, and *Drainage* packages. The preconditioned conjugate-gradient (PCG2) method was selected for flow calculations within MODFLOW. Within MT3DMS, the *Advection* and *Source/Sink Mixing* packages were utilized along with the third-order total-variation-diminishing (TVD) solution method to simulate the system solute flux and advective transport. To support this modeling, numerous collected data sets, along with data obtained from outside sources, were incorporated into the model or used for model calibration and verification. The model data sets are summarized in Table 4.

The MODFLOW model was calibrated to within a mean absolute elevation error of 1.0 m (3.28 ft) and an overall mean elevation error of 0.2 m (0.66 ft) using the collected 1999 data on groundwater elevation by adjusting the more uncertain parameters such as hydraulic conductivity and canal seepage (which is controlled by assigned conductance values in the *General Head* package). Values for these parameters were constrained within a range known by data collection and analysis as actually occurring within the study region. Additionally, model-calculated return flows for the river and its tributaries were compared to existing streamflow gaging data to insure that the model represents realistic behavior.

The MT3DMS model was adjusted to replicate the observed groundwater salinity data; however, because the contribution of salts from salt-bearing soil layers has not been investigated to the extent required for quantification and incorporation in the model, there is a significant degree of uncertainty inherent in modeled salinity input. Still, the solute-transport model is extremely useful for the purposes of examining solutions strategies on a comparative basis.

Weekly-varied recharge values for each field polygon were estimated based on the following factors: annual crop type data obtained from the Farm Service Agency (FSA); estimated crop ET requirements; a randomly-assigned, unique application efficiency selected from a truncated normal distribution with a mean equal to the overall canal command area irrigation efficiency (minimum cutoff = 0.15, maximum cutoff = 0.85, standard deviation = 0.2); irrigation frequency (dependent upon crop type); irrigation schedule (randomly assigned); deep-percolation fraction (based on Walter 1995); and effective precipitation. The equation used for recharge estimation for each field polygon for time step t is as follows:

$$[Q_R = DP(1 - E_A)(Q_{ET} - Q_P)]_t \tag{3}$$

Table 4. Model data set summary

Data Set	*Source*	*Use/Model Package*
Ground Elevation	USGS, GPS surveying	Block-Centered Flow
Bedrock Elevation	Weist '62, Major et.al '72	Block-Centered Flow
Shallow Hydraulic Cond.	Data Collection Program	Block-Centered Flow
Deep Hydraulic Cond.	Wilson '65	Block-Centered Flow
Crop Evapotranspiration	Farm Service Agency, Colorado Climate Center, CropFlex98 calculations (Broner and Lorenz 1998)	Recharge calculations, ET Package
Effective Precipitation	National Climatic Data Center	Recharge calculations
Aquifer Recharge	Calculated using estimated efficiency and leaching fraction	Recharge Package
Pumping	State Engineer's Office	Well Package
Seepage (Conductance)	Data Collection Program	General Head Package
Salinity	Data Collection Program	Source/Sink Mixing
Surface Water Levels	GPS surveying	General Head, River
Specified Head	Data Collection Program	Time-Variant Specified Head Package
Specific Yield/Porosity	USDA '72, Data Collection Program	Block-Centered Flow, Advection Package

where Q_R = recharge estimate (m), DP = deep-percolation fraction, E_A = application efficiency, Q_{ET} = evapotranspiration estimate (m), and Q_P = effective precipitation (m).

Modeled Scenarios

Six separate scenarios were modeled to investigate effects of various solution strategies. Scenarios were formulated in an attempt to represent realistic strategies which might be employed on a regional scale. They are as follows:

Scenario 1: *Baseline Conditions.* This is the baseline scenario which simulates actual conditions measured during the data collection program. This scenario is used to evaluate the comparative effects of changes to the system as modeled in the five other scenarios.

Scenario 2: *Reduce Recharge Rates by 25%.* This scenario simulates the impacts of uniformly increasing application efficiencies over the entire study region so that recharge rates are reduced by 25%.

Scenario 3: *Increase Pumping Rates by 50%.* This scenario examines the effects of increasing the pumping rates of currently active wells within the study region by 50%. Additional flow is assumed to by routed directly into nearby surface drains which flow back to the river.

Scenario 4: *25% Reduction in Canal Seepage.* This scenario models the aquifer response to reducing canal seepage through structural improvements or otherwise in all canals by 25%.

Scenario 5: *Subsurface Drainage Installed over 25% of Waterlogged Area.* In this scenario, it is assumed that effective subsurface drainage is installed in 25% of all fields which are waterlogged (i.e. the water table depth is less than 1.5 m). These fields were randomly selected, and it was assumed that "effective" subsurface drainage would lower the water table to a depth greater than 1.5 m (4.9 ft).

Scenario 6: *Combination of Scenarios 2 and 5*: This scenario explores the impacts of reducing recharge rates by 25% (Scenario 2) while simultaneously maintaining effective subsurface drainage over 25% of the currently waterlogged area (Scenario 5).

Modeling Results

The results of the modeling runs are summarized in Table 5. These results indicate that the current situation, although serious, is recoverable. As expected, Scenario 6 offers the most widespread benefits in reducing waterlogging problems; however, all modeled solution strategies yielded a net decrease in waterlogged area. The results of the preliminary salinity modeling were inconclusive (i.e. no significant changes in water table salinity were predicted). Model improvements are needed which will address uncertainty and which will extend the modeled time period to include a long planning horizon.

Table 5. Summary of Modeling Results

Performance Indicator	*Scen. No. 1*	*Scen. No. 2*	*Scen. No. 3*	*Scen. No. 4*	*Scen. No. 5*	*Scen. No. 6*
Predicted Seasonal Reduct. in WT Elev. (m)						
1999	NA	0.142	0.008	0.015	0.206	0.268
2000	NA	0.592	0.123	0.299	0.138	0.308
2001	NA	0.460	0.022	0.056	-	0.101
Predicted % of Area with WT Depth < 1.5 m						
1999	19.1	16.9	18.8	18.4	17.6	15.7
2000	13.1	11.5	12.9	11.6	12.0	10.9
2001	11.5	10.5	11.5	10.2	10.7	9.7

CONCLUSIONS AND FUTURE WORK

The on-going comprehensive data collection program has revealed a subregion whose agricultural productivity is currently hampered by significant waterlogging and salinity problems. If strategies are not employed to alter the current conditions, the future economic vitality of the area will be in jeopardy. Modeling indicates that regional-scale solutions do exist and can have very significant effects. These effects are amplified when strategies involve multiple approaches to reducing aquifer recharge or to artificially lower water table levels through subsurface drainage or increased pumping. Further study of more detailed solution strategies and further refinement of the existing models (including stochastic modeling of uncertain parameters) will continue, and should yield solid evidence upon which the best solutions can be formulated and implemented. Specifically, work is underway at Colorado State University to develop detailed unsaturated zone modeling of fields within the study area. This work will give insight into the impacts that solution strategies will have on soil salinity, and, therefore, on actual crop productivity. Also, studies are underway to examine the overall economic viability of regional-scale solutions. In concert, these studies should allow for informed planning which will insure the sustainability of the Lower Arkansas Valley's agricultural productivity and protect the livelihood of its rural communities.

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REAL-TIME WATER QUALITY MODELING AND MANAGEMENT IN THE SAN JOAQUIN RIVER

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Michael C.S. Eacock[2]

ABSTRACT

The San Joaquin River (SJR) is one of the most regulated flowing waterbodies in the nation. Current regulations include minimum seasonal flows for salmon migration and water quality objectives for salinity, selenium, boron, sediment, temperature, and most recently dissolved oxygen. Modeling tools have been developed in the past decade to quantify contaminant sources and to help improve coordination of high quality east-side Basin reservoir releases and degraded west-side Basin agricultural and wetland drainage flows. Other projects in the watershed have allowed enhancements to be made in modeling capability. The Grassland Bypass Project, initiated in 1996 has significantly reduced salt, selenium and boron loading from west-side agricultural sources through innovative drainage management, reuse and water conservation measures . The Vernalis Adaptive Management Program (VAMP) has shown that SJR flow and tributary releases can be manipulated and controlled for short periods of time in a highly deterministic manner. CALFED has supported a real-time water quality management project in Grasslands Basin wetlands that is investigating the flexibility of wetland management practices to control discharges of salt and oxygen-demanding biological constituents without negatively impacting wetland habitat values. In the Panoche-Silver Creek watershed rainfall gauges and real-time flow and water quality monitoring have been installed to provide early warning and initial estimates of flood flows during large storm events. An initiative, to resolve dissolved oxygen deficit problems in the Stockton Deep Water Ship Channel (DWSC), has created an opportunity to expand the current flow and salinity forecasting effort to consider factors affecting biochemical oxygen demand. This paper reviews progress made in the past decade towards development of a real-time management of water quality system for the SJR and considers the work ahead to develop a comprehensive decision support system.

BACKGROUND

The San Joaquin River Basin (SJRB) is bounded by the crest of the Sierra Nevada on the east, the Coast Range on the west and the Kings River on the south. The

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drainage area is about 32,000 square miles, the size of Maine. It provides about 16 percent of the State's water supply and has ten major tributaries: Cosumnes, Mokulumne, Calaveras, Stanislaus, Tuolomne, Merced, Chowchilla, Fresno, San Joaquin, and Kings Rivers. The average annual runoff from the mainstem is 1.7 million acre-feet. The SJR Basin reservoirs store snowmelt later used for hydropower generation and water supply. Of the 57 major reservoirs in the basin, four can store up to 1,000,000 acre-feet or more each, another 12 can each store up to 100,000 acre-feet. Reservoir releases provide storage for flood flows, satisfy agricultural irrigation water demands, support a river rafting industry and provide pulse flows to aid migrating Chinook salmon. Adult Chinook salmon return to the tributaries to spawn, beginning in October, and the resulting salmon smolts leave the tributaries and pass through the Delta,to the Pacific Ocean , during the spring.

More than 95 percent of the water in the SJR is captured each year by Friant Dam and diverted south down the Friant-Kern Canal and north in the Madera Canal to farms and cities on the east-side of the San Joaquin Valley. Saltier water from the Delta is brought back into the Valley in the Delta-Mendota Canal and California Aqueduct to farmlands and managed wetlands on the west-side of the SJRB. This water has an average salinity of 350 ppm. A portion of this applied water returns to the river as surface or subsurface drainage increasing the river's salt load. The largest component of the salt load in the lower SJR is derived from west-side agricultural sources (50%) and wetland sources (10%). These west-side sources, particularly subsurface agricultural drainage and wetland drainage often cause SJR water quality objectives to be exceeded. Agricultural drainage discharges are highest during the irrigation season (March – October) with peak flows generated during the pre-irrigation season (March – May). Impounded water in seasonal wetlands are released according schedules that optimize conditions for waterfowl habitat.

It has been customary in the past for decisions affecting reservoir releases, agricultural drainage discharges and wetland releases to be made independently without regard to impacts to other water users and to water quality conditions in the SJR. Exceptions to this rule are commonly court-imposed remedies to tort litigation. For example the US Bureau of Reclamation has a legal obligation to meet Vernalis water quality objectives through releases from New Melones Reservoir on the Stanislaus River resulting from a court settlement reached between the agency and the South Delta Water Agency.

Water quality objectives (Table 1) for the SJR have been developed by the State Water Resources Control Board (SWRCB) for selenium and boron at Crows Landing (immediately downstream from the confluence with the Merced River), for salinity at Vernalis (below the confluence with the Stanislaus River) and for dissolved oxygen in the Deep Water Ship Channel (DWSC) which is used by the Port of Stockton to allow access to ocean-going cargo vessels. The assimilative capacity for each of these contaminants is defined as the contaminant load (in pounds or tons per year) that the river can still accept before exceeding the contaminant objective for compliance. Positive assimilative capacity means more of the contaminant can be discharged into the river without exceeding objectives. Negative assimilative capacity quantifies the load of the contaminant that must be removed to meet objectives. The impetus for real-time water quality management in the SJR Basin is to decrease the frequency with which water quality standards are exceeded along the main stem of the River.

Table 1 : San Joaquin River Water Quality Objectives

Water Quality Constituent	Compliance Monitoring Station	Seasonal Water Quality Objective Irrigation Season (April to September)	Seasonal Water Quality Objective Non-Irrig Season (October to March)
Salinity (EC)	Vernalis	700 mS/cm	1,000 mS/cm
Boron	Crows Landing	1 mg/l	1 mg/l
Selenium	Crows Landing	5 ug/l	5 ug/l
Dissolved oxygen	Channel Point – Deep Water Ship Channel	5 mg/l	5 mg/l

Real-Time Water Quality Management

Real-time water quality management on the SJR for a particular constituent can be defined as a collaborative process by which the assimilative capacity of the SJR for that constituent is shared among stakeholder dischargers in a manner that avoids violation of SWRCB water quality objectives. It comprises four major activities: data collection, data processing/modeling, analysis, and information dissemination. Raw data must be quickly assembled and checked for accuracy prior to data analysis. To be effective as a management tool, these activities must be executed in near real time. The data processing/modeling and analysis steps should consider the end users and facilitate dissemination of information results to stakeholders to assist coordination of water management activities in the Basin.

Typically, river stakeholders manage water according to a set of rules that dictate when water is stored, used, or discharged. These stakeholders normally operate independently, making decisions on water use that are based on the quantity and or quality of water supplies available to them and their own individual needs. For example, reservoir operators manage reservoirs based on criteria and rule curves that address flood control, water delivery scheduling, power generation, and recreation demands. Each criterion is further controlled by a combination of long term or seasonal specifications such as seasonal maximum storage allowances, as well as short-term specifications such as rainfall or reservoir inflow.

Real time water quality management in the SJR Basin relies on coordinated and cooperative actions that encourage manipulation of the scheduling of east-side reservoir releases and west-side agricultural and wetland return flows to meet improve compliance with numeric water quality objectives. Successful implementation assumes that stakeholders can be informed of one another's actions and that they have the flexibility and resolve to adjust their operational schedule to minimize conflicts. With better information on upstream conditions, downstream users can adjust their timing of use to avoid diverting irrigation water with high salt or high boron concentrations. Reservoir releases can be timed to improve flows for fish migration and agricultural uses. Releases of saline water may be matched to the assimilative capacity of the SJR to help reduce the frequency of exceedence of salinity water quality objectives at Vernalis. Finally, releases of saline water from wetlands and drainage systems can been scheduled to coincide with the need for outflows sought by fisheries managers.

Chronology of SJR Real-Time Water Quality Management Initiatives

The concept of Real-Time Water Quality Management in the San Joaquin Basin was first proposed by Alex Hildebrand in 1988 as a means of disposing of excess salts. Hildebrand's concept was a series of groundwater wells adjacent to the SJR that would evacuate a portion of the groundwater salt reservoir during wet years, when there was excess assimilative capacity. The use of SJR assimilative capacity was further advanced by the development of the San Joaquin River Input-Output (SJRIO) model (Kratzer, 1987). This model was used by the San Joaquin Valley Drainage Program to compare management actions to reduce west-side salt and selenium mass loads to comply with SWRCB river water quality objectives. The formation of the Water Quality Subcommittee of the San Joaquin River Management Program (SJRMP-WQS) in 1989 led to the first funded research projects, specifically geared to establishing a network of real-time SJR water quality monitoring stations and developing procedures to forecast short term changes in SJR assimilative capacity. In the past 5 years the implementation of a number of resource management initiatives have greatly enhanced the prospects for realization of this concept.

VERNALIS ADAPTIVE MANAGEMENT PROGRAM

The Vernalis Adapative Management Project (VAMP) is a 14 year-long experiment to improve scientific understanding of the relationship between flow and fish resources in the SJR. Fishery biologists developed a program of study to gather data on the impact of flows, Delta project export rates and delta diversions on the populations of salmon smolts in the lower SJR. The VAMP was developed as an alternative that provides an equivalent level of protection to the SJR flow objectives contained in the SWRCB's 1995 Bay-Delta Water Quality Control Plan. The VAMP agreement also identifies and quantifies the sources and volumes of water required to implement the VAMP study that are managed by the

San Joaquin River Group Authority, whose members are willing sellers of water supply. The primary uses of the VAMP water supply are to (a) provide a pulse flow for a 31-day period at Vernalis between April 15 and May 15 each year; and (b) to manage Delta export pumping and other diversion flows identified by the CVPIA water acquisition plan, during this period, to facilitate migration and attraction of anadromous fish. The VAMP is described as an adaptive management study that anticipates that the flow requirement changes annually in response to hydrologic and biologic conditions. The VAMP Agreement provides for up to 137,500 acre-feet of water annually.

The implications for real-time management of water quality are significant and positive during the April 15 to May 15 VAMP period, when east side reservoir releases become highly deterministic improving forecasting skill of SJR flows. It follows that by increasing the accuracy of the flow forecast the accuracy of salt assimilative capacity forecasts in the SJR also improves significantly.

GRASSLANDS BYPASS PROJECT

The Grasslands Bypass Project (GBP) was conceived in the late 1980's as a means of removing selenium contaminated agricultural drainage water from wetland channels within the Grassland Water District. The source of this drainage water are farms in the 97,000 acre Grassland Drainage Area (GDA). The GBP uses a portion of the federal San Luis Drain to convey this drainage water directly to Mud Slough (north), a tributary of the SJR. This has significantly reduced the selenium contamination in more than 93 miles of wetland supply channels. Use of the Drain by GDA farmers is contingent on compliance with strict monthly and annual selenium load targets and the formation of a regional drainage management authority. The monthly and annual load targets were established through a lengthy negotiated process between the water districts, state and federal resource agencies, the US Environmental Protection Agency, Environmental Defense, and activist organizations.

Selenium load objectives are based on average pre-project monthly selenium loads from the GDA. Annual load objectives were initially set at the 9-year mean of 6600 lbs per year - these targets and monthly load targets were to be reduced by 5% per year in the last 3 years of the 5 year project. The project has been re-authorized for another 8 years until the end of 2009. In 2005, the selenium load targets are reduced incrementally to the Total Maximum Monthly Load (TMML) limits, recently set by the California Regional Water Quality Control Board – Central Valley Region (CRWQCB). The 2005 TMML selenium load objectives will vary according to water year type (critical, dry/below normal, above normal, wet) which will offer some relief during wet years when assimilative capacity in the SJR is high (Figure 1). However the allowable selenium loads in a typical TMML approach are less than 70% of those allowable under an equivalent real-time water quality management system which takes advantage of about 80% of the SJR assimilative capacity for selenium. The project has been successful in meeting selenium load limits in normal water years as is shown in Figure 1. Selenium loads to the SJR have dropped steadily since September 1996.

In order to meet selenium load targets innovations have been made at the farm and water district level. Continuous flowmeters have been installed at main discharge points from the GDA. Telemetry systems were installed to allow real-time monitoring of individual water district's contribution to overall drainage flow and to project selenium mass load.. With this knowledge water districts have developed customized selenium load targets for each tile sump based on correlations between tile drain flow and selenium load. This has been accomplished at a fraction of the cost of daily selenium monitoring and has proved an efficient system for encouraging compliance from local farmers enabling the GDA to meet monthly selenium load targets.

The GDA water districts also enacted a moratorium on tailwater discharges into District owned canals. The Districts assisted farmers to design and construct tailwater return systems that allowed the blending of tailwater and surface water canal deliveries. Panoche Water District invested in a district-scale recirculation system that collects and conveys accumulated tailwater to the inlet diversion points along the California Aqueduct. The District closely monitors the salinity of the blended water to ensure that during periods of reuse that no farmers receives

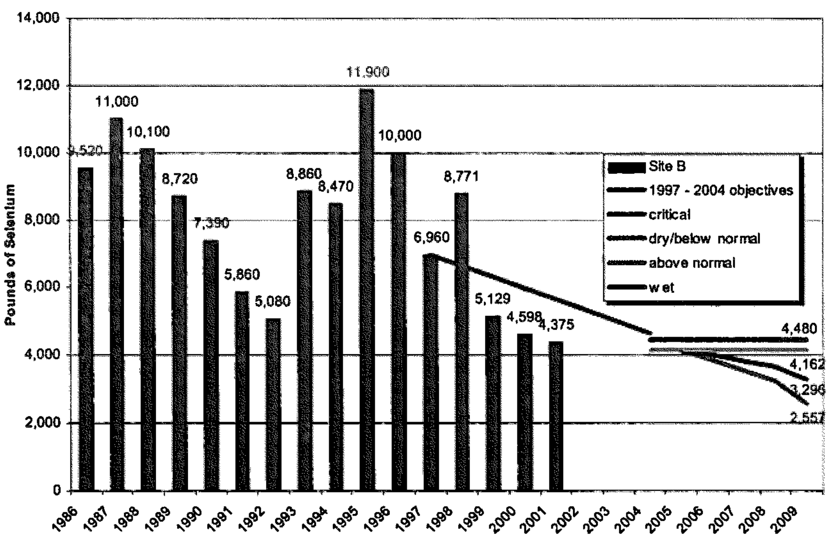

Figure 1. Selenium mass loading and the impact of the Grassland Bypass Project

canal deliveries with salt concentrations exceeding 600 ppm between the dates of June 30 – September 30. Drainage sump stage control sensors were raised so that they were activated only when water tables were within 5 feet of the ground surface. Color coded floating risers were installed in monitoring wells – to provide a visual indicator of water table depth. From the roadside. To control gravity drains that discharge directly to open ditches the main drainage lines were severed and weir control structures installed to help to store more drainage water beneath each field prior to discharge to the District's collector system.

Drainage management policies have been amended to allow the GDA water districts to cycle their sump pumps to help meet monthly selenium load objectives. Knowledge of the sumps that produce the highest selenium concentrations and loads allows the water districts some latitude in meeting load targets. This policy helped to produce greater than 40% reductions in selenium load during the first five years of the project (Figure 1).

The impacts of real-time drainage management are best illustrated using a simple dilution volume analysis which uses releases from New Melones Reservoir as a surrogate for river salt assimilative capacity for years before and during the GBP. Figure 2 illustrates how the various drainage selenium load management practices have altered the shape of the annual salt load hydrograph from the basin. Not only are the reductions in salt load also significant from the GBP water districts since 1996 but the policy appears also to have changed the shape of the salt load discharge hydrograph, damping the peak salt loads during pre-irrigation season – shifting some of this salt load export into the following months.

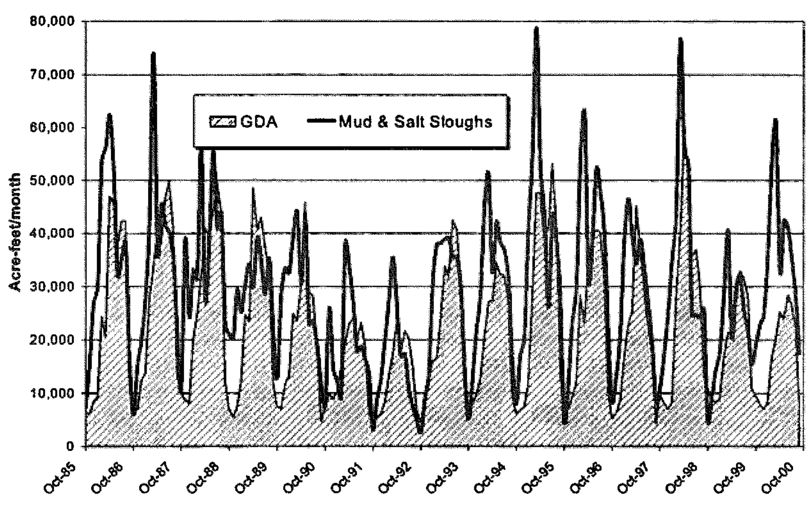

Figure 2. Monthly volume of water required to dilute drainage return flows from the Grassland Drainage basin to meet Vernalis standards (1988-2001)

The important lesson from the GBP is that given clear, unambiguous effluent load targets, water districts can manipulate the timing of their drainage exports. What is less certain is how to make the transition from a TMML drainage management approach to a more dynamic real-time system, which is better tuned to real-time and forecast river assimilative capacity.

WETLAND REAL-TIME WATER QUALITY MANAGEMENT

Private wetlands and state and federal refuges have not been exempted from the CRWQCB salinity control plan for the San Joaquin Basin. The Grassland Water District (GWD) has initiated a project aimed at quantifying salinity mass loading from the private duck clubs and cattle ranches that comprise the 51,500 acre northern half of the District. Management of salts from wetlands is more complicated than from agricultural land because of the larger number of system constraints, including diversity goals, poorly understood relationships between soil salinity and moist soil plant propagation, and the lack of a decision support system to assist with the selection of best management practices. A GIS-based salinity accounting model has been developed to simulate the dynamics of salinity concentrations in seasonal and permanent wetlands within the Northern GWD and to provide a decision support tool for real-time wetland salinity management.

Five new flow and salinity outflow monitoring sites and one inflow monitoring site have been installed in the Northern GWD. A parallel experimental monitoring system has been installed at the Salinas Duck Club, one of the most intensively managed and progressive duck clubs in the watershed. The Salinas Club experiment is providing information on salt mass loading during drawdown from shallow, deep, early season and late season wetland managements units that have been separately monitored. The long-term impacts of these management regimes on habitat quality is being assessed for each of these management regimes. These small-scale and large-scale monitoring systems will likely form the backbone of any future wetland real-time water quality management system.

PANOCHE-SILVER CREEK WATERSHED FORECAST SYSTEM

The 380 square mile Panoche-Silver Creek watershed is the largest west-side tributary to the SJR and the primary source of selenium to the SRJB. Measures to limit these contaminant loads including early-warning systems of significant runoff events can help to reduce the impact on SJR water quality. Recently installed monitoring stations and agency simulation models of precipitation, snowmelt and runoff for the east-side watersheds allow for advance forecasts of east-side tributary streamflow. The same capability is not yet available for the west-side. West -side ephemeral streams can deliver flows up to 10 % of the total flow into the SJR, carrying significant sediment, salt, and selenium loads. Implementing predictive modeling and measurement applications on the west-side will expand the San Joaquin water quality monitoring network.

A CALFED-sponsored project, administered by the watershed-based, Coordinated Resource Management Program, is adding to the existing real-time water quality management network by providing rainfall, flow and water quality monitoring stations within the watershed on three of the major tributaries to Panoche/Silver Creek. A GIS-based rainfall-runoff and streamflow model will be developed using data collected during significant storm events. The goal is to be able to forecast times of potential high runoff concentrations, maintain an expanded operational flow and water quality monitoring network, and provide advisories via the SJRMP-WQS web site and listserver.

SJR REAL-TIME WATER QUALITY MANAGEMENT

Real time forecasts of SJR assimilative capacity for salt are currently made using an existing monthly model, the San Joaquin River Input-Output (SJRIO) model (Kratzer et al., 1987), which has been adapted to calculate flows and water quality in the SJR on a daily basis (SJRIODAY model). The original SJRIO model provides a base case condition, monthly data files from which are overwritten with real-time data collected from a network of real-time flow and water quality monitoring stations in the San Joaquin Basin. Figure 3 shows the locations of these gauging stations.

The monthly SJRIO model performs a mass balance accounting of discharge, TDS, boron, and selenium for a 60-mile reach of the lower SJR, between Lander Avenue in the south to Vernalis. Lander Avenue was chosen as the upstream boundary of the model because the SJR is often dry upstream of the confluence with Bear Creek in all but wet years. The SJR near Vernalis was chosen as the downstream boundary because it is a water quality compliance monitoring station, it is upstream of Delta tidal effects and has a long-term historical data record. Water is supplied to agricultural water districts from three sources: diversions from the SJR, groundwater pumping, and pumpage from the Federal project located in the Delta. For its base case the model uses the average of each of four water year types - critically dry, dry, normal, and wet. SJR inflows, diversions and water quality are calculated for every tenth of a mile along the 60-mile reach of the River. Estimates for diversions from seventy-five diversion pumps on the SJR and the east side tributaries were based on water rights records available at the SRWRCB. Model inputs for which there are no available real time data were estimated using similar-year mean monthly hydrology. Annual drainage flows from west-side tile drains, which contribute over 50% of the annual SJR salt load, were estimated by multiplying tile drainage factors ranging from 0.65 to 0.85 acre-feet per acre per year by the tile drained acreage (Kratzer et al. 1987). Gauged surface agricultural return flow data were available for water districts on the east-side of the SJR. Average monthly TDS, boron, and selenium concentrations were determined based on a survey of available data for sumps and drains carried out by the CRWQCB.

DECISION SUPPORT SYSTEM

The decision to build a functional decision support system (DSS) to aid real-time water quality management in the SJR was made in 1995 by the SJRMP-WQS. System requirements were to retrieve pertinent data from field-based sensors, allow quality assurance checks to be performed on the downloaded data, fill data gaps, display data graphically, then transform the data into information useful to operational decisions. A useful feature of the DSS developed by Systech Engineering Inc. was the the Windows™ - based, graphical user interface (GUI) which was simple to use and served the needs of modelers and stakeholder's alike (Quinn et al., 1997). The purpose of the GUI was to assist stakeholders who make operational decisions on drainage discharges, recycling and temporary storage with respect to SJR assimilative capacity. The system facilitates the inspection of real time and forecast data, and features routines to expedite the collection of model input data and disseminate water quality forecasts (Figure 3).

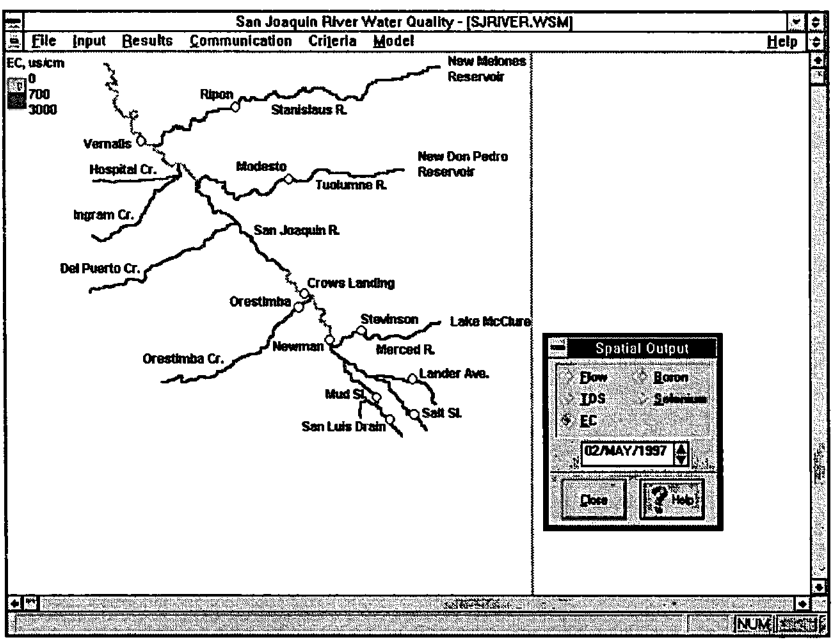

Figure 3. GUI visualization of monitoring sites on the San Joaquin River and the major tributaries and current SJR water quality conditions

Reservoir operators can enter daily schedules of diversions and discharges and upload these schedules every two weeks to the person making the flow and water quality forecasts. Likewise, this person can also use the GUI to download operational schedules from agencies where timely data is routinely posted to a public ftp site. The criteria menu may be selected to specify the color code used to display the spatial variations of water quality. Results from model runs can be viewed as a time series or by spatial location. When time series is chosen, a dialog box appears for the selection of flow and water quality parameters (TDS, boron, or selenium). The color representation of water quality is usually set according to the water quality objective.

In theory the GUI allows water managers to coordinate their operational decisions on a weekly basis by providing a spreadsheet-type entry of operational schedules consisting of the past week's operation and two weeks projected operation. Water district managers who decide to change their drainage or diversion operational schedules as a result of the model run output would be able to contact SJRMP-WQS members by telephone or email. The model run input data would be revised and SJRMP-WQS staff would rerun the model again, posting the revised run results on an FTP or project web site.

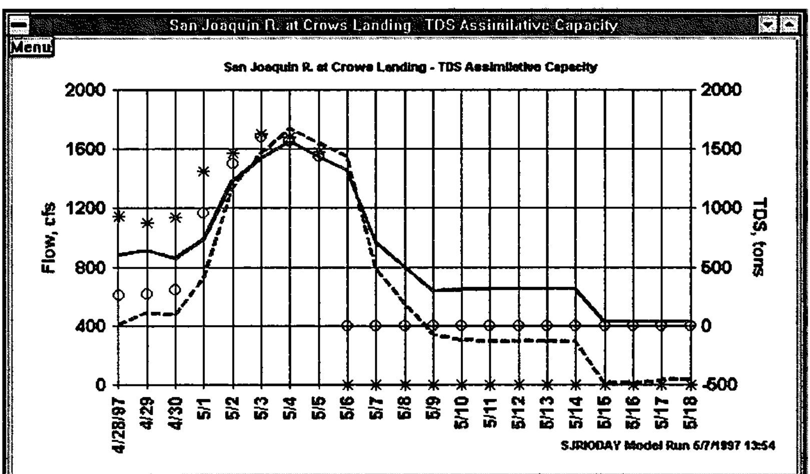

Figure 4. Assimilative capacity at Vernalis for the SJRIODAY run beginning 5/5/97

In early 1999 the SJRMP-WQS abandoned the GUI approach in favor of the world wide web. The GUI approach required that co-operators install the GUI on their home machines and update their input data files from a dedicated computer housed at DWR, Fresno. Forecasts using "push" technology, whereby an e-mail was distributed to stakeholders containing a url link to the forecast web site, was considered more effective than "pull" technology whereby the co-operator has to initiate the request for information. The web site also allows the distribution of associated data and supplemental information that would be difficult to communicate with the GUI-based approach.

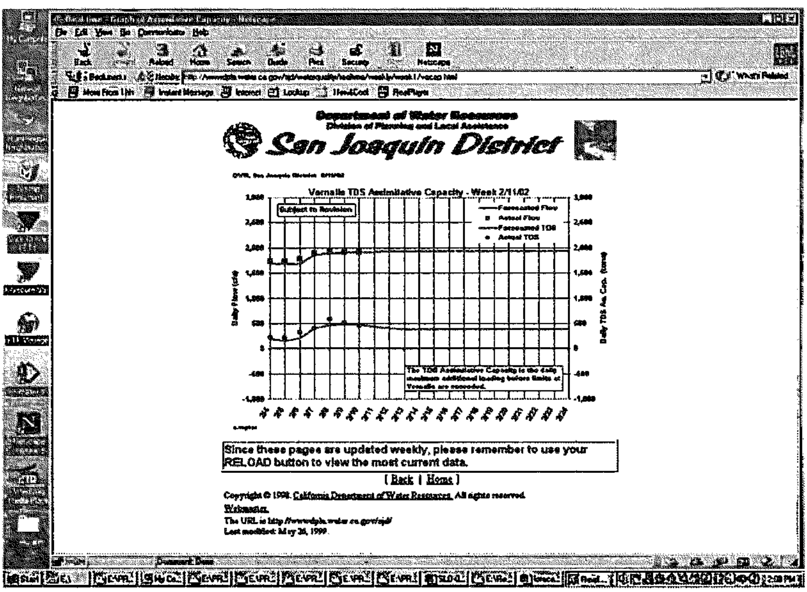

Figure 5. Weekly forecast of San Joaquin River assimilative capacity.

Forecasts continue to be made on a weekly basis by the SJRMP-WQS project team which comprise individuals from the DWR, CRWCB, Berkeley National Laboratory, UC Berkeley and the US Bureau of Reclamation. Active users of the system are the planners for the SJR VAMP and the San Joaquin River Dissolved Oxygen TMDL project.

DISSOLVED OXYGEN MANAGEMENT TMDL

Dissolved oxygen levels in the Deep Water Ship Channel operated by the Port of Stockton regularly fall below EPA water quality standards during the late summer and fall each year. The problem is created by a transition in hydraulic residence time as river water passes from the shallow well-oxygenated SJR to the deep, wide ship channel, a transition which promotes settling of suspended material including algae and encourages occasional channel stratification. When dissolved oxygen falls below 5 ppm the DWSC may create an effective barrier to fall run salmon migration – jeopardizing the viability of the fishery as well as CALFED investments in upper watershed fish habitat restoration. The problem is especially acute during dry and critically dry years when hydraulic residence times within the Ship Channel increase with diminished inflow from the lower SJR. Intensive monitoring of a number of water quality indicators has taken place since 1999 to attempt to develop a dissolved oxygen TMDL for the DWSC and improve compliance with EPA standards.

Management solutions to address this problem involve (1) recognition of the relative contribution to the problem by agricultural, wetland and municipal sources; (2) coordinated continuous monitoring of the factors contributing to low dissolved oxygen in the DWSC; and (3) development of a decision support and management tool that allows forecasting of future dissolved oxygen conditions in the SDWSC that will assist real-time management of techniques to address the problem when it develops. A goal of real-time forecasting of dissolved oxygen in the SJR will be to improve coordination of activities among those entities that directly benefit from and depend on the resources of the SJR system leading to an overall improvement in SJR water quality. There are many implementation options that could provide additional benefit if integrated into a real time management system. These include: operations of South Delta barriers, low head recirculation pumping at the Head of Old River, aeration in the DWSC, release timing of effluent discharge from wastewater treatment plants, release timing of flows from duck clubs, wetlands and wildlife refugees, release timing of flows from urban stormwater holding ponds, and release timing and flow levels from east side tributaries for fall pulse salmon attraction flows.

An improved hydrodynamic and water quality model of the lower SJR and Delta system is under development based on DWR's existing DSM-2 model. The model will be used as an aid to decision making - to compare aeration, source control and other management options and to simulate waste load allocation policies as part of a long-term management solution to the oxygen deficit problem. A calibrated DSM-2 model of the upper SJR was recently completed (DWR, 2001) which uses the same input database as SJRIODAY but which is capable of running with 15 minute data – allowing more realistic simulation of the hydraulics and water quality dynamics of the River system.

The model will play a role in the integration of the various real-time water quality management projects, described in this paper, within the SJRB.

CONCLUSIONS

The purpose of real-time forecasting of dissolved oxygen and salinity in the SJR is to improve coordination of activities among those entities that directly benefit from and depend on the resources of the SJRB leading to an overall improvement in River water quality. The major components of any real-time management system for the SJR require monitoring, modeling, information dissemination and resource management. Improvements in data automation, the cost effectiveness of data acquisition and dissemination, and coordination of modeling activities have been accomplished in the past 5 years with the formation of the inter-agency SJRMP Water Quality Subcommittee and the awards of grant funding to develop the present system. The most challenging ne: step will be to develop the institutional framework and water management policy to move the concept of real-time water quality management from theory to practice. This may eventually require the formation of an institution or appointment of a "water qualit czar" to maintain and manage a network of flow and water quality monitoring stations and the development of a integrated decision support system to set target flows and contaminant loading schedules for all entities discharging to the SJR.

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REMEDIATING IRRIGATION DRAINAGE PROBLEMS AT STEWART LAKE

Delbert M. Smith P.E.[1]

ABSTRACT

The Department of Interior has been addressing irrigation drainage water quality impacts at several wildlife sites in the west through the National Irrigation Water Quality Program (NIWQP). At Stewart Lake State Wildlife Area located near Jensen, Utah, the main drainage water quality issue has been the trace element selenium. The Jensen Unit of the Central Utah Project irrigates approximately 4,000 acres of land that is comprised mostly of Mancos shale derived soils. The irrigation of these soils leaches selenium into constructed drains and natural seeps. Selenium in the water entering the lake through four subsurface irrigation drains averages about 40 ug/L parts per billion (ppb), concentrations known to cause deformities in nesting waterfowl and impaired reproduction in most fish species. Over time, this drainage water has also greatly elevated the selenium concentrations in the shallow lakebed sediments, from background levels common in the area of less than 2 ppm (ug/g), to concentrations averaging 20 ppm (ug/g). These concentrations are also known to cause impairment to fish and wildlife resources.

The drains have been intercepted and routed around the lake, but there is still some ground water seepage to be intercepted before year-round water quality objectives can be maintained. Remediating the selenium in the sediments is the main challenge and priority of the NIWQP. Much has been learned by trial and error, but we have not yet been successful in significantly reducing the concentrations of the lake sediments. This paper will address what has been tried to date and what is being proposed to complete the remediation efforts begun in 1997 at Stewart Lake.

SELENIUM IMPACTS TO ENDANGERED FISH IN THE COLORADO RIVER

The trace element selenium has been identified in certain areas of the Upper Colorado River Basin at concentrations known to cause problems to fish and wildlife. Selenium is a trace element that occurs naturally in many geologic formations throughout the western United States. Animals and humans need selenium in small quantities to stay healthy; however, elevated amounts cause deformities in livestock, fish, and waterfowl.

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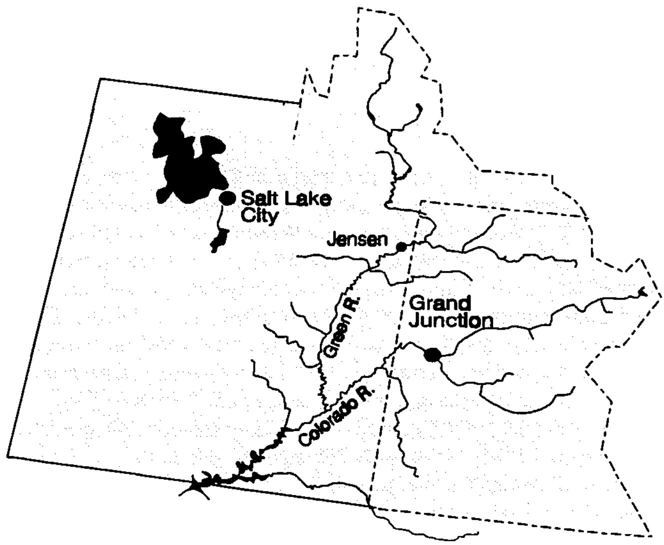


WHY STEWART LAKE IS IMPORTANT

One of the last naturally producing populations of razorback suckers is found in the Green River near Jensen, Utah. Stewart Lake is a large wetland that connects to the Green River during the runoff season, providing endangered fish nursery habitat. Although the water quality in the Green River is excellent, much of the backwater and flooded bottomland habitat has disappeared due to flow regulations created by Flaming Gorge Dam. Elevated levels of selenium negatively impact some of the critical razorback fish habitat that remains.

SELENIUM REMEDIATION AT STEWART LAKE IN EASTERN UTAH

Selenium in irrigation drain water from adjacent irrigated land entered the Stewart Lake Waterfowl Management Area from 1980 to 1998. The selenium has contaminated the bottom sediments and is causing damage to fish and wildlife resources in Stewart Lake, its outlet channel, and in the mixing zone in the Green River.

Two of the main backwater areas that the adult razorback sucker stage prior to spawning were found to be highly elevated with the trace element selenium. Both of these areas are part of the Stewart Lake Waterfowl Management Area. Stewart Lake provides both food and habitat to these fish in the spring of each year when the adults are getting ready to spawn and then in the summer when the larval fish seek out food. Selenium concentrations in the drain water average approximately

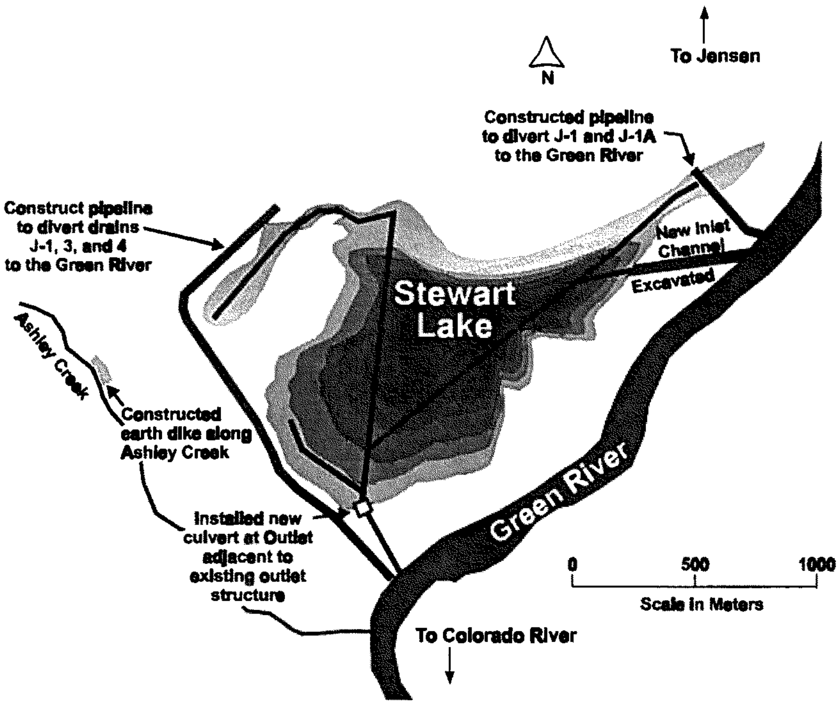
50 ppb (ug/L). Selenium in excess of 4 ppb (ug/L) in water has been found to cause impacts to wildlife. As a result, the food items in these two areas have high concentrations of selenium and cause the fish to bioconcentrate selenium in their eggs to levels that appear to be incapable of surviving. The lack of young fish caught supports this conclusion. To address this problem, an environmental assessment (EA) was completed by the NIWQP in September of 1997 and proposed action was selected. It was decided that the Stewart Lake Waterfowl Management Area would be remediated to benefit both the endangered fish as well as nesting migratory waterfowl. The main source of selenium to Stewart Lake is four subsurface drains, which provide drainage to 4,500 acres (1,800 hectares), of irrigated land in the Jensen Unit of the Central Utah Project.

The goal of the proposed action in the EA is to restore the biological productivity of Stewart Lake to the maximum extent possible and to eliminate selenium hazards to endangered fish and waterfowl. Stewart Lake becomes a large backwater of the Green River during and immediately after spawning by the razorback sucker. The proposed action, which has largely been implemented, is aimed at reducing the selenium levels in the water, sediment, aquatic organisms, plants, and invertebrates that fish and wildlife consume in the lake, its outlet channel, and the mixing zone of the Green River.

Remediation activities by the Bureau of Reclamation (BOR) began in May of 1997 and include: (1) excavating a new inlet channel, (2) excavating channels to drain the lake, (3) extending the drains to the Green River, and (4) monitoring and evaluation.

A new inlet channel was excavated to connect Stewart Lake to the Green River. Enlarging the outlet on the opposite side of the lake has allowed the Green River water to move through Stewart Lake more easily. This is part of a pilot study to see if "flushing" the lake with clean water can effectively reduce selenium in lake sediments and food for fish and waterfowl. Water from the Green River during the runoff season, late May to early July, usually has less than 1 ppb (ug/L) of selenium in it.

Complete drainage of Stewart Lake was not possible without lowering the outlet from Stewart Lake to the Green River by about three feet and excavating drainage channels. It was initially hoped that draining the lake would reduce selenium in sediments by enhancing oxidation. The inability to completely drain the lake required using an amphibious hydraulic excavator to dig drainage channels through the lake. The main drainage channel is more than 5,000–ft long and 15-ft wide, and was excavated about three feet deep. The existing outlet channel was deepened approximately two feet and bypassed the existing outlet structure. A temporary 6-ft diameter culvert crossing was installed to provide access to the southwest side of Stewart Lake. The lake was essentially dry three weeks after the drainage channels were completed.



Extending the existing agricultural drains to the Green River has eliminated the main source of selenium to Stewart Lake, and is a very critical element in its recovery. Two of the five irrigation drains were extended to the Green River during November 1997. Approximately 1500 feet of 18-in diameter pipe connect these drains to the Green River. The drain discharge of about 3.5 ft^3/sec is small compared to the 5,000 ft^3/sec flowing in the Green River, which will provide plenty of dilution capacity for the small increase in selenium load. In 1998, the three remaining agricultural drains were connected and extended around the southwest end of Stewart Lake to the Green River. Approximately 7,500 feet of 24-in diameter pipe connect these drains to the Green River.

Inlet and outlet water control structures were constructed for Stewart Lake during May of 1999. The outlet structure made it possible to store water in Stewart Lake again. The summer of 1999 was the first time since 1996 that the lake was flooded for a period longer than the runoff season. In 2000 the lake was flooded for 30 days and in 2001 the lake was flooded for about 20 days. The amount of flushing has decreased each year since 1997, as demonstrated in Table 1, due to low spring runoff.

Year	Acre-Feet	Meters^3
1997	36,000	44,000,000
1998	11,800	14,600,000
1999	13,000	16,000,000
2000	3,300	4,100,000
2001	2,000	2,500,000

Table 1. Volume of Water from Green River

Bottom sediment from the lake was sampled in 1995 and again in 1997 after the flooding. The 1997 flooding may have reduced the selenium concentration of the near-surface bottom sediment in Stewart Lake by 5 to 25 percent (1,000 to 5,500 lbs.). Additional monitoring is needed to verify this estimate and to assess what is happening to sediment concentrations over the long term.

Biological sampling has helped biologists from the Utah Division of Wildlife Resources and the U.S. Fish and Wildlife Service quantify selenium in the wildlife at Stewart Lake. Biological monitoring has included the collection of invertebrates, plants, fish, and water-bird eggs from various locations in Stewart Lake and the Green River both above and below the lake. The results have indicated that even with good quality water in the lake, the flooding action alone has not been very beneficial in reducing selenium in biota.

REMEDIATING SELENIUM IN STEWART LAKE SEDIMENTS

The attempts so far to mobilize the selenium in the Stewart Lake sediments have been predicated on the idea that exposure to the atmosphere would be sufficient to oxidize and mobilize the selenium in the sediments. Prior to 1997 the lake was not drainable which made it difficult to sample and provided no opportunity for wet and dry cycles for the sediments. With construction of the inlet channel in 1997 it was hoped that large volumes of fresh water from the Green River would flush selenium from the sediments. The 1997 Green River runoff was very high and prolonged. An estimated 36,000 acre-feet of water flowed into and out of the lake over an eight-week period, extending into July. Results from sediment sampling in September of 1997 looked promising. It looked like the selenium concentration in sediments had been reduced by up to 25 percent compared to sediment sampling in 1995.

Twenty permanent monitoring sites were established with 15-foot tall steel poles as markers during the September 1997 sediment sampling effort. The locations of these sites were saved using a Global Positioning System (GPS). This was the first dry sampling opportunity since the lake was drained for the first time. Previous lake-wide sediment sampling had been performed in 1995 from a boat. A GPS was used in 1995 to sample sediments on an 800-foot grid. Since 1997, the twenty sites have been sampled once a year.

During the 1997 runoff season, approximately 36,000 acre-feet (44,000,000 m^3) of high quality Green River water flowed through the lake over a period of six weeks. During the 1998 and 1999 runoff season, approximately 11,800 acre-feet (14,000,000 m^3) and 13,000 acre-feet (16,000,000 m^3) of Green River flowed through the lake, respectively. This flushing did not result in similar reductions of selenium in the sediments. Due to much lower runoff seasons and less efficient operation of the inlet gate, only 3,300 acre-feet (4,100,000 m^3) of water flowed into the lake in 2000, and 2,000 acre-feet (2,500,000 m^3) of water flowed into the lake in 2001.

PERSISTENT PROBLEM

Even after extending the buried pipe drains to the Green River, there appears to be enough highly concentrated selenium in ground water springs entering the north end of the lake to mask any gains from flushing. Both the endangered razorback sucker and Colorado squawfish have been documented to use Stewart Lake when Middle Green River flows were diverted to the lake, in 1997, 1998, 1999, and 2000. In 2000, 100 razorback suckers were released into Stewart Lake. These fish were raised in a hatchery and five were caught during the draining of the lake and had increased to 5 ppm (ug/g) selenium in their muscle tissue after only 30 days indicating that there is still a high selenium concentrated food source in the wetland.

TILLING SEDIMENTS

In November of 1999 it was decided to set up some test plots to determine if aerations of the sediments by tilling the top six-inches would increase selenium leaching during flooding. We selected three, 10,000 ft^2 sites with adjacent control sites and performed composite sampling before and after tilling for selenium analysis. We sampled the following increments: 0-3-inches, 3-6-inches, 6-12-inches, 12-18-inches, 18-30-inches, and 30-36-inches. Sampling the same increments followed by tilling was performed in April of 2000, before the lake flooded, and again in August of 2000, after the lake was drained. There was no significant reduction in selenium concentrations from the tilled sites.

APPLICATION OF QUICKLIME (CAO) AND SLAKED LIME (CA(OH)2)

To date the selenium apparently has remained predominantly in the Se IV state (selenite). Se IV is actually relatively soluble. However, it is strongly adsorbed, in which case it becomes immobile (James Yahnke, p. 16 of *Report on the April 2000 Sediment Samples*). Previous work done in the San Joaquin Valley in association with the selenium poisoning incident due to contamination at Kesterson Reservoir showed that selenium adsorption was highly pH dependent (Neal et al., 1987). That work showed that the percent of Se IV adsorbed declined to essentially 0 at a pH greater than 9. This was investigated in the Bureau of

Reclamation (BOR) Soils Lab in Denver on a bench scale. The results indicate that the soluble selenium in Stewart Lake sediments increased from 13 ppb (ug/L) at the unadjusted pH of 7.5 to 1920 ppm (ug/g) when enough lime was added to raise pH to 10.5. This result indicated that the selenium in the Stewart Lake sediments could be mobilized by adjusting pH. In May of 2001, we added 2000 pounds of CaO to 5,000 ft^2 of each of the tilled up test plots and 2,150 pounds of Ca(OH)2 to the other 5,000 ft^2 of the tilled plots.

Initial results indicate that selenium sediment concentrations were reduced by up to 10 ppm on these test plots. The CaO seemed to be the more effective soil amendment. A larger scale pilot study is planned for the spring of 2002 to see if similar results can be realized using CaO.

REMOVING SEDIMENT

As can be seen from Figure 1, reducing selenium concentrations to 4 ppm (ug/g) is going to take more than just flushing. If soil amendments with lime are not effective at lowering selenium concentrations to 4 ppm (ug/g), removing sediment may be the only way to reach this goal. This would involve removing up to 500,000 yd^3 of sediment over about 400 acres of the lakebed.

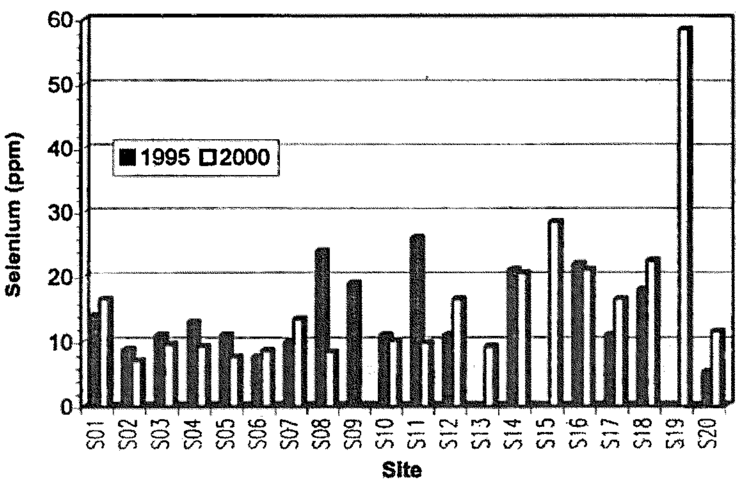


Figure 1. Plot of 1995 and 2000 Total Selenium in the 0-6 –inch Sediments

CONCLUSION

The process of remediating selenium impacts to fish and wildlife resources at Stewart Lake is complex. Adaptive management is the common approach being implemented to realize the greatest opportunity for success. With continued efforts to remediate selenium problems related to irrigation drainage, fish and wildlife resources, including the endangered fish found in the Colorado River, will benefit from one less variable working against their survival.

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BREAKING THE TECHNOLOGY BARRIERS IMPOSED BY CAST-IN-PLACE CONCRETE PIPE IN IRRIGATION DISTRICTS
- CASE STUDY OF SOUTH SAN JOAQUIN IRRIGATION DISTRICT-

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ABSTRACT

South San Joaquin Irrigation District (SSJID) in Manteca, California, is beginning an ambitious modernization program to increase its water delivery flexibility. The district has over 200 miles of 30-60 inch cast-in-place (CIP) concrete pipeline that currently allow for little flexibility. SSJID will install four reinforced concrete interceptor pipelines and regulating reservoirs to redistribute water among the CIP pipelines and provide improved flexibility. The district's goal is to improve efficiency and encourage farmers with pressurized irrigation systems to shift from well water to surface water.

INTRODUCTION

South San Joaquin Irrigation District (SSJID) consists of 71,000 acres of which 59,650 acres are irrigated. It is located in Central California near the town of Manteca. The primary crops are almonds (56%), corn/oats (10%), and grapes (7%). The farm irrigation methods are border strip and furrow irrigation (56%), drip (3%), microsprayer (6%), undertree sprinkler (32%) and miscellaneous (3%). Its water distribution system consists of a main canal that brings water into the service area, plus 3 smaller distribution canals. Although the canals do provide some direct delivery turnouts to farmers, the vast majority of deliveries are made from large diameter (762 mm – 1220 mm) monolithic, cast-in-place (CIP) concrete pipe. All flows are gravity; pumps are not used. The pipeline sizes and their lengths are shown in Table 1.

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Table 1. CIP Pipeline Diameters and Lengths in SSJID.

Pipe Diameter		Lengths	
inches	mm.	feet	meters
30	762	51,363	15,655
36	914	595,024	181,363
42	1067	21,050	6,416
48	1219	446,808	136,187
60	1524	8,124	2,476

CIP pipe is common in irrigation districts on the east side of the San Joaquin Valley, where approximately 2.3 million meters are in place (Burt and Wegener, 1994). The pipe is usually 40+ years old, and as a result suffers from a high incidence of cracking – hence the local joke that it is "cracked in place" pipeline. In SSJID, applying more than 1.5m of pressure on the pipe generally results in excessive leakage.

SSJID irrigation district is one of the few districts remaining in California that supplies water on a rotational schedule. Historically, farmers surface irrigated with large flow rates of .6 - .7 m^3s^{-1} (20 – 25 CFS). The CIP system was designed to deliver 1-2 "heads" of this size simultaneously through any one pipeline. Deliveries are often made to individual border strips directly from large alfalfa valves placed on top of the district pipelines. There are very few of the standard "turnouts" common in most irrigation districts; that is, there is not a single delivery point and flow meter for each 10-30 ha field. Instead, a one or two "head" flow rate enters at the head of each pipeline, and one or two farmers receive water simultaneously. The only flow rate measurements are estimates at the heads of the pipelines. Farmers are charged a per-acre fee rather than a volumetric fee for water.

The present pipeline system is very inflexible, and has bottlenecks in the lower ends of the district. Some of the pipelines are 10 miles (16 km) long (Figure 1). With the long pipelines it is difficult to quickly switch supplies from an area of temporary excess to one of deficit. As a result, spills occur on some pipelines while other areas of the district lack sufficient water. Because of the inflexibility, the system is incapable of providing surface water to the increasing number of growers who are switching to sprinkler or drip/micro irrigation. Those growers rely on groundwater instead of surface water – resulting in extractions from the aquifer while simultaneously reducing groundwater recharge.

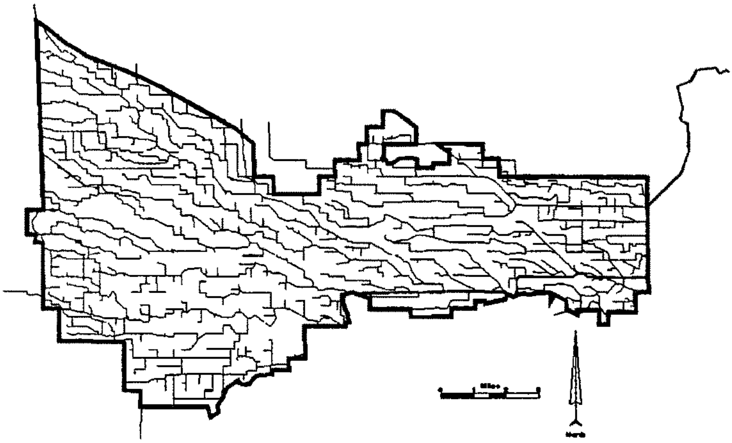

Fig. 1. Present Water Distribution System in SSJID.

SSJID has an existing Supervisory Control and Data Acquisition (SCADA) system on its canal system. The Main Canal (shown as a darker line in Figure 1, originating on the NE side of the district), which uses automatic overshot hinge gates, has Remote Terminal Units (RTUs) at each gate. The gates are automated for upstream control. ITRC provided the control algorithm and tuning constants for those gates at an earlier date.

MODERNIZATION OF SSJID

Background of the Present Modernization

SSJID's management and Board of Directors have recognized the need for improved flexibility and reliability in water deliveries. In 1997, the senior author was asked to do a Rapid Assessment of the district and brainstorm ideas for improvement. There were numerous small improvements that could be made, but the Rapid Assessment identified two strategic recommendations:

1. Eliminate several capacity bottlenecks in the distribution system.
2. Install two interceptor pipelines that would cross perpendicular to many of the existing CIP pipelines. These new interceptors would both receive and supply water from existing CIP pipelines. One of the interceptors would be sized to supply water to a new grid of pressurized lateral pipelines which would facilitate water delivery to drip/micro irrigation systems.

At the end of 1999, SSJID again contacted Cal Poly ITRC and expressed a desire to obtain a preliminary design that incorporated the 1997 recommendations. In

July 2000, ITRC provided an initial framework for the System Improvements for Distribution Efficiency (SIDE) project, with a rough cost estimate of $13.8 million. Further discussions resulted in the addition of several sub-projects. In late 2000, the California Energy Commission (CEC) funded Cal Poly ITRC to provide technical assistance to districts that wished to explore options for modernizing their CIP pipeline systems. In February 2001, SSJID, with ITRC participation, interviewed consulting engineering companies that could provide the final design drawings and project management; Provost and Pritchard Engineering Group of Fresno was selected. SSJID is using Northern Digital, Inc. of Bakersfield as the SCADA integrator.

The project will be installed in phases. One area (the "Northwest Pipeline") was chosen for the first phase because it incorporates almost all of the engineering features that are contained in the other larger sub-projects. Construction of some pipelines and a reservoir for the Northwest pipeline began in Fall 2001, and completion of this sub-project is expected in July 2002. A description of the project and some aspects of its implementation follows.

Modernization Strategy

The modernization strategy recognized the following points:

1. It is infeasible to replace all of the CIP pipelines. Replacement costs are approximately $100/ft ($328/m).
2. The field layouts for surface irrigation require the high flow rates that are currently being provided. Therefore, any design options must guarantee the same high flow rate deliveries.
3. Members of the Board of Directors are elected from geographical regions within the district, so it is politically important to improve service to all areas of the district.
4. The farmers at the downstream end of the system (the west side) generally farm annual crops. They believe that their primary constraint is the lack of flexibility (and water) because of bottlenecks upstream.
5. Farmers in the eastern side of the district, who primarily grow almonds and apples, are shifting away from surface irrigation and are installing drip/micro and sprinkler systems that use well water. They would use surface water if it could be supplied with more flexibility and better filtration.
6. Most of the CIP pipelines are operated partially full or at very low pressures to minimize seepage losses. This means that they are operated as upstream controlled canals. Many hours are required before a change in flow rate moves from the inlet end of a pipe to the downstream end.

The strategy for modernization treated the pipeline distribution system as if it was similar to a canal system. Several perpendicular interceptor pipelines, each with a regulating reservoir, were designed. Each interceptor pipeline will provide the following functions:

New Supply: First, they establish a "new supply" for the existing CIP pipelines. At its intersection with the CIP pipe, the interceptor pipeline will be able to add water from a regulating reservoir or from other pipelines that have an excess supply. This water will be available to downstream users regardless of the amount of water used by irrigators upstream of the interceptor pipeline. This not only improves the service to downstream users by introducing an additional supply of water into the existing pipelines; it also improves the service to upstream users because they can now use water that would otherwise have been reserved to supply growers further down the line.

Flexibility: Second, the interceptor pipeline can recover excess water in the existing pipelines and move it to other crossing pipelines that are short of water. If there are no shortages in other pipelines, it can move the water to the buffer reservoir where it will be stored until needed. However, this "minimization of waste" is actually a minor element when one considers the requirements for the future. The primary advantage of this project is that upstream CIP operators and users will no longer need to follow precise schedules in duration and flow rate; discrepancies will simply move downstream and be absorbed by the interceptor pipeline. System operators will be able to err "on the high side" of the amount of water sent down a pipeline without fear of losing the excess water to spillage, resulting in improved water delivery service for all the users along the pipeline.

Distribution System for Pressurized Irrigation: Finally, some of the interceptor pipelines will form the supply backbone of new distribution systems for irrigators that use sprinkler or drip irrigation. With the installation of new laterals, adjacent farmland will have a supply of water under moderate pressure that can be boosted to the appropriate irrigation pressure using private on-farm booster pumps.

Design Features

Interceptor Pipelines: The project includes 4 interceptor pipelines. The interceptor pipelines are constructed of reinforced concrete pipe. Sizes range from 60" – 36" (1524 mm – 914 mm). The pipelines will be continually pressurized by pumps at interceptor boxes and at the regulating reservoirs. The pipelines were initially sized using an economic pipe size method to obtain the lowest annual cost of power and fixed costs. However, the economic pipe sizes were small enough that, depending upon the magnitude and location of flows, considerable pressure fluctuations could occur in the pipeline. In order to select pumps and valves (described below) that could be automated easily, it was desirable to have less variation in pipeline pressures. Therefore, several pipe sizes were increased to lower the friction.

Interceptor Boxes: At each point where the interceptor crosses a key CIP pipeline, an interceptor box will be installed in the existing CIP pipeline. An interceptor box will have two functions:

1. It will serve as the beginning of the downstream CIP pipeline segment. The flow rate will be controlled and measured at this point.
2. It will serve as the end of the upstream CIP pipeline segment. Any excess flow from the upstream segment will be recaptured.

To accomplish these functions, there must be mechanisms to:

1. Supply the interceptor boxes with any additional flow that is needed downstream.
2. Remove excess flow that enters the boxes from the upstream segments.

The interceptor pipeline will serve both purposes, of supplying water and accepting excess flow. Figures 2-4 show the basic design of the interceptor boxes.

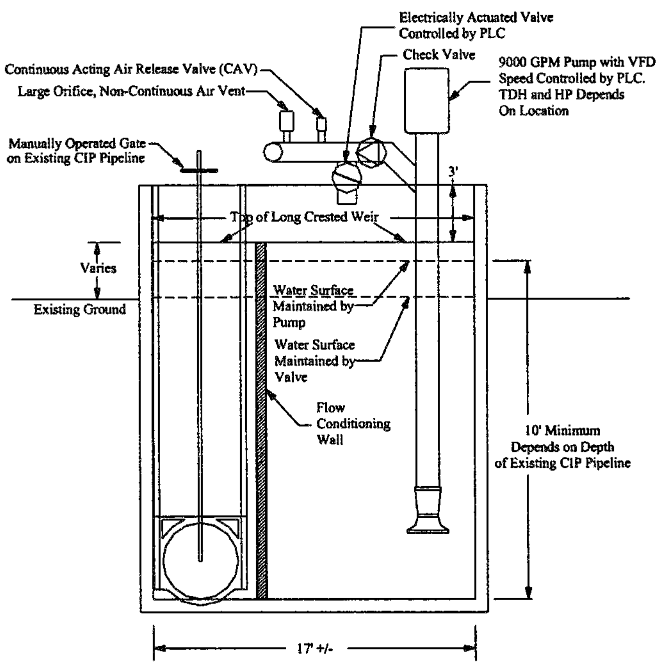

Fig. 2. Front View of an Interceptor Box (Not to Scale).

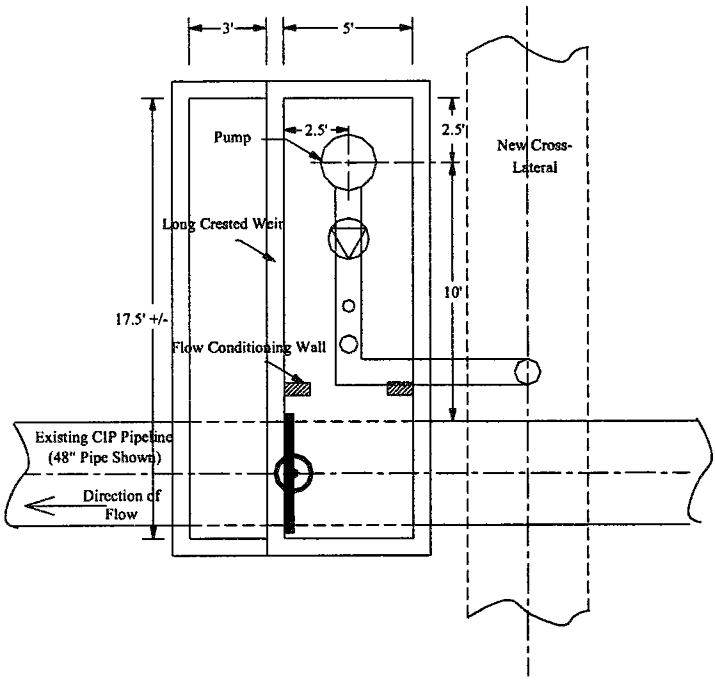

Fig. 3. Plan View of an Interceptor Box (Not to Scale).

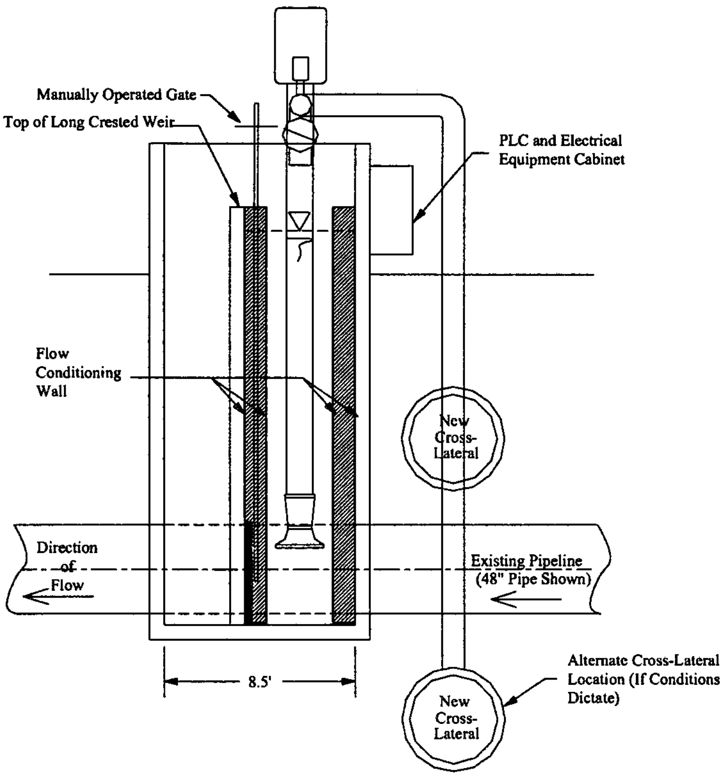

Fig. 4. End View of an Interceptor Box (Not to Scale).

Figures 2-4 illustrate the following physical features:

1. One or two pumps with variable frequency drive motors will be installed in each box. The pump will prevent the water level in the box, upstream of the weir, from rising too high when excess water enters from the upstream segment of the CIP pipeline. The pump will automatically place any excess flow that enters the box into the interceptor pipeline ("new cross-lateral"). The pump responds to the water level in the box.
2. A valve will automatically supply water from the new interceptor pipeline into the box if the water level drops in the box upstream of the weir. The valve responds to the water level in the box.

3. A wall with a long crested weir separates the upstream and downstream CIP pipeline segments. The design allows flows to pass through the box in case the pump malfunctions.
4. The flow rate into the downstream segment of the CIP pipeline will be controlled with a sluice gate built into the long crested weir wall. Initially, the flow rate will be manually controlled. Ultimately, the gate may be motorized and automated. The water level upstream of the sluice gate will be maintained sufficiently high and constant to provide a fairly constant, manually controlled flow into the downstream pipe segment.
5. A flow measurement device (not shown) will be installed on the CIP pipeline, downstream of the box. This will have both a visual and analog output. The meter will allow the operator to set the flow rate properly, and will eventually allow automatic flow rate control.
6. Remote monitoring (not shown). Each box will be remotely monitored through SSJID's SCADA system. From the control center, it will be possible to change any of the automatic pump or valve control parameters, or to bypass automatic features and remotely manually operate the pumps or valves.
7. A rotating trash screen belt (not shown) was designed to filter water before it enters the pump(s). This is needed to remove debris that enters the pipe between the source and the interceptor pipeline.

<u>Regulating Reservoirs</u>: Each of the 4 interceptor pipelines must have a reservoir to supply water to the interceptor pipelines when there is a deficit, or receive water from the pipelines when there is an excess. Excess and deficit flow rates will automatically shift within the interceptor pipeline between interceptor boxes that have excess and deficit flows. Three of the interceptor pipelines require new reservoir construction; the fourth will use the Main Canal as a reservoir. The Main Canal has a regulating reservoir built into it, downstream of the point where the new interceptor will connect to the Main Canal.

The regulating reservoirs will include pumps to supply water to the interceptor pipelines. The pumps at the regulating reservoir will be programmed to respond to the pressure in the interceptor pipelines. If the pressure drops to a pre-set level, the pumps will inject additional water into the interceptor pipeline. Likewise, the valves will open and discharge water into the reservoirs if the pressure in the interceptor pipeline rises too high.

<u>Supply to the Regulating Reservoirs</u>: In two locations, additional CIP pipeline inter-ties will be installed upstream of the interceptor pipelines to ensure that additional water can be quickly brought into the regulating reservoirs when needed. One of the regulating reservoirs is adjacent to a canal (the "R" canal). This canal's cross regulators (check structures) between the Main Canal and the regulating reservoir will be modified with long crested weirs and ITRC Flap Gates. These modifications will allow SSJID to easily vary flows down the R

canal to the regulating reservoir without needing frequent check structure adjustments. The changes can be made by simply changing the flow rate at the head of the R canal.

The water level in each regulating reservoir will be remotely monitored. The regulating reservoirs will have sufficient capacity to receive or supply a discrepancy in flow rate for about 5 hours. This is enough time for the SSJID operators to make adjustments to flows at the heads of pipelines or canals and to have those flows reach the regulating reservoirs before the reservoir dries up or overflows.

Example Interceptor

The Southwest Pipeline, shown in Figure 5, is an example interceptor. It is about 3.2 miles long, and is generally aligned in a north-south direction. It will intercept 5 CIP pipelines that notoriously have discrepancies between supply and demand. The TBB pipeline (on the north end of the interceptor) typically has excess capacity, while the southern pipelines often suffer deficits. The new reservoir will have a live storage capacity of about 41 Acre-feet.

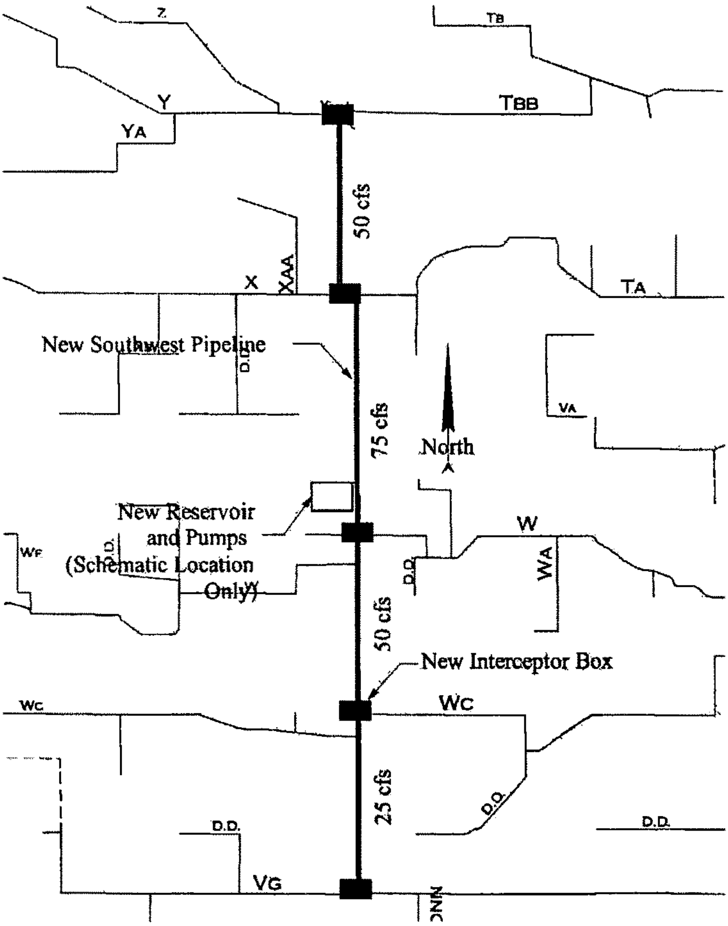

Fig. 5. Southwest Pipeline.

Implementation

Phase 1: Construction began on the Northwest Pipeline (Figure 6) in Fall 2001, and this first phase is expected to be operational by September 2002. Full operational status requires that the check structures in the R canal be modified. Those structures will be upgraded for improved upstream control in Fall 2002.

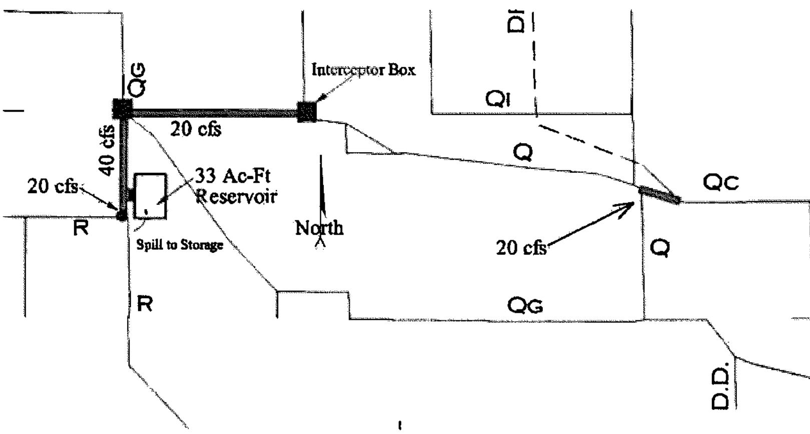

Fig. 6. Northwest Pipeline – Initial Layout.

Interesting aspects of the Northwest Pipeline implementation have been:

1. The operational concepts depicted in Figure 6 remain the unchanged (interception of the Q, Q_C, and Q_G pipelines and the R canal).
2. The actual locations of the new pipelines and the reservoir have been shifted because of the district's desire to purchase rights-of-way and land from willing landowners.
3. Obtaining rights-of-way that simultaneously satisfy land owner desires and the hydraulic requirements of the interceptor systems has required considerable effort in surveying, landowner discussions, land value assessment, identification of legal ownership records, etc.
4. The active involvement of SSJID operators has been considered to be a key ingredient for the eventual success of the project. The operators are involved in all meetings that involve the engineers and SSJID management. Because of the active solicitation by SSJID management of input from the operators, the operators have brainstormed among themselves and have come up with tough questions and excellent recommendations for modifications and improvement. Among other things, operators have identified several inter-ties between pipelines (similar to the Q – Q_C connection shown above) that they feel will enable them to maximize the benefit of the changes.
5. The proposed operation of the systems will involve considerably more flexibility than the operators are accustomed to. Their initial thinking focused on supplying extra water to some areas in times of shortage. The concept of also using reservoirs to <u>receive</u> temporary <u>excess</u> water as part of a flexible operation has been more difficult to understand.
6. While some districts would do much of the construction with in-house employees, SSJID has decided to contract out all of the work. This, of

course, requires very detailed engineering drawings for all aspects of the civil and electrical works.

Using Surface Water for Drip/Micro: It will be a challenge to convince many growers to use surface water rather than well water for drip/micro and sprinkler systems – even if the water is delivered with a high degree of flexibility. This is because the depth to groundwater is still relatively shallow (less than 30 meters) in most areas, and because the surface water requires much more elaborate filtration than well water. Based on discussions with farmers, the following points are noted:

1. Pressurized laterals can be installed on each of the interceptors to provide water to farmers with a minimum of 5' (1.5 m) of pressure.
2. The district has decided that it will install outlets on the interceptors at potential PVC lateral locations. However, it will only install the laterals when farmers join together and form improvement districts to pay for individual laterals.
3. Rather than install the moving trash screens at each interceptor box, SSJID is now considering the installation of the trash screens at the heads of the CIP pipelines – where they receive water from the canals. This will provide coarsely filtered water to all farmers, not just those taking water from the interceptors. Currently, some dairy farmers use the CIP pipes as drains – thereby introducing straw and manure into the pipelines. That practice will be prohibited in the future.
4. The shift to surface water will be stimulated in some areas of the district by the decreasing groundwater quality. In some areas, the groundwater is high in salts and nitrates – both detrimental to tree fruit growth.

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PARTNERSHIPS IN APPLYING NEW AND INNOVATIVE TECHNOLOGY

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ABSTRACT

The Roza Irrigation District (Roza), in the Yakima Valley of Washington State, received only 37 percent of its water right entitlement in the 1994 and 2001 irrigation seasons due to its junior water status in the basin. A reduction in water allocation of this magnitude places a great deal of stress on farmers and their crops, and hinders the District's ability to deliver water. Faced with having to operate with less than 50 percent (600 cfs) of design flow, Roza has become one of the most progressive irrigation districts in the Pacific Northwest with respect to water conservation by applying new and innovative technology.

Since 1991, Roza has developed numerous partnerships to help fund and apply resourceful technology to water conservation measures. Initially, Armtec and UMA introduced the automated single-leaf over shot gate, which was applied to numerous main canal checks to provide constant level control for pipeline turnouts and pumping plants. One of Roza's most successful partnerships has been with the United States Bureau of Reclamation (Bureau) Water Conservation Field Services Program in Yakima, and Aqua Systems 2000 Inc. (AS2I) from Lethbridge, Alberta, Canada. The partnership began in 1997 when an existing flashboard check structure was replaced with an automated Langemann gate. The success of this project, cost shared with the Bureau, prompted Roza to discontinue building new multi-bay single-leaf over shot structures and building or retrofitting eight structures with a single Langemann gate, the largest being 24-feet wide.

The partnership with the Bureau and AS2I expanded to include Supervisory Control and Data Acquisition (SCADA) on two existing main canal checks and a re-regulation reservoir. These innovative projects include flow control using undershot gates, spill flow monitoring using ultrasonic level technology, downstream canal level control using a variable speed pump and modulating valve, and remote communications via spread spectrum radio, dedicated phone line, and cellular digital modem technology. The SCADA system was recently expanded with the incorporation of an Acoustic Doppler Flow Meter (ADFM) downstream of the diversion point. In addition, Roza also partnered with MagnaDrive, Inc. (MagnaDrive) and AS2I to demonstrate variable speed pump control on an existing pumping plant. MagnaDrive supplied a magnetic coupler rather than a variable frequency drive (VFD), and AS2I provided the controls

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design and software to the flow control project.

The success of these projects has enabled Roza to reduce low-end operational flows from 600 cfs to 280 cfs in water short years and still provide reliable service to its water users.

INTRODUCTION

The Roza Irrigation District (Roza) is located in South Central Washington, along the eastern slopes of the Cascades Mountains in the lower Yakima River Basin. Water is supplied to the basin from the Yakima River and five storage reservoirs. Irrigation water is diverted from the Yakima River in the Ellensburg Canyon at the Roza Diversion Dam. The Roza Main canal is 94.8 miles long, serving 72,000 acres lying along the northern rim of the lower Yakima Basin. The District delivers water to 45,000 acres below the main canal by gravity and 27,000 acres above the main canal via 18 pumping plants.

Roza has a junior water right entitlement for 375,000 acre-feet of contract water. In water short years, senior water rights are met first and the remaining water is shared or prorated among the junior water right holders. There have been 9 years of pro-ration since the Roza was constructed; the worst by far were 1994 and 2001 when Roza received only 37% of its water entitlement. For Roza, conservation has always been a top priority due to its junior water right status.

The District has had a Bureau approved Comprehensive Water Conservation Plan (CWCP) since 1992. The plan includes the replacement of open ditches and concrete pipes with PVC pipes, termed the Enclosed Conduit System (ECS): lining earthen laterals with geo-membranes; constructing automated check structures; retrofitting existing Bureau structures into automated check structures; construction of re-regulation reservoirs; and SCADA. To date, Roza has converted 33,000 acres from open ditches or concrete pipe to the ECS, constructed seven new automated check structures, retrofitted 25% of the original Bureau check structures with automated gates, built two re-regulation reservoirs and nearly completed the SCADA system. This aggressive program has been completed in collaboration with the Bureau and private industry.

SINGLE-LEAF OVERSHOT GATES

Check structures are instrumental in conserving water by regulating water level at diversion points, regardless of the main canal flow rate. Without check structures, additional "operational" water would have to be diverted beyond that required to satisfy irrigation demand in order for the main canal to operate effectively.

Roza has constructed six 3-bay single leaf overshot check structures since 1992. The bays range in size from 7 to 10 feet, providing design capacity ranges from 350 cfs to 1150 cfs. For the first structure, Roza partnered with UMA, Inc. (UMA) and Armtec, both out of Lethbridge, Alberta, Canada, whereby UMA provided the structure design and gate sizing while Armtec provided the gates,

hoists, control equipment, gate installation supervision and commissioning. Roza provided the design criteria and the construction.
During this time, Hydrogate, Inc. began manufacturing overshot leaf gates, and the bid process was separated into two contracts, one for the purchase of the overshot gates and one for the supply, installation and commissioning of control equipment. In 1996 Roza partnered with UMA to purchase a control software site license for future overshot leaf gate structures.

In general, the overshot leaf gates have performed very well and continue to operate with very few problems. The partnerships formed with UMA, Armtec, Hydrogate and local electrical contractors were satisfactory.

LANGEMANN GATES

In 1997 Roza was approached by AS2I to utilize a Langemann gate. Due to the uniqueness of the gate, Roza approached the Bureau's Field Services Program to evaluate the utility of the gate in a Demonstration Project. The Demonstration Project Program is a "win-win" partnership for the Bureau and any interested district. The Bureau promotes water conservation projects that demonstrate applications of new technology or a unique application of an existing technology while the Districts acquire conservation projects at one-half the cost. The Langemann gate was a perfect candidate for a demonstration project, and with the Bureau sharing half of the project costs, the District's financial risk was minimized.

The unique and simple design of the Langemann gate makes it easy to retrofit into existing flashboard check structures. The double-hinged, two leaf folding Langemann gate operates in a vertical plane upstream of the structure sill as opposed to the 60-degree horizontal plane of the single leaf gate operating downstream of the sill. Compared to the single leaf over shot gate, installation of a Langemann gate requires very little structure modification. Initially, both the Bureau and the District had concerns about sediment and debris buildup upstream of the gate. However, the low power, standalone Langemann gates have made operation much easier for the watermasters and ditch riders, and concerns of sediment build up and weed and debris clogging proved to be unwarranted. Ditch riders seldom need to pull weeds from a Langemann gate structure.

The success of the first Langemann gate prompted the District to build a new 1150cfs Langemann gate check structure. By utilizing a single 24 ft Langemann gate rather than three 8 ft single leaf gates, Roza was able to reduce the concrete volume and save approximately 15% of the cost when compared to an 800cfs leaf gate structure 20 miles closer to the Roza main office. This was the first check structure in the world built solely for a Langemann gate, the site continues to operate problem free.

The simplicity and versatility of the initial Langemann gate projects convinced the District that all of the remaining check structure projects should be retrofit with Langemann gates. The partnership with AS2I also allowed the District to return to single source responsibility for supply of automated gates. To date, Roza has installed and commissioned a total of 9 Langemann gates of various sizes and flow capacities.

FLOW CONTROL WITH UNDERSHOT GATES

The Roza main canal was constructed with seven waste way sites that utilize undershot gates. For most of the sites, the primary purpose of the undershot gate is to dam up and spill water in case of a canal failure downstream. However, two sites, Waste Way 5 (WW5) and Waste Way 6 (WW6) utilize undershot gates to manage both operational spill and the flow rate downstream.

WW5 is situated at the midway point of the main canal. It consists of a two-bay, 15-foot wide by 7.5-foot radial gate check structure, an automatic siphon and waste way with a gauging station upstream of the check, and a canal gauging station downstream. Prior to automation, the site was operated manually based on upstream and downstream gauge readings, the time of year, and experience. Based on flow rate demand downstream, the gates were adjusted by "turns of the wheel" on the gate hoist. The experience to operate this site was limited to two or three people at the District. As a vital control point in the canal, the site would require adjustments several times a day during periods when main canal fluctuations were critical such as spring frost and drought years. Unwanted fluctuations were typically the result of changes in demand upstream, debris in the canal, or an emergency situation.

WW6 is located approximately three quarters of the way down the main canal. It consists of a single 6-foot wide sluice gate check structure and automatic siphon. There is a waste way and gauging station upstream of the check similar to WW5. Immediately downstream of the gate, the flow is super critical prior to entering a siphon that crosses Snipes Canyon There is a gauging station on the outlet of the siphon. This site was initially automated by the Bureau with Littleman technology. A level transmitter, located at the gauging station across the canyon, was connected to the process control equipment at the upstream check control building. Operation was generally reliable; however a failure of the buried cable crossing the canyon floor prompted the District to re-evaluate automatic control of the site.

To improve water management, Roza partnered with AS2I and the Bureau to implement automated flow control and SCADA monitoring of both sites. AS2I supplied complete controls packages including hardware, software, commissioning and training. Roza installed equipment, supplied and installed interconnecting cabling and assisted during commissioning. Once again, the Bureau provided incentive for the projects by funding them as Demonstration Projects.

The control packages consisted of programmable logic controller (PLC) based control panels, instrumentation and devices, control software, communication link and SCADA host. The control software is based on a multi-bay, undershot gate, level/flow control algorithm developed by UMA Engineering, which was selected because of its proven ability to provide reliable and stable automatic control over a wide range of site parameters and conditions.

The objective of the control software at WW5 was to i) maintain the desired downstream canal flow at an operator selectable set point, ii) provide upstream level overrides to maintain a maximum and minimum canal level upstream of the check and iii) provide flow measurement of the operational spill down the waste way. The objective of the control software at WW6 was similar except that an automated re-regulation reservoir located upstream of the check provided control of the upstream canal level.

Downstream flow control at WW5 is accomplished using the two radial check gates as flow measurement and control elements. Flow through the gates is calculated using a submerged orifice flow formula using the two gate openings and the upstream and downstream levels. Flow control is accomplished by adjusting the gates in response to changes in the measured gate flow. Under normal operations, the automatic siphon provides some hydraulic high level override and assists in maintaining a relatively constant upstream level. However, when canal flow changes dramatically, site control automatically switches to upstream level control. When the upstream canal level exceeds the high-level override set point, or falls below the low-level override set point, flow surpluses or deficits are passed downstream. The waste way flow is calculated using an ultra sonic level transmitter and Manning's equation.

Flow control at WW6 is accomplished using the slide gate as the flow measurement and control element. Flow is calculated using free discharge orifice formula using the gate opening and the upstream level. Flow control is achieved by adjusting the gate in response to changes in gate flow. An ultrasonic level transmitter is also used to measure the waste way level and the flow is calculated using Manning's equation.

A spread spectrum radio provides a real-time communications link between the site and a SCADA system host computer at the RID office. A Windows based graphical user interface manages bidirectional transfer of data from the site providing both graphical and text based representation of current site conditions, and allowing operational set point changes to be made.

A previous attempt to remotely monitor the re-regulation pond at WW6 utilized leased telephone lines for the communications link to the RID office. The original monitoring system was abandoned and the lease line was utilized as the link into the SCADA host computer for the first year of operation of the WW6 flow control system. Operating screens were added to the original WW5 configuration and the addition of a communications multiplexer at the host allowed concurrent communications to the two sites utilizing the different media. A new project completed the following year saw the leased line abandoned in favor of cellular digital wireless technology.

RE-REGULATION PONDS

Operational spills down waste ways 5, 6 and 7 were historically required as part of the operation of the Roza main canal. Manually read gauging stations on these waste ways provided a means to determine instantaneous spill flow once or twice in a day, but Roza knew this return flow would better serve the District needs if it could be stored and used at a later time. As such, Roza designed and constructed two re-regulation ponds upstream of WW6 and WW7. Even though the final configuration of the two systems were different, the intended operation of both ponds was to capture excess flow that would otherwise be diverted down the waste way and utilize the stored water to make up short falls in downstream flow demand.

The pond at WW6 is comprised of a 150 acre-foot reservoir, side spill weir and pump plant. The side spill weir is located upstream of the WW6 check gate and was constructed to allow excess canal flow to spill into the pond. The pumping plant consists of one 25hp variable speed and two 50hp fixed speed vertical turbine pumps with the pump intake structure located inside the banks of the pond and discharging back into the canal. The system was designed for stand-alone operation and was initially controlled using analog process controllers that would start, stop, alternate the pumps and, control the speed of the variable speed pump based on the signal from a level transmitter located in the canal near the side spill weir. Following construction and initial operation, a remote monitoring system was installed that relayed pump and alarm status to the RID office via leased telephone lines. Roza had originally partnered with two local contractors for the supply of the electrical, controls and monitoring systems, but those firms are no longer in business. Following the installation of the WW6 flow control automation, the District once again partnered with AS2I to assist with integrating the WW6 re-reg controls into the District SCADA system.

The pond at WW7 is comprised of an 11 acre-foot reservoir and combination pump/discharge structure on the main canal. This structure consists of a combination intake and discharge well constructed in the side of the canal bank that includes a variable speed 15hp vertical turbine pump and an 18 inch electrically actuated butterfly drain valve. The pump discharge and the drain valve are connected to the same pipe that connects to the low level inlet structure in the elevated pond. Initially, Roza partnered with local electrical and controls contractors to assist with automating the facility, but the system was never able to operate for more than a few hours at a time due to deficiencies in the installation and equipment.

With the onset of several consecutive water short years, Roza partnered with AS2I to provide design, control equipment, programming and commissioning of a new control system for the site. The original variable frequency drive was replaced by the District and an automated Langemann gate was installed at the top of the downstream waste way structure. The system was programmed to perform downstream level control in the canal upstream of the waste way gate by pumping excess canal flow into the reservoir and releasing stored water to make up flow deficits. The primary objective of no spill down the waste way was achieved.

HIGH LIFT PUMPING PLANT CONTROL

Initially, all gravity laterals were to be enclosed with PVC pipe prior to expanding the ECS to the pump laterals. However, specific right-of-way and water quality issues brought about the necessity to enclose the Pump 10 lateral. The Pump 10 pumping plant consisted of two 150hp, 480volt, 1760rpm motors driving horizontal split case pumps operating in parallel. Water was pumped 139 feet above the pumping plant into a head weir box and spilled over a 48-inch weir and into an open gravity ditch flowing to farmer deliveries. The flow rate was manipulated by manually adjusting gate valves on the discharge of the pumps until the desired flow rate was measured over the head weir. Piping the lateral and replacing weir blades with flow meters increases the farmer's flexibility. With weir blades, farmers were unable to shut off there water unassisted if there pipe broke or they needed to move a wheel line. But with the ECS system, farmers are equipped with valves and thus the ability to turn off their deliveries at any time. This would cause water to stack up at the head weir box and overflow if the pump flow rate were not immediately reduced.

Roza recognized the need for automatic flow control at the pumping plant, and decided to retain the head weir box with the ECS pipe delivering from the box and maintain a water level set point in the weir box, regardless of the demand for water. Traditionally, this would be accomplished using a VFD to vary the pump speed. However MagnaDrive was interested in using a Roza application as a showcase and offered a financial incentive to utilize their product on this project. Once again, the use of new technology provided an opportunity to enter into a Demonstration Project partnership with the Bureau as only nine MagnaDrives had been installed worldwide and none at an irrigation district.

The MagnaDrive utilizes the principle of magnetic induction or induced eddy currents. The MagnaDrive coupler (coupler) consists of two components, the magnet rotor assembly and the conductor rotor assembly, which make no physical contact. Relative motion between these two components results in a powerful coupling force between the two rotors, and precisely varying the air gap between the magnets and the copper conductor results in a controlled output. Based on this principle, the motor can operate at its optimal design speed. The coupler varies the pump speed and, thus the flow rate, by manipulating the air gap, by way of an electric actuator, between the rotors and thus controlling the amount of slip.

For the design of the hardware and software to provide level control using the coupler, Roza again partnered with AS2I. Based on the AS2I design, Roza purchased the necessary controller and instrumentation and fabricated and installed the system hardware. A magnetic insertion flow meter from MSR Magmeter was installed in the lower portion of the penstock and connected to the PLC in order to monitor total discharge flow from the pumping plant. An ultra sonic level transmitter, connected to the PLC with buried instrumentation cable, was used to measure the water level in the head weir box. The coupler actuator was controlled via an instrument signal from the PLC, and the software utilized a proportional + integral + derivative (PID) loop to vary the coupler air gap based on the difference between the water level and the set point.

The installation of the MagnaDrive required several modifications of the pumping plant and 4 unsuccessful attempts to complete the project due to miscues by both Roza and MagnaDrive. The magnetic flow meter did not perform as well as expected and in discussion with the supplier it was determined that the location of the sensor was likely too close to the discharge of the pumps creating excessive turbulence. The sensor was relocated further up the penstock and will be tested in the coming operating season.

Even though there was initial uncertainty regarding the final outcome of the project, Roza recognized that this was a demonstration project utilizing new technology in a unique application. In the end, both the District and MagnaDrive learned a great deal from this partnership and are pleased with the successful outcome.

DIVERSION FLOW METERING

Irrigation water for District use is turned into the Roza main canal at the 11 Mile bifurcation works, and was historically measured at the 11 Mile gauging station located just downstream of the bifurcation works. The gauging station is in a trapezoidal lined section of the main canal, and the Littleman controller that controls the radial head gates is housed in the gauge station building. There is also a metering bridge in a rectangular lined flume section of the main canal about ½ mile downstream of the gauge station.

The water depth at the gauge station was reported via radio to the water master once daily, and the water master determined the flow rate from a rating curve chart supplied by the Bureau. The Bureau developed the rating curve of corresponding flow rates and water depths by using a current meter at the measuring station. The Bureau also uses the same rating curve to set a flow rate for the District by determining the required gage height and using the Littleman controller to maintain the target elevation.

Providing flow control for the District in this manner has been contentious as accumulations of moss and debris downstream create backwater at the gauging station which results in overestimates of flow for a given gauge reading. The Bureau periodically performs flow measurements at the metering section to establish correction factors for the rating curve but these corrections sometimes came too late to rectify the difference between actual and measured flow.

Drought situations make it more critical that measured flow equal actual flow. The District decided the 2001 water short year was a good time to install a better metering device. Roza partnered with MGD technologies, Inc. (MGD) to purchase an Acoustic Doppler Flow meter (ADFM). The ADFM uses acoustic pulses and the Doppler affect to measure velocity along with an ultra sonic sensor to measure depth to calculate flow in a known canal cross section. Roza also partnered with AS2I once again to expand the SCADA system to incorporate the 11 Mile gauging station to acquire real time data on flow rate and velocity, and with the Bureau to include this site as a demonstration project for new technology.

Following the first season of operation, it was determined that the ADFM did not perform as well as anticipated due to wave action created by the meter's proximity to the diversion gates. MGD claims the ADFM should be accurate within 2 % of the average of the total flow readings, but instantaneous readings were found to be 4 % to 8 % off of the actual flow rate. In an attempt to improve performance, Roza moved the ADFM ½ mile farther downstream to the rectangular metering section where the wave action is substantially reduced, and AS2I added a moving averaging algorithm to the site RTU. If the ADFM proves to be a superior flow measurement instrument, the hope is that the Bureau will utilize the device to automate the canal inlet radial gates and ultimately solve the issue of backwater affecting the main canal flow rate.

SCADA SYSTEM & COMMUNICATIONS TECHNOLOGY

Over the course of implementing its Comprehensive Water Conservation Plan Conservation Plan (CWCP), Roza has moved towards implementing a SCADA system that connects the key operation points in the District to the Roza office. From the District's point of view, automated main canal checks performing level control do not warrant remote communications, but key flow measurement and control points, such as 11 Mile gauging station, WW5, WW6 and WW7 are of significant importance. Some consideration may be given to remote communications with the pumping plants associated with many of the main canal checks in the future, and a re-regulation reservoir planned for upstream of WW5. Early automation and remote monitoring systems were implemented with varying degrees of success, but a coordinated plan was not put together.

With the assistance of AS2I and the automation projects implemented at the key measurement and control points, an approach to implementing an open SCADA system has been mapped out. As such, the District has now standardized on Modbus communications protocol using two different types of wireless communications media. The implementation of Modbus allows the District to communicate with numerous brands of PLC's, including the Modicon and TeleSAFE units that are currently installed at various sites in the District.

Roza has also utilized various communications media including license-free spread spectrum radio, leased telephone line and cellular digital packet data (CDPD). The use of the communication multiplexer at the Roza office allowed all three of these media types to be implemented for a portion of one operating season, but eventually the leased telephone line to WW6 was abandoned in favor of CDPD once its viability was proven. This "hybrid" communications network allows the District to maintain a single host computer for the system, and the use of remote control PC to PC software provides remote access for operations personnel in all areas of the District.

SUMMARY

The partnerships that Roza has formed over the past decade for implementing new and innovative technology have assisted the District with the successful implementation of its CWCP, and have formed valuable relationships both inside and outside the District's sphere. The significant partners have worked with the District with the principal goal of water conservation in mind and have endeavored to provide support and solutions that are consistent with the philosophy of the District. With more projects planned for the future and the certainty that water will require more active stewardship, Roza has positioned itself so that it can call upon its conservation minded partners to help the District apply the right technology.

MODERNIZATION OF WATER MEASUREMENT SYSTEMS IN THE TURLOCK IRRIGATION DISTRICT

Brent Harrison, P.E, M. ASCE.[1]
Mike Kavarian[2]

ABSTRACT

The Turlock Irrigation District (TID), California's first irrigation district, was established in 1887. In 1997, the TID began to investigate what improvements could be made to its water measurement facilities, some of which dated to the early part of the 20th century. The plan that was developed consisted of the installation of telemetry to existing concrete weirs, construction of new long-throated flumes, and installation of solid-state devices to replace existing systems. The data collected in the field is transmitted via a spread-spectrum radio to the District's operation center where it is loaded into the Irrigation SCADA system. Office staff can access the data and use it to monitor and analyze the operation of the irrigation system. The experience gained in data monitoring and collection will be used as the foundation for further improvements in operation of the TID canal system.

BACKGROUND AND HISTORY

The Canal System

The Turlock Irrigation District was established as California's first irrigation district on June 6, 1887. After building the diversion and distribution facilities, the TID made its first delivery of irrigation water from the canal system on March 9, 1900. Today the TID irrigates 150,000 acres of land that consist of 7,500 parcels of property and approximately 5,000 individual irrigators. The District extends from the foothills of the Sierra Nevada on the east to the San Joaquin River on the west. The Tuolumne River forms the TID's northern boundary, while the Merced River forms the southern boundary. The TID canal system stretches from La Grange in the foothills of the Sierra Nevada mountains where water is diverted from the Tuolumne River, to Lateral 8 which ends 2 miles from the confluence of the Merced and San Joaquin Rivers. The canal system consists of 250 miles of concrete-lined and unlined canals, 380 check structures or drops, and 43 points where flow is measured in the system. Operational spills can be made in 14 locations, where discharges reach the Tuolumne, San Joaquin, or Merced Rivers. The maximum diversion from the river to TID's regulating reservoir, Turlock Lake, is 3,200 cfs. Irrigation releases from Turlock Lake to the

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distribution system range from 600 cfs in the spring and fall to 2,100 cfs during the summer.

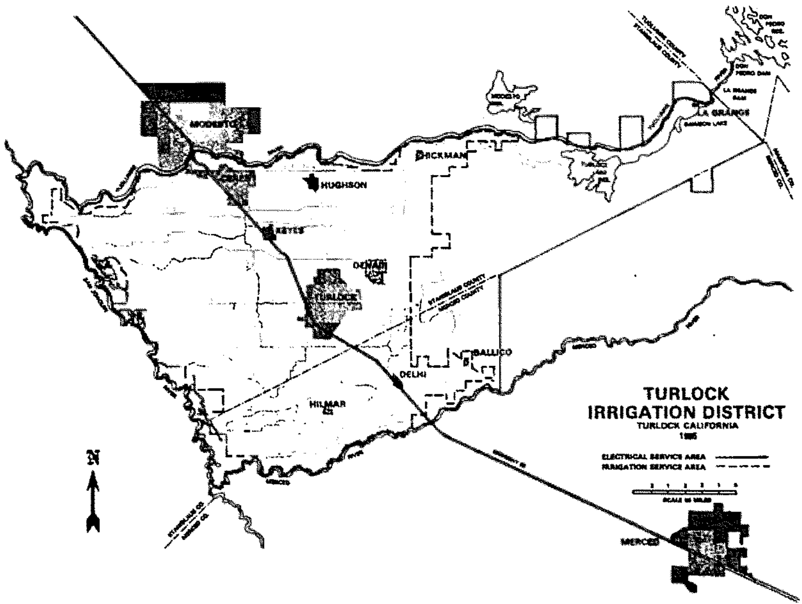

Figure 1 - Map of TID

Legacy Water Measurement System

To a great extent, the old water measurement system consisted of concrete Cippoletti weirs, some of which dated to the 1916 to 1923 period. Some of the older weir structures had wooden weir blades that required periodic replacement. The data recording system consisted mainly of Leopold Stevens chart recorders, which utilized a spring-operated mechanism that recorded float elevation as a pen line on a circular piece of paper. The recorders were placed in approximately 30 locations and recorded data for a one week period. One employee was assigned to change the paper charts in the recorders throughout the TID, a task that took 2-3 days each week to complete since many recorder stations required lubrication or other minor maintenance during the visit.

When the recorder sheets were returned to the office, the same employee was assigned to obtain flow data from the sheet and enter the data into the TID water tabulations. By the 1980's, the water tabulations had progressed to PC based spreadsheets thus making the data available in electronic format to other employees at the TID. Unfortunately, the time consuming nature of manually servicing the recorder stations, and other high priority assignments sometimes precluded the employee from timely reduction of the chart recorder data and entering it into the spreadsheets.

In the late 1980's, the District attempted to automate the reading of the flow measurement devices by installing twelve Telog instrumentation devices in the canal system. These devices used a solid-state probe connected to a local data recorder. Each site was visited on a weekly basis to transfer raw numbers to a portable data storage device, which was then connected to a PC for viewing and storage. Some of the problems with this system included not discovering problems with the data until it was downloaded and viewed on the PC, and the still time-consuming nature of the data collection and transfer work.

Other Problems with Water Measurement System

Along with the time consuming nature of collecting the data, the TID faced other problems with measuring the flow of water in the District. Many of the measuring structures were at the end of their useful life, having been built early in the 20th century. Concrete required patching and other maintenance procedures. Many of the measuring weirs were constructed too close to the headgates and were affected by high water velocity and turbulence. Some of the measuring weirs were too far away from the headgates which resulted in unacceptable time delays in setting up the canal for the needed flows. In other critical areas, there was no flow measurement available, and insufficient elevation drop was available to construct weirs.

In the mid 1990's, several outside forces also came to bear on the TID's methods of water measurement and record keeping. Government programs such as AB3616 increased outside scrutiny of the TID's operations and use of water. Irrigation customers expected higher levels of service than in the past. For example, they wanted more flexibility in delivery flow rate and time of delivery. In addition to these outside forces, TID managers questioned the resources assigned to obtain water measurements and other operating information for the canal system.

Turning Problems into Opportunities

TID staff came to the conclusion that major improvements would be required in the way water was measured and recorded. To determine the necessary improvements, TID staff discussed needs of the various stakeholders in the program. Customers were questioned to determine the service levels they required and the records needed to support their needs. TID canal system operators were asked to list their needs so that they might be able to operate the system more efficiently. TID staff visited other irrigation districts to learn what improvements had been successful. Staff also attended college and university workshops to determine the state of the technology available. Vendors were consulted to determine availability of devices and what budgets would be required to support the improvements needed.

TID staff developed a checklist of factors that would be used in evaluating new technology. The system must be user friendly, meaning it must be easy to learn and understand. The technology must be compatible with standard systems and protocols, and other applications already existing at the TID. In addition, the technology must have a proven track record and be operating in other similar applications. TID staff visited other irrigation districts to see first hand how well the various technologies functioned.

WATER MEASUREMENT IMPROVEMENT PLAN

The plan, finalized in 1997, consisted of a phased improvement program to the TID's water measurement activities. Improvements were scheduled over a 5-year period starting in the year 1998. Funding was requested as part of the capital budget for irrigation system improvements and with the TID Board of Directors approval, construction began in 1998. Forty-two sites were identified in the plan as sites that required water measurements to be transmitted to the central office, as shown in Figure 2. In order to accomplish this, improvements were needed in several areas such as structural improvements to water measuring devices in the canal system, installation of level transducers, installation of local electronics at each site to read the transducer, installation of a radio system to communicate with the remote devices, installation of base electronics to receive the data, and software to view and store the data. This comprehensive improvement plan became known as the Irrigation SCADA system and is scheduled for completion in 2002.

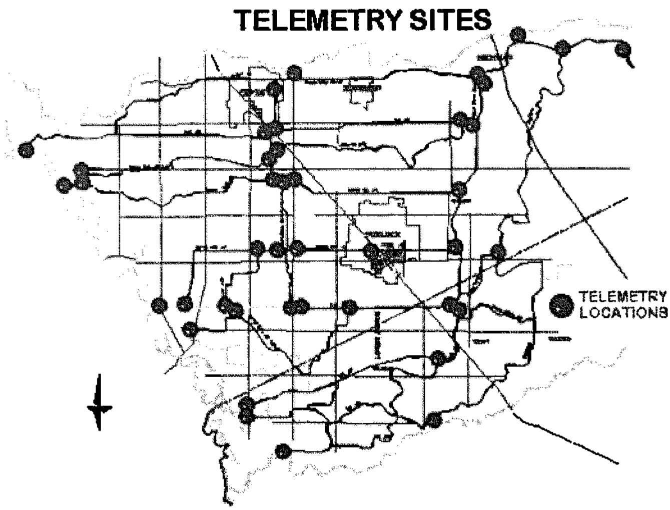

Figure 2 – TID Irrigation SCADA System

Structural Improvements

Structural improvements contained in the plan were:

- Rehabilitation of existing concrete measuring weirs. These structures included Cippoletti weirs that were installed strictly for flow measurement and Long Crested weirs that were installed for operational control and also used to measure flows.

Figure 3 - Cippoletti Weir

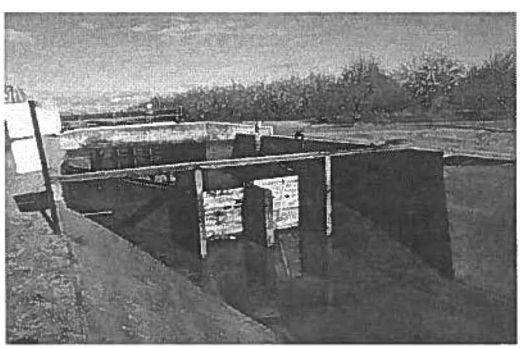

Figure 4 - Long Crested Weir

- Construction of five new long-throated flumes in locations where new measuring devices were needed and sufficient head existed for flumes to be constructed.

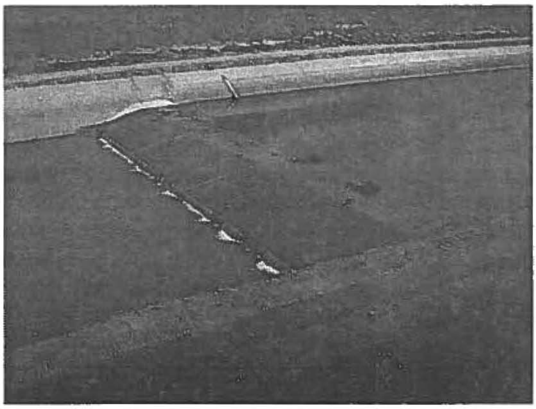

Figure 5 - Long Throated Flume

- Installation of six solid-state measuring devices where insufficient head was available for a weir or flume. The TID is currently installing measuring devices provided by Nivus. They consist of acoustic velocity sensors coupled with pressure transducers. One previous installation was provided by Accusonic, which utilizes acoustic velocity sensors coupled with an acoustic down-looking level sensor.

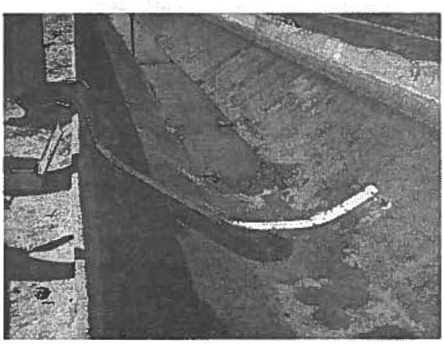

Figure 6 - Nivus Flow Measuring Device

Level Transducers

Several types of water level transducers are available commercially. Float-connected rotary transducers, ultrasonic down-looking devices and submersible pressure transducers were evaluated. The TID is currently using Druck PTX 1830 submersible pressure transducers because they have been found to be reliable and perform well in the TID's installations.

Local Electronics

The TID is using an RTU assembled by Sierra Controls. The RTU consists of a PLC, radio, antenna, operator interface/display, power supply and battery, lightning/surge protection, various terminal strips for input information, software and documentation. The RTU has the capability of 4 analog inputs, 3 digital inputs, and 2 digital outputs.

Radio System

The TID chose spread spectrum radios operating from 900 to 928 MHZ and are supplied by MDS. The TID has found the MDS radios reliable, with a favorable maintenance history.

Figure 7 - Remote SCADA site

Base Electronics

The signal from the spread spectrum radios is received at a central electric substation and transmitted over the TID's microwave backbone to the operation headquarters. From there the data is transmitted via a telephone system to the SCADA system PC.

Software

The TID utilizes Lookout software from National Instruments to provide the operator interface to the irrigation SCADA system. Operational personnel in the field can access the real-time data by calling the operations center. For example, one way the real-time data is used is to determine whether scheduled water has reached critical points in the distribution system. Operational and engineering staff can access the real-time data on their personal computers in their offices.

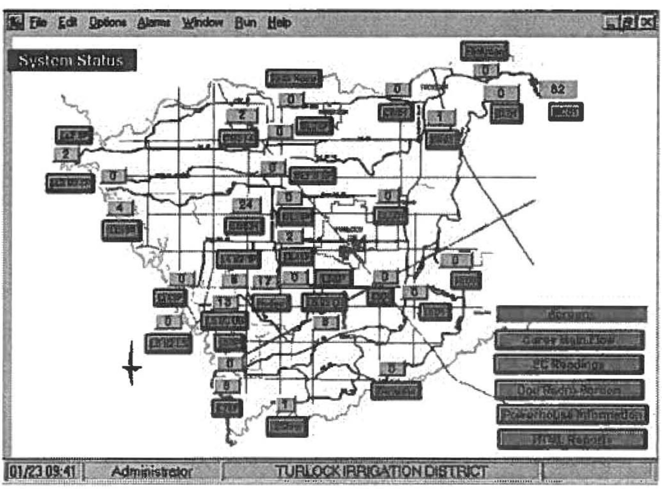

Figure 8 - Typical Operator Console for Irrigation SCADA

The software also provides the ability to download information as needed to a spreadsheet file, which is a key feature of the updated record keeping system. Flow data is generally downloaded on an hourly basis from the SCADA system to a spreadsheet. The hourly data is then manually averaged to provide daily flows for each site. Weekly and monthly reports are compiled in a similar fashion, with all data and computations in an electronic format. Reports are generated from the data and distributed to key personnel who use the information to make improvements in the management of irrigation water in the TID.

FUTURE PLANS

Completion of the Irrigation SCADA system in 2002 will fulfill the improvements envisioned for the Water Measurement System. The development of the Irrigation SCADA system provides a data collection and record keeping system that is timely and accurate. The system facilitates analysis and decision making to enable staff to efficiently operate the TID's irrigation system.

The TID is working on two improvements to the Irrigation SCADA system that will be implemented in the near future to improve communication with field operators. The software has the ability to send e-mail containing alarms and flow data to field personnel that can be received on their cellular phones. The other improvement is to convert HTML pages of flow data to web pages that can be read by web-enabled cellular telephones. It is anticipated that both these improvements will enable field personnel to improve the way they operate the TID irrigation system.

CORRECTING UNRELIABLE VELOCITY DISTRIBUTIONS IN SHORT CULVERTS AND CANAL REACHES

John Replogle[1]

ABSTRACT

Irrigation water management increasingly depends on good water flow measurement. Too frequently, flow disturbances from upstream elbows, the well pump, or other pipe fittings, produce distorted flow profiles that are detrimental to the proper installation and operation of common flow meters used in pipes, and the flumes and weirs used in canals associated with irrigated agriculture. Field conditions often force installation of pipe flow meters and flumes closer to these upstream disturbances than specified by standard installation recommendations. Special methods to condition flows to generate usable flow profiles over short distances are commercially offered for pipes meters Less expensive methods have been used in irrigation applications, but have been only partially studied to define their application limits. The historical installation recommendations for pipe meters and flumes are summarized and still recommended for inclusion in future constructions. For retrofit situations the field experiences and limited laboratory information on alternate approaches are presented. Suitable flow profiles for acceptable metering results in pipes can be obtained at distances about one-half to one-third the usual, historical, pipe-length requirements. These profiles, while nearly symmetrical, may be more uniform, or piston-like, than the fully developed shape.

INTRODUCTION

Water flow measurement is increasingly recognized as a major component for good irrigation water management. Too frequently existing field installations thwart retrofitting for proper meter operation because of the difficulty and expense of eliminating flow disturbances from upstream elbows or other pipe fittings, that produce distorted flow profiles that are detrimental to the proper operation of most common flow meters for pipes and canals in pipes, and also compromise the accuracy of flumes and weirs used in canals associated with irrigated agriculture. These existing field conditions and cost considerations often result in installation of pipe meters closer to upstream disturbances than specified by traditional installation recommendations. This also applies to canals where flumes are installed too closely to upstream disturbances. The traditional recommendations and more recent studies that contribute to the understanding and expectations of flow metering installations will be examined for possible

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application to situations that do not meet traditional recommendations. The objective is to identify the rate of deterioration in accuracy with anomalies in an installation. This is not always possible, but some general guidelines are proposed. Additionally, recent efforts to condition velocity profiles in short distances downstream from a disturbance in order to present an acceptable profile to a flow meter will be examined.

TRADITIONAL RECOMMENDATIONS

ASME and ISO Standards

The traditional recommendations for pipe flows are well summarized in ASME (1971). The highest accuracy assigned to a pipe meter depends on meeting these installation conditions. These recommendations specify piping arrangements that are meant to hold errors to within ±0.5%. Several recent studies contribute to understanding this accuracy problem (Miller, 1996; Hanson and Schwankl, 1998 ; and Johnson, et al., 2001). Miller (1996) states that several European and United States test programs have contributed data on specific kinds of upstream disturbances for application to installations of orifices, Venturi nozzles, and Venturis. These recommendations differ between the United States sources (ANSI/API 2530, 1995 and ASME MFC-3M, 1995) and those listed by ISO (ISO Standard 5167, 1991). The ISO standards are slightly more conservative.

The recommendations by ASME (1971) are basically for orifices, nozzles and Venturi tubes, and vary with the diameter ratio, $\beta = D_1/D_0$ (D_1= Diameter's of a meter throat; D_0= Diameter of pipe). An example of those recommendations in graphical form is shown in Figure 1, which is diagram "D" of eight such figures from ASME (1971). Again, this is for orifices and nozzles and does not address the deterioration of a result if these recommendations are not achieved. Also, none of the diagrams directly deal with the requirements of many modern meters that are used in irrigation, such as the ultrasonic flow meters, propeller meters, and vortex-shedding meters.

Other Considerations

Neither the ISO nor the other standards appear to address the question of deterioration of accuracy with deviation from the standard recommendations. Elsewhere, some limited discussion is offered as to the rate of accuracy deterioration with deviation from these standards. An attempt to quantify deterioration was made by the Kele Company (1998) based on published results in Benedict (1984) who in turn had provided estimates taken from studies in the International Journal of Heat and Fluid flow. These data indicate, for example, that eight pipe diameters (8*D*) of straight pipe upstream and 4*D* downstream are expected to add ±1% variance to a manufacture's stated accuracy for some situations, such as a single plane elbow, a tee, or a strainer. For most valves and two-plane elbows, 16*D* upstream and 4*D* downstream are recommended. These have been summarized and presented by the Kele Company (1998) in Table 2 for pipe Reynolds numbers in excess of 3000. Although some meters may be more sensitive to installation conditions than others, these are recommendations should present a suitable flow profile to any meter.

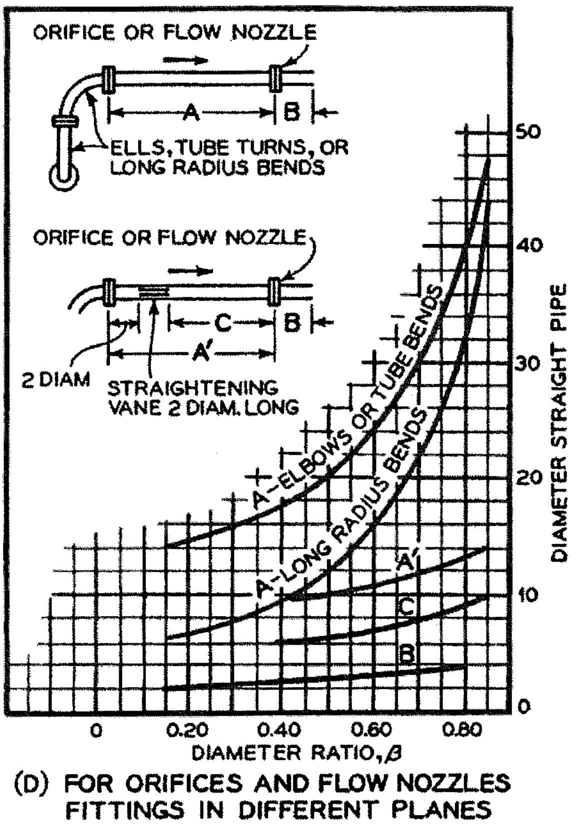

Figure 1. An example of recommendations of piping requirements for optimum meter performance as functions of Diameter Ratio, β. (From ASME, 1971)

For example, from Table 1, the grouping containing globe and gate valves would require16*D* of upstream straight pipe and 4*D* of downstream piping to maintain a variance of ±1%, with a deterioration to ±2% if only 8*D* are provided.

Translating these recommendations into irrigation practice, the consideration of heat exchangers, for example, can be likened to "Concentric Reducer or

Expander" and the corresponding values, ±1%, of in Table 1 can be used. Table 1 is not clear on how to apply the combinations of $8D$ and $2D$ instead of $8D$ and $4D$, etc., or if $4D$ downstream would really decrease the variance over $2D$. This approach of adding an increase in variance based on disturbance, however, may be somewhat oversimplified. It assumes that all meters respond similarly to a flow disturbance, which is not what Hansen and Schwankl (1998) reported.

Table 1. Additional variances for some common installation situations, not sorted for meter sensitivity. (After the Kele Company, 1998).

Upstream Obstruction	Straight Pipe Diameters Upstream	Straight Pipe Diameters Downstream	Resulting Variance
Elbow (single plane)	8	4	±1%
Tee, Strainer, Air	4	2	±2%
Separator	2	2	±5%
Globe Valve, Gate Valve	16	4	±1%
Two-Plane Elbows,	8	4	±2%
Bullhead Tee	4	2	±10%
Concentric Reducer	6	3	±1%
or	3	2	±2%
Expander	2	2	±4%

Hanson and Schwankl (1998) investigated the progressive increase in error of a variety of flow meters as they were placed at distances of $2D$, $5D$, $10D$ and $15D$ from sources of flow disturbance. The meters in this study were two propeller meters, a Doppler ultrasonic meter, a Hall pitot tube meter and a Collins gage pitot meter, which they classified as responding to average velocity. Additionally they included a velocity-gage flow meter, and two paddle-wheel meters, which they classified as point-velocity meters. The Hall pitot tube produces an average velocity by simultaneously sampling several points across the pipe and "hydraulically" averaging the resulting impact pressures. The Collins gage pitot meter is a point measuring device as part of a tube completely crossing the pipe through two holes on opposite sides of the pipe. The sensing part is moved from point to point across the pipe by sliding the tube through the two holes. This means that the tube crossing the pipe is always present and offers the same obstruction regardless of the sensing hole position. The combined result is used to obtain average velocity. Results showed that the propeller flow meters, the Collins meter and the Hall tube, using many points, were least affected by excess turbulence. The meters that depended on single point measurements were most vulnerable. One might think that the flow about the two sides of a butterfly valve may be symmetrical enough to cancel the effects of the valve plate and stem. However, a butterfly valve strongly affected meter performance when the meter was installed in close proximity. Apparently the centerline flow did not recover quickly. Their results reconfirm that meters should be installed upstream of most kinds of control valves. They noted that the installation of vanes downstream of a valve tended to transport distorted flow profiles further downstream, as has been noted by others (ASTM, 1971).

Johnson, et al. (2001) evaluated a transit-time flow meter with a variety of upstream disturbances and distances to the upstream disturbances. Their meter had a factory profile adjustment built into its internal software. A transit-time meter averages the velocity in a sampled slice across the pipe diameter and this slice must be translated into a "volume of revolution" to get the pipe's average velocity. For example a piston flow would have a factor of 1.0. An impractical conical profile would be 0.333, with the practical range between 0.75 and a value less than 1.0. Their results using the built-in velocity profile correction indicated discharge over-estimates averaging about 7%. This may mean that undeveloped velocity profiles after the flow disturbance were closer to the piston flow value of 1.0 than the manufacture's built-in correction value.

Their results showed that the transit time ultrasonic meter could be used with an inaccuracy of about ±5% for distances as close as 4.5*D* from disturbances caused by (a) single elbows, (b) two elbows not in the same plane, (c) a check valve, and (d) a 50% open butterfly valve.

Because the profiles were not fully developed, although consistent, at this distance, the internal correction for fully developed profiles was not applicable, and they present an adjustment ranging from about +6.5% for the profile at 4.5*D* and about +3.5% at 10*D*, reducing to no correction at about 30*D*. These are applicable to smooth pipes usually found in irrigation, such as PVC and aluminum. Standard iron pipe, which would be rougher, might be expected to produce yet different velocity-profile developments with distance, so these factors should be applied with caution in those cases.

Downstream distance requirements.

From Table 1 values presented by the Kele Company (1998), the effect of distance to a downstream obstruction seems to be important. These results were compared to the old data of Yarnell (1937). Figure 2 is one of several figures from that study and illustrates some of the information available therein. A pressure influence and a corresponding velocity disturbance are noted at the upstream plane of the 90-degree elbow shown, (Figure 2, upper row of diagrams, second from right end). At one-pipe diameter upstream (-0.5') no significant pressure or velocity influence is detectable. This was for Reynolds numbers ranging from 250,000 to 600,000.

For an obstruction such as a downstream gate valve, where the flow must bend significantly to reach the flow opening, the backpressure difference may extend further upstream, depending on the degree of valve opening. On the other hand, in this elbow installation, the entire flow is turned in a distance of one pipe diameter as it goes around the bend, so one may consider that this may be equivalent to one pipe diameter at the elbow beginning face. Thus, the one-diameter upstream is then equivalent to about two diameters upstream from the backpressure of the turning pipe's elbow.

Because of the splitting of the flow, one might expect a butterfly valve to cause less backpressure problems than a gate valve, and a ball valve opening smaller than the pipe itself could be considered similar to a "reducer" in that it passes flow near the pipe center. Using the old flow net process for potential flows in these situations, it would appear that elbows and butterfly valves would have less upstream requirement than a gate valve, depending on whether the gate valve was nearly open or nearly closed. Even gate valves seem harmless if they are about two diameters downstream, for the level of performance usually acceptable to irrigation.

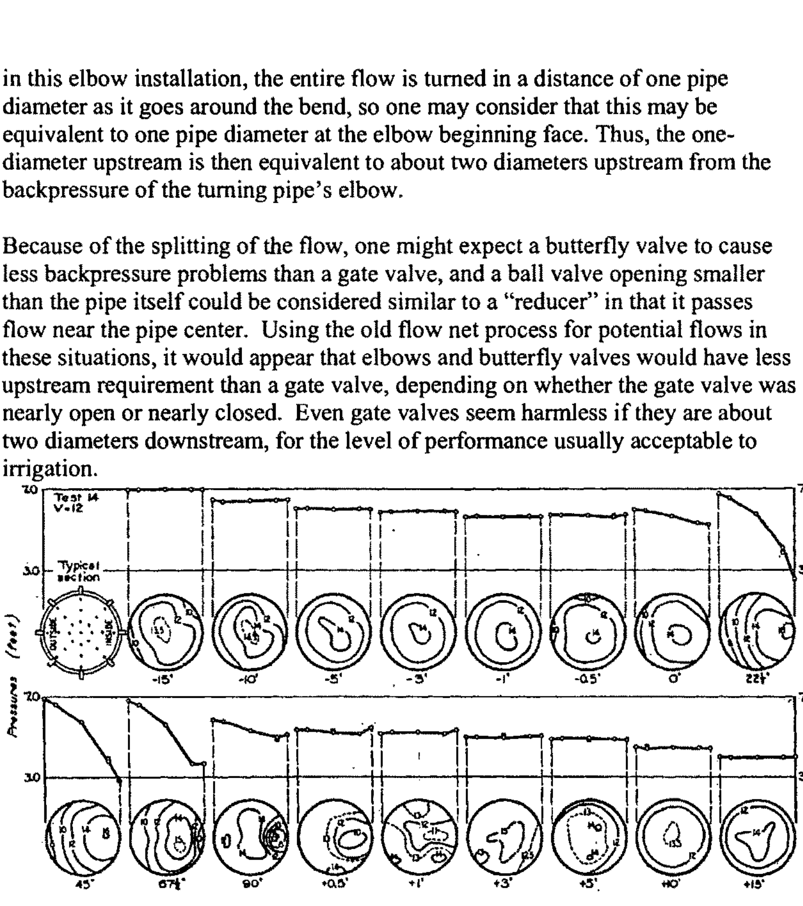

Figure 2: An example of velocity distribution and peripheral pressures in a 6-inch (15-cm) standard elbow bend with approximately uniform velocity distribution in the approach tangent. Mean velocity = 12 ft per second (3.66 m/s). (From Yarnell, 1937).

Can we support the upstream requirements stated in Table 1? Again using Yarnell (1937), Figure 2, the pressures and measured velocity distributions at 2 *D* downstream (+1' in Figure 2) from a 90-degree elbow still show some pressure differentials and asymmetrical velocity patterns. At 6*D* (+3' in Figure 2) the pressure differences around the pipe are nearly equalized, but the velocities are still slightly asymmetrical. Beyond 10*D*, some slight asymmetrical velocity patterns still persist but do not seem to improve more, even to 30*D*.

With the flow obstructed to force a jet to the inside of a 90-degree elbow (Yarnell, 1937), the disturbance persisted so that about 10*D* was required to approximately match the previous 6*D* behavior. With the jet on the elbow's outside, 20*D* was required with almost all of the effect gone by 30*D*. However, the jet at one side of the elbow bend, the disturbance was still significant at 30*D*. This supports the long pipe length recommendation associated with closely coupled elbows not in the same plane, called two-plane elbows in Table 1, and gate valves upstream of elbows with the gate valve opening to one side of the elbow bend plane.

Open-Channel Flow Metering

Flumes and weirs are the most common open channel devices used in irrigation. The recommended upstream requirements for flumes are given in Clemmens, et al., 2001, and are summarized below. They could apply as well to sharp-crested weirs.
(a) The Froude number should not exceed 0.5 at the gaging station and for a distance upstream of 30 times the maximum head reading, H_{1max}.
(b) The upstream channel should be straight and uniform for at least 30 H_{1max}.
(c) There should be no flow of highly turbulent water (e.g., undershot gates, drop strictures, hydraulic jumps) in the upstream channel for a distance from the gaging station of 30 H_{1max}.
(d) If there is a bend close to the structure (closer than 30 H_{1max}), the water surface elevations at the two sides are likely to be different. Reasonablely accurate measurements can be made with about 3% added error if the upstream straight channel is at least 6 H_{1max}. It is best to measure the water level on the inner bend of the channel.

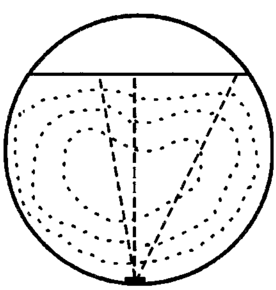

Figure 3. Doppler-based flow meter.

There is a relatively new acoustic Doppler-based flow meter, the ADFM Velocity Profiler™ (Acoustic Doppler Flow Meter, MGD Technologies, San Diego, CA). The ADFM uses, range-gated Doppler technology to measure velocity at many discrete points along several beams in the depth of flow in a channel or pipe cross section. These point velocities then are combined to determine a velocity profile and thus a flow rate for the channel or pipe, Figure 3. The developers claim that it tolerates distorted flow profiles well.

No attempt to specify conditions for other open-channel flow measuring methods such as current metering and open -channel, transit-time ultrasonic meters are given herein, although the above conditions for flumes should serve well. Sauer and Mayer (1992) have addressed part of this problem for current metering in canals and streams, and a literature review on accuracy of river measurements was prepared by Pelletier (1988).

CONDITIONING VELOCITY PROFILES

Various recommendations exist for conditioning velocity profiles to try to shorten the distance needed to improve meter performance. The idea being that if one could present a symmetrical, fully developed turbulent profile to a place that is to be used for a meter installation, then the meter would perform optimally. This means that one must provide either the suggested pipe lengths to achieve this desired profile or somehow artificially cause it. In many cases just providing a symmetrical profile whether or not it is properly shaped as a turbulent profile is a great improvement and will provide satisfactory results for applications in irrigation management.

Miller (1996) discussed in detail flow conditioners of many styles. Most of them are tube bundles or grid devices of complete cross vanes that are suited to clean flows. Many irrigation flows are pumped from canals and such straighteners are trash collectors. The eight cross vanes, two pipe diameters long will have a head loss of about one velocity head. More restrictive types can be as high as about 14 velocity heads (Miller, 1996).

Field experience

A transit-time ultrasonic meter was installed in an irrigation outlet that was 76 cm (30 in) in diameter and about 45 m (150 ft) long. A gate valve partly opened controlled flow into the pipe where it traveled about 10 diameters before turning 45-degrees left for a distance of about 40 pipe diameters. The result appeared to be a slowly rotating, distorted flow profile that the meter sampled for 16 seconds and recorded the result, which was on the order of 400 L/s (15 cfs.). This readout varied as much as 20% to 30% for consecutive readings,. The fluctuation was a random error that averaged to zero and did not affect the daily and weekly totals, but provided poor information for estimating the irrigation time for a particular field delivery. Using the reasoning, discussed later, we inserted a large opening orifice made of 2x2 inch (5x5 cm) structural metal angle forming the orifice of diameter ratio, β, of about 0.86. Because the ultrasonic meter was assumed to be fairly insensitive to rotation, unlike a propeller meter, no vanes were installed. The fluctuation in the readings was reduced to about ±3%. The head loss was negligible, calculated to be on the order of 1 cm.

A 3 m length of 15 cm diameter aluminum irrigation pipe was fitted with a propeller meter at about the 2.5-meter location. A routine check against a laboratory weigh tank system was planned. The meter location of about 16*D* from the upstream entrance was assumed more than adequate. However, a hasty connection to the water supply was made with some "lay-flat" tubing. This tubing caused enough swirl in the pipe, depending on the way it was twisted into position, that errors in the meter of as high as 30% were noted, both high and low. When vanes were installed, the meter read correctly. Even though vanes only can actually increase the action of a jet, the thought here was that a propeller meter is less sensitive to weak jets than to twisting flow, and we started with the least drastic "fix."

Highly agitated flow compromised the reading of a flume wall gage in a trapezoidal concrete channel with a 60-cm wide bottom and 1:1 sidewall slopes. The channel was flowing at about 400 L/s. A row of standard bricks was cemented down the sides and across the bottom of the canal to form an abrupt "bump" about 5 cm high. The flow surface was then damped by the construction of a small fixed bridge about 60 cm wide with its bottom inserted into the water surface by about 5 cm. This bridge was slightly downstream from the row of bricks by perhaps one flow depth of the canal. The process severely damped the downstream water surface and provided excellent reading ability at the flume gage (Clemmens, et al., 2001). Floating wave suppressors are not recommended.

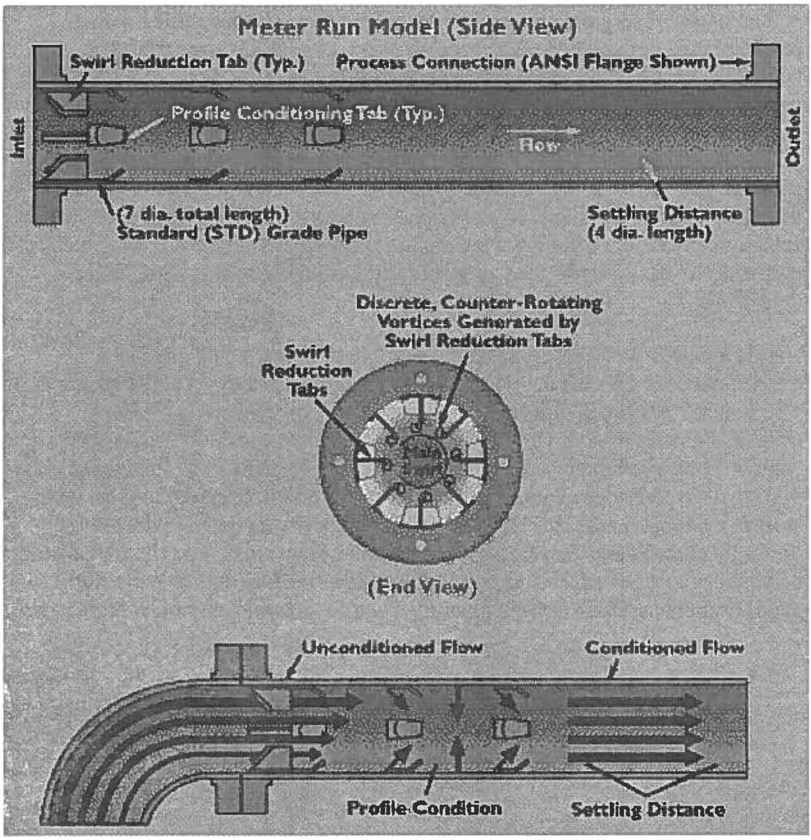

Figure 4. Vortab® flow-profile conditioner.

Commercial Flow conditioning Devices

A design objective could be to condition the flow using a length of one pipe diameter for the conditioning hardware followed by no more than two pipe diameters before reaching the metering device, for a total length of 3*D*. The objective is to present a symmetrical flow profile that may not be that of fully developed flow, but which many meters could use to produce satisfactory results for irrigation management.

A search for commercial offerings resulted in finding a device called the Vortab® flow profile conditioner, Figure 4. The head loss for most units is 0.5 to 1.0 pipe velocity heads. Swirl reduction tabs remove swirl by generating small vertices which tend to cancel the larger main swirl. Three sets of profile conditioning tabs produce cross-stream mixing, which mixes faster velocity regions with slower regions. This mixing produces a homogeneous flow profile. An additional 4-diameter length of pipe from the last profile condition tabs, allows the flow conditioning to fully occur. A flow meter would be installed somewhere at or beyond this distance. The total length is $7D$ of pipe, even for two-plane elbows, and this flow profile conditioner claims to present a fully turbulent flow profile at the$7D$ location. It may have a symmetrical profile at three diameters that can suffice for many applications if the "settling" distance is sacrificed.

Work in Progress

A study has been conducted by NEL (1999) to use numerical modeling (Computational Fluid Dynamics techniques, CFD) to determine the degree of flow disturbance at the inlet of a downstream meter. They claim to be able to apply the techniques to study flow meters themselves with disturbed inlet conditions to estimate the degree of measurement error. At the time of their report, studies of orifice plates and Venturis are finished. They intend to look at ultrasonic and electromagnetic meters. These results should offer insight to the accuracy deterioration when piping requirements are compromised.

Is there an alternate design that may be more simple to manufacture than the Vortab® and be applicable to irrigation technologies? Studies like the field experiences described above are underway at the US Water Conservation laboratory. One possibility is to use vanes extending from the pipe sidewalls to a distance of 0.5 times the radius ($0.5R=D/4$). This involves the outermost 75% of the flow area. The length of these vanes should be a function of the distance between them. For example, using eight vanes, the circumference is divided into $\pi D/8$ parts, and a guess that at least twice the flow length-to-width ratio may be sufficient to satisfactorily straighten the flow, the vanes would need to be $2(\pi D/8)$ long. In general terms with n vanes, this becomes $2(\pi D/n)$ long. This does not speak to a probable need for a 3:1 sloping upstream edge to shed trash. The length of this sloping part would be $3R/2$, or $3D/4$. If half the length of the sloping vanes is considered effective, then the total length will increase by half the vane length, or $2(\pi D/\mathrm{n})+3D/8$. For example, with eight vanes, the total vane length would compute to be about $1.06D$, with $0.75D$ of the length involved in the sloping part. The added head loss is expected to be less than the one velocity head for vanes completely across the pipe (Miller, 1996).

An orifice with a β ratio of 0.9 is to be added to the end of this structure. The resulting velocity fields at $1D$, $2D$, $3D$ and beyond for various disturbances are being studied. The pressure loss for an orifice to pipe ratio of 90%, ($\beta = 0.9$) is approximately 22% of the maximum differential pressure (ASME, 1959; Daugherty and Ingersol, 1954). Using this value for $\beta = 0.9$, this translates into a head loss of approximately 0.25 velocity head. However, while an orifice alone may not stop flow spin but may actually increase it, an orifice is important to block flow close to the wall (Miller, 1996). Thus, vanes placed upstream before the orifice to reduce spin would seem appropriate.

SUMMARY AND CONCLUSIONS

Several traditional recommendations for piping installation requirements for flow meters are reexamined for applications to new metering systems and to determine the expected accuracy when the traditional recommendations are not met, as is often encountered when trying to retrofit economical flow metering into exiting irrigation systems. These traditional recommendations are still valid, but are basically for older types of differential meters. They do not address the rate of accuracy deterioration as a function of deviation from the traditional piping recommendations. Similar types of recommendations for open channel devices, such as weirs and flumes, are still valid, but again do not fully address the rate of accuracy deterioration with installation anomalies. Newer types of meters for both open flow and pipe flow show variable tolerance to disturbed flow. These have not been fully studied. A limited number of recent studies do address both the accuracy deterioration and the meter tolerance issues. As might be expected, meters that sample large portions of the flow profile are more tolerant of distorted profiles than those that sample a single point and must depend on extrapolation to the entire pipe flow. For open channels, the production of smooth approach flow to weirs and flumes seems a sufficient measure to give adequate profile presentation to the metering station. Recent ongoing studies in numerical modeling are attempting to determine the rate of accuracy degradation for particular pipe installation anomalies. Commercial devices claiming to correct and present developed flow profiles within seven pipe diameters when placed downstream from flow disturbing fittings and valves are available. The design and construction of inexpensive pipe inserts are currently being tested and are meant to condition flows for most of the meters used in irrigation. These profiles, while nearly symmetrical, may be more uniform, or piston-like, than the usual, fully developed shape. These methods and their selection include effectiveness of flow conditioning, consideration of trash handling, and cost of installing the necessary equipment into the water supply system.

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SCADA EXPERIENCE IN MEXICO

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ABSTRACT

Since 1989, with the transfer of operation and maintenance of the irrigation districts to Water User Associations in Mexico, a big training and technological program began. Two large irrigation districts started their remote operation projects with unfavorable results. Poor equipment selection, based on extremely limited experience, and the absence of a supervisory system to verify the performance of the components were the root causes. The Mexican Institute of Water Technology (IMTA) is making headway in the use of SCADA systems for canal operation in Mexico. Studies include testing of the control algorithms required, development of the supervisory schemes, evaluation and adaptation of control elements available on the market and training of user association personnel. Two laboratory canals were used for these purposes. Since 1997, the Institute, supported by the National Water Commission (CNA), the Mexican federal agency responsible of water reclamation, worked with the farmers' association in the Carrizo Irrigation District to develop a remote flow monitoring system and remote control of four control structures

INTRODUCTION

In 1989, The Mexican government began the transfer of operation and maintenance of irrigation districts. This transfer was associated with a training, financial and technology program. In Mexico, water is a limiting factor for development. The marked competition between the municipalities, industry and agriculture is pushing toward more efficient use of this limited natural resource. Agriculture, the largest water consumer, requires new technologies to improve water and soil conservation. SCADA systems are one of these technological alternatives under analysis in Mexico.

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With the transfer process, a modernization program was started with limited results. The principal problem was that system operation (Plusquellec et al., 1994), the main aspect of modern project development, was not considered during design. New infrastructure and remote monitoring control systems were installed without considering well-defined and realistic operational plans based on the service concept. These problems and the lack of qualified staff doomed the modernization project to failure. These considerations and the limited experience with modern flow and level control alternatives caused two SCADA projects to fail.

PREVIOUS SCADA EXPERIENCES

In 1989, the National Water Commission (Comisión Nacional del Agua, CNA, the Mexican federal agency responsible of water reclamation) created two automation projects in the central planning area, "El Canal Alto" in the Yaqui River Irrigation District and "Bachimba" in the Delicias Irrigation District.

Río Yaqui Project

In 1994, IMTA carried out an evaluation of equipment and operating conditions in the Yaqui Project to determine why a recently built remote monitoring system had never been used and had been partially dismantled. The evaluation showed that an operation analysis had not been carried out, equipment was removed in two control structures, electronic components were faulty, and the communications components was inadequate for heavy-duty telemetry systems. These problems made the system very dangerous to use as it was. In the following years, the Users Association and CNA did not have the resources or desire to continue testing remote monitoring systems.

Bachimba Project

Bachimba project had a similar fate. The CNA's central office developed a project without the participation of personnel in charge of canal operations. When the project was finished, no one at the irrigation district knew how to use the new system or the purpose of the project. The proposed system involved the use of an unreliable communication system and a Remote Terminal Units (RTU) incapable of operating the control structure if communications with the master station were broken, a frequent occurrence with the communication equipment selected.

The reliability and performance of the equipment and the project architecture required for canal operation were underestimated. The lack of experience and interest in remote monitoring resulted in poor technical specifications in the request for proposals. Consequently, the projects were unreliable and useless. The cheapest solutions were taken while ignoring the contractors' lack of experience in canal operations.

IMTA RESEARCH PROJECTS ON REMOTE MONITORING AND CONTROL

IMTA has been involved in three aspects of remote monitoring and control of canal operation: 1) development of control and supervisory algorithms, 2) evaluation of control equipment and software and; 3) training of water users association personnel. The first component of this research was conducted with the simulation model SIC from the CEMAGREF, France. In 1995 and 1996, with the collaboration of the ITRC CalPoly, three slide gates were installed on a zero slope laboratory canal. Last year, with the assistance of the USBR Provo Office, a slide gate was installed on a trapezoidal canal.

Research Project On The Zero Slope Canal

Canal description. Three slide gates were installed (Fig. 1) in a zero slope canal, 60 cm wide, 40 m long and 1 m high at IMTA laboratory facilities. The canal inflow is regulated with a servo valve controlled by the RTU located at the slide gate. At the downstream end of the canal, the level is regulated by an overshot gate. Each slide gate is equipped with a linear actuator, two pressure sensors (upstream and downstream water level), a potentiometer for gate position and limit switches (maximum and minimum gate opening). The system was designed to allow manual operation and RTU operation. The RTU can be operated in the manual or local automatic mode. The linear actuator has a 12 Volt DC motor, controlled by an operation box that allows manual or RTU operation. Inside the box, a set of electromagnetic relays is controlled by the digital outputs of the RTU that specifies the direction of the motor. Two limit switches cut the power supply to the motor when the gate is at the upper and lower positions. The pressure sensors and gate position potentiometer are relayed to the analog inputs of the RTU. The RTUs used are the MODICOM PLC E984-245 and the SCADAPack from Control Microsystems. At the Pentium PC master station, the man-machine interface (MMI) was designed using the Lookout as SCADA (Supervisory Control and Data Acquisition) software. The master station and the PLC communicate using a cable communication called MODBUS+ or radio.

Control algorithms. With the control equipment installed, some control algorithms were evaluated. The initial tests were with a Proportional-Integral (PI) regulator in an upstream control application. The PI regulator performance changed with the head across the structure. The control parameters (KP and KI) had to be readjusted to maintain the desired closed loop performance. Burt et al. (1998) proposed a correction factor to account for the non-linearity of the upstream water level response to gate movement. The new algorithm is the PI with the universal factor (UF). This control algorithm is a gain scheduling PI regulator (Aström and Wittenmark, 1990).

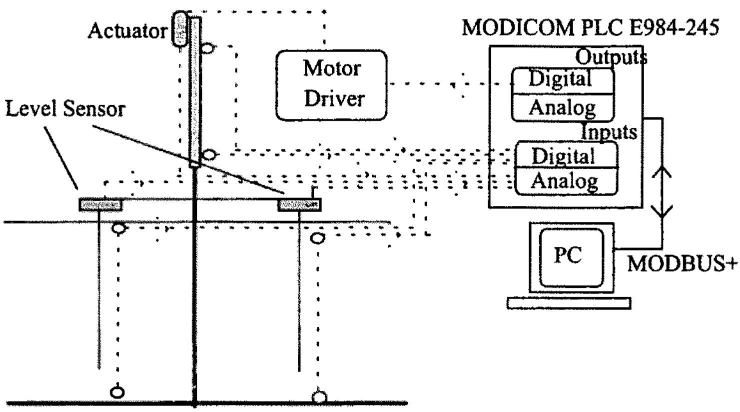


Figure1. Slide gate with control and supervisory components.

The performance of a PI control algorithm in the regulation of the water level upstream of the gates at the laboratory canal is presented in Fig. 2. The flow at the head of the canal was changed 25% in each case. The sampling time was 10 seconds, the control parameters were KP = 0.8, KI = 0.24 (gate one), KP = 0.4, KI = .08 (gate two), and KP = 0.43, KI = 0.04 (gate three). The references (y_{ref}) for gate 1, gate 2 and gate 3 were 70 cm, 55 cm, 40 cm, respectively. The ability of the PI - UF to maintain the performance of the control algorithm under different operating conditions can be clearly observed in Fig. 3.

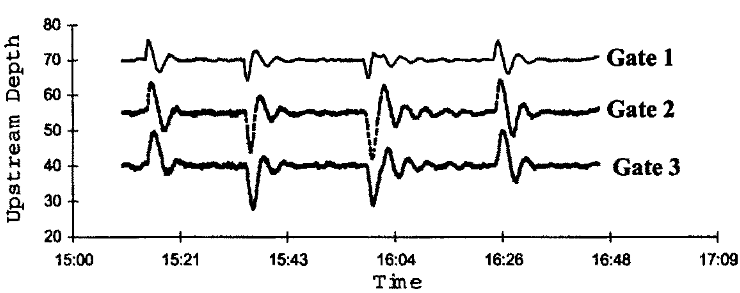
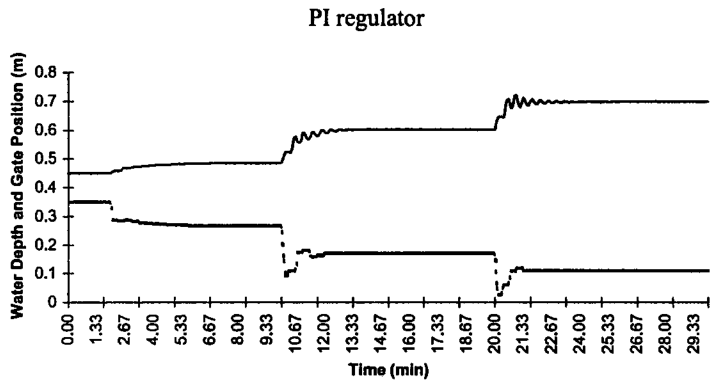


Figure 2. Evolution of the water depth upstream of the laboratory canal slide gates with a PI regulator when the flow at the head of the canal change 25 % (20 l/s)

Last year, a multivariable optimal regulator (Linear Quadratic Gaussian, LQG), was tested in simulation and in on the laboratory canal, as an alternative to canal control. The LQG regulator was introduced in the SCADA systems, using the Windows DDE options, the SCADA software Lookout was connected with Matlab were the control algorithms were developed. Fig. 4 shows the downstream level responses (outputs), when an inflow perturbation acts on the canal with the LQG regulator. The Fig. 4 presents the opening evolution of the control gates (inputs), and inflow variation (perturbation) (Begovich et al, 2002).



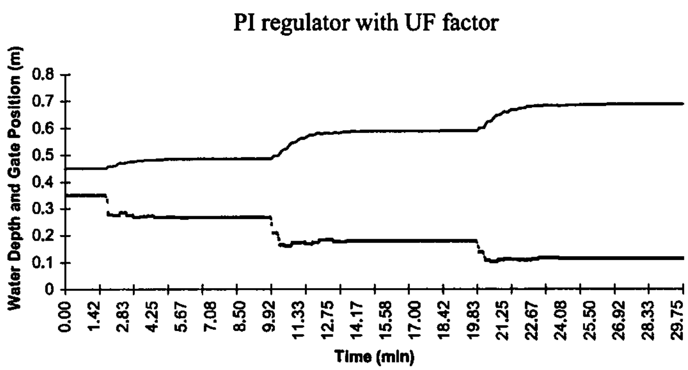


Figure 3. Evolution of the water level (continuous line) tracking a reference profile with different PI regulators. The gate position is presented in the dotted line.

Research Project On The Trapezoidal Canal.

Canal Description: A slide gate was installed, with water level sensors, gate position sensor and gate actuator, on a trapezoidal canal 25 m long, 0.0005 bottom slope, 1:1 slide slope,. The low cost sensors and actuator were provided by the group of Roger Hansen from USBR, Provo, USA. The canal also has scaled-down models of real regulators (AMIL gate, overshot gate) and turnouts (metering gates and distributor). The inflow to the canal is regulated by a manual valve. An X2 distributor (120 l/s max.) is located upstream from the slide gate. As in the zero slope canal, the operation system was divided on two boxes. The operation box drives the actuator. All the electronic equipment (RTU, radio. power supply, etc.) is located in the control box. The operation box was designed based on the example the USBR provided us with components found in Mexico.

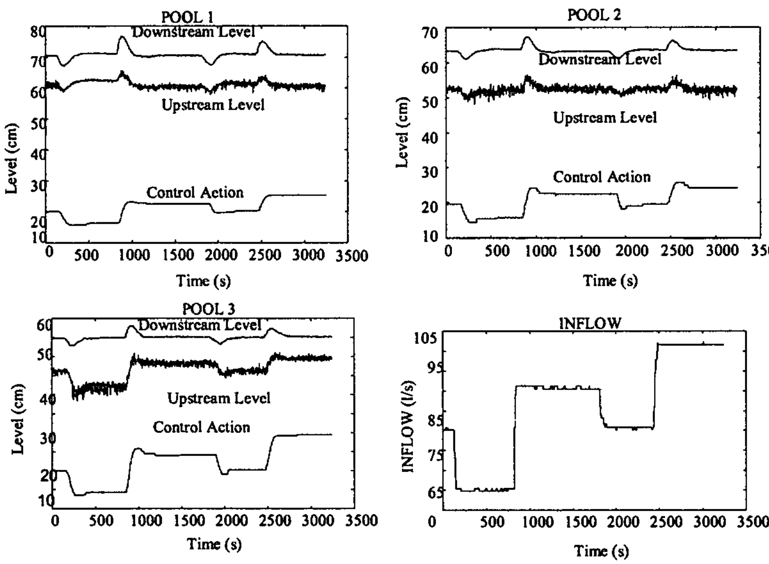


Figure 4. Responses of both downstream levels and positions of control gates when an inflow perturbation acts (LQG regulator performance).

Regulator test. As a first step, the PI block of the Telepace ladder logic was used for upstream control. The level was regulated at 35 cm from the bottom of the canal, the nominal level of the X2 distributor (Nerpyc Alsthom). Downstream of the distributor the discharge was free. The evolution of the level upstream the gate and gate position when the flow through the distributor changes is shown in Fig. 5.

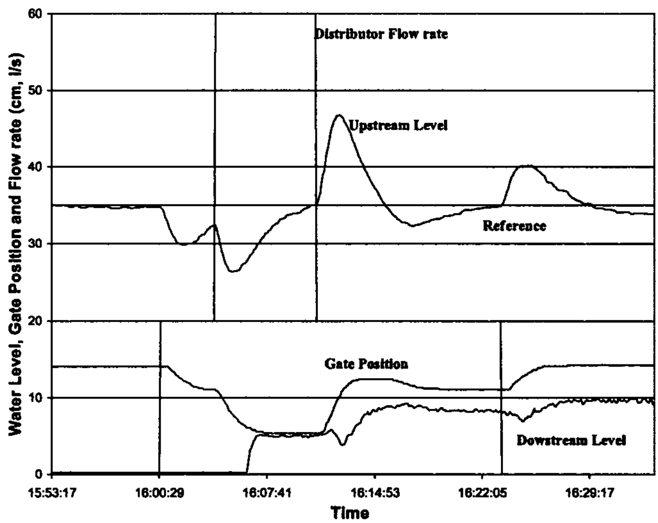


Figure 5. Performance of the PI regulator on the upstream water level in the laboratory canal

IMTA FIELD EXPERIENCE

Remote Flow Monitoring. The farmers' association in the Carrizo Irrigation district, under a drought assistance program financed by the Ministry of Agriculture (SAGAR), asked IMTA to help them develop a remote flow monitoring system (Fig. 6). Long-throated flumes were built at the head of the main canals and secondary canals, and at the beginning of the irrigation sections controlled by different farmers' associations. The water depth in the flume is measured at a stilling well using an ultrasonic sensor. From the water depth, a 2100 Badger Meter determines the flow and totals the volume passing through the flume using the flume calibration equation. The measured information is sent to the master station using a radio-modem transceiver (MDS 4310). The communication protocol used is MODBUS. At the master station, a man-machine interface (MMI, Lookout) program presents the flow information and the accumulated volume. The MMI displays on a district map the sites and the measured flow. There is a screen display for each flume that presents all the flow and volume information required by the water master.

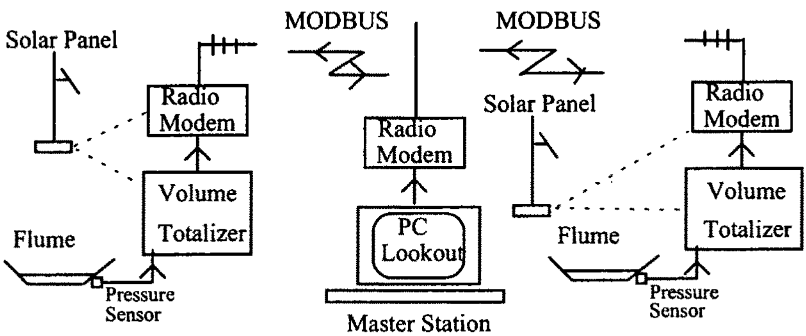


Figure 6. Flow measurement at the Carrizo Irrigation District.

Problems encountered during this project were all solved. The main problem for 3 years was the quality of the field equipment. This original field equipment (MGV from Hidronica) was replaced by 2100 Badger Meter.

Remote Operation. In November of 1999, IMTA began to work on the remote operation of control structures at the Carrizo District. The project is financially supported by CAN. Given the Carrizo District's experience with remote flow monitoring, IMTA considered that this could be a good place to test control equipment. Today, four control structures have remote monitoring and control. Two of the remote flow monitoring sites are based on the equipment tested on the trapezoidal canal. The two other sites are based on technology developed at the National Autonomous University of Mexico. Figure 7 shows the upstream level and gate movements requested since it started operation at the control structures located at the head of the North Canal.

CONCLUSIONS

SCADA equipment is an alternative for canal operation. Special care must be taken in the development of the project. Equipment maintenance, training programs, user capacity and social conditions must be carefully considered. Pilot projects, involving important remote monitoring but not crucial canal operation, can provide an estimate of the capacity of those responsible for building and operating the system.

After the experiences on canal operation and the possibilities Lookout gives to supervise operation through Internet and Intranet, CNA is very interested in the development of SCADA projects to supervise Water Users Asociation operation. This year some districts have requested IMTA to help them develop their remote monitoring system as part of the federal program to improve flow measurement at the control point where the water is provided to the water user associations. Until today the remote control projects have been supported 100% by CNA.

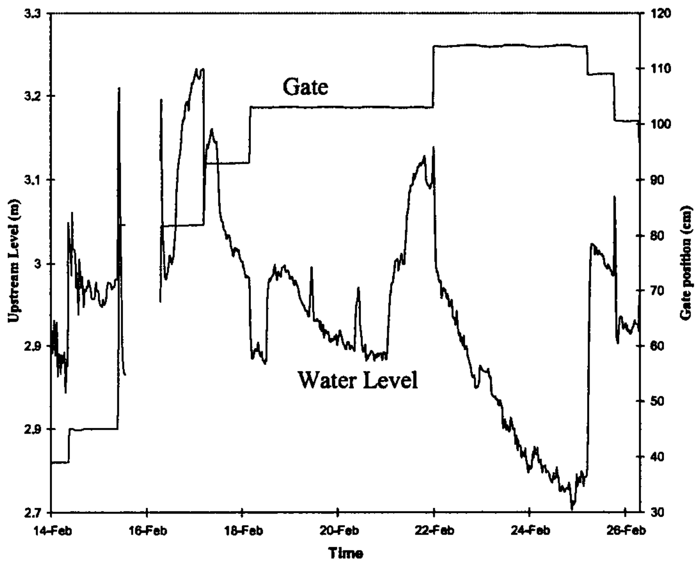


Figure 7. Evolution of the water level upstream and gate position at the head of the north main canal in the Carrizo Irrigation District.

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CASE STUDY: INSTALLATION OF CANAL CONTROL STRUCTURES ON THE GOVERNMENT HIGHLINE CANAL

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ABSTRACT

The Government Highline Canal Modernization Study was completed in April 2000 to evaluate options for reducing the flow rate requirement of the Government Highline Canal in order to increase the water supply in the "15-Mile Reach" of the Colorado River, thereby helping sustain habitat for fish species identified as endangered. The intent was to develop a design for which Colorado River diversions could be better matched to on-farm demands. It was determined that canal modernization, in the form of automated canal structures, in-system storage, and new operational procedures could significantly reduce operational spill. A systematic evaluation of the existing canal system and operations identified a potential savings of 28,500 acre-feet of water that could remain in the river or be returned to the river upstream of the 15-Mile Reach.

This paper is an update to one done in 1999 for USCID about the Modernization Study. This paper will focus on the lessons learned from the actual implementation of the recommended structural and operational improvements, in particular the construction of the main components of this project including canal control structures.

INTRODUCTION

The Government Highline Canal is part of the federal Grand Valley Project in the Colorado River Basin in west-central Colorado, USA. The project is located at the confluence of the Colorado and Gunnison Rivers at Grand Junction, Colorado (refer to Figure 1). The elevation of Grand Junction is about 4600 feet (above sea level). The United States Bureau of Reclamation (USBR) built the Government Highline Canal in the 1910s in cooperation with the Grand Valley Water Users

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Association (GVWUA). The Government Highline Canal receives water from the Grand Valley Project diversion dam. The canal travels in a mostly westerly direction for 55 miles (88.5 km) to the Badger Wash Check. The diversion dam is located about 8 miles (12.9 km) northeast of Palisade. The main canal has a diversion capacity of about 1,675 cfs (47.4 cms), about half of which goes to the Orchard Mesa Power Canal (OMPC) and half continues to GVWUA and other water users. The irrigated service area of the GVWUA is about 23,300 acres (9,400 ha). Project facilities are owned by the USBR; the facilities are maintained and operated by the GVWUA.

In 1987 the United States Fish and Wildlife Service (USFWS) developed the Recovery Implementation Program for the Endangered Fish Species in the Upper Colorado River Basin (Recovery Program), which identified a 15-Mile Reach of the Colorado River near Grand Junction, Colorado as an area needing additional water supplies during the late summer and fall months (August through November) to maintain habitat conditions for several identified endangered fish species (USBR 1998). The Grand Valley Water Management Project was established to evaluate potential improvements that could be incorporated into the USBR's Grand Valley Project in order to provide additional flows in the 15-Mile Reach.

As a continuation of the Grand Valley Water Management Project a team of five agencies completed the Government Highline Canal Modernization Study (Modernization Study). The project team included the United States Bureau of Reclamation (USBR) Western Colorado Area Office in Grand Junction, Colorado; the Irrigation Training and Research Center (ITRC), California Polytechnic State University in San Luis Obispo, California; the Grand Valley Water Users Association (GVWUA) in Grand Junction, Colorado; the USBR Technical Services Center (TSC) in Denver, Colorado; and the University of Iowa's Hydraulic Research Laboratory (IHRL) in Iowa City, Iowa.

A systematic evaluation of the existing Government Highline Canal, including current management practices and operation of project facilities, identified a potential savings of 28,500 acre-feet of water that could remain in the Colorado River or be returned to the river upstream of the 15-Mile Reach. An important aspect of the Modernization Study was the desire to maintain the current high level of water delivery service provided to the GVWUA customers, yet have less spill. The study made specific, pragmatic recommendations for reducing operational spill. The detailed analysis conducted as part of the Modernization Study is summarized in Styles et al. 1999. Funding for the initial modernization study was provided by the USBR. After the potential to reduce diversions by 28,500 acre-feet was identified, the Recovery Program agreed to fund the remainder of the study and the proposed improvements. The cost for the studies and improvements is approximately $7,000,000.

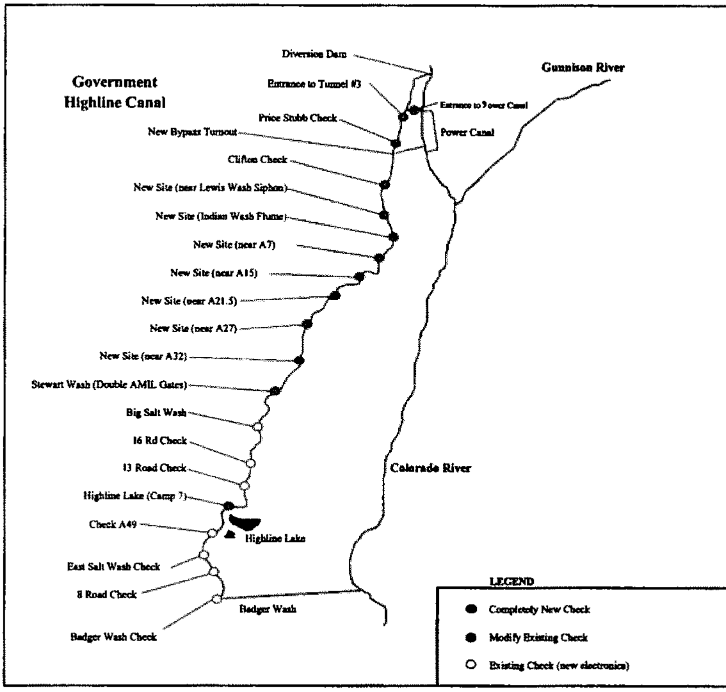

Figure 1. Government Highline Canal Check Locations

MODERNIZATION PROJECT: CANAL CONTROL STRUCTURES

The goal of the Government Highline Canal modernization project was to maintain the same level of flexibility and reliability of service to the GVWUA customers, yet have less spill. The proposed improvements would allow for more water to flow into the "15-Mile Reach" of the Colorado River. Implementation of the project is expected to cause a 28,500 acre-feet reduction in the operational spill requirements of the GVWUA during the months of August through October. The manager will have better access to information and will be able to make more frequent flow rate changes at the canal headworks. In addition, the manager will have new capabilities for flow rate control at Camp 7 and at the new Palisade Pipeline Turnout (described below).

The main features of the modernization project are:

- Seven new check structures (operational in 2001)

- New pump station at Highline Lake and the use of Highline Lake as a buffer reservoir (planned in 2003)
- New Palisade Pipeline turnout to the Colorado River upstream of the 15-Mile Reach (operational in 2001)
- Implementation of SCADA - Supervisory Control and Data Acquisition (planned for 2002)
- Twenty-one new Remote Terminal Unit (RTU) sites connected by radio
- New central office control facilities
- Modification to thirteen existing check structures
- Modification to the OMPC turnout

The following sections describe the structural details and operational characteristics of the new and modified canal control structures. The discussion begins at the canal headworks and proceed in a downstream direction.

Government Highline Canal Headworks

The Government Highline Canal headworks structure consists of nine sluice gates. A single motor, located in an adjacent structure, powers a shaft, which can drive all of the gates simultaneously, or as many as are desired. A single stage "Littleman" controller is used to control the flow rate into the canal. The water level in the canal at the Littleman float location approximately 500 ft downstream of the headworks is approximately calibrated to flow rate.

The improvements to the headworks will provide the GVWUA with the capability to monitor and make changes to the canal flow rate remotely from a central office. This will be accomplished with the installation of a new, reliable controller, a water level sensor to replace the float and stilling well for the Littleman, a new RTU to control the gate motor, a gate position sensor, and the incorporation of radio communications. The office computer will display the gate position and the flow rate and operators will have the ability to change set-point from the office or on site.

The modified operation of this site will be similar to the current procedures. Only one gate will be adjusted to fine tune control; the other eight gates are only for gross flow rate control. The flow rate will be measured using an MGD acoustic Doppler flow meter.

Inlet to Tunnel #3

At this site the canal splits between the Government Highline Canal and the Orchard Mesa Power Canal (OMPC). Diverted water flows through a radial gate, into an inverted siphon, across the Colorado River, and into the OMPC. A radial gate also controls the entrance to the Government Highline Canal. Previously the operation of the two gates was somewhat complex and the canal operators needed a day of adjustments to make a single flow rate change. The radial gate was manually adjusted with a wheel crank to maintain the desired water level in the OMPC.

A new RTU will be added at the Inlet to Tunnel #3 to control the two radial gates. The radial gate controlling the entrance to the Government Highline Canal will be automated with local closed-loop control to automatically maintain the upstream water level using the same RTU as the OMPC gate. The radial gate controlling the entrance to the OMPC siphon will be adjusted by operators at the district office (remote manual control) based on the flow rate into the OMPC. The flow rate will be determined from the water level in the OMPC downstream of the siphon under the Colorado River using a second RTU that communicates via 470 MHz spread spectrum radio to the controlling RTU.

Both gates will be equipped with new electrical gate actuators. The radial gate at the entrance to Tunnel 3 will be moved much more frequently. Therefore, the complete hoist mechanism will be replaced, and the gate hinges will be inspected thoroughly and repaired or replaced as necessary. A requirement for each of the check gates was the that the control of the gate must allow for the gate to be operated in a fully open mode such that the gate is out of the water.

Because of the critical nature of this control point redundant water level sensors were installed in the pool at the inlet to Tunnel #3 and in the OMPC across the river from the Tunnel 3 entrance.

Price-Stubb Check

The Price-Stubb check structure is located at the exit of Tunnel 3. The check structure consists of three stoplog bays to maintain the upstream water level. Current plans are to remove the stop log bays and install a new automated Langeman gate in the check structure. The gate will be locally controlled to maintain a desired upstream water surface elevation. The SCADA system will be set to provide an automatic alarm to the gate operator if the single automated gate position is either almost completely closed or completely open.

Palisade Pipeline Turnout (new facility)

Palisade Pipeline is a key structure of the modernization program - its simplicity and location will reduce the lag time normally associated with flow rate changes from several days to perhaps 12 hours. It was installed just below the Price-Stubb Check and permits simple fine-tuning of canal flow rate yet return any excess diversions to the river upstream of the 15-Mile Reach. The turnout to the Palisade Pipeline is a sluice gate under remote manual control based on the flow rate in the pipeline as shown in Figure 2. The flow rate is computed based on observations of the required and actual flow rates at the Indian Wash Flume, Camp 7 spill, Highline Lake storage, and Badger Wash spill.

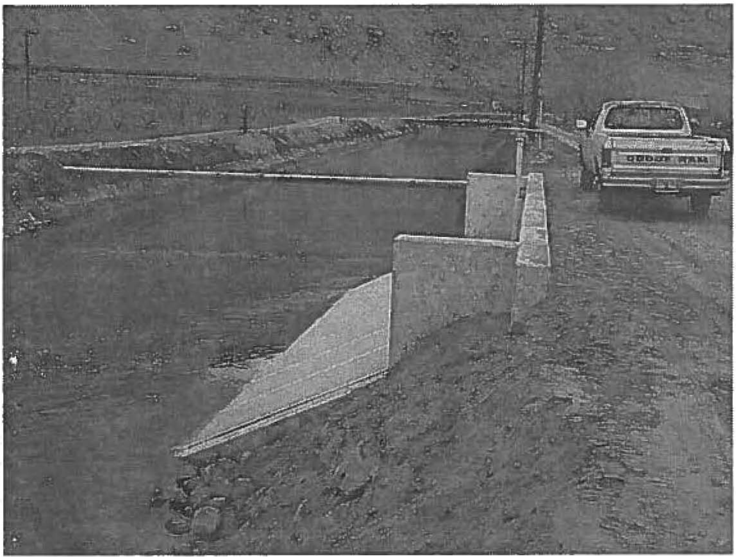

Figure 2. Palisade Pipeline Entrance

The pipeline was sized for a maximum flow rate of 120 cfs. Typical operation will have a 50 cfs by-pass flow rate. This will allow for immediate availability of an additional 50 cfs for downstream canal demands, and also provide capacity for removing 70 cfs from the canal at any time (50 cfs regular + 70 cfs extra = 120 cfs capacity).

Clifton Check (existing facility)

The Clifton check structure was modified with the addition of electric sluice gates in the six stoplog bays. The AMIL-450 automatic hydraulic gate in the center of the structure was retained. The first stoplog bays on each side of the gate will remain as stoplogs, since the catwalk structure could not accommodate a change to another type of gate. The layout of the modified structure is shown in Figure 3.

A new RTU was installed to monitor: i) position of the AMIL gate for an indication of stoplog maintenance, ii) position of the new sluice gates, iii) upstream water level, and iv) downstream water level in the Lewis Wash Check reach.

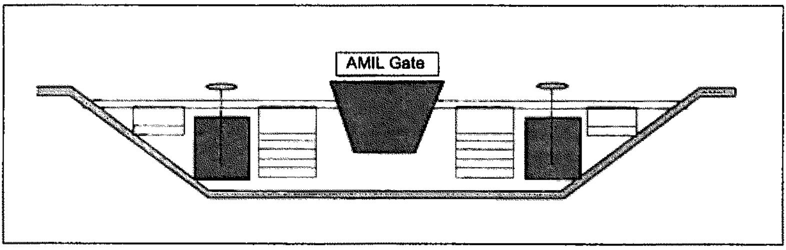

Figure 3. Clifton Check with new manual sluice gates

Lewis Wash Check and all new check structures

Automatic radial gates with two long crested weirs were the selected option for the new structures on the Government Highline Canal. The completed installation of a typical installation is shown in Figure 4. Figure 5 shows the general layout of the check structure. This provides the most flexibility for the GVWUA. The design included automatic electric radial gates in the middle with long crested weirs (LCW) on both sides. The combination of a radial gate(s) in the middle and LCW on each side provides the benefits of both undershot and overshot characteristics.

- By having a constant overpour, there would be significant flexibility in the control of the center gate(s). It could be operated manually or automated, although automation is definitely recommended.

- Because of the long crested weir sides, there will be a fair amount of latitude available in selecting the tuning constants for the automatic controller. The automatic controller tuning constants will be less stringent due to the water level being somewhat maintained by the weir.

- There will be much less gate movements with the weirs than without. This will extend the life of the motors, relays, etc.

- The design is similar to the gates used in the West End and Stage 1, but the weirs will have longer sides and will be submerged at maximum flows.

- A set point of **0.8 feet** above the weir crest will be used. This will provide some flexibility in the operating set point and also better flow characteristics.

- If there were an emergency situation where an additional amount of water came into the canal due to an incident such as a storm, the weir would pass additional emergency flows above the weir set point.

- The center gate will flush out silt while the sidewalls will allow for floating debris to pass over the crest.

Figure 4. Typical Installation of Automatic Radial Gates and Long Crested Weirs

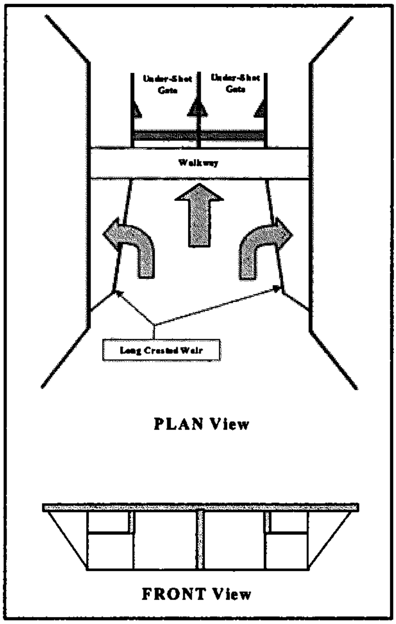

Figure 5. Basic Check Design

Except for the configuration of the long crested weirs and gates, control of the Lewis Wash check structure will be identical to the other check structures. The radial gates will operate under automatic upstream control, with the necessary RTU, gate position sensors, and upstream water level sensors. A separating wall will be installed from the check to the existing siphon so that each siphon pipe can be flushed independently.

Indian Wash Flume

This is a critical site because it is used to monitor the flow rate into the service area farmed by GVWUA customers. A new sensor was installed in the flume to measure the velocity and depth of flow to determine the flow rate. This site is a monitoring site only – there are no control functions proposed at this location. The required flow rate at Indian Wash Flume will be determined with an operational rule that will account for changes in demand, Highline Lake water level, and spill or pumping into/out of Highline Lake. The flow rate will be "controlled" at this point by making manual remote changes to the turnout flow rate at the Palisade Pipeline turnout.

Between the Indian Wash Flume and Stewart Wash

In addition to the Lewis Wash check, a total of six new check structures are proposed. New check structures are proposed to be located downstream of Lewis Wash, A1-1/4, A7, A15, A21.5, A27, and A32. The general structural data for the Government Highline Canal is summarized in Table 1.

Table 1. General Structure Data for Government Highline Canal

	High Flows			Low Flows	
Site	Design Check Q (CFS)	Submerged Weir Flow (CFS)	Rounded Chk. Width @ 0.1' Loss (ft)	Non-Submerged Weir Flow (CFS)	Length of Weir Req'd. w/ 0.8' Set Point (ft)
A1-1/4	579	77	28	154	70
A7	554	73	28	145	65
A15	522	65	28	130	60
A21-1/5	469	59	24	118	55
A27	433	55	24	110	50
A32	332	45	20	95	45

Stewart Wash (existing facility)

The only potential modification to this site might be to adjust the set point for the two AMIL 450 radial gates.

Checks Downstream from the Stewart Wash (existing structures)

No new checks will be constructed in this reach. Controllers on the eight existing checks will be replaced with new RTUs. The new RTU will automatically maintain the water level upstream of the screens using a single radial gate. The entrance conditions to the check were improved to reduce head loss across the structure.

Camp 7 Check

The Camp 7 check is a key structure under new operational procedures. The layout of the Camp 7 check structure and Highline Lake is shown in Figure 6. The Government Highline Canal will be operated under downstream control beyond this point. This Camp 7 check will serve as a new flow rate regulating structure. It will essentially serve as the first check for a new canal section. The check structure will be automated with a new RTU, water level sensors and gate position sensor. It will be able to function for either upstream or downstream flow control.

A new electronic flow measurement device was installed at the Camp 7 check on an existing concrete channel and a new Replogle flume will be added in the channel to the spillway. The required flow rate at Camp 7 check is determined by a rule that accounts for the change in demand downstream of Camp 7, plus the spill flow rate at Badger. A flap gate will be added upstream of the autosiphon structure to maintain the water level of the pool.

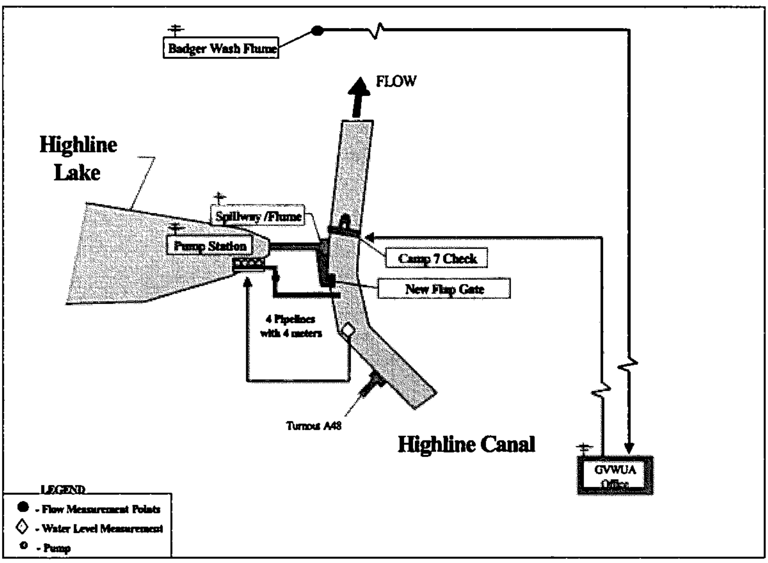

Figure 6. Government Highline Canal at Check 7

Highline Lake Pumping Station

The Highline Lake Pumping Station includes a pumping configuration with four pumps (25, 25, 12.5,and 12.5 cfs). The pump station utilizes a variable frequency drive (VFD) on one of the 12.5 cfs pumps to closely control the water level in the canal.

Each pump will have an individual discharge pipe with a propeller meter to measure flow rate and volume and will be tied into the SCADA system. The pumps discharge into the pool upstream of the Camp 7 Check. They will operate based on the water level in the pool. The RTU for the Camp 7 Check controls the upstream water level (upstream control) or the canal flow rate, and can start and stop pumps to maintain a water level. The control can change from upstream water level control to flow rate control "at the click of a button" by the operator at the GVWUA office. What makes this possible is proper programming of the RTU, measurement of the upstream water level, measurement of the flow rate, measurement of the gate position, limitations to gate travel, and a method of raising and lowering the gate. The only difference between this and any other automatically controlled check structure is that pumps are controlled, both upstream level and flow rate are measured, and programming for three functions are required.

Badger Wash Check

This is the end of the Government Highline Canal. Refer to Figure 7 for a general layout of the site. The modifications to this structure will include the addition of a Replogle flume to monitor spill flow rate and volume, a water level sensor to monitor the upstream water level and rehabilitation of one of the two existing gates (including actuator), limit switches, and a position sensor to provide automatic upstream water level control.

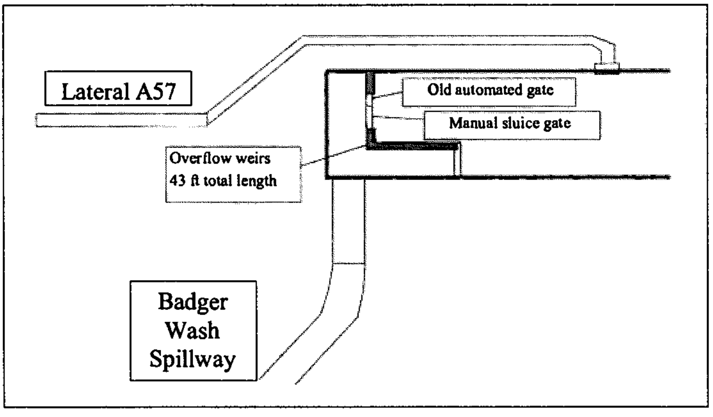

Figure 7. Badger Wash Check and Spill

SUMMARY

In summary, the modernization steps to the Government Highline Canal emphasized simple solutions for the new control strategy necessary to maintain the current level of water delivery service, yet have less spill.

The results of the project will be that about 28,500 acre-feet of water can be used for the 15-Mile Reach of the Colorado River (located above the Gunnison River and Colorado River). These additional flows aid in the restoration of habitat for fish species identified as endangered.

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EVAPOTRANSPIRATION FROM A SATELLITE-BASED SURFACE ENERGY BALANCE FOR THE SNAKE PLAIN AQUIFER IN IDAHO

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ABSTRACT

SEBAL (Surface Energy Balance Algorithm for Land) is an image-processing model comprised of 25 submodels for calculating evapotranspiration (ET) as a residual of the surface energy balance. SEBAL was developed in the Netherlands by Bastiaanssen and has been modified during Idaho studies for application to irrigated agriculture, rangeland, mountainous terrain and clear, cold lakes under semiarid conditions. SEBAL has been applied in many developing countries and has now been applied in southern Idaho to predict monthly and seasonal ET for water rights accounting and for operation of ground water models. Results from SEBAL have been compared and validated using precision-weighing lysimeter measurements from the U.S. Department of Agriculture – Agricultural Research Service (USDA-ARS) at Kimberly, Idaho, and from Utah State University for the Bear River. ET for periods between satellite overpasses was computed using ratios of ET from SEBAL to reference ET computed for ground-based weather stations. ET maps via SEBAL provide the means to quantify, in terms of both the amount and spatial distribution, ET from individual fields. The ET images generated by SEBAL show a progression of ET during the year as well as distribution in space.

Initial application and testing of SEBAL indicates substantial promise as an efficient, accurate, and relatively inexpensive procedure to predict the actual

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evaporation fluxes from irrigated lands throughout a growing season. ET from satellite images may replace current procedures used by Idaho Department of Water Resources and other management entities that rely on ground-based ET equations and generalized crop coefficients that have substantial uncertainty.

INTRODUCTION

SEBAL is an emerging technology that has the potential to become widely adopted and used by the world's water resources communities. ET maps created using SEBAL or similar remote-sensing based processing systems will some day be routinely used as input to daily and monthly operational and planning models for reservoir operations, ground-water management, irrigation water supply planning, water rights regulation, and hydrologic studies.

In Idaho, SEBAL is used to generate seasonal ET maps for predicting effects of irrigation on stream flow depletion in the Bear River Basin and the upper Snake River Basin. The ET maps are also used to predict recharge to ground-water systems and to extend pumpage records for ground-water diversions. The Snake River Plain aquifer system is large, spanning more than 30,000 square km (an area larger than the states of Massachusetts, Connecticut, and Rhode Island combined), with over 7,000 square km (1.7 million acres) of irrigated farmland.

Two SEBAL applications have been made in Idaho using funding from Raytheon Company and the National Aeronautics and Space Administration (NASA). The first application, during Phase I of the study, was to the Bear River Basin of southeast Idaho (Morse et al., 2000). The second application, during Phase II, was to the eastern Snake River Plain of southern Idaho, (Morse et al., 2001).

The theoretical and computational approach of SEBAL is well documented in Bastiaanssen et al., (1998), Bastiaanssen (2000) and Morse et al., (2000). Basically, ET for each image pixel is computed for the energy balance where ET $= R_n - G - H$, where R_n is net radiation, G is soil heat flux density, and H is sensible heat flux density. R_n is computed from satellite-measured broad-band reflectances and surface temperature, G is estimated from R_n, surface temperature, and vegetation indices, and H is estimated from surface temperature ranges, surface roughness, and wind speed using buoyancy corrections. The model was applied in Idaho using the ERDAS Imagine software with the Spatial Modeler. Modifications to SEBAL have included the method for selecting anchor pixels in the energy balance computation and the method for extrapolating ET from the time of the satellite overpass to adjoining periods (Allen et al., 2001).

BEAR RIVER APPLICATION

In 1958, the Bear River Compact was developed to establish how Idaho, Utah and Wyoming would equitably distribute and use water from the Bear River. The role of Idaho Department of Water Resources (IDWR) is to compute depletion by irrigated agriculture for the Idaho part of the basin to support Idaho's position in negotiations with the other two states. IDWR will continue to refine and apply SEBAL in the basin to assist in administration of the Bear River Compact.

In Phase I (2000) of our SEBAL study, ET maps were generated monthly for a 500 km x 150 km area (comprised of 2 Landsat images) encompassing the Bear River basin. Images were processed for 1985, coinciding with an ET study using lysimeters (Hill et al., 1989) that allowed for comparison to SEBAL. Lysimeters near Montepelier, Idaho, just north of Bear Lake, had been planted to an irrigated native sedge forage crop characteristic of the area and local surroundings. The lysimeters were measured weekly. ET from the three lysimeters was averaged to reduce random error and uncertainty in the ET measurements. Results for four satellite images during the 1985 growing season (July 14, Aug. 15, Sept. 16, Oct. 18) are summarized in Figure 1 and Table 1. The results compare well to lysimeter data for the last three image dates. The earliest date, July 14, compares well when examined in context of the impact of precipitation preceding the image date and rapidly growing vegetation during that period (Morse et al., 2000).

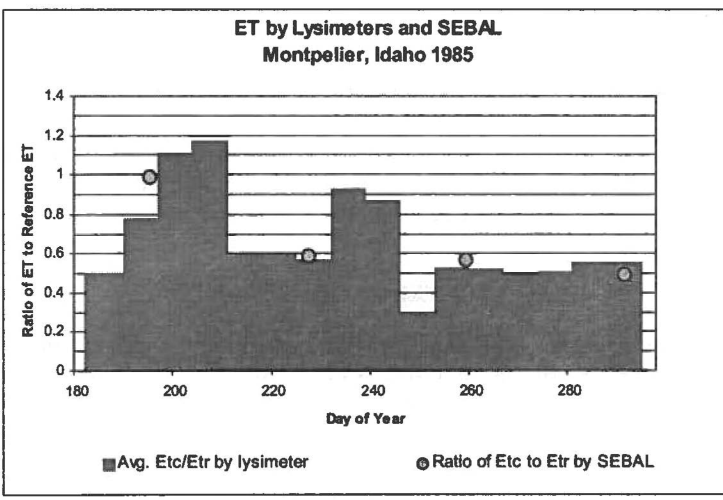


Figure 1. Comparison of ET_r fractions (i.e., K_c) derived from 7-day lysimeter measurements near Montpelier, Idaho during 1985 and values from SEBAL for four Landsat dates (ETc = crop ET).

The Fraction of Reference ET (ETrF) in Table 1 is defined as ET/ET_r where ET_r is reference ET based on an alfalfa-referenced Penman equation (Hill et al., 1989). ETrF values were computed for each pixel and used to extrapolate ET from the day of the satellite image to days between images. ETrF is synonymous with the well-known crop coefficient, K_c. ET_r accounts for changes in ET caused by weather variation between satellite image dates.

Table 1. Summary of SEBAL- and lysimeter-derived ET for weekly and monthly periods and the associated error for Bear River, 1985.

	7-day Lys. ET ave. for image date (mm d^{-1})	SEBAL ETrF on image date	7-day SEBAL ET for image date (mm d^{-1}	Diff. in 7-day ET (SEBAL – Lys) (%)	Monthly Alfalfa ET_r (mm)	SEBAL Monthly ET (mm)	Lys. Monthly ET (mm)	Diff. in Monthly ET (SEBAL – Lys.) (%)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(10)
July	5.3	0.98	6.8	28%	202	198	167	19%
Aug	3.5	0.59	3.7	6%	201	119	145	-18%
Sept	1.9	0.57	2.1	10%	115	66	54	22%
Oct	0.7	0.49	0.6	-14%	45	22	23	-5%
July-Oct.	2.9	0.73	3.3	15%	563	405	388	4%

Predicted monthly ET averaged +/- 16% relative to the lysimeter at Montepelier (Table 1). However, seasonal differences between SEBAL and lysimeters were only 4% due to impacts of reduction in the random error component present in each estimate.

SNAKE RIVER PLAIN APPLICATION

Managing water rights and irrigation on the Snake River Plain and tributary basins presents a challenge to IDWR. Water for irrigation comes from surface and ground sources. For various historical reasons, the use of surface water has been directly measured and regulated by IDWR while the use of ground water has not. This situation began to change in 1995 when the Water Measurement Information System Program was established within IDWR to measure ground-water use. IDWR has dedicated considerable resources to water measurement, including three full-time positions to monitor about 5,000 points of diversion, mostly wells. As useful as these data are, they do not provide all the information necessary for effective management of the resource. Information regarding the ET or consumed fraction of diversions is needed. SEBAL can be used in conjunction with Water Measurement data in an efficient program to help manage water development, use and stewardship. SEBAL covers large areas inexpensively and efficiently, thereby extending Water Measurement data in both time and space, and the Water Measurement data, in turn, can be used to validate or calibrate the SEBAL results.

This combined program offers advantages over present methods: 1) it offers the ability to monitor whether water has actually stopped being used for irrigation after a water shut-off order has been issued; 2) it can discover if more water has been used than authorized; 3) it can quantify and be used as proof of beneficial use of a right; 4) it can be used as an unbiased, quantitative record of historical use; 5) the consumed fraction and return of non-evapotranspired water to the resource can be quantified; 6) estimates of yield and productivity can be made to assess benefits of water development and tradeoffs in water management. In addition, resulting seasonal ET maps are utilized by the State of Idaho, University of Idaho, and U.S. Bureau of Reclamation ground-water modelers to predict recharge of irrigation water to the Eastern Snake Plain Aquifer.

A number of tasks during Phase II (2001) were directed at improving components of SEBAL to better predict ET for environments found in the western United States. These include prediction of net radiation and soil heat flux components and identification and assessment of the energy balance for "anchor" pixels used to define the overall energy balance for the image. Other improvements included determination of mean wind speeds in mountain areas, prediction of aerodynamic roughness for various vegetation covers, and development of an ET reference fraction (ETrF) approach for extending ET between images (Allen et al., 2001).

The production of ET maps having 30 m resolution for the Eastern Snake River Plain Aquifer was highly successful. ET images were created for 12 dates during 2000 and were integrated over the March – October period. Interpolation between image dates was done using ETrF from pixels of each image and multiplying these by ET_r computed for each day between images.

Images were purchased from both Landsat 5 and Landsat 7 archives for 2000 to increase the number available for the southern Idaho area. Often, dates for adjacent Landsat 5 and 7 paths were separated by just one day. Landsat 5 images were of immense value in providing ET for similar periods between paths. Algorithms were developed to correct individual reflectance bands of Landsat 5 to coincide with measurements by Landsat 7 to account for sensor deterioration.

Validation of SEBAL at Kimberly, Idaho

The validation of SEBAL on the Snake River Plain has centered on the use of two precision-weighing lysimeter systems for ET measurement in place near Kimberly, Idaho, from 1968 to 1991. The lysimeter system was installed and operated by Dr. James Wright of the USDA-ARS (Wright, 1982, 1996) and measured ET fluxes continuously. ET data are available for a wide range of weather conditions, surface covers, and crop types. Measurements of net radiation, soil heat flux and plant canopy parameters were frequently made near the lysimeter site. The lysimeter data sets provide valuable information to verify SEBAL over various time scales and for various conditions of ground cover.

Nineteen Landsat 5 satellite image dates were purchased for Kimberly, Idaho, covering the period between 1986 and 1991. These dates had quality lysimeter and cloud-free micrometeorological data and represent a combination of crop growth stages and times of the year. Eight images from 1989 are discussed here.

The lysimeter data for intervening periods between image dates were used to assess the impact of various methods for extending ET maps from a single day to longer periods. They have also been used to assess the variability in ETrF over a day. The success of SEBAL is predicated on the assumption that ETrF for a 24-hour period can be predicted from the ETrF from the instantaneous satellite image. ET_r was calculated for hourly and 24-hour periods using the ASCE standardized Penman-Monteith method for an alfalfa reference (EWRI, 2001), representing the ET from a well-watered, fully vegetated crop, in this case, full-cover alfalfa 0.5 m in height. The denominator ET_r serves as an index representing the maximum energy available for evaporation. Weather data were measured near the lysimeter and included solar radiation, wind speed, air temperature and vapor pressure. Lysimeter data analyses showed ETrF= ET / ET_r to be preferable to the evaporative fraction (EF) parameter used in previous applications of SEBAL (Bastiaanssen et al., 1998, Bastiaanssen 2000)), where EF = ET / (R_n – G). The better performance by ETrF was due to its consistency during daytime and agreement between hourly ETrF at satellite overpass time (~1030) and daily average ETrF. An illustration of ETrF for a day in 1989 is given in Figure 2 for clipped grass (alta fescue) and sugar beets. ETrF for many days was even more uniform than shown in the figure. In nearly all cases, the ETrF for the 24-hour period was within 5% of the ETrF at 1030.

T able 2 summarizes error between SEBAL and lysimeter measurements during 1989. Absolute error averaged 30% for the eight days. When April 18 was omitted, the average absolute error was only 14%. April 18 was before planting of the sugar beets and represented a period of drying bare soil following precipitation. The field at this time was nonuniform in wetness due to differential drying, and differences between lysimeter and estimate were only 1 mm. The standard deviation of error between SEBAL and lysimeter for dates from May – September was 13%. In comparison, a commonly quoted standard error for ET prediction equations that are based on weather data, for example, Penman or Penman-Monteith-types of equations, is about 10% for daily estimates. SEBAL was able to obtain close to this level of accuracy for the field surrounding the lysimeter. Results are illustrated in Figure 3, where ET is expressed in the form of ETrF. ETrF was used to normalize results for differences in climatic demand (i.e. ET_r). The round symbols and horizontal line segments in Figure 3 represent ETrF determined from lysimeter on the image date. These values are those directly compared with SEBAL predictions in Table 2. The triangular symbols in represent the ETrF predicted by SEBAL for the image date.

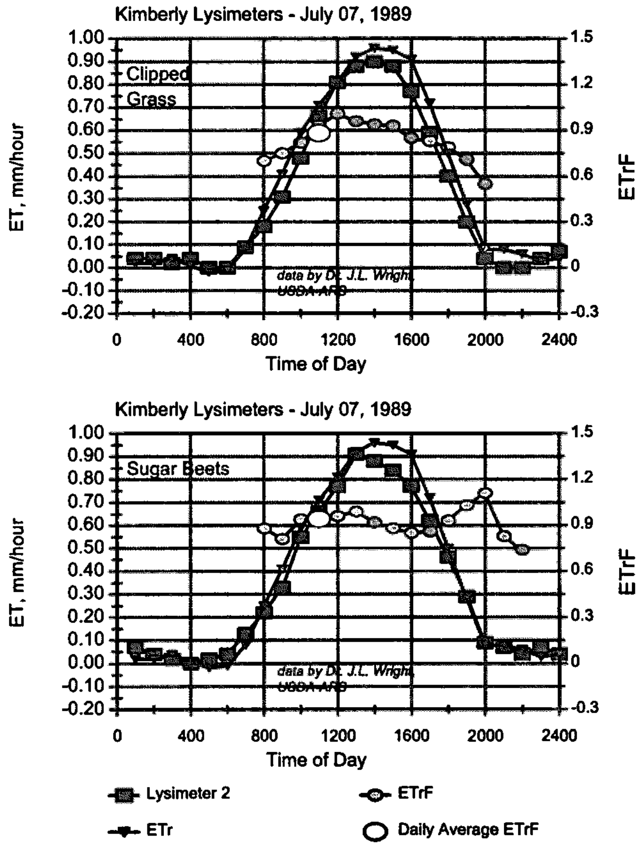


Figure 2. Hourly measured ET, ET_r, ETrF and 24-hour ETrF for July 7, 1989, for clipped grass (top) and sugar beets (bottom) at Kimberly, Idaho.

Table 2 summarizes the extrapolation of ET by SEBAL over the season (April 1 – Sept. 30, 1989). Most periods were 16 days, centered on the image date. April 18 was used to represent April 1 – April 25, July 23 was used to represent July 16 to August 24 and Sept. 25 was used to represent Aug. 25 through Sept. 30. What is surprising is the close agreement for seasonal ET for April 1 – September 30. The difference between SEBAL (714 mm) and the lysimeter measurement (718 mm) was less than 1% for the sugar beet crop. It appears that much of the error occurring on individual dates was randomly distributed, and tended to cancel.

Table 2. Summary and computation of ET during periods represented by each satellite image and sums for April 1 – September 30, 1989, for Lysimeter 2 (Sugar Beets) at Kimberly, Idaho.

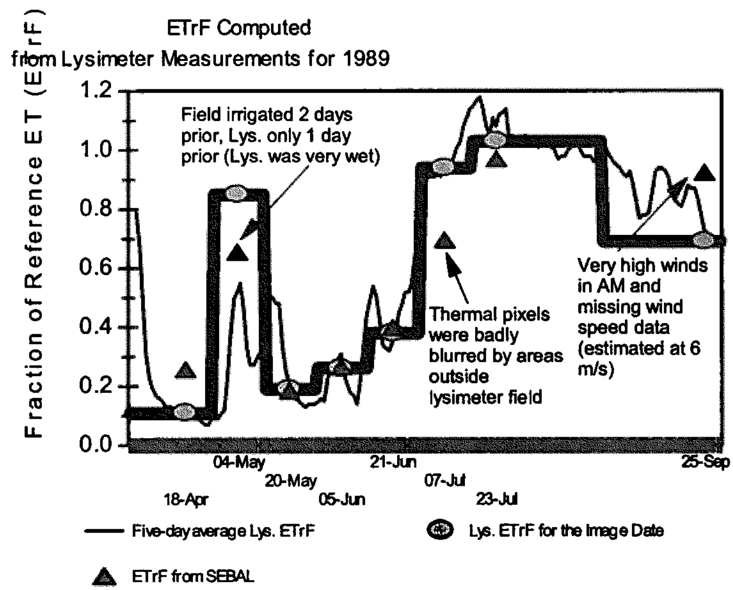
Image Date	Lys. ET on date (mm d^{-1})	SEBAL ET on date (mm d^{-1})	Error on Image Date (%)	ET_r on date (mm d^{-1})	ET_r for period (mm)	Lys. ET summed daily for period (mm)	Lys. ET for period based on image date only (mm)	SEBAL ET for period (mm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
4/18/89	0.73	1.74	139	6.78	147	28	16	38
5/4/89	6.61	5.09	-23	7.76	94	30	80	62
5/20/89	1.37	1.34	-2	7.27	90	22	17	17
6/5/89	1.73	1.78	3	6.68	118	24	30	31
6/21/89	2.39	2.54	6	6.33	127	62	48	51
7/7/89	7.96	5.89	-26	8.44	120	116	113	84
7/23/89	7.64	7.17	-6	7.38	253	266	262	246
9/25/89	5.51	7.40	34	8.00	201	171	138	186
4/1–9/30						718[a]	705[b]	714[c]
Percent Error						--------	-1.8%	-0.6 %

[a] The sum of daily measurements by lysimeter computed as the sum over all days between April 1 and Sept. 30.
[b] The sum of ET computed for each lysimeter period, computed by multiplying summed ET_r during the period by the ETrF for the image date.
[c] The sum of ET predicted by SEBAL for the lysimeter 2 field, computed by multiplying the summed ET_r during the period by the ETrF computed on the image date by SEBAL.

An illustration of the type of resolution for ET maps generated from Landsat imagery is shown in Figure 4 for a 4 km x 6 km area near American Falls, Idaho.

IMPACT

The SEBAL work is evolving. Nevertheless, there have been impacts. IDWR found the results of Phase I and II sufficiently compelling to request additional funding from the Idaho Legislature to include SEBAL as the ET source for recalibration of the Eastern Snake River Plain aquifer model and to generate ET maps to monitor ground-water pumage. The aquifer model uses 5 km grid cells, and aggregating ET up to a 5 km cell is preferable to disaggregating county-averaged data.



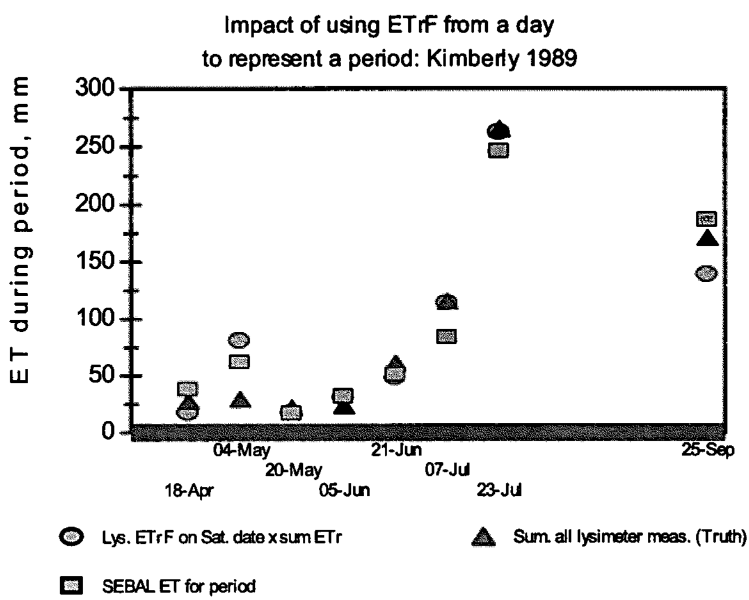


Figure 3. Results by SEBAL and ET by Lysimeter as ETrF (top). The thin line is the five-day average ETrF for lysimeter and the thick line is the assumption used in SEBAL to extrapolate between images. The bottom figure shows total ET for the image period.

Figure 4. Close-up of ET (left) with false color composite (right) from Landsat 7 showing variation within individual fields May 5, 2000.

COST SAVINGS

SEBAL ET data are less expensive to generate than are standard ET data. Since IDWR is still developing the SEBAL data, a quantitative cost-benefit analysis is premature. Nevertheless, it is possible to do a cost comparison based on some available figures. Current costs for monitoring water use on the eastern Snake River Plain are estimated to be about $500,000 per year. We estimate costs for remote sensing to be about $50,000 per year. This includes costs for 30 TM scenes representing 8 to 10 dates for the whole eastern Snake Plain (Landsat scenes cost about $400 each for images more recent than 1998 and about $4,000 each for images older than 1999. Geo-registration of images costs an additional $400 each, for a total procurement cost of about $24,000). SEBAL processing requires about 3 days per scene (90 days * 8 hours = 720 hours * $30.00 per hour = $22,000). The total for remote sensing is $46,000. Set-up and time for aggregation of ET results in a GIS structure results in a total remote sensing cost of $50,000. Using these figures, the estimate cost ratio of remote sensing to the current measurement program is $50,000/$500,000 = 0.10, i.e., remote sensing costs about 10% of the measurement costs. Measurement costs are for a subset of the total number of wells, all of which are not measured in a single year, whereas, SEBAL data cover the entire Snake River Plain and all places of use. The use of SEBAL ET will not replace the existing measurement program, per se. Pumpage data that can be related to individual water rights will be needed to regress against the SEBAL ET data for the same water rights to establish the relationship between volume pumped and volume of ET. That relationship can then be applied to all other non-monitored water rights and their associated wells to estimate both aquifer depletion and water use by individual water rights.

SUMMARY AND CONCLUSIONS

SEBAL uses digital image data collected by Landsat and other remote-sensing satellites that record thermal infrared, visible and near-infrared radiation. ET is computed on a pixel-by-pixel basis for the instantaneous time of the satellite image. The process is based on a complete energy balance for each pixel, where ET is predicted from the residual amount of energy remaining from the classical energy balance, where ET = net radiation – heat to the soil – heat to the air.

In Phase 1 for the Bear River Basin, the difference between SEBAL and the lysimeter, total, for the growing season was 4%. For the Phase 2 comparison with precision weighing lysimeters at Kimberly, differences were less than 2%. These comparisons represent a small sample, but are probably typical. Error as high as 10 to 20%, if distributed randomly, could probably be tolerated by IDWR and by the water user communities.

Comparisons of SEBAL predicted ET with precision weighing lysimeter data at Kimberly, Idaho, from 1989 have provided valuable information on the conditions required to obtain maximum accuracy with SEBAL and the best procedure for obtaining ET monthly and annually. ET has been calculated for the entire Snake River Plain of southeastern Idaho and has improved the calibration of ground-water models by providing better information on ground-water recharge as a component of water balances. Ground-water pumpage from over 10,000 wells has been estimated using ET from SEBAL by developing correlations between ET and pump discharge at measured wells and then extrapolating over large areas using ET maps from SEBAL.

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SPATIAL AND TEMPORAL VARIABILITY IN REFERENCE EVAPOTRANSPIRATION IN OKLAHOMA

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Ronald L. Elliott*

ABSTRACT

Reference evapotranspiration (ET_{ref}) is an important indicator of a region's climate, specifically its evaporative demand. Evapotranspiration (ET) is a key component of the energy and water balance of a given environment. Information on ET_{ref} is essential in the quantification of water use in agricultural, natural and urban landscape systems. Various formulations of ET_{ref} have been used over the past several decades. Recently, the Task Committee of American Society of Civil Engineers (ASCE) Evapotranspiration in Irrigation and Hydrology Committee developed a standardized procedure for computing reference evapotranspiration for grass (ET_{os}) and alfalfa reference crops. Accurate ET_{ref} computations using data from a network of weather stations require a careful check of the data quality and the similarity of site surface conditions to reference standards when data is acquired. Despite a lack of coastal effects or significant orographic influences, Oklahoma appears to exhibit considerable spatial and temporal variability in evaporative demand. The availability of the Oklahoma Mesonet, a comprehensive automated weather station network, provides an opportunity to study ET_{ref} patterns across the state and through time. The Mesonet is a network of 115 well-distributed and well-maintained stations. Using seven years of quality assured data from this network, daily ET_{os} has been calculated for 40 sites representing the diverse Oklahoma climate. Spatial and temporal variability in ET_{os} is discussed. This analysis is a precursor to on-line mapping of reference evapotranspiration and identification of geo-spatial ET_{os} zones.

INTRODUCTION

Quantification ET_{ref} in space and time is required in the analysis of irrigation water management in agricultural and urban landscapes. Evapotranspiration is also a major component in water quantity and quality models.

The rate of ET from soil and vegetated surfaces is dependent upon the atmospheric demand for water and the surface characteristics. In the commonly applied two-step approach to estimating ET, the atmospheric demand is quantified through the calculation of ET_{ref} and the surface characteristics are incorporated into a crop coefficient (K_c). The product of these two parameters provides an

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estimate of the actual ET. ET_{ref} has been defined as "the rate at which water, if readily available, would be removed from the soil and plant surface of a specific crop, arbitrarily called a reference crop" (Jensen et al., 1990). Thus, ET_{ref} computation forms an integral part of water balance studies.

Several ET equations are used to compute ET_{ref}, creating confusion about which to use. To minimize the confusion and facilitate ET comparisons across regions, the ASCE Evapotranspiration in Irrigation and Hydrology Committee has recently adopted a standard reference ET computation method and procedure to compute reference ET for grass and alfalfa crops (Walter et al., 2000). The standard also simplifies the transfer of K_c's from one region to another. The standard equation for daily ET_{os} (mm d^{-1}) is as follows:

$$ET_{os} = \frac{0.408\Delta(R_n - G) + \gamma \dfrac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34 u_2)} \qquad (1)$$

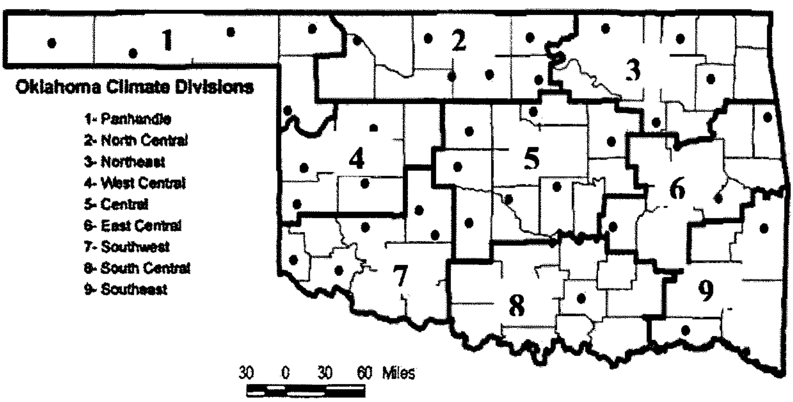
where R_n is net radiation (MJ m^{-2} d^{-1}); G is soil heat flux density at the soil surface (MJ m^{-2} d^{-1}) ; T is mean daily temperature at a 1.5 to 2.5 m height (°C); u_2 is mean daily wind speed at 2-m height (m s^{-1}); e_s is mean saturation vapor pressure at a 1.5 to 2.5-m height above the surface (kPa); e_s is the average of e_s at maximum and minimum air temperature; e_a is mean actual vapor pressure at a 1.5 to 2.5-m height above the surface (kPa); Δ is the slope of the vapor pressure-temperature curve (kPa) and γ is the psychrometric constant (kPa $°C^{-1}$).

As the need for timely ET information continues to grow, the advancement in electronic instrumentation and communication have expanded real time operated automatic weather stations. These stations measure weather variables that enable one to compute ET_{ref} directly. The Oklahoma Mesonet is a unique network with a high density of well-maintained and operated stations. It has 115 stations covering the entire state with an average spacing of 32 kilometers (Elliott et al., 1994). At each site, standard weather and soil variables are sensed every few seconds (Brock et al., 1995), logged at 5-minute intervals, transmitted to a central facility every 15 minutes, and then verified and made available to customers in near real-time. The availability of the Mesonet's quality data allows the study of the spatial and temporal patterns in ET_{os} across the state, as well as the ability to provide on-line ET_{os} tables and maps for customers. ET_{ref} computations require good quality weather data collected under standard reference surface conditions, i.e., an extensive open surface covered by green grass or other short vegetation that is not short of water. Thus, it is necessary to understand site conditions before computing ET_{ref} directly from data measured at any weather stations.

CLIMATE OF OKLAHOMA

Oklahoma climate is somewhat correlation with its topography. The land surface elevation generally decreases from the west to the east. The vegetation becomes gradually more dense from the semiarid western plains in the Panhandle to the eastern woodlands. Mean annual precipitation ranges from 380 mm in the western plains to 1270 mm in the southeast, and there can be large seasonal and inter-annual variations. Most of the moisture for precipitation is carried from the Gulf of Mexico by the southerly winds. The average annual temperature gradually increases from north to south. The state is divided into nine climate divisions (OCD) as shown in Figure 1.

Figure 1. The Oklahoma climate divisions (OCD) and the locations of the 40 Mesonet sites used in the study.



METHODOLOGY AND PROCEDURE

A total of 40 Mesonet sites with adequate fetch and representing the different climatic regions of the state were selected using site information and photographic documentation (http://okmesonet.ocs.ou.edu/siteinfo/). Three to seven best reference sites with continuously measured data for 1994 to 2000 were chosen from each climate division. For the selected sites, data quality and reference condition checks were made and ET_{os} values were computed for each site daily for seven years. Then, the monthly mean daily ET_{os} ($ET_{os\text{-}m}$) for each month of each year was calculated for each site. Finally the seven years mean ($ET_{os\text{-}7Yav}$), maximum, and minimum of the $ET_{os\text{-}m}$ were determined and their spatial and temporal variability analyzed.

Data quality assurance procedure

The Oklahoma Mesonet has a comprehensive data quality assurance system that incorporates an instrument laboratory, routine and emergency field visits, automated computer routines, and manual inspection of data (Shafer et al., 2000). Each Mesonet datum is archived together with a quality control flag that indicates its degree of accuracy. In our computations of ET_{os}, we used only data that were flagged to be of good quality. In this study, additional data integrity checks such as comparing measured clear day solar radiation to theoretical solar radiation envelopes, and average dew point temperature to daily minimum temperature, were made using guidelines given by Allen (1996).

Check of reference conditions

The standard weather variables that need to be measured under reference conditions and required for ET_{ref} computation include solar radiation, air temperature, wind speed, and humidity. The weather variables, especially the temperature and humidity to be used in the Penman type one-dimensional flux equations, need to be measured within the internal boundary layer that is in complete equilibrium with the reference surface. Thus, it is critical that measurements of reference weather data be collected above a standard reference surface.

Some of the site selection standards used by the Mesonet specified that the sites be placed in a rural and relatively large flat open area free of obstacles within 300 m, and covered with a uniform low-cover, preferably grass vegetation (Shafer et al., 1993). Thus, most of the Mesonet sites are covered with short native grass and have a good fetch in all directions. However, the Mesonet sites are not irrigated to keep the vegetation green and transpiring fully at all times during the growing season. This is likely to be more of a problem for shallow rooted grasses as compared to deep rooted and drought-resistant native vegetation. During the growing season, the extent that a Mesonet site will be at or near reference conditions depends on the availability of adequate soil moisture for reference vegetation to meet its ET demand. Hence, the net rainfall (that infiltrating into the soil) is critical. Depending on the net rainfall at a site, the surface condition can vary from a reference type to one covered with dried vegetation and with negligible ET. Thus for the Mesonet sites, the rainfall amount, frequency and distribution become key in determining whether the site surface condition is that of reference. It is necessary to identify any extended dry periods and examine the data and ET_{os} computed during such periods carefully.

On clear days, the partitioning of the energy balance on a non-reference surface generates high sensible heat due to the relatively less or non-transpiring vegetation, thus increasing the temperature measured above the surface. The high temperature and the low humidity due to the dry air above the surface result in a higher calculated vapor pressure deficit than what it would have been under a normal reference condition. Vapor pressure deficit can also be transported from a surrounding dry area. This non-reference effect will result in the over prediction of ET_{os}. For example, Brown (2001) reported the impact of extreme site aridity at Parker, AZ, that showed significant increase in monthly total ET_{os} (18-26%) during June through September. The non-reference effect of over predicting ET_{os} is particularly important during peak ET periods because they directly affect the size of irrigation systems. For irrigation scheduling, unrealistically high ET_{ref} values can translate into inefficiencies due to over watering, but that may in fact be more desirable than under irrigation that results in crop stress.

Since reference surface conditions directly modify the temperature and humidity measurements at a given weather stations, comparing these two measurements with their corresponding ranges under standard reference conditions helps us to identify potential non-reference conditions. Under reference conditions, the maximum relative humidity (RH_{max}) generally exceeds 90% during early morning hours, especially when skies are clear and winds are light, and minimum relative humidity RH_{min} is expected to be above 25%. The minimum temperature (T_{min}) will also approach the dewpoint temperature (T_{dew}) and T_{min} minus T_{dew} (T_{min}-T_{dew}) is often less than 2°C and in a semiarid environment it is likely to be less than 4 or 5°C (Allen, 1996; Allen et al., 1998). Although most of Oklahoma, especially the western part, is characterized by high winds that strengthen the mixing of the boundary layer and increase the transport of air properties from the surroundings making the processes more dynamic and complex the assessment variables mentioned above could help us identify non-reference days. Thus, in our checks for reference conditions, plots of RH_{max} and RH_{min}, as well as the trend in both T_{min} and T_{dew} combined with the precipitation data were used to identify days where the site conditions are more likely to be of a non-reference condition.

Checking the daily data for the 40 sites, the growing season's T_{min}-T_{dew} for days with significant rainfall (greater than 8 mm) is on average lower than 7°C for the semi-arid sites in the western part of the state and 4°C for the more humid sites in the east. The RH_{max} for these reference days is above 90% most of the time and in particular approaches 100% for the sites in the east. On non-reference days T_{min}-T_{dew} reached as high as 20°C in the west and 10°C in the east. Querying the growing season daily data for a site, using T_{min}-T_{dew} of greater than 5°C and RH_{max} of less than 90% appears to best capture the non-reference days across the sites. In this study, if both of the above criteria were met, the day was assumed to be a non-reference day. In general, the sites in the western part of the state which are characterized by relatively less rainfall and high winds were found to have a greater occurrence of non reference conditions compared to the sites in the east. Moreover, sites within the same OCD showed different patterns of non-reference periods through the months and the years due to the variation in the amount of rainfall received at each site. For most of the sites the month with most non-reference days during the seven years period was September, followed by August. The non-reference days at a site per month varied from zero days up to three weeks. A long dry period in August and September 2000 affected all 40 sites. During this dry period, the non-reference days at a site in August and September varied from 39 days in the Panhandle to 2 days in the southeast. On the other hand, June and July 1999 were without non-reference days for most of the sites.

ET_{os} for each site was computed using all the data for the seven years as well as after removing all non-reference days identified at each site. The results showed that the ET_{os-m} without removing the non-reference days was higher by 2-18% compared to the ET_{os-m} after removing the non-reference days. The effect of not filtering non-reference days is high in our ET_{os} especially during the peak ET_{os} days. The yearly peak daily ET_{os} overestimation at a site varied from 0 to 49%. Most of the peak daily ET_{os}s computed at a site were found to be associated with non-reference days. Allen et al. (1998) and Temesgen et al. (1999) suggested simple empirical adjustment procedures for air temperature and humidity data measured under non-reference conditions. In this study, no attempt was made to correct data for non-reference conditions. But, all ET_{os} for the non-reference days, as identified above, were excluded in the analysis to be discussed in the next section.

Spatial and temporal trend analysis procedures for computed ET_{os}

Spatial and temporal trends of ET_{os-m} and $ET_{os-7Yav}$ were studied. Analysis were limited to growing-season months as these periods are of most interest to agriculture and are characterized by active vegetation growth consistent with the reference ET computation. The spatial and temporal analysis of variance of the ET_{os-m} and $ET_{os-7Yav}$ were analyzed using Statistical Analysis Software (SAS), (SAS Institute Inc., 2000) and other standard statistics that measure variability and the extremes. Separation of means was determined by the least significant difference (LSD) method. For the SAS analysis of the $ET_{os-7Yav}$, a probability level $\alpha = 0.05$ is implied whenever significant difference is mentioned in the text. The coefficient of variation (CV), the ratio of the standard deviation to the mean, is used to measure spatial variability and consistency of ET_{os} across the sites. In addition, the maxima and the minima of ET_{os-m} and $ET_{os-7Yav}$ provide insight about extreme values. Using the above statistics the temporal variability of $ET_{os-7Yav}$ across the growing season months and years and the spatial variability across the OCDs were identified and discussed.

RESULTS AND DISCUSSION

The seven years (1994-2000) growing season $ET_{os-7Yav}$ computed for the 40 sites representing the nine climatic divisions of Oklahoma are in Figure 2. The elevations of the sites are included in the figure. On the abscissa of Figure 2, the sites within each OCD (the character following site ID on the abscissa designates its OCD) are arranged from left to right in a decreasing longitude (west to east) and in a decreasing site elevation. The top and bottom smooth lines enclosing the growing season's $ET_{os-7Yav}$ in Figure 2 indicate the seven-year maximum and minimum $ET_{os\text{-}m}$ at each site. In the same figure, the maximum $ET_{os\text{-}7Yav}$ for most of the sites occurred in July followed by June or August, and the minimum occurred in October. $ET_{os\text{-}7Yav}$ of 7.34 mm d^{-1} in July and 2.36 mm d^{-1} in October are the extremes. In general, $ET_{os\text{-}7Yav}$ for all the months uniformly decreases with decreasing elevation from the western part of the state to the east. Linear regression of each month's $ET_{os\text{-}7Yav}$ on elevations for all sites resulted in r^2 that ranged from 0.66 in October to 0.36 in April with an overall value of 0.54 for all the months. Analysis of the 40 sites, seven years monthly averages showed vapor pressure deficit and wind speed to be the major factors determining the spatial and temporal distributions of ET_{os}. The linear regression of each month's seven years average vapor pressure deficit in kPa and U_2 in m s^{-1} on site longitude in decimal degrees resulted in an average r^2 of 0.64 and 0.62 respectively.

Temporal variability

The temporal variation of $ET_{os\text{-}m}$ and $ET_{os\text{-}7Yav}$ through the growing season months and the seven years is summarized in Tables 1a and 1b. Table 1a is arranged such that each year's $ET_{os\text{-}m}$ mean, maximum, minimum and the CV are given in rows followed by their seasonal mean at the end of the row. The rows at the bottom of the table show similar statistics for the $ET_{os\text{-}7Yav}$. The bottom of Table 1a shows that the mean $ET_{os\text{-}7Yav}$ for all the 40 sites is the highest in July with 6.27 mm d^{-1} and the lowest in October with 3.06 mm d^{-1}. The extremes in the $ET_{os\text{-}m}$ for each year show that the highest was in June 1998 with 8.81 mm d^{-1} at site ALTU (Altus) in OCD 7, and the minimum in October 1994 with 1.58 mm d^{-1} at IDAB (Idabel) in OCD 9. The growing season average $ET_{os\text{-}7Yav}$ is given as 4.81 mm d^{-1}. Analysis of variance on $ET_{os\text{-}m}$ for the growing season months and $ET_{os\text{-}7Yav}$ for the seven years showed significant temporal variability. Pair-wise comparisons on the means based on LSD showed that only June and August are not significantly different from each other (with p value < 0.0001). See the first column in Table 1b. Similar statistical analysis on the yearly mean of $ET_{os\text{-}m}$ showed that some of the years are not significantly different from one another as shown in the second column of Table 1b.

Spatial variability and ET_{os} grouping

As shown in Figure 2 and as discussed above, the $ET_{os\text{-}7Yav}$ variation across the state as we go from west to the east is evident. This variation is also related to the gradually decreasing site elevations from west to east. It appears that a cursory comparison of the growing season $ET_{os\text{-}7Yav}$ for all sites and the respective OCD, as depicted in Figure 2 results in three groupings of the OCDs depending on the magnitude, and the pattern with which the $ET_{os\text{-}7Yav}$ changes within OCD, and the elevations of the sites. The first group could include the sites in the eastern third of the state, those within OCD 3, 6 and 9. These sites show a relatively low and uniform magnitude of $ET_{os\text{-}7Yav}$ compared to the sites in the rest of the central and west OCDs. The second group could consist of the sites in the central part of the state in OCD 2, 5 and 8, which is in a transition from the other zones in the west to those in group one in the east. The third group could be the sites situated in the western third of the state (OCD 1,4 and 7), which are those with a relatively higher ET $_{os\text{-}7Yav}$. However, analysis of variance based on the 40 sites ET $_{os\text{-}7Yav}$ showed significant difference among the nine OCDs ET $_{os\text{-}7Yav}$s. Pair-wise comparisons on the means of the OCD's average ET $_{os\text{-}7Yav}$ based on LCD resulted in four groups as shown in the last column of Table 1b, agreeing with group one and group three discussed earlier. But OCD 2 was identified to be unique, and OCD 8 and OCD 5 could join group one.

In Table 1a, the CVs for each month and in each year indicate the spatial variability of $ET_{os\text{-}m}$ across the 40 sites. Similarly the CVs at the bottom of the table show the spatial variation of ET $_{os\text{-}7Yav}$.across all the sites for each month. The seven years average CVs show that May and June, have relatively higher variability across the sites than the months of April and August.

SUMMARY

The Oklahoma Mesonet provides a high quality, spatially dense, and temporally continuous quality weather data set that allows the study of spatial and temporal patterns in ET_{os} across the state. Rainfall amount and frequency influence the number of reference quality data available to compute ET_{os} at a site. Non-reference days were identified as those with daily T_{min} minus T_{dew} of greater than 5 °C and RH_{max} of less than 90%. Sites in the western part of the state had more non-reference days than those in the east. Analysis of the growing season $ET_{os\text{-}m}$ and $ET_{os\text{-}7Yav}$ over a seven-year period showed a significant temporal and spatial variation across the state. Overall the highest $ET_{os\text{-}7Yav}$ is in July and the lowest in October. In general $ET_{os\text{-}m}$ increased from east to west with increase in site elevation. Vapor pressure deficit and wind speed are the major factors that determine the spatial and temporal distributions of ET_{os} across Oklahoma. The comparison of the climate division means of the $ET_{os\text{-}7Yav}$ resulted in three groups, thus identifying the potential ET_{os} zones for the state.

Figure 2. Growing season ET $_{os\text{-}7Yav}$, maximum and minimum $ET_{os\text{-}m}$ for the 40 sites

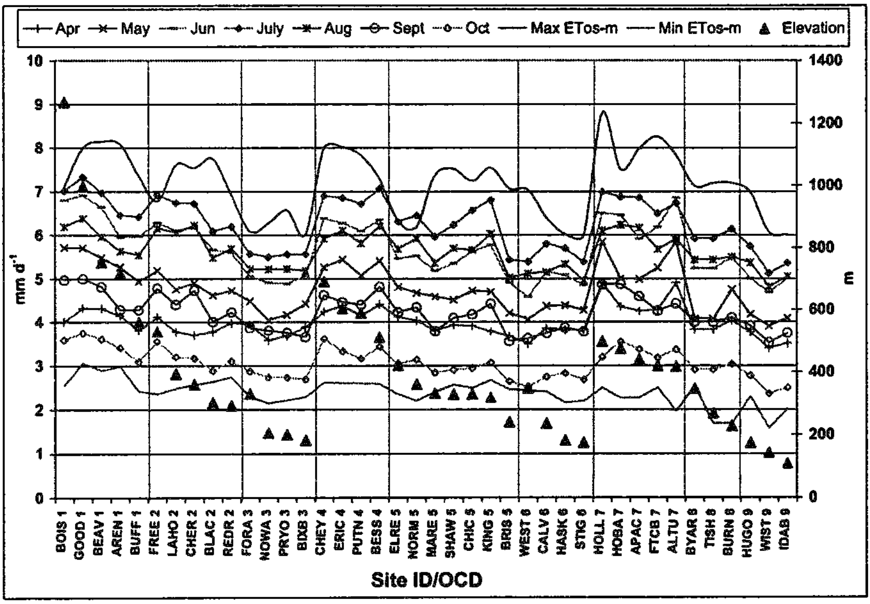


Table 1a. Statistics for monthly and yearly $ET_{os\text{-}m}$ and $ET_{os\text{-}7Yav}$ for the 40 sites

Period	Statistic	April	May	June	July	August	Septembe	October	Mean
		$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$	$mm\ d^{-1}$
1994	Mean	4.30	4.26	6.52	6.28	5.86	4.36	2.71	4.90
	Maximum	5.84	5.83	8.21	7.98	7.19	5.98	3.82	6.41
	Minimum	3.31	1.68	5.24	4.83	4.39	3.35	1.58	3.48
	CV	0.13	0.21	0.15	0.15	0.16	0.17	0.15	0.16
1995	Mean	3.69	3.77	5.28	6.32	5.77	3.45	3.67	4.57
	Maximum	4.31	4.83	6.67	7.34	7.39	4.67	4.68	5.70
	Minimum	3.04	3.20	4.40	5.16	5.06	2.99	1.96	3.69
	CV	0.08	0.11	0.09	0.10	0.09	0.10	0.15	0.10
1996	Mean	4.77	5.73	6.13	5.47	4.70	3.52	3.24	4.80
	Maximum	6.68	7.94	7.35	6.34	5.51	4.37	3.96	6.02
	Minimum	3.69	4.17	4.82	4.50	3.89	2.88	2.14	3.73
	CV	0.13	0.16	0.12	0.08	0.08	0.11	0.14	0.12
1997	Mean	3.39	4.76	5.31	6.21	4.94	4.26	2.73	4.52
	Maximum	3.65	5.70	6.20	7.66	5.81	5.23	3.83	5.44
	Minimum	3.02	4.06	4.43	5.09	4.03	3.22	2.01	3.70
	CV	0.05	0.08	0.09	0.12	0.11	0.12	0.16	0.10
1998	Mean	3.97	5.26	6.83	6.79	5.65	4.65	2.88	5.15
	Maximum	4.96	6.62	8.81	8.00	6.52	5.94	3.75	6.37
	Minimum	3.21	4.17	5.31	5.57	4.91	3.39	2.24	4.12
	CV	0.09	0.12	0.12	0.09	0.08	0.14	0.17	0.11
1999	Mean	3.86	4.68	4.92	6.65	6.03	4.14	3.53	4.83
	Maximum	5.08	5.92	6.79	7.84	6.96	5.63	4.90	6.16
	Minimum	3.18	3.69	3.92	5.14	5.18	3.28	2.82	3.89
	CV	0.12	0.12	0.15	0.10	0.07	0.12	0.15	0.12
2000	Mean	3.94	4.97	4.76	6.24	6.62	5.16	2.65	4.90
	Maximum	5.09	6.26	6.68	7.81	7.83	6.09	3.06	6.12
	Minimum	3.07	3.92	3.80	5.04	5.78	4.34	2.26	4.03
	CV	0.10	0.14	0.15	0.13	0.08	0.09	0.07	0.11
7 years average	Mean	3.99	4.77	5.68	6.27	5.65	4.22	3.06	4.81
	Maximum	4.91	5.86	6.91	7.34	6.39	5.00	3.75	5.74
	Minimum	3.40	3.91	4.58	5.13	4.81	3.52	2.36	3.96
	CV	0.08	0.12	0.12	0.10	0.08	0.10	0.11	0.10

Table 1 b. Summary of statistics for temporal (yearly, monthly) and spatial (OCD) variability for the 40 sites.

Month	*Mean ET_{os-m} mm d^{-1}	Year	*Mean ET_{os-m} mm d^{-1}	OCD	*Mean $ET_{os-7Yav}$ mm d^{-1}
April	3.99^{a}	1994	4.90^{a}	1	5.27^{a}
May	4.78^{b}	1995	4.57^{b}	2	4.97^{b}
June	5.68^{c}	1996	4.80^{a}	3	4.33^{c}
July	6.28^{d}	1997	4.52^{b}	4	5.25^{a}
August	5.65^{c}	1998	5.14^{c}	5	$4.68^{d,c}$
September	4.22^{e}	1999	4.83^{a}	6	4.29^{c}
October	3.06^{f}	2000	4.91^{a}	7	5.29^{a}
				8	4.57^{c}
				9	4.18^{c}

*Means with the same letter, within a column, are not significantly different at the probability level $\alpha = 0.05$.

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IMPACTS OF POPULATION GROWTH AND CLIMATE CHANGE ON CALIFORNIA'S FUTURE WATER SUPPLIES

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ABSTRACT

CALVIN, an economic-engineering optimization model of California's water supply system, is used to assess the interaction of potential impacts of population growth and climate change on water supplies in California. CALVIN includes the state's surface and groundwater system and is driven by economic values for agricultural and urban water use. Future agricultural and urban demands are represented by water value functions estimated from separate agricultural and urban economic models. In addition, values for hydropower generation and flood damage reductions are being developed for CALVIN to represent these additional water management benefits. This allows CALVIN to re-operate statewide facilities and allocate water supplies to minimize economic losses to agricultural, urban, hydropower, and flooding sectors throughout the system for changes in climate or water demands. Methods for developing economic values of the different water functions and for adjusting hydrologic inputs for climate change in CALVIN are presented. Water supply impacts to agricultural and urban sectors are assessed for a range of population and climate scenarios.

INTRODUCTION

Concurrent with expected climatic changes in California over the next 50 to 100 years are likely increases in population. How these two dynamic forces may interact and affect future water supplies and the operation and management of California's water resource system at local, regional, and statewide scales, is the focus of this research project. Funded by the California Energy Commission (CEC), the study explores the potential for adaptation to impacts of climate change and population growth on the state's water resource system and the associated economic costs of these impacts. By studying both effects, the sensitivity of climate impacts to population assumptions can be evaluated.

Already, water supplies are strained in dry years in California creating substantial water scarcity and severe problems balancing urban, agricultural and

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environmental demands (California Department of Water Resources (DWR) 1998). Flooding in the Central Valley is an important water management concern with potential damages in the tens of billions of dollars (US Army Corps of Engineers (USACE) 2002). Reservoir flood reserve pools are an integral component of flood protection, while reservoir-based hydropower supplies significant amounts of the state's electric generation.

Consequences of global warming for California's climate include higher maximum temperatures, more intense precipitation, increasingly dry summers, and increased risk of drought (see Miller et al. (2001) for a review of climate change studies). Currently, California's water supplies, water infrastructure, and the management of water resources are finely tuned to historic runoff timing and magnitude, especially snowmelt-driven Sierra Nevada runoff. Investigations of California streamflow response to climate change indicate that runoff is likely to peak earlier in the season (Miller et al. 2001). However, the full extent of shifts in runoff timing and magnitude across the state are uncertain (Roos 2001).

This study examines a range of climate scenarios for California's water supplies, including two atmospheric general circulation model (GCM) scenarios and combinations of temperature and precipitation changes. Runoff changes are simulated by Miller et al. (2001) for representative headwater basins, forced by precipitation and temperature variables using coupled soil moisture accounting and snow accumulation/ablation models. Six watersheds capture a variety of altitude and location conditions to represent basin scale sensitivities to climate changes. Simulated runoff changes, and associated temperature and precipitation changes provide a set of monthly perturbation parameters to drive hydrologic changes for input to large-scale water resources system models such as CALVIN.

Long-term population growth depends on demographic changes and net migration, both difficult to forecast for California beyond 20 years. Official projections for 2040 span a low of 46.8 million to a high of 63.4 million, reflecting a large range of uncertainty (Johnson 1999). For this study, low and high population scenarios for 2100 are developed, along with their projected spatial footprints (see Landis 2000) to test the sensitivity of climate change impacts to population uncertainties and bookend a range of impacts to the state's water resources.

CALVIN MODELING APPROACH FOR CLIMATE CHANGE

Adaptation to changing climatic and water demand conditions will require system flexibility as well as physical changes to water management infrastructure in California (VanRheenen et al. 2001). The need to explore flexible operating and management policies that balance multiple water resource objectives and identify promising long-range system changes has motivated the choice of the CALVIN optimization model for this study. Economic optimization, in contrast with simulation models that require specifying operating rules a priori, allows the system to adapt operations and allocations within the physical capacities of infrastructure to maximize benefits (minimize costs) across hydropower generation, flood damage mitigation, agricultural and urban water supply. Economics is the compensatory tool to make such tradeoffs, with the exception of environmental demands that are represented as required flows.

CALVIN Model

CALVIN is an economic-engineering optimization model of California's water supply system, explicitly integrating the operation of water facilities, resources and water demands across the state's entire inter-tied surface and groundwater system from Trinity-Shasta in the north to the Mexico border in the south. The CALVIN model uses the US Army Corps of Engineers' HEC-PRM network flow with gains optimization solver to find water operations and allocations that maximize regional or statewide economic benefits (Jenkins et al. 2001).

Covering 92% of California's projected 2020 population and 88% of its estimated irrigated acreage in 2020, the CALVIN model contains roughly 1,200 spatial elements, including 51 surface reservoirs, 28 groundwater basins, 19 urban economic demand areas, 24 agricultural economic demand areas, 39 environmental flow locations, 113 surface and groundwater inflows, and numerous conveyance and other links representing the vast array of California's water management infrastructure (Jenkins et al. 2001). The model's schematic is available at http://cee.engr.ucdavis.edu/faculty/lund/CALVIN/.

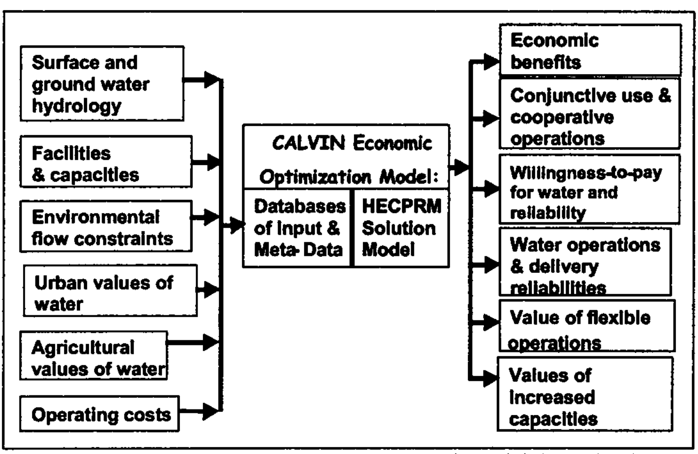

Figure 1. CALVIN Data Input, Output and Model Organization

The optimization is currently driven by economic values for agricultural and urban water use across the network system, subject to environmental flow constraints and flood storage operations. CALVIN is a data driven model as shown in Figure 1 that produces a variety of economic and water supply reliability information. For this study, enhancements to CALVIN include adding economic values for reservoir-based hydropower generation and damage costs for flooding along the Sacramento and San Joaquin River systems, as described below. The economic representation in CALVIN of these additional water resource management objectives allows us to more fully explore the potential for operational and policy changes along with infrastructure expansion to adapt to and mitigate impacts of climate change on water resources for California.

Modeling Approach for Climate Change

Figure 2 depicts the modeling approach, data flow, and linkages among modeling components of the CEC climate study. This paper addresses the shaded boxes and their linkages to the CALVIN model in Figure 2. Other CEC researchers are developing the modeling components shown by unshaded boxes.

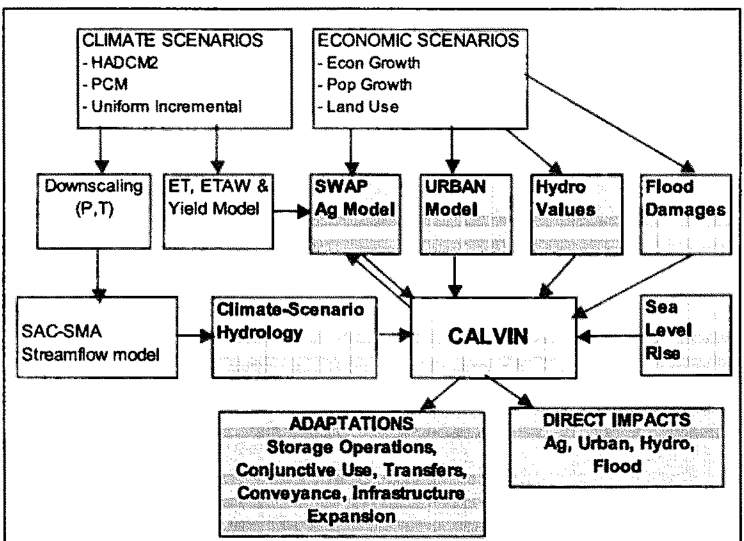

Figure 2. Model Linkages and Data Flow - CALVIN Climate Impacts Study

The climate and economic scenarios are combined to define a set of perturbations, including changes to temperature, precipitation, runoff, population, income and land use. These are then translated into spatially disaggregated inputs to drive the CALVIN model as described below.

Hydrology

CALVIN hydrology consists of rim inflows from outside the modeled system, net local stream accretions from within the modeled system, and net groundwater recharge (exclusive of agricultural and urban return flows) (see Technical Appendices, Jenkins et al. 2001). All three of these components are perturbed for climate change. The 38 rim inflow points are mapped to the six representative headwater climate change basins for which monthly runoff ratios are available from the National Weather Service Sacramento Soil Moisture Accounting Model (SAC-SMA) streamflow simulations (see Miller et al. 2001). The appropriate runoff ratios are then used to perturb the 1922-1993 monthly historic time series for each CALVIN inflow. Central Valley groundwater recharge and local accretions are perturbed using precipitation changes. Changes in the historic monthly volume of precipitation over each of 21 groundwater basins are partitioned into deep percolation and runoff using empirical relationships for each basin. The historical hydrologic time series of monthly groundwater inflows and local accretions for each basin are then adjusted accordingly. No attempt is made to adjust stream-aquifer gains. Adjustments to local accretions and groundwater recharge for land use changes are handled within CALVIN explicitly through return flows from deliveries to agricultural and urban demands within each basin.

Urban Demands

CALVIN uses economic water value functions to represent the demands of 19 urban areas included in the system. These value functions are developed from estimates of population, per capita use, sector water use breakdowns (residential, commercial/public, and industrial), industrial water production values, and monthly use patterns for each urban area, and estimates of seasonal residential price elasticities of demand in California (see Appendix B, Jenkins et al 2001). For the CEC study, spatially disaggregated urban land use and population changes for each urban area in CALVIN are computed from statewide projections developed for each economic scenario. Estimated changes in per capita use are based on location-specific population density and income effects using relationships derived from California Department of Water Resources urban water use forecasting data (DWR 1998). These estimates are then combined to develop new urban water value functions for each economic scenario studied. No attempt is made to adjust residential price elasticities or sector breakdowns. In some scenarios, new CALVIN urban demand areas are added in the Central Valley.

Agricultural Demands

The Statewide Agricultural Production model is used to develop agricultural water value functions for CALVIN (Appendix A, Jenkins et al. 2001). CALVIN includes 24 production areas. In the CEC study, climate effects on yield are estimated and used to adjust crop production functions. Land use changes from the economic scenarios are also input to the model along with shifts in commodity demand, to develop new agricultural water demand functions for each of the 24 areas consistent with a scenario's land use changes and urban demands.

Hydropower

Energy production models and estimates of the value of energy are combined to develop hydropower values as a function of month, storage level, and flow for the major reservoirs with variable head hydropower facilities in CALVIN. Fixed head hydropower facilities are represented more simply as monthly linear functions of flow. Seasonal variation in the price of electricity reflects winter vs. summer demand as it has historically affected electricity prices in California. Sensitivity to energy prices will be examined in the course of the study. An iterative solution technique in HEC-PRM is used to find an optimal network solution for CALVIN when variable-head hydropower is included.

Flooding

Flood damage data from the Sacramento-San Joaquin River Basin Comprehensive Study (USACE 2002) is used to derive monthly flow-damage functions along Central Valley river reaches with flood damage potential in CALVIN. The 2000 level-of-development data is scaled using spatially disaggregated population projections for the scenarios modeled in this study. Damage-discharge relationships for flood events are converted to monthly damage functions by relating data on flood peak flows with associated monthly flow volumes. While reservoir flood reserve pools are maintained in CALVIN at current levels based on historical hydrology, the addition of flood damage functions on downstream reaches will drive CALVIN to increase flood reserve pools if this is economically optimal with climate induced changes to runoff hydrology.

Sea Level Rise

Temperature rise is expected to raise sea level (Titus and Narayanan 1995). The effects of projected sea level rise on water quality in the San Francisco-Bay Delta and subsequent operational changes to maintain salinity levels below regulatory thresholds and legal requirements for Delta exports are estimated in this modeling component. The output will produce a modified monthly time series of required Delta outflows for input into CALVIN as part of adjusting environmental requirements for climate change. The higher expected outflows are likely to produce operational changes on the Sacramento and San Joaquin River systems in the process of adapting to changing climate conditions.

Model development and testing is currently under way to generate the various CALVIN inputs (disaggregated economic value functions for urban demand areas, agricultural production areas, hydropower facilities, and flood damage areas; sea level rise changes to required Delta outflows; perturbed climate hydrologic data) needed to model the economic and climate scenario in CALVIN.

MODIFYING CALIFORNIA'S HYDROLOGY FOR CLIMATE CHANGE

Twelve climate scenarios are investigated in this study (see Table 1), capturing a range of potential temperature and precipitation shifts anticipated for California. The two GCM scenarios represent a warmer, wet alternative (HadCM2 run 1, Johns et al. 1997) and a cooler, dry alternative (PCM run B06.06, Dia et al. 2001). Three different time slices from each GCM projection are considered, centered on 2020, 2060, and 2100. Uniform incremental scenarios span the range of anticipated temperature and precipitation uncertainty.

Estimated change in the historical 1922-1993 total average annual rim inflow volume of 28.2 maf in CALVIN under the various climate scenarios (Table 1) varies from a low of –25.5 % for the PCM 2080-2099 period (a loss of 7.1 maf/year of water supply) to a high of +76.5.% for the HadCM2 2080-2099 period (a gain of 21.6 maf/yr of water supply). More important perhaps, is the consistent seasonal shift in streamflow volume arising from temperature increases in all scenarios, including the cooler, dry PCM projections. The seasonal shift in timing and volume for each inflow point in CALVIN is more or less pronounced depending on the characteristics of the basin to which it is mapped, in particular the freezing elevation.

Table 1. CALVIN California Historic and Climate Perturbed Rim Inflows

Climate Scenario	Description	Change T °C	Change P %	Volume (maf/yr)	% Change Total	% Change Oct-Mar	% Change Apr-Sep
Historical	1922-1993	0	0	28.2	0	0	0
1	Uniform Incremental	1.5	0	28.6	1.1%	15.6%	-13.4%
2	"	1.5	9	32.4	14.6%	31.7%	-2.7%
3	"	3	0	28.5	0.9%	28.0%	-26.5%
4	"	3	18	36.2	28.1%	64.4%	-8.7%
5	"	5	0	27.9	-1.1%	37.1%	-39.7%
6	"	5	30	40.6	43.7%	103.8%	-17.0%
7	HadCM2[a] 2010-2039	1.4	22	38.5	36.4%	54.9%	17.6%
8	HadCM2[a] 2050-2079	2.4	32	41.3	46.4%	82.0%	10.4%
9	HadCM2[a] 2080-2099	3.3	62	49.8	76.5%	134.3%	18.1%
10	PCM[b] 2010-2039	0.5	-9	26.5	-6.2%	-6.7%	-5.7%
11	PCM[b] 2050-2079	1.5	-14	24.4	-13.6%	-3.8%	-23.5%
12	PCM[b] 2080-2099	2.4	-24	21.1	-25.5%	-14.2%	-36.9%

[a] Hadley (Johns et al., 1997); [b] Parallel Climate Model (Dai et al., 2001)

Rim inflows are the largest component of water supplies in CALVIN, followed by groundwater recharge from natural sources (7.8 maf/yr) and net local accretions (5.2 maf/yr). Work is under way to finalize estimated changes in natural deep percolation to groundwater (1.7 maf/year in CALVIN for the Central Valley) and runoff contributions to local accretions for CALVIN from precipitation changes under each climate scenario.

CALIFORNIA'S WATER DEMANDS UNDER POPULATION GROWTH

Future economic scenarios are considered for 2020, 2060, and 2100. The 2020 scenario is the baseline condition for CALVIN, developed from California Department of Water Resources (DWR) Bulletin 160-98 projections for 2020 (DWR 1998; Jenkins et al. 2001). For later years, two population trajectories are considered: a High Scenario with high population growth combined with low income growth (reduction in historic rate to 1% growth in per capita income), and a Low Scenario with lower population growth stabilizing in 2060 combined with high income growth (continuation of historic 2% growth in per capita income). These bracket the anticipated population uncertainty for the purpose of measuring climate change impacts in this study.

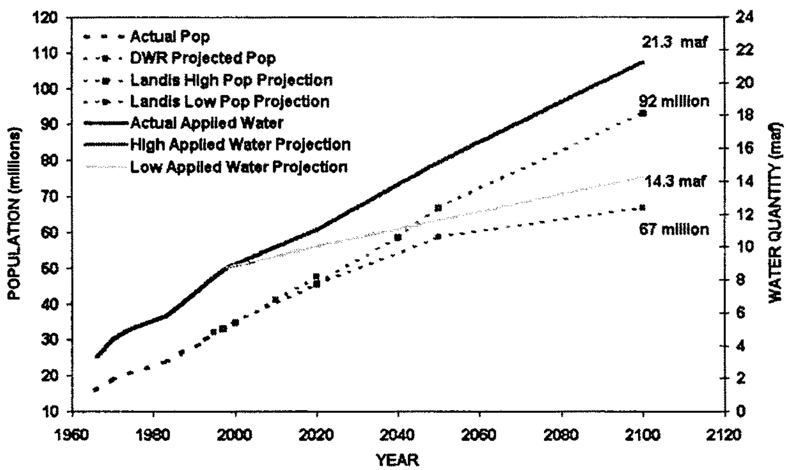

Figure 3. California Projected Population and Urban Water Use Ranges

Key parameters to estimate urban water demands across the CALVIN model for each of these periods, and the associated effects on agricultural land and production, are: total population, where that population occurs, how densely it is arranged, and income levels. Figure 3 shows the range of population projections and associated water use estimates for the 2100 period considered in this study. Table 2 compares DWR population and water use data for 1995 and projected for 2020 across the 10 hydrologic regions of California with this study's 2100 High and Low economic scenarios. Projected increases in population-driven water demands for 2100 range from 8 to 11.3 maf/yr, or 1.8 to 2.3 times current (1995) levels estimated by DWR.

Table 2. California Population and Water Use - Economic Scenarios

Hydrologic Region	M&I Urban Water Use (maf/yr) DWR 1995	DWR 2020	2100 L	2100 H	Population (millions) DWR 2020	2100 L	2100 H
Central Coast	0.3	0.4	0.7	0.9	1.9	3.2	4.7
Colorado River	0.4	0.7	0.9	1.2	1.1	3.2	5.0
N. Coast	0.2	0.2	0.3	0.3	0.8	1.3	1.8
N. Lahontan	0.0	0.1	0.0	0.0	0.1	0.1	0.1
S. Coast	4.3	5.5	7.4	8.7	24.3	31.3	42.8
San Francisco	1.3	1.3	1.9	2.2	7.0	9.7	12.3
San Joaquin	0.6	1.0	1.1	1.2	3.0	3.8	5.6
S. Lahontan	0.2	0.6	1.3	1.7	2.0	3.8	5.8
Sacramento R.	0.8	1.1	1.7	2.0	3.8	5.5	7.7
Tulare Lake	0.7	1.1	1.5	1.9	3.3	5.0	7.4
TOTAL CA	8.8	12.0	16.7	20.1	47.5	66.9	93.1

ASSESSING WATER SUPPLY IMPACTS

The list of climate-economic scenarios for CALVIN modeling that will be assessed in this study is given in Table 3. Outputs produced by CALVIN, shown in Figures 1 and 2, will be assessed for each case. These include direct water resource-related economic and reliability impacts on urban and agricultural water supplies across the system, on hydropower generation and on flood protection.

Table 3. CALVIN Climate Impacts Modeling Runs

Run	Period	Economic Scenario Population	Income	Urban Demand	Climate Scenario #	Rim Inflows (maf)
0	2020	47 million	Base	12 maf	Historical	28.2
1	2100	93 million	Low	20.1 maf	4	36.2
2	2100	67 million	High	16.7 maf	4	36.2
3	2100	93 million	Low	20.1 maf	5	27.9
4	2100	93 million	Low	20.1 maf	12	21.1
5	2100	93 million	Low	20.1 maf	9	49.8
6	2060	67 million	Low	14.3 maf	11	24.4
7	2060	67 million	Low	14.3 maf	8	41.3
8	2100	93 million	Low	20.1 maf	2	32.4
9	2100	93 million	Low	20.1 maf	3	28.5
10	2100	93 million	Low	20.1 maf	6	40.6
11	2100	93 million	Low	20.1 maf	1	28.6
12	2020	47 million	Base	12 maf	7	38.5
13	2020	47 million	Base	12maf	10	26.5

In addition, changes in facility operation and usage, and allocations across the system taken by CALVIN in balancing competing water resources management objectives under these scenarios will be examined for promising adaptations to climate change and opportunities for expanding infrastructure across the range of uncertain climatic conditions and population projections anticipated for California.

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NATURE-BASED TOURISM
ECONOMIC OPPORTUNITIES ON A FEDERAL WATER PROJECT
A CASE STUDY

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ABSTRACT

The Garrison Diversion Unit (GDU) is a multipurpose water project in North Dakota. Over the past 35 years, its purpose has been to provide economic opportunities for the state. In 2000, the Garrison Diversion Conservancy District (Garrison Diversion) initiated a plan whereby the natural resources associated with Garrison Diversion lands and waters were to be evaluated for their economic potential, using a nature-based tourism philosophy. The plan was to develop an innovative model for the 33,000-acre (13,355 hectors) Lonetree Wildlife Management Area (Lonetree) in central North Dakota, with eventual application to all the GDU lands. Garrison Diversion, working with the Bureau of Reclamation (Reclamation) and the North Dakota Game and Fish Department (Game and Fish), contracted with Fermata, Inc., Austin, Texas, which has extensive experience in ecotourism, or nature-based tourism. It was envisioned that this plan would explore opportunities for rural and nature tourism, identify natural resources that would be of interest to tourists, inventory the local infrastructure such as roads, lodging and restaurants, and make recommendations on how those resources could be used to the economic benefit of the rural communities associated with the GDU project.

Garrison Diversion worked with the local communities surrounding the LWMA, agencies, organizations and the public over a one-year period. The resulting report recommended many physical and marketing changes at the site that would increase the usage by making the area more accessible and enjoyable to the public. The suggestions included new interpretive centers and signs, hiking trails, canoe trails and development of the area's cultural and historical resources. A strong emphasis was also placed on program and web development catering to specific markets.

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Opportunities were evaluated for local businesses to expand, and the interest of local communities was determined through public meetings and individual discussions. Community leaders expressed much interest, and Garrison Diversion will be convening with interested parties to help facilitate development of business plans.

The Garrison Diversion project has identified many economic opportunities for the public and private sector to explore and expand their horizons on irrigation, municipal, rural and industrial water supply, fish and wildlife, and recreation projects for the benefit of a worldwide audience. On existing and future irrigation projects, approaching our natural resources with this broad view of the potential development can greatly supplement rural economies.

INTRODUCTION

History of the Project

The purpose of this paper is to describe the GDU Project in the state of North Dakota and the economic opportunities, which have evolved during its long history, as they relate to nature-based tourism, which is promoting a balanced approach among economics and the natural, cultural and historical resources in the area. The initial stage of the GDU was authorized in 1965 with the primary purposes being flood control, navigation, irrigation, and hydropower production. In addition, it had a large fish and wildlife mitigation and enhancement plan (146,520 acres/59,296hectors) and a substantial recreation plan (up to $13 million on 12,000 acres/4,856 hectors).

Over time, the emphasis of the plan changed toward the delivery of drinking water to rural areas, towns and cities while retaining major portions of the wildlife and recreation plans. Lands previously acquired for the 33,000-acre (13,355-hector) Lonetree re-regulating reservoir were converted into a wildlife management area along with areas along the canals and other Garrison Diversion project lands. These lands totaled approximately 70,000 acres (28,329 hectors) and were abundant with natural and cultural resources available to the public. While the wildlife opportunities were being developed and recognized, it became evident that these lands were not being fully utilizing for their economic benefits and public use potential.

In 1999, Garrison Diversion, working with Reclamation, Game and Fish, and local communities, took the initiative to explore ways to better define those potential benefits and opportunities. The initial effort was to focus on the LWMA. Later efforts would be expanded to all the GDU lands.

The objectives were to document the existing natural and cultural resources in the Lonetree area and examine ways to increase public use without compromising the wildlife management programs. The partners recognized that since development of the area, public use was not as high as it was. It was believed that a plan was needed that would not only anticipate higher use but plan for the management of the public. Most related public agencies are experienced at developing and managing natural resources but are not necessarily trained in people management. For this reason, it was decided that a plan was necessary to protect the natural and cultural resources while assuring that the public has an opportunity to enjoy the lands and resources.

Initial Assessment

In spring 2000, the three managing partners of the LWMA: Garrison Diversion, Reclamation and Game and Fish contracted with Fermata, Inc., and Ted Eubanks. The goal of the first meeting was to gain an initial assessment of the economic potential for development of a nature-based tourism program for the LWMA and other Garrison Diversion project lands. Fermata, Inc. was selected based upon research of companies in this field and strong recommendations from other resource agencies. Fermata offered more than 30 years of business and personal expertise in the area of wildlife watching and tourism development and was working on projects in the United States and throughout the world. Fermata, Inc. had also developed and trademarked Applied Site Assessment Protocol. This model has been used to determine recreational values for nature assets, to identify the market segments that would most be attracted to those resources and to define the constraints for nature tourism on other public and private lands.

The assessment was brought about by the need for Reclamation and Garrison Diversion to evaluate the public lands and how public use could be managed. The goal of this process was to prepare for expanded public use. Available statistics and national trends indicated that an increase in interest in all natural areas is on the rise and that North Dakota has a significant potential for being a destination of naturalists and birders. This concerned the partners, who were not equipped to manage public use as well as we could manage natural resources. The goal was to evaluate the potential for economic development that would benefit the people, state and natural resources of North Dakota.

In his initial assessment, Mr. Eubanks spent several days touring the area and gathering information. He reported to the three managing partners that there was potential for development of nature/experiential tourism, but it would take a great deal of work.

With this understanding in mind, the three cooperating partners agreed to pursue a larger contract with Fermata, Inc. to prepare a nature-based tourism plan for the LWMA. This contract included developing a comprehensive strategic plan that focused on nonconsumptive activities, such as wildlife viewing. In that plan, the creation of a model for future development of Garrison Diversion and other public lands would be included. The eventual need for a business plan for the project was also recognized. The contract included paying special attention to private interests in the region. One of the avenues used to reach this audience was holding public meetings. Eight meetings were held in four separate communities. The first meetings were held to explain the process, and follow-up meetings were held to discuss the report and its impacts. The expected timeframe for completion of this plan was early 2001, at a total cost of $80,000.

Resources Inventory

The first step was to determine the zone of influence for the plan/study and then conduct an inventory of the natural and cultural resources within that zone. The three partners conducted the inventory. It included gathering and delivering to the contractor, data and previously published information on all the natural wetlands, both fresh and alkali, streams and rivers, native and planted grasslands, woodland shelter belts, riparian woodlands, and the predominate wildlife associated with each habitat type. Cultural and historical sites such as Indian tepee rings and homesteads of the Europeans who settled the area were also inventoried. It was also important to understand what was available to the public in the form of transportation infrastructure, recreation sites, and facilities, food and lodging within a close proximity, and support agencies and institutions to accommodate the public needs.

The 33,000-acre (13,355-hector) area possesses a dramatic undulating terrain, forming an attractive view shed. The Sheyenne River and its headwaters define the land, carving an undulating valley through the site. Marshes have formed in the lower reaches of this valley, and numerous water birds as well as odonates (dragonflies and damselflies) and butterflies congregate in these ephemeral wetlands.

Working with Fermata, the agencies gathered the data. The data were analyzed as to their role in the study, but mostly to determine if all the necessary data were available. A majority of the requisite data was available, but there were some gaps. One of the data sets lacking on Lonetree was information on current public use, which was needed not only to recognize the current level but also to establish techniques for collecting, monitoring, and managing future public use. The most critical data came from the surrounding areas, including information on food service, accommodations support personnel, and visitation. This data shared a lack of knowledge or ways to provide information to visitors already in the state, visitation during winter, interpretation of the natural and cultural resources within Lonetree, ways to provide information to potential remote visitors, and knowledge of the potential impact of tourism on the local economy. Based upon the data collected and the experience of the contractor, it was determined that the most pressing data gaps were in the area of visitor services and support personnel. Infrastructure objectives were determined to meet minimal standards because most roads are gravel. During winter months and various other times of the year, these roads are not available for travel on a guaranteed basis.

It was important to understand the demographics of the area. During summer and fall of 2000, the partners met with a selected number of local communities to get feedback and opinions on the project and to learn whether they believe there was a need for a study of the potential for nature-based tourism. This was the first in a number of public meetings held throughout the study. The meetings were very successful. The local communities not only supported the study, but also felt that this effort might consummate into a significant economic opportunity for the declining agricultural community.

The Model

Fermata's Applied Site Assessment Protocol is a new technique that can be used on many types of water projects such as irrigation, flood control or other natural or cultural resource projects. It takes into account the typical resources in an area and also looks beyond the boundaries of public lands and considers the effects on and opportunities of atypical economic development. It also provides guidance on developing partnerships between public/private/nonprofit groups that are not typical of water projects.

Current observations on many water projects may be that of recognizing the value of either guiding or public use of hunting/fishing resources. Beyond those uses and seasons, the opportunities have not been explored. Hunting and fishing provide some economic revenue, but that is only realized during a short window of time in the summer or fall. During the other months of the year, economic benefits from other resources are not being recognized. One of the main values of a study such as this is to gain an understanding of opportunities outside the fishing and hunting season in order to develop a marketing plan that will meet the needs and provide benefits to the communities.

Additionally, Fermata recognizes that to the average person, tourism conjures the image of specific destinations and attractions, such as Las Vegas or Disneyland. This Disney view of tourism is without a doubt an important economic component in the travel and tourism market. Yet, natured-based tourists are searching for the natural, historical, and cultural heart of a region, their defining principle is authenticity. The ambition of this expanding segment of the travel and tourism market is to be immersed in a rich natural experience, and there is no need to invest in an attraction but only to preserve and manage what is already present. This idea was prescribed to in the study. (Fermata, Inc. 2001)

Results from Lonetree and Results from State Plan: Economic Opportunities

With the completion of the LWMA report, multiple areas and groups were identified as benefiting from development of a nature-tourism plan, not only in the Lonetree area but in other parts of the state. A group of 15 project stakeholders were identified as potential partners in the planning process. State, private and non-profit interests, and local communities were considered an important part of any future planning process.

The report identified zones of influence for destinations that were geographical and represented the resources that attract visitors' attention and time. The first and immediate area was the LWMA. The Fermata report stated that the LWMA area did not have the variety of resources to support a sustainable nature tourism industry. This became clear to the sponsors early in the process and thus Fermata, Inc., was authorized to look beyond the LWMA boundaries and consider the entire state of North Dakota. The report recommended building a zone in which the areas surrounding Lonetree could be developed.

The second area of influence includes the communities surrounding the wildlife management area, which have the highest vested interest in the development of nature-based tourism. Once again, these areas do not currently have the infrastructure to sustain a large-scale development, but it is believed that this prerequisite can be developed in a methodical manner that brings the most benefit to the communities and rural area.

The third area stretches out to include a variety of public lands that, combined with the LWMA, would provide a package destination for the nature tourist. The final zone was the entire state of North Dakota. The issues and observations made by Fermata, Inc. carried through with the entire state. The report stated that without the cooperation of the state, the LWMA would struggle as a destination on its own. Any development would have to be done in a graduated schedule. Changes, such as better signing of the roads and destinations, along with organization of resources and education to the areas affected are important first steps.

The resources of Lonetree, though limited, can be developed into unique experiences for visitors. The headwaters of the Sheyenne River are in the LWMA. There are several freshwater wetlands and alkali lakes, along with upland grasslands and a diverse set of habitats for species across the area that could be marketed and explored for education. The area holds several cultural and historical sites that can be developed for interpretation. There is also a diverse opportunity for businesses associated with canoeing, hiking, and bird watching. It is important that with any future development, we assure a long-term balance between the valuable natural and cultural resources and any real or perceived economic development.

The overall objective of the report, along with any development of the area, is sustainability of resources. These resources range from the land and species that currently have a habitat there, to the people who live in the surrounding rural areas and communities. Any future development or progress is to be done in stages in as simple a manner as possible. The cooperating agencies do not want to begin any development until there is a firm understanding of the outcome and pitfalls, nor do they want to mislead the private sector or spend money on an unproven benefit.

One area to be examined is how to market the resources and create an awareness to travelers, not only from the local commuting area, but also worldwide. This is an ongoing effort and, to date, that objective is far from being met.

Implementation

For the LWMA, the first goal is to develop a list of action items, which can conceivably be implemented in the near future. These action items are intended to improve public use access, information and appreciation of the LWMA and will be implemented by mutual consent of all major partners (Garrison Diversion, Reclamation, Game and Fish) and in cooperation with many different stakeholders. A list of these potential stakeholders was included in a draft implementation plan. A timeframe for developing the final plan, designing features of the plan, and implementing specific action items included in the comprehensive implementation plan has yet to be developed and will be discussed and mutually agreed upon by all the major partners. The final implementation plan will tie into a nature tourism and public use strategy that integrates the ecology of the prairie potholes with the ecology of the Great Plains.

Some of the initial plans will include working on directional signage, upgrading an existing auto tour route, and creating and upgrading the trail system around wetlands, woodlands, grasslands and riparian areas. Some of these will be hiking trails and others could be canoe trails. Other plans include improving the birding checklist and creating new wildlife and nature-type checklists, such as those for mammals, native flowers, native grasses, woodlands, wetlands, and insects, to meet the needs of the public. The report also suggested that an evaluation of the potential for developing detailed plans on a major interpretive center be considered. Year-round access is a major consideration in the northern climate due to long, cold winters. County road systems will need to be evaluated, with consideration given to improvements. While this may seem like a major investment, it may be the practical solution to assure year-round access to the area.

The draft implementation plan is still in its infancy; but by taking small steps this year, it is expected that the momentum may develop into bigger projects in the near future.

The issue of marketing is another step. The three sponsoring agencies are all government entities in some format and have a variety of authorities and abilities when working with the public. Given those restrictions, a straightforward implementation plan was developed that focuses on helping the communities and individuals with resources.

Creating an environment for the communities to work together is needed. Traditionally, the rural communities surrounding Lonetree have competitively worked against each other, ranging from high school sports to attracting businesses. They will now work together in order to develop a sustainable tourism industry.

Providing educational forums, focusing on community and individual needs will also be one of the directions taken. Classes such as plant and bird identification, marketing, web based development and customer service are tools that will be offered to the residents who are interested in developing private businesses to work with the development plans on the wildlife management area.

Local communities have already started creating a base of knowledge. The small rural community of Anamoose started a birding club and is actively seeking grants and funding for business development. The community of Harvey has several single-owner businesses that will benefit from increased traffic. It also has a variety of hunting lodges that are expanding and pursuing natural, cultural and historical tourism opportunities. Several rural families are working on providing rural tourism opportunities and will benefit from development of the region and the LWMA.

The expansion of nature-based tourism at LWMA will not create large-scale business development or Disney lands in North Dakota. It will provide sustainability of resources for existing and future generations.

Applications

Garrison Diversion has learned a great deal over the past two years in an area that, for water project sponsors is outside the normal field of vision. North Dakota's resources can reap great benefits for its people, but it will take a major effort. Probably the most important idea brought forward is that no matter what is envisioned, we will be setting ourselves and others up for failure if we believe the agencies can be successful by themselves. The local communities need to take the lead in this effort and any detailed planning needs to be developed by the communities with the agencies providing guidance and funding only if necessary.

This concept has potential for any water project, and we encourage others to expand their horizons and look to ideas such as this to supplement the future of agriculture. Garrison Diversion will continue to search for new ideas and ways to implement the project and share the information with the water community.

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RECLAIMED WATER—A NEW IRRIGATION SOURCE FOR CITRUS IRRIGATION IN FLORIDA

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ABSTRACT

Many communities in Florida considered wastewater to be a disposal problem before 1980. When it was proposed to convert wastewater to reclaimed water for crop irrigation, citrus growers refused to accept the water because of fears of heavy metals, flooding, or disease. Ultimately, several reclaimed water projects were started, and Water Conserv II west of Orlando has become one of the world's largest agricultural reclaimed water irrigation projects of its type. This project provides irrigation for more than 4300 acres of agricultural crops and two golf courses. The water is chlorinated, is odorless and colorless, and has been used successfully for crop irrigation for 15 years. Excess reclaimed water is discharged to rapid infiltration basins (RIBs). The water meets drinking water standards for a number of compounds including nitrate, sulfate, Na, Cl, Cu, Zn, Se, and Ag. Initial fears that reclaimed water would cause problems were unfounded. In the sandy well-drained soil, excessively high irrigation rates with reclaimed water (100 inches/year) promoted excellent tree growth. Because of a recent severe drought in Florida, attitudes toward reclaimed water have changed. Once believed to be a disposal problem, reclaimed water is now considered to be a viable resource that can meet irrigation demands. Average statewide reuse flow rates have increased by 116% in ten years.

INTRODUCTION

As water shortages become more common, competition for water among various sectors becomes more acute. Increasing urban growth, along with agricultural and industrial needs, lead to greater competition for limited water resources. Interest has increased in developing new water resources to meet the greater demand. Florida has relied heavily on groundwater pumping and concerns have arisen regarding declining aquifer levels. This has led to serious discussions on developing alternate water supplies such as desalination, aquifer storage and recovery, and reclaimed water. Reclaimed water use has evolved in an interesting way in Florida, and the objective of this paper is to briefly discuss one project, Water Conserv II, which illustrates how attitudes toward reclaimed water can change when water supplies get short.

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Background

Disposal of wastewater is a problem for many urban areas. In the 1980s, disposal of wastewater effluent was considered to be a growing problem, primarily because of environmental concerns about lake degradation. Urban wastewater disposal had commonly been handled by treating the wastewater to a certain level and then disposing of it in the most convenient or cheapest manner. Usually, this meant discharging the water into a nearby river or lake, spraying it onto a field, or loading it into a percolation pond. Disposal was the primary consideration since the amount of wastewater continued to increase as an unavoidable consequence of population growth. As wastewater volume increased, concerns were raised about the effects on discharge sites. This led to consideration of alternate uses such as irrigation. While the idea of converting wastewater to reclaimed water for irrigation was not a new one, using reclaimed water for irrigation was a relatively small-scale activity in Florida before 1980. Eventually, increasing disposal problems led to several large Florida projects set up to reclaim water from wastewater treatment plants for irrigation of agricultural crops or landscape vegetation. Examples include projects in Tallahassee, St. Petersburg, and the Water Conserv II project of Orlando and Orange County (Allhands et al., 1995; Parnell, 1988; Roberts and Vidak, 1994).

Before 1987, Orlando and Orange County each discharged treated wastewater from their treatment plants into Shingle Creek that flows into Lake Tohopekaliga, a lake with high recreational value. Concerns were raised over the potential eutrophication of the lake due to nutrient loading. Thus, the U. S. Environmental Protection Agency required Orlando and Orange County to develop an alternative plan for the disposal of the wastewater they were discharging into Shingle Creek. Several plans for the effluent were proposed, such as: 1) building a pipeline approximately 68 miles long to carry the effluent to the Atlantic Ocean, 2) establishing a "Groundwater Conservation Program" which would inject reclaimed water meeting primary and secondary drinking water standards into the Floridan aquifer, 3) purchasing large tracts of land for rapid infiltration basins (RIBs), 4) increasing the treatment level to convert the wastewater to meet reclaimed water standards and have growers apply it to their citrus groves, and 5) injecting the wastewater into deep wells over 3000 feet deep using high pressure. None of the plans by themselves proved to be acceptable for a variety of reasons. Following further review, a combination of citrus irrigation and RIBs was determined to be effective. This combination was selected and named Water Conserv II.

Citrus grove owners initially rejected the plan because of concerns about possible heavy metal contamination, potential virus or disease problems, flooding, and lack of flexibility in water application during periods of high rainfall. Growers also raised concerns over psychological aspects and feared that there might be a degradation of fruit quality from trees irrigated with reclaimed water. Ultimately, Orlando, Orange County, and the growers developed a plan that provided for the establishment of reclaimed water standards, regular monitoring of the water, greater grower flexibility on timing of use, and research on the effects of the reclaimed water on citrus tree performance. In addition to applying the reclaimed water to citrus groves, the project also included the purchase of land for Rapid Infiltration Basins (RIBs) or percolation ponds for disposal of excess water. Water Conserv II has since become the largest reclaimed water agricultural irrigation projects of its type in the world and was the first project in Florida to be permitted to irrigate crops for human consumption with this water (McMahon et al., 1989).

At present, the reclaimed water is applied primarily to citrus, but it is also used for irrigation of several other crops. At the Orange County National Golf Center and West Orange Country Club, golf courses with a total of 45 holes have RIB sites incorporated into them and use the reclaimed water for irrigation. At present, over 4,300 acres of citrus, 12 nurseries and tree farms, and two landfills use this reclaimed water for irrigation. One hundred acres of willow is irrigated in a "browse farm" to provide feed for the Walt Disney World Animal Kingdom theme park. New pipeline has been installed to extend the reclaimed water to additional areas.

Water Treatment, Distribution, and Quality Standards: Two treatment facilities receive the wastewater and process it to meet reclaimed water standards. These facilities were upgraded to meet the stricter water quality standards. In addition to the normal treatment, advanced secondary treatment capability was added to meet high-level disinfection standards. This involves coagulation and filtration facilities similar to potable water treatment plants. Pump stations at both reclamation facilities transmit the reclaimed water through a pipeline about 21 miles long to a distribution center in western Orange County. The distribution center is located in a citrus production area with deep, well-drained, sandy soils. The center can store up to 20 million gallons of water in four large covered concrete tanks. A computerized control system monitors the distribution of reclaimed water continuously. Water is pumped from the distribution center to either grower's fields or to RIBs. Under current conditions, about 60% of the water goes to citrus groves and the remaining 40% goes to the RIBs. This project presently delivers about 30 million gallons per day (mgd). Permitted average daily flow capacity is 44 mgd with ultimate average daily flow capacity of 50 mgd with peaks to 75 mgd. The source of the wastewater is primarily restaurants, motels, and tourist attractions in western Orlando and Orange County. There is very little factory or heavy industry input into the incoming wastewater.

During freezes, there is a high demand for irrigation water, and the reclaimed water provided to citrus groves is supplemented with well water for frost protection in order to meet that demand. Most groves are irrigated with undertree microsprinkler irrigation which can provide some frost protection (Parsons et al., 1982; 1991) as well as normal irrigation for citrus trees.

Under the current contract, growers agreed to accept either 25 or 50 inches of water per acre per year for 20 years. Water is delivered at no charge to the edge of the grower's property at a minimum pressure of 40 psi. Growers can terminate their participation in the 20-year agreement at any time through a buy-out clause by repaying the city and county $3600/acre the first year with the repayment decreasing by 5% each following year. To date (15 years into the project), no grower has chosen to opt out of his contract. This indicates grower satisfaction with the reclaimed water.

The University of Florida established water quality guidelines for citrus trees. They are rigorous and apply only to the Water Conserv II project. The maximum average concentration limits (MACLs) for some elements such as sodium, chloride, barium, chromium, copper, selenium, silver, sulfate, and zinc are more stringent than Florida drinking water standards (Parsons et al., 2001). Drinking water standards, Conserv II standards, and typical values are presented in Table 1. The treatment facilities have been required to meet the drinking water standard of 10 mg/L for nitrate nitrogen. In terms of crop mineral nutrition, meeting the nitrate drinking water standard is a disadvantage because this reduces nitrogen supplied to the tree.

The water is chlorinated which provides virtually complete removal of viruses and bacteria. The water is colorless and odorless. Florida regulations presently state that only indirect contact methods such as drip, subsurface, or ridge and furrow irrigation can be used to irrigate the "salad crops." Any type of irrigation method can be used to irrigate tobacco, citrus, or other crops that will be "peeled, skinned, cooked, or thermally processed" before human consumption (York et al., 2000). Most of the oranges in Florida are processed for juice, but some do go to the fresh market.

<u>Reclaimed Water Research at Water Conserv II:</u> Growers have now used Water Conserv II reclaimed water successfully for over 15 years. At the request of growers, studies were initiated to determine the effects of this reclaimed water on citrus trees. The first studies were conducted in commercial groves to make comparisons between reclaimed and well water (Zekri and Koo, 1990). In these plantings, growers using reclaimed water commonly used more water than those using well water. Hence, soil water content was usually higher in the groves using reclaimed water. Appearance of trees irrigated with reclaimed water was usually better than the trees irrigated with well water (Koo and Zekri, 1989; Wheaton et al., 1996).

Table 1. Florida drinking water standards, typical well water values, Conserv II maximum average concentration limits (MACL), and typical values in Conserv II water. All values are in mg/L except for pH, EC, and SAR.

	Drinking water Max. Contam. Level (mg/L)	Well water typical values (mg/L)	Conserv II water MACL (mg/L)	Conserv II water typical values (mg/L)
Arsenic	0.05	–	0.10	<0.005
Barium	2	–	1	<0.01
Beryllium	0.004	–	0.1	<0.003
Bicarbonate	–	–	200	105
Boron	–	0.02	1.0	<0.25
Cadmium	0.005	–	0.01	<0.002
Calcium	–	39	200	42
Chloride	250	15	100	75-81
Chromium	0.1	–	0.01	<0.005
Copper	1	0.03	0.2	0.002-0.05
EC (umhos)	781	360	1100	720
Iron	0.3	0.02	5.0	0.01-0.37
Lead	0.015	–	0.1	<0.003
Magnesium	–	16	25	8.5
Manganese	0.05	0.01	0.20	0.006-0.042
Mercury	0.002	–	0.01	<0.0002
Nickel	0.1	–	0.2	0.01
Nitrate-N	10	3	10	6.1-7
pH	6.5-8.5	7.8	6.5-8.4	7.1-7.2
Phosphorous	–	0.01	10	1.1
Potassium	–	6	30	11.5
SAR	–	0.6	–	2.5
Selenium	0.05	–	0.02	<0.002
Silver	0.1	–	0.05	<0.003
Sodium	160	18	70	50-70
Sulfate	250	23	100	29-55
Zinc	5	0.02	1.0	0.04-0.06

Since disposal of wastewater was of concern early in this project, it was important to determine if citrus could tolerate high application rates of reclaimed water. In research plantings, very high rates were applied to two citrus varieties, 'Hamlin' orange and 'Orlando' tangelo trees on four rootstocks. In addition to normal rainfall of approximately 48 inches/year, these trees were irrigated with rates of up to 100 inches/year (~2 inches/week). Application of 2 inches/week of reclaimed water in a 20-acre experimental planting significantly increased canopy volume and fruit yield compared to 0.3 inch/week of well and reclaimed water applications (Parsons et al., 2001). Because of the scheduling method used, the lower irrigation rate did not provide adequate water for optimum tree growth and production. The excessive irrigation diluted the juice soluble solids somewhat, but because of the greater total fruit production, total soluble solids per acre were increased at the 100-inch irrigation rate (Parsons et al., 2001).

Weed growth was rank because of the high reclaimed water irrigation rate (Parsons and Wheaton, 1992; Zekri and Koo, 1993). Such growth has been controlled with proper herbicide use and mowing.

Irrigation with reclaimed water increased soil P, Ca, Na, and pH (Parsons and Wheaton, 1992; Zekri and Koo, 1993). Most fibrous roots are located in the top three feet, and much of the Na in the soil was leached below this depth. This reclaimed water supplies all the P, Ca, and B required by trees in central Florida soils. While levels of some elements have increased in the soil, they have not built up over the years (Zekri and Koo, 1993). This lack of buildup is attributed to low soil organic matter, low cation exchange capacity, and leaching rainfall. Leaf P and Ca levels were also increased. Leaf levels of Na, Cl, and B were elevated but remained below toxic levels.

Because the nitrate-N level is low (less than 10 mg/L), the amount of nitrogen extracted from this reclaimed water is unknown. In a small grower test, young trees that were given no fertilizer and irrigated only with Water Conserv II reclaimed water took 2 to 5 years to show nitrogen deficiency symptoms and yield declines (Ross, 1993, pers. comm.). Other work in the Vero Beach area showed that reclaimed water alone did not provide adequate nutrition for young grapefruit tree growth (Maurer and Davies, 1993). Preliminary data showed that high application rates of reclaimed water maintained yield for one year, but yields declined in the second year without additional fertilizer application (Wheaton et al., 1996).

Have Attitudes Changed? Attitudes in Florida toward reclaimed water have changed since the mid-1980s. Once considered to be an urban disposal problem with no beneficial use, treated wastewater effluent was discharged into a water body or spray field as a low cost method of disposal. Environmental concerns ended such disposal. Growers were initially opposed to the Water Conserv II because of fears about salts, heavy metals, odors, contaminants, flooding, disease, and potential tree damage. Once water quality standards were established and the initial fears of flooding, disease, and tree damage were proved to be unjustified, research went on to show that reclaimed water had no adverse effects. Sufficient flexibility was also given to growers so they could acceptably manage their water in a region that has quite variable rainfall.

The benefits of this project are now apparent. Orlando and Orange County benefit by meeting the mandate for zero discharge of effluent into surface waters.

Withdrawal from the Floridan aquifer for irrigation has been reduced. Recharge of this aquifer has been accelerated due to the application of reclaimed water to the RIB sites. Because reclaimed water has been used safely and effectively, some groups and agencies are promoting the use of reclaimed water as a way to make up for water shortages. A serious drought in central Florida lasting from 1999 through spring, 2002 has greatly increased interest in water reuse. Statewide reuse flow increased by 116% to 575 mgd from 1990 to 2000. By 2000, agricultural reclaimed water irrigation reached 14,414 acres of edible crops and 20,868 acres of other crops (Fla. Dept. Environ. Protec., 2001).

Reclaimed water is no longer considered to be a disposal problem, but a limited resource of value. Quality of the water, along with supply and demand forces, will ultimately determine how much reclaimed water is used for irrigation or other purposes. Some growers still have concerns that there is a psychological stigma attached to reclaimed water that may damage the market quality of Florida citrus that has been built up over the years. Nevertheless, initial opposition to use of reclaimed water has decreased as demand for the water has increased. In the case of Water Conserv II, reclaimed water has been used in a productive and environmentally safe manner in a successful cooperative effort between growers and government agencies that has solved problems for both and proven the value of reclaimed water as a resource.

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APPLICATION OF AUTOMATION TECHNOLOGY FOR ENSURING STREAM FLOWS FOR ENDANGERED SPECIES IN THE LEMHI RIVER

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ABSTRACT

The paper work involved in establishing water banks and water leasing programs seldom consider the difficulty in measuring and controlling precise amounts of water. The relatively small amounts of water transferred through water banks or leased for instream use programs further increases the difficulty of shepherding the water through the river system. The proper application of automation technology can be a valuable tool for the watermaster in administering the resource.

INTRODUCTION

Background

The Lemhi River in central Idaho is an important spawning site for endangered and threatened chinook salmon and steelhead. These two anadromous fish species are hatched in the Lemhi's headwaters, swim downstream to the Pacific Ocean as smolts and then, as adults, return to their birthplace to spawn after 2 or 3 years. The lower 60 miles of the Lemhi River also provides water for irrigating 40,000 irrigated acres of agricultural lands. Near the mouth of the river the irrigation diversions significantly impact river flows and affect fish passage.

The 2001 Idaho State Legislature established the Lemhi Basin Water Bank, a legal and administrative tool that permits irrigators to lease a portion of their irrigation water right for other beneficial uses, particularly for maintaining in-stream flows to help facilitate migration of anadromous fish. Automation technology was used to regulate river flows and to assist the water master in ensuring that water leased for stream flows remains in the river.

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Idaho water law, like in most western states, is based on the Doctrine of Prior Appropriation. By law, the date on which water was originally diverted from a stream and put to □beneficial use□" determines the priority of that right over other rights on the same stream. Under this system, an irrigator may divert water at a rate up to the limit of his water right, provided that there is sufficient water in the stream to fill the rights of other appropriators senior to his in time. State statutes determine what types of activities constitute □beneficial use□ and permitted diversion rates and quantities are based on the nature of use and the point of use. In Idaho river basins, in-stream flows have only recently been considered a □beneficial use□ and water rights for in-stream flows are junior to most of the other water rights.

Most river basins in Idaho have a watermaster who oversees water rights and assures that water is used according to the law. Many of the senior water rights on the Lemhi River are irrigation diversions on the lower eight miles of the Lemhi River. The watermaster assures that adequate stream flows reach these senior water rights. This situation usually ensures adequate fishery flows above river mile six. The lower six miles of the Lemhi River have historically experienced low water conditions due to drought and irrigation diversions upstream of this reach. Recently water users, watermaster and the model water shed have cooperated to establish more fishery flows in the lower portion of the Lemhi River.

Many issues involving competing uses came to light in the Pacific Northwest in the summer of 2001 as much of the region experienced drought. The establishment of the Lemhi Basin Water Bank provided a mechanism to help balance competing needs of irrigation and riparian habitat. The proper operation of the Water Bank required improved water measurement and control to ensure that both irrigation and in-stream demands were handled in a fair manner within the statutes of the Water Bank.

Operation of Idaho's District 74 Water Bank

Idaho has water banks established in several river basins. Through the water bank, a water right holder may lease a portion of his water to other water users. In nearly all cases in Idaho, leased water comes from reservoir storage. The Lemhi River contains no storage facilities so the water bank was established to deal with diversions of natural river flow rights. The water bank is operated by a Local Rental Committee with procedures approved by the Idaho Department of Water Resources. The Committee is responsible to facilitate the entire rental process.

The Committee accepts applications from parties desiring to place water in the bank and from parties wishing to lease water from the bank. The Committee then considers:

A. Whether the rental would cause injury to other water rights;
B. Whether the rental would constitute an enlargement of the rented water right.
C. Whether the rented water would be put to a beneficial use;
D. Whether the water supply available from applicable rights in the bank is sufficient for the use intended;
E. Whether the rental is in the local public interest; and
F. Other factors determined to be appropriate by the Committee.

The Committee also works with the lessor and lessee to facilitate an agreement if the asking rental rate and the offered rental rate are different. The Lemhi River District 74 Water Master is part of the Committee and serves as the manager of the water bank and provides valuable information in the logistics of delivering rented water to the appropriate point of use. The Water Master also has field responsibilities to deliver and verify the use of rental water.

Water Leased for Instream Flows for the 2001 Season

In 1995 the National Marine Fisheries Service (NMFS) issued a Biological Opinion for endangered Snake and Columbia River Chinook Salmon. The Biological Opinion required the Bureau of Reclamation to annually provide 427,000 acre-feet of water to augment stream flows in the Lower Snake and Columbia Rivers to aid migration of ocean-bound smolts from the Snake River Basin. In most years Reclamation provided a significant portion of this amount from storage water rentals from water banks on the Payette and upper Snake River basins. Due to drought conditions in 2001 there was little storage water available for lease in the Payette and Upper Snake River water banks and Reclamation fell short of the 427,000 ac-ft flow augmentation target.

Through the recently established Lemhi Basin Water Bank, Reclamation was able to lease 22 cfs from the Lemhi Water Bank for maintaining stream flows in the lower 6 miles of the Lemhi River from July 1st through the end of the season. The water not only enhanced fishery flows in the lower Lemhi but the leased water also flowed through the Salmon River and Lower Snake River to help meet part of Reclamation's flow requirements from the Snake River Basin. The majority of the water was rented from water users in the lower portion of the Lemhi Basin. These water rights were generally senior rights and were anticipated to have water available throughout the season.

The Committee evaluated the water rights being leased along with the other required diversions in this section of river and made a determination that the entire water right could be leased, not just the consumptive use portion.

Difficulties With Delivery of the Leased Flows

The point of delivery of the leased flows was established as immediately downstream of the L-6 diversion, located 6 miles upstream from the mouth of the Lemhi River. About 1/4 mile downstream from L-6 is an active USGS stream gage. This stream gage collects flow data every 15 minutes and relays this data by satellite to an office in Boise, Idaho where it is available over the Internet. With stream flows being a critical issue the National Marine Fisheries Service, Idaho Fish and Game and other interested parties check the flows on the web-site often.

To assure that the leased flows reached the critical river reach, the water master was charged with ensuring that the flows below L-6 did not fall below 22 cfs. During snow melt, runoff flows reaching the river fluctuate daily due to the close proximity to the high mountains and large diurnal temperature swings. These diurnal changes result in a continually changing river stage and can result in flows downstream from L-6 dropping below 22 cu-ft/sec. The water master could see that this would quickly become a frustrating situation which would consume a great deal of time and attention. If low water conditions were to result in harm to any endangered species in the river, NMFS could declare a "take" and implement fines or other legal action which would further complicate and frustrate the situation. The water master had even considered parking a camp trailer at the site so the water level could be checked throughout the night.

Automation Ensures That Leased Flows Remain Instream

Reclamation worked with the water master to develop the means to use inexpensive solar-powered canal automation technology to regulate flows in the critical river reach below the L-6 diversion dam. The L-6 diversion dam spans the river just downstream of the L-6 Canal headgates. The diversion was constructed in 1996 using inflatable bladder weirs to raise the water surface as well as provide fish passage. Retrofitting the inflatable weirs with new automation was not considered feasible. Time and funding for the project were limited. The dam facility, designed to maintain a constant upstream water level, was equipped with an automatic level control system. This system was not easily adapted to regulating downstream flows.

Reclamation worked with the Idaho Department of Water Resources and the water master in making the determination to automate the L-6 turnout gate instead of the diversion dam. Reclamation agreed to supply the design and technical installation and the State of Idaho agreed to pay for the cost of the automation components ($6,000). Many of the water rights leased by Reclamation came from the L-6 canal. However, the water users that did not lease their water rights still required diversions from the river. The automation would be designed to ensure a minimum of 22 cfs would constantly pass the diversion structure during the flow augmentation period. Any temporary shortages would occur in the L-6 canal.

The decision to automate the L-6 turnout gate came in June and the split season leasing was to begin on July 1. This left very little time to complete the project design, purchase the necessary components, develop the programming and install the equipment. Staff resources from the Reclamation's Pacific Northwest Regional Office and the Provo Area Office were enlisted to complete the job.

The L-6 canal headworks consisted of two manual screw-operated slide gates and is located on the upstream end of the L-6 diversion dam. Canal diversions are periodically measured with a current meter as it does not have a permanent measurement structure. For the project, river flows downstream of the L-6 diversion dam would provide feedback for making adjustments to one of the canal gates. The weirs at the dam were to be fixed at a level which would provide adequate head to operate the canal as well as provide water past the dam for fish passage. A water level sensor would be placed in a pool just downstream of the dam. This level could then be correlated with flows measured at the USGS gage further downstream. Limits on the automated canal gate opening would prevent too much water from being diverted into the canal.

The automation worked as designed and ensured that the water leased for the stream stayed in the stream. Figure 1 contains a plot of the flows during the lease period at the USGS Lemhi River gage located 1/4 mile downstream from the automation. The plot also compares the flows from July 1st through October 1st for 1999 and 2001. The data shows that the flows did not drop below 22 cfs at any time during the 2001 lease period.

The project utilized mostly readily available, off-the-shelf components which were obtained within a couple of weeks. Key components included a 1/15th horsepower 12-volt D.C. motor (Figure 2), a 30-watt solar panel with voltage regulator, a programmable logic controller, a cell phone, a modem with synthesized voice communications (Figure 3) and gate position indicator (Figure 4). A complete list of the components and their functions are included in Appendix A. The control program was developed while parts were being ordered and assembled. The Reclamation crew and the water master completed the field installation in three days. Following installation a field tour of the newly installed automation was conducted for local water users and government agencies (Figure 5).

The water master now has a valuable tool to control the instream flows. In addition to providing 24-hour river regulation, the new equipment and installation allows the water master to check on the diversion remotely and make diversion adjustments as necessary. Figure 6 shows the L-6 Diversion running in automatic mode ensuring that the leased water (22 cfs) remains in the river and that the pools for fish passage are operating in 1-foot steps. After operating the site during the season, the water master has now identified other sites where automation might assist in resolving other water management problems.

Summary

The water leased from the Water Bank during the 2001 irrigation season was leased by the federal government to maintain adequate instream flows for anadromous fish. Water measurement sites and flow monitoring standards were established to ensure that the proper in-stream flows were maintained. Low-cost automation technology was used to regulate a diversion headgate just upstream of a critical river reach. [The use of off-the-shelf components enabled the project to be designed and completed in less than a month.] The control logic of an existing diversion structure was modified to provide better control of in-stream flows and still regulate irrigation diversions. The technology also allowed water managers to remotely monitor and control the facilities. This resulted in more precise control of the water.

Results from operations during the 2001 irrigation season illustrate the effects of utilizing improved measurement and control technologies to regulate river flows and irrigation diversions. Relatively simple technologies can be an effective tool for managing a complex river situation with competing water uses.

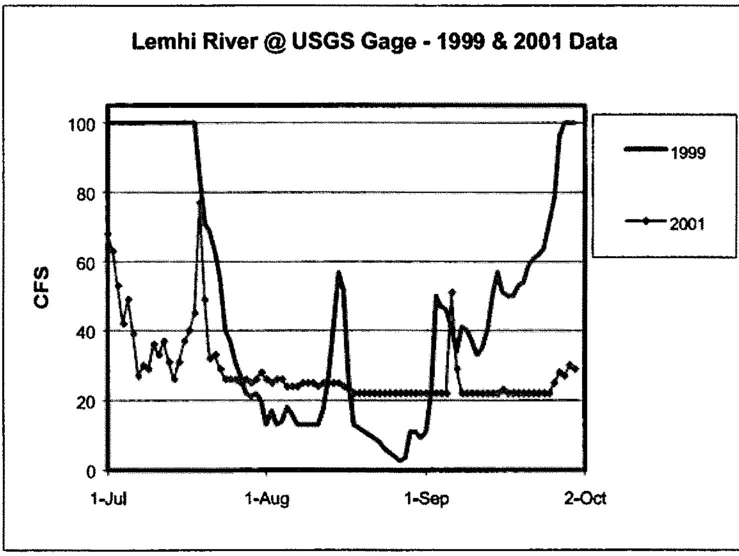

Figure 1 The 1999 flow line shows examples of low flows that occurred before the automation project. The 2001 flow line illustrates the effectiveness of the installed automation in preventing extreme low flow conditions from occurring in the Lemhi River.

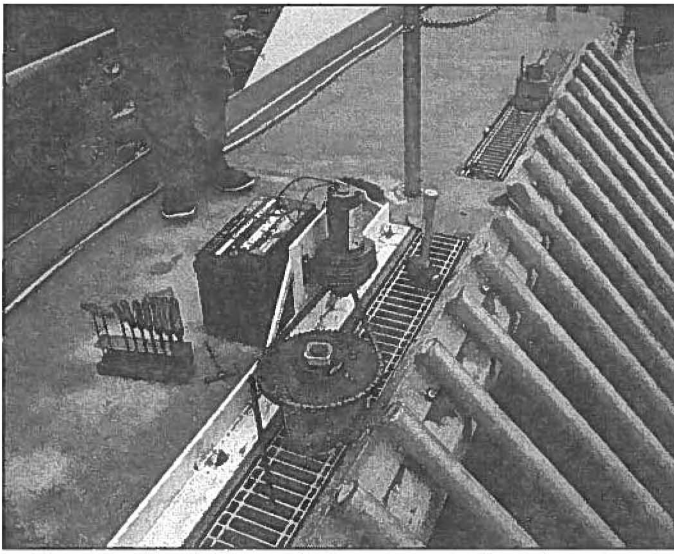

Figure 2. The 1/15th h.p. - D.C. motor controller is being tested.

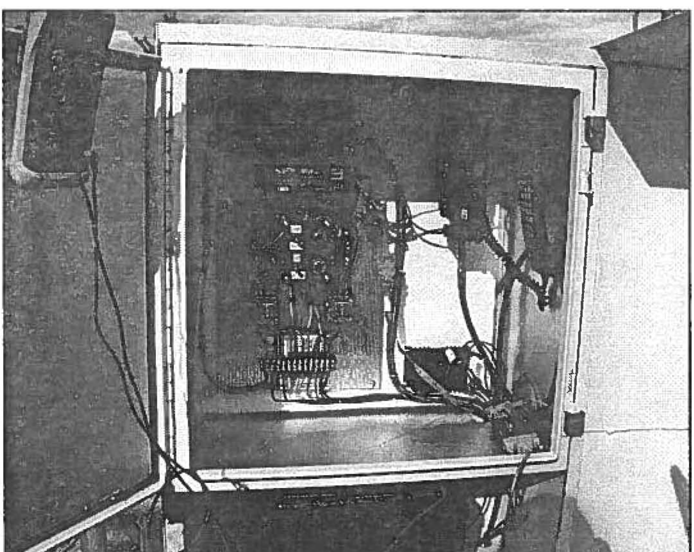

Figure 3. This view shows the control box which includes a cell phone, CR-10X Data Logger, voltage regulator, signal control, and battery.

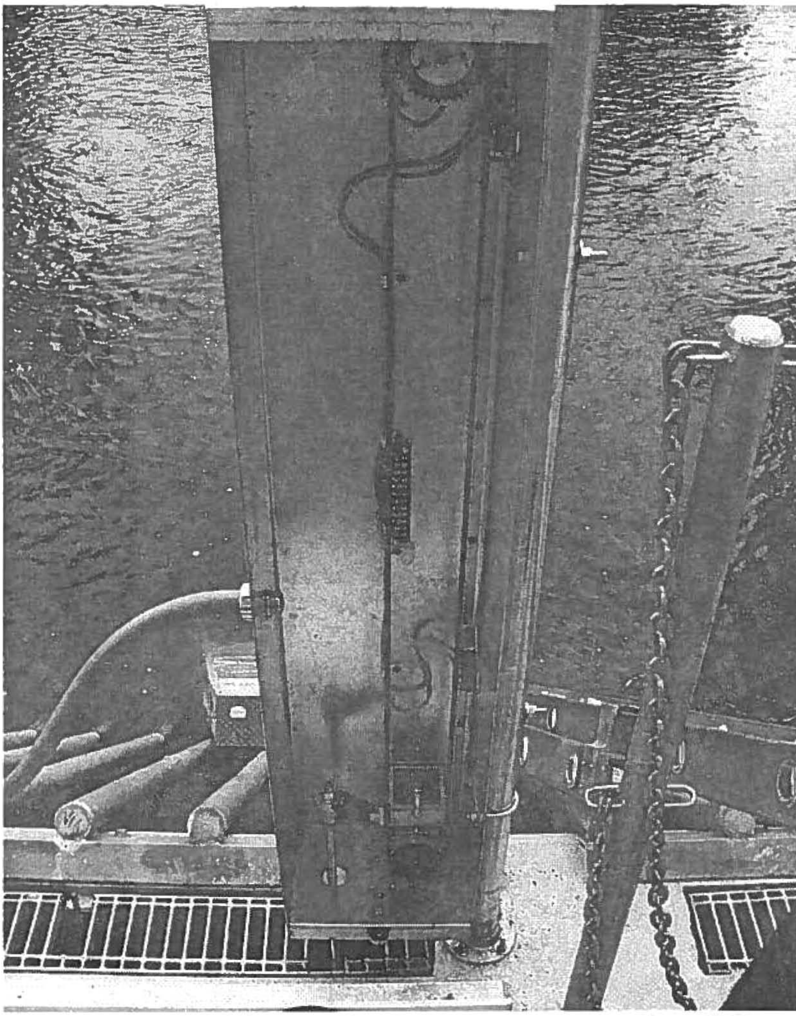

Figure 4. The gate position indicator contains limit switches as backup protection for the equipment.

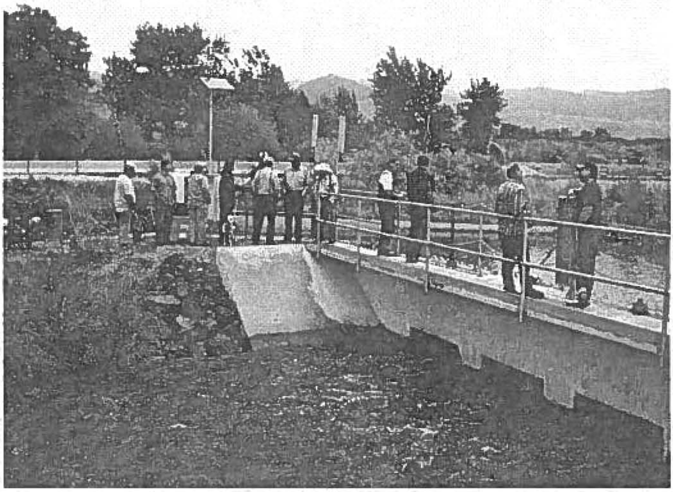

Figure 5. On July 12, 2001 a field demonstration of the newly installed automation was given to water users and government agencies.

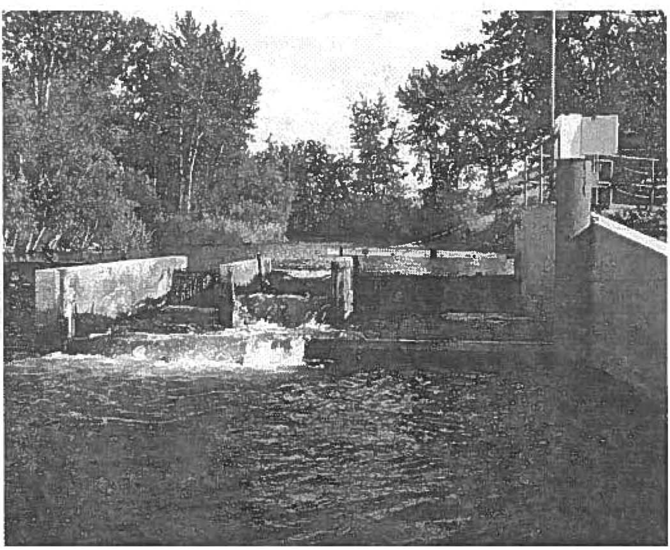

Figure 6. The L-6 Diversion running in automatic mode ensuring that the leased water (22 cfs) remains in the river. Note also the pools for fish passage.

APPENDIX A

The following list reflects the components and material that were used to complete the automation at the L-6 diversion on the Lemhi River, Salmon Idaho.

Component:	**Supplier:**	**Purpose:**
CR10X Data Logger	Campbell Scientific Inc.	Data storage/control
Com 300 Voice Modem	Campbell Scientific Inc.	Voice and data access
JL-2 Voltage Regulator	Stevens Water	Battery protection
20 Watt Solar Panel #MSX-30LT	Atlantic Solar Products	Charges battery
31-MHD Battery	Interstate Batteries	100 Ah 12 volt
ASP-962 Yagi Antenna	Decibel Products	8dBd Yagi
Analog Bag Cell Phone	Motorola	Telemetry link
Data Link # S1936C	Motorola	Cellular interface
12VDC Gearmotor #1L472	Grainger	Drive motor for gate 20RPM
Drive Sprockets #40BS12 & 40B60	Kaman Bearings	Provides 5:1 ratio
Nema Enclosure #N12242008	Automation Direct	Weather Tight
Water Level Sensor	USBR Provo Area Office	Measures level
Gate Position Sensor	USBR Provo Area Office	Gate position data
Control Panel	USBR Provo Area Office	Manual/Automatic
PC208W 3.3	Campbell Scientific Inc.	Support software

The components and suppliers that were used to complete this project are by no means the only suppliers and components available, as many companies supply comparable products.

SIMULATING PESTICIDE LEACHING WITH GLEAMS

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ABSTRACT

Ninety-six percent of the corn produced in the United States has pesticides applied to it, with atrazine and metolachlor being the most common herbicides used for weed control. Previous research has shown that there is a rapid movement of pesticides through many coastal plain soils after rainfall or irrigation. Numerous models have been developed to simulate pesticide movement through the soil. The GLEAMS (Groundwater Loading Effects of Agricultural Management Systems) model was chosen for this study to simulate pesticide movement through the root zone. A precipitation model was developed to generate a typical precipitation year for input into GLEAMS. The GLEAMS model was calibrated with data acquired from pesticide and nutrient leaching studies at the University of Delaware Research and Education Center in Georgetown, Delaware. The simulated rainfall data was inputed into the calibrated GLEAMS model to develop base levels of atrazine and metolachlor pesticide leaching. Fifteen additional scenarios were developed using 2-, 10-, and 100-year return period rainfall amounts at 3, 7, 15, 30 and 90 days after pesticide application. The results of the simulation reveal the potential for excessive leaching of pesticides after major rainfall events on sandy loam soils typical of southern Delaware. All storm events within thirty days of application indicate a leaching of greater than two percent of the applied atrazine and greater than one percent of the applied metolachlor. A 100-year return period storm three days after application leached 5.89 and 3.44% of the applied atrazine and metolachlor, respectively. The model showed that by ninety days after application the pesticides had degraded to approximately 10% of their original levels, and so had minimal leaching losses. However, all scenarios produced leaching amounts that were considerably larger than these values if there was not a major storm. These base values were 0.16% of the applied for atrazine and 0.01% of the applied for metolachlor.

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INTRODUCTION

In Delaware, there are approximately 238,000 hectares in agricultural production with 123,300 hectares in Sussex County (USDA, 2001). Corn production represents 62,700 and 40,000 hectares in Delaware and Sussex County, respectively (USDA, 2001). These soils in Sussex County are typical of the soils of the coastal plain region of the Eastern United States. National surveys have indicated that pesticides are applied to 96 percent of all corn grown to aid in weed and insect control (Farm Chemicals, 1993). Nationally, the two most widely used pesticides are atrazine [2-chloro-4-ethyllamino-6-isopropylamino-s-triazine] and metolachlor [2-chloro-N-(2-ethyl-6-methyl-phenyl)-N(2-methoxy-1-methylethyl) acetamide] which are used on 69 and 30 percent of the corn producing fields, respectively.

The recommended application rates for sandy loam to loamy sand soils, typical of Sussex County, are 2.24 kg/ha active ingredient for atrazine and 1.68 kg/ha active ingredient for metolachlor (C&P Press, 1993). Based on these rates and the estimated pesticide usage, 24,725 kg of atrazine and 10,750 kg of metolachlor are applied to Sussex County farmland each year. Beneath the surface of Sussex County is an unconfined aquifer which supplies most of the domestic water for the county. In the Nanticoke watershed, which encompasses approximately 50% of Sussex County, the depth to the water table ranges from 0 to 5.5 m with an average depth of 1.8 m (Boggess and Adams, 1964).

Current United States Environmental Protection Agency standards for drinking water permit a maximum contaminate level for atrazine of 3µg/L for lifetime consumption (Pontius, 1992). Metolachlor has a health advisory level of 100µg/L (EPA, 1989). Using the estimated application rate for atrazine and an aquifer porosity of 30 percent (Fetter, 1994), it would only take the leaching of 0.4% of the applied atrazine to contaminate the top meter of the aquifer under the field to which it was applied. In areas of concentrated corn production, the domestic wells in the area could potentially be contaminated with high levels of these pesticides in the water.

Climatic factors are a major component in the movement of pesticides through the soil and into the groundwater. A better comprehension of the influences of rainfall events will help in understanding the transport of pesticides. This will allow for proper planning of applications and prompt action to prevent or limit contamination from any human-made or natural occurrence. With the outputs from modeled farming regimes, improved growing practices can be developed that would control any harmful influences on the environment. A simulation study was completed using the GLEAMS model to evaluate the effect different size rainfall events have on pesticide leaching in relationship to the timespan between when the rainfall occurs and when the pesticide was applied.

The Groundwater Loading Effects of Agricultural Management Systems model (GLEAMS) is a field scale model which is capable of tracing the movement of pesticides through the root zone. GLEAMS was developed by the United States Department of Agriculture, Agricultural Research Service in Tifton, Georgia in cooperation with the University of Georgia. The model consists of four components: hydrology, erosion, nutrient, and pesticide (Leonard et al., 1986). These components are interactive to provide a complete simulation of field conditions.

Numerous studies have assessed the reliability and accuracy of the model. Original model calibration by Leonard and Knisel proved that GLEAMS simulated pesticide movement and leaching within the variable range of the field data (Leonard et al., 1986). Sichani et al. (1991) validated GLEAMS on a silt loam soil in Indiana and found that the overall results of the simulations were close to the actual values. However, the model did not predict the leaching of pesticides after the first storm event accurately. The authors also stated that the partitioning coefficient had a significant effect on simulation results. Smith et al. (1991) compared GLEAMS and PRZM (Pesticide Root Zone Model), both with input parameters not optimized or calibrated, and found agreement of the GLEAMS results within an order of magnitude and in most cases within a factor of two or three to actual field data. On the coastal plain region of Virginia, Zacharias and Heatwole (1994) found that GLEAMS did well in predicting the pesticide mass in the soil, but not as satisfactory in describing its distribution in the soil. This study was also done with an uncalibrated version of the program. Truman and Leonard (1991) did an in-depth study of the parameters of half-life and partitioning coefficient in the GLEAMS model. They stated that these inputs, along with rainfall greatly affected the results of the model and careful attention should be paid to calibrating these values for any particular location.

MODEL DESIGN AND CALIBRATION

In order to predict the outcome of differing scenarios in the future, past climatic data may not be useful. When long-term studies of many years are to be run, historical data would be acceptable, because any yearly abnormalities would be averaged out. However, if only short-term simulations are to be run, actual data may not be appropriate due to the high significance of rainfall in the GLEAMS model calculations. Any single year probably would not be a typical average year, and bias would be introduced in picking the year of data to use. Therefore, in this study a first-order Markov chain and gamma distribution were used to generate the climatic data that was used in the GLEAMS model. The first-order Markov chain was used to determine the days of precipitation. A gamma distribution was used to estimate precipitation amounts on the days selected by the Markov chain. Both the Markov chain and gamma distribution are proven and accepted methods for analyzing climate data (Scarborough, 1995).

For GLEAMS to model pesticide movement in the soil accurately, four major areas of parameters are required: climatic data, soil data, crop growth and farming characteristics.

The climatic data for the calibration of the model were obtained from the National Oceanographic and Atmospheric Administration monitoring station located at the University of Delaware Research and Education Center at Georgetown, DE. These date included the daily temperature extremes and the daily precipitation. The monthly radiation, dew point, and wind movement values were provided with the program for the Georgetown area. Since the program is not highly sensitive to these parameters, average values are acceptable. The location data included latitude and longitude of the field and elevation above sea level. For the simulation runs of the program, the modeled precipitation was used along with the average daily temperatures as computed from historical data.

The soil data were gathered from a series of sample pits located in the field used for model calibration at the University of Delaware Research and Education Center near Georgetown, Delware. The soil is classified as an Evesboro sandy loam (mesic, coated, typic quartzipsamments). The base permeability and saturated conductivity of the soil were taken from the USDA-SCS Soil Survey of Sussex County (Ireland and Matthews, 1974). Previous application by the author, using the program for nutrient leaching had adjusted the soil parameters for GLEAMS to simulate most closely the conditions of southern Delaware. These included the analyses of soil porosity and saturated conductivity values which are the primary factors for water movement in the soil. The values of 0.375 cm/cm and 2.36 cm/hr resulted in the highest correlation with actual values. The field capacity and wilting point were also available from the soil survey. The moisture content was assumed to be at field capacity at the beginning of each run (January 1). The Evesboro soil type used for the calibration is also the major soil type of crop production land in Sussex County.

The agronomic factors of the calibration trials included field preparation by means of chisel plowing followed by two diskings immediately prior to planting. A full season corn hybrid was planted with the pesticides being applied on the following day. The crops were irrigated, with the irrigation amount entered in the rainfall data file as additional rain. The simulated crop data consisted of corn planted on April 20. The simulation field was prepared identically to the calibration fields. Irrigation was applied by the program whenever the soil moisture fell below 50 percent of field capacity. Enough irrigation was applied to restore the soil moisture to field capacity. The crop was fertilized as recommended to simulate actual plant and root growth and evapotranspiration conditions.

For the calibration and simulated runs, no metabolites were considered. All application program runs used rates of 2.24 and 1.68 kg/ha active ingredients for atrazine and metolachlor, respectively.

A sensitivity analysis of the plant uptake variable indicated only a 1.5% change in the amount of pesticide in the soil after 51 days. Therefore, plant uptake was set at one to show the best-case scenario. The inputs for foliar application and foliar wash off were ignored because the pesticides were applied to bare soil only. The method of application being considered was a broadcast surface spraying over the entire field. The two variables that greatly affect the movement of the pesticide in the soil are the soil half-life of the pesticide and the partitioning coefficient, or the ratio of the concentration of pesticide on organic matter to concentration of pesticide in water.

Published values of half-life vary from 18 to 120 days for atrazine and from 15 to 132 days for metolachlor (Ware, 1992).These values are greatly controlled by soil moisture, temperature, and microbial activity in the soil. The GLEAMS soil half-life calculation are based on the exponential decay formula:

$$f(x) = a\, e^{bx}$$

Therefore these values could easily be determined separately from the GLEAMS model. Soil samples were collected at depths from 0 to 15 cm in 1987 and 1988 over a period of time, for atrazine and in 1987 for metolachlor from a pesticide leaching experiment. The pesticide field data was fitted using a decay equation for each pesticide. After the best-fit decay curve was determined, pesticide half-life of 31.3 days for atrazine and 27.6 days for metolachlor were calculated. These values were used for all computer simulations. The r^2 values were 0.833 and 0.780 for atrazine and metolachlor, respectively.

With the amount of pesticide in the soil set prior to any leaching, the pesticide profile in the soil could then be calibrated. The GLEAMS output includes a pesticide mass output for twelve soil layers after any rainfall that causes pesticide movement in the soil. The outputs were correlated with the corresponding soil sampling dates, and the soil layers were modified to correspond to identical depths. A series of runs of the program were completed with differing values for the partitioning coefficient (KOC). An analysis of variance was carried out to select the value that best represented the actual pesticide profile in the soil. KOC values of 25.0 and 50.0 with correlation coefficients of 0.92 and 0.87, were selected for atrazine and metolachlor, respectively.

A typical corn production field on an Evesboro soil, and the agronomic variables set to recommended standards was used for the GLEAMS simulation.

A 2-. 10-, and a 100-year return period 24 hr storm was modeled for major rainfall events occurring 3, 7, 15, 30 and 90 days after application of the pesticides. These corresponded to rainfall amounts of 9.5, 14.1, and 20.4 cm, respectively. A total of sixteen simulations were run.

RESULTS AND DISCUSSION

A summary of the simulated results for atrazine and metolachlor for base precipitation, 2-, 10- and 100-yr return periods are presented in Tables 1 and 2. For all simulations an application loss of 5% is assumed to account for drift and volitalization losses.

Table 1. Atrazine Fate Based on the Interval from Application to Storm Event for 10-Year and 100-Year Return Periods.

		10-Year	100-Year
3 Day	% application loss	5.00	5.00
	% degrade + other loss	8.50	8.38
	% in soil	84.47	82.73
	% leached	2.03	3.89
7 Day	% application loss	5.00	5.00
	% degrade + other loss	19.57	19.48
	% in soil	73.73	72.17
	% leached	1.70	3.35
15 Day	% application loss	5.00	5.00
	% degrade + other loss	34.93	34.86
	% in soil	58.38	57.07
	% leached	1.69	3.08
30 Day	% application loss	5.00	5.00
	% degrade + other loss	58.28	58.25
	% in soil	35.33	34.43
	% leached	1.38	2.33
90 Day	% application loss	5.00	5.00
	% degrade + other loss	89.05	89.05
	% in soil	5.55	5.32
	% leached	0.39	0.63
3 Day	Annual % leached	4.19	5.89
7 Day	Annual % leached	3.67	5.17
15 Day	Annual % leached	3.26	4.44
30 Day	Annual % leached	2.54	3.27
90 Day	Annual % leached	0.82	0.96

Table 2. Metolachlor Fate Based on the Interval from Application to Storm Event for 10-Year and 100-Year Return Periods.

		10-Year	100-Year
3 Day	% application loss	5.00	5.00
	% degrade + other loss	9.49	9.42
	% in soil	84.36	83.20
	% leached	1.14	2.38
7 Day	% application loss	5.00	5.00
	% degrade + other loss	21.53	21.39
	% in soil	72.53	71.60
	% leached	0.94	2.01
15 Day	% application loss	5.00	5.00
	% degrade + other loss	38.06	38.00
	% in soil	56.01	55.17
	% leached	0.94	1.84
30 Day	% application loss	5.00	5.00
	% degrade + other loss	61.66	61.65
	% in soil	32.61	32.02
	% leached	0.73	1.33
90 Day	% application loss	5.00	5.00
	% degrade + other loss	90.60	90.57
	% in soil	4.24	4.15
	% leached	0.16	0.28
3 Day	Annual % leached	2.22	3.44
7 Day	Annual % leached	1.90	2.96
15 Day	Annual % leached	1.70	2.53
30 Day	Annual % leached	1.27	1.78
90 Day	Annual % leached	0.31	0.41

Analysis of the base precipitation output revealed that a total of 122.8 cm of rainfall and irrigation occurred throughout the year with 41.8 cm of percolation below the root zone of 152 cm and, theoretically, into the water table. The percolation caused 3.53 g/ha or 0.16% of the atrazine to leach into the water table. The first leaching did not occur until 40 days after application when a 3.8 cm rainfall event occurred. The amounts leached from that event were 0.44 and 0.11 g/ha of atrazine and metolachlor, respectively. This was the greatest single loss of atrazine and metolachlor for the year.

After three days, the base model indicated 87.7% of the atrazine and 86.8% of metolachlor remaining in the soil, with neither pesticide leaching. Simulations with all the storm events increased the degradation and other loss to slightly more than 1%. The average degradation losses for all return periods was approximately 8 and 9% for the atrazine and metolachlor, respectively. The atrazine had leaching percentages of 0.69, 2.03, and 3.89 for 2-, 10- and 100-year storms, respectively. Metolachlor had leaching rates ranging from 0.27 to 2.01% for the three storm scenarios.

Seven days after the application of the pesticide, the atrazine had degraded approximately 19% and metolachlor 21%. For the 2-, 10- and 100-year storms 7 days after application 0.55%, 1.70% and 3.35% of the atrazine was leached and 0.27%, 0.94% and 1.84% of the metolachlor was leached, respectively. Similar trends in the data are found with storms 15 and 30 days after application as shown in Tables 1 and 2. One exception to this trend is for the 2-year return period storm. These values do not indicate a specific trend, due to the greater influence of the base rainfall amounts versus the 2-year storm, as compared to the significantly higher differences with the 10- and 100-year storms versus the base data.

For all scenarios ninety days after application, the amount of pesticide left in the soil had decreased to between 4 and 6% of the application rate. Atrazine had a 89% degradation rate, while metolachlor had degraded 90%. The potential leaching from a major storm event had become less than 0.7% of the applied atrazine and less than 0.3% of the applied metolachlor.

The annual summary of data from date of application until December 31 showed the 0.16% leaching rate of atrazine for the base data. The total annual leaching for atrazine ranges from 0.38 to 5.89% of the applied pesticide with the 100-year storm, three days after application being the highest. All values except the 90-day interval and the 30-day, 2-year storm indicate leaching potentials of greater than 2.0% of the applied atrazine. The base value for annual metolachlor leaching is 0.01% with the storm leaching potentials ranging from 0.09 to 3.44%.

There is also a common crop production practice that can compound the leaching problems. If there is not adequate weed control, often another application of pesticide is made. When these pesticides are applied pre-emergent, they work by killing the weeds through root uptake and at germination (Hughes, 1982). Therefore the pesticides must be present in the uppermost layer of the soil, where the weed seeds are located, at high enough levels to be effective. The base rainfall simulation showed all the pesticides remaining in the top two layers through the first 15 days. After 15 days the crop should be large enough to shade out any weeds that emerge. A 2-year return period storm 3 days after application is capable of moving 65% of the available atrazine out of the effective soil layers, while a 100-year storm will move over 76% of the atrazine away from the soil surface. These percentages remained consistent with the 7- and 15-day simulations. The values for metolachlor were slightly better with an average of 55% and 67% remaining in the top two layers after 2- and 1000-year storms, respectively. The potential loss of the effective use of the available pesticides are actually worse since the values are for the top 2 layers of output from GLEAMS (0 to 9 cm). However, the active zone of the pesticide is the top 3-5 cm (Hughes, 1982). These loss values show potential lack of weed control and the probably need for another application of these or other pesticides to control the weeds, and the potential for more leaching of pesticides into groundwater.

SUMMARY AND CONCLUSIONS

The GLEAMS model simulations that were run show the great potential for pesticides leaching into the water table of southern Delaware due to the combination of sandy soils, high water table, and intense crop production. There is no way to predict the possible occurrence of a major storm more than 7 days in the future, but concern should be shown in the weather forecast for the week following an application of pesticides. The effects of potentially high amounts of pesticide leaching should be combined with the very dynamic nature of the water table before any conclusions about contamination can be assumed. The surrounding areas, lateral movement of the aquifer, pumping, and other water budget considerations will greatly affect pesticide concentrations and movement in the water table. However, there is a high potential for problems on the micro-scale of a farmstead that draws water from the water table aquifer and is surrounded by production fields. Another question concerns multi-year scenarios. Since pesticide degradation in water is limited to chemical processes, which is an extremely slow process, the potential of ever increasing levels of contamination in the aquifer is possible.

Further investigation is needed to analyze and model the movement of pesticides after they reach the water table aquifer. This would involve the introduction of numerous parameters including climatic factors, aquifer movement, wells and other discharge points, and surrounding physical and cultural practices. There also needs to be an in depth investigation of pesticide half-life based on length of times to rainfall after application, and the influence of soil moisture and temperature on rates of degradation. With this added knowledge, a detailed scenario to assess the cultural and environmental impacts of current pesticide application practices for substantially larger areas could be developed.

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WATER QUALITY OF SURFACE IRRIGATION RETURNS IN SOUTHERN IDAHO

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ABSTRACT

The Clean Water Act and Safe Drinking Water Act and the Environmental Protection Agency of the United States Federal Government have provided means and encouragement to irrigation projects and entities to improve the quality of surface water returns flowing to river systems. In Idaho, state and federal partnerships have set maximum concentration limits for suspended sediment (52 mg/l) and phosphorus (0.1 mg/l) on portions of the Snake River system and its tributaries through the total maximum daily load process. The irrigation community has responded by implementing programs and partnerships for improving quality of returning water. These programs include education, water quality improvement facilities and best management practices, and monitoring.

BACKGROUND

The Snake River reach between Milner and King Hill in southern Idaho is locally referred to as the Middle Snake River. Water quality in the 94-mile reach has been impacted various ways resulting in some of the designated beneficial uses not being attained within the reach (**Figure 1**). This stretch of the Snake River is impacted by return flows from irrigated agriculture, fish hatchery effluent, hydroelectric development, sewer treatment plant discharge, spring flows, grazing and recreational activities.

The Middle Snake River often is referred to as a working river because of its highly regulated stream flow. Reservoirs upstream of Milner have a combined

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storage capacity in excess of 4.5 million acre-feet[4]. Impoundments and diversions, primarily for irrigation and hydroelectric power generation, have resulted in smaller stream flows and stream flow velocities thus limiting the Middle Snake River's ability to assimilate sediment and nutrient inputs from non-point and point sources.

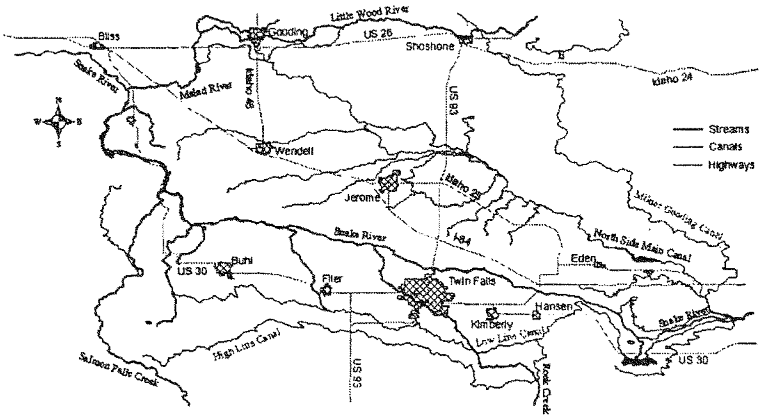

Figure 1. Map showing general location and area of interest.

Water quality in the Middle Snake River reach is impacted by a variety of non-point and point sources of pollution. Non-point sources account for most of the sediment reaching the river and its tributaries. Primary non-point sources of pollutants are agricultural activities, animal grazing and irrigation. Primary point sources of pollutants in the reach are municipal wastewater-treatment facilities and aquaculture related activities. The degraded water quality in the river results from a combination of excessive nitrogen, phosphorus, pesticides, and sediment; and the cumulative effects of decades of non-point and point source activities. Excessive aquatic vegetation, low dissolved oxygen, and high water temperatures prevent water in this reach from meeting State water-quality criteria[5].

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Geologic and Hydrologic Setting

The Middle Snake River flows through an incised canyon in the basalts of the Snake River Group. The present location and configuration of the river is the result of ancient canyon filling processes and erosion and deposition during the Pleistocene Bonneville flood. Flowing west of Milner Dam, the river becomes incised in the basalts, and the elevation of the river drops below the ground water table elevation of the aquifers bordering the river. The primary outflow from the Eastern Snake Plain Aquifer occurs along the north side of the river. Springs issuing from the canyon walls contribute significantly to the river flow for the lower three fourths of the reach. These spring inflows contribute approximately 5,000 to 6,000 cfs to the river. Depending on the water supply in the Upper Snake River Basin, the flow entering the reach or passing Milner Dam may be as low as 200 cfs or as great as 30,000 cfs during floods. To meet storage and irrigation diversions upstream of Milner Dam the U. S. Bureau of Reclamation and Idaho Water District Number 1 control the flow passing Milner Dam and entering the reach.

Adjacent to the canyon, 360,000 acres are surface irrigated with sprinkler and traditional gravity methods using water diverted from the Snake River above Milner Dam. The Northside Canal Company operates the diversion and delivery system on the north side of the river for 160,000 acres and the Twin Falls Canal Company operates the diversion and delivery system on the south side of the river for 200,000 acres. The majority of the farms are sprinkler irrigated on the north side (90%) compared to the majority of farms under furrow irrigation on the south side (75%). The crops include alfalfa, dry beans, corn, small grains, sugar beets, and potatoes. The soils are loessal in origin varying between 6 to 40 inches in depth. Most of the soils are classified as a silt-loam or sandy-loam and are highly erosive. The climate is considered semi-arid with most of the precipitation occurring during the winter and spring months.

Irrigation return flow from the two systems enters the Snake River directly, from numerous surface drains on both sides of the canyon and tributaries, and indirectly through springs along the river and tributaries. These drains represent surface return flows from irrigated agricultural ranging from individual fields to irrigation canal company lateral returns. Excluding the individual field and farm drains, the canal companies estimate maximum surface return flows at approximate 420 cfs for both the north and south sides. Individually, the return flow drains typically have highly variable flows reflecting daily and seasonal patterns in water use.

Water Quality Awareness

In the early 1990's, local citizens and concerned agencies formed a working group to address water quality problems within the Middle Snake River reach. As an outgrowth of the working group's activities, the State of Idaho implemented the nutrient management planning process. The working group served as the basis for the initial planning group. Representatives from the various industries, municipalities, irrigated agriculture, recreational and environmental groups served on the planning committee. The parties took a proactive stance on improving water quality in the reach. Many of the participants, including agriculture, developed proactive plans of action to reduce their impact on water quality in the reach and a few started implementing their plans. As the nutrient manage planning process was transformed into a Total Maximum Daily Load (TMDL) determination and allocation process overseen by Idaho Department of Environmental Quality (IDEQ) and U.S. Environmental Protection Agency (EPA), the planning group became in part the Mid-Snake Watershed Advisory Group.

As a result of the efforts during the 1990's, two TMDLs were written by IDEQ and approved by EPA for the Middle Snake River and it's associated tributaries. The first TMDL addressed total phosphorus within the Middle Snake River and set a target concentration of 0.075 mg/l. Irrigated agriculture commitment to the first TMDL was to reduce suspended sediment entering the river via surface returns by 27 percent. Irrigated agriculture representatives and advisory staff felt that there would be an associated 10 percent reduction in total phosphorus by attaining the suspended sediment goal. Based on the commitment by irrigated agriculture, sixteen indicator drains were assigned specific phosphorus load targets. The second TMDL[6] addressed then entire Middle Snake watershed formally known as the "Upper Snake/Rock Watershed". This TMDL set suspended sediment concentration target level at a mean monthly 52 mg/l with a daily maximum of 80 mg/l for the river and it's tributaries. Additionally, total phosphorus target concentration of 0.1 mg/l was set for tributaries. The analysis was based on the assumption that the quantity of return flow would not change.

Canal Company Actions

Over the past decade both canal companies have pursued a vigorous proactive course with regard to water quality in the Middle Snake River. The canal companies were instrumental in the formation of the Irrigators' Water Quality Committee (IWQC). This committee was comprised of representatives from canal companies, local soil conservation districts, local farmers, the University of Idaho, the USDA Agricultural Research Service, the USDA Natural Resources Conservation Service and IDEQ. The committee served to guide the irrigated agriculture representatives on the various advisory groups associated with watershed management plans and total maximum daily load determinations for

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[6] Buhidar, Balthasar B., et al., 1999, The Upper Snake Rock Watershed Management Plan, The Upper Snake Rock Subbasin Assessment and The Upper Snake Rock Total Maximum Daily Load, Idaho Division of Environmental Quality, Twin Falls Regional Office, Twin Falls, Idaho.

the Middle Snake River and its watershed. The committee also served as a clearinghouse on activities by the various entities on their efforts to improve water quality. The committee was instrumental in developing the irrigated agriculture management plan identified in the two Middle Snake River TMDLs. The committee additionally served in an oversight role for an irrigation return flow water quality project funded by the two canal companies and operated by the University of Idaho.

The canal companies have been diligent in efforts to build water quality improvement facilities on the various lateral returns and or drains prior to their discharge into the Snake River or its tributaries. The facilities are typically large-scale sediment ponds and have retention times on the order of 4 to 36 hours. Some of the facilities have constructed wetland components. The partnerships developed by the canal companies to construct and operate these facilities have included: The Nature Conservancy, Idaho Power, Idaho Department of Fish and Game, and others besides local property owners.

WATER QUALITY PROJECT

The water quality project funded by the canal companies and operated by the University of Idaho received support from the U.S. Bureau of Reclamation who provided assistance in laboratory analyses. The project's approach identified by the canal companies, the university and the IWQC consisted of three key elements: education, water quality practices, and monitoring. The three components were intended to increase the project's potential for successfully improving irrigated agriculture's impact on water quality in the Mid Snake watershed.

Education

The education program focused on education and training on three levels: irrigators (landowners and operators), general public, and canal company staff. The two major goals of the education program were: (1) to bring about improved water quality in the Mid-Snake by emphasizing improved tillage and water management practices on the land; and, (2) to educate the general public about good-faith efforts underway by the canal companies and irrigators to improve water quality of irrigation return flows.

The educational effort associated with farm owners, operators, and irrigators had two primary focus points. The first was to increase the awareness that irrigated agriculture along with other industries and municipalities does impact water quality in the Middle Snake River. The other focus of the irrigator education program was to provide information about the water quality situation and information on new technology and management practices for improving water quality in irrigation return flows. Proposed, enacted and adopted regulatory requirements regarding irrigation return flow water quality was provided to irrigators and landowners. Procedures were emphasized for management of delivery systems for sediment and nutrient control, on-farm tillage and irrigation management practices, and operation of sediment ponds and vegetative areas for both water quality improvement and wildlife habitat enhancement.

The public education program provided a historical perspective of water development in Magic Valley and associated benefits, and an understanding of the irrigation process and it's impact on sediment and nutrient loading to the river. The operation of irrigation systems, constraints to change, and trade-offs in water management, energy, and water supply were presented to various public service groups, schools, and other organizations. Water quality improvements efforts by the canal companies and shareholders (landowners) were identified at those presentations and addition to various tours conducted for interested parties and regulatory agencies. Additionally, presentations were made to educate the agriculture support sector (consultants, business leaders, and lenders) on activities and practices that control erosion and reduce sediment and associated nutrients from surface return flows at the farm level.

The educational effort associated with canal company ditch riders focused on water management, recognizing farmers and irrigators with successful water quality management plans, and identifying those who may need help. The hope was that the ditch rider would be able to discuss water quality practices with individual irrigators and farmers when the opportunity arose. They would be able to relay information needed or identify sources of help to those needing assistance. Information sessions on how canal company activities impact water quality within the canal system and finally the Middle Snake River were developed and presented to the employees. By increasing the water quality awareness of the canal company staff, opportunities for improving water quality by changing day-to-day operations and minor system changes were identified by the staff, some of which were implemented.

Water Quality Improvement Facilities and Best Management Practices

A majority of the farm level best management practices (BMPs) have centered on controlling erosion on the field and treating the surface runoff. These practices have changed little over the past years. They include modifying traditional crop management and water management practices. New BMPs have incorporated usage of chemical additives such as polyacrylamides to stabilize the soil structure. The historical control and treatment practices effectiveness has been documented in previous studies by various agencies. The effectiveness of new practices are being evaluated by the USDA Agricultural Research Service and others.

University personnel provided assistance to the canal companies in locating potential water quality improvement facility sites, designing the facilities, and evaluation of alternatives. After implementing several facilities, the canal companies developed expertise in locating sites and facility design. The university monitoring program documented the performance of typical facilities on a case-by-case basis.

WATER QUALITY MONITORING

The water quality monitoring program developed by the University of Idaho for irrigated agriculture consisted of four components: attitudes, best management practice and facility performance, surface return flow quality, and in-stream water quality of the Middle Snake River. The first two components were to assess the effectiveness of the educational program and implemented practices. The remaining components of the monitoring program were to assess water quality trends and improvements related to the Mid Snake and irrigation return flows.

Attitudes

An attitude survey was developed by University of Idaho Extension staff and mailed to irrigators within each canal company service area. The attitude survey attempted to quantify the impact of the educational program of the project. Surveys were mailed to all canal company shareholders irrigating more than 5 acres. The surveys were conducted four times during an eight-year period. The survey was not a rigorous survey where similar statements were repeated in different forms. From the results of the four surveys, the attitude of the irrigation community appeared to have changed. Initially, the owners and operators were of the opinion that sediment control practices either cost too much, required too much time, or reduced yields. By the last survey the respondents had moderated their opinions. The level of agreement to statements in the survey expressing that practices took too many resources changed from agreement to disagreement.

Based on the surveys and the educational efforts, the canal companies were able to adopt bylaws governing the quality of surface water returning to the distribution and return flow systems. The stockholders (landowners) of the two companies have supported management's allocation of more resources to water quality programs within each company.

Water Quality Improvement Facility Monitoring

While the irrigated agriculture nutrient management plan identified best management practice implementation on the irrigated fields, the irrigation companies have constructed water quality facilities on the drainage systems. The typical facility is a sediment pond; however, constructed wetlands have been built in cooperation with landowners and conservation groups. Components from a constructed wetland concept have been implemented where applicable. The monitoring program addressed the need for facility monitoring to document performance.

In-stream Monitoring

The in-stream water quality-monitoring program was based on the 1990-91 monitoring work performed by the University of Idaho on the Middle Snake River for the IDEQ[7]. Four selected locations along the Snake River were monitored on a biweekly schedule. The monitoring locations along the reach were Milner Dam (inflow), Blue Lakes Bridge (Twin Falls), Clear Lakes Bridge (Buhl), and Shoestring Bridge (Bliss). The monitoring package consisted of field parameters and laboratory parameters. The parameter list included air and water temperature, dissolved oxygen, pH, electrical conductivity, transparency, total suspended solids (sediment), ammonia nitrogen, nitrate nitrogen, total Kjeldahl nitrogen, dissolved orthophosphate, total phosphorus, and chlorophyll-A. These four monitoring sites were to provide trends on water quality in the river and provide a data set for evaluating the success of all efforts, not only irrigated agriculture, in improving water quality.

Over the decade of monitoring, the discharges in the Middle Snake River ranged from historical lows to historical highs. Figure **2** shows the average daily discharge for the Snake River at the USGS station near Kimberly for the period of monitoring.

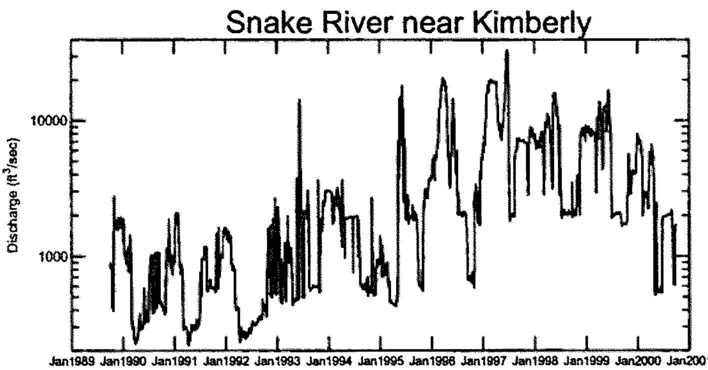

Figure 2. Snake River daily discharge near Kimberly, ID.

The wide variation in flows over the decade has resulted in difficulty in determining trends in suspended sediment loads within the reach. During the early 1990's, the region experienced a drought with very low flows. Sediment was being deposited in various areas of the river. These deposits along with deposited

[7] Brockway, Charles E. and Clarence W Robison, 1992. Middle Snake River Water Quality Study, Phase I, Idaho Water Resources Research Institute, University of Idaho, Moscow, Idaho 83843.

sediment from irrigation return flows during the twentieth century were eroded during the high flows in 1996 and later years. Figure 3 shows average quarterly suspended solids loads in the river at Blue Lakes Bridge near Twin Falls. These loads are based on the bi-weekly monitoring results and represent between six and seven measurements. Clearly, there does not seem to be a trend in the loading at the site not associated with stream flow.

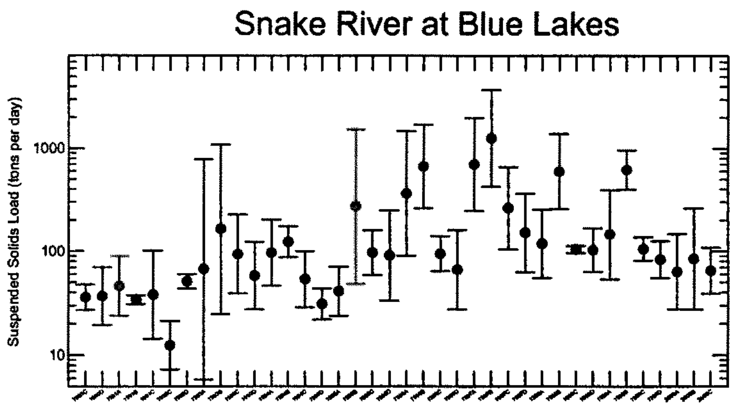

Figure 3. Quarterly Suspended Solids Load near Twin Falls

Return Flow Monitoring

The surface return flow-monitoring program focused on 16 irrigation return flow drains discharging directing to the Middle Snake River reach. The drains were identified by the IDEQ and the University of Idaho in a 1990-91 study as contributing over 60 percent of the irrigation return flow volume to the river. The drains were identified in the first TMDL developed for the Middle Snake River as the key indicator drains for irrigated agriculture and had individual phosphorus loads allocated to them. The program targeted these drains to be monitored on a biweekly basis every three years for temperature, dissolved oxygen, electrical conductivity, flow, total suspended solids (sediment), ammonia nitrogen, nitrate nitrogen, total Kjeldahl nitrogen, dissolved orthophosphate, and total phosphorus. On the off years, some of the drains were monitored for total suspended solids and total phosphorus. Figures 4 and 5 show the concentrations over time of suspended solids and total phosphorus for a typical southside irrigation return flow drain. This drain is showing improvement from the various activities on its watershed (farmland) over the past decade.

A web site has been created and maintained to make the collected water quality data available to the public (http://www.kimberly.uidaho.edu/MidSnake). The site contains summary data for the various parameters at each monitoring site along with scatter plots of the data over time.

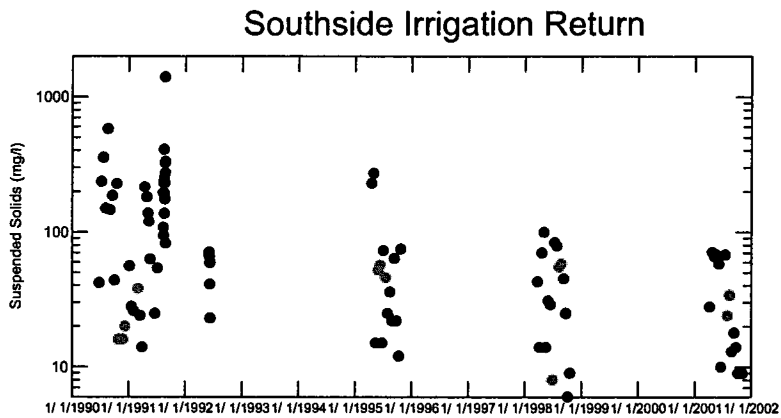

Figure 4. Suspended solids concentration for a southside drain.

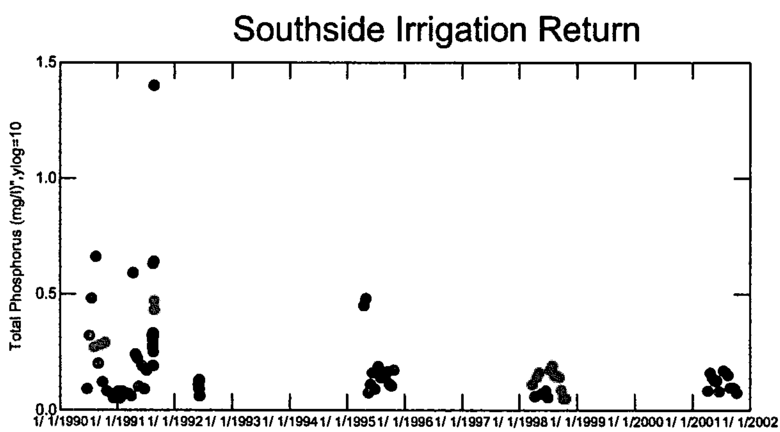

Figure 5. Total phosphorus concentrations for a southside drain.

SUMMARY AND CONCLUSIONS

Water quality monitoring of irrigation return flows has provided valuable information on magnitudes of and trends in the quality of surface waters returning to the Snake River of southern Idaho. Data show a general improvement in return flow quality resulting from adoption of best management practices and proactive water management programs by local canal companies. The water quality of the mid Snake River should also improve over the near future as surface returns improve and as other programs supported by the TMDL process evolve.

IMPACT OF FARMER-MANAGED MAINTENANCE OF SECONDARY CANALS ON WATER DISTRIBUTION EQUITY: A CASE STUDY FROM SINDH, PAKISTAN

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D. Hammond Murray-Rust[2]

ABSTRACT

International Irrigation Management Institute (IIMI) carried over a pilot action research study on farmer managed irrigation system (FMIS) in Sindh Province of Pakistan. Overall fourteen Farmer Organizations (FOs) on distributary channels were formed. In order to ensure success of this participatory management, the FO members were trained in organizational management, operation and maintenance (O&M) of channels and financial aspects. The study focuses on impact of farmer-managed maintenance on water distribution equity and resources mobilization.

In conjunction with a program to organize water users at secondary canals in Sindh, IIMI staff made observations on the physical conditions before and after the maintenance campaign in January 2000 and actual inputs made by members of Farmer Organizations. In a one-week period water users contributed over 7,800 man-days of labor and 582 hours of tractor operation in eight secondary canals, and removed over 43,000 cubic meters of sediment. The imputed cost of these contributions exceeded over Rs.900,000 ($15,000) or almost Rs.30 per hectare ($0.50).

The hydraulic benefits were substantial. Comparing water deliveries into the head and tail reach of each canal before desilting, head end areas received roughly 68% more water than tail enders. After desilting the head end areas only received 14% more water, and in six of the eleven canals where water measurements were taken, tail end areas actually received more water than head end areas.

Despite the benefits that accrued, there is concern for the long-term sustainability of the improved performance. There is no systematic monitoring program that enables operation and maintenance to be linked and no proper maintenance of control infrastructure to complement desilting. Until these institutional changes occur, operation and maintenance will remain a largely ad-hoc activity.

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INTRODUCTION

Maintenance in the Context of the Design of Canals in Pakistan

There can be no doubting the importance of maintenance as an integral component to the successful performance of irrigation systems. There are many generic manuals available such as those produced by ASCE (1980), ICID (1989) and Snellen, for FAO (1996), and there are innumerable O&M manuals for national and provincial Irrigation Departments and for specific projects for new or rehabilitated irrigation systems. Few textbooks fail to stress the importance of maintenance, and some books are largely focused on the problem (Skogerboe and Merkley, 1996, and Verdier and Millo, 1994, are good examples).

Yet the stark reality is that irrigation performance in most irrigation systems appears to continue to decline, and guidelines or standards for maintenance remain ignored or only partially followed. Further, this is taking place in many countries where there is in process a transfer of operation and maintenance responsibilities that brings in a new set of managers and system operators who may not have had prior experience in running a focused and systematic maintenance program.

To ensure that canals did indeed function as designed, three pieces of data were required to be collected that helped the operational staff of the irrigation agencies to determine whether or not canals were performing as expected (Sindh Irrigation Department, 1995):

- Daily water levels at the gauge at the head of secondary canals, converted to discharge using a rating table that was meant to be revised at least once a year;
- Daily water levels at the gauge installed at the tail of the secondary canal, which are normally designed to be 0.6 feet (18.3 cm)
- Monthly measurements of water levels in the secondary canal adjacent to each outlet

Using these three pieces of information it is easy to determine whether or not the canal system is functioning properly. The head gauge shows whether or not the incoming discharge is at designed level, and if not then the control structure at the head of the secondary canal must be adjusted. The tail gauge should read the desired target level if the head discharge is at design. If it is not, then something has gone wrong and a field inspection is required to find out why this discrepancy has occurred.

However, if there are persistent discrepancies the problem is likely to be one of sedimentation or erosion that has caused water levels along the canal to deviate from design, and therefore change the discharge passing through each outlet structure. The third piece of data, the monthly water level readings entered in the "H" register, was specifically collected to allow engineers to determine where such problems were occurring and to take necessary remedial measures such as desilting or bank restoration.

As long as the three sets of data showed no problems, engineers could be assured that water distribution was at or very close to the equity expected of the system. As long as illegal actions at individual watercourses could be kept under control, and as long as discharges were at or close to design, inequity could only occur through improper maintenance. From this we can conclude that maintenance of secondary canals is the key to water delivery performance in supply-based canals such as those of Pakistan and Northwest India.

STUDY AREA

Pilot Farmer Organizations in Sindh

The pilot program not only dealt with the creation of the structure of watercourse organizations at watercourse level and federated at distributary level and formed the Farmer Organizations (FO), but also provided technical support and training in various aspects of irrigation and drainage system operation and maintenance. FO members received training on water measurement, canal operation, development of a business plan, collection of irrigation fees and development of effective and manageable maintenance plans. The FOs formed at different distributaries are indicated in Figure 1. However, the salient features of the pilot distributaries where hydraulic and diagnostic survey were carried out, are given in Table 1.

Table 1. Salient Features of the Pilot Distributaries.

Distributary	**Length (Km)**	**Area (ha)**
Heran	9.75	4994
Rawtiani	8.38	3658
Bareji	11.98	5797
Mirpur	14.63	6566
Potho	10.06	3264
MAW	5.18	1552
Khadwari	5.18	1245
Dhoro Naro	9.84	5418

Although the different FO groups had no legal requirement or responsibility to undertake maintenance at secondary canal level, they were keen to do something because they all suffered some form of water distribution inequity between watercourses. So as part of the pilot program it was agreed between the Irrigation

Department and each FO that was established by January 2000 to undertake self-help maintenance activities with technical support from the IWMI staff in the field. A total of eight FOs participated in structured maintenance planning, while an additional 4 undertook less structured maintenance where no hydraulic surveys were undertaken.

The approach followed is summarized in Table 2, that contains a set of logical steps to involve the water users in a program that should result in properly functioning canal once maintenance has been completed.

By and large each FO was able to undertake these activities with only minimal technical support from IWMI. The main areas of involvement of IWMI was in doing the surveys of the canals, estimating the work to be done and the actual work accomplished, and continuing the routine water measurement activities before and after maintenance.

Table 2. Maintenance Program for Farmer Organizations at Secondary Level Canals

Activity	**Tasks**
Problem Identification	• Meeting of FO committee before canal closure to identify problems resulting from degraded canal; • Motivate members to participate and contribute labor, funds and tractors
Hydraulic Survey	• Undertaken as soon as canal is closed • Measure cross-section every 1500 m • Measure long-section from head to tail • Compare to design drawing of canal
Walk-Through Survey	• FO committee members identify sections where desilting or bank repairs are required
Prioritization of Needs	• FO general meeting to discuss scale of work required • Consensus obtained on what to do as a priority • Estimation of costs involved and whether some work has to be contracted out • Set date for maintenance
Resource Mobilization	• Collection of necessary funds • Agreement for labor commitment of members • Identification of who will provide tractors
Maintenance	• Communal desilting and bank restoration until design conditions are restored
Evaluation and Feedback	• Assessment of labor and machinery inputs • Estimation of earthwork and bank work undertaken • Measurement of hydraulic performance

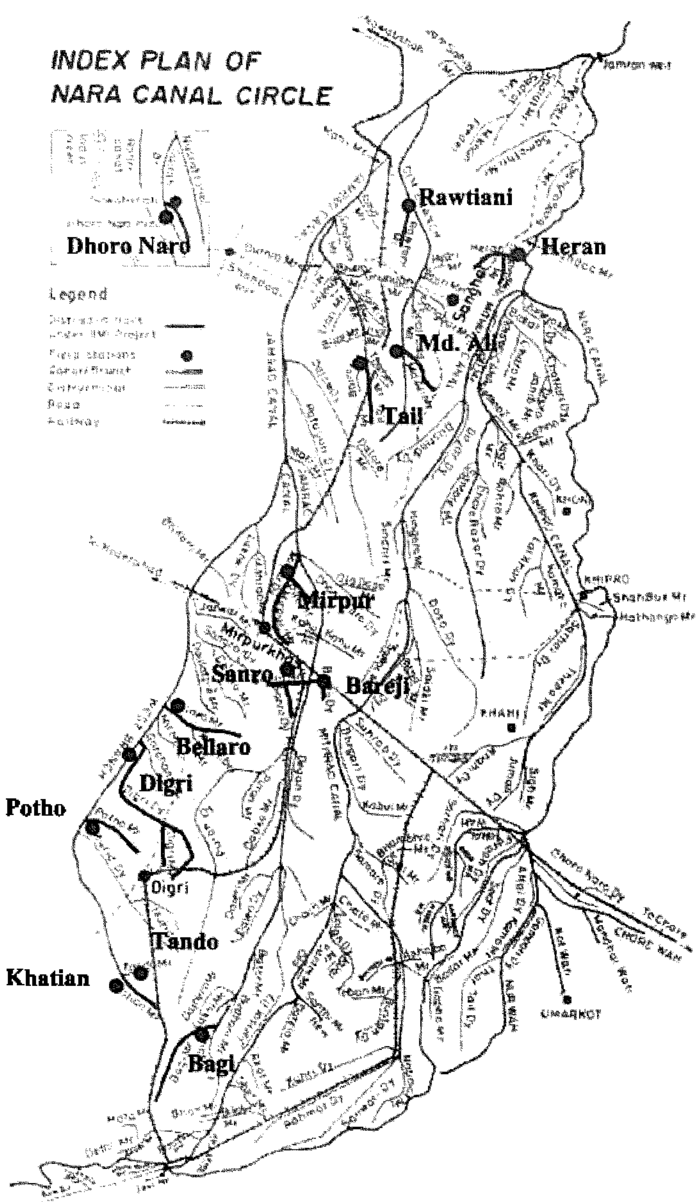

Figure 1. Location of IWMI Sample Secondary Canals in Sindh

Canal Conditions before the Maintenance

The diagnostic survey was conducted for each distributary by the FOs to identify the real problem of the distributary and prioritized the activities. The information gathered is shown in Figure 2.

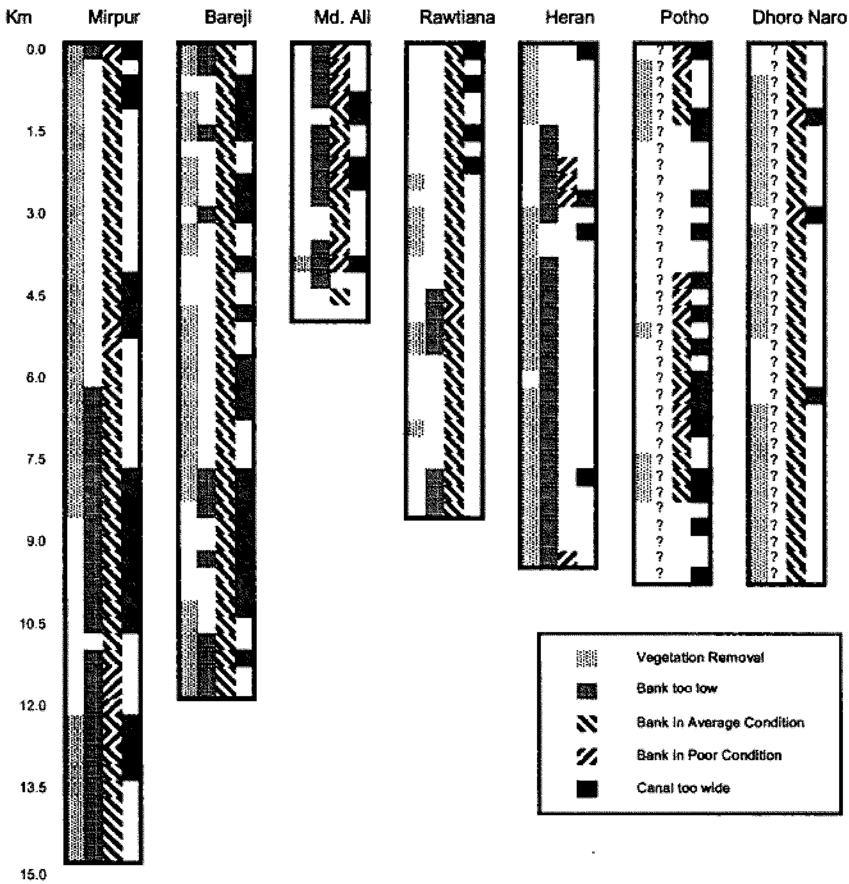

Figure 2. Maintenance Requirements.

RESULTS AND DISCUSSION

Maintenance Inputs

Labor Inputs. For an area exceeding 32,000 ha farmers provided over 7,000 man hours of voluntary labor and donated nearly 600 hours of tractor time. This represents an average of just over one person-day for 4.5 hectares (11 acres). It is clear that labor inputs were not directly matched to the amount of work actually required. In all canals there was a great deal of solidarity among water users who were keen to show the Irrigation Department that they were able and willing to undertake this work, and that they would be able to do so in the future once management transfer had occurred.

Table 3. Maintenance Inputs into Pilot Secondary Canals.

Distributary	Man-days	Tractor-hours	Imputed Cost (Rs)	Earthwork (m^3)	Work (man-days per ha)	Cost (Rs/ha)	Cost (Rs/ m^3)	Volume (m^3 per man-day)
Heran	1157	58	124100	7411	0.23	24.85	16.74	6.41
Rawtiani	586	35	64025	1351	0.16	17.50	47.40	2.31
Bareji	1020	14	105700	5601	0.18	18.23	18.87	5.49
Mirpur	1311	120	172650	9993	0.20	26.29	17.28	7.62
Potho	979	17	113611	8138	0.30	34.80	13.96	8.31
MAW	427	30	44625	3806	0.28	28.76	11.72	8.91
Khadwari	301	16	49275	n/a	0.24	39.59	n/a	n/a
Dhoro Naro	2055	292	249375	7376	0.38	46.03	33.81	3.59
Total	7836	582	923361	43678				
Average					0.25	29.51	22.83	6.09

Imputed and Actual Costs

The imputed cost of this activity is calculated on the typical labor and machinery hire rates prevailing at the time of the survey. Based on an average of Rs. 100 per day per person and between Rs.150-175 per tractor-hour, the grand total is just over Rs. 800,000. On an average basis the cost is almost Rs.25 per ha ($0.45) which represents about 40% of the typical irrigation water fee or abiana that farmers are expected to pay. This is a substantial saving for the government who would otherwise have had to pay those labor rates to accomplish the amount of work done. If the inputs were typical for all of Sindh then the total cost of maintenance for the Province would be something on the order of Rs.125 m or $2.25 m.

Sediment Removal

The volume of sediment removed was substantial, with over 40,000 cubic meters of material being cleaned out of the canals. Most of this material was used to repair banks and canal roads so the total distance of haulage was small. On average, for each 10 man-days of labor and 1 tractor-hour it was possible to remove over 70 cubic meters of sediment. Given that most sediment has to be manually lifted at least 1 m it is clear that people really did work extremely hard.

Hydraulic Impact of Desilting

Before desilting the average deliver performance ratio (DPR) at the head of the eleven canals was 1.29 (i.e. 29% above design), ranging from 213% of design at Bareji which had been remodeled in 1995 and could cope with much larger than designed discharge to 58% of design at Bagi. However, the DPR at the head of the tail sections averaged only 97% of design indicating that in most canals all of the extra water was being captured by the head and middle sections of the canal (Table 4).

Looking at the ratio between head and tail DPR values the degree of inequity can be clearly seen. In only two canals (Potho and Bagi) were tail end DPR values higher than the head: in all other canals head end values were higher than tail end and on averaged were 68% higher. At Heran and Belharo head end values were over three times as high as tail end values, showing gross inequity between head and tail.

After desilting the picture changed considerably. Average discharges into canals were only 20% above design: overall in the area discharges are low after desilting because it is the coolest season of the year and wheat in some areas is beginning to mature. However, tail end DPR values were, on average, also at 120% of design indicating almost uniform distribution. Data demonstrate that the inequity between head and tail was substantially reduced. However, many tail end areas got more water than the head, but in reality this will slowly be reversed as canals silt up again during the year.

Table 4. Hydraulic Condition of the Distributaries before and after Maintenance

Distributary	Before Desilting			After Desilting		
	Head	Tail	Ratio of Head:Tail	Head	Tail	Ratio of Head:Tail
Heran	1.36	0.38	**3.53**	1.31	0.51	**2.55**
Rawtiani	1.71	1.71	**1.00**	1.54	1.71	**0.90**
Tail	1.49	1.20	**1.23**	1.15	0.96	**1.20**
Mirpur	1.02	0.39	**2.64**	0.94	0.66	**1.44**
Bareji	2.13	1.63	**1.30**	2.13	2.36	**0.90**
Sanrho	1.29	1.11	**1.16**	1.34	1.58	**0.85**
Belharo	1.11	0.36	**3.07**	1.07	0.79	**1.35**
Digri	1.17	1.12	**1.04**	1.04	0.90	**1.16**
Potho	1.02	1.28	**0.79**	0.74	0.98	**0.76**
Khatian	1.31	0.65	**2.00**	1.25	1.35	**0.92**
Bagi	0.58	0.80	**0.72**	0.71	1.36	**0.52**
Average	**1.29**	**0.97**	**1.68**	**1.20**	**1.20**	**1.14**

CONCERNS FOR THE FUTURE

It would, however, be unwise to be complacent about the situation that was measured and observed during the January 2000 maintenance period. A number of issues remain that continue to cast doubt on the ability of Farmer Organizations to maintain their facilities now that management transfer has occurred.

Even on those canals where IWMI had undertaken physical surveys of cross-sections and longitudinal sections desilting remained more a matter of eyeballing than of systematic and controlled establishment of design sections. Yet at no time were physical measurements taken to determine whether widths, depth or slopes were consistent with what should be required to provide effective water levels at each outlet when the canal operates at design discharge.

Although hydraulic conditions improved in most canals, these results did not become incorporated into the daily actions of water users or the Irrigation Department, instead remaining more or less as a separate and unrelated measurement exercise. So the link between maintenance and performance remains weak or non-existent, and there is no sign of any major effort to try to link them again.

Based on these concerns it would be premature to suggest that one the basis of a single activity within the context of a fairly intensively managed pilot project that the Farmer Organizations can undertake all aspects of maintenance into the future. There is still a long way to go before they develop the technical skills and the managerial capacity to maintain canals, repair infrastructure, and upgrade it as and when the need arises.

CONCLUSIONS

In systems with a high degree of control over water there is some opportunity for a trade-off between operation and maintenance in order to achieve the desired water distribution pattern. In the supply-based systems of the Indus Basin and northwest India this option is not available: if canals are not maintained so that their physical condition approximates the original design, it is impossible to achieve a reasonable degree of equity of water distribution.

Irrespective of who is given operation and maintenance responsibility, be it the Irrigation Department, Farmer Organizations or private companies, the basic maintenance requirements remain the same in these supply-based systems. If ownership or management responsibility changes, there is no hydraulic basis for altering the rules of operation and maintenance unless there is a change in design.

There is no shortage of available advice on how to operate and maintain irrigation canals, either at the general level through maintenance manuals, or through specific procedures laid down in manuals of the different Irrigation Departments. These procedures are available to all parties and there is no technical reason why any particular organization cannot implement them.

There is also no shortage of information on performance parameters, their values and tolerances, that should form the basis of an integrated operation and maintenance program that achieves the desired levels of water distribution equity and predictability that are the hallmarks of a well-managed supply-based irrigation system.

We therefore have to conclude the poor or inappropriate maintenance remains essentially an institutional problem. It is not that the technical knowledge does not exist, it is that people choose not to follow the technical advice. While accepting to some extent this may reflect poor financing of irrigation services, it seems much more a managerial and motivational matter. If performance is not part of the accountability process, then maintenance will not be done effectively.

In this context, improvements will not come because of improved manuals or greater attention to asset management. It has to come through recognition of the need for defined levels of service at each point where water transfers from one organization or group of people to another. If people believe in a level of service as the ultimate objective of an irrigation system, then it will be incumbent upon managers at each level to do whatever is necessary to accomplish that level of service. In the Pakistan context, with large supply-based systems with little opportunity for operational intervention, maintenance will continue to be the key to delivering levels of service that meet the needs and aspirations of all members of the water using community.

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MANAGEMENT OF SALTS IN CLAY SOILS

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ABSTRACT

Management of salinity in drainage waters is a major challenge for growers irrigating crops in arid and semi-arid areas. To successfully manage salinity, growers require knowledge of the salts' origin and how the salts move through the rootzone. In most cases, the main source of salt is the applied irrigation water although in places with high water tables, salts may be brought into the rootzone by capillary rise of shallow groundwater. Traditionally, salts are managed by applying water in excess of crop water requirement so as to leach salts vertically through the rootzone. The leachate or deep percolation carries salts into the regional groundwater, or artificial drainage system for disposal elsewhere. In heavy clay soils, salt management is more difficult as the vertical water movement is constrained by the very small soil permeability. To improve the basis for developing salt management strategies for clay soils, detailed measurements were made of salt movement in the rootzone of alfalfa and sudangrass hay crops grown on irrigated clay soils in the Imperial Valley, California. We found that salt movement was complex in these clay soils. Although a fraction of the salt moved horizontally along the length of the border or furrow irrigated fields, the primary salt movement pathway was vertically through the soil profile. Even when minimizing, or eliminating surface runoff, soil salinity could be managed to prevent an excessive build up of salts in the rootzone and maintain crop yields. Salts may not be the only contaminant in the drainage water and other factors such as sediment load may be of concern. Eliminating surface runoff has additional benefits related to water conservation and disposal of poor quality surface drainage waters.

INTRODUCTION

Irrigation schemes developed in arid and semi-arid areas utilise the climate and soils favorable to growing crops. Irrigation water brought in to supplement sparse rainfall is used in large quantities to sustain plant growth, but results in salts left

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in the rootzone. Over time, the management of these salts become a major challenge for growers.

To manage salinity, growers need an understanding of the origin of the salts and how salts move through the rootzone. In most cases, the main source of salts is the applied irrigation water, although in places with high water tables, salts may be brought into the rootzone by capillary rise of groundwater. The environmental influence of the different ways that salts can move in the soil profile are discussed by Grismer (1990).

Traditionally, salts are managed by applying water in excess of crop requirement and to leach salts through the rootzone. The leachate or deep percolation carries salts into the regional groundwater or artificial drainage system for disposal elsewhere. In heavy clay soils, salt management is more difficult as vertical movement of water is constrained by very low soil permeability. Water and salt movement during irrigation of clay soils is dominated by their cracking characteristics (e.g. van der Tak and Grismer, 1987; Bali et al 1994, Grismer and Tod, 1994 and Shouse et al 1997).

Rhoades et al (1997) monitored salt 'pick-up' by flood irrigation water advancing along the soil surface of cultivated fields with heavy-textured soils and found that salinity increases were remarkably non-uniform, though there was a tendency for salinity to be greater towards the lower section of the field. They expected that this accumulation would result in substantial yield reductions, but they did not quantify salt mass applied, that leaving the field, or crop yields.

To improve the basis for developing salt management strategies for clay soils, detailed measurements were made of salt movement in the rootzone of alfalfa and sudangrass hay crops grown on irrigated clay soils in the Imperial Valley to follow up on previous reports (Bali et al., 2001 & 2001a; Bali and Grismer, 2001; and Grismer 2001a).

RESEARCH METHODOLOGY

The field studies were conducted at the University of California Desert Research Station and Extension Center (UCDREC) located near Holtville, California. Alfalfa and sudangrass hay were grown in two 3-ha fields at two locations (Area 80 for the alfalfa and Area 70 for the sudan grass). Each field had 4 border checks and each border check was 381 m long, 19.8 m wide and on about 0.1% slopes.

The soils of the field used for the alfalfa studies were classified as Glenbar clay loam (moderately slow permeability and very high water available water capacity); Holtville silty clay loam (slow permeability in the clayey and moderately rapid in the underlying material, high to very high available water capacity) and Glenbar silty clay loam (moderately slow permeability and very high available water capacity). The soils of the fields used for the sudangrass studies were classified as Imperial silty clay (slow permeability and very high available water capacity) and Glenbar silty clay loam (moderately slow permeability and very high available water capacity). The soils of the fields selected for the trials are representative of those in the Valley as Glenbar silty clay loam is found on 21% (82,482 ha) of the Valley, while Holtville silty clay is found on 7% (28,572 ha), Imperial silty clay on 12.5% (51,417 ha), and Glenbar clay loam on 0.4% (1,717 ha) (Zimmerman 1981).

Alfalfa (cv CUF 101) was planted in November 1995. The agricultural practices of soil preparation, planting rates, varieties selection, fertilization and pest control followed normal agricultural practices at UCDREC and were typical of agricultural practices found in the Valley, with the exception of irrigation management. Alfalfa hay was cut at approximately 10% bloom and baled at moisture contents of approximately 10-15%. The first cutting was in March 1996. The crop was grown continuously until the final cutting in August 1998. Corn, a more salt sensitive crop, was planted in February 1999 to determine if there were any residual salinity impacts in the field.

Sudangrass hay was grown for three consecutive summer seasons in 1996, 1997 and 1998. In each year, the sudangrass was planted in April and the last cutting was taken in October or November. After the first year, an oat hay crop was grown during the winter season. As with the alfalfa, the agricultural practices used to cultivate the sudangrass hay mirrored Valley wide practices.

Changes to the hydrologic regime at both fields were monitored by measuring the inflow of irrigation water, the advance rate of the surface-wetting front along the border, and tail-water runoff (if any). The field irrigation turnouts were calibrated to establish the pressure head-discharge relationship and then the on-flow of irrigation water was measured using the method described by Tod at al (1991). The reduced-runoff irrigation method for clay soils (Grismer and Tod, 1994 and Bali et al 2000) was used to estimate the duration of each irrigation event, except for the first two irrigations after seed bed preparation when the traditional two-point volume-balance method was used to estimate the irrigation water cut-off time.

Changes in soil water content were monitored using a neutron probe in 8 access tubes located along each border before and after every irrigation and direct analyses of soil samples. The irrigation water was untreated Colorado River water supplied by Imperial Irrigation District and its salinity ranged from 1.05-1.15 dS/m. Depth to the watertable was monitored regularly using observation wells. Salinity changes in the soil profile were determined from analyses of soil samples collected every six months at depths to 2.74 m in 8 locations along each border. Groundwater and drainage water quality was also monitored throughout the three-year trials. Weather data, including rainfall and evaporation rates, were obtained from the CIMIS weather station located on an adjacent field at the Center.

Hay yields were determined by cutting and weighing the crop from representative sample areas (6 m by 0.90 m) located along each border prior to cutting of the entire field, as well as by commercial harvesting methods. The quality (e.g. crude protein, acid detergent fibre) of the alfalfa was also measured from 16 samples taken from bales harvested along the four borders. Full details of the methodology and results of the field trials are given in Bali et al (1999).

Before analysing salt movement in the field, the overall impact of the water management practices used on the field trials is considered. The annual irrigation water applied is summarized in Table 1 along with the hay yields.

Table 1. Summary of annual applied water and hay yields.

Crop	Year	Number of Irrigations	Total Applied Water (m)	Number of cuttings	Yield ($Mg.ha^{-1}$)
Alfalfa	1995	1	0.189	-	-
	1996	10	1.332	8	23.96
	1997	11	1.137	8	15.83
	1998	11	1.106	7	15.08
Sudangrass	1996	7	0.953	3	16.90
	1997	9	1.204	3	14.50
	1998	7	0.899	2	11.90

Project alfalfa hay yields were ~3% greater than average Valley yields during the same period (see Grismer, 2001b). The project irrigation water applied was ~34% less than typical Valley water applications. The frequency of irrigation was reduced from 16/year for the typical Valley to 10/year for the project. Similarly, total alfalfa hay production costs for the project were ~1 % less than typical Valley production costs. Water management costs of the project were ~3 % greater than the typical Valley water management costs, while labor costs were ~13% lower than the typical Valley labor costs (Tod and Grismer, 1999).

The sudangrass hay yields of the project were ~1 % greater than average Valley yields. The project irrigation water applied was ~31% less than typical Valley water applications, though irrigation frequencies were similar. Overall sudangrass hay production costs for the project were ~8% higher than typical Valley production costs, as water costs were ~ 7% higher and labor costs for the project were ~28 % greater than typical Valley costs (Tod and Grismer, 1999).

Considering the soil salt movement during the project, the main source of salt introduced into the system was from applied irrigation water. During the nearly three-year period of the project, 3.79 m of water was applied to alfalfa containing about 25.43 tons/ha of salt. Similarly, for the sudangrass, 3.12 m of water was applied along with 21.91 tons/ha of salt. As there was virtually no surface runoff, clearly, significant quantities of salt were introduced via the irrigation water to the crop rootzone. Additional salts accumulated in the rootzone from upward rise of shallow saline groundwater (Bali et al., 2001 & 2001a). Thus, the question remaining is what happened to all the salt, as there was no apparent adverse effect on crop yields?

Firstly, consider the water moving along the surface of the field during flood irrigation of the border checks. Salinity of the irrigation water moving across the surface only increased slightly (from 1.2 dS/m to 1.4 dS/m) within the initial 100 m from the outlet, but remained fairly constant thereafter, as shown in Figure 1.

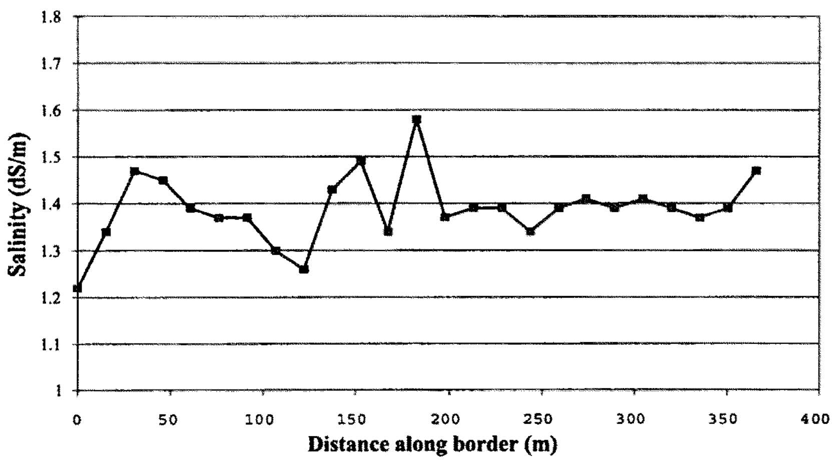

Figure 1. Changes in Surface Water Salinity

The increase or 'pickup' of salts is probably due to the nature of flow near the wetting front. In clay soil, water moves across the surface of the soil peds, and, on reaching a crack edge, cascades down into the crack. The crack fills rapidly as the rate of incoming water exceeds the rate of infiltration into the clay forming the sides of the crack. There is no 'even' wetting front as cracks are interlinked in a mosaic pattern, and as cracks start to fill, some water moves into adjacent, connected cracks. Once cracks fill, water movement is controlled by the low hydraulic conductivity of the clay soil, followed by crack closure as the clay wets and expands. During the filling, the salt deposited on the surface of the crack is washed down into the crack and further into the soil profile. The flow at the wetting front is turbulent and salt is 'picked up' as water moves down the field.

Soil salinity at different depths in the sudangrass field in November 1997 is shown in Figure 2. In general, the salinity of the upper layers (0.15 m and 0.3 m) increases from the head to the tail end of the borders, but salinity of the deeper layers is more uniform across the whole field.

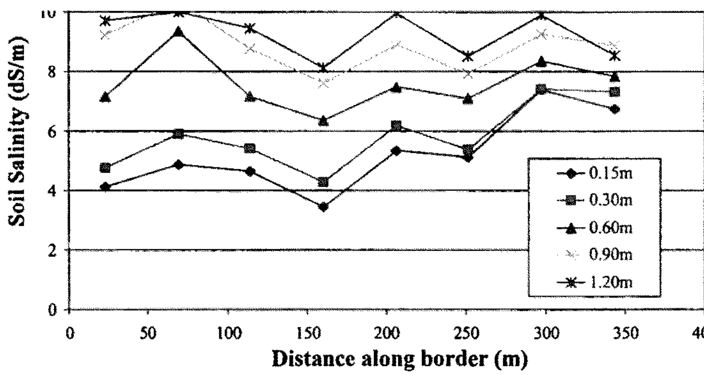

Figure 2. Average Salinity of Sudangrass Field-Nov 97

Average soil salinity in the top 0.6 m along the borders of the sudangrass field during the entire project is shown in Figure 3. Soil salinity increases marginally along the border except during the last year when there was a significant increase towards the lower end after the leaching irrigation.

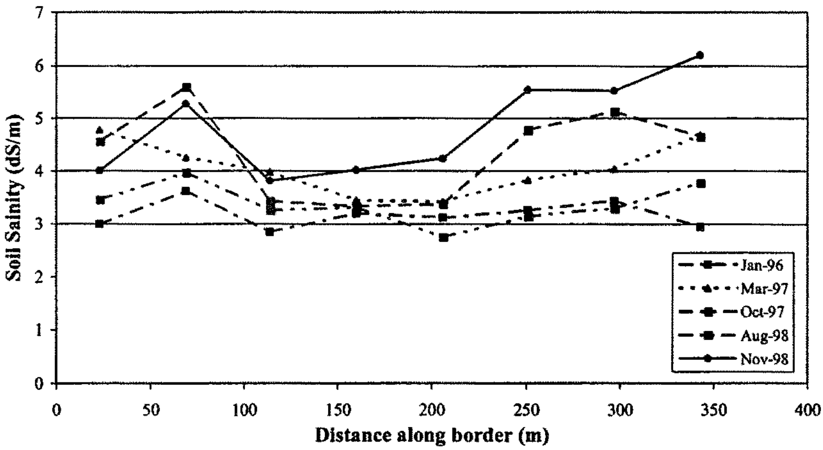

Figure 3. Salinity of Soil Layer (0-0.6 m)

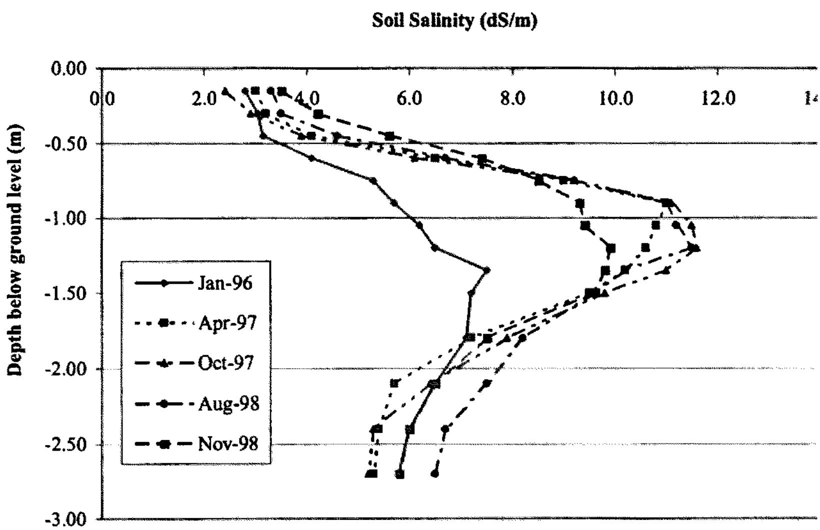

Figure 4. Average Soil Profile Salinity in Alfalfa Field

Movement and distribution of salts in the alfalfa fields were similar to that for the sudangrass as discussed by Bali et al (2001a). Average salinity changes in the soil profile of the alfalfa trials are shown in Figure 4 for the period January 1996 to November 1998. Salts accumulated at the 0.9-1.5 m depth in the soil profile, with soil salinity increasing from 6 dS/m to about 14 dS/m. However, the field was disked after the last harvest and then a leaching irrigation was applied before corn, a more salt sensitive crop, was planted in February 1999 to determine if there was any residual salinity impacts on the fields. The corn grew normally with satisfactory yield and no adverse salinity effects were detected.

CONCLUSIONS

The reduced-runoff irrigation hydrologic regime followed during the three years of the project resulted in hay yields similar to that obtained in the Imperial Valley. Though the studies were conducted on moderately saline fields, the amount of irrigation water applied
to the alfalfa crop was 28% less than that usually applied to alfalfa in the Valley, and tail-water was reduced from a Valley-wide average of about 17% to about 2 %. The soil salinity regime during the project remained favorable for crop production.

The process of water and salt movement through clay soils is complex, but overall the main pathway for salt movement through the soil profile is vertically through the soil profile as the soil dries, cracks and is re-wetted by the application of more irrigation water. Salts appear to be manageable within a field and tail-water is not required to maintain soil salinities that are suitable for crop production.

Salts may not be the only contaminant in surface runoff and other constituents such as sediments or fertilizers may be of environmental concern, as is the case in Imperial Valley. Eliminating surface runoff has additional benefits related to water conservation and problems with disposal of poor quality water. Furthermore, applying less irrigation water and re-assigning the 'saved' water of known quality is economically more efficient than recovering degraded tailwater of unknown quality for re-use as irrigation water elsewhere.

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PESTICIDE CONTAMINATION OF GROUNDWATER ON A COASTAL PLAIN SOIL

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ABSTRACT

The leaching of the herbicides alachlor, atrazine, cyanazine, simazine and metolachlor were investigated on a rainfall simulator study. Treatments investigated included ridge tillage, conventional tillage, conventional tillage with a winter cover crop and conventional tillage with poultry manure. Rainfall events of 10 years, 1 hour duration under dry conditions, (75 mm) 10-years, 1 hour duration under wet conditions and 10 years, 14 days were applied by solid set irrigation five days after the herbicides were applied. Groundwater and pan lysimeter samples were collected one day after the rainfall application. All of the herbicides moved below the root zone by preferential flow after 75 mm of rainfall was applied. Alachlor was detected more frequently than the other herbicides in the groundwater and lysimeters.

INTRODUCTION

Poultry is a $1.58 billion (DPI, 1997) business on the Delmarva Peninsula with 609 million broilers produced in 1997. Since there is a tremendous demand for feed grains in the poultry industry, corn and soybeans are the major crops grown on the Delmarva Peninsula. The three most common herbicides used on corn nationally are atrazine, alachlor and metolachlor. Many of the Coastal Plain soils on the Delmarva Peninsula are loamy sand or sandy loam. The Peninsula has an average rainfall of 110 cm. These soil and rainfall conditions make the watertable aquifer very vulnerable to groundwater contamination. The water-table aquifer generally fluctuates from 0 to 5 m below the surface on many of the Coastal Plain soils.

Since 1984, the Bioresources Engineering Department has been involved in a pesticide transport research. Ritter, et al. (1994) found that atrazine, simazine, cyanazine and metolachlor were leached to the groundwater shortly after they were applied in 1987, but not in 1988. A total of 30.5 mm of rainfall occurred within 9 days of herbicide application in 1987, when the herbicides were detected in the groundwater. In 1988, no rainfall occurred within the first week after

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application. In another three-year study, dicamba leaching to groundwater was measured under no-tillage (NT) and conventional tillage (CT) Ritter et al. (1996). In the first year of the study, dicamba was detected in all monitoring wells at concentrations ranging from 2.0 to 37.0 ug/L 12 days after it was applied. A total of 54 mm of rainfall occurred in the 12 days. The other two years dicamba was detected infrequently in the groundwater because of the rainfall distribution. In both of these experiments the herbicides moved rapidly to the groundwater by macropore flow.

During the 1990's the Bioresources Engineering Department has conducted several studies on the impact of groundwater by herbicides. This paper will summarize results from a rainfall simulation field study.

METHODS AND MATERIALS

Field Experiment

The research site was located at the University of Delaware Research Center in Georgetown, Sussex County, DE. Soils at the research site were classified as Evesboro, Fallsington and Rumford loamy sands, derived from Pleistocene fluvial deposits. (Ireland and Matthews, 1974). The principal aquifer system beneath the research center consists of the sands of the Columbia formation. The aquifer varies in thickness from 27 to 61 m. Depth to the water table generally ranges from 1.5 to 3.1 m. The average transmissivity is about 1000m^2/day. The average storage coefficient is around 0.05. (Sundstrom and Pickett, 1969).

In the fall of 1990, the 2 ha field was divided equally into eight plots, which were subjected in pairs to four tillage treatments. Ridges were established on two of the plots and a winter cover crop of rye and white clover was planted on two other plots in October of 1990.

Treatments used on the plots included ridge tillage (RT), CT with a winter cover crop of rye and white clover and CT with poultry manure applied at a rate of 6 t/ha. The cover crop was planted each year in October and killed with paraquat the latter part of April. Previous research has shown that if sufficient rainfall occurs shortly after some herbicides are applied to sandy coastal plain soils they will move rapidly to the groundwater by macropore flow. The winter cover crop and poultry manure experiments were included because it was thought that increasing the soil organic matter may decrease herbicide leaching. (Ritter et al, 1994).

Corn was planted in May from 1991 to 1993. The herbicide alachlor, simazine, atrazine, metolachlor and cyanazine were applied preemergence after corn planting at rates of 2.24, 2.24, 2.24, 1.7 kg/ha respectively. A solid set sprinkler irrigation on a 9 m x 9 m spacing was installed in the plots several days later. In 1991, five days after the herbicides were applied a total of 75 mm of water was applied to simulate a one-hour, 10-year return period rainfall. In 1992, the same irrigation treatment was used five days after the herbicides were applied. Dry conditions occurred before the irrigation was applied in 1991, while in 1992 the soil moisture was at field capacity before irrigation. The 1993 treatment simulated a 14-day rainfall for a 10-year return period. The irrigation water was applied during three irrigations, with the first irrigation occurring five days after the herbicides were applied.

Each plot had three monitoring wells that were installed for a previous project. The wells were constructed from 3.25 cm diameter PVC pipe and installed to a depth of 4.5 m with the bottom 0.75 m of the wells screened with 0.75 mm slot size screen. A wick lysimeter was installed in each plot at 1 m depth in April of 1991. The wick lysimeter consisted of a 50 cm long wick attached to a lysimeter with a width of 10 cm and the length equal to 60 cm. There were three wicks on each lysimeter that drained to three sample bottles. The design and installation of the wick lysimeters are described by Boll et al. (1991). From 1991 to 1993 samples were collected from the monitoring wells and lysimeters one day after the simulated rainfall.

Groundwater for the rainfall simulation studies were collected with a battery operated peristaltic pump. The monitoring wells were pumped dry and allowed to recharge before samples were collected. The samples were transported on ice to the Bioresources Engineering Department Water Quality Laboratory in Newark and frozen at-18°C until they were analyzed.

Analytical Methods

The herbicides were analyzed by a method that was modified from a procedure developed by the Pesticide Residue Laboratory of Cornell University at Geneva, NY. (Spittler, 1987). The method was developed to analyze large volumes of soil and water samples at a reasonable cost. The water samples were removed from the freezer and allowed to thaw at room temperature and then filtered through a 1.0 micron pore size glass fiber filter. Fifty ml. of the filtered samples were loaded onto a Sep-Pak cartridges at a rate of 3.0 to 4.0 ml/min. The Sep Paks were activated with 5 ml of high-pressure liquid chromatography (HPLC) grade methanol followed by 5 ml of double-distilled water. After the Sep-Paks was loaded and dried 5 minutes with an air aspirator, the herbicides were eluted from the Sep-Paks into a 15 ml graduated test tube with 12 ml of HPLC grade benzene. Samples were concentrated to dryness in a warm water bath under a hood with a gentle flow of high purity N_2 gas. Chlorothalonel spiked HPLC benzene (1.0ml) was added to each dry test tube.

A 1 ml of sample was injected into a capillary column gas chromatograph with a ^{63}Ni electron capture detector. A SPB-15 ml column with a 0.53 mm ID was used. The detector temperature was 300°C. The inlet temperature was programmed to start at 120°C for 1 minute and rise at 8°C/minute to 240°C and hold for 10 minutes. Helium was used as the column gas at a flow rate of 1 m/min. Nitrogen was used as the carrier gas. The minimum detection limit for each herbicide was 0.1ug/L.

RESULTS AND DISCUSSION

In 1991, samples were collected from the monitoring wells and wick lysimeters the day after 75mm of simulated rainfall were applied when the soil was at 50 to 60% of available soil moisture or 6 days after the herbicides were applied to the plots. No atrazine, cyanazine, simazine or metolachlor were detected in the groundwater. Alachlor was detected in 15 of the 24 wells at concentrations ranging from 0.46 to 1.69 ug/L. There was very little difference in alachlor concentrations among the different treatments.

All the herbicides were detected in the lysimeters (Table 1). Alachlor and metolachlor were detected in all 28 samples collected. Alachlor concentrations ranged from 4.36 to 26.9 ug/L. Metolacher concentrations ranged from 0.60 to 4.72. Cyanazine was detected in 7 samples with concentrations ranging from 0.17 to 12.1 ug/L, while atrazine was detected in 2 samples and simazine in a single sample.

In 1992, when soil moisture was at field capacity all herbicides were detected in the groundwater 6 days after they were applied and after 75 mm of simulated rainfall. Alachlor, cyanazine and simazine were detected most frequently in groundwater. Average alachlor concentrations ranged from 0.38 ug/L for CT with poultry manure to 1.91 ug/L for CT with a cover crop. Only two of the samples had concentrations above the EPA drinking water standard of 2.0 ug/L (MDE , 1994). Average cyanazine concentrations ranged from 0.22 ug/L for the RT to 1.14 ug/L for the CT and cover crop. Individual cyanazine concentrations ranged from 0.15 to 3.80 ug/L with 2 samples above the recommended maximum contaminant level (MCL) of 1.0 ug/L. Average simazine concentrations ranged from 0.52 ug/L for RT to 1.99 ug/L for the CT and a cover crop treatment. Only one sample was above the EPA drinking water standard of 4.0 ug/L (10). Atrazine was detected in 12 out of 24 samples with concentrations ranging from 0.10 to 6.31 ug/L. Four of the samples had concentrations above the EPA drinking water standard of 3.0 ug/L (MDE, 1994). Metolachlor was also detected in 12 out of 24 samples with concentrations ranging from 0.28 to 3.29 ug/L.

Table 1. Average Pesticide Concentrations In The Lysimeters Six Days After Application in 1991.

Treatment	Simazine	Atrazine	Alachlor	Metolachlor	Cyanazine
RT	0.45	1.76	9.54	0.81	0.50
CT	0	2.16	6.52	0.98	0
CT(WC)[a]	0	0	9.80	0.70	0
CT(PM)[b]	0	0	6.11	0.58	0

[a] Winter Cover Crop [b]Poultry Manure

In 1992, a total of 16 samples were collected in the lysimeter bottles (Table 2). Simazine was detected in all 16 samples with concentrations ranging from 0.36 to 24.7 ug/L. Metolachlor was detected in only 2 of the samples and cyanazine was detected in 9 of the samples. Cyanazine concentrations ranged from 0.21 to 5.74 ug/L. Atrazine and alachlor were detected more frequently than cyanazine and metolachlor. Alachlor was detected in 10 samples with concentrations ranging from 0.21 to 3.88 ug/L and atrazine was detected in 12 samples with concentrations ranging from 0.36 to 4.14 ug/L. There are several possible explanations as to why cyanazine and metolachlor were detected more frequently in the groundwater than the lysimeters. Some of the herbicide may have degraded sitting in the lysimeter bottles exposed to air before the samples were collected and frozen. The soil was disturbed to install the lysimeters, so the flow paths would not be the same as the entire plots. In general there is more variability in samples collected from lysimeters in the vadose zone than from groundwater monitoring wells.

Table 2. Average Pesticide Concentrations In the Lysimeters Six Days After Application in 1992.

Treatment	Simazine	Atrazine (ug/L)	Alachlor (ug/L)	Metalachlor	Cyanazine
RT	5.34	0.46	0.55	0.11	2.15
CT	1.06	0.44	0.22	0	0.38
CT(WC)[a]	1.50	0.57	0.47	0.06	0.09
CT(PM)[b]	7.93	0.67	1.02	0	1.24

[a]Winter Cover Crop [b]Poultry Manure

In 1993 all the herbicides were detected in nearly all the monitoring wells after an input of the 229 mm of rainfall and irrigation water. Simazine and atrazine had the highest concentrations. Average concentrations for simazine ranged from 1.28 ug/L for the CT to 5.96 ug/L for the CT with poultry manure treatment. Six samples had simazine concentrations above the EPA drinking water standard of 4.0 ug/L and 5 samples had atrazine concentrations above the EPA drinking water standard of 3.0 ug/L (MDE, 1994). The highest measured atrazine concentration was 19.1 ug/L and the highest simazine concentration was 12.9 ug/L. Metolachlor had the lowest concentrations in the groundwater. Average concentrations ranged from 0.16 ug/L for RT to 0.82 ug/L for the CT with poultry manure experiment. Alachlor concentrations ranged from 0.14 to 3.54 ug/L with the highest average concentration of 2.06 ug/L for the CT treatment. Average cyanazine concentrations ranged from 1.47 ug/L for RT to 3.86 ug/L for the CT with poultry manure treatment with 19 samples having concentrations above the MCL of 1.0 ug/L.

A total of 17 samples were collected from the lysimeters (Table 3). There was a great variability in the herbicide concentrations for the different lysimeter bottles. Simazine and atrazine concentrations were the highest in the lysimeters. Simazine concentrations varied from 2.89 to 78.3 ug/L and atrazine concentrations ranged from 1.92 to 82.9 ug/L. Metolachlor was detected in 14 of the 17 samples with concentrations ranging from 0.12 to 3.95 ug/L. Alachlor was detected in all the samples with concentrations ranging from 0.44 to 6.39 ug/L. The variability of cyanazine concentrations was similar to the variability for simazine and atrazine. Cyanazine concentrations ranged from 0.33 to 79.7 ug/L.

Table 3. Average Pesticide Concentrations In the Lysimeters Twenty Days After Application in 1993.

Treatment	Simazine	Atrazine (ug/L)	Alachlor (ug/L)	Metalachlor	Cyanazine
RT	13.1	12.9	4.04	0.45	2.97
CT	32.9	29.1	3.92	1.72	25.5
CT(WC)[a]	4.99	3.68	1.94	0.39	3.20
CT(PM)[b]	14.0	18.6	4.83	0.93	2.32

[a]Winter Cover Crop [b]Poultry Manure

Over the three different rainfall simulation events there was no clear difference among the different treatments. In 1993, four of the herbicides had the highest concentrations in the CT with poultry manure treatment. In 1991, there was very little difference in the alachlor concentrations for the different treatments. In 1992, the CT with poultry manure treatment had some of the lowest herbicide concentrations.

These rainfall simulation studies show that if enough rainfall occurs shortly after herbicides are applied on sandy soils, they will move rapidly below the root zone and may be leached to the groundwater. Most of the movement is probably by macropore flow. Dye and tracer studies on the plots showed that up to depths of 0.6 m, water and solutes moved according to the connective dispersive equation with some preferential movement due to root channels and perhaps some small perturbations of the wetting front. Below 0.6 m two types of preferential flow were active, preferential flow due to large root channels and to coarse sandy layers. (Ritter and Steenhuis, 1994).

SUMMARY AND CONCLUSION

Herbicides moved below the root zone from an intense rainfall shortly after they were applied in the rainfall simulation studies. Alachlor was detected more frequently in the lysimeters and groundwater than metolachlor; atrazine, simazine and cyanazine but the maximum concentrations were generally lower. There appears to be no apparent relationship between herbicide movement and tillage or other cultural practices tested. Dye studies indicated preferential flow occurred on the site due to large root channels and to coarse sand layers.

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MODERNIZING IRRIGATION FACILITIES AT SUTTER MUTUAL WATER COMPANY: A CASE STUDY

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ABSTRACT

In 1999 Sutter Mutual Water Company (SMWC) began an effort to modernize its water-distribution system in an attempt to reduce operation and maintenance costs and conserve water and power resources. The primary technical support was provided by professionals from the Irrigation Training and Research Center (ITRC), California Polytechnic State University (Cal Poly), San Luis Obispo. Additional technical expertise was provided by Concepts in Controls of Visalia, California and Wilson Pumps of Woodland, California. This modernization project was partially funded by the United States Bureau of Reclamation (USBR), Mid-Pacific Region, Northern Area Office, through a Field Services Program Grant and technical support agreement with the ITRC.

The effort encompassed two projects within the company's service area located within the boundaries of California's largest reclamation district, Reclamation District 1500. The projects were (1) the automation of the pumping plant at Portuguese Bend with a new Variable Frequency Drive (VFD) pump and Supervisory Control and Data Acquisition (SCADA) system and (2) the demonstration of new SCADA-compatible electronic flow measurement technologies for both canals and pipelines.

The anticipated, and ultimately realized, benefits of the modernization effort was a savings to the company due to a reduction in the amount of water diverted,

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power consumed and number of personnel required to operate and maintain its system.

INTRODUCTION

Sutter-Mutual Water Company (SMWC), a farmer-owned, non-profit water company, decided in 1998 to begin modernizing its irrigation facilities in an attempt to reduce its increasing operation and maintenance costs while conserving water and power resources. The following paper is a 1999-2001 status report on what has become an ongoing effort.

The work completed to date has been possible due to a coordinated effort between company personnel and professional engineers from the Irrigation Training and Research Center (ITRC), California Polytechnic State University (Cal Poly), San Luis Obispo, Concepts in Controls, Visalia, California and Wilson Pumps, Woodland, California. This modernization project was partially funded by the United States Bureau of Reclamation (USBR), Mid-Pacific Region, Northern Area Office, through a Field Services Program Grant and technical support agreement with the ITRC.

Background

For over 80 years the company has operated and maintained its irrigation facilities, initially using mostly vintage technology, which has proven to be very reliable. In the 1960-1970s, three new pumping stations were built and more efficient turbine pumps were installed to help reduce power consumption and to increase water diversion efficiency. For economic and operational reasons, in 1999 it was decided to begin installing additional technology in some of the plants in order to take advantage of the substantial savings offered by such technology.

The first equipment chosen by the company was a Variable Frequency Drive (VFD) unit and a Supervisory Control and Data Acquisition (SCADA) system, which were installed in the Portuguese Bend plant during 1999-2001. While this project was being completed a second project was also initiated in order to measure flows in two different canals. Both of these projects are described below.

Description of Company

Formed in 1919, SMWC is one of the first water companies to be established in the state of California. The company's physical location (Figure 1) is approximately 45 miles northwest of Sacramento, California and is bordered in the north by the Tisdale Bypass, in the west by the Sacramento River and in the east by the Sutter Bypass. The southern boundary is located at the southern end of Sutter County near the Fremont Weir where the Sacramento River and Feather

River come together. The company's 46,746 irrigable acres (18,917 ha), which are part of the 67,850 (27,470 ha) gross service area that is maintained by Reclamation District 1500 flood control and drainage personnel, are served by approximately 400 turnouts. Approximately 200 miles (322 km) of canals and laterals in the distribution system convey water to the fields.

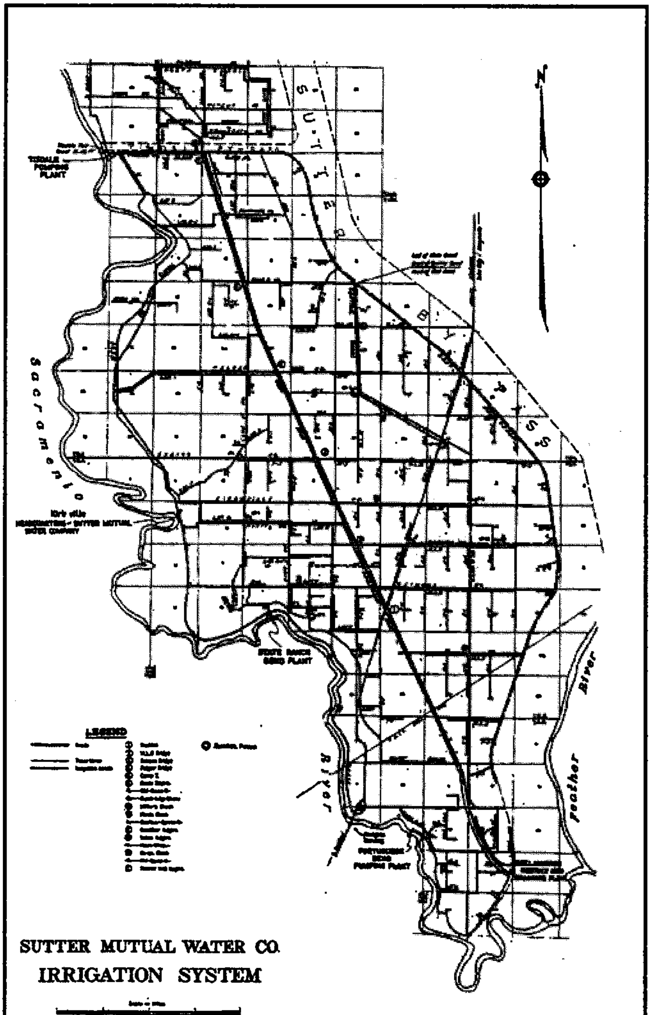

Figure 1. Map of Sutter Mutual Water Company service area

PROJECT #1: INSTALLING A VFD UNIT AND SCADA SYSTEM AT THE PORTURGUESE BEND PUMPING PLANT

In early 1999 the company decided to proceed with the installation of a Variable Frequency Drive (VFD) unit and a Supervisory Control and Data Acquisition (SCADA) system in the pumping plant at Portuguese Bend (Figure 2) following an explanation of the technology by ITRC staff who detailed the work to be done, the equipment to be used and the cost and benefits of the project. The VFD, in a manual mode, was successfully installed by the end of the year after three unique problems critical to the successful operation of the new technology were identified, evaluated and resolved: (1) an adequate radio signal between the office and field site, (2) proper siphon breaker operation, and (3) adequate cooling of the VFD unit. A description of the VFD and SCADA system is as follows.

The Variable Frequency Drive (VFD) Unit

Constant-speed AC motors drive many pumps used for water distribution and delivery at the district or grower level. When flow control is needed to accommodate changes in downstream demand, typically two methods are employed to control the flow rate and pressure: (i) a downstream throttling valve is used to alter the system curve, and (ii) some of the output is by-passed back into the intake.

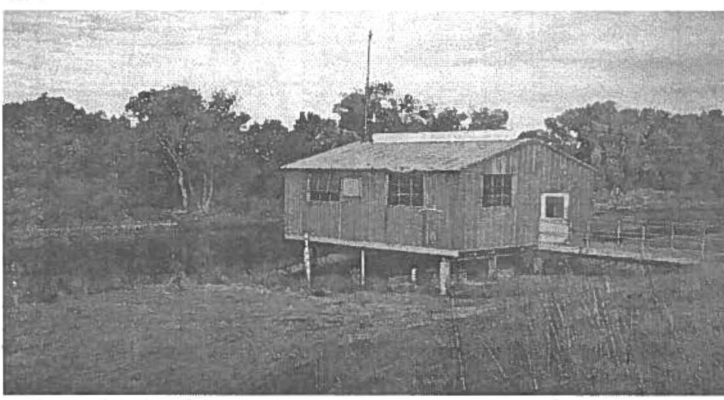

Figure 2: Portuguese Bend Pumping Plant on the Sacramento River

With these two methods a considerable amount of energy could be wasted in doing work that is not needed just to achieve the desired flow rate. VFD units provide an effective way of reducing the speed of the pump drive motor, thereby allowing the flow rate or pressure to be adjusted to the desired level without the additional energy from throttling or by-passing. Basically the VFD is an electronic device that is used in conjunction with a constant-speed AC motor.

The VFD accepts the standard line voltage and frequency then converts the signal into a variable frequency and voltage output that allows the standard constant-speed AC motor to be varied in speed.

Advantages of a VFD: VFDs provide the potential for system automation of pumping plants such as Portuguese Bend. Water level sensors can be used as feedback into the controller to continuously adjust the VFD speed for varying downstream conditions. In general, this permits the ability to provide water deliveries to growers on-demand. In turn, growers are able to schedule irrigations to match the crop water requirements rather than district limitations. This type of VFD operation also offers the potential for labor savings over manual adjustment. The further advantages of VFD systems include the following:

- Softer starting: the device limits the current inrush to the motor providing for a smooth non-shocking acceleration of the pump shaft speed up to its operational RPM
- Elimination of pressure surge: bringing the system up to operating speed slowly removes the pressure surge caused by an almost instantaneous acceleration of the water to its operational flow rate
- Reduction of operating costs: by reducing the energy input over previous control methods (by-pass) operating costs can be reduced
- Reduction of motor stress: reduces mechanical stress on motor windings
- Reduction of peak demand charges: by reducing the energy loads the overall peak demand of the facility can be reduced

Disadvantages of a VFD: There are important issues to consider when VFD devices are being used, as follows:

- Increased motor stress: electrical stress increased due to the steep voltage wave that forms in the power supplied by the inverter. Older motors with inferior insulation may have problems. Typically the motor should be dipped and baked twice. Newer VFDs that include "soft switching output technology" can significantly reduce motor stress and interference from harmonics.
- Increased maintenance: while VFD units are very reliable they are an additional item requiring maintenance. In critical applications, it is essential to have spare parts and maintenance expertise or retain the ability to by-pass.
- Harmonics concern: with the increasing number of control systems going on-line, the line interference produced by some VFD units can cause problems.
- Environmental conditions: most units require relatively dust free enclosures with some type of temperature control. Most of the pumping VFD applications can utilize a simple water-to-air radiator type cooling system (simple and effective).

Energy Savings: VFD units usually reduce pumping costs by reducing the pump drive motor speed to match the desired operating conditions thereby reducing energy input. Without a VFD device this is typically accomplished by using either a by-pass set-up or a downstream throttling valve. The system layout for a typical by-pass installation consists of a pump and by-pass piped into a standtank. With this arrangement the by-pass maintains a constant head in the standtank regardless of flow, as there is less downstream demand. The excess flow is by-passed to the pump intake to maintain a constant head in the standtank or canal. Determining how a pump will operate in a given situation requires an understanding of the pump and system curve. The pump characteristic curve for a standard centrifugal pump shows that the pump, at a fixed speed, has a flow rate associated with a particular pressure; high flow, lower pressure vs. low flow, higher pressure. The intersection of the pump curve and the system curve shows the point of system operation.

If, instead, the pump speed is modified using a VFD device to control head just below the by-pass, flow and head are reduced together along the system curve. A comparison of the relative pump water horsepower with (i) VFD, and (ii) by-pass installations are shown by the shaded areas in Figure 3.

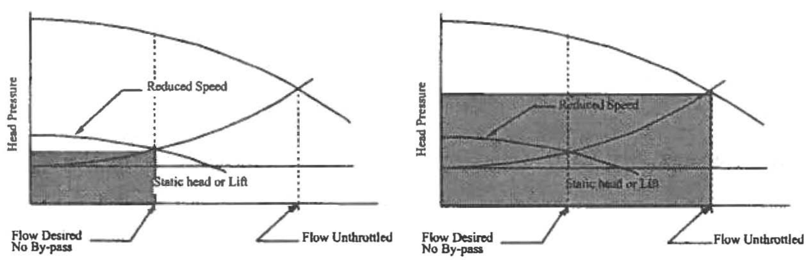

Figure 3. Water horsepower (shaded area) for pumping plant with (i) VFD and (ii) by-pass

However, the water horsepower differences above are only some of several factors to consider. To properly compare the actual cost savings of a VFD system, the overall plant pumping efficiencies with and without the VFD device also need to be considered. The major additional losses that must also be considered in determining the overall pumping plant efficiency are as follows:

- As the system curve changes or the pump speed is reduced, the operating efficiency of the pump changes. Therefore, for each operating point the pump efficiency must be checked.

- VFD units have some losses associated with the conversion process to the new operating frequencies. In general the units are relatively efficient, 95%. But, as the frequency is lowered, some units do become less efficient. The individual specifications for the VFD being considered should be obtained.
- Electric motors if sized properly near maximum loading, can be very efficient. With VFD units used in pumping, the motor loading is reduced as the speed is reduced. This reduction in motor loading can reduce its efficiency and drive motor losses will result.
- Drive friction losses can be reduced with VFD applications. As the speed is reduced the mechanical friction on drive shaft components is reduced.

In addition to the items above, it is important to consider the relative volume pumped each season. Small pumping volumes generally produce small savings and do not justify VFD installations.

Cost Savings Analysis: To determine the total savings due to the VFD unit, a detailed cost savings analysis was begun on the VFD installation on pump #1 (100 hp motor) at the Portuguese Bend pumping plant. Initial savings have already been realized with the reduction of one employee who was needed to constantly monitor and reset the plant's three pumps as dictated by flow requirements out of the plant's main canal. The main costs now under evaluation involve two other components as follows: (1) energy savings as a result of eliminating the by-pass practice (before meter) to control delivery flow, and (2) reduction of spilled water out of the canal, which reduces metered pumping of a purchased volume, plus the additional energy savings associated with the reduction in pumped volume. Both portions of the savings analysis do not take into consideration the specific time of use rates; they are based on the total monthly values. The reduction in canal spill and the associated energy savings are only achievable with the new SCADA system.

Table 1 shows the expected estimated annual energy savings based on the reduced pumping costs associated with a VFD unit. The cost savings analysis is based on an estimate of the average pumping costs before and after the installation of a VFD unit. In addition, an estimate of the average pumping cost with a VFD using a simple control algorithm is included to illustrate the expected additional cost savings during the first year operating with the new SCADA system. Table 2 shows the anticipated annual savings of approximately $2,000 to the company on Portuguese Bend's main canal due to the reduction in spilled water along the canal. Total cost savings from reducing both canal spill and from overall energy cost savings should be between $14,000 and $18,000 per year and will increase even more in the future as energy costs continue to increase.

Table 1. Average annual energy savings expected from VFD operations

Item	Pumping Cost $/AF	Annual Energy Savings based on 6,000 AF/ Season	Annual Energy Savings based on 8,000 AF/ Season
without VFD	4.7	---	---
with VFD	3.0	$10,200	$13,600
with VFD and simple algorithm	2.7	$12,100	$16,200

Table 2. Average annual savings anticipated from a reduction in Portuguese Bend canal spill with the new VFD and SCADA system

Description	Amount
Approximate annual spill at end of canal (acre-feet)	200
Possible reduction in spill with VFD (acre-feet)	120
Water value as missed opportunity to sell ($/acre-feet)	$12.00
Possible revenue from missed sales annually ($)	$1,440
Approximate pumping cost $/acre-feet (from pump test data)	$4.18
Energy savings from reduced pumped volume ($)	$502
Total Anticipated Canal Spill Savings	**$1,942**

The SCADA System

Overview: The basic objective of the automation at Portuguese Bend was to vary the pump flow rates from the pumping plant in order to maintain a target water level in the canal. This required the integration of a VFD unit at the pumping plant and a new SCADA system. Specifically, this involved the ability to remotely monitor the system (water levels, flow rates, pumps on/off, etc.), manually control operations from SMWC's administration office, and to eventually control the system automatically using the new VFD unit. This required the integration of data acquisition components (sensors for water level, electronic flow meters, etc.) with computerized controllers for implementing supervised commands. Monitoring and controlling operations at a remote site such as Portuguese Bend further required a two-way communications network between the remote office location and the control site. Such a system is often referred to as a Supervisory Control and Data Acquisition (SCADA) system.

SCADA is a tool that allows irrigation companies or districts to acquire real-time information and control operations at remote sites from a central location, usually in the main office or at an operations center. By having this real-time information available at the office, the system can also be managed on a real-time basis, thereby providing the ability to achieve maximum water conservation and operational flexibility.

In the water industry, the SCADA systems installed just a few years ago were one-of-a-kind systems custom designed for a specific job. As a result, these systems were not in most cases industrially hardened. Their relatively short-term design efforts did not address all of the day-to-day conditions the components would be subjected to. Consequently, system reliability was low. In addition, the communications protocols were all unique within these proprietary systems; therefore, no interchangeability between components and different vendors was possible. The overall communication systems used also added to the unreliability of early SCADA systems. The older systems typically used lower frequency voice radios for data transmission that were prone to many outside disturbances.

Current SCADA systems are now being designed under a term called "open architecture". This new approach uses off-the-shelf industrially hardened components, which can be linked together using common communication protocols. One such protocol currently adopted by the industry is Modbus. The current systems configuration assembles individual components, called Remote Terminal Units (RTU's), to control or monitor at each site independently. These standard components are then configured (programmed) for the specific task. The site RTU information is then linked back to the central location via radio communication. The open architecture and industrially hardened components have allowed increased scalability and reliability.

Radio communication for the SCADA systems has also improved. Equipment and FCC regulations have allowed the operation frequencies to increase, thereby improving reliability. One notable advance in radio communication has been the FCC approval of a technology known as Spread Spectrum radio. This is an unlicensed 900 Mhz frequency 'hopping' technique that provides reliable communication within about a 15-mile range. The range can be extended with a repeater configuration.

Project Phases: Due to the complex nature of installing a SCADA system into an irrigated area, successful implementation is best accomplished in phases. Initiating change in the routine operation of key facilities and altering the day-to-day activities of company or district personnel can create significant uncertainty. It is therefore necessary that this uncertainty is addressed during each step of the process and a level of confidence is gradually built-up in the participants. Achieving this critical "buy-in" from the people who will actually use the system is essential for the success of modernization projects. This phased approach has the important benefits including maximizing reliability while allowing an irrigation company or district to prioritize critical modernization needs and implement components on a site-by-site basis. Styles et al. (1999) outline further advantages of the phased approach based on experience in modernization projects in irrigation districts. The Project Phases used for installing a SCADA system for SMWC were as follows:

Phase 1 (Completed in April 2001): The first phase of the SCADA part of the project was to install, test and calibrate a new water level sensor in the head of the main canal at Portuguese Bend. The new sensor located in a stilled area at the start of the canal was connected to the RTU/PLC at the Portuguese Bend pumping plant. The new sensor installation was setup so that water levels in the canal were measured once per second and transmitted via radio to the RTU/PLC, where it was stored in a data table and averaged over a one-minute time interval. Upon completion of this task, the Lookout® screens at the district office included information on the canal water levels at two locations, the river stage, the status of each pump (on/off and speed) and target depth (water level setpoint). In addition, the Lookout® screens were configured so that the target depth could be remotely changed from the office (for future automatic control), and so that up to 15 coefficients used for the distributed automatic control could be remotely changed from the office. However, the ability to change these coefficients were "hidden" so that only authorized personnel will be able to change the values.

Phase 2 (Completed in June 2001): This was the first step toward automating the site but nothing automatic was introduced at this stage. The VFD pump was tested on-site in manual mode with occasional remote manual operation. Manual VFD operation means the operator sets the motor speed control using the percent speed control located in the pumping plant. This phase facilitated testing of the new communications equipment, sensors, VFD controls, connection to the office computer, a new air/vacuum relief valve, etc.

Phase 3 (Completed in October 2001): This was the second step toward automating the site. There was a continuation of the remote manual mode of operation, but it was expanded to include a new flow meter. Rather than using only water levels as feedback, the operators now had information on specific flow rates at the pumping plant. A new electronic flow measurement device (Panametrics acoustic meter) was installed on one of the three pumping units and the flow rate was available to the operator. The digital display screens for the new Panametrics meters are shown in Figure 4. This did not mean that operators were expected to make hourly changes from the remote office location. This step required at least two months of operational testing extending into the peak irrigation season.

Phase 4 (Completed in December 2001): This was the third and final step of automating the Portuguese Bend pumping plant. A Proportional-Integral-Filtered (PIF) algorithm for control of the site was programmed into the RTU/PLC and implemented. The control algorithm was a PIF algorithm supplied by the ITRC and not the internal Proportional-Integral (PI) equation supplied by the VFD's manufacturer.

The Lookout® screens in the office necessary to support this automation were already in-place. The ladder logic and additional site programming were completed during this stage. At this time the effect of fluctuations in the Sacramento River level was factored in and added to the ladder logic programming, allowing the minimum VFD speed to shift with the river level. The Lookout® screens were modified to allow a person in the remote office location the ability to shift the pumps to automatic or manual control. In the case of remote manual control, this meant the ability to control the speed of the VFD and the number of pumps operating from the office. This final step allowed for the fullest possible (or desirable) automation of the site.

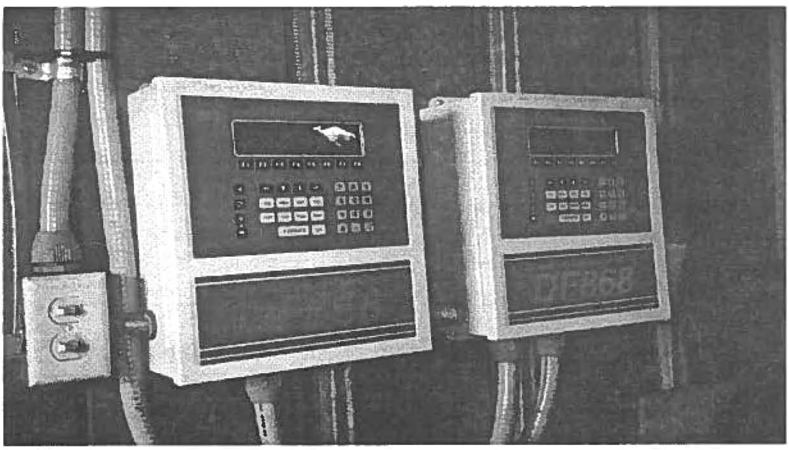

Figure 4. Digital display screens for new Panametrics flow meters inside the Portuguese Bend pumping plant

<u>CANALCAD Modeling</u>: During the modernization effort, the ITRC completed several unsteady flow hydraulic simulations of the first pool of the Portuguese Bend canal were conducted to determine the optimum control scheme for the new VFD unit. The algorithm uses PIF control logic based on water depth measurements 1,800 feet downstream of the Portuguese Bend pumping plant. The algorithm controls that water depth using the VFD and single stage pumps in the pumping plant.

The following is the control logic with optimized algorithm parameters:

VFD pump speed change: DS =1.3 * Round (DQ, 3)
Required flow rate change: DQ=35.315*[KP*(FE1-FE2)+(KI*FE1)]

with: FE1 =fc*FE2+(1-fc)*ENOW
FE2 = FE1 of previous step.
KP = -6.50
KI = - 0.18
fc = 0.84

Simulation Results: Figure 5 summarizes the best modeling results and algorithm for controlling the water depth about three-quarters of the way downstream from the Portuguese Bend canal.

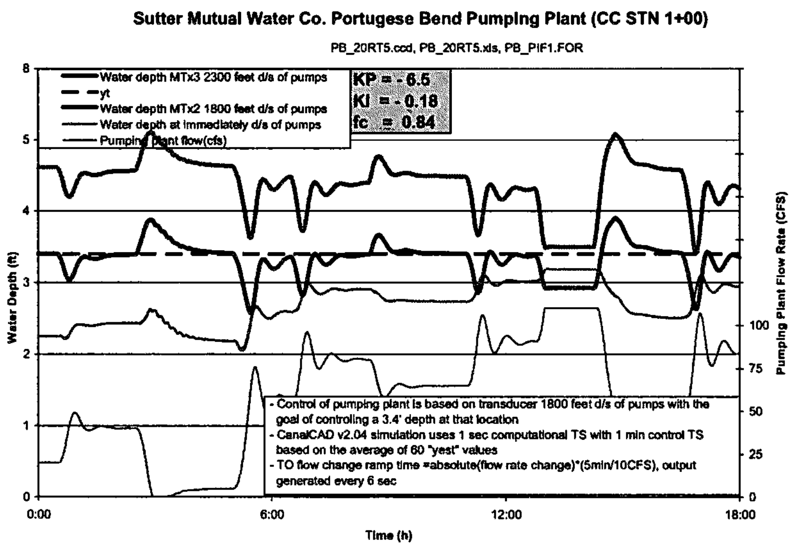

Figure 5. Water level control results when turnout flow changes occur at a rate of 5 minutes for every 10 cfs change

The control action occurs once a minute based on the average of at least 60 measurements of water depth. The graph presents the control results for nine simulated end-of-canal turnout flow changes that range from 5 to 110 cfs and that occur over an 18-hour period. The turnout flow changes occurred based on five minutes per every 10 cfs change in flow.

The graph shows the following information:

- Water depth immediately downstream of the pumping plant,
- Target water level of 3.4 ft at 1,800 ft downstream of the pumping plant,
- Water depth at 1,800 ft downstream of the pumping plant,

- Water depth at 2,300 ft (end of the pool), and
- Pumping plant flow rate

Figure 5 demonstrates satisfactory water level control with frequent flow rate changes over a relatively short period of time using the ITRC selected control algorithm. The target water level to maintain is 3.4[6] ft and the control location is 1,800 ft downstream of the pumping plant. This is the location of two transducers for measuring water depth[7].

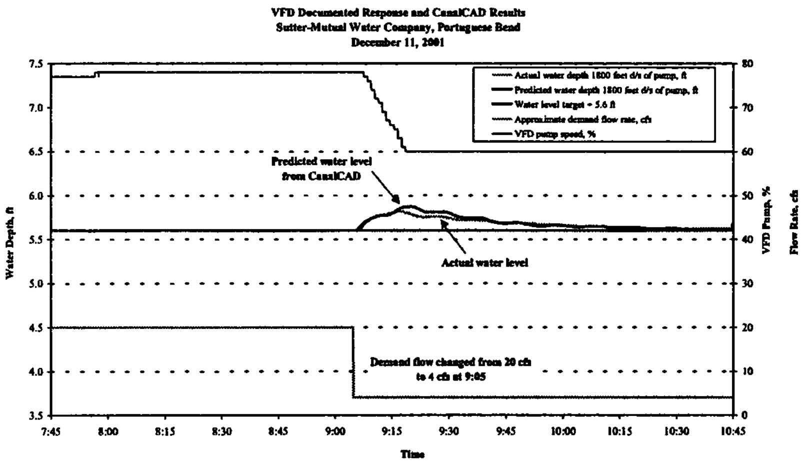

Figure 6. Documented VFD response, water level control results and predicted water depth with an 80% change in demand flow

Documented VFD Response: The documented response of the Portuguese Bend pumping plant from field tests conducted in December 2001 is shown above in Figure 6. During the final evaluation, the demand flow was varied with multiple flow rate changes to test the response time, stability and robustness of the VFD and SCADA systems. The flow changes were made manually by the operator adding or removing weir boards and opening or closing the gate at the check structure located at the downstream end of the first pool of the Portuguese Bend canal.

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[6] The target water level was later changed to 5.6 ft after the canal was de-silted and the sensor height adjusted.

[7] A redundant measurement (Y2) is used to check the integrity of the (Y1) measurement, which is used in the control logic.

PROJECT #2: ELECTRONIC WATER FLOW MEASUREMENT

The second project of the modernization program at the SMWC was the successful utilization of advanced flow measurement technologies. Accurate flow measurement is an integral part of the scientific management of water and energy resources. New electronic flow measurement devices provide a cost-effective and practical means to precisely measure flows at critical locations such as the pumping plant at Portuguese Bend. Panametrics, SonTek Argonaut™ SL and Unidata Starflow ultrasonic flow meters were used to determine flow rates and volumetric flow in the discharge pipeline and canal at Portuguese Bend in conjunction with the evaluation of the new VFD and SCADA system. A brief overview of the deployment of these devices is presented in the following sections.

Testing in the Portuguese Bend Pumping Plant's Main Canal

The new VFD pump permits excellent control of the water level in the first pool of the canal by allowing an unlimited flow rate range. However, in order to correctly program the RTU/PLC of the VFD, the relationship between "change in pump speed" and "change in flow rate" must be known. In practice, this is neither a constant nor a precisely known value. The ITRC used ultrasonic flow measurement equipment on the VFD pump discharge pipeline and pump affinity laws to estimate the relationship between pump speed and flow rate. A Panametrics acoustic flow meter was installed on the VFD pump discharge pipeline in order to integrate real-time flow data into the new SCADA system, in addition to providing flow rate via a digital display in the pumping plant (refer to Figure 4).

Testing in the Tisdale Pumping Plant's Main Canal at the Tisdale Bridge

A SonTek Argonaut™ SL Doppler current meter (Figure 7) was deployed in the Tisdale main canal near Tisdale Bridge from April 18 to July 31, 2001. The canal flow rate was measured in 10-minute intervals and the daily flow volume was calculated. The daily flow volume measured by the SonTek Argonaut™ SL was compared to data provided by SMWC. Drawings of the canal cross-sections at the deployment location were prepared and used in the calculation of volumetric flows. The Tisdale Bridge location is shown in Figure 7 below.

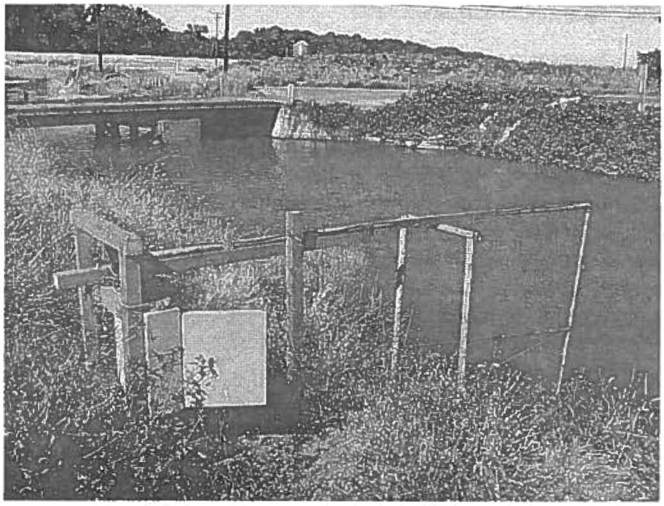

Figure 7. SonTek Argonaut™ SL Doppler current meter at Tisdale Bridge

The percent difference in the measured volume of delivered water ranged from 1.4 to 2.6% per month while the percent difference in total delivered volumes during the four months was less than one percent (–0.9%) as shown in Figure 8. In both monthly and total volumes, the meter registered the slightly higher amount.

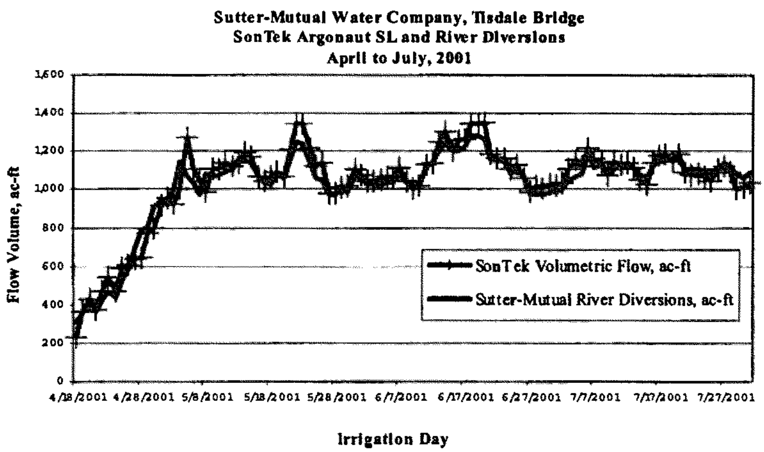

Figure 8. Comparison of Sutter Mutual Water Company and the SonTek Argonaut SL volumetric flows at Tisdale Bridge

Testing at Portuguese Bend Canal

To facilitate the final evaluation testing of the VFD and SCADA system at Portuguese Bend, a Unidata Starflow acoustic Doppler flow meter (Figure 9) was installed in the canal approximately 200 ft downstream of the pumping plant. The Unidata Starflow ultrasonic flow meter provided the total flow rate from the pumping plant during the test period. This was necessary because the new Panametrics meter had not yet been installed on pump #3 (single stage) at the pumping plant. The Unidata Starflow meter was field calibrated using the Panametrics flow meter on pump #1.

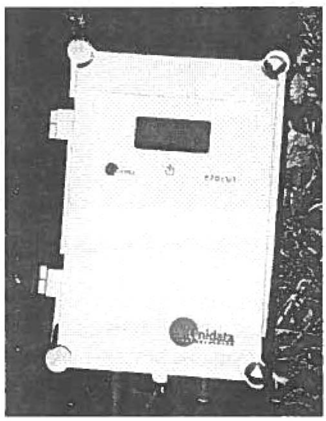

Figure 9. Digital display LCD screen on the Unidata Starflow acoustic Doppler flow meter

SUMMARY

In summary, the modernization effort at SMWC is still continuing in the year 2002 when the company hopes to install a SCADA system in its Tisdale pumping plant. Savings resulting from the installation of the VFD and SCADA system at the Portuguese Bend pumping plant are being closely monitored and already have shown important benefits to the water company and reclamation district as a whole, especially in the area of conserved water and reduced energy costs.

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CASE STUDY: PATTERSON IRRIGATION DISTRICT'S USE OF SCADA FOR TOTAL WATER & ENERGY MANAGEMENT

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ABSTRACT

Making accurate, informed operational decisions in water and energy management can have significant resource and fiscal impacts on irrigation districts. The need for accurate and reliable real-time and historical data is key in making these vital decisions. The use of every acre-foot of water and every kilowatt-hour of energy, resource management, has become the topic of scrutiny in today's world. The protection of these valuable water and energy resources, held in trust and managed by the irrigation district, on behalf of its' landowner constituents, is one of the vital functions of the Patterson Irrigation District (PID). Plant Control and Supervisory Control and Data Acquisition (SCADA) systems can provide the link between data and effective District operations and management.

This case study will outline the initial development, expansion and subsequent upgrade of the Patterson Irrigation District's Plant Control and SCADA systems, the role in data acquisition and daily district operations, the benefits the district and its water users have accrued from accurate real-time and historical data and finally, the lessons learned in the development, implementation and evolution of a state-of-the-art Plant Control and SCADA system for irrigation district use.

In its first full year of operation, 1999, historical data verified an increase of 23% in total Station #1 pumping plant efficiency on a kW-hr per acre-foot basis.

INTRODUCTION

Patterson Irrigation District holds pre-1914 water rights on the San Joaquin River which serves as its major source of irrigation water supply. The water diverted from the San Joaquin River must be pumped and lifted through a series of five – (5) pumping plants. Pumping Plant #1 at the San Joaquin River consists of seven –(7) pumps ranging in flow from 15-35 ft^3/sec (0.42 – 0.99 m^3/sec), typically lifts the water 28-35 feet (8.5-10.7 meters) and its downstream pool length is 7,300 feet (2,224.9 meters). Pumping Plant #2 consists of seven – (7) pumps ranging in flow from 11-44 ft^3/sec (0.31-1.25 m^3/sec), typically lifts the water 11-12 feet and

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its downstream pool length is 5,400 feet (1,645.8 meters). Pumping Plant #3 consists of six – (6) pumps ranging in flow from 6-44 ft^3/sec (0.17-1.25 m^3/sec), typically lifts the water 11-12 feet and its downstream pool length is 4,200 feet (1,280.1 meters). Pumping Plant #4 consists of five – (5) pumps ranging in flow from 7-30 ft^3/sec (0.20-0.85 m^3/sec), typically lifts the water 11-12 feet and its downstream pool length is 3,000 feet (914.4 meters). Pumping Plant #5 consists of four – (4) pumps ranging in flow from 7-21 ft^3/sec (0.20-0.59 meters), typically lifts the water 11-12 feet and its downstream pool length is 650 feet (198.1 meters).

Water can also be introduced into the Main Canal system from the Delta-Mendota through a concrete monolithic gravity pipeline system. This pipeline system runs parallel with the Main Canal on its north side and allows water to be added into the system at any reach or any lateral. The Main Canal supplies irrigation water to 13 laterals as shown in Figure 2.

The size of the district is 13,500 irrigated acres (4,925.2 ha) and the capacity of the main pumping plant is 180 cfs (5.1 cms). There are 772 turnouts approximately 49 miles (79 km) of open laterals and 84 miles (135 km) of pipeline sub laterals. The District has over 450 landowners and 625 water users with an average irrigated field size of approximately 20 acres (7.3 ha), with only five –(5) individual irrigated fields over 100 acres (36.5 ha). The flow rates and demands on the main canal pumping system are ever changing making it very difficult to manually operate the Main Canal pumping system for optimum efficiency and water level consistency.

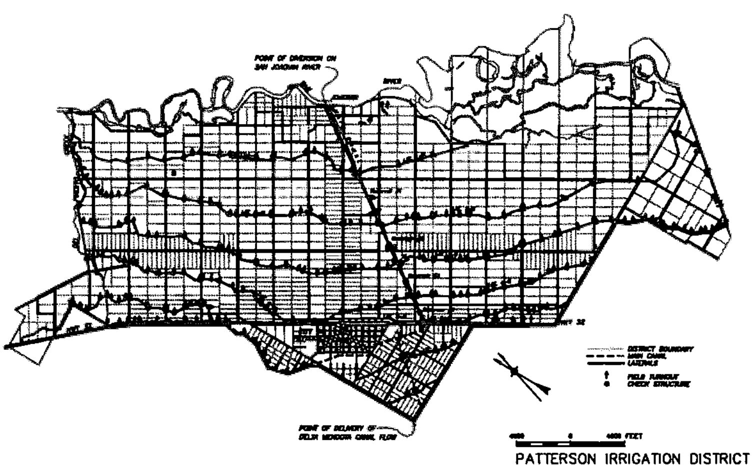

Figure 1. District layout.

INITIAL AUTOMATION FAILURE

Patterson Water District (now PID) installed a proprietary automated pump control system on its Main Canal prior to the 1996 irrigation season. The original automation scheme was never modeled for hydraulic stability, therefore it did not provide hydraulic stability. In addition, this original automation system was not scalable, did not provide for remote monitoring or control, did not gather or collect data and was not designed with readily available off the shelf parts and materials. The system was operated in 1996 and very shortly in 1997. The system flaws and short-comings created frustration with field personnel and doubt about the feasibility of automation at the Board level. Due to constant changes in water levels, pumps kicking on and off, lateral flows varying greatly, general system alarms being triggered constantly, and multiple flooding occurrences the automation system was scrapped and the pumps were once again operated manually with still no remote monitoring. Typical manual operations provided for controlling the pool depths just over the operational spills allowing 5-8 cfs (0.14-0.23 cms) to spill to the next lowest pool and be re-circulated. Although not energy efficient, this operation scheme provided the best fit between existing pumping flexibility and manual system operational errors.

CONTROL SYSTEM BEGINNINGS

In the beginning of the 1997 irrigation season District staff presented the District Board with a phased modernization plan and received the go ahead to begin modernization. The plan consisted of the District working with the Irrigation Training and Research Center (ITRC) and the United States Bureau of Reclamation (USBR) to modernize and automate key District facilities, primarily the Main Canal pumping and distribution system. The District would receive technical support and expertise from the Irrigation Training and Research Center through its contract to provide technical assistance on behalf of the Mid-Pacific Region of USBR to its water service contractors. This support included hydraulic modeling in CanalCAD and development of control algorithms for the Districts pumping plants. In addition, the District would apply for USBR grants through the Mid-Pacific Region Field Services Program. The District dedicated field and technical personnel plus significant capital resources to implement the modernization plan.

The first phase consisted of the automation of Pumping Plant #1 including a Remote Terminal Unit (RTU), Spread Spectrum Radio communications, a variable frequency drive (VFD) on the largest flow rate pump, SCADA software and a Main SCADA computer terminal at the District Office. All hardware and software utilized was open architecture, industry tested, readily available and manufacturer supported. The new RTU at the pumping plant was hard wired into the pump contactors, water level sensors, and level cutoffs. The RTU processed information, controlled pumps (including the VFD) and provided real-time

monitoring, alarms, and supervisory control at the District office. All pump control programming and logic resided in the RTU. This eliminated the possibility of a computer, radio communications or office power problem from halting or interfering with automatic system operation. The office computer terminal acted only as a monitoring and data gathering station. In the event of office system failures the worst that could happen was lack of remote monitoring as the automation system continued to work. A basic layout of the initial SCADA concept is shown in Figure 2. A key safety component of the automation system was an emergency level cutoff switch in the return canal. This switch would shut-down operations at the pump station in the event the return canal water level became dangerously high. The RTU would send an immediate alarm to the SCADA terminal notifying the District of the occurrence. The SCADA terminal would also send a text specific alarm message to alpha-numeric pagers. These pagers are carried and monitored by two District field employees at all times including weekends, nights and holidays. This would allow the District to react immediately by shutting down other pumps stations, troubleshooting the problem and getting the system back up and running quickly and safely.
In its first full year of operation, 1999, historical data verified an increase of 23% in pumping efficiency on a kW-hr per ac-ft basis with the automation of Pumping Plant #1 utilizing a VFD and a downstream controlled algorithm. This reliable control and monitoring system also allowed the District to divert pump operator personnel hours to other vital maintenance and District operational needs. Project results included money saved, efficiencies improved and a shift in labor to other pressing District system needs; thereby improving service and reducing costs to the customer.

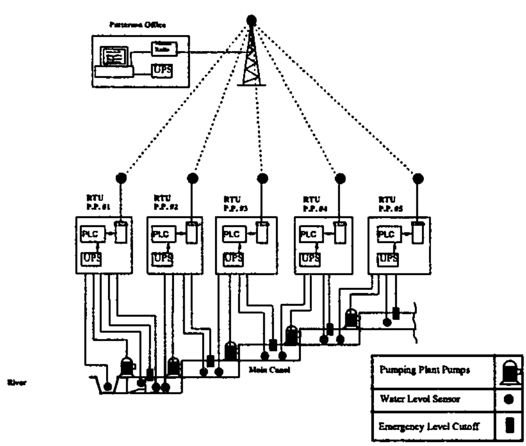

Figure 2. Plant Control and SCADA system layout.

THE INTEGRATION APPROACH

The District chose to implement the first phase of the control systems using an integrator. An integrator is a person or entity familiar with all aspects of such systems including hardware, software, communications, RTU programming, Human-Machine Interface (HMI) software programming, computer set-up, etc. An integrator was solicited to provide a cost estimate based upon a project concept and on a list of system desires and functions. In this case the ITRC helped the District find an integrator and the District, the ITRC and the integrator sat down and developed the "project scope". The District performed all the field work within its collective employee's abilities such as trenching, laying conduit, installing junction boxes, pulling signal cable and installing stilling wells and pressure transducers. District staff also learned to program the HMI software allowing the District to use its own labor for some initial programming debugging and subsequent additional system function desires. The ability to program the HMI software allowed District staff to set-up an alarming system using alpha-numeric pagers and develop data logging applications which tracked water diversions, electrical usage, pumping hours, water elevation data, pump starts/stops, etc.

The approach requires adequate technical expertise and understanding at the District level to manage all aspects of the project construction and integration. The District must also decide on the level of detail it can live with regarding project documentation such as material lists, catalog cuts, equipment and fabrication layouts and details, schematics, wiring and interconnection diagrams, software and hardware programming printouts and electronic copies, system and component test reports, operation and maintenance manuals and as-built drawings. In the District's experience adequate documentation is a key component to a successful project and system. As there are many integrators, electrical contractors, and engineering companies who service, design and upgrade these systems documentation will provide flexibility when needing additional control system services. Money and man hours saved on non-inclusion of documentation in the beginning of a project can be costly and time consuming when service, troubleshooting, component replacement, modifications, additions and upgrades are desired in the future, as somebody has to figure the system out to do anything with it.

ADDITIONAL SCADA PHASES

By using the open architecture system approach, the District was positioned to expand on its successful first phase. Prior to the 2000 irrigation pumping season, using Pumping Plant #1 as the model, Pumping Plants #2 and #3 were automated and tied into the SCADA system. With some gathering of water elevation data and continuing to work with the ITRC, the first three pumping plants were working in unison and debugged in the first couple months of the pumping season. The District continued to operate Pumping Plants #4 and #5 by hand at the plants, but there were now no full-time staff dedicated to pump operations. The pump operation duties were shared by the water department personnel.

Prior to the 2001 pumping season the District added five -(5) solar powered remote RTU sites along the Main Canal. These sites provide the District with monitoring of all the flows in main laterals coming off the Main Canal; either by 4-20 mA output signals received from McCrometer open flow propeller meters or 4-20 mA output pressure transducers reading water elevations over broad-crested weirs. By logging volume data the District can now look at individual lateral delivery efficiencies on any time basis and move toward further delivery efficiency improvements where needed. The District can also monitor flow and remotely operate its three deep wells on its main canal system and gather and store volume data for any time period reference. This reduces field personnel time needed to travel and perform these tasks as well as staff time needed to update historical diversion spreadsheets. While trenching and installing conduit and signal cable for the pressure transducers on each lift the District pre-wired for future use of gate actuators at the heads of the main laterals.

The District also added a sixth remote RTU site for monitoring and control of its turnout on the Delta-Mendota Canal. This site is 5 miles from the District office and was equipped with an old flow meter that did not have instantaneous flow reading capability. The site is now equipped with two –(2) digital display McCrometer open flow propeller meters and two –(2) Limitorque electric gate actuators. The gates can be operated both remotely and on-site and have full flow control capability. The District now has valuable real-time data allowing more precise and frequent analysis of data for water and energy management purposes without leaving the office.

The District favors the phased implementation approach for a small District like PID. This phased approach served the following needs of the district:

- Allowed the system to be implemented in stages providing maximum reliability and benefits to the district. Each stage can be evaluated and debugged before the next step is started.
- Economics – as capital costs for SCADA systems can be rather large, phasing these costs in over a period of years can provide less "sticker shock" than doing a complete, sophisticated SCADA system at once.
- Allowed the district to become familiar with the system one piece at a time
- Allowed the district to maximize its in-house personnel for installations as they became familiar with the equipment and the system from phase to phase
- Allowed the District to maintain maximum flexibility with critical site installations on an as-needed basis.

UPGRADING TO INTELLIGENT MOTOR CONTROL CENTERS

In the Spring of 2000 the District hired an electrical engineering firm to complete a system study of the District's Main Canal Electrical Distribution System. The District receives preference power from the Western Area Power Administration (WAPA) for its pumping loads along the Main Canal. The interconnection point with Pacific Gas & Electric Co., which provides transmission of the District's WAPA power, is at Pumping Plant #2. From this interconnection point the District owns its own distribution lines and wires which travel east to Pumping Plant #1 and west to service Pumping Plants #3 through #5. The study documented installed hardware, including motor control centers, power factor correction capacitors and the pole mounted transformers (installed on concrete pads on the ground) used to transform the power from 11.47 kV to 480 volts. As may be the case in many older Districts some records were not kept up as well as others. The study also looked at reliability, remaining useful life and reliability of all system components. The study clearly showed neglect and age was quickly catching up with the electrical equipment and reliability would quickly become an issue. Safety issues were also identified. Of main concern was the remaining useful life of the 1960's vintage GE Motor Control Centers, the ineffectiveness of the existing capacitors and the safety and liability issues involved with the transformer installations.

As the District is almost entirely reliant on its Main Canal electrical and water system to accomplish its water deliveries, replacing the Motor Control Centers (MCC) and the transformers were made top priorities. In developing the specifications for new motor control centers, the District expressed interest in more advanced intelligent motor control centers with soft start capabilities, a central Programmable Logic Controller (PLC) and both power monitoring and radio communications capabilities. Included in the specifications were desired documentation, warranties, standards, enclosures and bus, electrical ratings, solid state motor controller alternatives, power factor correction capacitors, circuit breakers & protectors, digital multifunction meter to monitor the incoming WAPA power, and PLC requirements. Using these specifications requests for bids were sent out to major manufacturers for replacement of the existing motor control centers at Pumping Plants #1 and #2. Bids were received from only two manufacturers. Of the bids received it was evident that the standout product and also the most cost effective was the IntelliCENTER line by Allen-Bradley, Rockwell Automation. The IntelliCENTER was a complete hardware, software and communications system fully configured and tested, as close to plug and play as we could get, a complete solution which met the District's needs as outlined in the specifications for bid. The District was sold on the IntelliCENTER's as a complete product including the communications and software which would allow integration into the Districts current SCADA system monitoring and controlling main canal operations. Considering all factors the Board approved the purchase of the IntelliCENTER's for Pumping Plants #1 and #2 in late 2000. The District concurrently awarded the contract for the MCC installations. They were installed and ready for operation prior to the 2001 irrigation season.

INTELLICENTER SOFTWARE

One of the main features of the IntelliCENTER system is the IntelliCENTER software which provides the following:

- preconfigured screens which provide MCC line-up with nameplates and indicators so users can get needed information at a glance (Figure 3)
- Individual unit views allow for easy changing of parameters to be viewed and monitoring (Figure 3)
- Allows MCC monitoring locally or remotely; using Ethernet, ControlNet and DeviceNet
- Optimized polling to ensure software system performance, real time status and historical trending
- AutoCAD documentation including unit wiring diagrams, elevation drawings and single line drawings
- User Manuals
- Event Logging such as warnings, faults and parameter edits

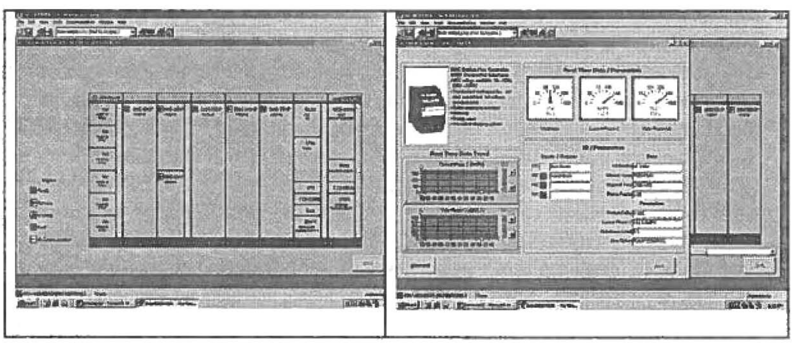

Figure 3. MCC Line-up showing nameplates, indicators and individual unit view allowing monitoring, parameter viewing and editing.

Monitoring the MCC's in real time would provide the district with real answers on a SCADA PC as opposed to locked in an enclosure in the field, such as in the event of an MCC or unit failure. This would simplify troubleshooting and minimize downtime and eliminate the need to have a dedicated electrician at the District. With the IntelliCENTER software, User Manuals, drawings, diagrams and parts lists, existing District personnel can handle maintenance and troubleshooting of the MCC's.

SCADA RTU AND MCC PLC CONNECTION

As the canal control equations and logic were still housed in the original RTU's the District was at a crossroads. Should the District move all canal control programming into the MCC PLC? This would eliminate the need for two PLC's, and the associated software knowledge maintenance requirements. As the District was comfortable with the current RTU's from the prior year and the current irrigation season was upon us, the District decided to postpone moving the canal control programming from its current RTU. It was originally intended that the digital I/O from the RTU would be hardwired to the MCC and continue to monitor and control pumps as it had prior. This would likely have been the best idea, but the District chose to have an integrator develop a cable connection and have the existing RTU poll and direct the MCC PLC for pump status and control commands via Modbus communications. As it turned out having two PLC's communicate over a cable connection was not in our best interest. There were some bugs in the new I/O addressing and Ladder Logic modifications. The District learned to live with the limitations created by this decision and knew there would be time to correct the problem after the irrigation season.

UTILIZING THE INTELLICENTER'S

Throughout the first year of operation the IntelliCENTER's proved their worth. As the year 2001 began it was clear California was facing uncertainty in the electric utility arena due to problems associated with the States deregulation of energy markets. In response to these conditions the California Energy Commission (CEC) began an energy conservation grant and funding assistance to encourage irrigation and water districts to improve efficiencies and reduce loads. The District participated in this program by testing all pumps on its main canal for pumping plant efficiency. The IntelliCENTER ability to display load information made it extremely simple, safe and quick to determine pumping loads at Pumping Plants #1 and #2. In contract it was difficult to obtain good load data at Pumping Plants #3 - #5 as hand held power meters were required using complicated procedures. There were also safety issues as the 480 volt panels had to be opened while charged and two sets of hands hand to be used to make all required connections. The poor condition of the capacitors at Pumping Plants #3 - #5 also proved a hindrance in collecting good load data, as power quality was a problem. As the District has approved the installation of IntelliCENTER MCC's at Pumping Plants #3 - #5, pump testing will be redone at those plants. Obtaining load data in the future for frequent pump testing of all Main Canal pumps will be easier, safer, more reliable and far less time consuming than in the past.

As with any complicated electrical device, the District did experience problems with individual MCC units and motors. The soft starters proved valuable in displaying fault conditions such as thermal overload, current imbalance, device status, warning status, trip history, etc. Looking up fault conditions in the user manual, helped guided District personnel through the proper procedures to get the motor back up and running or determine if there were greater problems in need professional attention. In the past if a motor were to trip, standard procedure was to reset the breaker and start it back up. Only call for help if you smell smoke or see fire. It is anticipated that the IntelliCENTER system will help the District better protect motors and electrical panel equipment by pinpointing problems that may otherwise have gone unnoticed and attending to them at the proper time – not upon complete failure.

UTILIZING INTELLICENTER'S FULL CAPABILITIES

Going into the 2002 irrigation season, the District will perform the following tasks to allow the District to utilize the complete capabilities of the IntelliCENTER's for total water and energy management:

- Install IntelliCENTER MCC's at Pumping Plant #3, #4 & #5; PLC's will be Control Logix with Ethernet Radio capabilities
- Add a Power Monitoring SCADA PC and Terminal at the Office which will run IntelliCENTER software and monitor all MCC's via Ethernet Radio communications
- Upgrade the PLC's at Pumping Plants #1 and #2 to Control Logix and add Ethernet Radios and antenna's
- Add new Modified Bi-Val Control (Proportional Integral with Filter) algorithms developed by the ITRC using CanalCAD to IntelliCENTER Control Logix PLC's at Pumping Plants #1, #2, #3, #4 and #5.

The District also had to modify its existing office network (Figure 5) for receiving of both serial and Ethernet radio communications. Ethernet communications are required at the pumping plants to make this total SCADA system effective for real-time remote operation. High bandwidth and frequent polling is required as there are 5 MCC's with 29 individual motor buckets with over 100 I/O points to monitor for each bucket. This is in addition to all the water system data being polled. The remote RTU sites will continue to communicate via Modbus (Figure 5). The original RTU equipment removed at Pumping Plants #1, #2 and #3 will be saved and used as spare parts for existing remote RTU sites or utilized for the creation of additional remote SCADA sites in the future, such as lateral spills or a proposed tailwater recovery reservoir.

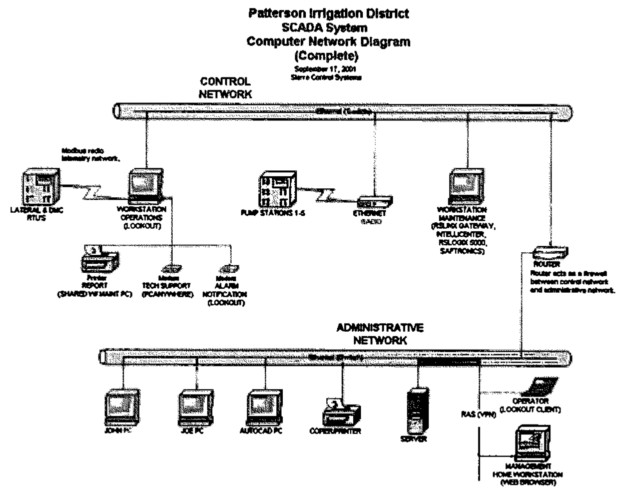

Figure 5. SCADA System Network with Firewall Router

SUMMARY

This paper documents the development of a Plant Control and SCADA Systems using a phased approach. These systems have changed the way water distribution systems can be run and automated the once manual arduous task of gathering vital field data used to make resource management decisions. In this case a Plant Control and SCADA system integrated into an intelligent motor control center, combined with the technical expertise to develop effective downstream canal control algorithm's, has proved to modernize a small irrigation district, improve distribution system operational efficiencies and help it manage its resources. The remote monitoring functions of SCADA also allow for real-time reaction to system malfunctions and shut-downs which in the past may have gone un-noticed.

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www-BASED MONITORING AND CONTROL FOR OVER-STRESSED RIVER BASINS

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ABSTRACT

The Bureau of Reclamation (Reclamation), StoneFly Technology (StoneFly), Utah State University (USU), and several Utah water user groups are working to create "virtual" river basins on the Internet. Websites with accurate real-time representations of the Sevier, San Rafael, and Duchesne river basins are being developed. These river basin websites, coupled with low-cost automatic remote-control on all major structures, allow for nearly instantaneous decisionmaking. The ability to see what is happening throughout a river basin and to react promptly to changing hydrologic and weather conditions is dramatically improving the way rivers and irrigation canals are operated. Additionally, these websites are important elements in building trust and encouraging collaboration between the various stakeholders.

INTRODUCTION

In the 19th Century, most irrigation systems in the Western United States were run-of-the-river, and they frequently ran short of water in the late summer and fall as the natural flow of the river declined. In the 20th Century, significant man-made water storage became an important ingredient in water resource development projects, and there was a concerted effort to match water supply and demand. When new water storage and distribution systems were constructed, the complex interactions and impacts associated with modifying the natural behavior of river systems and the challenge of sustaining water resources, in an environment of increasing demands for water, were not always foreseen or understood. But in the 21st Century, the American public is clamoring for intense operation of river basins to deliver the right amount of water at the right time.

What is therefore needed is a coordinated effort to operate existing facilities with a basin-wide perspective in an optimal fashion that balances diverse objectives: (1) meeting increasing municipal and industrial (M&I) water needs;

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(2) maximizing power generation; (3) improving water quality; (4) meeting in-stream flow and other biological requirements; (5) protecting endangered species; (6) enhancing recreational opportunities; (7) improving aesthetics; and (8) addressing public safety concerns, all while protecting the viability of the agricultural sector. The authors feel that with more intensive management, it will be possible to meet a wide range of multi-objective goals without too strongly penalizing any one user or interest.

Reclamation's Provo Area and Denver Offices have been working with StoneFly, USU, and several Utah water user groups to develop low-cost real-time monitoring and control systems to address the need for improved operation of existing river basin control facilities.

TECHNOLOGY DEVELOPMENT AND APPLICATION

One combination of technologies that can enhance the way river basins are managed is the "virtual" river basin: an accurate, real-time representation that can be displayed over the Internet. Critical parts of a "virtual" river basin include: (a) comprehensive real-time environmental monitoring system (including real-time images); (b) low-cost automation systems; (c) Internet displays that provide accurate real-time visualizations of basin conditions; (d) enhanced and alternative methods of real-time database access; and (e) decision-support software. We have made significant progress with items (a) and (b), and have a good start on (c), (d), and (e). Results to date show that better and timelier information leads to better decisionmaking, and with automatic remote-control, required actions can be quickly taken. The "virtual" river basin thus helps meet the growing need for a constant and precise matching of water supply and demand.

Real-Time Environmental Monitoring

In the 1990's several Utah water user groups began programs to closely monitor their irrigation canals and rivers. This was accomplished by adding dataloggers and radio telemetry equipment to existing flow monitoring sites, water quality monitoring sites, and weather stations. These diverse smaller projects eventually coalesced into something approaching basinwide monitoring systems (see Table 1.) These monitoring systems proved useful for improving water management, particularly with river commissioners and the larger canal companies.

Low-Cost Automatic Remote-Control

The dataloggers used for the above monitoring systems had the ability to control as well as datalog. This opened up the possibility of automatic remote control on all major water control structures. Unfortunately, at the onset of these projects, there was little to control as most gates were operated manually. One major obstacle to adding motors was the lack of affordable commercial power.

Table 1. Utah River Basin Websites
General Information

River Basin	Website (www.-)	Since	Sponsor	Water Monitor/ Control	Weather/ Webcam	Total
Sevier (including San Pitch)	sevierriver.org	1998	Sevier River WUA*	19/21	4/5	49
San Rafael	ewcd.org	1999	Emery WCD**	60/4	5/2	72
Duchesne/ Strawberr y	duchesneriver.org	2002	Duchesne/ Strawberry WUA*	5/5	1/0	11

Another obstacle was the dearth of commercially available gate actuators that could be easily solar-powered. Reclamation developed several designs for low-cost, 12-VDC gate actuators that can be easily retrofitted onto existing slide and radial gates, (Hansen et. al., 2001). In the meantime, several gate actuator manufacturers developed their own 12 and 24-VDC models that can be powered by small solar panels and deep-cycle batteries. These developments have made the jump to automatic remote control very doable and cost-effective. Today, every major water control structure in the Sevier River Basin is fully automated, most sites using solar-powered gate actuators.

River Basin Website Development

In the past, monitoring and automation systems generated substantial amounts of data, but it was unavailable to all but a few water managers. This was a constant source of frustration to the excluded water managers and others who needed the information to improve their operations. Meanwhile, the rapid rise in the development and use of the Internet meant that many water managers were either getting "on-line" or considering it. It became apparent that getting the real-time data onto the Internet would be a good way to distribute the information to a wide audience without requiring the purchase of specialized equipment.

In 1997, StoneFly developed a plan to connect the real-time databases to the Internet. Reclamation agreed to assist with the project. The initial effort began in 1998 for the Sevier River Basin, Utah. (The development of the Sevier River website is being partially funded through a grant from the Technology Opportunity Program of the U.S. Department of Commerce). A second website was installed for the San Rafael River system in 1999. By the start of the 2002 irrigation season, a similar site has been installed for the Duchesne Rivers system.

Any water manager or interested individual is now able to sit down at a computer and survey hydrologic and weather conditions throughout these three river basins.

The websites created by StoneFly are designed to serve a variety of users with a variety of displays. The log-in page gives the user several options. One popular display gives hourly flow data for the previous 7 days (see Fig. 1). Current river and canal flow information is displayed in spatial diagrams (see Fig. 2). Another popular display shows the real-time status of all major reservoirs throughout a Basin (see Fig. 3). Web cams are being integrated into all three websites.

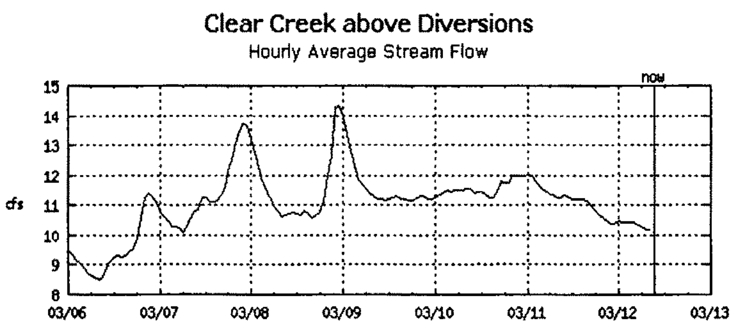

Fig. 1. This time-series plot displays hourly flows at a River gaging site for the previous 7 days.

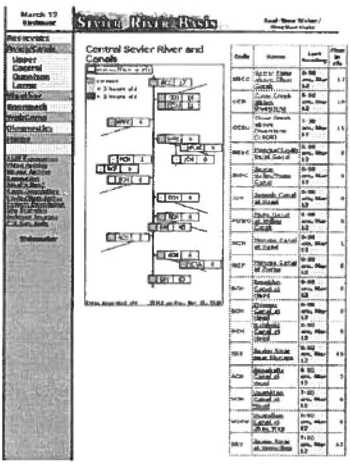

Fig. 2. Spatial diagram displaying real-time Sevier River

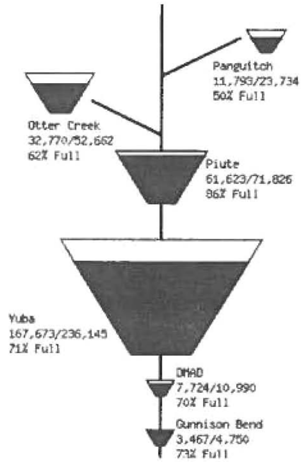

Fig. 3. Real-time status of all major water storage reservoirs in the Sevier River Basin

Progress made to date on these three Utah automation/Internet projects is outlined in Table 1. Fig. 4 shows the general locations served by the three websites.

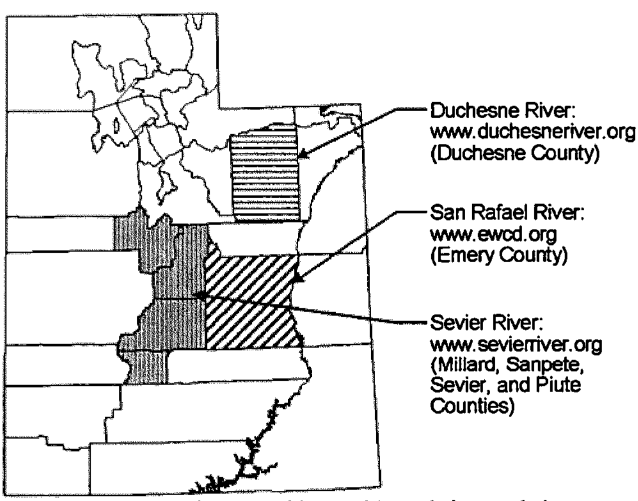

Fig. 4. Counties served by Utah's real-time websites.

Enhanced and Alternative Methods of Database Access

The value of the data on river basin websites is enhanced by improving Internet access. When information is available over high-speed, full-time connections, water managers make much greater use of the websites. Because these three Utah projects are in rural areas, access to broadband Internet service was extremely limited. It was determined that connecting a few of the key water managers' homes to the Internet full time would be an important step in the project's success. This was achieved by using low-cost, license-free spread spectrum radio links. The water managers with "full-time" connections access the website frequently throughout the day, sometimes more than 20 times.

One planned enhancement to the real-time data access system is the addition of telephone touch-tone data retrieval, since water managers often find themselves in need of access to the information while in the field. StoneFly is in the process of adding equipment which will enable anyone to call in on a telephone and retrieve specific real-time data by entering the proper number on a telephone keypad. This will allow cell phone access to the system.

A second alternative method of remote real-time data access is the web-enabled cell phone. These phones allow the users to view very small renditions of web pages. At the moment, the availability of web service on the rural cellular systems is limited. In the future, however, this service will be more wide spread so there are plans to create alternative versions of data display pages which would be optimized for cellular phone web browsers.

Decision-Support

Within the next few years, we anticipate being able to build a system that is capable of operating a river basin with little or no direct human intervention required, by tying decision-support models to the automation systems. This will allow a system to operate at maximum efficiency around the clock. To this end, we are working on several decision-support models. Two of these are discussed below.

Decision Support for Water Rights. All western water rights are established by legal decree or statute. In Utah, regulation of water rights is an administrative procedure executed by river commissioners acting under the direction of the State Engineer. In the Sevier River Basin, the river is managed by two commissioners using procedures so complex that few people fully understand them. The most difficult aspect of the allocation procedure involves the determination of the primary flow and the segregation of this water from storage water. Another confusing aspect is the division of the flow into zones.

In view of the importance of water rights in the management of water resources, Dr. Wynn Walker, Head, Department of Biological and Irrigation Engineering at USU, and Roger Walker, retired Sevier River commissioner, developed a computerized water rights allocation model—SEVIER—for use by the river commissioners and others (Walker, 1991). SEVIER duplicates the computations and record analyses performed by the two river commissioners.

To provide water rights updates in a timely manner, Dr. Walker is currently working with StoneFly to connect his model to the Sevier River monitoring system (river flows, canal diversions, and reservoir storage). This will provide the river commissioners, water users, and others with water rights information that is updated daily. By posting this information on the Internet, each irrigator will have continually updated information on the status of his/her water rights and will be notified when additional water is available. This Internet software is being tested during water year 2002.

Decision Support of Long-Term Water Forecasts. Improving the management efficiency of complex systems requires better information about both current and potential future water availability. Since 75 percent of available fresh surface water in the western United States comes initially in the form of snowfall (McManamon et al., 1993) the ability to forecast future water availability is limited by uncertainty in snowpack estimation and the timing and quantities of future runoff from snowmelt.

To address this issue, Dr. Mac McKee, Associate Director at the Utah Water Research Laboratory, USU, has proposed using military decision-support systems to improve forecasts of snowpack and snowmelt. The situation faced by water managers at the river basin scale is in many ways analogous to that of a battlefield commander. Just as battlefield commanders must make decisions about the deployment of troops and weaponry in the face of uncertain enemy strengths and intentions, the water manager must make decisions based on uncertainties related to future water supplies.

Considerable progress has been made in recent years in developing automated and interactive sensor data fusion techniques for military and intelligence applications. Sensor fusion involves a deductive process whereby data is interpreted in terms of models of situational elements, relationships, and behaviors (Bowman and Steinberg, 2001). These techniques can be applied to problems in estimating and predicting the state of regional terrestrial processes, such as water supply.

The USU project would employ information dissemination techniques to make the improved database readily available to water managers, decision-makers, and stakeholders in the three river basins (initially concentrating on the Sevier River Basin). This will require an outreach component that will enable the water users to transfer the analytic capability that is developed and make it available via the river basin websites.

ADDITIONAL BENEFITS

In addition to optimizing management of a water distribution system, these new automation/Internet/decision-support systems can: (1) build trust through complete disclosure; (2) encourage collaboration in water resource management; and (3) allow for adaptive management practices.

Complete Disclosure

Observers of our evolving global economy claim that the concept of transparency, or complete disclosure, is essential to economies which hope to prosper in the future. They argue that information about financial data and transactions should be available to all in a timely and consistent fashion. This ensures that sound decisions can be made by investors and that financial problems don't fester until they explode with unfavorable consequences. We believe that the same principal applies in the case of water resources. When all the stakeholder groups — irrigators, municipalities, sports enthusiasts, boaters, and environmentalists — are privy to the same information, no one can hide their actions. While this may be painful to some in the short run, in the long term we believe it to be good for all. When the www.sevierriver.org went live, there were concerns voiced about making the information available to everyone. After more than 3 years of operation, however, we have not yet heard of anyone wishing for the days when information was scarce and out of date.

Collaboration

It has been widely reported in the national news media that attitudes are changing in the western United States. For example, a recent edition of *Time Magazine* (MacCarthy, 2001, p. 21) quoted Patricia Limerick, a history professor at the Center of the American West in Boulder, CO: "There has been a tremendous surge in collaborative conservation groups and watershed alliances in the past 10 years. This evolution toward broader input into decision-making is, in part, the result of over-allocated or over-stressed resources and changing values.

The trend toward collaborative and localized decision-making is well served by real-time monitoring/Internet technologies because the latter provides information to everyone with access to the Internet (which is rapidly becoming everyone). There is no more information elite. Better and timelier data, universally available, is leading to better decision-making and improved water management. Websites like www.sevierriver.org reports real-time conditions throughout the Sevier River Basin (including river and canal flows, reservoir storage, snow and weather conditions, water quality, etc) for everyone to see. Decisions are made with a better understanding of present (and recent past and historic) conditions.

Adaptive Management

Environmental monitoring websites can provide timely feedback on important management issues like the effectiveness of salinity control programs or erosion control projects. The benefits of "adaptive management" have long been touted. But adaptive management depends upon carefully monitoring the effects of management actions on the environment, and then using that information to refine our understanding of the system and to adjust our decision-making and management plan (Western Water Policy Review Advisory Commission, 1998, pp 30-31). What better way to assess the effectiveness of management strategies than with real-time monitoring systems coupled with comprehensive decision-support software.

CONCLUSIONS

At the start of the river basin automation/Internet project, Reclamation and Stonefly staffs were hesitant to speculate on where the projects might be headed, for fear of scaring off the water users. Today the water users are frequently ahead of the technologists. The water users are continually inventing new uses and innovations for their river basin websites.

Admittedly, the process of using low-cost automation/Internet technologies to improve water management is still in its infancy. But, any measure, the river basin websites have been successes. According to one water user: "When something goes down and I have to go back to the old way of doing things, it is like being blind after being able to see."

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EVAPOTRANSPIRATION OF FULL-, DEFICIT-IRRIGATED AND DRYLAND COTTON ON THE NORTHERN TEXAS HIGH PLAINS [1]

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ABSTRACT

Cotton (*Gossypium hirsutum* L.) is beginning to be produced on the Northern Texas High Plains as a lower water-requiring crop while producing an acceptable profit. Cotton is a warm season, perennial species produced like an annual yet it requires a delicate balance of water and water deficit controls to most effectively produce high yields in this thermally limited environment. This study measured the water use of cotton in near-fully irrigated, deficiently irrigated, and dryland regimes in a Northern Texas High Plains environment, which has a shortened cotton producing season, using precision weighing lysimeters in 2000 and 2001. The irrigated regimes were irrigated with a lateral-move sprinkler system. The water use data were used to develop crop coefficient data and compared with the FAO-56 method for estimating crop water use. Cotton yield, water use, and water use efficiency was found to be as good in this region as other more noted cotton regions. FAO-56 ET prediction procedures performed better for the more fully irrigated treatments in this environment.

INTRODUCTION

Irrigation supplies from the northern High Plains Aquifer (Ogallala Aquifer) are declining due to water mining and the limited aquifer recharge. Producers are seeking alternate crops in the northern portion of the Southern High Plains that might reduce water consumption and extend the aquifer's useful life. Corn *(Zea mays* L.) is widely produced in the region with exceptionally high yields (USDA-NASS, 2001), but it has a large irrigation requirement (Howell et al., 1997). Cotton (*Gossypium hirsutum* L.) offers potentially equal gross incomes while requiring less irrigation water and the ability to be produced under dryland conditions while corn is not a reliable dryland crop in this region. The Northern

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Texas High Plains is adjacent to the largest contiguous cotton-producing region in the U.S., but it has a growing season length and thermal environment that is marginal for cotton. Nevertheless, producers are moving cotton production farther north in search of an alternate, economical crop. This region is far from ideal for cotton (Peng et al.,1989) with its short season, cool temperatures, high evaporative demand, and water scarcity (both from irrigation and growing season rainfall).

FAO-56 evapotranspiration (ET) methods (Allen et al., 1998) replaced the FAO-24 (Doorenbos and Pruitt, 1975) methods for estimating crop water use and proposed using the dual crop coefficient approach based on Wright (1982). FAO-56 used more precise definitions for the separation of soil water evaporation and crop transpiration from the lumped crop evapotranspiration and used the "straight-line" crop coefficient (K_c) approach (segmented lines opposed to curves) from FAO-24. Both FAO-56 and FAO-24 are based on "grass reference" ET (termed ET_o) with FAO-24 being based on a Penman equation and FAO-56 being based on the Penman-Monteith (PM) equation for a specified grass height [4.7 in. (0.12 m)], surface resistance [230 s ft^{-1} (70 s m^{-1})], albedo (0.23), and constant latent heat flux [585 cal g^{-1} (2.45 MJ kg^{-1})]. These ET methods are intended to improve irrigation scheduling programs such as Jensen et al. (1970) and Jensen et al. (1971). Although several methods are employed to express the time base for K_c curves, FAO-56 used a day scale while others have used a thermal scale based on growing degree days (GDD) (Sammis et al., 1985; Stegman, 1988; Ayars and Hutmacher, 1994; Slack et al., 1996; and Hunsaker, 1999). The GDD scale has been reported to improve inter-site and inter-seasonal transferability of K_c curves. Methods for computing GDDs differ significantly, including time base (hour or shorter to daily values), methods for computing the GDDs (Fry, 1983), and varying base and upper threshold temperatures used.

Hunsaker (1999) developed K_c curves for a short-season cotton variety in Arizona based on the California Irrigation Management System (CIMIS) hourly Penman equation (Snyder and Pruitt, 1985) for both the FAO-56 "straight line" and GDD based K_c methods. Their K_c values were larger than those proposed in FAO-56 for cotton. Allen (1999) applied the FAO-56 procedures to a large irrigation district in the western U.S., and he found an 8% over-estimate, which he attributed to actual crop conditions not fully representing the more "pristine" conditions assumed in FAO-56. Tolk and Howell (2001) found the dual K_c approach for sorghum (*Sorghum bicolor* (L.) Moench) superior compared with the single K_c approach using the FAO-56 methodology. The FAO-56 soil water evaporation procedures tended to over-estimate evaporation early in the season, and the "straight line" water limits on ET (based on Kerr et al., 1993) tended to over-estimate simulated effects on ET, particularly at the end of the season. Grismer (2002) reported that cotton K_c values that were measured in Arizona and California exceeded those reported in FAO-56 by 30-35% under non-water-stressed conditions, by 30% in CA under water stress, and by 20-25% in desert environments in AZ and CA.

Few studies besides Allen (1999 and 2000) and Tolk and Howell (2001) have evaluated the FAO-56 methods independently. The purpose of this paper is to report cotton water use amounts and rates in an environment not optimum for cotton and to compare the resulting water use rates in terms of the FAO-56 dual K_c approach across three water regimes.

MATERIALS AND METHODS

Agronomy and Treatments

The study was conducted at the USDA-ARS Laboratory at Bushland, TX [35° 11' N lat.; 102° 06' W long.; 3,840 ft (1,170 m) elev. above MSL]. ET was measured with two weighing lysimeters (Marek et al., 1988) each located in the center of 10.9-ac (4.4-ha) [700 ft (210 m) E-W by 700 ft (210 m) N-S] fields [four fields arranged in a square pattern] during the 2000 and 2001 seasons. The soil at this site is classified as Pullman clay loam [fine, mixed, superactive thermic Torrertic Paleustoll] (Unger and Pringle, 1981; Taylor et al., 1963) which is described as slowly permeable because of a dense B22 horizon about 1.0 to 1.6 ft (0.3 to 0.5 m) below the surface. The plant available water holding capacity within the top 6.6 ft (2.0 m) of the profile is approximately 9.4 in. (240 mm) [7.9 in. (~200 mm) to 5.0-ft (1.5-m) depth). A calcareous layer at about the 5.0 ft (1.5 m) depth limits significant rooting and water extraction below this depth. This soil is common to more than 2.9 million ac (1.2 million ha) of land in this region and about 1/3 of the sprinkler-irrigated area in the Texas High Plains (Musick et al., 1988). Weighing lysimeters offer one of the most accurate means to measure ET (Hatfield, 1980). Predominate wind direction is SW to SSW, and the unobstructed fetch (fallow fields or dryland cropped areas) in this direction exceeds 0.62 mi. (1 km). The field slope is less than 0.3 percent.

Two adjacent lysimeter fields (designated west and east) each contain two weighing lysimeters (designated NW & SW and NE & SE, respectively) were planted to cotton in each season. Each lysimeter field with its two sub-fields contained a weighing lysimeter in its center (Marek et al., 1987) that was 100 ft^2 (9 m^2) in area and 8 ft (2.3 m) deep with monolithic cores. Both lysimeter fields were planted to the same variety and managed similarly. The west lysimeter field was dryland (DRY) with the north half (NW) in 30-in. (0.76-m) spaced rows and the south half (SW) in 10-in. (0.25-m) spaced rows in 2000 and twin rows 10 in. (0.25 m) apart on 30 in. (0.76-m) spaced rows in 2001. The NW field was sown at rate of 183 seeds ft^{-2} (17 seeds m^{-2}) [6.1 seeds per ft of row (13 seeds per meter of row)]. Data from the SW field are not being used in this study [i.e., only the cotton fields in 30-in rows (0.76-m) spaced rows is being used herein]. The east lysimeter field was irrigated in both years with the south half (SE) being irrigated to meet the crop water use (FULL) but allowed to reach boll cutout and dry down for maturity while the north half (NE) was irrigated at one-half the FULL rate, except for a few initial irrigations for establishment at the FULL rate, on the same days by using smaller sized nozzles on the irrigation spray heads to achieve approximately one-half the flow rate (i.e., one-half the peak application rate and one-half the application amount). The sowing rate was the same for the FULL and DEFICIT treatments at 226 seeds ft^{-2} (21 seeds m^{-2}) [7.5 seeds per ft of row (16 seeds per meter of row)] in 2000, but it was reduced slightly to 215 seeds ft^2 (20 seeds m^{-2}) [7.2 seeds per ft of row (15 seeds per meter of row)] in 2001. The lysimeters were sown at a thicker rate and hand thinned about two weeks after emergence to match field plant densities.

Table 1 summarizes the agronomic and management details. Cotton was grown in both lysimeter fields (Paymaster[3] 2145 in both seasons) on rows spaced 30 in. (0.76 m) apart. In the east lysimeter field (SE and NE fields), rows were on raised beds and the furrows were diked to store irrigation and rainfall. In the NW field, rows were flat without beds or dikes. All field operations were performed with standard 15 ft (4.6-m) row-crop field equipment, except in the immediate 320-ft^2 (30-m^2) area at each lysimeter where hand-cultural methods were required. Fertility and pest control were applied uniformly to the field area.

Table 1. Agronomic and management information.

Category	2000		2001	
	Irrigated	Dryland	Irrigated	Dryland
Apply herbicide	Apr. 27	Apr. 26	Apr. 27	Apr. 30
Plant	May 17	May 16	May 16	May 17
Emergence	May 26	May 28	May 28	May 29
Installed neutron tubes	May 31	June 1	May 29	May 29
Cultivate	July 6	July 10	NA	NA
Furrow dike installation	July 7	NA	NA	NA
Begin deficit treatment	July 26	NA	July 2	NA
Harvest	Nov. 14	Oct.18	Oct. 30	Oct. 22

Irrigations

Irrigations were applied with a 10-span lateral-move sprinkler system (Lindsay Manufacturing, Omaha, NE) with an end-feed hose and aboveground, end guidance cable. The sprinkler system was aligned N-S, and irrigated E-W or W-E. The system was equipped with gooseneck fittings and spray heads (Nelson D3000, Walla Walla, WA) with medium grooved, concaved spray plates on drops

[3] The mention of trade or manufacturer names is made for information only and does not imply an endorsement, recommendation, or exclusion by USDA-Agricultural Research Service

located about 5 ft 1.5 m above the ground and 60 in. (1.52) m apart. Each spray head was equipped with a 15-psi (100-kPa) pressure regulator and a 2.2-lb. (1-kg) polyethylene drop weight. Irrigations were scheduled to meet the ET water use rate (based on the lysimeter mass of the FULL treatment) and were typically applied in one to two 1.0 in. (25-mm) applications per week. Irrigations were managed on the FULL treatment to minimize early water deficits with the available irrigation capacity while allowing the soil water profile to deplete in order to initiate boll cutout and to use the readily available soil water by maturity or just before frost. The FULL treatment did no completely meet the "potential" water demand late in the season to reduce vegetative growth in favor of boll filling and eventual opening of the bolls likely to mature by the season's end.

Plant and Yield Sampling

Plant samples from 11-16 ft^2 (1.0-1.5-m^2) areas were obtained periodically to measure crop development. These field samples were taken at sites about 30 to 60 ft (10 to 20 m) away from the lysimeters in areas of the field representative of the lysimeter vegetation. Leaf area index (LAI), crop height (CH), and aboveground dry matter (DM) were measured from three samples. Final yield was measured by harvesting all the open bolls and aboveground plant matter from each lysimeter [97 ft^2 (9 m^2)], and dry matter and yield at harvest were measured from adjacent plant samples. The seed cotton was ginned on a small research gin at the Texas Agricultural Experiment Station at Lubbock and fiber samples were analyzed by the Texas Tech University International Textile Center (data not reported here).

Lysimeter Measurements

Lysimeter mass was determined using a Campbell Scientific (Campbell Scientific, Inc., Logan, UT) model CR-7X data logger to measure and record the lysimeter load cell (Interface, Scottsdale, AZ) model SM-50, signal sampled at 0.5-Hz (2 s) frequency. The load cell signal was averaged for 5 min and composited to 30-min means (reported on the mid point of the 30 min, i.e. data were averaged from 0-30 minutes and reported at 15 min), and the lysimeter mass resolution was 0.004 in. (0.01 mm), and its accuracy exceeded 0.002 in. (0.05 mm) (Howell et al., 1995a). Daily ET was determined as the difference between lysimeter mass losses (from evaporation and transpiration) and lysimeter mass gains (from irrigation, precipitation, or dew) divided by the lysimeter area [97 ft^2 (9 m^2)]. The lysimeters were calibrated prior to the experiment similarly to the methods used by Howell et al. (1995a) but not as detailed. A pump regulated to -17 in. H_2O column (-10 kPa) provided vacuum drainage, and the drainage effluent was held in two tanks suspended from the lysimeter (their mass was part of the total lysimeter mass) and independently weighed by load cells (drainage rate data are not reported here). ET for each 24-h period was multiplied by 1.02 to adjust the lysimeter area to the mid point between the two walls [0.39 in. (10 mm) air gap; 0.37 in. (9.5 mm) wall thickness; 98 ft^2 (9.18 m^2) area instead of 96.9 ft^2 (9.00 $m^{2)}$ area)]. This correction would be applicable for full-cover crops, but it would not be necessary for bare soil conditions. Nevertheless, it was applied to all data uniformly.

Soil Water Measurements

Soil water contents were measured periodically using a neutron probe (Campbell Pacific Nuclear, Martinez, CA) model 503DR Hydroprobe at 8 in. (0.2-m) depth increments with 30-s counts. Two access tubes were located in each lysimeter [read to 6.2 ft (1.9 m) depth] and four tubes were located in the field surrounding each lysimeter [read to 7.5-ft (2.3-m) depth]. The probe was field calibrated for the Pullman soil using a method similar to that described by Evett and Steiner (1995).

Climatic Data, Reference ET, and Crop Coefficients

Solar radiation, wind speed, air temperature, dew point temperature, relative humidity, precipitation, and barometric pressure were measured at an adjacent weather station (Howell et al., 1995b) with an irrigated grass surface (cool-season lawn mixture containing bluegrass, perennial rye-grass, etc.). Reference ET (ET_o) was computed with the FAO-56 equation using the exact formulas in Allen et al. (1998).

The crop ET (ET_c in mm d^{-1}) was computed as

$$ET_c = (K_{cb}K_s + K_e)ET_o \quad (1)$$

where K_{cb} is the "*basal*" crop coefficient, K_s is the soil water deficit factor, K_e is the soil water evaporation factor, and ET_o is the grass reference ET in (in. day^{-1} or mm day^{-1}). Values for K_{cb}, K_s, and K_e were derived following Tolk and Howell (2001) (Table 2) for the Pullman soil and using guides from Allen et al. (1998) in the FAO-56 manual. A spreadsheet patterned after Appendix 8 in the FAO-56 manual was used for this similar to one developed for use in Tolk and Howell (2001). Stage lengths were estimated from the measured FULL treatment and based on phenologic growth stages of cotton (Hake et al., 1990). The K_{cb} values were fit to the few mean K_c values for days without irrigations and were selected to match as closely as possible to those in FAO-56. The value for "p" was reduced from 0.65 in FAO-56 to 0.55 to initiate an ET reduction to better match field observations.

Table 2. Pullman soil parameters used with FAO-56 dual K_c model (Tolk and Howell, 2001). See FAO-56 manual for parameter definitions (Allen et al., 1998)

Parameter	Definition	Value and Unit
FC	Field capacity	0.33 m m^{-3}
PWP	Permanent wilting point	0.20 m m^{-3}
Z_r	Root zone depth	1.5 m
Z_e	Evaporation zone depth	0.15 m
TEW	Total evaporative water	34.5 mm
REW	Readily evaporative water	10 mm
TAW	Total available water	195 mm
RAW	Readily available water	107 mm
p	Water stress initiation	0.55 (fraction)
in. ft^{-1} = 12*(m^3 m^{-3})		
ft = 3.28*(m)		
in. = 0.03937*(mm)		

Growing degree-days were computed as the mean of the daily maximum and minimum air temperatures less the base temperature of 60°F (15.6°C) (Hake et al., 1990; Peng et al., 1989) that is widely used in the cotton community in the Southern High Plains. This GDD method differs from that used by Hunsaker (1999), and the methods described by Fry (1983), who provided some conversions for differing GDD methods.

Model Performance Evaluation

Tolk and Howell (2001) explained the desirability of the Legates and McCabe (1999) statistical procedure (E; modified coefficient of model efficiency), but both that procedure and the Willmott (1981) method [D; coefficient of agreement] that used the error square terms were included. Also, standard statistical parameters — coefficient of determination (r^2), standard deviation, mean, and root mean square error (RMSE) were used to characterize the data and the FAO-56 model performance.

RESULTS AND DISCUSSION

Weather and Climatic Conditions

Both of the growing seasons were drought seasons for Bushland, but they were not atypical of the climatic variations experienced on the Southern Great Plains. The climatic conditions are given in Table 3 for the seasons, and the Bushland historical data are presented for comparison. Mean monthly temperatures were not greatly different from long-term monthly means despite the dry summers. After the slightly larger than normal rain in June of 2000, the growing season was devoid of significant rains until late October, which was too late to help the 2000 crop. The 2001 rainfall was again below normal although early rains in May and June reduced the need for early irrigations. Wind speeds at the 2-m elevation were greater than normal in the early 2000 season. The mean seasonal FAO-56 reference ET (ET_o) was almost identical in both years, although they had slightly differing temporal trends in daily ET_o.

Crop Development

Figures 1 and 2 illustrate the cotton development in each season, respectively. The 2000 crop was planted following alfalfa (*Medicago sativa* L.), which may have affected the growth and development. The alfalfa was plowed out during the 1999 fall and winter, but a few alfalfa plants remained that had to be treated post-planting with herbicides. In addition, the deeper rooting alfalfa had depleted the deep soil water [>5 ft (1.5 m)]. The 2001 FULL treatment achieved a greater LAI, CH, and DM than it did in 2000. However, the DRY and DEFICIT treatments had almost the same growth patterns in both years. These cotton growth patterns are typical for the Texas High Plains, although we expected LAI for the FULL treatment in 2000 to be more like the pattern in 2001. The FULL treatment achieved a closed canopy in both seasons; however its canopy was taller in 2001 with significantly greater row width spread (as indicated by the LAI values; see Figs. 1 and 2).

Table 3. Monthly climatic data summary of daily mean values for 2000 and 2001compared with the 20-yr Bushland historical mean data.

Month	Max Temp. Tmax °C	Min. Temp. Tmin °C	Dew Point Tdew °C	Solar Rad. MJ m^{-2} d^{-1}	2-m Wind m s^{-1}	Barometric Pressure kPa	ETo mm d^{-1}	Rain mm
2000								
May	29.5	11.0	5.6	26.6	5.12	88.1	8.17	11.4
June	28.9	16.0	14.2	21.7	5.12	88.4	6.45	96.8
July	33.2	17.9	14.2	26.1	3.91	88.5	7.68	26.2
August	33.9	17.3	10.8	24.5	3.59	88.6	7.75	0.5
September	31.1	12.6	5.2	21.2	3.68	88.5	6.74	0.0
October	20.7	8.1	6.9	12.3	3.66	88.6	3.08	66.0
2001								
May	25.4	10.4	10.6	24.3	4.04	88.3	5.38	75.7
June	32.7	16.0	11.0	27.5	4.33	88.4	8.38	33.5
July	35.1	19.3	12.7	26.6	3.64	88.5	8.36	3.6
August	31.9	16.8	13.9	22.1	3.04	88.7	6.19	28.2
September	29.2	12.4	10.7	20.5	3.44	88.6	5.52	12.2
October	24.0	5.5	1.9	16.1	4.16	88.5	4.78	1.5
20-yr Bushland Historical Means								
May	25.7	9.6	NA	24.7§	4.34¶	NA	NA	59.9
June	30.1	14.7		26.3	4.26			76.2
July	32.3	16.9		25.9	3.73			73.4
August	31.4	16.4		22.9	3.44			70.9
September	27.6	11.9		19.3	3.61			56.4
October	21.8	5.3		15.6	3.77			40.1

§ 28-yr mean
¶ 12-yr mean

°F = 1.8*(°C) + 32
cal cm^{-2} day^{-1} = 23.89*(MJ m^{-2} d^{-1})
in. Hg (60°F) = 0.292*(kPa)
mph = 2.237*(m s^{-1})
in. d^{-1} = 0.03937*(mm d^{-1})
in.= 0.03937*(mm)

Water Use, Yield, and Water Use Efficiency

The seasonal water use, yield, and lysimeter water use efficiency (WUE) data are presented in Table 4. Grismer (2002) recently reviewed these types of data for cotton, emphasizing AZ and CA locations, but he included studies conducted in

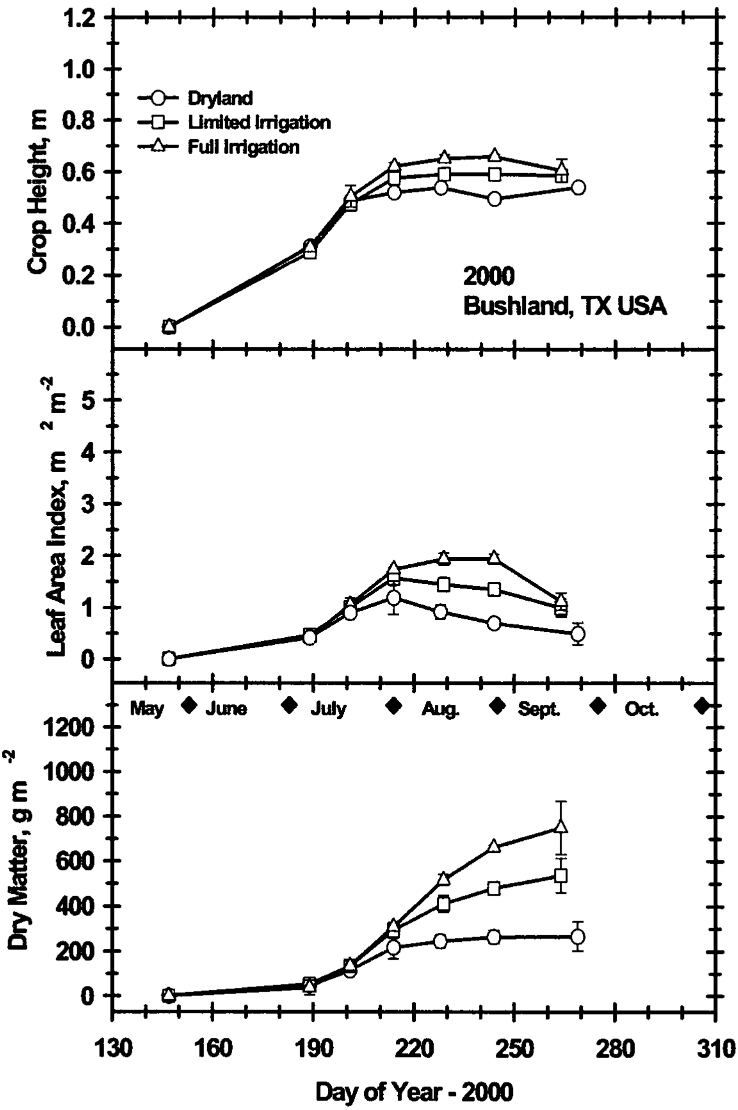

Figure 1. Cotton growth parameters in 2000 at Bushland, TX.
[lb ac^{-1} = 8.92*(g m^{-2}); ft = 3.28*(m)].

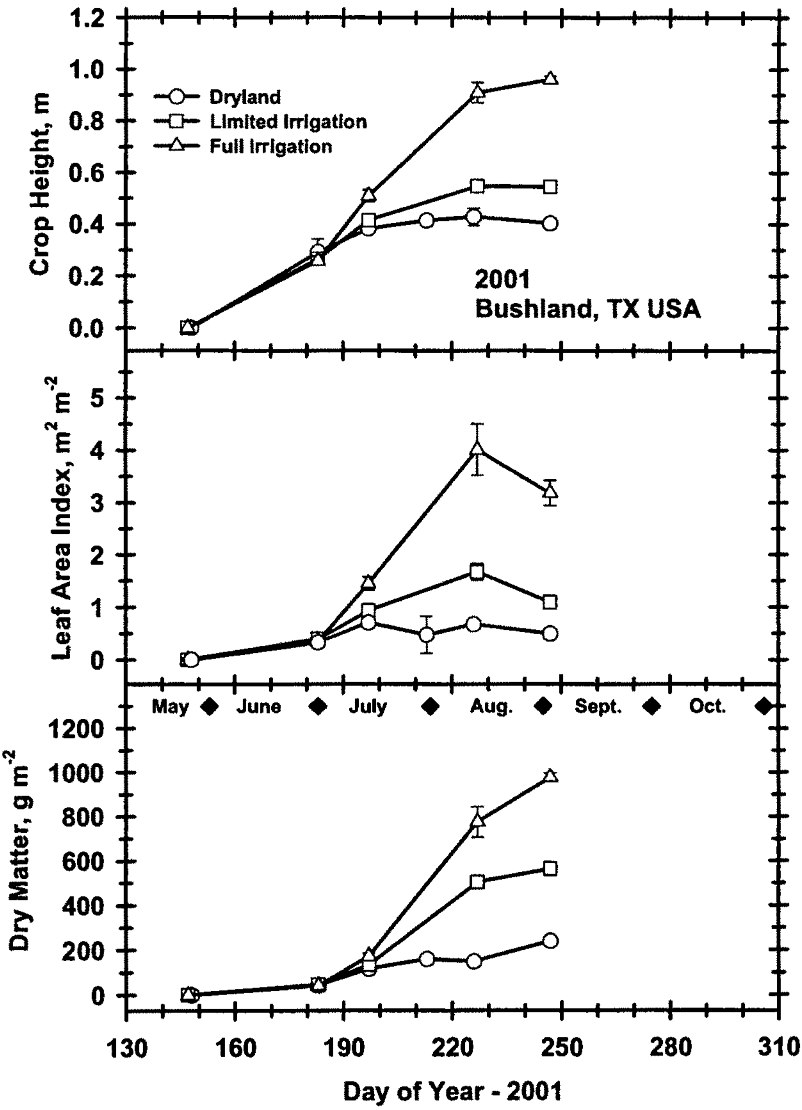

Figure 2. Cotton growth parameters in 2001 at Bushland, TX.
[lb ac^{-1} = 8.92*(g m^{-2}); ft = 3.28*(m)].

cotton regions around the world. Our ET and WUE for the FULL and DEFICIT treatments are similar to his summary. He indicated WUE values of 43.1 to 47.6 lb ac^{-1} $in.^{-1}$ (0.19 to 0.21 kg m^{-3}) required a net irrigation amount (after subtracting rainfall) of about 27.6 in. (700 mm) in the San Joaquin Valley in California. This is considerably greater than our irrigation requirement for cotton on the Northern Texas High Plains [~20 in. (~500 mm) or less depending on rainfall]. We attribute this partly to our shorter growing season; however, it is difficult to argue that our ET demand is less than the Central Valley of California or the deserts of High Plains due to high winds, low humidity, relatively clear skies, and the high elevation (low barometric pressure). Peng et al. (1989) indicated in the Southern Texas High Plains, a heat unit accumulation of approximately 2610 °F-days (1450 °C-days) with a total water supply rainfall plus irrigation of 22 in. (550 mm) are needed to achieve optimum yields exceeding 624 lb ac^{-1} (70 g m^{-2}). Figures 3 and 4 indicated we did not exceed a cumulative GDD of 1980 °F-days (1100 °C-days) in either season. It is unlikely that a cotton crop can consistently accumulate enough heat units to fully mature all the bolls on the plants in the Northern Texas Arizona or California with the extreme advection experienced in the Southern High Plains environment. It is critical that the first and second position bolls (Hake et al., 1990) be developed by minimizing early crop stresses and that careful insect and disease control measures are utilized to avoid the loss of these primary fruiting positions. Despite the environmental limitations for producing cotton on the Northern Texas High Plains, excellent yield potentials are possible even with DEFICIT irrigations and WUE values exceeding that for many others

Table 4. Water use, yield, and lysimeters WUE data for the 2000 and 2001 seasons at Bushland, TX.

	2000			2001		
Treatment	FULL	DEF.	DRY	FULL	DEF.	DRY
Parameters						
Measured ET, mm	775	622	397	739	578	386
FAO-56 Computed ET, mm	770	619	356	736	639	415
Irrigation, mm	470	307	12	385	208	14
Rainfall, mm	201	201	201	214	214	214
Lysimeter yield, g m^{-2}	150.0	89.4	36.4	111.9	126.5	39.7
WUE, kg m^{-3}	0.194	0.144	0.092	0.151	0.219	0.103
Field mean yield, g m^{-2}	131.3	64.6	25.8	102.2	91.9	28.4
Field std. dev., g m^{-2}	13.3	4.8	3.7	9.6	9.0	21.0
Lysimeter yield within ± 2 std. dev. from the field yield	yes	no	no	yes	no	yes
in. = 0.03937*(mm) lb ac^{-1} = 8.92*(g m^{-2}) lb ac^{-1} $in.^{-1}$ = 226.6*(kg m^{-3})						

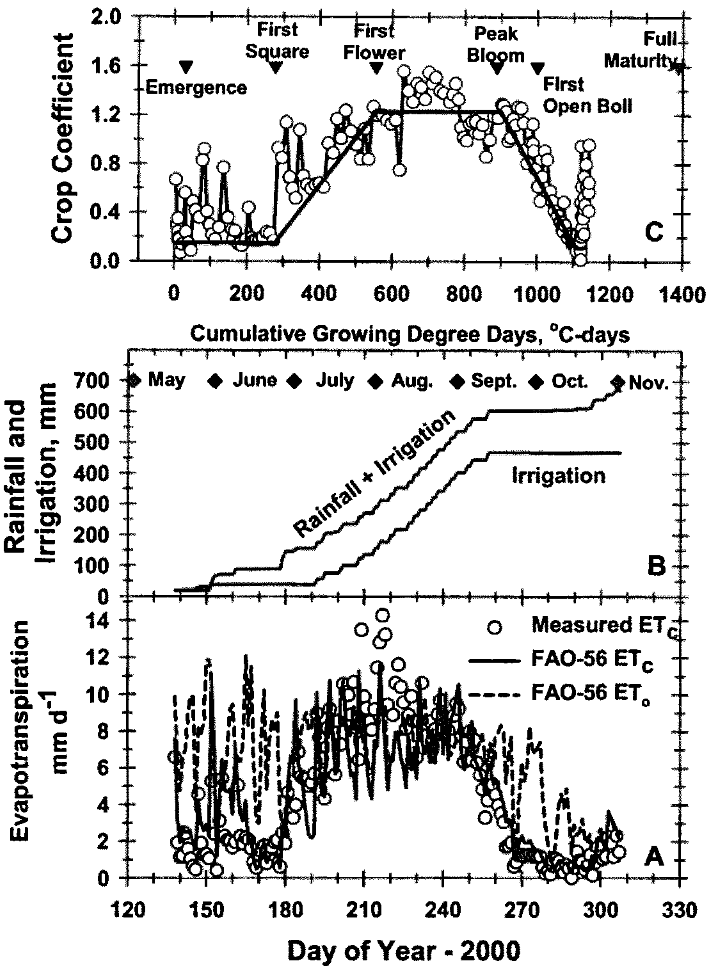

Figure 3. Cotton Water Use for the Full Treatment in 2000at Bushland, TX. (A) Shows the Daily ET (ET_c) Measured and Computed by FAO-56 and the FAO ET_o Reference ET; (B) Shows the Cumulative Irrigation and Rainfall Data; and (C) Shows the Cotton Crop Coefficient in Relation to the Cumulative GDD for the Base Temperature of 60°F (15.6°C). ET in in. $d^{-1} = 0.03937*(mm\ d^{-1})$; in. = 0.03937*(mm); GDD (base 60°F-days) = 1.8*GDD (base 15.6°C-days).

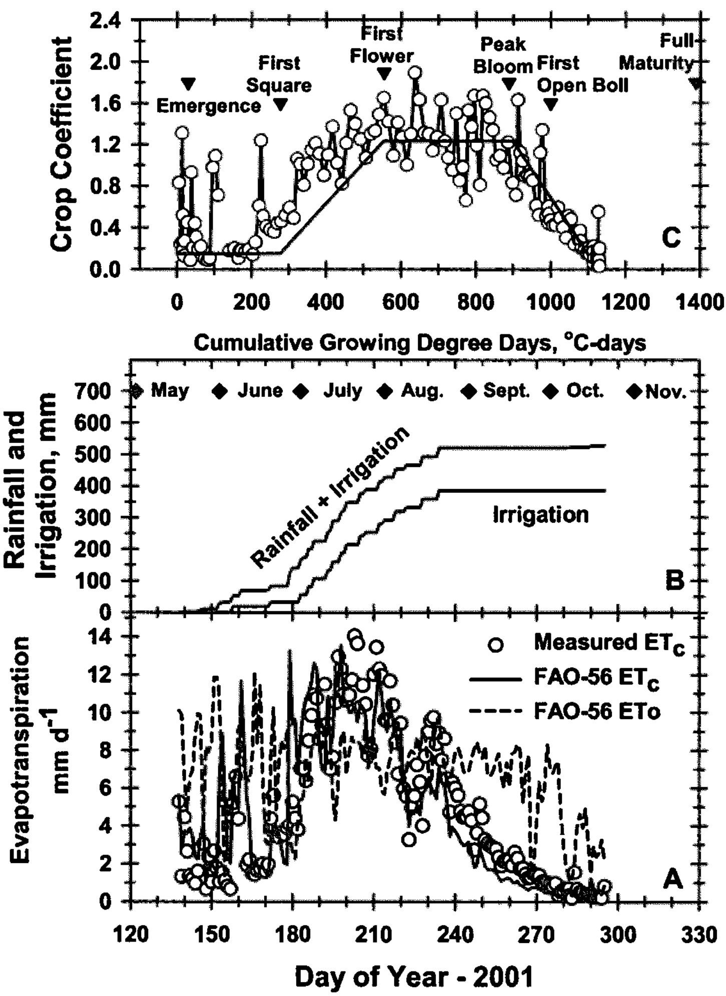

Figure 4. Cotton Water Use for the Full Treatment in 2001 at Bushland, TX. (A) Shows the Daily ET (ET_c) Measured and Computed by FAO-56 and the FAO ET_o Reference ET; (B) Shows the Cumulative Irrigation and Rainfall Data; and (C) Shows the Cotton Crop Coefficient in Relation to the Cumulative GDD for the Base Temperature of 60°F (15.6°C). ET in in. d^{-1} = 0.03937*(mm d^{-1}); in. = 0.03937*(mm); GDD (base 60°F-days) = 1.8*GDD (base 15.6°C-days).

regions with better environments for cotton (Table 4). Cotton offers regional producers another crop option that has a lower water requirement yet a high income potential depending on the fiber quality and price.

The FAO-56 used the computed reference ET_0 values for the site with the beginning soil water contents matched to the early season measurements. The FAO-56 model fit the FULL treatments considerably better than the water deficit DRY treatments (Table 5). We believe, without the benefit of a thorough analysis, that the simple "straight line" water stress function, K_s, exaggerated the

Table 5. Model evaluation parameters for the FAO-56 procedure for cotton on the Northern Texas High Plains.

	2000			2001		
Treatment	FULL	DEF.	DRY	FULL	DEF.	DRY
Parameters						
D (Willmott, 1981)	0.773	0.469	0.391	0.961	0.529	0.274
E (L & M, 1999)	0.562	0.897	0.007	0.710	0.311	-0.498
RMSE, mm d^{-1}	1.98	2.48	1.83	5.83	2.39	1.87
Mean, mm d^{-1}	4.59	3.71	2.44	4.74	3.68	2.45
Std. Dev., mm d^{-1}	3.66	2.35	1.51	3.82	2.48	1.21
Coeff. of Determin., r^2	0.708	0.519	0.432	0.758	0.356	0.078
in. d^{-1} = 0.03937*(mm d^{-1}).						

on-set of ET stress, although we found the "p" value (stress set point) rather insensitive in our case with trials for $0.4 < p < 0.8$. The soil water stress function is critical in our case because of deficit, declining water supplies and dryland production. In addition, like Tolk and Howell (2001), we found that the early soil water evaporation was over-estimated which caused the simulated and measured ET_c values to depart from synchronization WITH THE fao-56 model. The index of agreement (D) (Willmott, 1981) had higher values for the FULL treatments while the modified index of model efficiency (Legates and McCabe, 1999) indicated poorer model agreement, except for the DEFICIT treatment in 2000.

For the Northern Texas High Plains, Table 6 presents a starting point in the use of FAO-56 methods for cotton in this unusual region for cotton. Figures 3 and 4 illustrate the superiority of the GDD basis for crop K_c curves because the GDD scale spreads the critical mid-season period while maintaining the needed precision on the season ends. Although we did not present the K_c curves based on a time scale (see Table 6), they required some greater skill in defining the water stress at the end of the mid-season and through the late-season periods. The late season crop coefficients are typically not "adjusted" in FAO-56. But cotton production in this region is often terminated by chemical applications to hasten boll opening and to terminate vegetative growth. Early frost can terminate growth, too, in this region.

Table 6. Length of cotton growth stages, K_{cb}, and $K_{cb}adj$ values for use with the FAO-56 methods for the Northern Texas High Plains.

Cotton Growth Stage	Length of Stage (days)	Basal Crop Coefficient (K_{cb})	Adjusted Crop Coefficient ($K_{cb}adj$)§
Days			
Initial	40-50	0.08	0.15
Development	40	na	na
Mid-season	50	1.10	1.23
Late-season	28-30	0.15	0.20
GDDs (°C-days)			
Initial	0-277	0.08	0.15
Development	277-555	na	na
Mid-season	555-900	1.10	1.23
Late-season	900-1100	0.15	0.20
§ Adjusted according to FAO-56 (Allen et al., 1998) °F-days =1.8*(°C-days).			

CONCLUSIONS

Cotton appears to be a viable alternate crop for the Northern Texas High Plains that can use less water than other crops. The WUE and yield obtained at Bushland rivals those from more noted cotton production regions while offering a crop alternative that responds well to both rainfall and irrigation. The WUE was almost doubled by irrigation. It is noted that these were unusually dry summers.

The FAO-56 ET procedures performed considerably better under the more "well-watered" conditions suggesting the need for additional studies on the model's performance or environmental characterization for deficit irrigation and dryland conditions.

Acknowledgements

The assistance of Karen Copeland, Soil Scientist; Keith Brock, Biological Technical; Brice Ruthardt, Biological Technician; and Don McRoberts, Biological Technician at the Conservation and Production Research Laboratory were vital to this work as well as advice from Dr. Louis Baumhardt, Soil Scientist, on technical aspects of cotton production.

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A SIMULATION MODEL FOR EVAPOTRANSPIRATION OF APPLIED WATER

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ABSTRACT

The SIMETAW program was written to provide a new and innovative tool for estimating evapotranspiration of applied water (ETAW), which is a seasonal estimate of the water requirement for evapotranspiration of a crop minus any water supplied by effective rainfall. ETAW information is needed to determine consumptive use requirements. In addition to using measured weather data, the SIMETAW program simulates daily weather data from monthly climate data. Simulation of daily weather data where only monthly means exist is a good tool for filling missing data points. In addition, the simulation program is useful for studying the effects of climate change on ETAW. All of the ETAW calculations are done on a daily basis, so the estimation of effective rainfall and, hence, ETAW is greatly improved over earlier methods. In addition, the use of the widely adopted Penman-Monteith equation for reference evapotranspiration (ET_o) and improved methodology to apply crop coefficients for estimating crop evapotranspiration is used to improve ETAW accuracy.

INTRODUCTION

The 'Simulation of Evapotranspiration of Applied Water' program (SIMETAW) was developed to help the State of California to plan for future water demand by agriculture and landscape irrigation. Using Borland Professional C++, the program was written by Sara Sarreshteh based on a design by R. Snyder, M.

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Orang, S. Geng, and S. Matyac. SIMETAW has a user-friendly design and, while mainly empirical, it accounts for many factors affecting crop coefficients that are generally ignored in other programs. Rainfall, soil water-holding characteristics, effective rooting depths, and ET_c are needed to determine effective rainfall. Combining crop evapotranspiration (ET_c) with effective rainfall estimates provides net water application requirements for various crops. When divided by the weighted mean application efficiency, the result is a site-specific total irrigation requirement to produce a crop. Weather generators allow us to investigate how changes in weather will affect the water demand in the state. This paper will discuss how the simulation model uses monthly input to generate daily weather data over variable periods of record and the advantages of the new model over traditional long-term ET_c estimates.

ENTERING CROP AND SOIL INFORMATION

Crop and soil information are input into a data file and the data are stored under a filename using the 'Detailed Analysis Unit' or 'DAU', which is used by the State of California as a region for determining ETAW. The input data include the crop name, planting and ending date, initial growth irrigation frequency, pre-irrigation information, immaturity factors, presence of cover crops, soil water holding characteristics, maximum soil and rooting depths, etc. Data are saved as a row of data in the DAUnnn.csv file before going onto the next crop-soil combination entry. Each row of data in the file contains information on crop growth, crop coefficients, irrigation frequency, cover crops, crop maturity, etc.

CALCULATING THE YIELD THRESHOLD DEPLETION

Crop rooting depth, soil depth, and water holding characteristics are used to determine the yield threshold depletion (YTD), which is used for determining an irrigation schedule. A user selects one of three general categories for the soil water holding characteristics. If a light soil is selected, the program uses 0.075 inches per inch for the available water holding capacity of the soil. A value of 0.125 inches per inch is used for the water holding capacity of a medium textured soil. For a heavy soil, a value of 0.175 inches per inch is used. The selected value is multiplied by the smaller of the rooting depth or the soil depth to determine the plant available water (PAW) within the soil reservoir at the maximum rooting depth for the crop. Although not strictly correct, the water holding content at field capacity for the soil reservoir is estimated as twice the available water holding content. This is only done to simplify graphing of the results. The YTD for the crop is calculated as the product of the allowable depletion (expressed as a fraction) and the PAW. In reality, the rooting depth and PAW increases as the roots extend, but, because of the additional complexity, this is ignored in the SIMETAW model.

ENTERING CLIMATE DATA

Either daily or monthly climate data are used to determine ETAW in SIMETAW. The daily data can come from CIMIS (California Irrigation Management Information System) or from a non-CIMIS data source as long as the data are in the correct format, which is described in the HELP files. After reading the data, ETAW can be calculated directly from the raw daily data. In addition, the monthly means can be calculated from the daily files and then daily data are generated using the simulation program. Since daily data were input directly, the calculation of monthly data for use in simulation of daily data is unnecessary. However, it was included to test if similar results are obtained using raw or simulated data.

The monthly data can be read from a file or calculated from daily CIMIS or non-CIMIS data files, or from some other source. The monthly data file must have the proper, comma-delimited format as described in the HELP files. SIMETAW will generate daily weather data for a specified period of record from the monthly data.

SIMETAW either generates a daily data file from monthly data or uses a raw data file consisting of daily solar radiation, maximum, minimum and dew point temperature, and wind speed for calculating daily ET_o. After calculating ET_o, if the data were generated, the program sorts the rainfall data within each month to force a negative correlation between rainfall amount and ET_o rate. Only the rainfall dates are sorted and there is no change in the dates for the weather and ET_o data. The results are output to a file with the extension 'wrk'. For non-simulated (raw) data, the data are directly saved in the file with the 'wrk' extension without sorting the rainfall dates.

WEATHER SIMULATION

Weather simulation models are often used in conjunction with other models to evaluate possible crop responses to environmental conditions. One important response is crop evapotranspiration (ET_c). Crop evapotranspiration is commonly estimated by multiplying reference evapotranspiration by a crop coefficient. In SIMETAW, daily data are used to estimate reference evapotranspiration. Rainfall data are then used with estimates of ET_c to determine ETAW. One can either use raw or simulated daily data for the calculations.

Characteristics and patterns of rainfall are highly seasonal and localized, so a making a general, seasonal model that is applicable to all locations is difficult. Recognizing the fact that rainfall patterns are usually skewed to the right toward extreme heavy amount and that the rain status of previous day tends to affect present day's condition, a gamma distribution and Markov chain modeling approach was applied to described rainfall patterns for periods within which rainfall patterns are relatively uniform (Gabriel and Neumann, 1962; Stern, 1980; Larsen and Pense 1982; and Richardson and Wright, 1984). This approach consists of two models: two-state, first order Markov chain and a gamma distribution function. These models require long-term daily rainfall data to estimate model parameters. SIMETAW however, uses monthly averages of total rainfall amount and number rain days to obtain all parameters for the Gamma and Markov Chain models.

The simulation of wind speed is a simpler procedure, requiring only the gamma distribution function, as described for rainfall. While using a gamma distribution provides good estimates of extreme values of wind speed, there is a tendency to have some unrealistically high wind speed values generated for use in ET_o calculations. Because wind speed depends on atmospheric pressure gradients, no correlation between wind speed and the other weather parameters used to estimate ET_o exists. Therefore, the random matching of high wind speeds with conditions favorable to high evaporation rates leads to unrealistically high ET_o estimates on some days. To eliminate this problem, an upper limit for simulated wind speed was set at twice the mean wind speed. This is believed to be a reasonable upper limit for a weather generator used to estimate ET_o because extreme wind speed values are generally associated with severe storms and ET_o is generally not important during such conditions.

Temperature, solar radiation, and humidity data usually follow a Fourier series distribution. Therefore, the model of these variables may be expressed as:

$$X_{ki} = \mu_{ki} (1 + \delta_{ki} C_{ki}) \quad (1)$$

where k = 1, 2 and 3 (k=1 represents maximum temperature; k = 2 represents minimum temperature; and k =3 represents solar radiation). μ_{ki} is the estimated daily mean and C_{ki} is the estimated daily coefficient of variation of the i^{th} day, i = 1, 2 ..., 365 and for the k^{th} variable.

SIMETAW simplifies the parameter estimation procedure of Richards and Wright (1984), requiring only monthly means as inputs. From a study of 34 locations within the United States, the coefficient of variability (CV) values appear to be inversely related to the means. The same approach is used to calculate the daily CV values. In addition, a series of functional relationships between the parameters of the mean curves and the parameters of the coefficient of variation curves, which made it possible to calculate C_{ki} coefficients from μ_{ki} curves without additional input data requirement, were developed.

REFERENCE EVAPOTRANSPIRATION CALCULATION

Reference evapotranspiration (ET_o) is estimated from daily weather data using a modified version of the Penman-Monteith equation (Allen et al., 1999, Walter et al., 2000, and Itenfisu et al., 2000.). The equation is:

$$ET_o = \frac{0.408\ \Delta\ (R_n - G) + \gamma \dfrac{900}{T + 273} u_2\ (e_s - e_a)}{\Delta + \gamma\ (1 + 0.34\ u_2)} \tag{2}$$

where Δ is the slope of the saturation vapor pressure at mean air temperature curve (kPa $^oC^{-1}$), R_n and G are the net radiation and soil heat flux density in MJ $m^{-2}d^{-1}$, γ is the psychrometric constant (kPa $^oC^{-1}$), T is the daily mean temperature (^oC), u_2 is the mean wind speed in m s^{-1}, e_s is the saturation vapor pressure (kPa) calculated from the mean air temperature (^oC) for the day, and e_a is the actual vapor pressure (kPa) calculated from the mean dew point temperature (^oC) for the day. The coefficient 0.408 converts the $R_n - G$ term from MJ $m^{-2}d^{-1}$ to mm d^{-1} and the coefficient 900 combines together several constants and coverts units of the aerodynamic component to mm d^{-1}. The product 0.34 u_2, in the denominator, is an estimate of the ratio of the 0.12-m tall canopy surface resistance (r_c=70 s m^{-1}) to the aerodynamic resistance (r_a=205/u^2 s m^{-1}). It is assumed that the temperature, humidity and wind speed are measured between 1.5 m (5 ft) and 2.0 m (6.6 ft) above the grass-covered soil surface. For a complete explanation of the equation, see Allen et al. (1999).

CROP COEFFICIENTS

While reference crop evapotranspiration accounts for variations in weather and offers a measure of the 'evaporative demand' of the atmosphere, crop coefficients account for the difference between the crop evapotranspiration and ET_o. The main factors affecting the difference are (1) light absorption by the canopy, (2) canopy roughness, which affects turbulence, (3) crop physiology, (4) leaf age, and (5) surface wetness. Because evapotranspiration (*ET*) is the sum of evaporation (*E*) from soil and plant surfaces and transpiration (*T*), which is vaporization that occurs inside of the plant leaves, it is often best to consider the two components separately. When not limited by water availability, both transpiration and evaporation are limited by the availability of energy to vaporize water. During early growth of crops, when considerable soil is exposed to solar radiation, ET_c is dominated by soil evaporation and the rate depends on whether or not the soil surface is wet. If a nearly bare-soil surface is wet, the ET_c rate is slightly higher than ET_o, when evaporative demand is low, but it will fall to about 80% of ET_o under high evaporation conditions. However, as a soil surface dries off, the evaporation rate decreases considerably. As a canopy develops, solar radiation (or light) interception by the foliage increases and transpiration rather than soil evaporation dominates ET_c. Assuming there is no transpiration-reducing water stress, light interception by the crop canopy is the main factor determining the ET_c rate. Therefore, crop coefficients for field and row crops generally increase until the canopy ground cover reaches about 75%. For tree and vine crops the peak K_c is reached when the canopy has reached about 70% ground cover. The difference between the crop types results because the light interception is somewhat higher for the taller crops.

During the off-season and during initial crop growth, *E* is the main component of *ET*. Therefore, a good estimate of the K_c for bare soil is useful to estimate off-season soil evaporation and ET_c early in the season. A two-stage method for estimating soil evaporation presented by Stroonsnjider (1987) and refined by Snyder et al. (2000) is used to estimate bare-soil crop coefficients. This method gives *Kc* values as a function of wetting frequency and *ETo* that are quite similar to the widely used bare soil coefficients that were published in Doorenbos and Pruitt (1977). The soil evaporation model is used to estimate crop coefficients for bare soil using the daily mean ET_o rate and the expected number of days between significant precipitation (P_s) on each day of the year. Daily precipitation is considered significant when $P_s > 2 \times ET_o$.

Field and row crops

Crop coefficients are calculated using a modified Doorenbos and Pruitt (1977) method. The season is separated into initial (date A-B), rapid (date B-C), midseason (date C-D), and late season (date D-E) growth periods (Fig. 1).

Tabular default K_c values corresponding to important inflection points in Fig. 1 are stored in the SIMETAW program. The value *Kc1* corresponds to the date B *Kc* (*KcB*). For field and row crops, *Kc1* is used from date A to B. The value *Kc2* is assigned as the *Kc* value on date C (*KcC*) and D (*KcD*). Initially, the *KcC* and *KcD* values are set equal to *Kc2*, but for tree and vine crops, the values for *KcC* and *KcD* are adjustable for the percentage shading by the canopy to account for sparse or immature canopies. During the rapid growth period, when the field and row crop canopy increases from about 10% to 75% ground cover, the K_c value changes linearly from *KcB* to *KcC*. For deciduous tree and vine crops, the Kc increases from KcB to KcC as the canopy develops from leaf out on date B to about 70% shading on date C. During late season, the K_c changes linearly from *KcD* on date D to *KcE* at the end of the season. The values for *KcB* and *KcC* depend on the difference in (1) energy balance due to canopy density and reflective qualities, (2) crop morphology effects on turbulence, and (3) physiological differences between the crop and reference crop.

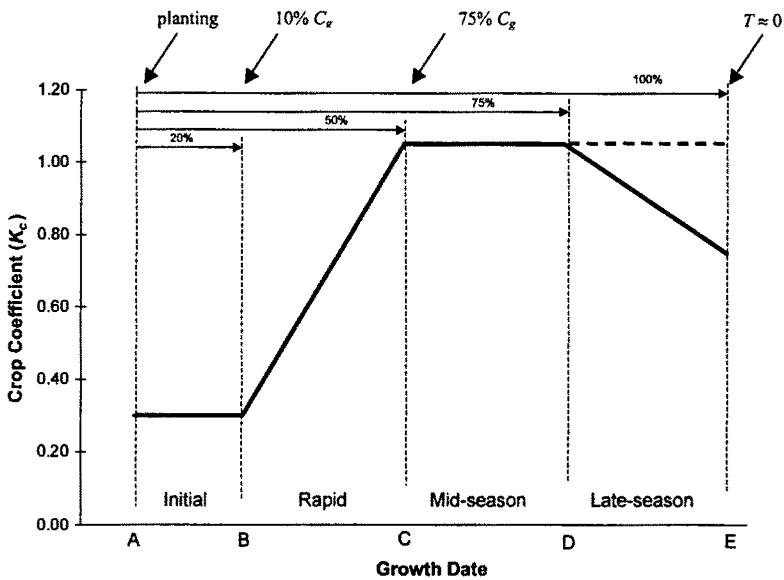

Figure 1. Hypothetical crop coefficient (K_c) curve for typical field and row crops showing the growth stages and percentages of the season from planting to critical growth dates.

Field crops with fixed crop coefficients

Fixed annual K_c values are possible for some crops with little loss in accuracy. These crops include pasture, warm-season and cool-season turfgrass, and alfalfa averaged over a season. In the SIMETAW program, these field crops are identified as type-2 crops.

Deciduous tree and vine crops

Deciduous tree and vine crops, without a cover crop, have similar K_c curves but without the initial growth period (Fig. 2). The season begins with rapid growth at leafout when the K_c increases from *KcB* to *KcC*. The midseason period begins at approximately 70% ground cover. Then, unless the crop is immature, the K_c is fixed at *KcC* until the onset of senescence on date D (*Kc2*=*KcC*=*KcD*). During late season, when the crop plants are senescing, the K_c decreases from *KcD* to *KcE*. The end of the season occurs at about leaf drop or when the tree or vine transpiration is near zero.

The initial K_c value is refined by using the K_c for bare soil evaporation on that date based on ET_o and rainfall frequency. The assumption is that the ET_c for a deciduous orchard or vineyard at leaf out should be about equal to the bare soil evaporation. The *Kc2* and *Kc3* values again depend on energy balance characteristics, (2) canopy morphology effects on turbulence, and (3) plant physiology differences between the crop and reference crop. The *Kc1* corresponds to *KcB* and *Kc3* corresponds to *KcE*. Again, the K_c is initially fixed at *Kc2* during midseason, so *Kc2=KcC=KcD*. However, the *KcC* and *KcD* can be adjusted for sparse or immature canopies. Adjustments can also be made for the presence of a cover crop.

With a cover crop, the K_c values for deciduous trees and vines are increased depending on the amount of cover. In SIMETAW, adding 0.35 to the in-season, no-cover K_c for a mature crop, but not to exceed 1.15, is used.

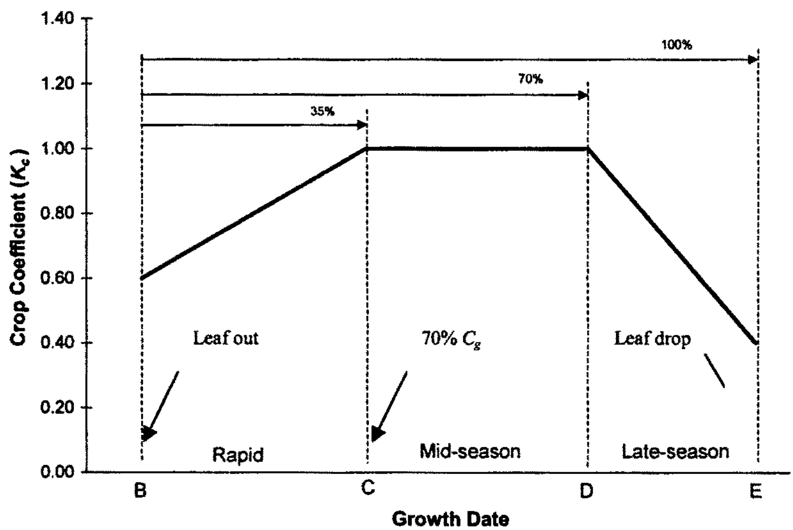

Figure 2. Hypothetical crop coefficient (K_c) curve for typical deciduous orchard and vine crops showing the growth stages and percentages of the season from leaf out to critical growth dates.

Subtropical Orchards

For mature subtropical orchards (e.g., citrus), using a fixed K_c during the season provides acceptable ET_c estimates. However, if higher, the bare soil K_c is used for the orchard K_c.

ET OF APPLIED WATER CALCULATIONS

The ET_o data come from the 'name.wrk' file, which is created from either input raw or simulated daily weather data. The K_c values are based on the ET_o data and crop, soil, and management specific parameters from a row in the 'DAUnnn.csv' file. During the off-season, crop coefficient values are estimated from bare soil evaporation as previously described. It is assumed that all water additions to the soil come from rainfall and losses are only due to deep percolation. Rainfall runoff as well as surface water running onto a cropped field is ignored. Because the water balance is calculated each day, this assumption is reasonable.

During the off-season, if the soil water depletion (SWD) is less than the YTD, ET_c is added to the previous day's SWD to estimate the depletion on the current day. However, the maximum depletion allowed is 50% of the PAW in the upper 30 cm of soil. If the SWD at the end of a growing season starts at some value greater than the maximum soil water depletion, then the SWD is allowed to decrease with rainfall additions but it is not allowed to increase with ET_c (Fig. 3). If half of the available water is gone from the upper 30 cm, it is assumed that the soil surface is too dry for evaporation. Once the off-season SWD is less than the maximum depletion, it is again not allowed to exceed the maximum off-season depletion.

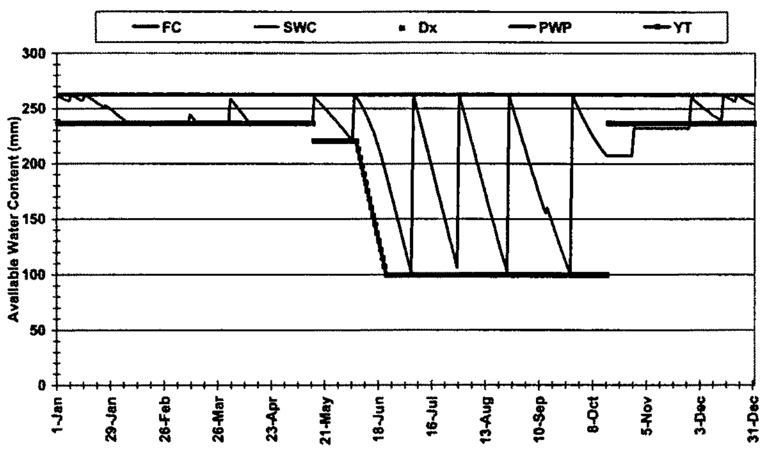

Figure 3. An annual water balance for cotton showing fluctuations in soil water content between field capacity and the maximum depletion during the off-season and between field capacity and the YTD during the season.

If a crop is pre-irrigated, then the SWD is set equal to zero on the day preceding the season. If it is not pre-irrigated, then the SWD on the day preceding the season is determined by water balance during the off-season before planting or leafout. It is assumed that the SWD equals zero on December 31 preceding the first year of data. After that the SWD is calculated using water balance for the entire period of record.

During the growing season, the SWD depletion is updated by adding the ET_c (or by subtracting ET_c from the soil water content 'SWC') on each day (Fig. 3). If rainfall occurs, SWD is reduced by an amount equal to the rainfall. However, the SWD is not allowed to be less than zero. This automatically determines the effective rainfall as equal to the recorded rainfall if the amount is less than the SWD. If the recorded rainfall is more than the SWD, then the effective rainfall equals the SWD. Irrigation events are given on dates when the SWD would exceed the YTD. It is assumed that the SWD returns to zero on each irrigation date. The ETAW is calculated both on a seasonal and an annual basis as the cumulative ET_c minus the effective rainfall. The calculations are made for each year over the period of record as well as an overall average over years. The results are output to a summary table.

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EVAPOTRANSPIRATION AND IRRIGATION WATER REQUIREMENTS FOR JORDAN'S NATIONAL WATER MASTER PLAN: GIS-BASED ET CALCULATIONS

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ABSTRACT

Evapotranspiration (ET) from irrigated cropland is a significant component of water consumption in Jordan. During 2000 and 2001, the MWI[5] created a large GIS-based system that includes components for processing ET and net irrigation requirements (NIR) for various agroclimatic zones in Jordan. The NIR system, named NIR_Calculator, is programmed within Microsoft Access database using a Visual Basic interface and is highly flexible.

Incorporation of irrigation-specific and culture-specific characteristics for crops has improved the prediction of evaporation and transpiration inside greenhouses and under plastic mulch. Computations include partitioning of irrigated crops into drip, sprinkler and surface irrigated classes, with evaporation from soil surfaces during initial crop development periods and during nongrowing periods calculated for each class separately. A monthly soil water balance determines the impact of stored soil moisture and off-season precipitation on NIR. Evaporation during nongrowing periods is considered when computing annual effective precipitation.

INTRODUCTION

Studies of projected water demand and supply in Jordan have shown that the water deficit is increasing with time, with demands on a finite quantity of good quality water ever increasing. Per capita availability of renewable water today is less than 175 cubic meters, already far below the projected regional average for the year 2025, and will decrease to 90 cubic meters by the year 2020 if water projects are not implemented (Taha and El-Nasser, 2002). In order to meet minimum current water demands for basic uses, over-pumpage of groundwater

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resources in Jordan is estimated to be at 100% above the safe-yield. Of the total groundwater supplied to all uses in 1998 (485 MCM), irrigated agriculture consumption accounted for about 53%, at some 258 MCM. Nearly 80% of the safe yield of the renewable groundwater resources and 40% of the non-renewable groundwater are currently used for irrigated agriculture throughout the country.

Agriculture constitutes about 70% of the overall water demand in Jordan. Therefore, it is important to obtain accurate and well-organized estimates of current and future water consumption by agriculture. This has motivated MWI to create a software tool for the projection and management of irrigation demand in the nation, given available water quantity and quality information.

BACKGROUND AND OBJECTIVES

In the early 1990's, a UN-DP program assisted MWI in refining a computer-based procedure for structuring and housing water and crop-related data for the country. Software was in the form of early data-bases and most planning computations and summaries used spread-sheets. Beginning in the mid-1990's, and under the scope of work of the Water Sector Planning Support project funded by the GTZ[6], a suite of digital water balance tools was designed and implemented[7]. The objective is to enable the Ministry to carry out nation-wide water balances using recent data and various development scenarios to support efficient water sector planning. Digital water balance tools incorporating nine modules were developed for the assessment of various water demands as well as water resources including wastewater and water losses.

One of the most important tools implemented for the calculation of water demand at MWI is the Irrigation Model. The Irrigation Model consists of 3 separate modules for pre-processing climatic data, calculation of reference ET (ET_0) and computing monthly net irrigation requirements of crops (NIR). These calculations are combined to estimate present and future irrigation demands for the whole country and selected planning/development regions. The Irrigation Model is GIS-based (Jacobi, 2001) and is linked to a Relational Database Management System under Oracle thus allowing for the updating of data and visual presentation of the results.

PRINCIPAL DESIGN OF THE TOOL

The Irrigation Model is implemented under the Microsoft Access database, and is linked to a central Oracle database and GIS databases containing Arc View shape

[6] German Agency for Technical Cooperation
[7] Conceptual Design was done by Ministry of Water and Irrigation, whereas software development was by AHT International GmbH in close cooperation with GTZ.

files. The software system models irrigation water demand in the future based on existing information regarding cropping patterns, irrigated areas, information on crop water requirements, in combination with application and conveyance methods, and leaching requirements given water quality. Prediction of future water demand is tied to a reference year demand assessment. This permits the evaluation of the current irrigation demand situation in the spatial unit under consideration, with respect to water availability and water quality, prior to proceeding with assumptions regarding the future. Irrigation demand data are aggregated to demand centers such as towns and villages, developed irrigated areas and project areas that are geographically represented via point information in ArcView GIS.

The Irrigation Model requires entry of various data from databases, including:

- Irrigated areas for each crop group and demand center
- Distribution of irrigation methods for irrigated areas under the various application methods
- Leaching requirements for every crop group and salinity class
- Monthly net water requirements (NIR) for each crop group, given the agroclimatic zone in which it is grown
- Application efficiency tables

During operation of the Irrigation Model, selection of individual irrigation centers for which irrigation demand is to be calculated is enforced and performed within Arc View (Figure 1). The selection of demand centers is possible for various spatial units of analysis, such as agroclimatic zone, governorate, groundwater basins, surface water catchments, and subcatchments.

CALCULATION OF NET IRRIGATION REQUIREMENTS (NIR)

Besides statistics on irrigated areas and efficiencies, net irrigation requirements are the principal factors for determining irrigation demand. Net irrigation requirements are calculated using an MS Access-based application, namely the NIR_Calculator and exported to Oracle for further use by the irrigation demand model. The NIR_Calculator uses climatic data and crop factors for various growing stages of crops, with consideration for the application method (surface, drip, and sprinkler) and type of cultivation (open and plastic houses). NIR is calculated on a monthly basis for each crop group, application method, and agroclimatic zone under various year types (median, dry or wet). The application uses monthly ET_0 computed using an early version of the FAO-56 Penman-Monteith equation (FAO, 1990) with monthly precipitation, cropping calendars, and FAO crop coefficients (K_c) for each agroclimatic zone in the country.

The NIR_Calculator was originally coded in a Lotus 123 data base during the early 1990's UNDP work, but has been since converted to an Access application using Visual Basic programming (Jacobi, 2001). Various information compiled by NIR_Calculator includes:

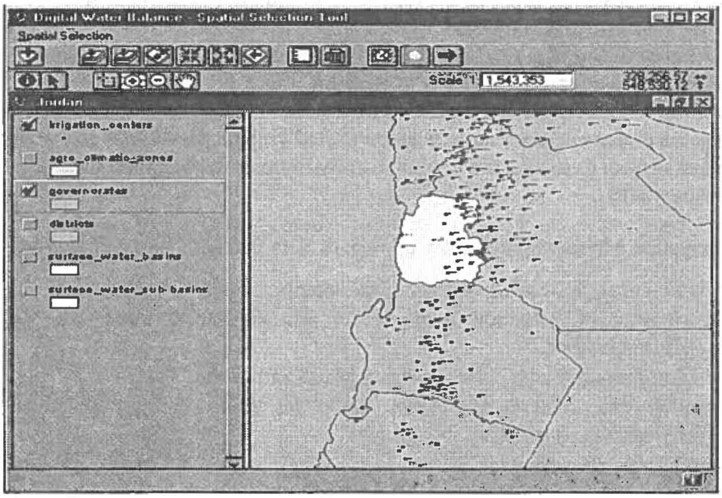

Figure 1: Selection of AgroClimate or Demand Zone within the Irrigation Model (from Taha and El-Naser, 2001)

Crop Factors:

- Length of the crop development stage; initial, development, mid and late seasons (from FAO).
- K_c during the initial, mid and late season
- Minimum possible irrigation depth
- Maximum late season depletion
- Initial root depth of the crop and depth at full development
- Available moisture in the soil
- User specified maximum allowed depletion depending on the crop type.

Meteorological data:

- Reference crop evapotranspiration ET_o
- Number of rainy days
- Rainfall amounts
- Effective rainfall.

Crop coefficients are organized and specified by crop and irrigation method (surface, sprinkler, drip), and cultural environment (open field, greenhouse, open field with plastic mulch, and time of year). This organization allows the values for K_c to be customized to reflect effects of the specific irrigation system type and environment on soil evaporation.

Two separate means for processing rainfall and ET were applied by the Ministry and GTZ: a) the calculation of effective rainfall and ET for historic years and b) a statistical evaluation for dry, median and wet years calculated for agro-climatic zones rather than for individual stations.

The calculations in the NIR_Calculator are subdivided into 5 steps (Figure 2):

Step 1: Calculation of the crop coefficient during the initial growth period, $K_{c\ ini}$

Step 2: Calculation of crop evapotranspiration using ET_0 and crop factors

Step 3: Calculation of net effective precipitation during non-growing season

Step 4: Calculation of NIR

Step 5: Export of NIR results to the relevant database table.

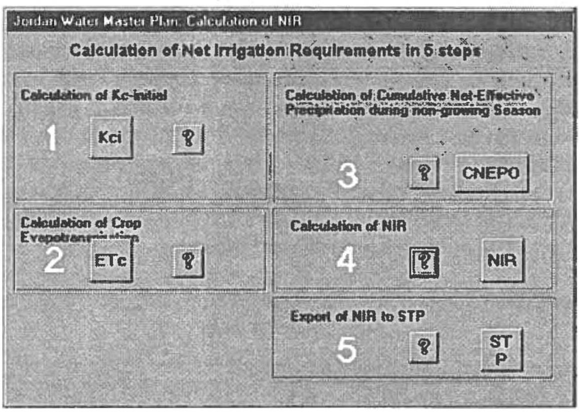

Figure 2: Control screen of the NIR_Calculator Form of MS-Access

PERTINENT CALCULATIONS

This section describes some of the ET and NIR procedures that were developed during the study for use in NIR_Calculator. Several enhancements have been made to the NIR_Calculator to account for effects of late season depletion, off-season precipitation, and preplant irrigation on annual NIR.

Net Irrigation Water Requirement Equation

The NIR for any month "i" is calculated for outside environments as:

$$NIR_i = \text{Maximum}\left((ET_{c\ i} - R_{eff\ i})\,Days_i\ ,\ -0.5\,RAW_i\right) + PPI_i - LSD_i \qquad (1)$$

where $ET_{c\,i}$ and $R_{eff\,i}$ are daily crop ET and effective rainfall for the month (or period) i, $Days_i$ is the number of days in the month, RAW_i is the maximum readily available moisture for month i, PPI_i is any net preplant irrigation requirement for month i (0 unless immediately prior to planting), and LSD is late season depletion during month i (0 unless during the month or months immediately prior to harvest).

The maximum of net $ET_{c\,i}$ and -0.5 RAW_i in (1) insures any excess precipitation during rainy months (where $ET_{c\,i}$ - $R_{eff\,i}$ is negative) does not exceed the average ability of the soil to retain the excess precipitation in the root zone. This average ability (or capacity) is estimated to be one half of the maximum allowable depletion of the soil for any month. This way, the total NIR includes the benefit and effect of useable excess precipitation during the negative value months.

Limits on Negative NIR

When negative values of NIR are encountered that are caused by effective precipitation exceeding ET requirements, the NIR sub-module calculator compares the negative values to those for the following month to insure that the sum of negative NIR for two or more consecutive months is never more negative than the –RAW. This prevents more storage in the soil than is possible. Excess storage is assumed to become deep percolation from the root zone, and ultimately, ground-water recharge. The RAW used in (1) is based on the maximum rooting depth for the crop. Negative values for cumulative CNIR at the end of the growing season are discarded and set to zero. Table 1 illustrates calculation of NIR with the –RAW limit invoked during wet months.

Table 1. Example NIR calculation for citrus in zone 5 of Jordan for a wet season, months Dec. - Feb.

Month	ET_c, mm/d	R_{eff}, mm/d	$ET_c - R_{eff}$, mm	NIR, mm	□NIR, mm
12	0.97	3.3	-72	-51	-51
1	1.48	3.1	-51	-51	-102
2	1.95	3.2	-35	0	-102

In the case of the citrus example in Table 1, the RAW equals 102 mm, with only one-half of this allowed to be added to soil storage during any month. Beginning with December (the first month having a negative NIR), one would have, by the end of January, a summed negative NIR = -102 mm, which is just OK (i.e., less than or equal to RAW). However, by the end of February, which is the third consecutive month having negative NIR, one would have a summed negative NIR over the three months = -137 mm, which is more "carryover" than the soil can hold. Therefore, the NIR for February is set to 0 mm.

This example presumes that the soil is depleted to RAW at the time of harvest. This is a reasonable assumption for many field crops. For some crops, however, management may keep the soil moisture at higher levels, for example for fruit quality or to make harvesting easier, so that RAW is too large a value to use in estimating month to month storage carryover. One may tend to use a smaller value for RAW in those instances.

Late Season Depletion (LSD)

Late season depletion accounts for the practice of utilizing moisture stored in the root zone near the end of the growing season, without replacing it. This has the effect of reducing the within-season NIR. End of season soil moisture depletion is assumed to be later replenished by precipitation during off-season or by a future pre-plant irrigation. Late season depletion is computed in the NIR Module as the minimum of RAW and a user-specified maximum allowed depletion (USMD) due to cultural or other requirements (Allen, 1991):

$$\mathrm{LSD} = \mathrm{Minimum}[\mathrm{RAW}, \mathrm{USMD}] \tag{2}$$

The RAW in mm is computed as:

$$\mathrm{RAW} = \mathrm{AM}\frac{\mathrm{MAD}}{100}R_{z\,max} \tag{3}$$

where AM is the total available water stored in the root zone (one or two days after irrigation) represented by the soil water content between field capacity and wilting point, and $R_{z\,max}$ is the maximum rooting depth. AM varies with the type of soil and averages about 150 mm/m soil depth. The MAD represents the level of maximum allowed soil water depletion tolerated to maintain potential crop growth and it varies with the type of crop. Values for MAD and $R_{z\,max}$ are found in Doorenbos and Pruitt (1977) and Allen et al., (1998).

The USMD was set to 10 mm for root crops such as carrots, potatoes and onions to assure moist soil during harvest to facilitate extraction of the crop from the soil without damage. For other crops, such as wheat and barley, which tolerate higher depletion levels, USMD was set to 100 mm.

Cumulative Net Effective Precipitation During The Offseason (CNEPO)

Cumulative Net Effective Precipitation during the off-season is used to predict soil moisture depletion at the beginning of a new crop growing season (i.e., at the planting date). CNEPO is based on the estimated late season depletion amount and net precipitation accumulated between harvest and planting dates. Net effective precipitation for each month is calculated as effective precipitation less ET, where effective precipitation is taken as 0.7 P and ET for each month is calculated using the estimate for $K_{c\,ini}$ during the nongrowing season, referred to as $K_{c\ ngs}$, and which is based on rainfall frequency:

$$CNEPO = \sum_{1}^{n}\left[R_{eff\ i} - K_{c\,ngs\ i}\ ET_{o\,i}\right]Days_i \qquad (4)$$

where n is the number of months or periods outside the growing season, and $K_{c\ ngs\ i}$ is K_c during month i of the nongrowing season. Beginning season depletion is calculated as LSD – CNEPO, with a lower limit of 0.

Allowable Depletion During the Initial Period (AD_I)

AD_I is the moisture required to moisten the seed bed and is computed as:

$$AD_I = AM\frac{MAD}{100}R_{z\,I} \qquad (5)$$

where $R_{z\ I}$ is root depth during the initial period. $R_{z\ I}$ for annual crops is estimated as the planting depth of the seed plus 5 to 10 cm to represent upward movement of moisture toward the seed and rapid root development. A typical value is 0.15 m. For perennial crops, a typical value is 0.7 m.

Preplant Irrigation Depth (PPI)

The preplant irrigation depth is determined by comparing the minimum physically possible net irrigation depth (MPID) with difference AD_I - P_{eff} and difference LSD – CNEPO. The PPI calculation presumes that the same crop is planted each year, and just once per year, on the same land unit. It is calculated as:

$$PPI = Minimum[MPID, Maximum(AD_I - R_{eff}, LSD - CNEPO)] \qquad (6)$$

Minimum Possible Irrigation Depth (MPID)

MPID is the minimum depth of water that can physically be added to the soil due to constraints in the irrigation application system. A surface system may have to apply 40 mm average irrigation depth to push enough water across the basin or along the furrow to just infiltrate 5 mm at the furthest point. Because drip and sprinkler irrigation systems are better controlled, the MPID for these systems is 0.

NIR under greenhouses

Inside greenhouses, there is no effective rainfall, so that:

$$NIR_{inside\ i} = ET_{c\ i}\ Days_i + PPI_i - LSD_i \qquad (7)$$

The K_c during midseason ($K_{c\ mid}$) and during the late season ($K_{c\ end}$), i.e., the second and third anchor points for the FAO style of K_c curve, are adjusted for predicting ET inside plastic greenhouses by multiplying by 0.75 based on work by Mazahrih (2001) in Jordan Valley.

Calculation of the Crop Coefficient during the Initial Period

K_c during the initial period, $K_{c\ ini}$, is calculated using Cuenca (1987) for reproducing Fig. 6 of Doorenbos and Pruitt (1977). This equation was employed for consistency with past usage by the Ministry. However, the application can also be applied using $K_{c\ ini}$ equations of FAO-56. Wetting events include both rainfall and irrigation. In NIR_Calculator, the $K_{c\ ini}$ method was applied to both the initial period of crops and to periods between crops (nongrowing periods).

Following Cuenca (1987),
If $I_M < 4$ days then:
$K_{c\,ini} = [-0.27\ \mathrm{Ln}(I_M) + 1.286]\ \mathrm{Exp}[(-0.042\ \mathrm{Ln}(I_M) - 0.01)\ ET_o]$ Otherwise, if $I_M >= 4$ days then:

$$K_{c\,ini} = 2\ (I_M)^{-0.49}\ \mathrm{Exp}[(-0.04\ \mathrm{Ln}(I_M) - 0.02)\ ET_o] \quad (8)$$

where I_M is the mean wetting interval within the initial cropping period and is based on rainfall frequency and a water balance of the evaporation layer or seedbed of the soil. The depletion layer during the initial period is generally assumed to be 10 to 15 cm in depth, or deeper for perennials, and the water readily available in this layer is AD_I, calculated by (5). Units of ET_o are mm/day. The ET_o, wetting interval and R_{eff} parameters are averages over the initial period.

$K_{c\ ini}$ is calculated using an iterative procedure to find the unknown irrigation interval, I_{irrig}. During the first iteration, $K_{c\ ini}$ is estimated using I_M based on rainfall only, and the subsequent $ET_c = K_{c\ ini}\ ET_o$ is compared to total rainfall:

$$\text{If } K_{c\,ini}\ ET_o > R_{eff} \text{ then } I_{irrig} = \frac{AD_I}{K_{c\,ini} ET_o - R_{eff}} \quad (9)$$

Otherwise, if $K_{c\ ini}\ ET_o <= R_{eff}$, then I_{irrig} is set to an arbitrary 50 days. The I_{irrig} is considered to be necessary, and followed by farmers, to maintain adequate moisture in the seedbed for germinating and establishing the crop. In this manner, impacts of I_{irrig} on evaporation are incorporated.

An average wetting interval is calculated to consider wetting by both rainfall and irrigation, based on a geometric mean (Allen, 1991). The interval mimics an irrigation schedule during the initial period that maintains sufficient moisture in the upper soil layer conducive to seed germination and root development:

$$I_M = \mathrm{Int}\left(\frac{1}{\frac{1}{I_{rain}} + \frac{1}{I_{irrig}}}\right) + 1 \quad (10)$$

This new estimate for I_M is reinserted into (8) for $K_{c\ ini}$ and a new value for $K_{c\ ini}$ is calculated. The product $K_{c\ ini}\ ET_o$ is again recomputed and compared to R_{eff} and a new I_{irrig} is computed. The process is repeated until $K_{c\ ini}$ and I_M have stable values.

$K_{c\ ini}$ for Perennials

For perennials, the calculation of $K_{c\ ini}$ requires modification, since the ground surface is not entirely bare, so that the basal condition, $K_{cb} > 0.15$. The calculation of $K_{c\ ini\ perennials}$ considers evaporation from both rainfall and irrigation. However, the evaporation is reduced due to the shading effects of vegetation. The "optimal" irrigation frequency is determined to retain sufficient soil moisture in the initial root zone, which for a perennial is assumed to be substantially close to maximum rooting depth. In the following computations for $K_{c\ ini\ perennials}$, the values for ET_o, I_{rain}, and R_{eff} are averages over all months in the initial period. This procedure, based on Allen (1991 and 2001), uses the Cuenca equation to retain consistency with early work within the Ministry, but is applied to evaporation from a partially vegetated surface from FAO-56, and can be applied with the $K_{c\ ini}$ equations of FAO-56.

Evaporation is regulated under perennials by the fraction of exposed area not covered by vegetation. A simplification of the FAO-56 approach is applied to predict f_{ew} for use with Cuenca's equation, where f_{ew} is the fraction of ground surface that is exposed and wetted by precipitation or irrigation, predicted as:

$$f_{ew} = 1.2 - K_{cini\_residual} \tag{11}$$

with limits of $0.01 < f_{ew} < 0.99$, where $K_{c\ ini\_residual}$ is the same as the basal K_{cb} for a perennial crop, and represents ET for the crop when it has a dry soil surface.

It is assumed that evaporation from rainfall occurs in the exposed portion of the field, in between beds (i.e., in f_{ew}), when there is no plastic mulch or when the plastic mulch is covered by sufficient soil. For surface or sprinkle irrigation, the total "drying" of the soil is computed as a weighted average based on f_{ew}. The irrigation frequency of perennials for drip with plastic mulch along the beds is less coupled with rainfall. For drip, it is assumed that the bed is mulch covered so that irrigation does not substantially impact $K_{c\ ini}$. The same basic equation for $K_{c\ ini}$, i.e. that by Cuenca (1987) is used, but with ratioing according to f_{ew}.

The evaporation from the f_{ew} area is added to the basal $K_{c\ ini\ residual}$

$$K_{c\,ini\,peren} = K_{c\,ini\,(Eqn.8)}\ f_{ew} + K_{c\,ini\,residual} \tag{12}$$

where $K_{c\ ini\ (Eqn.\ 8)}$ is $K_{c\ ini}$ from (8). Limits are placed so that:

$$K_{c\,ini\,peren} = \text{Minimum}\left|K_{c\,ini\,peren\,(Eqn.12)}, \text{Maximum}(1.25,\ K_{c\ ini\ residual})\right| \quad (13)$$

Similar to the condition for bare soil, an "optimal" irrigation interval, I_{irrig}, is recommended if predicted evaporation exceeds effective rainfall:

$$\text{If } K_{c\,ini\,peren}\,ET_o > R_{eff} \text{ then } I_{irrig} = \frac{AD_I}{K_{c\,ini\,peren}\,ET_o - R_{eff}} \quad (14)$$

where AD_I is water that is readily available in the rooting zone during the initial period. If $K_{c\ ini\ peren}\ ET_o <= R_{eff}$, then I_{irrig} is set to an arbitrary 50 days. (14) is not invoked for drip irrigation, where it is assumed that the surface is generally wetted within the shade of the vegetation, so that no extra evaporation from irrigation occurs. Therefore, for drip, I_{irrig} = 50 days. Once a value for I_{irrig} is predicted, a geometric mean wetting interval (I_M) is calculated using (10). The process iterates on $K_{c\ ini\ peren}$ and I_{irrig} and I_M until values are stable.

NonGrowing Season ET

The $K_{c\ ini}$ function (8) is applied during the nongrowing season to predict total evaporation and associated effectiveness of precipitation during the nongrowing season. This calculation is part of the prediction of the net change in soil water storage during the period from harvest of one crop until the planting of another. Application of the $K_{c\ ini}$ calculation is made on a monthly basis. The calculation assumes that the soil surface is void of green vegetation. Where green vegetation exists during the nongrowing season, $K_{c\ ini\ peren}$ from (12) can be used to approximate $K_{c\ ngs}$. In the application to the nongrowing period, it is assumed that there is no irrigation, so that I_M in (8) is set equal to I_{Rain}, and no iteration is necessary. However, an upper limit is applied, so that:

$$\text{If } K_{c\,ngs}\,ET_o > R_{eff} \text{ then } K_{c\,ngs} = \frac{R_{eff}}{ET_o} \quad (15)$$

where $K_{c\ ngs}$ is the K_c during the nongrowing season and is from (8). This conditional limits evaporation to effective rainfall.

OPERATION OF NIR_CALCULATOR FOR IRRIGATION DEMAND

Following the calculation of monthly NIR, results are exported and saved in the relevant tables in the Central Oracle database. Irrigation demand assessment proceeds using the irrigation demand model, which can be linked on-line to this database. Figure 3 illustrates the user interface used in the irrigation model to specify irrigation types and efficiencies, and resulting water demands.

Figure 4 illustrates how monthly water demand data are presented to the user by main crop group (MCG) and month, in tabular and graphical form, under dry, median and wet conditions.

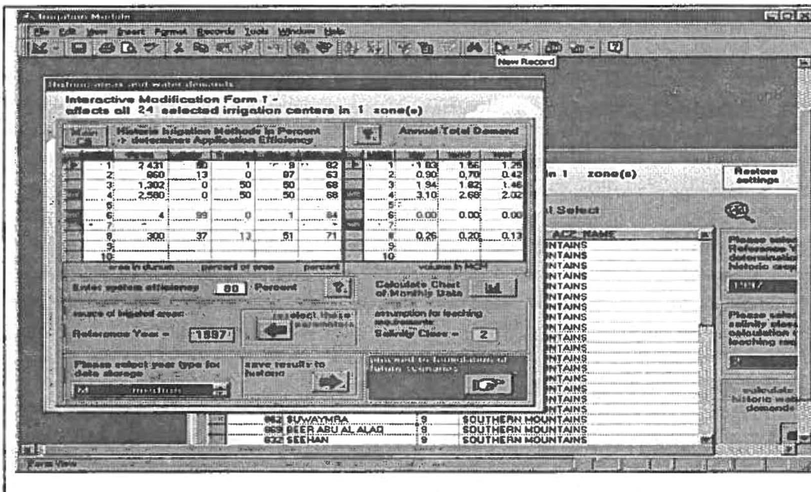

Figure 3: Example of selection of irrigation system type and efficiency and resulting water demand (from Taha and El-Naser, 2001).

Calculation of Future Irrigation Demand

Up to three development scenarios can be considered for the projection of future demand. The scenarios cover the time span until the year 2040 and are performed in five-year intervals. Parameters that can be entered for each interval include:

1) Projected increase or decrease in irrigated areas shown against current figures for the reference year.
2) Projected distribution of irrigation methods; percent drip, percent sprinklers and percent surface.
3) Projected gains in conveyance and application efficiency
4) Projected irrigation water salinity class

Comparisons between irrigation demand predicted from the irrigation model and water diversions and deliveries recorded by the Ministry are providing feedback concerning any need to modify (i.e., reduce) K_c values in NIR_Calculator to reflect impacts of water stress, salnity, low plant density, and water management on total ET. In some areas, the "pristine" assumptions of FAO K_c factors (Allen et al., 1998) may overpredict total ET from an area. Therefore, reducing factors may need to be developed.

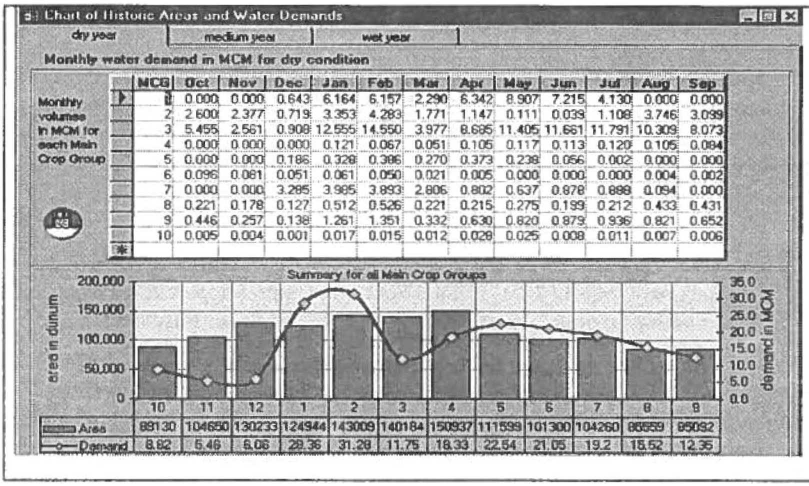

Figure 4: Example computation of monthly historic demand for main crop groups (MCG) (from Taha and El-Naser, 2001).

SUMMARY AND CONCLUSIONS

The irrigation demand model is one of several tools that have been developed to enable the management of the present and future water balance in Jordan. The model aims at prediction and management of irrigation demand given certain development scenarios and based on recent data. As such, the model can be used to evaluate present irrigation demands with respect to actual water use and availability, prevailing cropping patterns, irrigation methods and efficiencies. The tool's reporting flexibility allows digital output of various types of data for additional processing. Being GIS-based, spatial examination and analysis of both the present and future irrigation demands can be made. In addition, the digital nature of the model and its database dependency allows for easy updating of the tool. These features permit more flexible responses to merging new realities and changing conditions.

The interactive feature of the model allows testing the impact of various development scenarios on irrigation demand and subsequently on water balancing, thus allowing for review of water strategies. In addition, scenarios reflecting various water sector strategies and policies can be examined using the model, and as such, the tool can be used as one of several elements to support decision making with regards to demand and supply management strategies, and hence the optimization of water resources.

The NIR calculator is flexible in regard to programming different relationships among NIR components, so that it can be updated according to various countries' needs and to reflect the latest findings in the field.

ACKNOWLEDGEMENTS

Mr. Lothar Nolte of GTZ, Amman, provided valuable coordinating and interfacing. Dr. Peter McCornick of ARD, Inc., Amman, provided background and support. Engineer Rasha Sharkawi of MWI assisted with Access data base operation and NIR_Calculator debugging. Work by R.G. Allen was supported by ARD, Inc. with funding by US-AID.

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ROUTING DEMAND CHANGES WITH VOLUME COMPENSATION: AN UPDATE

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Albert J. Clemmens
Theodor S. Strelkoff

ABSTRACT

Using the gate-stroking method, this paper shows that a complex open-channel flow feedforward control problem can be treated as a series of linearly additive single flow-change control problems. A key element of this approach is determining the initial conditions for each single flow-change problem. An inadequate choice of initial conditions will result in under or overestimation of the canal storage volume change needed for the new steady-state conditions. These findings provide support to a simple feedforward control scheme based on volume compensation and time delay. An example is used to demonstrate that the simple scheduling approach is nearly as effective in controlling water levels as the complex gate-stroking approach.

INTRODUCTION

Bautista and Clemmens (1998) proposed a simple method for routing known demand changes through an open-channel water delivery system (the feedforward control problem) using the concept of volume compensation. Volume compensation refers to the volume of water that needs to be added or removed from a canal pool in going from an assumed initial steady-state to a desired new steady-state condition. That volume is delivered through a small number of step changes in inflow rate. The magnitude of those changes depends on estimates of the time needed for the flow changes to travel the length of the channel (the travel delay time τ). A key problem of volume compensation is determining this delay, and thus, the timing of the inflow changes.

Simulation studies have demonstrated the application of the volume-compensating feedforward control method to specific water delivery systems (Bautista and Clemmens, 1998; Bautista and Clemmens, 1999a). Additional research is needed to generalize those results and to identify limitations of the method. A recent study used gate-stroking (Wylie, 1969) and volume compensation to examine the characteristics of feedforward control solutions for single-pool canals of uniform geometry (Bautista et al, 2002). The gate-stroking

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method solves the governing equations of unsteady open-channel flow inversely in space. The study considered a wide range of canal geometries and flow configurations. The gate-stroking method can fail to find a solution or can produce a solution requiring discharges exceeding the canal capacity or flow reversal under conditions where the time needed to supply the canal volume change is small relative to the disturbance wave travel time. Volume compensation offers a solution under those conditions and the resulting water level control is satisfactory. There are also conditions under which upstream flow changes travel with little attenuation and, therefore, the inflow hydrograph computed by gate-stroking nearly matches the desired outflow hydrograph. Under those conditions, a volume-compensating schedule can be easily identified and will produce water level control comparable to that obtained with gate-stroking.

Bautista and Clemmens (1998) outlined a volume-compensation strategy for multi-pool canal systems subject to multiple changes, but provided no justification for the approach. Recent tests, not reported here, with canal systems subject to multiple flow changes have resulted in adequate control for some demand changes but less adequate for others, suggesting problems with the original approach. The purpose of this paper therefore is to reexamine the basic concept used and to refine the method.

MULTI-POOL SYSTEMS: ADDITIVITY OF SOLUTIONS

The volume-compensating feedforward control method for multi-pool systems suggested by Bautista and Clemmens (1998) treats the multiple flow change problem as a series of linearly additive single flow change problems. Because the governing equations of unsteady open-channel flow are nonlinear, one can not expect this assumption to hold in general. This section analyzes the linearity of feedforward control solutions, using the full Saint Venant equations (the gate-stroking method) under a specific set of flow conditions. Determining conditions under which gate-stroking solutions are additive should suggest conditions under which the feedforward control problem can be treated as a linear problem.

This analysis uses one of the test cases proposed by the ASCE Task Committee on Canal Control Algorithms (Clemmens et al, 1998), ASCE Test Canal 2, Scenario 2. Canal characteristics and test details are given in Table 1. The canal is 28 km long and relatively flat. The canal's geometry, together with the specified flow conditions, results in a low Froude number for all pools. All pools are entirely in backwater for the initial flow conditions. This means that disturbances can travel up and down the canal for a long time and, thus, flow levels can oscillate for a long time. In a previous study, a finite-difference gate-stroking model for multiple pools (Bautista et al. 1997) was used to compute a feedforward flow schedule for this test case and was shown to produce satisfactory water level control (Bautista and Clemmens, 1999b). In this paper, rather than processing all demand changes simultaneously as was done in that reference, each flow change was processed individually, as is described next.

Table 1. ASCE Canal Control Test Case 2-2: geometric§ and flow data

Pool	Pool Length (km)	Pool Bottom Width (m)	Pool Target Downstream Depth (m)	Initial Pool Inflow (m^3/s)	Initial Offtake Flow (m^3/s)	Offtake Flow Change (m^3/s)
1	7.0	7.0	2.1	2.7	0.2	1.5
2	3.0	7.0	2.1	2.5	0.3	1.5
3	3.0	7.0	2.1	2.2	0.2	2.5
4	4.0	6.0	1.9	2.0	0.3	
5	4.0	6.0	1.9	1.7	0.2	
6	3.0	5.0	1.7	1.5	0.3	0.5
7	2.0	5.0	1.7	1.2	0.2	1.0
8	2.0	0.6	1.7	1.0‡	0.3	4.0

§ For all pools, bottom slope = 0.0001, side-slope = 1.5, and Manning n = 0.02

‡ Flow past the canal's tail end is 0.7 m^3/s.

In the example, flows change at six of the eight turnouts three hours after the beginning of the test[2]. Since all demand changes take place at the same time, it is clear that the change in the most-downstream pool has to be routed first (i.e., requires the earliest change in inflow at the head of the canal). Initial conditions for that sub-problem are, simply, the time-zero initial conditions (discharges and levels). The second demand change to be routed is that originating in the penultimate pool, 7. Assuming a new steady-state as a result of the demand change in pool 8, initial flows for this second sub-problem are the sum of the initial flows and the demand change for the first sub-problem (a flow increase of 4.0 m^3/s in all pools). Initial water levels depend on these flows and the prescribed downstream target level. The same logic can be applied to determine the initial conditions of all remaining flow changes.

Solutions were combined for each check structure by adding all *flow increment* hydrographs for that particular check structure to its time-zero initial discharge. As an example, for the head gate, the time-zero initial discharge is 2.7 m^3/s (table 1). Since six individual offtake flow changes need to be processed, six different hydrographs are computed for the head gate. The flow increment hydrograph

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[2] The Test Case originally requires changes to occur two hours after the beginning of the test (Clemmens et al., 1998). This time was modified to allow the initial flow changes at the head gate to occur at a time greater than time zero.

resulting from each demand change is the difference between the gate-stroking solution and the initial conditions for that particular sub-problem. Since demand changes at a location do not affect check flows downstream from that location (once unsteadiness caused by the change has dissipated), the number of flow increment hydrographs that needs to be combined decreases as the check is located farther downstream. For example, for the check structure between pools 6 and 7, the combined hydrograph is simply the solution to the individual demand change in pool 8 plus the flow increment hydrograph due to the change in 7.

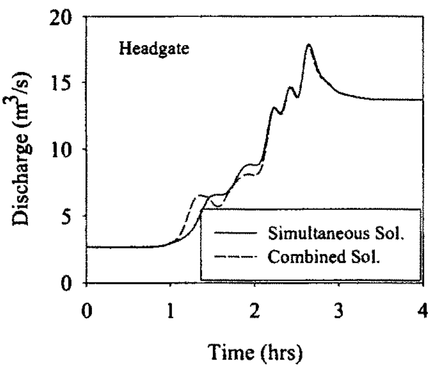

Figure 1. Gate-stroking inflow hydrographs for ASCE Test Case 2-2: simultaneous and combined solutions

Figure 1 compares the linearly combined and nonlinear simultaneous solutions obtained for the head gate. The solutions are nearly in agreement for most of the hydrograph. The mismatch in the initial part of the hydrograph suggests that the difference is related to the demand change or changes at downstream pools, since those changes would require the earliest flow changes at the head gate.

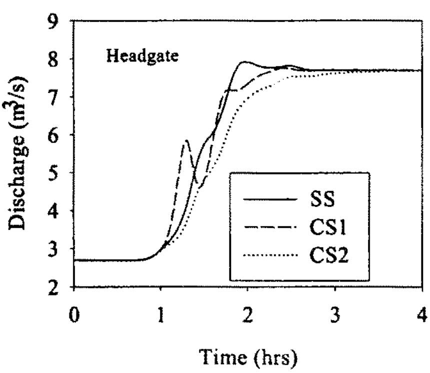

Figure 2. Gate-stroking inflow hydrographs for two offtake flow change problem: simultaneous (SS) combined (CS1, CS2) solutions

To understand the above mismatch, gate-stroking solutions were developed for a simpler problem, consisting of the demand changes in pools 7 and 8 only. Two different combination solutions (CS1, CS2) for the head-gate are shown in Figure 2, along with the simultaneous solution (SS). Solution CS1 is based on the same assumption used in the preceding analysis, namely that in processing the demand change in pool 7, prior changes (i.e., the change from pool 8) have reached steady-sate conditions. In contrast, solution CS2 assumes that the prior change in pool 8 has not taken place. That change is larger than the initial canal flow so it is likely that the resulting steady state will not be reached until after the change in pool 7 takes place. Because initial conditions are difficult to identify, the same initial conditions used to process the change in pool 8 were applied to process the demand change in pool 7. In comparison with the hydrograph from the simultaneous solution (SS), the CS1 hydrograph shows a large flow rate increase and then a large decrease. Those oscillations are not present in the CS2 hydrograph and, the hydrograph's shape is closer to the simultaneous solution. Notice however that the volume of water delivered to the canal with CS2 is less than that delivered by the simultaneous solution (the volume can be calculated by integration of the hydrograph with respect to time). This volume mismatch should cause water levels to temporarily deviate from their target value. Clearly, the steady conditions assumed by the original approach, CS1, result in an incorrect estimation of the transient response, however they do account more accurately for the needed volume change (the resulting volume is in close agreement with the volume delivered by the simultaneous solution hydrograph).

Determining the initial of conditions of each sub-problem is easy for the Test Case and the order in which each demand change needs to be routed is evident. If the demand changes take place at different times, determining the order in which they need to be routed, and the resulting impact on initial conditions of subsequent flow changes, is less obvious. This problem was solved as follows: individual gate-stroking solutions were generated for a set of demand changes (with changes in the pools at different times) using the time-zero initial conditions for each individual sub-problem. The solution requiring the earliest flow change at the head gate was then assumed the first to be routed. The final conditions resulting from this first demand change were then used to define new initial conditions for the remaining set of demand changes, from which the next demand change to be routed was identified. The process was continued until all demand changes were processed. This approach was applied to modified versions of the Test Case, with demand changes taking place at different times. Results of these tests, which are not presented here, again showed reasonable agreement between the hydrographs computed by routing all changes simultaneously and those computed by routing the changes individually and then combining them.

These results show that the complex feedforward control problem, consisting of multiple pools and flow changes, is somewhat linear. Difficulties in applying this approach are likely to be encountered when dealing with very large flow rate changes, as such changes would result in long-lasting unsteady flow. In such cases, one could consider interpolation, to estimate a more representative set of initial conditions for a given flow change. While that approach may reflect better the dynamics of the transient, it will not satisfy its volume compensation requirements. The simpler and more consistent approach is to assume that each individually routed demand change completely defines the initial conditions for the next change.

SIMPLE VOLUME COMPENSATION SOLUTION

A volume-compensating feedforward control schedule for a single demand change in a single-pool canal can be obtained by dividing the pool's volume change ΔV by the travel delay τ (Bautista and Clemmens, 1998; Bautista et al. 2002):

$$\Delta Q_1 = \frac{\Delta V}{\tau} \tag{1}$$

ΔQ_1 represents the flow rate change at the upstream check structure. The desired final steady-state check discharge, Q_f, is the sum of the initial steady-state check discharge, Q_0, and the demand change, Δq_d. Depending on the value of τ, $Q_0 + \Delta Q_1$ may not match Q_f. Therefore, a second check-flow change, ΔQ_2, will likely be needed to adjust the check discharge to Q_f.

$$\Delta Q_2 = \Delta q_d - \Delta Q_1 \tag{2}$$

For the range of conditions examined in Bautista et al (2002), suggested bounds for τ are:

$$\tau_{DW} \leq \tau \leq \tau_{\Delta V} \tag{3}$$

τ_{DW} is a delay estimate based on dynamic wave theory,

$$\tau_{DW} = \frac{L}{v_0 + c_0} \tag{4}$$

where L is the canal length, v_0 the average flow velocity under the initial flow conditions, and c_0 average celerity under the initial flow conditions. $\tau_{\Delta V}$ in (3) is a delay based on the time needed to supply ΔV at a rate equal to the demand change:

$$\tau_{\Delta V} = \frac{\Delta V}{\Delta q_d} \tag{5}$$

In cases where the wave introduced by upstream flow changes travels with little attenuation, $\tau_{\Delta V}$ can also be interpreted as a kinematic shock travel time. With τ in (1) given by (5), $\Delta Q_2 = 0$.

Bautista and Clemmens (1988) computed τ using kinematic and dynamic wave theory. That approach requires estimates of the pool length affected by backwater for the given flow conditions. When applied to the Test Case, this approach proved inappropriate as it yielded discharge changes at the check structures greater than the canal capacity as a result of very small delay values. A simpler and more conservative approach was used here, by using (5) as the delay. As noted, this reduced the inflow schedule to a single change,

$$\Delta Q = \Delta q_d \tag{6}$$

and, more importantly, bounded the magnitude of the check-flow change.

If the canal has multiple pools and a single demand change occurs in pool *J*, then a schedule of inflow changes needs to be computed for all check structures upstream from pool *J*. The schedule of check *J* (pool *J*'s upstream check) is a function of pool *J* only. For pool *J-1*, the schedule is a function of the sum of volume changes and accumulated delays of pools *J-1* and J. For *j*-th check structure, the expression for the discharge change is (Bautista and Clemmens, 1998):

$$\Delta Q_j = \frac{\sum_{k=j}^{J} \Delta V_k}{\sum_{k=j}^{J} \Delta \tau_k} \tag{7}$$

This equation applies to the general case in which τ in (1) is obtained by any reasonable procedure. In such case, the timing for ΔQ_1 for structure j is given by:

$$t(\Delta Q_1) = t_d - \sum_{k=j}^{J} \Delta \tau_k \tag{8}$$

while the timing for the second check-flow change, $t(\Delta Q_2)$, is the demand change time, t_d. If the delays are given by (5), then application of (7) yields simply Δq_d (Eq. 6) while $\Delta Q_2 = 0$. For a canal subject to multiple demand changes, each change has to be processed separately. The resulting time sequence of ΔQ_js then defines the feedforward control schedule for check structure *j*.

Bautista and Clemmens (1998) applied this approach to situations with multiple demand changes by assuming that a pool's flow was equal to the time zero discharge plus all demand changes ordered prior to the time of the requested Δq_d. Only demand changes in the pool being processed or in pools downstream from it were included in this sum. That approach was modified to properly identify the initial conditions that need to be used to process each individual demand change, as discussed in the previous section. However, instead of using gate-stroking solutions, accumulated delays (the denominator of (7)) were used to determine the order in which individual demand changes needed to be routed.

The head-gate inflow hydrograph obtained with this method is shown in Figure 3 along with the hydrograph obtained via gate-stroking. It should be noted that the

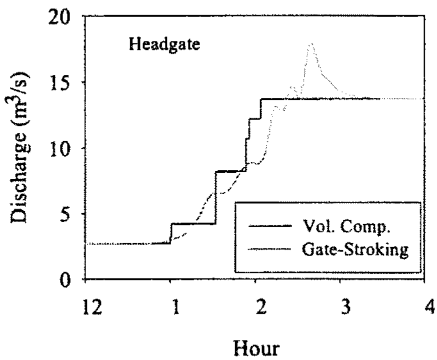

Figure 3. Volume-compensating and gate-stroking inflow schedules

final steady-state conditions of the test case are close to the canal's maximum discharge capacity (Clemmens et al., 1998) and, therefore, the gate-stroking solution exceeds temporarily that maximum value.

Water level control produced with the gate-stroking and volume-compensation feedforward control schedules are shown in Figures 4. These results were computed with the unsteady flow simulation model CanalCAD (Holly and Parrish, 1995). The simulator used the control schedules to determine check flow rate setpoints as a function of time and internally computed a gate position for the new flow setpoint. Flow through the gravity offtakes varied in response to water level fluctuations in the canal.

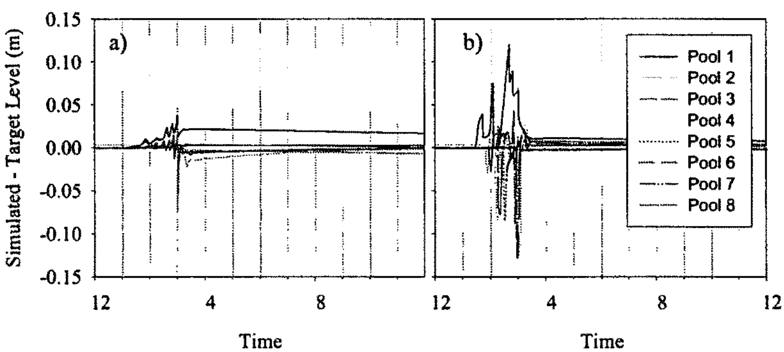

Figure 4. Difference between simulated and target water levels with a) gate-stroking and b) volume-compensating feedforward control schedules

Three things are evident from Figure 4. First, water-level deviations were much larger with the simple approach (Figure 4b) than with gate-stroking (Figure 4a). Second, despite these large deviations, near-steady-state conditions were achieved shortly after the time at which the offtake flow changes occur. Lastly, in both cases the deviations were small relative to the target levels (Table 1).

Table 2. Maximum Absolute Error (MAE) and Integrated Average Error (IAE) for test case, from simulation with gate-stroking and volume compensating solutions

	Pool 1	Pool 2	Pool 3	Pool 4	Pool 5	Pool 6	Pool 7	Pool 8
Gate-Stroking								
MAE	1.8%	0.8%	1.5%	0.4%	0.4%	0.7%	1.1%	4.5%
IAE	0.8%	0.2%	0.1%	0.1%	0.1%	0.1%	0.1%	0.3%
Volume-Compensation								
MAE	5.7%	4.0%	3.7%	4.4%	4.4%	5.2%	7.6%	7.2%
IAE	0.6%	0.1%	0.2%	0.1%	0.1%	0.3%	0.2%	0.2%

Two performance measures recommended by the ASCE Task Committee on Canal Control Algorithms (Clemmens et al, 1998) were computed for these tests. The Maximum Absolute Error (MAE) is a measure of the maximum water level deviation relative to the target. The Integrated Average Error is a measure of the average absolute error relative to the target. Results are summarized in Table 2. The MAE for the simple feedforward control is as much as ten times greater than with gate-stroking, however these errors are short lived and have little impact on the average performance. The average error for all pools with both feedforward control methods is less than 1% of the target level.

CONCLUSIONS

For the example presented, similar gate-stroking results were obtained by processing all demand changes simultaneously and by treating the problem as a linear combination of single-flow change problems. The analysis assumed a succession of steady states and, thus, differences in results were due to unsteady flow effects not accounted for in defining initial conditions for individual flow change problems. Results show that even under conditions where strong unsteady effects would persist for long times, reasonable results can be obtained by assuming that each demand change creates a new set of steady initial conditions for the next flow change to be routed. Such an approach also assures volume compensation. It has been previously shown that a simple feedforward control method based on volume compensation can produce reasonable water level control in single-pool canals subject to a single demand change. A strategy was developed to apply the volume compensation method to multiple-pool canals subject to multiple flow changes. The resulting water level control over the test period was, on the average, comparable to that obtained with gate-stroking. This suggests that the proposed volume compensation approach is both practical and effective.

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IMPROVING THE RELIABILITY OF CANAL DE PROVENCE HYDRAULIC MEASUREMENTS BY DATA RECONCILIATION

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ABSTRACT

Measurement network on hydraulic system includes many sensors subject to failure or deviation, and spread over a huge area. In addition discharge and volume measurements in open channel hydraulic networks are characterized by large uncertainties. To overcome this kind of problem, in process control industrial applications, data reconciliation is more and more used. The objective of the data reconciliation is to take advantage of information redundancy on a system to make a cross-checking of real-time measurements.

Using this information redundancy, a data reconciliation module allows to detect inconsistent measurements, measurement deviations and provides corrected values whether the initial measurements are valid, biased or invalid. A derived consequence is to better schedule the maintenance of sensors.

A data reconciliation module, based on the measurements from the hydraulic network, has been recently developed and implemented in the SCP's supervisory system. The software has initially been used on a daily basis to check the measured flow on the main canal. It has then been adapted in order to run every 15 minutes on a distribution network including pipes, canals, and tanks.

The paper presents first the theory of the Canal de Provence data reconciliation application. The basic model is an hydraulic network with a series of nodes corresponding to balance equations (inflows, outflows, and storage). Constrained data reconciliation is used in order to satisfy the non-negativity of the hydraulic variables and the mass balance relations. The results are corrected values for measured variables and proposed values for non-measured quantities. A statistical

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analysis of the results is performed. This analysis allows to evaluate the uncertainties attached to the estimated flows and volume values. It allows also to detect invalid measurements, drift of sensors and to decide which maintenance operations to perform. Secondly, field examples are presented : measured and re-estimated flow values with their standard deviations, detection of invalid sensors, performed maintenance operation. The data reconciliation is situated just after the measurement process and takes place in the decision process for diagnosis, identification and control.

INTRODUCTION

The Canal de Provence is situated in the South-East of France. It supplies water to 80,000 ha of farmland, 110 towns and villages and 400 industries. The water distribution strategy is user oriented, without resorting neither to rotations nor to any sort of priority allocation.

All the main structures are monitored and remote controlled from the General Center by a SCADA system including a module of "Dynamic Regulation" which provides automatic and permanent control of canal flows and safety systems.

A data reconciliation procedure can be implemented as long a redundancy among measurements exists. This redundancy can come from multi-measurements of some physical quantities in the system. It can also come from system equations linking the measured variables due to the existence of a model of the system. In our application, we are essentially concerned by this last case, which is closely connected with the concept of system observability.

Usually one distinguishes two main cases. The static case, where the model equations concern only measured quantities at the same instant. The dynamic case, where the system equations concern measured quantities on different instants. In the dynamic case, the model equations are most of the time those of the dynamic modeling of the system. In any case the relevance of the resulting re-estimated data will depend , of course, on the accuracy of the model.

The first implementation of data reconciliation on the hydraulic system of Canal de Provence, concerns daily volumes on the canal, see figure 1. Therefore, the model equations can be the static volume conservation equations at the nodes of the hydraulic network, since propagation time can be neglected.

More recently, it has been extended to a distribution network of pipes and canals, where it runs every 15 minutes. This second project required to take into account the storage in canal and tanks and the transit time of water along canals.

The next section is devoted to a presentation of the data reconciliation basic principles. The third section presents typical and illustrative field cases.

DATA RECONCILIATION PROCESS

The model equations can be presented as follow. First we number the daily measured discharges. Let us call v_i^* the true (unknown) value of the discharge on branch number i and v_i the value from sensor. Let us call V^* and V the respective vectors. The volume conservation relations at the nodes of the network lead to linear mass balance relations between these branch discharges:

$$MV^* = 0 \tag{1}$$

where M is the network matrix the elements of which have the values 1, -1 or 0. To each node of the network corresponds a row of M. The value of an element of a given row is 0 if the corresponding discharge v_i^* does not appear in the node relation, +1 if it is an entering discharge (inflow) and –1 if it is an outgoing one (outflow).

Of course this relation does not hold for the vector V. We rather get:

$$MV = E$$

generating then a vector of residues E.
At this stage some statistical hypotheses must be done. The v_i are considered as Gaussian, independent random variables, the standard deviations σ_i of which characterize the measurements precision. Therefore the covariance matrix W of the random vector V is

$$W = \mathrm{diag}\left(\sigma_i^2\right)$$

The data reconciliation problem is now : find the best estimate $\hat{V}$ of V^*, which minimizes the quantity

$$\frac{1}{2}\left(V - \hat{V}\right)^t W^{-1} \left(V - \hat{V}\right)$$

subject to the equality constraints:

$$M\hat{V} = 0$$

and to inequality constraints:

$$\hat{V} \geq 0$$

Numerical solution of this problem is obtained in the framework of the Lagrangien multipliers method (Ragot 1992).The inequality constraints are taken into account using an heuristic method (Gill 1981).

We call $D = V - \hat{V}$. Four statistical tests have been implemented in order to detect a value of D outside the confidence interval, a jump of mean value of D on a sliding window, a too large variance of D, and a value of $\hat{V}$ outside its confidence interval.

We check also for some points that value of $\hat{V}$ does not exceed physical known thresholds for these points.

In addition, in order to test the global consistency of measurements, a khi2 test is performed on the quantity

$$K = E^t M\ W\ M^t E$$

Ill-conditioned numerical problem can occur when measured values are missing in the database. In order to deal with this problem we have implemented an independent method of reconstruction of missing values based on a Principal Component Analysis (PCA) of a complete historical sample of measurements (Fenelon 1982, Canivet 2002)

FIELD EXAMPLES

The data reconciliation module runs actually on the main canal network on a daily basis and on a hourly basis on the Aix-Nord distribution network including pipes, canals and tanks. The next paragraphs will present these two cases, with a short description of the networks and the constraints.

The main canal application

The geographical location of Canal de Provence is shown in Figure 1.

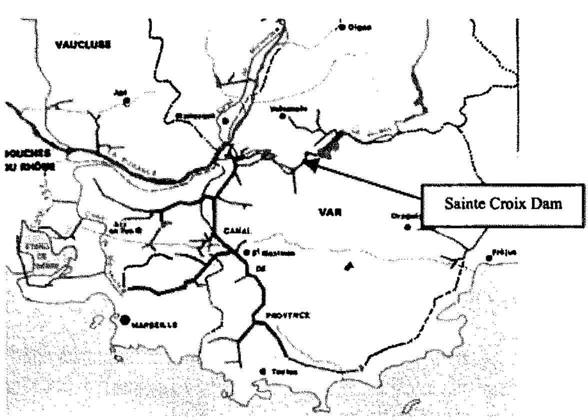


Figure 1: Canal de Provence, location map

The water is taken from Sainte-Croix dam at Boutre intake. The Canal de Provence supplies the Provence and Côte d'Azur region (PACA region). One can distinguish three main areas: the Aix-en-Provence, Marseille and the French Riviera areas.

Figure 2 shows the diagram of the network with the location of measurements. Different categories of sensors are used in order to measure the discharge. Ultra-sonic and electromagnetic flow meters are essentially implemented at the head of distribution networks. At cross structure locations gate formulas are used for the discharge calculation. In addition, calibrated weir, flumes (venturis), pumps tabulated formulas and an open-channel ultra-sonic sensor at the Boutre intake, are also present on the Canal.

The data reconciliation software is executed every morning by the control operator to check the overall discharge coherence of the previous day. This procedure is operational since 1999. Previously, discharge measurements were used for control purpose which needs only relative knowledge of discharge variations. This implementation represents the first step to the final absolute knowledge of the discharge values.

In the following we choose to present three typical examples: the abnormal Saint-Maximin and Pourcieux cross structures behavior and the Boutre discrepancy between two different flow measurements.

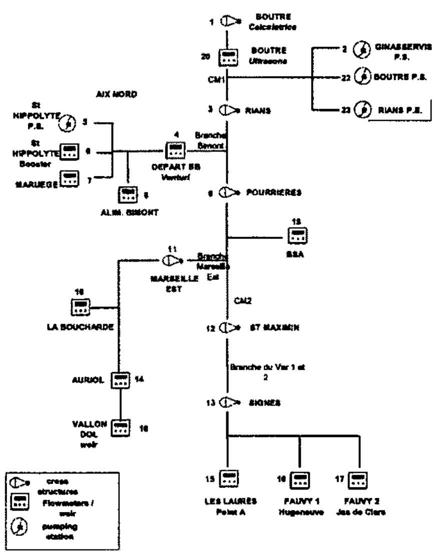


Figure 2: Diagram of network

The Saint-Maximin cross structure consists of two gates which are used alternatively on a monthly basis, for maintenance reasons. The discharge value is obtained from gate opening and upstream level since the structure works in a free flow condition. The software detects inconsistencies depending on the gate used. Figures 3(a) displays the differences between reconciled and measured discharge values, together with the gates status.

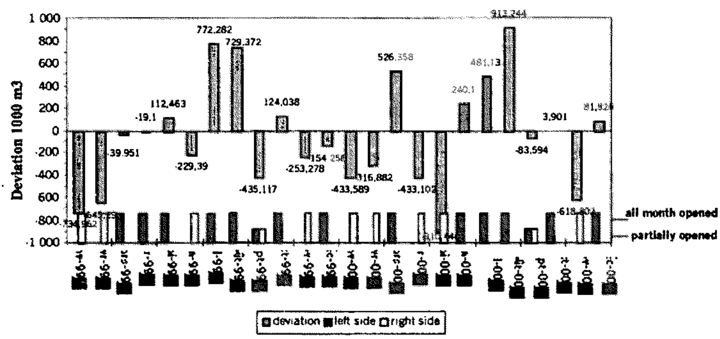


Figure 3(a):Deviation between reconciled and measured values at St Maximin cross structure, year 1999/2000

Inconsistencies correlated with the gate used appear very clearly. A field investigation, point out a slight error on position measurement of the right side gate. This error was small in comparison of the total gate opening. However, since the structure operates at low rate, it has a large effect on discharge calculation. Figure 3(b) shows the actual situation, after the correction mentioned above.

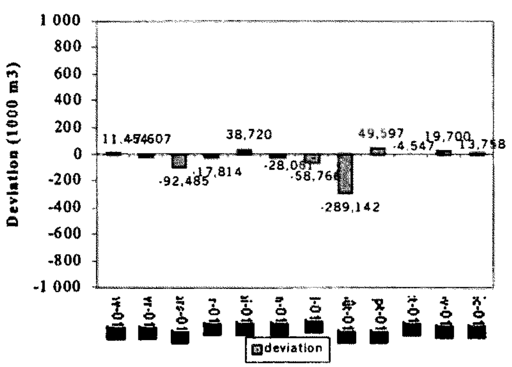


Figure 3(b):Deviation between reconciled and measured values at St Maximin cross structure after correction

At Pourcieux the structure was initially designed for free flow condition operation. The software detected that the discharge calculated at this point was to low in comparison to reconciled value. This was confirmed by a gauging based on flow velocity measurement. We then diagnosed that the canal downstream has an effect on the gate, although the canal slope was high. Now, the discharge calculation formula takes into account submerged condition.

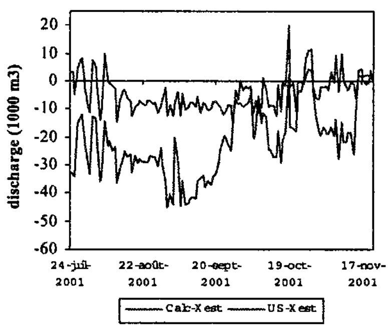
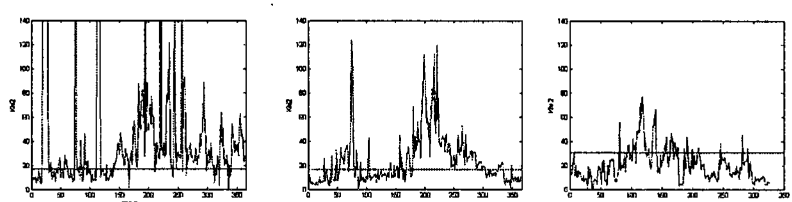


Figure 4:Boutre:deviation between measured and reconciled values

Boutre is the main intake of the canal. The discharge at this point was previously calculated from a gate formula established from a scale model thirty years ago. On one hand the accuracy of this system has been improved by replacing the gate opening measurement. On the other hand, the operating staff decided to add at this point an ultrasonic flow meter located 150 meters (492 ft) downstream.

The calibration of this sensor happened to be hard due to backwater effect. Unexpectedly, the software established that the renewed method gives values closest to the reconciled ones. Figure 4 shows the ultrasonic-reconciled and formula based-reconciled discharge deviations. A new calibration of the ultra-sonic sensor is now in progress.



(a) year 1999 (b) year 2000 (c) year 2001

figure 5: Evolution of the khi2 value

These previous examples illustrate the various improvements, which have been undertaken on the canal. As presented before, the global khi2 test is a valid indicator of consistency of measurements. Figures 5(a), 5(b) and 5(c) display the evolution of the khi2 values for the last three years, evidence of consistency enhancement is clearly seen in the consecutive graphs.

Aix-Nord application

Figure 6 shows the Aix-nord network. It consists of two sub-networks the intakes of which are respectively the Saint Hippolyte pumping station (SHPS network) and the Saint-Hippolyte booster (SHB network).

SHPS supplies two reservoirs which feed pipe networks for agricultural and domestic users.

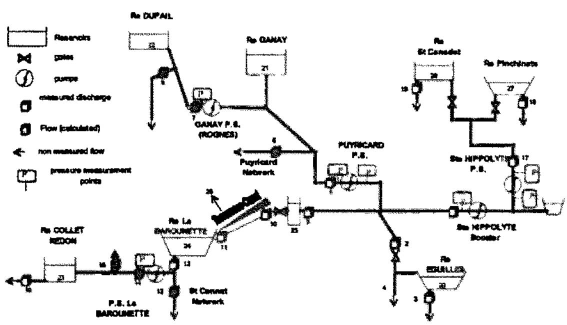


Figure 6:Aix-Nord network

The SHB net work is more complex. It includes pipe networks, pumping stations, reservoirs and the Trevaresse canal which behaves as a pure water transit delay.

In this case, the reconciliation software is automatically executed every fifteen minutes. This application requires to take into account the storage in canal and tanks, and the transit time of water along the canal. To avoid measurements noises, especially on level measurements, the reconciliation has been performed on an hourly basis (mean flow for the last hour, difference in stored volume for the last hour). The Aix-Nord application is running since July 2001. It automatically activates alarms on detection of abnormal situation.

In the following, three typical examples are described: an abnormal backward discharge when the Puyricard pumping station is stopped, the estimation of Saint Hippolyte booster discharge during a period of lack of information at that point, the detection of an abnormal increase of Eguille tank outflow.

Puyricard pumping station lifts the water into Ganay reservoir which supplies the Ganay pumping station. The Puyricard network is supplied with water at a position between Puyricard pumping station and Ganay reservoir. Discharges of the Puyricard and Ganay pumping station are measured by flow meters, a sensor measures the level in Ganay reservoir, whereas the Puyricard network discharge is not measured but deduced from other measurements. Figure 7 shows, when Puyricard pumping station is stopped, a deviation of 20 l/s between measured and reconciled discharge, before 9 January 2002. This was due to backflow measured by the flow meter and send to the supervisory system as pumped. The maintenance department corrected the flow meter configuration to eliminate backflow measurement and eliminated the source of error that caused the deviation mentioned above. The physical malfunction itself may come from leakage through exhaust valves, and the correction is now in progress.

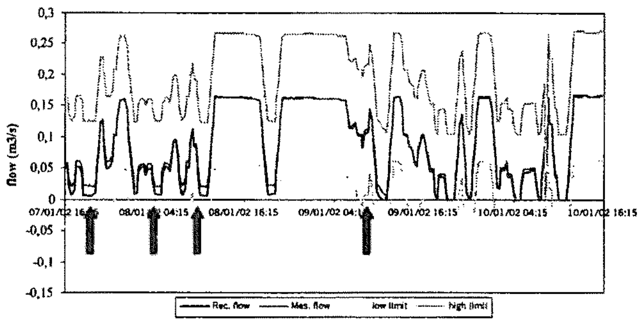


Figure 7:Deviation appearing in Puyricard site with no pump in operation

Because of renewing works, we haven't been able to measure the Saint Hippolyte booster discharge. As presented before the software has estimated this value using

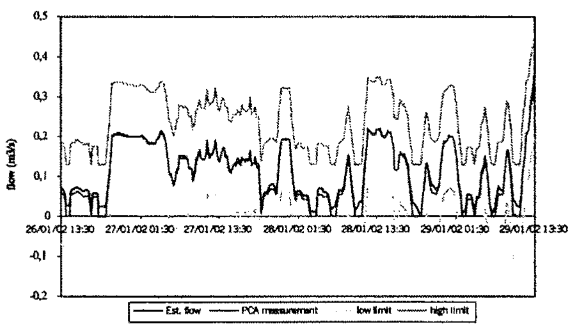


Figure 8: Reconciled values in absence of measurements

PCA method and reconciliation with the remaining measurements of the network. Figure 8 display the behavior of the reconciled values in absence of measurement, reconciled values are closed to initial calculated PCA values.

The software is able to detect abnormal variation in time of reconciled discharge. Particularly, the goal of this function is to alert the operator in case of a pipe breaking on the network. Figure 9 presents the trigger of an alarm on December 20th because of an abnormal increase of discharge flowing from Eguille tank Although on that day the discharge increase was linked to the filling of a pipe which had been emptied for maintenance reason, this confirms the effectiveness of that mode of detection.

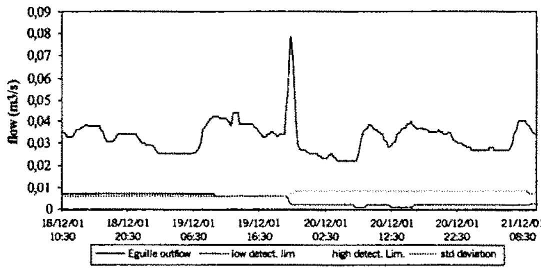


Figure 9:Abnormal time variation of Eguille flow

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PRELIMINARY RESULTS FOR DOWNSTREAM WATER-LEVEL FEEDBACK CONTROL OF BRANCHING CANAL NETWORKS

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ABSTRACT

Over the last 40 years researchers have made various efforts to develop automatic feedback controllers for irrigation canals. However, most of this work has concentrated on feedback controllers for single, in-line canals with no branches. In practice it would be desirable to automate an entire canal network and not just one of the branches. Because the branches in a network are hydraulically coupled with each other, a branching canal network cannot be controlled by designing separate controllers for each branch and then letting them run simultaneously. Changing the gate position in one pool on one branch can affect the water levels in pools on other branches. Because of this effect, the controllers designed for each of the in-line branches of the network will interfere with each other and potentially create instabilities in the branching canal network. Thus, the controller must be designed for the network as a whole and the branching flow dynamics must be explicitly taken into account during the controller design process. This paper presents preliminary simulation results on three different downstream feedback controllers on a branching canal network. The first controller is a series of Proportional-Integral (PI) controllers, one per pool. The second is a fully centralized PI controller. The third controller uses Model Predictive Control (MPC) to determine the appropriate control actions.

INTRODUCTION

The main purpose of an irrigation water delivery system is to deliver water to users at the desired time, rate, frequency, and duration. Most operators of irrigation water delivery systems operate the canals using manual techniques. Routing known flow changes and accounting for unknown flow disturbances and flow measurement errors using manual control is a difficult and time-consuming process. Thus, some canal operators have turned to automatic control techniques in an attempt to more efficiently control irrigation water delivery systems.

Over the past 40 years, researchers have proposed a wide variety of algorithms for automatic control of water levels in irrigation canals (Malaterre *et al.* 1998). These control algorithms include classic proportional-integral (PI) controllers, heuristic controllers, predictive controllers, and optimal controllers. Despite the

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large number of proposed algorithms, automatic control of irrigation water delivery systems has been successful under only a limited range of operating conditions (Rogers and Goussard 1998 and Pongput and Merkley 1997).

One drawback to the existing attempts to automatically control irrigation water delivery systems is that these control algorithms are only applicable to single in-line canal systems. If they are applied to branching canal networks, then the controllers will interfere with each other and may adversely impact the overall system. Thus, it is desirable to develop an automatic control system that can be applied to branching canal networks.

This paper presents preliminary simulation results on three different downstream feedback algorithms on a branching canal network. The first algorithm uses Proportional-Integral (PI) control, with each check controlled by errors in the water level at the downstream end of the downstream pool. The second algorithm is the fully centralized PI controller, which is a PI controller with complete hydraulic decoupling that explicitly takes the delay times of the pools into account (see Clemmens and Schuurmans 1999 for more details on this controller). With this controller, the flow rate at any check structure can be influenced by water level errors in any pool and by prior control actions at any check structure. These first two controllers use optimization techniques off-line to determine the controller constants. The third algorithm uses Model Predictive Control (MPC) to determine the appropriate control actions. For this controller, optimization techniques are performed on-line at each control time step. All of these controllers use a lumped-parameter linear approximation of the Saint Venant equations as their underlying linear process model.

BACKGROUND

Linear Process Model

The core of any automatic control system is the underlying process model that is used to model flow in open channels. Open-channel flow is described by the Saint Venant equations, which are a set of hyperbolic, nonlinear, partial differential equations that are distributed in time and space. However, nonlinear feedback control is not as easy to define as linear feedback control. Thus, the control problem is greatly simplified if the process model is linearized and linear feedback control is utilized.

Schuurmans *et al.* (1995) developed the integrator-delay (ID) model to describe flow in open channels. The ID model is a lumped-parameter linear response model that can handle backwater effects as well as normal flow conditions. For one pool, the ID model can be expressed as:

$$\frac{de}{dt} = -\frac{q_{out}(t)}{A_s} \qquad \text{for } t \le \tau \tag{1}$$
$$\frac{de}{dt} = \frac{1}{A_s}\left(q_{in}(t-\tau) - q_{out}(t)\right) \qquad \text{for } t > \tau$$

where e is the deviation of the downstream water level from its desired steady-state level, q_{in} is the deviation of the upstream inflow to the pool from its steady-state value, q_{out} is the deviation of the downstream outflow from the pool from its steady-state value, A_s is the backwater surface area of the pool, τ is the delay time of the pool, and t is time. The ID model depends on only two hydraulic parameters per pool: the delay time, τ, and the backwater surface area, A_s. Once these two properties are determined, the hydraulic characteristics for the pool are completely defined. See Clemmens *et al.* (1997) for details on determining A_s and τ.

State-Space Representation

There are many benefits to using the ID model as the underlying linear process model. First, it is a lumped-parameter model, so it is mathematically easier to handle compared to a distributed model. Second, the ID model can be used to define the discrete state-transition equations commonly used in linear system theory without the need for state estimation techniques:

$$\mathbf{x}(k+1) = \ddot{\mathbf{o}}\mathbf{x}(k) + \tilde{\mathbf{A}}\mathbf{u}(k) \tag{2}$$

$$\mathbf{e}(k) = \mathbf{C}\mathbf{x}(k) \tag{3}$$

where $\mathbf{u}(k)$ is a vector of changes in control actions at time k, $\mathbf{x}(k)$ is a vector of changes in the state of the system at time k, $\mathbf{e}(k)$ is a vector of water level errors at time k, and $\mathbf{\Phi}$, $\mathbf{\Gamma}$, and $\mathbf{C}$ are the state-transition matrices, which are defined by discretizing the ID equations. A convenient way to define the state vector is to have it consist of changes in water level errors, previous incremental control actions, and previous water level errors (Clemmens and Schuurmans 1999).

Controller Objectives

Both the optimal PI controllers and MPC determine the appropriate control actions by minimizing an objective function, J:

$$J = \sum_{k=0}^{\infty} \mathbf{e}(k)^T \mathbf{Q}\mathbf{e}(k) + \mathbf{u}(k)^T \mathbf{R}\mathbf{u}(k) \tag{4}$$

where **Q** is the penalty function for water level errors and **R** is the penalty function for changes in control actions. Although these controllers are tuned using optimization techniques, the user still needs to determine the appropriate values for the penalty function matrices **Q** and **R**. Typically, this is done using trial-and-error techniques. To simplify this process, **Q** is set equal to **I**, and $\mathbf{R} = \rho$ **P** where ρ is the penalty weight for the controller and **P** is the identity matrix weighted by the relative capacity of each pool in the canal (see Clemmens *et al.* 1997 for more details).

Optimal Proportional-Integral Controllers

Using linear system theory, the typical control law for PI controllers in state-space form can be expressed as:

$$\mathbf{u}(k) = -\mathbf{K}\mathbf{x}(k) \tag{5}$$

where **K** is the controller gain matrix determined by minimizing the objective function, J. The solution for **K** is subject to the dynamic characteristics of the physical system, as described by the state-transition equations (2) and (3). This optimization procedure is performed once and the same gain matrix, **K**, is used throughout the simulation. The form of **K** defines the type of PI controller that is used. For example, if all of the elements of **K** are nonzero, then this represents the fully centralized PI controller. If only the elements of **K** that correspond to the proportional and integral constants of the controller are nonzero and the rest of the elements are zero, then this form of **K** represents a series of simple PI controllers, one per pool (see Clemmens *et al.* 1997 for more details). For these optimal PI controllers, the value of ρ needs to be determined through trial-and-error techniques.

Model Predictive Control

MPC originated in the late 1970s and has been used extensively in the process control industry. MPC has three basic components, summarized as follows (Camacho and Bordons 1999):

1. A linear process model, the ID model in this case, is used to predict the system output for some time into the future, □$(k+i|k)$. The time that is predicted into the future is called the prediction horizon, p. The output predictions have two components: a free response and a forced response (Clarke 1994). The free response is the expected behavior of the system

assuming no future control actions. The forced response is the additional component of the process output that is due to the unknown future changes in control actions, $\mathbf{u}(k+i|k)$. The forced response is considered over a time horizon called the control horizon, m, while the free response is considered over the entire prediction horizon, p. The control horizon is less than or equal to the prediction horizon. While in the control horizon, the process model is used to obtain output predictions based on both the free and forced responses. After the control horizon has passed, the remaining output predictions are based on only the free response of the system. This prediction strategy is shown in **Figure 1**.

2. An objective function, similar to equation (4), is minimized by adjusting the future control actions, $\mathbf{u}(k+i|k)$. This optimization problem is subject to the many constraints that may be imposed on the system.
3. Once the sequence of future control actions that minimizes the desired objective function is determined, only the first set of control actions is implemented on the system, $\mathbf{u}(k+1|k)$. The system is then updated and the process is repeated. This is known as the receding horizon strategy.

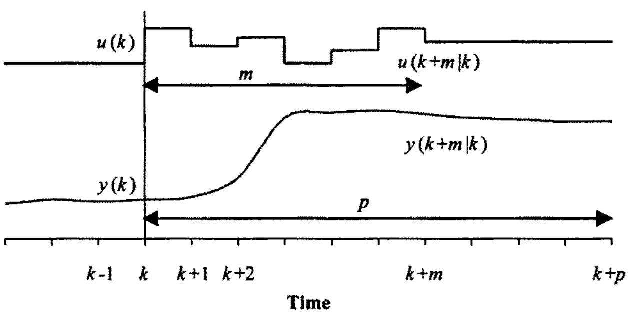

Figure 1. MPC prediction strategy (from Camacho and Bordons 1999)

MPC differs from the optimal PI controllers in that there is no explicit control law such as equation (5), and the optimization problem is solved on-line at each time step during the simulation. Implementing MPC on-line may present some difficulties because the optimization problem may be very complex and require an extensive amount of computing power to solve. Also, feasibility is an issue with MPC. If the constraints imposed on the optimization problem are too restrictive, then the problem may become infeasible and the controller will not function. Tuning the MPC controller consists of determining the appropriate values for the control horizon, the prediction horizon, and the penalty weight for the controller (ρ).

CONTROL OF BRANCHING CANAL NETWORKS

The first step in developing a controller for a branching canal network is to make sure that the state-transition equations capture the branching canal hydraulics by modifying the underlying ID equations. In the pool where the branch occurs, equation (**1**) must be modified to include the outflow into both branches of the canal system. This methodology is best explained by looking at an example of a branching canal system. Consider ASCE test canal 1 (see Clemmens *et al.* 1998 for more details on the ASCE test canals) and assume that there is a branch that occurs at the downstream end of pool 4. One of the branches contains pools 5 and 6 from the ASCE test canal 1 while the other branch contains pools 7 and 8, as shown in **Figure 2.**

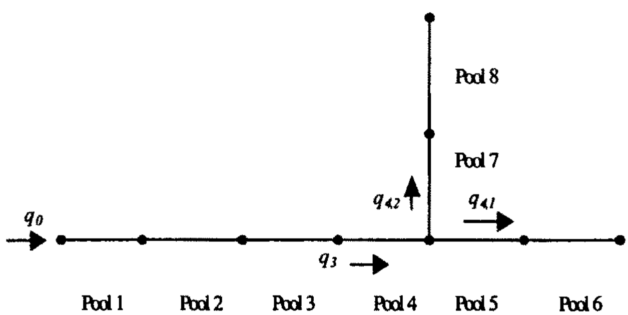

Figure 2. Schematic diagram of branching canal network

Without the branch, the ID equation for pool 4 can be expressed as:

$$\frac{de_4}{dt} = \frac{1}{A_{s4}}\left(q_3(t-\tau_4) - q_4(t)\right) \tag{6}$$

When a branch is present at the end of pool 4, the underlying ID equation is modified to become:

$$\frac{de_4}{dt} = \frac{1}{A_{s4}}\left(q_3(t-\tau_4) - q_{4,1}(t) - q_{4,2}(t)\right) \tag{7}$$

The ID equations for pools 5 and 7 can be developed with q_{in} equal to $q_{4,1}$ and $q_{4,2}$, respectively. The ID equations for the remaining five pools retain the form of equation (**6**). The eight ID equations are then discretized and placed in the state-space form of equations (**2**) and (**3**) (see Clemmens and Schuurmans 1999 for more details on how this is done). Once the proper modifications have been made to the underlying ID equations, optimal control theory or MPC can be applied to the system. For the optimal PI controllers, an additional adjustment needs to be made. Elements in the gain matrix, **K**, which are not feasible for the branching canal network, need to be set to zero. For example, for a fully centralized PI controller, a portion of the control actions at gate 7 would be passed to gate 5. For an in-line system, this is appropriate. However, for the system shown in **Figure 2**, gate 7 is hydraulically isolated from gate 5 and it would not be appropriate to pass a portion of the control actions at gate 7 to gate 5. Thus, these infeasibilities need to be identified in the branching canal network and the corresponding elements in the gain matrix need to be set to zero.

Control Tests

The authors performed simulations on the branched version of test canal 1 using the same initial conditions specified for test case 1 (see Clemmens *et al.* 1998 for details on ASCE test case 1). To test the effectiveness of the controller, an offtake change occurred in each section of the branching canal network (*i.e.*, upstream from the branch and in each of the two branches). Six hours into the simulation, the offtake flows at pools 3 and 8 increased from 0.1 m^3/s to 0.2 m^3/s, while the offtake at pool 5 was shut off. Simulations were carried out using the simple PI controller, the fully centralized PI controller, and MPC. All simulations were performed using the hydrodynamic model SOBEK (Delft Hydraulics 2000). SOBEK has the ability to simulate branching canal networks and to be linked to MATLAB (MathWorks 2000). All of the control routines were written as MATLAB m-files that interfaced with SOBEK. Because the goal of this paper was to determine the feasibility of these types of feedback controllers on branching canal networks, several simplifications were made to the ASCE test case 1. The simulations were performed only under tuned conditions, the minimum gate movement constraints were not enforced, and all of the flow changes were considered unscheduled (*i.e.*, no feedfoward routine was implemented).

Two constraints were imposed on the simulations: 1) the gates were not allowed to be completely closed and 2) the gates were not allowed to be taken out of the water. For the optimal PI controllers, these constraints were imposed after the control calculations were performed. In other words, the control law was used to determine a set of changes in control action variables. If these changes caused the constraints to be violated, then the control actions were adjusted until the constraints were satisfied. For MPC, the constraints were written explicitly into the constrained optimization problem.

RESULTS

Optimal Proportional-Integral Controllers

From past experience, the authors found that setting $\rho = 20$ works well for the steep test canal 1. Because no control actions are passed to other gates for the simple PI controller, no further modifications need to be made to the gain matrix and constrained optimization techniques can be used to determine the coefficients of **K**. About 10 hours after the disturbances, the PI controller returned the water levels to their setpoints and had a maximum deviation from the setpoint of about 0.2 m (**Figure 3**). Overall, these results are similar to other simulation results obtained using simple PI controllers on the unscheduled flow changes for test case 1-1 on the in-line test canal 1. For example, both Clemmens and Wahlin (1999) and Wahlin and Clemmens (1999) present simulation results that have similar maximum water level deviations for the unscheduled portion of the test case 1-1 as reported here for the branching canal network. However, the PI controllers for the in-line test canal 1 were not as sluggish as the PI controller for the branched system, and the water levels were returned to their setpoints in about six hours. In practice, these water level deviations and settling times may not be acceptable. Prescheduled delivery changes with a feedforward routine would improve the overall performance of this controller.

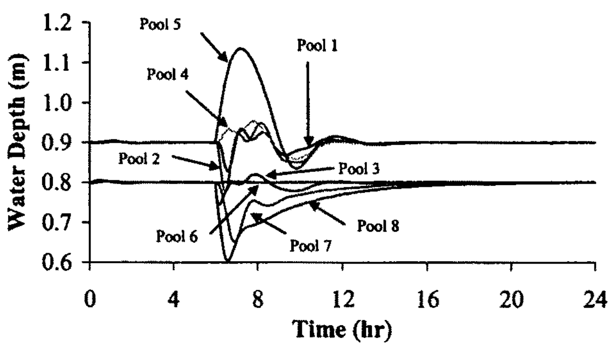

Figure 3. Simulation results using the simple PI controller ($\rho = 20$)

For the fully centralized PI controller, the infeasibilities that occur due to the branching canal network dynamics were identified and the corresponding elements in the gain matrix were set to zero. The gain matrix was then determined using constrained optimization techniques. There is a marked improvement in using the fully centralized PI controller (**Figure 4**) over the simple PI controller (*e.g.*, less overshoot, less oscillations, faster settling time, *etc.*). Within six hours of the disturbances, the fully centralized PI controller restored the water levels to their setpoints. These results agree fairly well with the unscheduled simulation results for the fully centralized PI controller on the in-line test canal 1 (Clemmens and Wahlin 1999). Again, overall controller performance would be improved with the addition of a feedforward routine.

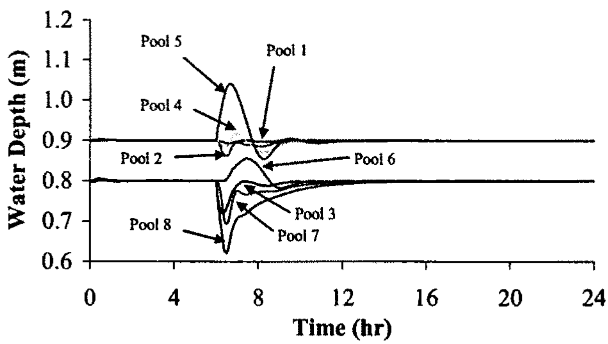

Figure 4. Simulation results using the fully centralized PI controller ($\rho = 20$)

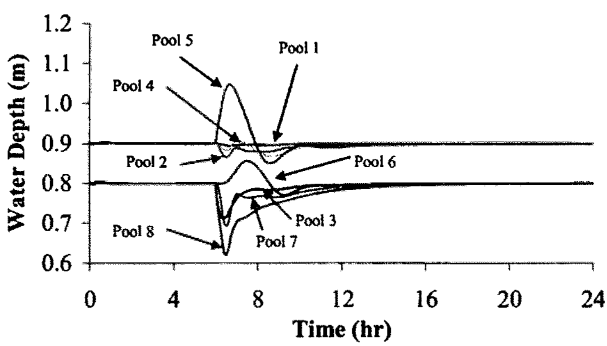

Figure 5. Simulation results using Model Predictive Control ($\rho = 10$)

Model Predictive Control

For the MPC simulations, the control horizon, m, was set to 20 time steps while the prediction horizon, p, was set to 40 time steps. Unlike the optimal PI controllers, better results were obtained with $\rho = 10$ instead of 20. The simulation results for MPC (**Figure 5**) are similar to those for the fully centralized PI controller, and, within six hours of the disturbances, the MPC controller restored the water levels to their desired setpoints. One of the benefits of MPC is that it has a feedforward routine built into the algorithm. Utilizing this option, the overall performance of the controller would improve.

CONCLUSIONS

Several conclusions can be drawn from these simulation studies:

1. Automatic controllers can be developed for branching canal networks by considering the branching dynamics in the underlying linear process model.
2. The fully centralized PI controller and the MPC controller adequately controlled this simple branching canal network under the given flow conditions.
3. The simple PI controller worked on the branching canal network; however, its performance was not as good as the fully centralized PI controller or MPC.
4. The controller performance was almost identical for the fully centralized PI controller and MPC.

The performance of these controllers on this simple branching canal network is encouraging. Additional simulation studies need to be performed to better define the robustness of these controllers.

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NEW CALIBRATION METHOD FOR SUBMERGED RADIAL GATES

A.J. Clemmens[1]

ABSTRACT

Calibration equations for free-flowing radial gates typically provide sufficient accuracy for irrigation district operations. However, many districts have difficulty in determining accurate discharges when the downstream water level begins to submerge the gate. Based on laboratory studies, we have developed a new calibration method for free-flowing and submerged radial gates that allows for multiple gates and widely varying upstream and downstream channel conditions. The method uses the energy equation on the upstream side of the structure and the momentum equation on the downstream side. An iterative solution is required to solve these two equations, but this allows calibration from free flow to submerged flow right through the transition. Adjustments to the energy equation for free flow are described, along with an additional energy adjustment for the transition to submerged flow. An application is used to describe the new procedure and how it overcomes the limitations of current energy-based methods.

INTRODUCTION

Radial gates are a common water control structure in much of the western United States. Their advantage over vertical sluice gates is that the lifting force is minimal. The U.S. Bureau of Reclamation has used these as a standard structure for nearly a century. They are also used in private irrigation projects, and projects of the U.S. Army Corps of Engineers. These structures are pervasive in canals and regulated streams in the central and western United States.

Calibration methods for free-flowing radial gates are available in standard references and have been used with reasonable success to measure flow. Calibration of submerged flows, however, has had mixed success, with errors up to 50% reported in some cases. These calibration methods are based exclusively on the energy equation. Some use the momentum equation to define the limit between submerged and free flow. However, a major flaw with all these methods is that they are all based on upstream and downstream channels that are the same width and have the same floor elevation as the gate. This rarely occurs in practice. Where multiple gates occur, submerged calibration has proven successful only when all gates are open the same amount and their total width is similar to the width of the downstream channel (e.g., the head of the All American Canal).

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In 1999, we conducted a study on the calibration of radial gates in the Hydraulics Lab of the U.S. Water Conservation Laboratory. Details of the experimental setup are provided in Tel (2000). In this paper, we present a solution method for submerged radial gates that uses the energy equation on the upstream side of the gate (the same as for free flow) and the momentum equation on the downstream side. A new transition between submerged and free flow is defined as an adjustment to the energy equation.

FREE FLOW

The calibration of flow under a vertical sluice gate is a classic problem in hydraulic engineering and has been studied for more than a century. Montes (1997) provides an excellent summary of the theoretical and experimental studies that have been conducted under free-flow conditions. However, our theoretical understanding of even free-flowing sluice gates is incomplete. For practical application, the errors associated with the theoretical disagreements are relatively small, being at most ± 5%, and within field calibration needs. Field calibrations are often needed anyway because of J-seals, etc. The foundation for the research on submerged flow is a reliance on the free-flow theory, thus it is worth discussion.

The complexity of the problem stems from our inability to theoretically determine the free surface configuration downstream from the gate, even in free flow. The jet emanating from under the gate reaches a minimum depth at the vena contracta (Section 2 in Figure 1). The theoretical difficulties are associated with defining the contraction coefficient δ (ratio of minimum depth y_j to gate opening w) for the variety of flow configurations encountered. In general, the contraction coefficient varies with the angle of the gate θ and the ratio of gate opening to upstream energy head, w/H_1.

If we apply the energy equation between Sections 1 and 2, we get

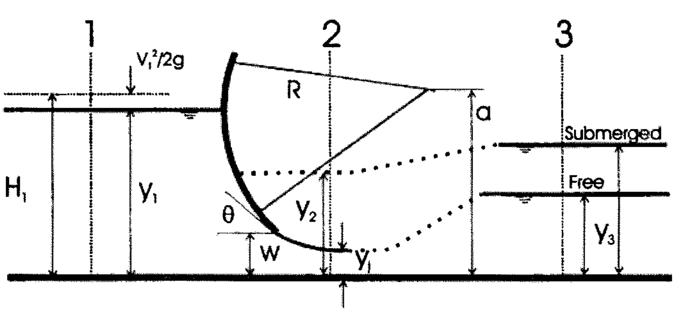

Figure 1. Definition sketch for radial gate.

$$H_1 = H_j + \Delta H = y_j + \alpha_j \frac{v_j^{\,2}}{2g} + \xi \frac{v_j^{\,2}}{2g} \tag{1}$$

where H_1 is the total head at section 1, H_j is the energy head at the vena contracta, • H is the head loss between Section 1 and the vena contracta, y_j is the depth at the vena contracta or jet, v_j is the average velocity in the jet, α_j is the velocity distribution coefficient for the jet (i.e., correction to the velocity head due to nonuniform velocity), g is the acceleration of gravity, and ξ is the energy loss coefficient. For simplicity, we will assume $\alpha_j = 1$. Any deviation from unity by α_j will end up in ξ, giving a combined coefficient.

Since the discharge is equal to velocity times area, in the jet we have $Q = \delta w b_c v_j$; where $y_j = \delta w$, and b_c is the gate width. Substituting for v_j and y_j in Equation 1 and solving for discharge gives:

$$Q = \delta w b_c \sqrt{\frac{2g(H_1 - \delta w)}{1+\xi}} \tag{2}$$

For a given geometry, this provides a relationship between discharge and upstream energy head, with only the contraction coefficient, •,•and the loss coefficient, ξ, to be evaluated.

This differs substantially from prior solutions of the radial-gate energy equation in several ways. First, it is expressed in terms of upstream energy head rather than depth. This allows one to have an upstream velocity head that is not related to the flow in any one gate, for example when multiple gates and weirs are used. Second, it includes as energy loss term rather than relying on empirical discharge coefficients. And finally, the contraction coefficient is not buried in the discharge coefficient. These differences allow us to determine the head-discharge relationships for a wider range of conditions than other methods.

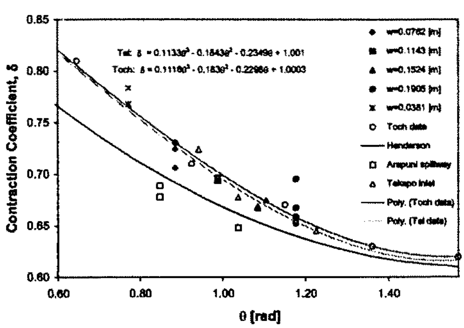

Figure 2. Radial gate contraction coefficient, •,• as a function of gate angle, θ.

In our study, the contraction coefficients were found by measuring the gate opening and measuring the pressure in the jet at the vena contracta with a Prandtl tube. For several runs, we measured the pressure distribution within the jet, and it was essentially hydrostatic, verifying that our single pressure readings were sufficient to define the contraction coefficient. Our resulting contraction coefficients (Fig. 2) were in excellent agreement with those found by Toch (1955) in laboratory studies. The contraction coefficient is strongly influenced by gate angle. We found almost no influence of w/H_1 on the contraction coefficient, in keeping with the results for planar sluice gates (Montes 1997).

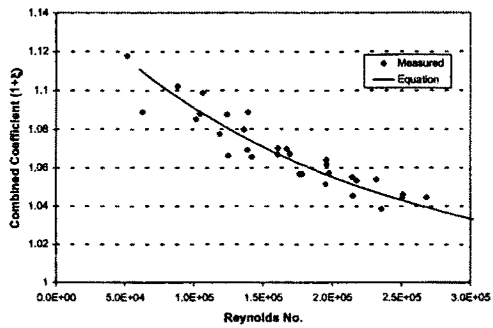

Figure 3. Combined velocity distribution and energy coefficient as a function of entrance Reynolds number for free flowing radial gate.

We chose to relate the measured energy loss to the velocity head in the jet, since the jet velocity head seems to be the most representative of the overall energy head that is causing/influencing the loss (Fig. 3). We found a strong relationship between the energy loss coefficient and the Reynolds number at the upstream face of the gate. Of significance to this research is that these energy losses are relatively high for laboratory models where Reynolds numbers are low. For prototype gates, Reynolds numbers may be an order of magnitude higher, suggesting energy losses on the order of 1% or 2%.

Solution of Equation 2 with the coefficients from Figures 2 and 3 agreed with laboratory data to within about 1% in all but a few cases (Fig. 4).

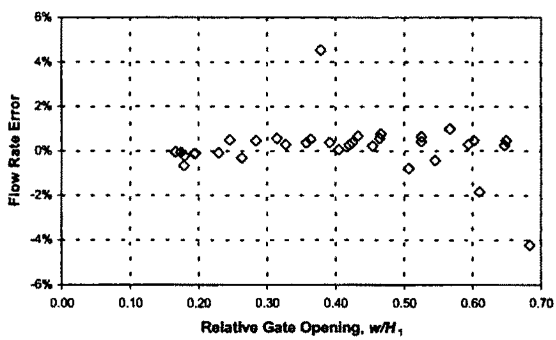

Figure 4. Accuracy of radial gate free-flow discharge computed with energy equation and curve fits for contraction coefficient and energy loss coefficient.

SUBMERGED CONDITIONS

Only a few studies on submerged radial gates are available in the literature. The most common approach has been to use an empirical discharge coefficient according to the amount of submergence. This approach was suggested by Henry (1950) and Rajaratnam and Subramanya (1967) for vertical sluice gates and used by Buyalski (1983) for radial gates. One difficulty with this approach is that the curves are very steep, resulting in a large change in discharge coefficient for a small change in upstream depth or gate opening. Another approach (Bos 1989) is to use the same discharge coefficient as for free flow, but with the water level difference across the gate replacing the upstream depth. A challenge with this approach is to determine when to use the upstream head and when to use the head differential. The standard textbook approach is to use the conjugate depth equation for a rectangular channel to determine whether or not the gate is submerged (e.g., as suggested by Bos (1989)).

These approaches have a major flaw when applied to practical situations. All of the studies and the conjugate depth relationship assume that the downstream channel is of the same cross section as the gate. The calibration results are highly dependent upon this condition, even though it is rarely found in practice. The current approaches cannot easily deal with these real-world conditions.

Where a hydraulic jump occurs, energy losses are difficult, if not impossible, to predict with an energy-based equation. This usually requires solution of the momentum equation. However, it is also not practical to solve the momentum equation from the upstream section to the downstream section since the forces on the gate are unknown. Instead, we propose to use the energy equation from the upstream side to the vena contracta, where we think we can capture the essential flow conditions, and the momentum equation from the vena contracta to the downstream section. Under normal operation, the depth and velocity at the vena contracta will not be measured. Instead, those conditions must be inferred from the equations.

Starting with the energy equation on the upstream side of the gate, we postulate

$$H_1 = y_1 + \frac{v_1^2}{2g} = y_2 + \frac{v_j^2}{2g} + \xi \frac{v_j^2}{2g} \tag{3}$$

In Equation 3, the subscript j refers to the "live stream" conditions in the jet at the vena contracta and the subscript 2 refers to the vena contracta location, whether free or submerged. Equation 3 implies a nearly linear relationship between y_1 and y_2 for any constant discharge and gate opening. However, laboratory results differ substantially from this result, as shown in Figure 5. At high submergence, the relationship looks reasonable, suggesting that the jet is the same size as under free flow, even though submerged.

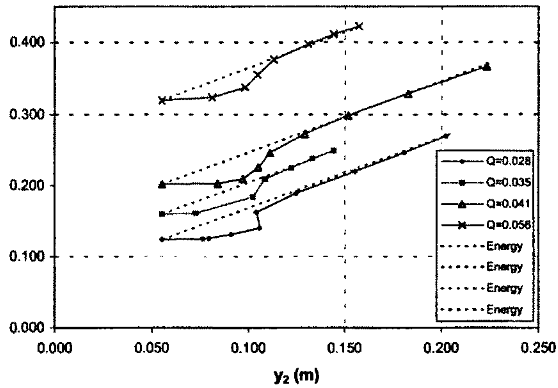

Figure 5. Preliminary application of energy equation from upstream side of radial gate structure to vena contracta.

Such results were also found by Rajaratnam and Subramanya (1967a), among others. At the beginning of submergence, the flow just downstream from the gate, as y_2 is increased holding all else constant, comprises an incomplete jump gradually approaching the classical wall jet (Rouse et al. 1959), and finally becoming the standard wall jet with the jet similar in configuration to the jet under free flow. While the partial jump is present, increases in tailwater elevation have almost no effect on upstream water level. But as the "wall-jet" condition is approached, further increases in tailwater depth are reflected in upstream depth changes.

Numerical modeling of this behavior between Sections 1 and 2 with the energy equation requires a reduction in jet velocity, and for a constant discharge, an expansion in the jet thickness (as suggested by Tel (2000)). In a simpler, and ultimately equivalent procedure, we postulate a kinetic energy correction term, E_{Corr}, for the transition zone, such that

$$H_1 = y_2 + \frac{v_j^2}{2g} + \xi\frac{v_j^2}{2g} - E_{Corr} \tag{4}$$

with v_j held fast at the free-flow value. E_{Corr} is evidently zero under free flow, as well as under fully submerged flow. In our preliminary analysis, we determined values for E_{Corr} as a function of relative submergence (shown below). Solving Equation 4 for discharge yields

$$Q = \delta w b_c \sqrt{\frac{2g\left(H_1 - y_2 + E_{Corr}\right)}{1+\xi}} \tag{5}$$

Solution of Equation 5 for submerged discharge requires, in addition to what is needed for free flow in Equation 2, an estimate of the energy correction, E_{Corr}, and an estimate for the depth, y_2, at the vena contracta. This depth is extremely difficult to measure in the field. The flow there is highly turbulent, rolling and "*frothy*" such that a surface measurement is insufficient to determine the true depth (i.e., as reflected in the pressure below the surface). This depth is not currently measured in the field and likely will never be. Instead, the water depth in the downstream channel, y_3, is measured. To utilize the measured depth, y_3, instead of y_2, a momentum relationship between Sections 2 and 3 is introduced.

Conservation of momentum, applied from the section with the vena contracta to Section 3, can be written as:

$$Qv_e + b_c g \frac{y_2^{\ 2}}{2} + \frac{F_w}{\rho} = Qv_3 + \frac{F_3}{\rho} \tag{6}$$

where v_e is the effective velocity in the jet (discussed below), v_3 is the downstream velocity, ρ is the density of water (mass per unit volume), F_3 is the hydrostatic-pressure force exerted by the downstream water depth, and F_w is the component of the force of water on all surfaces between Sections 2 and 3 in the direction of flow, including hydrostatic forces on all walls. This surface can be determined by taking the downstream area and projecting it back to Section 2 (assuming the section only expands from Section 2 to Section 3). Projected surfaces include the edges of the piers that separate the individual gates, closed gates, weir overfall sections, and the canal walls where the cross section expands. For rectangular cross sections, the force terms reduce to $bgy^2/2$, with subscripts 3 or w on b and y. For the short distances involved here, we can ignore the channel friction and bed slope effects.

Equations 5 and 6 represent solutions for flow on the upstream and downstream sides of the vena contracta, with Q and y_2 unknown, and the rest derivable from the measured water depths, gate opening, cross-section shapes, and empirical •,• ξ, and E_{Corr}. Application of these two equations is complicated by 1) quantification of the energy loss coefficient, 2) application of the energy equation under slightly submerged conditions, and 3) estimation of the wall forces for application of the momentum equation. The force on the walls is assumed to be based on a water depth there -- hypothesized to be between the depths at Sections 2 and 3. The effective water depth at the walls is found as the weighted average of these two depths, with weighting coefficient p:

$$y_w = py_3 + (1-p)y_2 \tag{7}$$

Laboratory experiments were performed to test the applicability of these equations. We solved Equation 6 for E_{Corr} with the measured values of Q, y_1, and y_2, and with ξ from the free-flow tests. The resulting energy correction relative to the change in depth at the vena contracta $[E_{Corr}/(y_2-y_j)]$ is shown in Figure 6 as a function of this change in depth relative to the free-flow jet thickness $[(y_2-y_j)/y_j]$.

The consistency of the relationship shown in Figure 6 for different discharges (and w/H_1 values) is encouraging! It essentially says that for small submergence depths, the contribution of that depth increase to the apparent increase in energy is small (i.e., almost 100% of the depth increase is canceled with E_{Corr}).

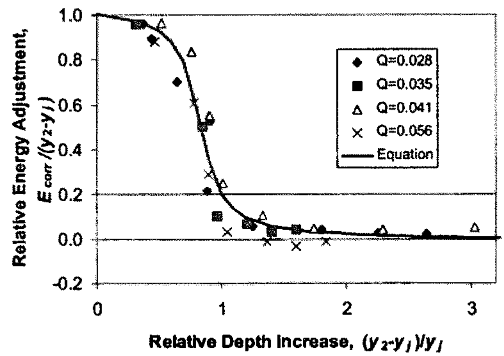

Figure 6. Relative energy adjustment required to apply free-flow energy equation to submerged flow for a radial gate up to the vena contracta.

A curve was fit to the relationship in Figure 6 to compute E_{Corr} for use in Equation 5. An equivalent jet velocity was determined by replacing the second and fourth terms on the right hand side of Equation 4:

$$\frac{v_e^2}{2g} = \frac{v_j^2}{2g} - E_{Corr} \tag{8}$$

The equivalent velocity then replaces the jet velocity in the momentum equation. This formulation leaves the computed upstream energy loss unchanged, except with Reynolds number (which changes with discharge and upstream depth as the gate becomes submerged).

The measured data were used to determine the coefficient p for the effective wall pressure. While there was some scatter in the data, a strong trend was not apparent. For the remaining analysis, an average value of 0.643 was used.

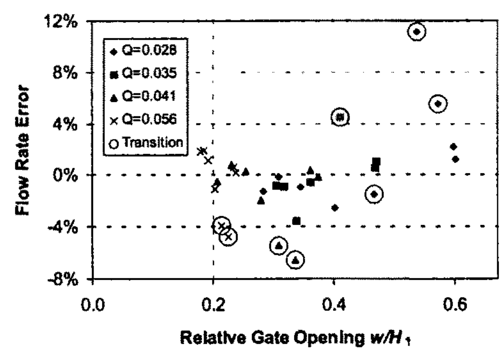

Figure 7. Error in discharge resulting from use of the Energy-Momentum method as a function of relative gate opening.

At this point, some verification of the relationships was attempted. The energy and momentum equations

(5 and 6) were solved with only knowledge of the upstream and downstream water levels. The relationships in Figures 2, 3 and 6 were used, along with p =0.643. The resulting errors in discharge are shown in Figure 7. The circled values are those that fall within the sharp transition range in Figure 6 (energy adjustment values between 0.2 and 0.8). Those within the transition zone have errors that ranged from –8% to 12%, while all the other values are estimated to within –4% to 3%. We also speculate that the relative depth at which the transition shown in Figure 6 occurs is a function of the ratio w/H_1, although there are not enough data points to define such a relationship (i.e., Figure 6 would have to have a family of curves). Further studies are needed to define this relationship as a function of w/H_1. Also, additional studies of submergence are needed at values of w/H_1 above 2/3, a theoretical limit at low submergence, but which can be greatly exceeded at high submergence.

APPLICATION

As an example, we show an application at the Salt River Project (SRP) in Arizona. At some check structures, operators report errors in computed flows as high as 50%. SRP has been using the submerged radial-gate calibration method suggested by Bos (1989). Under this method, the gate is assumed to be submerged when the downstream water level reaches the conjugate depth. In Figure 8, the calibration relationships for a fixed gate and given upstream water level are shown as a function of downstream (afterbay) depth. The horizontal line at the top represents free flow (i.e., not influenced by downstream depth). The far right point of this horizontal line represents the depth conjugate to the free-flowing jet thickness at its vena contracta for a downstream channel of the same width as the gate. The lower heavy line is the energy-based submerged-flow

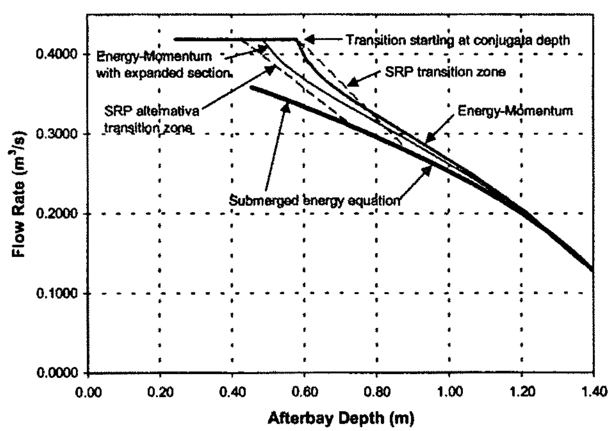

Figure 8. Radial-gate flow rates computed with energy equation (as used by SRP) and the Energy-Momentum method. (Fixed gate opening and upstream depth).

solution recommended by Bos (1989), where discharge is proportional to the square root of the head difference. No recommendation is given on transitioning from the conjugate depth point to the submerged-flow line. A straight drop is implied, but is clearly unreasonable. SRP chose to use a 1 foot (0.3 m) transition zone, described essentially by a straight line. When compared to the Energy-Momentum solution proposed here (heavy line to upper right), this is not an unreasonable approximation. Note that both solutions start the transition at the same downstream depth, as they should. However, SRP found that their transition equation did not fit their field data very well. As an alternative, they chose to place the transition zone in the middle of their 0.3 m interval (0.15 m on either side of conjugate depth). For some cases this provided a better fit. (An additional transition zone in between these two has also been used). Also shown in Figure 8 is the Energy-Momentum solution when the downstream channel is twice as wide as the gate. Submergence occurs at a lower downstream depth – i.e., the conjugate depth has changed. Their need to provide different transition zones for these gates can be entirely explained by application of the E-M method, as shown by the range of downstream depth at which submergence starts. The energy-equation solutions for submerged flow are not able to predict such performance.

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ADVANCES IN PLC-BASED CANAL AUTOMATION

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ABSTRACT

A short history of canal automation is given. PLC-based canal automation is relatively new. Advances in PLC-based canal automation are listed. Also listed are some of the remaining challenges. Recent advances have been made in understanding unsteady flow simulation procedures, the form of the control algorithms used, the tuning procedures for these control algorithms, and the field programming of the algorithms into PLCs. The experiences of the Cal Poly Irrigation Training and Research Center (ITRC) in automating a variety of canals with upstream and downstream control are given.

INTRODUCTION

Canal Automation History

Canal automation in this paper refers to closed-loop control in which a gate or pump changes its position/setting in response to a water level, flow rate, or pressure because that level/rate/pressure is different from the intended target value. "Closed loop" means that the action is performed without any human intervention. The automation may be performed through hydraulic, electrical, electronic, or a combination of these, means.

Early canal automation (pre-1950's) was characterized by the use of hydraulic gates. Flap gates were investigated in The Netherlands by Vlugter (1940). Cal Poly ITRC has recently reported the history of these gates and a new design procedure for them (Burt et al., 2001). Danaidean gates have been used in California since the 1930's and are still used in many irrigation districts for both automatic upstream control and downstream control.

The Nerytec Company from France became famous in the 1950's and 1960's for its hydraulic gates, such as the AVIS, AVIO, and AMIL models. These robust gates have been used around the world for upstream control and level top downstream control (Goussard, 1987).

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In the 1960's and 1970's, canal automation in the U.S.A. proceeded in 4 aspects:

1. Electro-mechanical controllers (commonly called "Littlemen") were developed and installed on projects throughout the western U.S. The legacy of these controllers continues, as many new automated sites with PLCs retain the old Littleman logic – with its inherent simplicity and limitations.
2. A few large water conveyance canals were installed with remote monitoring and remote manual control. Most notable is the California Aqueduct, which has been mistakenly identified as an automated facility for decades.
3. With the advent of computers, a few researchers were able to develop unsteady open channel flow simulation models – which began to open up possibilities for studying new methods of canal automation.
4. A few engineers and researchers began to try to apply control theory to the actual automation of canals. Perhaps most notable are the early attempts by staff from the US Bureau of Reclamation to install HY-FLO and EL-FLO on several canals for downstream control.

In 1987, a landmark American Society of Civil Engineers (ASCE) specialty conference was held in Portland, Oregon. Entitled "Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems", this conference brought together specialists from around the world to discuss various canal automation techniques.

Since 1987, there has been an evolution in the understanding of the nature of canal automation. This evolution has been assisted through various specialty conferences, primarily sponsored by the former Irrigation and Drainage Division (now the Water Resources Division) of ASCE. The evolution can be describes as follows:

1. In 1987, we had a simplistic view of automation. We understood the basic ideas of upstream and downstream control, and simulation models were becoming popular. Furthermore, personal computers were becoming common and there was a general feeling that computer-based automation would rapidly sweep the irrigation world.
2. In 1991, an ASCE specialty conference was held in Honolulu to discuss the challenges with selecting an unsteady flow simulation model. We were beginning to understand that the problem of simulation was more challenging than we had previously thought, and that there were substantial differences between simulation models. Partly as result of this awareness, authors of these programs began to make them more user-friendly and flexible.
3. By the mid-1990's, the interest in downstream control of many forms and in centralized control was quite strong – at least in theory. Numerous papers were published with various algorithms that had been simulated. But very few successful PLC-based automation schemes had been

successfully implemented. In fact, it appeared that most PLC-based automation schemes were failures rather than successes.

4. By the mid-1990's we also began to realize that, although the automation of the gate was very important, we had not paid sufficient attention to how the harmonics of flow disturbances in individual pools could cause instabilities in control. Therefore, attention shifted to understanding pool dynamics, with the hope that standard control theory (used in other industries) could be applied to irrigation canals. Terminology such as "PID" (Proportional-Integral-Derivative control logic) was heard much more frequently.
5. Upon arrival of the late 1990's we recognized that:
    a. There were very few successful applications of PLC (Programmable Logic Control) based automation systems. Certainly there were good examples of individual gates being automated with the use of PLCs, but there were very few whole systems with gates in series.
    b. SCADA (Supervisory Control and Data Acquisition) had advanced tremendously in the past 5-10 years, and the equipment and communications were much better than before. However, it was difficult to locate a good integrator company (a company that is responsible for the selection of all components, the installation, and commissioning of a SCADA system) even in the U.S.
    c. We had underestimated the complexities of going from an algorithm to actual implementation in the field.
    d. In spite of all of the tremendous theoretical work on downstream control, we still did not have adequate control algorithms for the most simple canal automation procedure – upstream control. Or if we had the algorithms, we were missing essential ingredients and we did not have good ways to tune the controller constants.
    e. The bottom line is that we did not have a package for local upstream or downstream control with PLCs that was easy and quick to implement in irrigation district canals for gates in series.

ITRC CANAL AUTOMATION WORK

The California Polytechnic State University Irrigation Training and Research Center (ITRC) has been involved in canal automation training, technical assistance, and research since the 1980's. ITRC believes in the "Keep It Simple" rule, and continues to recommend simple solutions such as hydraulic gates, long-crested weirs, regulating reservoirs, and remote monitoring where appropriate. But there is an increased need for tighter and more flexible control that often cannot be accomplished with those simple techniques. Therefore, ITRC has actively participated in PLC-based irrigation district automation implementation since the mid-1990's.

With PLC-based control, ITRC has attempted to work with excellent companies, and has tried to incorporate the best simulation models, equipment, control algorithms, HMI (Human-Machine Interface) software, and training that is available commercially and theoretically. There are so many challenges to successful implementation of PLC-based control that it would be fool-hardy for ITRC to work with anything but the best in cooperators, hardware, and software. In general, ITRC's role is to:

1. Select the control logic to be used for a particular project.
2. Select, develop and tune the control algorithm that dictates the gate movement.
3. Assist the irrigation district in specifying the SCADA system characteristics.
4. Work with the district in locating a good SCADA integrator.

Our ultimate objective is to make the technology much more user-friendly, simple to implement, and robust, so that commercial companies can implement it rapidly and effectively in irrigation districts.

Working in this way with the USBR and individual irrigation districts, we have helped to implement the following types of PLC-based canal automation:

1. Upstream control
    a. With overshot gates in series
    b. With radial gates in series
2. Flow rate control
    a. With Replogle flumes
    b. With Acoustic-Doppler Flow meters (ADFM)
3. Downstream control with the control point at the heads of the pools (i.e., level top control)
    a. With overshot gates in series
    b. With pumps in series
    c. For a single pool.
4. Downstream control with the control point at an intermediate location within the pools, with pumps in series

Each case has been different. These cases represent the majority of conditions that can be encountered for distributed control – our preferred method of automation at this time. Distributed control utilizes a PLC at each control structure/pump. The PLCs operate automatically and independently (one per site), but they are remotely monitored via a SCADA system at the irrigation district office. From the office, a person can switch any device to "manual" operation, or can change target water levels/pressures/flow rates.

The three points below summarize our understanding of PLC-based automation at this point in time.

1. Canal automation is much more complex than we had earlier believed.

2. Significant challenges remain in meeting our goal of developing simple procedures and guidelines for automation of irrigation district canals.
3. We have abandoned the dream of developing "simple" procedures for PLC-based canal automation. Our focus is now on developing a transferable procedure – one that irrigation districts and consulting engineers can successfully use without needing a tremendous theoretical background in control theory and hydraulics.

Having stated the challenges, it is also clear that over the past 3 years there have been significant improvements in our understanding of PLC-based canal automation. In addition to having learned from actual field implementations, funding through the California Energy Commission has allowed us to work with others to improve the theoretical understanding of the PLC-based canal automation process. In the remainder of this paper, we will share some of what we have recently learned for each of these PLC-based components:

1. Simulation models
2. Simulation procedures
3. Algorithms
4. Tuning of algorithms
5. PLC and sensor constraints
6. PLC programming by integrators

PLC-BASED CANAL AUTOMATION COMPONENTS

Simulation Model

ITRC presently uses CanalCAD, which utilizes the simulation technology based on algorithms developed by Cunge, Preissmann, Chevereau, Holly, and others at SOGREAH of France. CanalCAD has a user interface that was developed through a combined effort of ITRC, Imperial Irrigation District, and the Iowa Hydraulic Research Institute. We are examining other models for their suitability as a replacement.

In the past 2 years, CanalCAD has been upgraded at ITRC expense so that we can select any location within a pool as the "target" control location. This means that in our control algorithm subroutine, we can extract water levels or flow rates from any designated point. In the past, we were limited to 5 specific locations within any pool.

Key ingredients for any acceptable model include:

- Hydraulic correctness of steady and unsteady flow conditions.
- One second simulation timesteps
- Capability to simulate at least 20 pools in series.
- Capability to solve for initial steady state conditions automatically, including all water surfaces, flow rates, and gate positions.

- Ability to program the control algorithm within the simulator, as a subroutine.
- Ability to model a wide range of structure combinations at any single location.
- Quick computational speed.

With the type of work ITRC does, we have never needed or wished for a simulation program that includes branching canals.

We have occasionally seen discrepancies between actual canal hydraulics and CanalCAD-simulated hydraulics. These discrepancies have been noticed during extreme conditions, but in fact we do not know how extensive they are. We know that in most cases, the control that we have predicted with CanalCAD is what we have obtained. We have also verified some basic facts, such as wave travel characteristics in long canals.

In practice, it is difficult to obtain good data for comparison of simulated versus actual hydraulics because of the uncertainties of actual flow rates, canal dimensions and roughnesses, and water levels. We have recently revised our "commissioning" procedures for new installations so that we can obtain better checks on actual vs. simulated values. We now utilize a specific step-by-step procedure for testing PLC control at each structure, one at a time, for a new installation. Each structure must be "commissioned" by testing the algorithm's ability to maintain a steady state condition, and then progressing into small flow rate changes, and then to larger flow rate changes. This process requires several days. We monitor the actual water levels and the control response of the PLC and then duplicate the control response in CanalCAD. In part, we do this to confirm that there has been no programming error in the PLC.

Figure 1 shows an example of a discrepancy that we cannot yet explain. It is for a single pool that has downstream control at Sutter Mutual Water Company. The water level is controlled at about 82% of the distance down the pool, and a variable-frequency drive (VFD) pump controls the inflow to the pool. The commissioning of the installation required that a flow rate change be made at the far downstream end of the pool. Figure 1 shows that the actual control is better than what was predicted in CanalCAD.

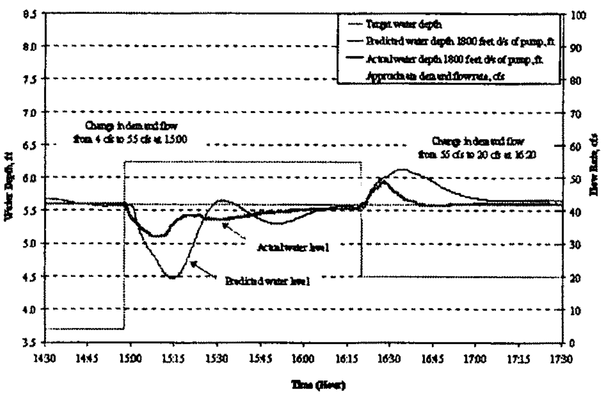

Fig. 1. Simulated and Actual Water Level Changes at Portugese Bend Canal, Sutter Mutual Water Company.

Another discrepancy that we have noticed occurs at conditions of low flow rates. In several canals, the irrigation district personnel have noticed a slow oscillation of gates or pumps with downstream control. We have not been able to duplicate the oscillation in CanalCAD. Our immediate solution is to reduce the magnitude of the response at very low flow rate conditions (e.g., at 5% of the maximum flow rate).

Simulation Procedures

An important lesson we have learned is that on some canals the simulation timestep must be as small as one second. In prior literature and our own previous experiences, it appeared that one-minute simulation timesteps were adequate. The case that first caught our attention was the Stanfield-Furnish Branch (SFB) Canal near Umatilla, Oregon. This is a relatively short canal of 4 pools with minimal storage, having very large flow rate changes (greater than 50%) at the tail end. The gates are overshot bottom-hinged gates, with the exception of radial gates at the head of the canal. The control technique was local downstream control, with the target located immediately downstream of each gate. Control was to be executed once/minute.

With the SFB Canal, we tuned a PI (described later in this paper) algorithm for downstream control in CanalCAD using a one-minute simulation timestep (Figure 2). When the algorithm was implemented in the field, the gates had such excessive movement that the panel heaters overheated and prevented more gate movement. Fortunately, when we heard about this problem it coincided with some observations we had made when doing some other development work that indicated a need for smaller simulation timesteps. When we simulated the operation again, but with a 1 second timestep, the oscillations appeared (Figure 3). It was obvious that we had missed the pool resonance problems in our prior simulations. Using a 1-second timestep to simulate one-minute control, we retuned our control algorithm and added a filter (making it PIF control), with the results seen in Figure 4.

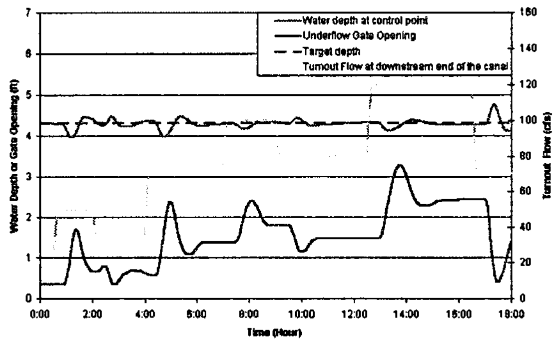

Fig. 2. Downstream Control Simulation Results of Initial Tuning of PI Algorithms on the SFB Canal Using a 1-Minute Simulation Timestep and a 1-Minute Control Timestep.

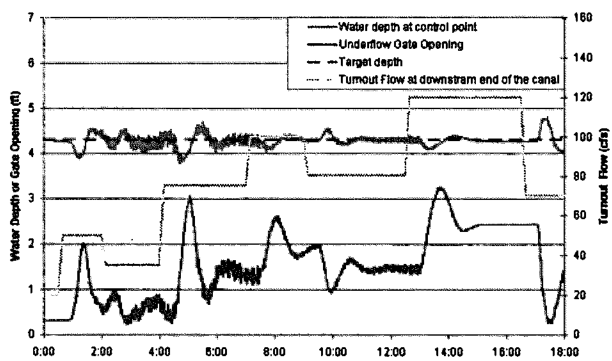

Fig. 3. Downstream Control Simulation Results on the SFB Canal Using a 1-Second Simulation Timestep and a 1-Minute Control Timestep, Using the Original PI Algorithm.

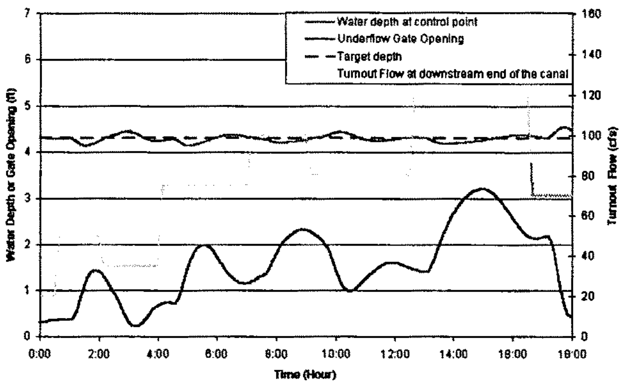

Fig. 4. Downstream Control Simulation Results on the SFB Canal Using a 1-Second Simulation Timestep and a 1-Minute Control Timestep, Using a New PIF Algorithm.

Algorithms

ITRC uses PI (Proportional-Integral) for many cases, PIF (Proportional-Integral plus Filter) equation when it encounters conditions with resonance problems, and once or twice we have used PID – which includes the derivative component. With PIF, the filter accounts for the previous error, and all previous errors, but as the errors become more distant in time, their influence is less and less. The form of the PIF equation can be described as follows:

$$\Delta u = UD \cdot UF \cdot [-KP \cdot (FE1 - FE2) - (KI \cdot FE1)]$$

where

Δu = change in gate position, feet

KP is the first PI constant, determined in MatLab

KI is the second PI constant, determined in MatLab

FE1 = The filtered error for the present timestep

$$FE1 = (FC \cdot FE2) + [(1 - FC) \cdot ENOW]$$

where

FC = a filter constant

FE2 = the value of FE1 for the previous timestep

ENOW = present unfiltered error

= (Actual water level - Target water level)

where

Actual water level is the average of at least 60 measurements taken over the last minute.

UF is a factor that determines how much the gate must be opened for a certain flow rate change. Its value depends on the gate position. It is generally of the form:

$$UF = [a1 \cdot (a2 \cdot u^2 + a3 \cdot u + a4) + a5]$$

UD is a factor (+1 or −1) that depends on whether the gate is an overshot gate or undershot gate

Tuning of Algorithms

We continue to try to develop or locate a procedure that tunes the PI or PIF algorithms satisfactorily and quickly. Historically, we systematically ran simulations starting with one set of KP and KI constants, and step-by-step varied the KP and KI constants in an attempt to bracket the best values. This was a trial-and-error technique that was complicated by the need to use different KP and KI constants for each pool. Because of pool and gate interactions, the tuning of all controllers must be done simultaneously.

The KP and KI combination that produced a quick stabilization with minimal overshoot was chosen if adjacent KP and KI combinations also provided stable solutions. Because of our uncertainties regarding simulated versus true results, we never want to select tuning constants that are on the verge of causing instability. This procedure typically required a minimum of one month of systematic tuning using CanalCAD, and often took longer.

Recently, ITRC has been working with Jan Schuurmans and Peter-Jules van Overloop from The Netherlands to develop a systematic tuning procedure that will be better and also much quicker. The procedure is to first determine resonance and wedge storage characteristics for each individual pool using CanalCAD, and then to use a special MatLab® routine to simultaneously tune the unique PI or PIF constants for each gate or pump controller. To date, we must still do about a week's worth of manual trial-and-error procedure to fine tune the MatLab results. However, we are optimistic that this tuning procedure can be improved further.

PLC and Sensor Constraints

We have encountered several PLC and sensor constraints. On the PLC side, we have learned that many PLCs require a significant part of a minute just to run through various checks of equipment, to read values from sensors, and to communicate with SCADA systems. Furthermore, the manufacturers are often unable to tell us how much time is required. Because our simulations should duplicate the actual computations, this is problematic. We have also found that some brands will take 60 sensor values per minute and provide an average, but those 60 sensor values are all read within the last 1-5 seconds of a minute.

For applications, we now insist on redundancy of key items. Specifically, we state that all of the key sensors be duplicated, plus the sensors must be wired into different power supplies and A/D converter modules in the PLCs. We assume that it is not a question of "if" a sensor will fail, but a question of "when". Although there are numerous techniques to use software to check for problems with single sensors, we have found out that this adds a tremendous complexity to the programming of the PLC that is unnecessary if redundancy exists.

On the sensor side, there are the classic problems with accuracy, calibration, and resolution. But a new challenge is presented when one uses an ADFM or similar device to measure flow rates at the head of a canal (for control purposes). Figure 5 shows that there is tremendous noise in their signals. We have examined numerous filtering techniques, but in the end we have concluded that we need at least 10 minutes of continuous readings before we can use an average value for a control decision. This complicates the control of headworks on some canals. In contrast, using a Replogle flume to measure flows at the head of a canal has the advantages of (i) little or no random noise in the signal, (ii) inexpensive redundancy of the water level sensor, and very importantly for control (iii) because the Replogle flume is a critical flow device, the new flow rate stabilizes very rapidly, so it is easy to determine how much to change the flow control device.

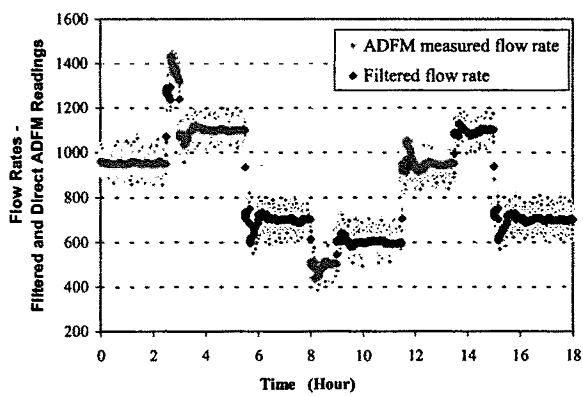

Fig. 5. ADFM and Filtered Signals at Headgate Rock Dam – CRIT Irrigation Project, Arizona.

PLC Programming by Integrators

The complexity of dealing with integrators has been a surprise. Integrator companies in the irrigation market have been quite independent, with little independent review of their work. Their procedures for documentation of programming, their neatness of organizing wiring and panels, their usage of programming languages, and their exposure to PI algorithms for canal automation are quite varied. This means that nothing can be taken for granted – even if an integrator can list numerous completed projects.

Three items are of particular concern to us:
1. A good integrator will always understand hardware, installation, communications, and programming quite well. But it is rare that an integrator is familiar with modern canal control algorithms, and how they are tuned within a simulation model. This can be a problem if the integrators take unwarranted liberties in the programming of the control algorithms that we supply as well as with the tuning constants that we provide.
2. Integrators sometimes embed numerous checks into their code with various hidden constants (of their own selection) that can shut down a gate or pump operation. The irrigation district operators (i) do not know these constants exist, (ii) do not know how to access them, (iii) must generally personally visit the PLC to change the constants, and (iv) do not really understand how the constants should be changed. We believe all constants and alarms should be transparent and changeable from within the office via the SCADA system. A portable PC with a copy of the office HMI (Human-Machine-Interface) software can be used in the field to change constants if it is desired to make in-field adjustments.
3. The "control algorithm" for a gate (the algorithms that are published in irrigation literature) may only occupy 10% of the total programming that most integrators put into the PLC. The remainder of the programming handles numerous checks of equipment and sensors, consideration of gate inertia, and other factors. We are working on specifications that will minimize unnecessary programming.

SUMMARY

In summary, what we started out thinking was a simple algorithm challenge is in fact quite complex. As a profession, we underestimated the complexity of automating canals with PLCs – even with simple upstream control. We now understand that there are challenges in equipment, simulation models, tuning procedures, and field programming. Having identified the weaknesses, we are systematically solving each one.

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ON TRACKING SEDIMENT PARTICLE SIZES IN FURROW-IRRIGATION INDUCED EROSION FOR MODELING PHOSPHORUS TRANSPORT

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L. Mateos[3]
J.V. Giráldez[4]
A. J. Clemmens[5]

ABSTRACT

An aspect of surface-irrigation performance is the erosion and transport of soil from which furrows are formed. From western Nebraska to the Pacific Northwest, as well as in other parts of the world, loss of soil fertility and discharge of eroded sediments, sometimes with phosphorus adsorbed, into receiving streams are significant problems. Like other measures of performance, sediment movement depends upon field hydraulic properties and system design and management. Simulation models applied to individual hypothetical events can be of great assistance in developing proper designs and management scenarios. Issues which arise when extending the single representative-particle-size approach in the erosion component of the simulation software, SRFR Version 4, to track individual size-fractions of the sediment mix through their entrainment, transport, and deposition, are described herein. In particular, the extension of existing empirical transport-capacity formulas to small silt particle sizes is detailed. The assumption that Laursen's (1958) formula is sufficiently well

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grounded theoretically to warrant some application to sediment sizes smaller than those in his empirical database leads to plausible results. Ultimate justification will be based on comparisons of simulated sediment-load hydrographs with field measurements.

INTRODUCTION

Erosion science was introduced into the version 4 series of the general surface-irrigation simulation model, SRFR (Strelkoff, Clemmens, and Schmidt, 1998), generally along the lines developed in Fernandez Gomez (1997), but narrowed to a single, representative particle size (Strelkoff and Bjorneberg, 2001). In SRFR, the flow of water in furrows is predicted by numerical solution of the unsteady equations of mass and momentum conservation coupled to generally applicable empirical equations describing infiltration and soil roughness and to given furrow configuration and inflow hydrograph. Selection of appropriate field parameters yields infiltration distributions and runoff in reasonable agreement with measurements. The erosion component consists in applying the simulated flows to site-specific empirical determinations of soil erodibility, to general empirical sediment-transport relations, and to physically based deposition theory to provide estimates of soil erosion, flux, and deposition at various points along the furrow as functions of time. Total soil loss off the field and ultimate net erosion and deposition along the furrow follow. In version 4, a single representative particle or aggregate size was assumed adequate. Results of comparisons with measurements of sediment concentrations in the furrow quarter points and in the tailwater showed excessive sensitivity to selection of empirical parameters.

In addition, adsorption of phosphorus (P) to eroded sediment particles in furrows and their subsequent transport off site is an environmental problem drawing increasing attention from regulatory agencies concerned with nutrient loading of surface water bodies. In furrow irrigation in which erosion is a significant factor, sediment in the tailwater is the primary vector for P. The concentration of P on the sediments is directly related to the combined surface areas of the particles in transit. Thus, both for refined calculation of sediment transport in its own right, and its effect on P fate and transport, predictions of particle/aggregate size distributions in the transported mix are essential.

This paper describes the issues which arose when the general approach of SRFR version 4 was extended to tracking individual sediment sizes in the distribution. Most of the issues are essentially logical in nature, considering, e.g., size-dependent entrainment (taking into account the likelihood of larger-size particles shielding smaller ones beneath), individual transport downstream, and potential deposition. However, a theoretical problem arose in connection with the transport-capacity formula when applied to the smaller-size fractions of the distribution. The paper closes with an examination of the problem and a potential solution.

PROCEDURES

Subdivision of measured particle-size fractions; fitting a continuous distribution

As noted in Strelkoff and Bjorneberg (1999), treatment of erosion/transport/deposition phenomena in terms of a single representative particle size leads to results overly sensitive to the selected size. While it is possible to match field-measured hydrographs of total load with a reasonable size in the range of measured values, selection of equally reasonable values, greater and lesser, can lead to significant changes in simulated loads. For example, increasing somewhat the representative particle size can lead to loss of *all* sediment in calculated tailwater loads. A mix of particle sizes circumvents this problem and leads to gradual changes in total sediment load as the fractions of each size are varied.

In view of the experimental difficulties of measuring many fractions of particle/aggregate sizes in a mix, a first cut at providing the desired gradual influence of particle sizes is achieved by fitting a continuous distribution to a coarse measurement. Material retained on a number 300 sieve (about 50 microns) was defined as sand, while particles smaller than 8 microns -- as established by standard pipette techniques -- were called clays, with all the material in between, silt. For example, Figure 1, a cumulative-distribution graph prepared for a mixture of 25% sand and 60% silt (Bjorneberg, 2002) shows by open-square symbols 75% of the mix finer than 50 microns, and 15% finer than 8 microns. To subdivide this coarse separation into plausible finer fractions, a continuous mathematical function was fitted to the field-measured mix, in the given instance, a normal distribution, with two parameters, mean, •,•and standard deviation,•.• In general, with more than two measured points, the cumulative normal distribution $p(D_s,$•,•) would be a best fit, in the least-squares sense – a nonlinear mathematical exercise, since p=p(x), with

$$x = \frac{D_s - \mu}{\sigma} \tag{1}$$

Here, p is percent finer than, and D_s is the particle size. With just two points to fit, the best fit, arrived at by the same nonlinear procedures, is perfect.

In Figure 1, the heavy open squares at particle sizes D_s=8 and 50 microns indicate the size-fraction boundaries in the field. The light grey curve is the straight (on arithmetic graph paper) line connecting the field points. The heavy solid curve is the fitted normal distribution. The dotted line segments running vertically, arbitrarily between p=0 and p=10%, represent boundaries selected for the theoretical particle-size fractions constituting a finer subdivision of the field distribution. And finally, the lengths of the heavy vertical line segments located between these theoretical boundaries represent the percentages in the theoretical

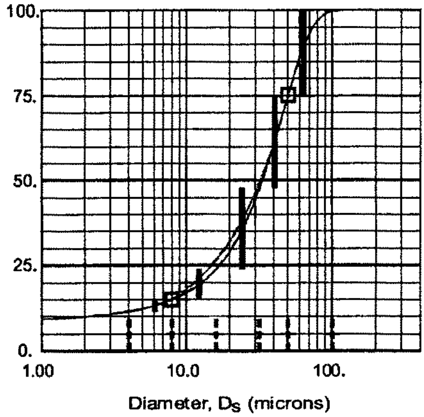

Figure 1. Theoretical continuous (Gauss normal) distribution fitted to field measurements (open squares) of particle sizes. Portneuf soil (Bjorneberg, 2002)

mix of the pertinent portion represented by the average size given by the D_s-location of the line segment. These averages, found for each fraction by integration of $p(D_s)$ and division by • p, represent the theoretical fractions for calculation of entrainment, transport, and deposition. Besides the fractions shown, another 11%, with an average size of 0.7 microns, lies off the graph; 0.3%, with an average size of 109 microns, is too small to see on the graph.

The mathematical operations described above were facilitated by a standard fit to the cumulative normal distribution $p(x)$ (formula 26.2.19, Abramowitz and Stegun, 1964).

Selective entrainment of particle classes; shielding

Following Borah et al. (1982) and Fernandez Gomez (1997), the susceptibility to detachment of the different particle/aggregate size and density classes in the mix of furrow bed material depends upon both the class, and its distribution throughout the mix. In principle, of the particles exposed to the flow, the smaller ones are more susceptible to entrainment than large ones, and in the conceptual model adopted, those will be swept up into the flow first, to be followed then, by ever larger ones, up to the transport capacities of the flow, individually for each class, and in total for all. The distribution influences the process of entrainment because, in the mix, large particles can shield smaller ones beneath.

The process of detachment is calculated starting with the smallest class ("1") available in the mix. A fractional portion is directly available to the flow (is not covered). If this is taken up without exceeding the transport capacity, erosion of class 2 is calculated, first, the fraction directly available to the flow. With this swept up, a fraction of class 1 particles is uncovered, and this is entrained, provided transport-capacity constraints are met. Then the fraction of class 3 particles directly available to the flow, is calculated. Once these are entrained, another fraction of class 1 particles becomes available, as well as another fraction of class 2, entrainment of which then exposes still another fraction of class 1 particles. The process of entrainment stops for a particular class if either the transport capacity for that class is exceeded, or the total transport capacity is exceeded, or the mix in the depth of soil interacting with the flow runs out of that class.

Transport capacity; fine-grained soils

The transport-capacity formula selected for the model was Laursen's (1958), because it was intended for total load (bed load and suspended load together), its database included particle sizes in the silt range (in contrast to other popular formulas developed for river sands, gravel and cobbles), and because it was recommended by Alonso *et al* (1981) for long straight channels in agricultural soils. The maximum possible percent mass concentration of homogeneous particles of size D_s in a flow, without deposition, is given by the product of three terms,

$$100\frac{G_{CH}}{\rho Q} = \left(\frac{D_s}{R}\right)^{\frac{7}{6}} \left(\frac{\tau_s}{\tau_{Cs}} - 1\right) F\left(\frac{\sqrt{\tau / \rho}}{w_s}\right) \qquad (2)$$

In this expression, in any consistent system of units, G_{CH} is the transport capacity (mass per unit time), ρ is the density of water (mass per unit volume), and Q is the volumetric rate of flow (volume per unit time); the cross section of flow is characterized by the hydraulic radius, R (length); τ_s is the tractive force on the grains of sediment per unit boundary area (boundary shear). The critical shear (at incipient movement), τ_{Cs}, is given by the Shields diagram (see, *e.g.*, Rouse, 1950) as a function of particle Reynolds number, $\left(D_s\sqrt{\tau / \rho}\right) / \nu$, in which the square-root function is the *shear velocity*, based on the total shear on the bed,

$$\tau = \rho g R S_f \qquad (3)$$

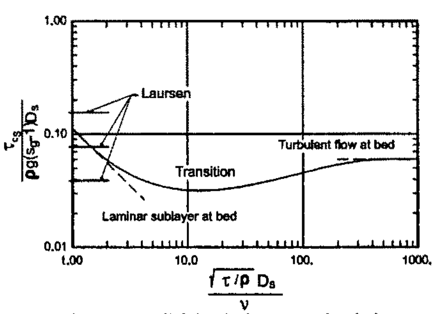

Figure 2. Critical shear (at incipient particle motion) -- after Shields, cited by Rouse (1950) -- and Laursen approximations

and ν is the kinematic viscosity. In eq. 3, g is the ratio of weight to mass, and S_f is the friction slope, essentially equal to bottom slope in furrows steep enough to experience erosion. Figure 2 shows the plot of a cubic spline (Press et al. 1992) fitted to Shields' data.

The factor, F, is a function of the ratio of shear velocity to sediment fall velocity w_s, and is given by an empirically determined curve in Laursen (1958) reflecting his data

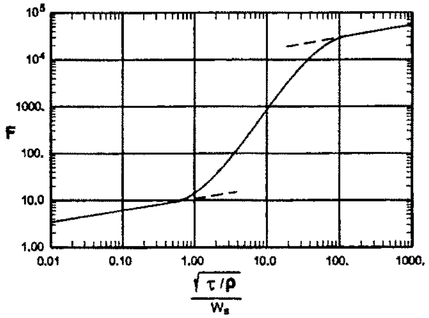

Figure 3. Empirical sediment transport capacity function (after Laursen, 1958)

sets of measured total loads. In developing the curve, Laursen assumed a specific gravity of all sediments as 2.65, and expressed the results in percent concentration. Figure 3 shows the plot of a cubic spline fitted to the graph in Laursen, 1958. The graph of the *F* function appears as a monoclinal rising curve with increasing values of $\sqrt{\tau / \rho} / w_s$. Total shear, •;• includes drag of bed forms and is given by eq. 3.

The fall velocity, if not given as input data, is calculated in SRFR by the general formula of Rubey (Simons and Senturk, 1992). However, in the range of particle sizes considered herein, these formulas mathematically reduce almost exactly to the far simpler

$$w_s = \frac{g\left(s_g - 1\right)}{18\nu} D_s^{\,2} \tag{4}$$

(Stokes Law) in which s_g is the specific gravity of the sediment. Thus, all else being equal, *F* increases with decreasing sediment size, in keeping with intuitive conceptions and observations of transport capacity.

Now, the first term on the right of eq. 2 clearly decreases with decreasing sediment size, but for the most part, F increases faster, so the product of the two terms increases with decreasing sediment size, except, inevitably, at very small values, as D_s approaches zero.

The second term on the right side of eq. 2 contains the ratio of tractive force •;•on the particles on the bed to critical tractive force, •$_{cs}$. In the data leading to Laursen's formula, transported sediment particles had always been entrained from the bed, so the tractive force always exceeded the critical (the formula was not intended for washloads), and the second term was always positive. In evaluating the empirical *F* function from measured transport, Laursen specified that •;•was the tractive force (expressed as a shear stress) on the grains themselves, as opposed to bed forms and other factors affecting the total shear on the bed; •;•was specifically calculated by the formula,

$$\tau_s = \frac{\rho V^2}{58.2}\left(\frac{D_s}{R}\right)^{\frac{1}{3}} \qquad (5)$$

derivable from Strickler's relation between Manning n and equivalent sand-roughness size (see, e.g., Henderson, 1966, p. 98). Here, V is the water velocity.

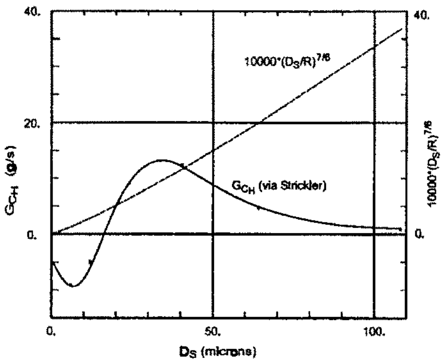

Figure 4. Example transport capacity with tractive force calculated by Strickler/Manning relation (eq. 5)

When eq. 5 is applied to the small grain sizes at the lower end of the fitted size distribution in Figure 1, the calculated tractive force on the grains proves smaller than the critical shear stress, and the resultant calculated transport capacity turns negative[6]. The transport capacity in Figure 4 shows the effects of both the sign reversal of the second term of eq. 2 and the decreasing size of the first term, inexorably drawing the transport capacity to zero as particle sizes decrease. It was noted, however, that the small particle sizes are characterized by that portion of the Shields diagram at the far left of Figure 2. Although the flow in the stream is generally turbulent, the particle Reynolds numbers are sufficiently small that the thickness of the laminar sublayer adjacent to the boundary is of the same order of magnitude as the particle diameters (Rouse, 1950, p. 790), with the smallest ones totally enveloped by the laminar region.

The Manning relationship for surface drag, on the other hand, assumes that the roughness elements -- the grains at the soil surface -- are so large, relative to the thickness of the laminar sublayer that would exist at a smooth boundary, that they completely disrupt the orderly, viscous flow in the thin layer that it would describe. The actual drag, then, would be determined by "fully turbulent," or "wholly rough" conditions in standard terminology, in which viscosity, characterized by Reynolds number, plays no role at all; dimensionless shear then depends solely on relative roughness, the ratio of grain size to hydraulic radius.

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[6] Laursen (1958) did not use the Shields data of Figure 2 directly, but instead, several constant values of dimensionless critical shear. For two of his data sets, he used the values, 0.08 and 0.16, and for the remainder, 0.04. The few runs for which the transport capacity approached zero or turned negative were rejected.

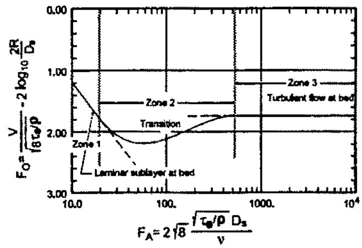

Figure 5. Dimensionless particle shear as a function of relative roughness and Reynolds number (after Rouse, 1946)

Thus, for the small particle sizes, there is a contradiction between the hydraulic conditions assumed for calculating τ_{cs} and those for τ_s. Now, Figure 5, derived from a set of experimental data (again fitted with a cubic spline) for pipes artificially roughened by uniform-sized sand grains, cited in Rouse (1946, Fig, 109, p. 206), shows the full range of interplay between viscosity (Reynolds number), relative roughness, and dimensionless shear. In Rouse (1946), the abscissa and ordinate variables are given by $F_A = \frac{\mathbf{R}\sqrt{f}}{r_0 / k}$ and

$F_O = \frac{1}{\sqrt{f}} - 2\log_{10}\frac{r_0}{k}$, respectively, exactly equivalent to those in Figure 5, but expressed specifically for round pipes. In the pipe-flow variables, f is the Darcy-Weisbach friction factor, a dimensionless shear stress, such that

$$f = 8\frac{\tau_s}{\rho V^2} \tag{6}$$

Ordinarily, eq. 6 is expressed in terms of the total wall shear, τ. The particle tractive force (per unit area), τ_s, is used here because it is *its* ratio to critical tractive force that appears in eq. 2. With the rigid uniform-sized-sand boundary of Figure 5, τ and τ_s are the same. The pipe radius is r_0, while k is the sieve size just passing the sand-grain roughness elements; in terms of our present variables, k=D_s. The pipe-flow Reynolds number is $\mathbf{R}$=VD/ν, with the pipe diameter D just 4 times the hydraulic radius. The unusual abscissa and ordinate, as compared to the standard Moody diagram (a plot of $f[\mathbf{R}, r_0/k]$) for artificially roughened pipes (Rouse, 1946, Fig, 108, p.205), were developed in order to collapse the set of curves in Fig. 108 into the single curve of Fig. 109 -- our Figure 5. The horizontal portion of the curve shown at the right in Figure 5 corresponds to a roughness so large that the laminar sublayer is completely disrupted, and flow resistance is governed solely by relative roughness, independent of viscosity. It is in this range that the basis for eq. 5, the Manning formula, with no reference to viscosity in its formulation, is an approximation to the pipe-flow data (Henderson, 1966, p.97).

In the hope that Laursen's dimensionless formulation is sufficiently general to allow some extrapolation of the F-function to a range of particle sizes lying outside those in his database, eq. 2 was modified by calculating τ_s by the entire function shown in Figure 5, rather than by eq. 5 alone. Intuitively speaking, we divide all of Figure 5 by Figure 2, instead of just the right-hand end of Figure 5 by all of Figure 2. Worthy of note, for plane beds of homogeneous material, $\bullet = \bullet_s$, and the abscissa scales of the two figures differ only by the factor, $2\sqrt{8}$; the transition range for both figures is seen to be approximately the same. Bogardi (1965) lends support to this approach by identifying $\bullet_s$ simply as the tractive force on a particle, without specifying that it is to be computed with the Strickler relationship.

In order to compute $\bullet_s$ from Figure 5, given the relative roughness and Reynolds number, an iterative scheme proves necessary, because of the appearance of $\bullet_s$ in both abscissa and ordinate of the figure. Indeed, with a cubic spline (on semi-logarithmic paper) fitted to that data cited in Rouse (1946, Fig. 109) lying between the limiting straight lines, F_O is given in terms of F_A by the following functions in the three zones, in which $\bullet = \log_{10} F_A$:

In *Zone* 1, *i.e.*, $F_A<19.4$: $F_O = 2.0 \bullet - 0.8$

In *Zone* 3, $F_A>525$: $F_O = 1.74$ (7)

In between, *Zone* 2, the fitted cubic is: $F_O = 1.002 \bullet^3 - 6.722 \bullet^2 + 14.34 \bullet - 7.67$.

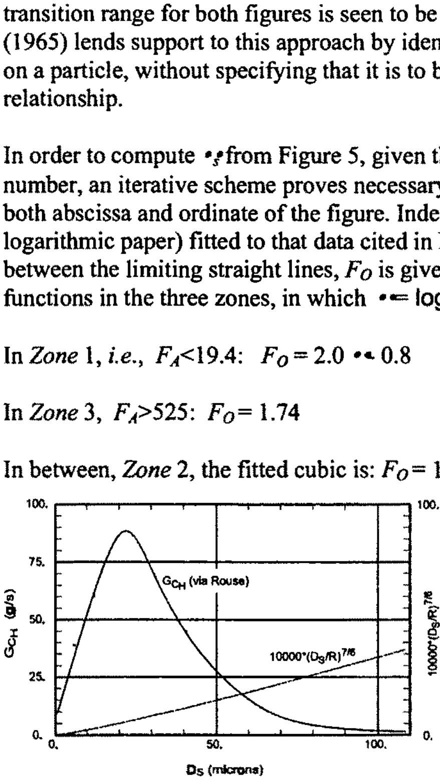

Figure 6. Example transport capacity with tractive force computed as per Rouse (1946)

Newton's method is used to find f and, hence, with eq, 6, $\bullet_s$ for substitution into eq. 2. This procedure, as can be seen in Figure 6, yields increased transport capacities as particle sizes (and fall velocities, w_s) decrease, for all but the smallest particle sizes of Figure 1. At that size, the first term on the right-hand side of eq. 2 always draws the calculated transport capacity to zero.

With the Laursen formula supplying the transport capacity of a flow for a single, homogeneous sediment, the transport capacities for each of the fractions making up a mix are found (following Fernandez Gomez, 1997) from the formulation of Wu and Meyer (1989), a reasonable way of apportioning total transport capacity amongst the size classes. The transport capacity, $G_C(i)$, for each of the constituent fractions, i, in a mix of N fractions, is found by multiplying the transport capacity for a homogeneous soil, G_{CH}, weighted as follows,

$$G_C(i) = \frac{G_{CH}(i)}{\sum_{j=1}^{N} G_{CH}(j)} G_{CH}(i) \tag{8}$$

The sum of all of the $G_C(i)$ is the total transport capacity, for all fractions.

Application to Portneuf soil

Bjorneberg (2001) cites entrainment data for a local Portneuf soil leading to an erodibility coefficient K_R of 0.0056 *s/m*, in the site-specific empirical equation for rate of detachment per unit area ($[Kg/s]/m^2$) by clear water, D_p,

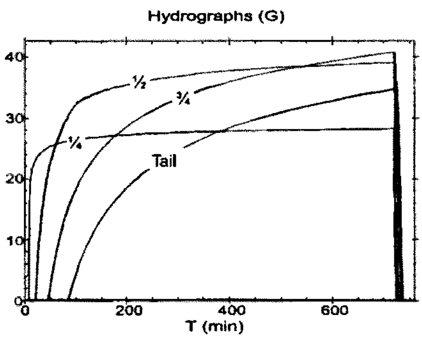

Figure 7 Calculated sediment loads at the quarter points and tail end of a furrow. Erodibility based on Bjorneberg 5-min. data; rec'd 11-26-01

$$D_p = K_R\left(\tau - \tau_{cD}\right) \tag{9}$$

The threshold value of shear for detachment was τ_{cD} = 1.01 Pa. Coupled to typical irrigation data on field roughness and infiltration, furrow geometry, and inflow hydrograph, the data led to the simulated sediment-load hydrographs shown in Figure 7, for the quarter, half, and three-quarter points of the furrow, as well as the tail end. The rise in average hydrograph ordinate in the upper reaches of the furrow testifies to the gradual addition of sediment to the furrow stream in that portion. The subsequent fall, on the other hand, is evidence of deposition, typical of Kimberly field experiments.

CONCLUSIONS

A significant start has been made in tracking the fate and transport of sediment particle-size fractions in furrow irrigated soils. Reasonable postulates for entrainment and transport, have been made. In particular, it appears that the Laursen formula for transport capacity can be extended to silts smaller than those in his database, if the tractive force on the smaller sediment grains is allowed to be influenced by the laminar sublayer of the stream enveloping them, in accord with standard fluid-mechanics analyses of turbulent flow in rough pipes. Extended comparisons between simulated and field sediment load hydrographs will prove or disprove this postulate.

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ELECTRIC LOAD SHIFTING IN IRRIGATION DISTRICTS – CALIFORNIA'S PROGRAM

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Ricardo Amón[2]
Darren Cordova[3]

ABSTRACT

During the 2000-2001 winter, California experienced a severe imbalance in electricity supply and demand that resulted in blackouts and brownouts. The state legislature initiated a number of emergency programs, one of which (Senate Bill 5x) was targeted for irrigation districts. The primary goal of SB 5x is to reduce peak period electricity demand. The California Energy Commission (CEC), acting under authority of Section 5(b) of the legislation, developed the "Agricultural Peak Load Reduction Program". The program was announced on June 1, 2001, and Cal Poly ITRC administers the irrigation district portion of the program for CEC. During the first 9 months of implementation, the irrigation districts voluntarily participated in load shifting, utilizing approximately $6.2 million in cost-sharing grant money. In addition, approximately 550 pumps were tested and pump repairs were made, resulting in an estimated savings of 16 million kWh.

PROGRAM OVERVIEW

Legislation and Peak Load Reduction

California Senate Bill 5x ("SB 5x") was enacted in April 2001 as urgency legislation in response to an imbalance in electricity supply and demand in the State. The goal of SB 5x is to reduce peak period electricity demand. The California Energy Commission (CEC), acting under authority of Section 5(b) of the legislation, has developed an Agricultural Peak Load Reduction Program.

One part of the Agricultural Peak Load Demand Program provides incentive grant payments to agricultural irrigation districts to install energy efficient hardware or make other conservation efforts to reduce peak period electricity demand. "Peak Period" is defined as weekdays, excluding holidays, from 12:00 p.m. to 6:00 p.m. during the months of June through September. Cal Poly ITRC administers the

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irrigation district component of the SB 5x program. The Center for Irrigation Technology (CIT) at Fresno State University administers the on-farm component. This paper focuses on the irrigation district program.

Three categories of projects have received grants under this program. The categories are:

1. Category 1. High efficiency electrical equipment and other overall electricity conservation efforts. Projects in this category must reduce peak load. An example project in this category is the construction of a regulating reservoir, into which water is pumped during off-peak hours and from which water flows by gravity during peak hours.
2. Category 2. Pump efficiency testing and retrofit/repair
3. Category 3. Advanced Metering and Telemetry. The majority of these projects were very simple, and paid for equipment that would allow pumps to be shut off as requested by California's Independent System Operator (ISO) Demand Relief Program. Districts participating in the ISO program agreed to shut off pumps if requested, and in return they received electricity at a reduced rate. The CEC program paid for special meters that confirmed the participation in the program, as well as telemetry for remote on/off operation of pumps.

A fourth category – retrofit of natural gas-powered equipment to alternative fuels – has not been utilized by the irrigation districts.

Program Schedule and Cost Sharing

The legislation was passed in April, with the desire to reduce peak electric loads immediately – to avoid summer power brownouts and outages. Contracts between CEC and ITRC were not completed until mid-May. Guidelines needed to be developed, application processes written, quality control measures implemented, verification procedures defined, etc. Obviously, civil works under Category 1 and pump testing/repair under Category 2 could not be implemented within a few weeks of the announcement of the program.

ITRC and CEC had discussed this project since December 2000, because the legislature needed to have some idea of how much money was needed for irrigation districts. ITRC had canvassed the major California irrigation districts for information, and had informed them that this program was in the works. Therefore, many of the districts were ready to act almost immediately – even if the application process had not yet been refined.

The program was originally intended to last until March 2004. However, in March 2002 the funding was reduced to from $10 million to $6.5 million, and December 2002 was declared the last date for applications to be approved. The

compression of the program is due to the financial crisis of the state of California – largely due to expensive power purchases by the State.
For projects in Categories 1 and 3, the grant can pay up to 65% of the project cost. The maximum reimbursement per kilowatt load reduction depends upon the date of project implementation – as a means of encouraging quick implementation of the projects. The reimbursement could be up to $350 per kilowatt for projects completed by July 31, 2001, $300 by September 30, 2001, and $250 by May 31, 2002. In reality, the maximum reimbursement has been almost always limited by the 65% cost sharing rule rather than by the dollars per kilowatt.

For Category 2 (pump testing and repair/retrofit) projects, the program reimburses up to 80% of the total pump tests, up to $200 for a "standard" test, and up to $250 for a special test that required two transects of data for flow measurement. For pump repair, up to 65% of the total cost has been reimbursed.

All projects are limited to installations that have existing connected electric load with a history of electricity consumption. Projects are approved on a first come-first served basis. In reality, the demand for the grant money has been less than the dollars available, so the issue of priority hasn't arisen. In addition, ITRC prepared and delivered a new pump test training program and new pump test standards. Pump test companies are required to meet the standards in order to be paid for their services.

Applications and Paperwork

The urgency of resolving the power crisis in California required quick implementation. That, in turn, required a well-defined yet simple application and verification process. Each of the categories of the program was unique, and within each category there were a variety of possibilities that would require different verification procedures.

Category 1: The program was designed as follows for Category 1:
1. Application forms were developed and placed on the ITRC web pages. This work was coordinated with CIT in Fresno, which has a similar program for on-farm and agricultural processing customers.
2. Computational spreadsheets were developed, and example computations for the value of the reimbursement were developed.
3. Districts submitted the application forms directly to ITRC for technical review and determination of eligibility and administrative completeness. This review process has gone quite smoothly, with a minimum number of questions. The districts have done an excellent job of submitting high quality applications. Several consulting engineering firms actively worked with their clients to fill out the applications.
4. ITRC reviewed the applications and defined the steps and data that would be needed for verification.

5. The program administrator for CEC was asked his approval; this approval was given within a few days. The district was given an e-mail approval to proceed – with financial reimbursement pending approval of the final contract.
6. The irrigation district was sent a contract document from ITRC. Technically, the contract was between the district and the Cal Poly Foundation. This is a key aspect to the program – by contracting directly between Cal Poly and the irrigation districts, the typical state paperwork and processing lag times were eliminated.
7. The contract document was signed by the district and returned to ITRC. The project received final approval.
8. Payment of 50% of the estimated incentive grant payment is made after completion of construction and full operations. Copies of all invoices, service contracts, personnel time records, and other relevant information to prove the final installation of the project are required.
9. The final grant payment is made after verification of the project's actual peak period demand reduction. This generally requires one full peak period of operation (June through September) after construction and operation.

In general, the Category 1 application process required a shortened but typical engineering application that provided historical information on peak electricity consumption, a plan for reduction of the peak load, a cost analysis, and agreement for verification. Very few problems were encountered in the program administration.

There were some challenges in determining the proper verification techniques, and in deciding exactly how to compute the eligible kilowatts. For example, if a pump is only operated 5 hours per year during the peak period it cannot receive the same rebate as one operating several hundred hours. Many pumps were not equipped with time-of-use meters to establish a historical basis of peak load usage. Also, 2001 was a dry year, meaning that irrigation districts would pump more during 2001 than they had during previous wetter years. Therefore, if one only looks at historical records one can lose opportunities for savings during a dry year.

Category 2: This category has 2 components:

1. Pump efficiency testing – Financially, this is a simple rebate program. Pump testers are required to follow specified pump efficiency testing requirements, and then submit properly completed paperwork for a rebate. No prior approval by the Grant Administrator to the Pump Owner/Operator is needed. Pump testing has been conducted by irrigation district employees, consulting engineers, and individuals and companies that specialize in pump testing. The paperwork requires completion of a form, as well as photo verification of the actual point of flow rate measurement. This should have been a very simple program, but many of the pump testers were unaccustomed to following rigid pump efficiency testing requirements and were also not used to presenting

their results in a specified manner. Therefore, this category was the most problematic in the entire program. It is discussed in more detail later.

2. Pump repair/retrofit – Prior approval is required for these rebates. Approval requires documentation of certain items, including computations showing the potential rebate, results of pump tests, and/or verification of historical electric power usage. Three options are available for computing the rebate – all of which are limited to a payment of 65% of the repair cost. A very simple EXCEL spreadsheet is available on the ITRC web page, which automatically computes the best rebate option. Table 1 below shows that Options A and C do not require pump tests. Option C assumes that there will be a 25% reduction in kWh due to the pump repair. About 70% of the rebates have fallen under Option C. None of the rebates used Option A. 24% of the rebates were limited by the 65% cost share limit.

Table 1. Data Requirements for Various Pump Repair Rebate Options.

Data Needed	Data Requirements for Each Rebate Option		
	A	B	C
Hours of Peak operation during the summer	*	*	
Pre-repair kW	*	*	*
Post-repair kW	*	*	
Pre-repair pump efficiency		*	
Post-repair pump efficiency		*	
Annual hours of operation			*

Grants are made for pump repairs, pump bowl/impeller lining, motor or pump replacement and other actions to improve pump efficiency (not to include motor rewinding, unless it is necessary for proper operation of a variable frequency drive [VFD] control). Also, well cleaning that reduces draw down and removal/replacement of valves and fittings with high-pressure losses will be considered. To qualify for the incentive for motor replacement the new motor has to be rated "High Efficiency Premium".

Category 3 - Advanced Metering and Telemetry. This program has been extremely simple to implement and verify. It does not require documentation of actual load shifting. It merely requires verification of a contract between the irrigation district and the ISO, and the installation of the equipment.

Overall. All the programs have a built-in "reasonableness" economic safeguard – the cost sharing. The irrigation districts must pay at least 35% of the cost for all Category 1 and pump repair expenses.

PROGRAM RESULTS

The results as of April 2002 are found in Table 2, reflecting encumbered grant funding and peak energy reductions, rather than actual savings to date.

Table 2. SB 5x Results for Irrigation District as of April 2002.

Category	Encumbered $, million	Encumbered kW Peak Energy Reduction	$/ peak kW Reduction	Annual kWh Reduction
1 – High Efficiency Electrical Equipment/Other Overall Conservation Efforts	1.9	11,000	173	
2 – Pump Efficiency Testing and Pump Retrofit/Repair	2.3			16,000,000
3 – Advanced Metering and Telemetry	1.0	43,700	23	
Totals	5.2	54,700		16,000,000

It is clear that the most cost-effective category is the advanced metering and telemetry. This is the most simple for districts to implement, assuming that they have sufficient pumping capacity and control systems.

While the pump efficiency testing and retrofit/repair category may not result in any peak energy savings, the annual energy savings are large. Blaine Hanson of Univ. of California extension (personal comm.) has documented that typical agricultural pump repairs often do not save power. This is because farmers are often able to pump more water with rebuilt pumps, but they do not reduce the hours of pumping after a pump is rebuilt – they just pump more volume per year. But irrigation district pumps are not operated the same as on-farm pumps. With irrigation districts, there is generally a specified volume of water that must be pumped per year. Therefore, improving pumping plant efficiency truly saves energy in irrigation districts.

PUMP TESTING

Quality Control for Pump Testing

The Category 2 – Pump Testing program component has required the most interaction with participants from an administrative point of view. In particular, several companies and individuals with many years of experience felt that inexperienced testers would not be able to competently test pumps. ITRC was concerned about the quality of pump testing, regardless of who did the testing. This program offered the opportunity to "raise the bar" of pump testing, which was approached in two ways:

1. Pump test training. ITRC developed a 2-day class that has been offered twice. Another is scheduled for the end of summer 2002. The class includes classroom and laboratory activities that focus on safety, obtaining the input kW and power factor, and measuring the flow rates. A complete training manual accompanies the class. While this class is not mandatory, it has been attended by almost all of the pump testers. The class ends with an exam. ITRC and CEC only acknowledge if a person has passed the exam or not; we do not "certify the pump tester". Both inexperienced and experienced pump testers passed the exam; likewise, both inexperienced and experienced pump testers failed the exam.
2. Development of strong requirements for pump testing and reporting. Working with experienced pump testers, criteria were developed for the testing of flow rates in pipelines. Specifically, we developed criteria for various upstream conditions (check valve, elbow, etc.) and stated that within a certain distance downstream no test would be acceptable; within another distance range, 2 transects would be needed with a Collins tube or Hall tube or acoustical device; beyond that range, 1 velocity transect would be sufficient. Applications for rebates must be accompanied by photos of the test section, and by the field data.

Pump Test Results

As of March 2002, 554 pumps had been tested in 22 irrigation districts. The average overall pumping plant efficiency (including motor and impeller, but neglecting any column losses [which are typically small for irrigation district pumps]) was 59%. When weighted by horsepower, the average weighted efficiency was 67% – quite high.

Figure 1 shows the range of reported overall pumping plant efficiencies (OPPE). Some of the first tests results were unrealistically high – certainly indicating the need for the improved pumping standards and training that were incorporated by the end of the summer 2001.

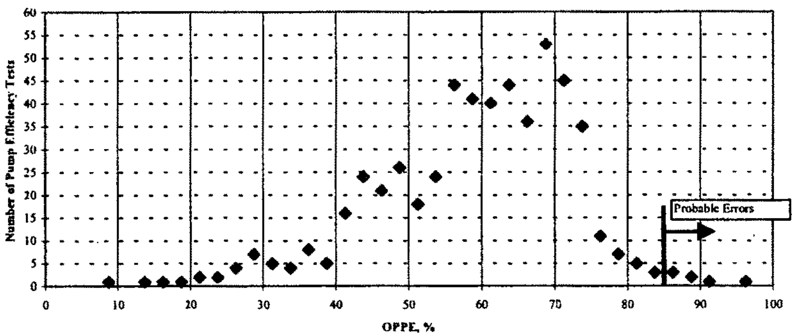

Fig. 1. Frequency of Distribution of Tests by Overall Pumping Plant Efficiency (OPPE).

Figure 2 displays the range of pump sizes that were tested. While the average kW was 125, it can be seen that some of the districts have very large pumps, and there is also an abundance of small pumps.

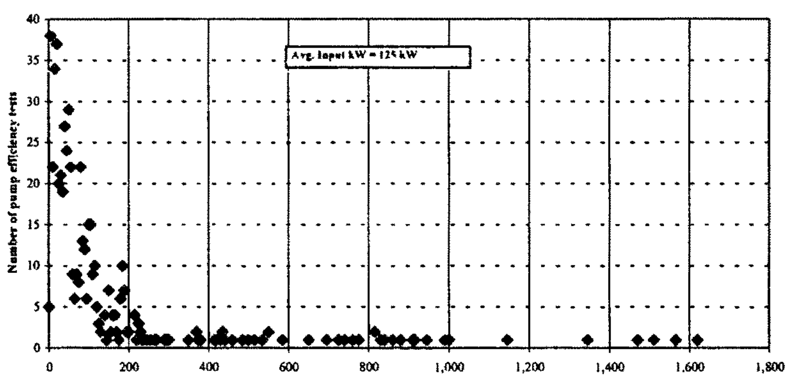

Fig. 2. Number of Pump Efficiency Tests Per Range of Input Kilowatts.

Five hundred nineteen (519) of the 554 pumps tested had less than 75% OPPE – a number we estimate is a reasonable goal for OPPE. If all of these 519 pumps were repaired and brought up to 75% OPPE, a net savings of 9,030 kW would be achieved – assuming no change in flow rate or in total dynamic head (TDH). The distribution of potential energy savings for these 519 pumps is shown in Figure 3. Figure 3 does not show 1 pump each of 81, 102, 141, 144, 150, 177, 186, 230, 353, and 296 kW.

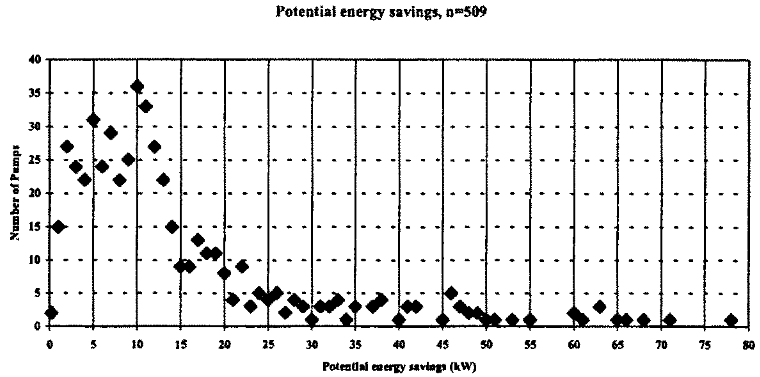

Fig. 3. Potential Energy Savings Resulting from Pump Repairs on the Smallest 509 of the 519 Pumps of Less Than 80 kW.

FINAL NOTES

The program was launched quickly and has resulted in major reductions in peak electrical load consumption by irrigation districts. The actual load reduction exceeded the expectations. However, there was less dollar demand (less participation) than expected. The lower-than-anticipated participation is probably due to these factors:

1. The short lead time for the program was unrealistic for many districts. Some districts have projects with considerable potential for load shifting, but the projects would require construction permits, decisions by the district Boards of Directors, design, etc.
2. The 65% cost sharing is substantial, but so is the remaining 35% cost sharing. The 35% is more than some districts can afford, even if the projects have a 3- or 5-year payback. Prices for many agricultural commodities are at record or near-record lows. Farmers and districts often only invest in projects with immediate or one-year paybacks.
3. The Category 1 projects required innovative solutions, and for some districts the innovations could not be conceptualized or appreciated.
4. Some irrigation districts that could have participated receive extremely inexpensive power from the Western Area Power Administration (WAPA), so there was little apparent incentive to participate.

Districts that participated in Category 1 projects were quite enthusiastic. The electricity bills for these districts were typically substantial. This program provided a relatively inexpensive path to achieving long-term savings through reduced power rates (because they will no longer use electricity during peak hours).

We had anticipated that districts would be able to organize farmers along pipeline or canal laterals to shut off their pumps during peak hours. This would result in removing both irrigation district and farmer pumps from the peak demand. It has high potential in areas with pumped pipeline laterals serving drip systems. This was just too difficult for districts to organize by the beginning of the 2001 summer irrigation season.

For Category 2, the new pump testing requirements have helped to improve the quality of future pump testing programs. Prior to this program, pump testers had little or no external quality control constraints.

Detailed information on this program can be obtained by accessing ITRC's web page (www.itrc.org) and then selecting the "CEC Agricultural Peak Load Reduction Program".

DESIGNING AND IMPLEMENTING A LARGE-SCALE GRANT PROGRAM FOR ENERGY EFFICIENCY

Peter Canessa[1]

ABSTRACT

This is a discussion of some of the more important issues that arose in the design and implementation of a large-scale grant program that was funded by general tax revenues and intended to encourage energy efficiency. It is shown that programs of this sort are complicated by issues that are legal, economic, practical, technical, and political in nature. Compromises in accuracy and accountability are necessitated by economics. Interpretation of often vague legislative language may hinge on specific political issues as well as individual opinions of the overall spirit and intent of the program. Grant levels may be set due to cost-based or market-based factors.

"One man's checks and balances are another man's government bureaucracy"

INTRODUCTION

The Agricultural Peak Load Reduction Program (Ag Program) is a cost-sharing, grant program sponsored by the State of California. It is authorized by legislation that is commonly referred to as "Senate Bill 5x" (SB5x). SB5x was signed by Governor Davis in April 2001. The Ag Program is active and accepting project applications until December 31, 2002. Projects accepted must be operational by May 31, 2003.

SB5x was passed as emergency legislation that was intended to address the then-critical imbalance in energy supply and demand in the state. It authorized in the range of $850 million dollars from the general fund for different programs. Although there was some overlap in the different sections of the bill, at least $75 million dollars was allocated for agricultural programs. However, as of February 2002, much of the original funding was rescinded due to a shortfall in the California state budget and the realization that the original authorization was much in excess of what could be absorbed in the field in the time allotted.

The agricultural portions of the bill are implemented through the California Energy Commission (CEC). CEC has retained two Grant Administrators (GA):

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1. The Irrigation Training and Research Center at California Polytechnic State University, San Luis Obispo (ITRC), which handles all grants for public water agencies dealing with agricultural water supply or drainage.

2. The Center for Irrigation Technology at California State University, Fresno (CIT), which handles all other applications.

This paper identifies some of the important issues involved in the design and implementation of such a large-scale grant program. It is important to realize that this process is/was as much political as practical and economical. The funding authorized was sometimes at the behest of different interest groups. In persuading legislatures to sponsor and vote for the bill, these interests had specific intentions in mind. Thus, these specific intentions, as well as statewide issues, had to be considered by CEC and GA.

The author is the Program Manager for the Center for Irrigation Technology and the paper is written from this perspective. The views set forth herein are the author's only and may not reflect those of the CEC, California State University, Fresno, or ITRC.

DEFINING THE OVERALL PROGRAM OBJECTIVE

Assuming that the law has been passed and monies actually authorized, the first step in program implementation is determining legislative intent in order to develop the program objectives. For electrical energy efficiency programs the question is whether the problem is not having enough generating capacity or not having enough fuel to power the generating plants.

If the situation is not enough (or too expensive) fuel, the objective is to reduce overall energy use. However, if the situation is not enough generating capacity, the objective is to reduce the peak demand. The latter situation is demonstrated by Figure 1, which is a typical relationship between electrical energy demand and time of day. To prevent blackouts, there must be enough generating capacity to meet the peak demand. To the extent that generating capacity is not available, system managers have to reduce the peak demand. (In Figure 1, anytime the demand curve goes above about 16.5 Relative kW Demand there is the risk of a blackout.)

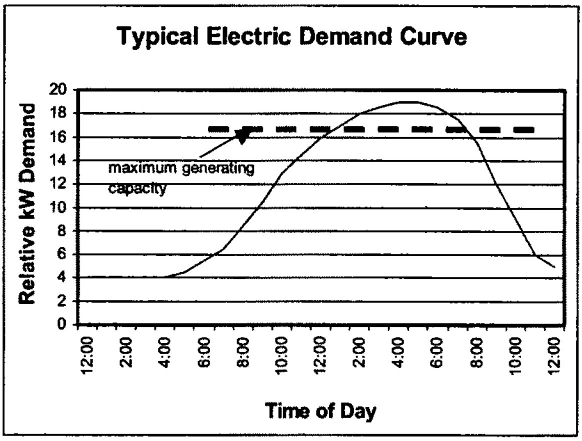

Figure 1. Typical electric demand curve

SB5x included these passages in the introduction:

> "*SECTION 1. The Legislature finds and declares as follows:*
> *(a) California is currently experiencing an energy crisis that threatens to adversely affect the economic and environmental well being of the state.*
> *(b) One of the most cost-effective, efficient, and environmentally beneficial methods of meeting the state's energy needs is to encourage the efficient use of energy.*
> *(c) The purpose of this act is to ensure the immediate implementation of energy efficiency programs in order to reduce consumption of energy and to assist in reducing the costs associated with energy demand.*
> *(d) To the maximum extent feasible, the expenditure of funds appropriated pursuant to this act shall be prioritized based upon immediate benefits in peak energy demand reduction and more efficient use of energy.*"

Note the phrase "...to encourage the efficient use of energy." in paragraph (b), the phrase "...in order to reduce the consumption of energy..." in paragraph (c), and the phrase "...based on immediate benefits in peak energy demand reduction and more efficient use of energy." in paragraph (d). All would seem to indicate an interest in reducing overall energy use. However, blackouts caused by extreme supply-demand imbalances during the peak period were the primary concern of politicians during the energy crisis. The interpretation by CEC was to use SB5x primarily to reduce peak period demand.

This decision has been a point of some contention throughout the program. As will be discussed further, focusing on peak period demand creates complexity, both for program designers and applicants. It also reduces the potential grant award in many situations. Finally, it reduces the pool of potential participants to only those with current energy use in the peak period.

DEFINING THE PEAK PERIOD

Since the decision was to focus on peak period demand, another issue was the meaning of the term "peak period". The term has significant meaning to utilities in the design of their rate schedules. "Time-of-Use" (TOU) rate schedules create tiers of different unit energy costs depending on the timing of energy use (i.e. between 12:00 PM to 6:00 PM, 6:00 PM to 8:00 PM, Monday through Friday, etc.). The terms "on-peak", "partial peak", and "off-peak" are commonly used to delineate the time periods creating the tiers. The highest unit energy rates are for on-peak period use and the lowest for off-peak use.

Depending on the utility and the time of year, there are different peak periods. For example, Southern California Edison (SCE) defines its main summer peak period as 12:00 Noon to 6:00 PM, Monday through Friday, June through October. Pacific Gas & Electric Company (PG&E) also uses 12:00 Noon to 6:00 PM but also has a rate with a peak period of from 1:00 PM to 5:00 PM. Most rates set Monday through Fridays as peak period days but one rate sets the peak days as either Monday, Wednesday, and Friday or Wednesday, Thursday, and Friday. In all cases the peak months for PG&E are May through October.

The CEC, in a predecessor peak load reduction program, defined the peak period as 12:00 Noon to 6:00 PM, Monday through Friday (except holidays), in the months June through September. This definition was continued for the Ag Program.

However, holding the peak period to the months June through September would mean that some segments of agriculture, specifically some types of processors, would not be able to take advantage of the program. For example, cotton gins do not start operations until late September into October. Thus, they would have little on-peak energy use under the original definition.

(It is interesting that in late January 2002, after much lobbying, the decision was made by CEC to extend the peak period into October. This only occurred because several cotton gins implemented projects that were verified to save substantial amounts of energy- and then made the effort to educate CEC as to that fact. However, the grant awards for October are significantly less - $50 per kW saved in October versus $250 per kW saved in the June through September period.)

IDENTIFYING THE TARGET AUDIENCE

Another important factor in the objective is determining the target audience. The Ag Program is an effort to implement four specific sections of the legislation. Each of these sections contained the word "agricultural" (e.g.. "*...($10,000,000) shall be used for the California Pump Energy Program to facilitate the efficiency testing of existing agricultural water pumps...*"). The question is: what is the exact meaning of the term "agriculture"? That is, exactly who is eligible to apply for the available grant money?

Utility accounts are classified by use for an important reason; different ratepayer classes (residential, industrial, commercial, and agriculture) have different types of rate schedules and pay different unit energy and demand costs. Generally, the utilities restrict the use of the agricultural class to those in <u>primary commodity production</u> activities (e.g. farms, dairies, nurseries). The large utilities also offer energy efficiency programs. However, usually when they offer a program for "agriculture" the only eligible accounts are those strictly classified as agricultural.

For the Ag Program the term "agriculture" was interpreted to mean anyone with an agricultural account as well as anyone who is a <u>primary handler</u> of an agricultural commodity. Thus, grants are available to such businesses as food processors, refrigerated warehouses, dehydrators, cotton gins, breweries, and wineries as well as traditional production agriculture.

A political implication of this decision is that it opened the Ag Program to participation by many, very large corporations. These corporations generally can afford the engineering and management effort needed to participate, as well as the efforts needed to remain aware of programs of this sort. Thus, they would be considered much more likely to participate in this type of program and receive the benefits available.

However, the specific intent by several important legislatures who voted for SB5x was that small farmers would benefit from the Ag Programs as much as possible. It has been a continuing issue as to how to ensure significant participation in the Ag Program by small farmers.

CHOOSING THE SPECIFIC PROGRAM COMPONENTS

Energy efficiency programs are generally implemented on three levels to energy users. These levels can be termed "information", "prescriptive", and "custom".

Information program components are designed to make people aware of particular problems and/or solutions. Providing pump tests are an example of a program at this level. Developing and distributing informational brochures, simple analytical tools, billboards, and mass media advertising are other aspects.

A prescriptive program component provides automatic reimbursement for project implementation based on verified purchase and installation of specified equipment. That is, the program prescribes what is eligible for participation. The reimbursement amount will be based on a standard engineering analysis, differentiated for region and other aspects as required.

An example of a prescriptive component would be retrofit of low-pressure sprinkler nozzles to standard high-pressure, field sprinkler systems. Here, the applicant is reimbursed at a set amount per nozzle installed. Other examples of prescriptive-type projects are high efficiency lighting, variable frequency drives installed in dairy milking systems, and premium high efficiency motors.

Prescriptive programs have the lowest administrative costs since the application, the award process, and the grant calculation are standardized. The prescriptive level will be particularly applicable to targeted technologies where energy savings can be estimated on an average-condition basis due to the (expected) large number of program applicants. Other important aspects are that the grant amount is based on a technology-specific metric ("a nozzle", "a ton of refrigeration") and the grant is made after purchase and installation.

The custom program component is most often implemented in the form of a "performance contract". In this situation, individual projects are proposed to the granting agency based on current or emerging technologies. Standard engineering analyses will indicate the feasibility and estimated energy savings of the project. A contract will be written if a project is accepted. Under the contract the applicant agrees to implement the project and the granting program agrees to provide a grant based on verified savings in energy. There may be a minimum project size (in terms of anticipated energy conservation) for these types of projects and project proponents will bear some of the engineering and verification costs.

The performance contract is the most flexible level but also the most expensive and complex to implement. Individual analyses must be performed for each application. There must be individual verification of energy savings and the means of verification may differ widely for the same type of technology implemented in different facilities.

Retrofit or New Construction?

In addition to the three main types of program components, there is the question whether to provide grants for retrofits of existing facilities, high efficiency design of new facilities, or both. It is much simpler to implement energy efficiency programs based on retrofit of existing equipment since before and after-project energy use comparisons can be made. With new facilities where there are no design standards there is no "before" to compare to an "after".

"New construction" programs are very hard to implement for agriculture due to the absence of any legal efficiency standards for design and construction of most agricultural facilities. This is in contrast to commercial and residential building design where there are minimum energy efficiency standards that have to be met as a matter of law.

The Ag Program was designed as a mix of information and performance contract, and for retrofit of existing facilities only. Information is provided by subsidized pump efficiency tests. All the other participation categories require a project application and the development of a performance contract, which may include a plan for verification of actual energy savings.

THE GRANT AMOUNT

Given that reducing peak period energy use is the objective of the Ag Program, the question is how much to "pay" for that reduction. That is, what level of incentive is required to achieve the Ag Program objectives? There are many factors to consider when setting the grant award. These include whether or not the grant should be on a cost-sharing basis, the criteria to be used to calculate the grant; and whether the grant should be market-based or cost-based.

This language appears in the bill [the underlines are the author's]:

> *"...[guidelines shall establish]...<u>matching fund criteria</u> to...ensure that entities eligible to receive funds...<u>pay an appropriate share of the cost</u> of acquiring or installing...measures to achieve... energy conservation..."*

Thus, grants awarded by the Ag Program would have to be on a cost-sharing basis.

Market-based incentives depend on what it takes to achieve the objective. For example, if the grant is to be tied to a percentage of the project cost (since cost-sharing is mandated) the question is whether to set the incentive at 25%, 50%, 75%, etc., of the project cost. What level is needed to achieve participation in the Ag Program (and achieve the objectives of the legislation)?

The maximum grant under this Program is 65% of the project cost. This level was set subjectively after discussions with interested parties, including legislative staff, industry lobbyists, CEC staff, and GA staff. Basically 50% was felt to be too little and 75% was felt to be too much.

On the other hand, during the California energy crisis, there was an issue of whether or not there was a true shortage of energy, or just a very high market price (manipulated or not) for that energy. Thus, the question for the state was whether to go to the market and buy the resource in order to develop more supply, or pay for conservation and reduce demand. Given that tax money would be used for either it would seem logical that the incentive level should not be higher than the cost to buy the resource on the open market.

Thus, although the maximum grant could be 65% of the project cost, the initial grant calculation would be based on the actual energy savings. The actual grant would be the smaller of the two. The metric chosen for the cost-based grant was kiloWatts (kW) of load reduced during the peak period. Note that the metric chosen matches the overall objective of the Ag Program.

Given the chosen metric, how much money would be paid for a kW of reduced peak period demand? The analysis was along these lines:

1. There are about 500 peak period hours per year as defined by the Ag Program. Thus, a kW of peak load would generate about 500 kiloWatt-hours (kWh) savings through the critical summer months.

2. The adopted measure should be in place a minimum of two years, thus, in the range of 1000 kWh, a megaWatt-hour (mWh), would be saved for each proven kW of reduced peak period load.

3. The average cost of a peak-period mWh during the critical months of the energy crisis was above $500/mWh and was projected to be in the range of $300-500/mWh during the summer of 2001.

4. It was projected that the cost of energy would be going down with time.

Thus, a tiered rate of award was adopted; paying more for projects that would be operational sooner. The actual grant rates were:

- $350/kW for projects that were operational by July 30, 2001

- $300/kW for projects that were operational by September 30, 2001

- $250/kW for projects operational between September 30, 2001 and May 31, 2003.

As a final note, power is now being bought (as of February 2002) for in the range of $30/mWh. There are some that question the current need for the Ag Program on the basis that the needed energy can be bought for less on the open market than it costs to conserve. This is much more a valid concern given that SB5x did not mandate long term persistence of any conservation project. That is, the Ag Program does not "care" if the conservation activity exists past May 31, 2004.

CALCULATION OF ENERGY SAVINGS

Probably the most onerous aspect of the Ag Program from the GA point of view is the process adopted for calculation of energy savings. The decision was made to calculate energy savings based on the average day throughout the entire peak period, not just a day when the project facility was in operation. This was termed "prorating" the existing demand.

The exact equation published in the Ag Program Description is:

$$kW_{reduced} = (\text{kWh Usage}_{\text{pre-project}} - \text{kWh Usage}_{\text{post-project}}) / 6 \text{ hours} \quad [1]$$

Where:

$kW_{reduced}$ = the Reduced Electrical Demand used for calculating the grant.

kWh Usage = average peak-period daily consumption of kilowatt-hours of affected system(s) during the entire peak period with typical operating conditions.

As a practical example, consider an irrigation pump with a 100 kW load, operating 24 hours per day in June and July, but completely shut off in August and September. This equates to about 25,800 kWh (21.5 peak days/month x 6 peak hours/day x 2 months x 100 kW) of on-peak energy use. Assume that the project results in this pump operating completely off-peak - that is, it does not operate between 12:00 noon and 6:00 PM. The grant award would be based on 50 kW, not the full 100 kW. This is because there are approximately 516 peak hours in the peak period (as defined by the Ag Program). The project reduces peak period energy use by 25,800 kWh. This equates to 50 kW using the equation noted above (25,800 kWh / 516 hours = 50 kW).

This is a simple example. Difficulties arise when projects are proposed in multi-load facilities, or where time-of-use energy use information is not available (i.e. current on-peak energy use cannot be accurately determined). The cost of processing a grant increases as the difficulty of verifying on-peak energy use for existing facilities increases.

Note also that this can be a relatively complex process that is difficult for the layperson to comprehend. Thus, a large percentage of applications contain incorrect calculations and must be revised. A certain level of public relations problem is created by this complexity. In effect, the GA becomes another "government bureaucracy" to many agriculturalists.

The economic effects of proration extend to the Ag Program's success, the GA, and the participant. The Ag Program gets less credit for connected load being taken off-peak, it adversely impacts the economics of administering the program, and the participant is awarded less money than expected.

The philosophy behind this method of determining the potential grant is that a reduction of a load that occurs throughout the peak period is more "valuable" to the state than for one that just occurs at certain times in the peak period. Thus, the 100 kW irrigation pump that runs all four months of the peak period is more "valuable" to the state than a pump that only runs two months of the peak period-assuming that both loads can be avoided.

Proration has also been a point of contention. This author points out that no matter how little a load runs during the peak period, when it is on it contributes to the risk of a blackout- that is, the system has to be able to supply that demand. Note also that in many cases it costs no more to remove a 100% on-peak load than a load that is running only 50% of the peak period.

VERIFICATION OF ENERGY SAVINGS

Given the fact that this was primarily to be a performance contract program, the question is raised as to what it takes to verify that energy efficiency was actually achieved. There are at least three types of problems that the Ag Program faces with verification:

1. Cost of verification - If the grant is $5000, should the program require the same accuracy of verification as a project where the grant is $50,000? Can the applicant afford the cost of verification?

2. Actual measurements - Isolating the effects of a change to one sub-system in a facility with multiple loads can be impossible in certain situations.

3. Natural changes in year-to-year energy use due to the nature of agricultural operations. These changes may be due to climate directly (as with heating and air-conditioning) or indirectly (high or low rainfall years leading to more or less well pumping or higher or lower crop yields). Market forces may be at work also. The question is filtering out these variations so that the verification involves an "apples-to-apples" comparison.

The International Performance Measurement and Verification Protocol is a commonly used standard for verification of energy savings. It describes four types of verification:

1. Engineering or statistical analysis of the projects to be installed and a stipulated proxy for determining actual energy savings.

2. Long term and direct measurement of the installed project.

3. Inspection of overall energy use of a facility before and after a project is installed.

4. Modeling of the installed project, calibrated by specific measurements of facility performance.

The Ag Program utilizes all of these. The problem for the Ag Program is that the vast majority of applicants are unaware of this protocol or the options. Many times the requirement for submittal of a verification plan is ignored by the applicant and Ag Program staff has to develop the plan. Thus, program implementation costs are increased.

OVERALL PROGRAM ECONOMICS

Frankly speaking, it takes money to spend money. That is, there is a certain amount of overhead to a grant program. Thus, funding authorizations necessarily recognize that a certain amount of the funds will be spent by the implementing organization. However, language in SB5x directed that CEC administrative costs should be no more than 2 ½ % of funds expended and that generally, not less than 85 % of funds expended overall should be for direct rebates, buy-downs, loans, or other incentives. Basically the bill's intent was that only 15% of the money authorized was to be used to implement the bill.

Also, CEC was specifically precluded from hiring additional staff to implement the bill. Thus, an early CEC decision was to identify and contract with Grant Administrators (GA) to actually implement the Ag Program- and the Grant Administrators would have to perform the function for 12 ½% overhead.

The question for the GA in deciding to provide services was whether or not they could perform for the money available. Note also that the implementation was to be done on an essentially commission basis. That is, only if a grant were authorized would the 12 ½% overhead be paid to the GA. The practical impact on the GA is that they would have to estimate how much staff to hire, office space to rent, computers, filing cabinets, etc. without actually knowing how much money they would be given to do so.

To put this situation in concrete terms, consider a grant for $50,000. Under the strict terms of the bill, some $6,250 (12 ½%) would be available to the GA for processing that grant. This includes "overhead" costs of program development and implementation, plus salaries, office rent, legal advice, accounting services, office equipment, and reporting, plus the grant-specific costs of review, data basing, and verification.

Now, the economics of a $50,000 grant probably "work". That is, the $6,250 that is paid to the GA may cover all costs of developing and implementing the Ag Program while ensuring that the individual grant is reasonable and verified. However, consider a $5,000 grant, which only provides for a $625 payment to the GA. The economics of this grant are questionable at best.

Note two aspects of the Ag Program previously discussed which affect the economics:

1. Prorating the peak demand, which tends to reduce the award – thus, reducing the commission amount.

2. Complexity of the process, especially development of viable energy-savings verification plans, which requires more time on the part of GA staff.

One part of the spirit and intent of SB5x is that money be available for the small agriculturalist as well as the large. Thus, this program has provided grants as large as $300,000 and as small as $113.32. It is hoped that the "commission" for the large grants will be able to offset the costs of servicing the small grants. (As of February 5, 2002, the Ag Program had approved approximately 255 grants, 135 of which were less than $5,000, 45 of which were between $5,000 and $10,000, and only 9 of which were greater than $100,000.)

CONCLUSION

This has been a discussion of some of the more important aspects of designing and implementing a large-scale, grant program for energy efficiency. Importantly, the funds for this particular program come from general tax revenues. As was seen, the process is complicated by issues that are legal, economic, practical, technical, and political in nature. Compromises in accuracy and accountability are necessitated by economics. Interpretation of often vague legislative language may hinge on specific political issues as well as perceptions regarding the overall spirit and intent of the entire bill.

To the extent that the application and verification procedures are perceived to be onerous to the participant, the Ag Program is another example of government bureaucracy. On the other hand, it is certain that some segments of society view the Ag Program as another government give-away.

LOAD SHIFTING OPPORTUNITITES AVAILABLE TO CALIFORNIA WATER AGENCIES

Hilary Armstrong Reinhard[1]
Michael J. Day, P.E.[2]

ABSTRACT

In response to the increase in the cost of electricity and the shortage of supply of energy, some agricultural water agencies in California's Central Valley have modified their distribution systems and operations to increase their ability to shift energy usage to hours when the cost is lower and power is more available. This paper provides suggestions for opportunities available to agencies for load shifting and energy conservation.

During 1999 and 2000, the California Legislature passed three important energy conservation bills, AB 970, SB 5X and AB 29X. Along with other energy initiatives, these bills supplied funding for projects that reduce the amount of demand on the California electric grid. This funding assists local agencies to pay for the capital costs associated with distribution system modifications. The load shifting program allows the agency to save money on its power bills each year by minimizing energy usage during peak hours, when the cost of energy is much greater.

One of the most cost effective load shifting projects involves increasing the amount of water storage the agency has available. This can be accomplished in a variety of ways, including the construction of new reservoirs, increasing the volume of existing reservoirs, and reducing the amount of constrictions that cause head loss in the system.

To determine the feasibility of a project, several factors must be taken into consideration. These factors include: the volume of water deliveries the agency provides during a maximum water usage day, volume of existing water storage, the feasibility of constructing additional water storage facilities, the physical layout and topography of the district, the capacities of pump stations, and the possibility of replacing constrictions that cause significant head loss in the distribution system such as undersized culverts.

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One example of an agency that has effectively utilized these principles to decrease peak energy used is Lost Hills Water District. This district increased its water storage capacity by constructing a 50 acre-ft reservoir. The construction of the new reservoir provided enough storage to enable the district to curtail all pumping at its Sarad Pump Station between the hours of noon and 6 p.m. on non-holiday weekdays from May 1st to October 31st.

INTRODUCTION

During the past year agricultural water agencies in California have been faced with numerous challenges. Two of the most significant issues have been the increasing cost and decreasing reliability of electricity. As a result, these agencies have been forced to allocate more money for electricity bills and deal with the uncertainty that electricity may not be available on high demand days.

Fortunately, in response to the challenges facing the State's electrical grid and power supply, the State Legislature has provided funding to assist water agencies to reduce their electricity usage during peak hours[3]. During 2000, the legislature passed three bills that provided funding for energy efficiency and load shifting projects; AB 970, SB 5X and AB 29X. Several agricultural water agencies in Central California utilized funding from these programs to implement successful projects that have removed demand from peak hours. In addition to providing relief to the State's overburdened power grid, these agencies have also realized substantial savings on their own electricity bills. One example of a load shifting project is the construction of a water storage reservoir that is filled during off-peak hours, and drained during peak hours. Storage reservoir projects do not decrease the total amount of electricity the agency uses; rather they decrease the cost of the electricity by shifting it to off-peak hours. To be effective, the load shifting program must not negatively impact water deliveries.

Load shifting projects take advantage of the reduced electrical rates that utilities generally offer during off-peak hours. For example, Pacific Gas and Electric Company's (PG&E) AG-5B[4] rate, a common rate for irrigation pumps, charges $0.04088 per kWh for off-peak electricity and $0.14294 per kWh during peak hours. In addition to the savings per kWh, these projects also decrease the peak demand charges. When these savings are considered, many of these projects pay for themselves within a matter of a few years.

[3] Peak hours defined as noon to 6:00 p.m. on non-holiday weekdays.
[4] Electricity rates information taken from PG&E tariffs dated January 1, 2002

PROJECT SELECTION

Establishing the Energy/Water Connection

To estimate the amount of energy use that can potentially be shifted as a result of the project, the amount of energy use per unit of water pumped must first be established. If the pumping plant consists of a single pump, information from a pump efficiency test can be used. In many cases, though, the lift stations consist of multiple pumps that operate in combination to deliver varying amounts of flow. Frequently the information available is insufficient to determine when each pump operates. Thus, the amount of energy used per unit of water pumped for the entire pump station is often difficult to determine. An alternative to using pump test results is to utilize pump station flow meter records and electric use information from utility bills. The total amount of water pumped during the electricity billing cycle can be divided by the energy usage during the billing cycle to calculate energy usage per unit of water pumped. This method may have some error if the time of the electric meter reading does not coincide exactly with the flow meter reading time. However, averaging the information for several time periods can reduce the error to acceptable levels.

When available, daily or hourly water delivery records should be used to determine the feasibility of a project. Adjustments to the records may need to be made if future water demands are expected to vary from past records. Flow information can be combined with the average energy usage per unit of water pumped to create a computer spreadsheet model of the district's daily or hourly delivery demands and electricity usage.

A storage reservoir project will allow pumping to be curtailed during peak hours, followed by increased pumping during off-peak hours. The maximum flow rate that can be curtailed during peak hours is equal to the volume of useable storage added by the project divided by the duration of the curtailment (usually six hours). For example, 50 acre-feet of useable storage divided by six peak hours gives a maximum curtailment of 8.33 acre-feet per hour, or 100.8 cubic-feet per second (cfs) [8.33 acre-feet/hour x 43,560 cubic-feet/acre-foot / 3,600 seconds/hour]. The average increase in flow rate needed to make up for the curtailment during off-peak hours is equal to the flow rate actually curtailed multiplied by the ratio of on-peak hours to off-peak hours. For example, a curtailment of 100.8 cfs for six hours requires 33.6 cfs (110.8 x 6/18) of increased pumping during the eighteen off-peak pumping hours.

It should be noted that additional storage in the regulation reservoir may be needed for purposes other than load shifting. For instance, storage may be required for regulating water supply and customer demand variations; this must be factored into the analysis.

The spreadsheet model can be used to calculate the post project peak and off-peak energy usage and demands. The savings on electricity bills is then computed by comparing the post-project energy usage pattern with the pre-project pattern. The savings can then be compared with the estimated project cost to determine its cost-effectiveness.

Considerations for Water Storage Load Shifting Projects

Water storage projects for load shifting are well suited for agencies with large elevation differences and/or high pipe friction losses between the water source and its destination. Additionally, agencies that obtain a large quantity of water from deep wells can gain significant benefits from a water storage program, provided the wells can be turned on and off daily to avoid peak pumping. In general, the more energy it takes to move the water from its source into the distribution system, the more effective a water storage project is going to be for load shifting.

Location: The topography of the district should be considered when determining the feasibility of adding storage. Ideally, the storage should be placed at a location with sufficient elevation to allow the distribution system to be fed by gravity during peak hours, thereby significantly reducing or eliminating the average peak demand of the district. In some cases, a low head lift pump to return stored water to the distribution system may be justified, but this adds to the project's capital cost and reduces energy cost savings.

Placing storage at the last reservoir in a chain of lift stations allows the greatest amount of curtailment for the least amount of additional storage. This is because curtailing pumping at the last lift station allows the previous lift stations to also be curtailed.

The reservoir size should not exceed the amount of water required to shift load for the highest water use day in that service area, as excess capacity will not aid load shifting.

Determining Optimum Storage Capacity: The amount of daily deliveries within an agency's distribution system typically fluctuates during the summer season. As these deliveries fluctuate, the amount of water that must be pumped into the system also varies. A reservoir that is sized to allow an agency to deliver water to its customers on its highest demand days and shift the entire load to off-peak hours, will have unutilized storage space during periods of lower demands. Thus, there is a diminishing return on the investment in additional storage. The reservoir should be sized to maximize the project's return on investment. Figure 1 is a typical graph showing the amount of savings achieved for each incremental increase in reservoir size. As the reservoir size increases, the corresponding additional savings in energy bills begins to decreases due to under utilization on days with lower water demand. Eventually, no additional savings can be achieved by increasing storage capacity.

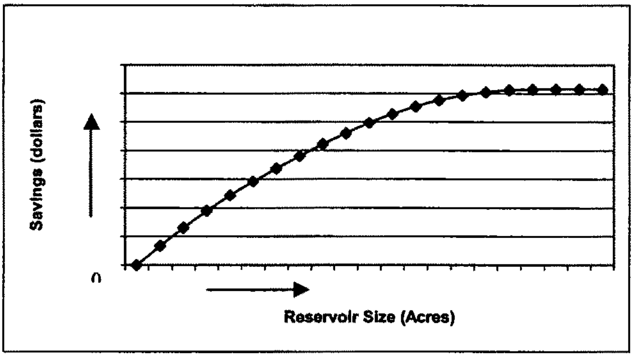

Figure 1. Typical Energy Savings vs. Additional Reservoir Capacity

A comparison of the energy bill savings to the cost of for each additional acre-foot of storage capacity added to the reservoir should be performed to determine the optimum reservoir size.

Supply System Capacity: The post-project operation of the supply system requires the system to have enough capacity to deliver the water to the reservoir during off-peak hours. Filling the reservoir during off-peak hours increases the flow rate required by the system. Accurate as-built drawings and information are essential for evaluating the capacity of the supply system.

If supply system modifications are not necessary, the additional flow required to fill the reservoir will increase the velocity of the water and, therefore, cause the pumps to operate at a different point on their pump curves. The additional friction caused by the increased velocity causes an increase in the Total Dynamic Head (TDH) required by the pumping plant; this increases the energy used per acre-foot. Figure 2 shows the relationship between the flow rate and the kWh per acre-foot.

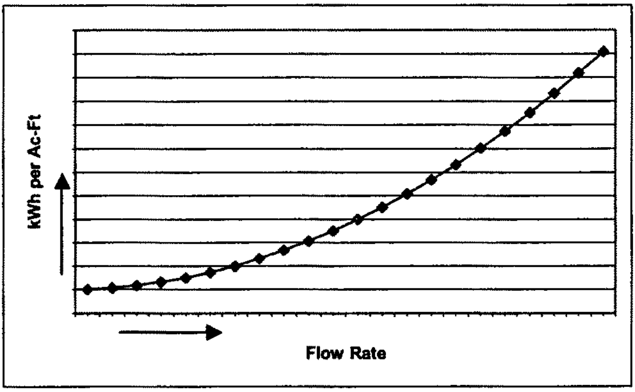

Figure 2. kWh per Acre-Ft vs. Flow Rate

This change is generally small, but becomes more substantial in systems where water is pumped long distances, pipes are undersized for the system requirements, or flow rate is greatly increased. A small increase in TDH during off-peak periods should not create a significant impact on the profitability of the project, provided that the delivery system has sufficient capacity.

Delivery System Capacity: In addition to the capacity increases necessary in the supply system to the reservoir, modifications may also be necessary to the delivery system to ensure that water can be drained from the reservoir during peak hours and used to service customers. Although the reservoir is filled in eighteen hours, it will be drained in six; this requires more capacity for emptying than filling. If the reservoir is located at the end of the delivery system, it may be necessary to increase the capacity of the canal or pipeline to ensure delivery of the water from the reservoir to the customers. In the case of canals, the capacity of culverts, weirs and other structures must be checked to determine if any additions or modifications are necessary.

Other Opportunities to Shift Load or Conserve Energy

Capacity Restrictions: Capacity restrictions in the water distribution system can limit the amount of storage capacity that is available to the agency. In a gravity fed system, constrictions such as undersized culverts can cause large losses of head. Head loss can cause the minimum water surface in the reservoir to be artificially high. Eliminating the constriction allows the minimum water surface in the reservoir to drop. Agencies can take advantage of existing storage capacity without constructing additional storage by eliminating points of head loss. These projects also aid in the management and flexibility of the distribution system.

Water Losses: The energy that is expended to pump water is wasted if the water is subsequently lost to beneficial use. Water is generally lost at the distribution level through seepage or by spills that are not recovered.

By lining canals and reservoirs, the amount of water lost to deep percolation can be virtually eliminated. Distribution systems located on sandy soils with high percolation rates can benefit significantly by lining. The benefits of reduced deep percolation include increased water conservation and a reduction in the amount of water that needs to be lifted. Spills also reduce the efficiency of energy use. Again, the investment of energy in water that has been pumped should be conserved. Improving the distribution system's flow control can help eliminate spillage. Although often difficult, matching customer demands and the water delivered is one of the most important methods of reducing spills. Investing in control technology increases the efficiency of the delivery system and can reduce the cost of water delivery.

Energy Losses: Energy is also lost through valves that are throttled to reduce the flow rate. Projects that conserve pumped water or reduce unnecessary pressure losses may also be considered by water agencies looking to save money on energy bills.

Pumps with standard drives that are required to deliver a wide range of flow rates frequently operate inefficiently. When the flow rates exceed the system requirements it can cause water to spill back into the source or outside of the distribution system. To avoid this, agencies should consider installing variable frequency drives (VFD). Although these drives require a large initial investment, they allow the output flow rate of the pump to match the system requirements. By matching the flow rates the spillage can be eliminated. This reduces the amount of electricity and money that is needlessly expended. Similarly, if a valve is being closed to restrict the flow rate from a pump to compensate for a large range of flow rates required, a VFD should be considered. Although VFD's reduce the efficiency of pumps at their maximum flow rate, their ability to match the system requirements without wasting energy through a partially closed valves or spillage makes VFD's more efficient in projects that require large variations in flow.

CONCLUSION

By analyzing their infrastructure, water agencies can identify and implement energy efficiency and/or load shifting projects to lower costs, while providing sufficient supplies to customers. Due to the wide variety of operating systems, layouts, and topography that exist it is impossible to discuss all of the feasible energy efficiency and load shifting projects. However, by following the principles presented in this paper, determinations can be made with respect to the most cost effective methods to meet the customers' needs while decreasing system energy costs.

ENERGY USE FOR MICROIRRIGATION

Tom Trout
Jim Gartung[1]

ABSTRACT

Microirrigation systems can operate with low pressure. Microirrigation emitters require only 7 - 15 psi. However, cleaning, regulating, and delivering the water to the emitters often consumes an additional 20 - 40 psi. A survey of 127 California Central Valley microirrigation systems showed that 50% of the systems operate with more than 35 psi pump pressure. Pressure was lost at the filter station, in the distribution system, and at the emitters. Some devices such as filters and pressure regulating valves require substantial pressure or pressure loss to operate. Extra pressure is required to irrigate undulating land. Reducing system pressure by 10 psi in a system would save about $30 per acre per year in electricity costs. It will often be economical to invest more in the system to save pressure and energy costs.

INTRODUCTION

Electric energy rates increased by about 30% in California in 2001. Cost for pumping irrigation water now exceeds 12 cents per kw-hr in most cases. Electricity shortages and the high cost of marginal supplies on the spot market induced California to offer irrigators financial incentives to reduce peak electricity demand.

Energy is used to lift water from groundwater wells to the fields and to pressurize the irrigation system. Gravity irrigation methods generally do not require pressurization and are often the lowest energy option. However, gravity systems may be less water use efficient than pressurized systems, resulting in higher well pumping costs. Most sprinkler systems require 50 - 80 psi to operate efficiently. The development of low pressure sprinklers and sprayers, largely in response to energy cost increases in the 1970s, reduced pressure requirements of moving lateral (center pivot, lateral move) sprinkler systems to 30 - 50 psi.

Microirrigation is a low pressure alternative to sprinkler systems that can efficiently and uniformly apply irrigation water. Microirrigation is used on 1.7 million acres in California and 3 million acres in the U.S. (Irrigation Journal 2001). Drip emitters require 7 - 10 psi, and most microsprayers operate well at 15

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psi. However, unlike sprinkler systems which are usually designed for minimal pressure losses between the pump and sprinkler, most microirrigation systems use 20 to 40 psi to clean, regulate, and deliver water to the emitters. Thus, pressure requirements are sometimes no lower than with low-pressure sprinkler systems. Most microirrigation systems operate at over 30 psi, and many, over 40 psi. There may be potential to reduce pressure, and thus energy, requirements of microirrigation systems through alternative equipment, design, or management. A 10 psi pressure reduction would reduce power requirements by about 36 kw-hr/ac.ft. of water pumped or about 150 million kw-hr/yr for California's 1.7 million acres of microirrigation.

Energy use in irrigation can be reduced by reducing the amount of water pumped (increased efficiency or deficit irrigation), by increasing the efficiency of the pumping plant, or by reducing the system pressure. For example, for a 100 ac. California orchard that requires 3 ft. of water annually with a well pump with a 100 ft. lift, energy costs can be reduced by about $2000 per year either by increasing the irrigation efficiency from 75% to 85% (ie: reducing the water pumped from 4 to 3.5 ac-ft/ac), by increasing the pumping plant efficiency from 60% to 68%, or by reducing system pressure requirement from 40 to 30 psi. If the water supply is surface water, the irrigation efficiency would need to be increased to 100% or the efficiency of the booster pump to 80% to gain the same savings as reducing system pressure by 10 psi.

The objective of this study is to determine the pressures used with microirrigation systems, sources of pressure losses, and ways to reduce pressure requirements.

PRESSURES IN CALIFORNIA MICROIRRIGATION SYSTEMS

Several hundred California irrigation systems have been evaluated by mobile irrigation labs using procedures developed by the Irrigation Training and Research Center (Burt et al. 2000). As part of these evaluations, system pressures are measured at several points in the irrigation system. We summarized pressure data from evaluations of 127 microirrigation systems in California's Central Valley carried out by the Irrigation Training and Research Center[2] (Cal Poly, San Luis Obispo) and Kings River Conservation District (Fresno) over the last 3 years. These systems were primarily drip systems in vineyards and microspray systems in orchards. We included only systems that irrigated more than 15 ac.

In these evaluations, pressures were recorded downstream from the pump and at several locations in the irrigation laterals (at the emitters). Measurements were often also taken downstream from the filter and at the outlet of the filter station

[2] Funding provided by U.S. Bureau of Reclamation and CA Dept of Water Resources

(inlet of the distribution system). In addition, all evaluations recorded the type of emitters and if the irrigated land was "undulating", and most recorded the type of filters and the presence and location of regulating values.

Figure 1 shows the range and distribution of system inlet pressures (downstream of the pump and, in most cases, upstream of the filter and any valves) in these 127 microirrigation systems. "Level" systems refer to those that were not considered to be irrigating undulating land. These systems are separated out because systems on undulating land often require additional pressure to overcome elevation differences. The figure shows a cumulative distribution curve, or the percentage of the total systems with pressures smaller than the x-axis value. The range of pressures is wide. About 15% of the systems operated with less than 25 psi pressure, and 35% had over 40 psi pump pressure. Many of the highest pressure systems were on undulating land, but about one-quarter of the level systems also had over 40 psi pump pressure.

High system pressures should result in improved irrigation water distribution uniformity (DU). Figure 2 shows the relationship between low quarter DU as calculated in the irrigation evaluation (Burt et al. 2000) and pump pressure for the systems. There is some decrease in both maximum and average DU as system

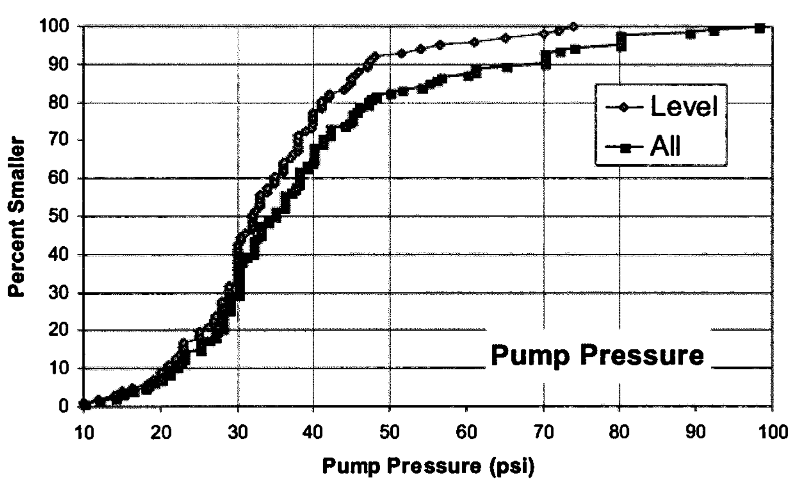

Figure 1. Cumulative distribution of pump pressures in 127 California Central Valley microirrigation systems, and for the 109 systems irrigating level ground.

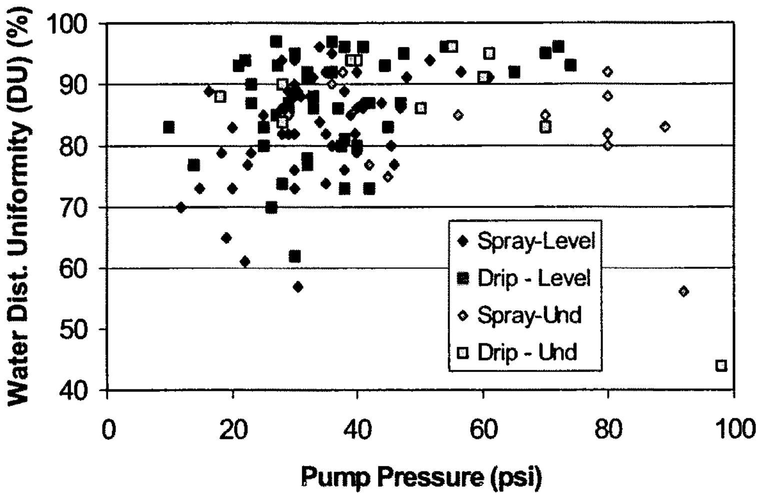

Figure 2. Measured water distribution uniformity as a function of pump pressure for 127 California microirrigation systems.

pressure drops below 30 psi, but there are systems with DU > 90% at pump pressures as low as 20 psi.

Sources of Pressure Losses

Table 1 shows pressure losses through microirrigation system components. The range of losses is large depending on the type and size of component used. High pressure losses generally result from undersized pipe and components and the use of pressure regulating valves and emitters. Pressure regulation is often required for uniform application to undulating land, but is also used to compensate for small (less expensive) distribution piping or long lateral lengths.

Filter station loss is an allowance for the filter to accumulate contaminates without frequent backflushing. Backflushing is typically set to occur when pressure drop across the filter reaches 7 psi. When the filter is clean, losses should be small, and with clean water, the allowance can be reduced. These loss values do not include minimum pressure requirements to backflush filters, which typically vary from 25 to 45 psi, depending on the type of filter.

Also shown in Table 1 are pressure losses in a "typical" well-designed microspray system on a level, 40 ac almond orchard. The system has 9 psi of pressure loss in the distribution pipes, fittings and hoses, and a 7 psi pressure "allowance" for the filter station. A similarly-designed drip system for a vineyard or row crops would require about 5 psi less pressure at the field and pump.

Table 1. Pressure losses for microirrigation systems.

System Component	Range *(psi)*	Well-designed System* Microspray *(psi)*	Drip *(psi)*
Emitter (microspray, dripper)	7-25	15	10
Lateral hose	1-5	1	1
Manifold	1-5	1	1
Sub-main pipeline	1-5	4	3
Main pipeline	1-5	3	3
Filter station piping	1-10	1	1
Filter	3-10	5	5
Press. Regulators, Chem. injectors, Control Valves, Flow Meters	0-25	1	1
Total Cumulative	15-90	31	25

* 40 ac. orchard or vineyard on level land with a sand media and screen filter.

Pressure measurements downstream from the filter station and in the laterals in the mobile lab evaluations allow estimates of where the pressure losses in those microirrigation systems occurred. Figure 3 shows losses at the filter stations of 109 of the evaluated systems for which pressure data were available. Note that these data are measured losses, and thus in most cases would include less than design allowed (maximum) losses through the filters. In 10 of 49 systems in which filter losses were noted, filter losses exceeded 7 psi. Eight systems had automatic control valves at the filter station, but only 3 of these systems had high filter station losses. Two of the systems had partially closed manual valves with losses greater than 15 psi, indicating the pump generated too much pressure for the microirrigation system.

Figure 4 shows the pressure losses in the distribution system, including the main, sub-main, and manifold pipelines and in the laterals. The curves represent losses to the emitter with the highest and lowest pressure, in both level and undulating systems. About half of the level systems exceeded 10 psi losses in the pipelines and lateral hoses, and 20 percent exceeded 17 psi losses to the lowest pressure emitter.

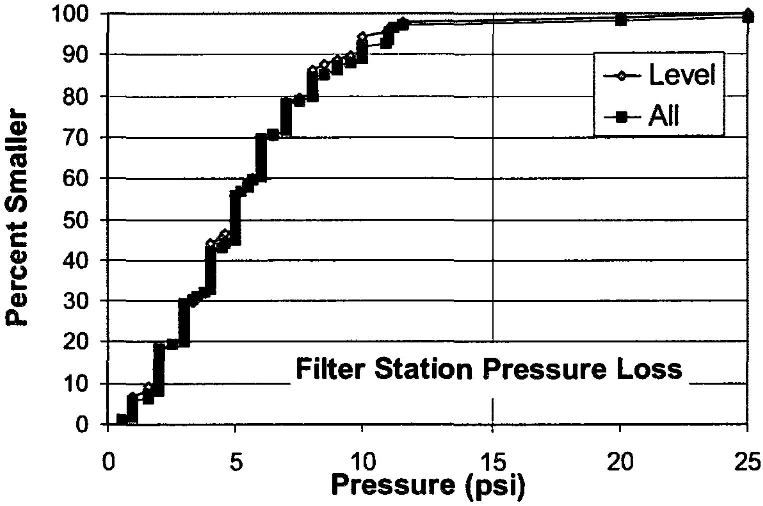

Figure 3. Cumulative distribution of pressure loss at the filter station of 109 microirrigation systems and 88 systems that irrigate level ground.

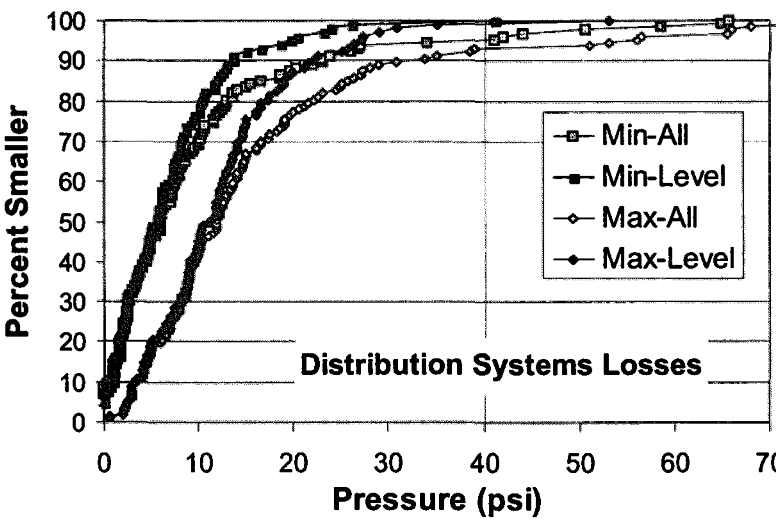

Figure 4. Cumulative distribution of pressure loss in the distribution system (between the filter station and emitters) for the highest and lowest pressure emitters for all 127 systems and 102 systems on level ground.

Losses in the distribution system can be from friction loss in the pipelines and fittings or regulating values. High friction losses indicate undersized pipelines. Thirty of the systems (21 of which were on level ground) had pressure regulation at the inlet to manifolds, and 15 systems (10 on level ground) had regulators at the lateral inlet. In addition to these regulators, 30 systems (17 on level ground) used regulated emitters. Sixteen systems (7 on level ground) had two sets of regulators inline. Regulators are often needed for good water distribution on undulating ground. On level systems, regulators are only needed to overcome the effects of friction losses in the distribution system.

Another source of pressure loss in the distribution system is plugged hose screens at the lateral inlets. On 17 systems, these screens generated over 2 psi of pressure loss, and on 6 systems, the loss was greater than 5 psi. The screen itself should generate little head loss, so these losses are assumed to result from trash caught in the screens that restrict the flow, an indication of poor system maintenance.

Figure 5 shows the difference between the minimum, average and maximum measured pressures in the laterals and the "design" emitter pressure. These values assume drip emitters require 10 psi, microsprays require 15 psi, and Dan 2000® pressure regulated microsprinklers require 22 psi. These excess pressure values did not appear to depend on whether the systems were on level or undulating ground. The pressures varied widely from 10 psi below to over 20 psi above required pressures. The minimum pressures distributed fairly evenly around the design pressures, and 75% of the average pressures exceed requirements. Lateral pressures a few psi below design pressures are acceptable, but pressures more than 5 psi below design likely result in poor water distribution.

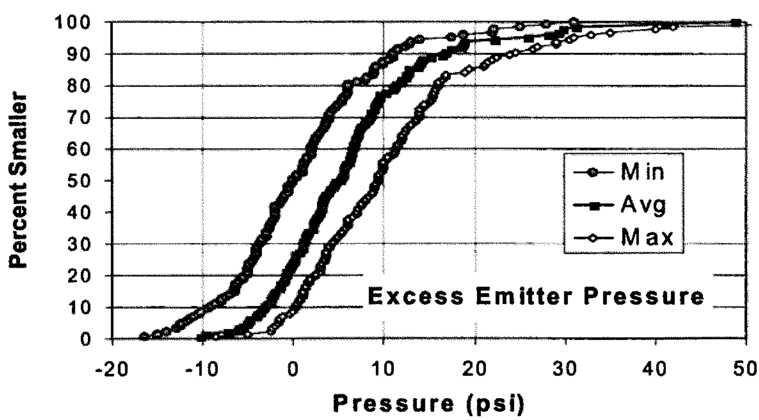

Figure 5. Cumulative distribution of minimum, average, and maximum pressures in the laterals compared to required emitter pressures for 127 microirrigation systems.

POTENTIAL FOR REDUCING SYSTEM PRESSURES

The evaluation data indicate that, although a portion of the systems operate at low to moderate pressures, there appears to be potential to reduce system pressures in many systems. Fifty-seven percent of the systems on level ground operate at greater than 30 psi. Reducing pressure to 30 psi in theses systems would save an average of 11 psi. If these data are representative and pressures could be reduced by 11 psi on 57% of the 1.7 million acres with microirrigation in CA, this would save 160 gigawatt-hr/yr of power and 64 megawatts of peak demand. The potential for reducing energy use is substantial.

There are many choices made during system design that impact pressure requirements. Some are easy to evaluate, such as the economic evaluation of higher initial cost vs. energy savings of increasing pipe sizes in the distribution system, or reducing lateral lengths by increasing the number of manifolds to avoid the need for pressure regulating emitters. For example, in the 40 ac microspray system in Table 1, the 3 psi loss in the 1000 ft long mainline could be reduced to less than1 psi by using 8" rather than 6" pipe. The 2.5 psi savings could reduce annual pumping costs by about $160 per year and increase system cost by about $800. At a 7% interest rate, the investment would pay off in 7 years - a good investment at electricity rates . Most choices that reduce energy use increase system cost, and require economic analysis to evaluate fully. Designers should make these evaluations and discuss them with the grower. Unfortunately, designers, who commonly work for equipment dealers, sometimes propose low cost systems without revealing the high energy costs in order to win a contract. Growers, even when given the economic information, sometimes choose to save initial costs in spite of higher deferred energy costs.

Some design choices that impact energy use are not easy to evaluate. Self-cleaning screen and disk filters require less backflush water than media filters, but 10 to 20 psi more pressure for effective backflushing. The choice whether to set the automatic backflush for a filter station at the standard 7 psi pressure differential or a lower level, will depend on the cleanliness of the water supply and the cost and reuse of the backflush water. Pressure regulators can improve water distribution uniformity, but require 5 to 10 psi pressure loss to operate. In some systems, good system design and maintenance can eliminate the need for regulators. Good regulation, like good filtration, reduces risks of costly failures, but come at a high price in energy costs.

Many designers allow for a safety factor when designing systems or pumping plants. Although it is easy to reduce pressures in an over-pressurized system (ie: partially close a valve), it is very difficult to increase pressures in an under-pressurized system. While growers will fault the designer if a system has inadequate pressure at the field, they may prefer to have a little extra pressure in case "things change". Thus, designers often build in a few extra psi of pressure to cover the unexpected and build in flexibility. This contingency can be expensive in terms of energy costs.

A cause of high pressure in some microirrigation systems is pump output that exceeds system requirements or was designed to meet multiple conditions. Before using a pre-existing pump, the designer should evaluate the energy benefits of modifying the pump or installing a new one designed for the system. In annual crops, the pump may also be used for sprinkler systems. In perennial crops, the flow rate or pressure required for frost control may be much higher than that required for irrigation. Systems that irrigate undulating ground require a pumping plant to irrigate the area at the highest elevation, and thus, the pump is over designed for the rest of the area. For these types of cases, designers should evaluate the energy benefits of multiple pump pumping plants or variable frequency drives.

Variable frequency drives (VFD) may be beneficial in systems that operate over a range of pressure or flow rate requirements. For example, on systems with varying flow rate requirements, VFDs can maintain a constant pressure. In systems on undulating ground that irrigate equal-sized (and flow rate) sets at varying elevations, they can maintain a constant flow rate and vary speed to automatically maintain the pressure required in the system. In both cases, not only does the VFD reduce energy when full capacity is not required, but it also avoids the use of pressure regulation and the pressure loss required by regulators. A VFD can also provide the extra flow or pressure required when the filter backflushes. Variable frequency pumping plants must be well designed to insure that the pump operates most of the time in its high efficiency range. Variable frequency drives also have energy losses (3 - 5%) that must be considered when evaluating their benefits.

In the next phase of this study of energy use in microirrigation, we will review assumptions and choices system designers make that impact energy use. We will also review pressure loss specifications of control valves, filters, and injection systems, and opportunities in equipment selection or use that can reduce pressure loss. Finally, through case studies, we will evaluate conditions under which either multiple pumps or VFDs are economic options.

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ACKNOWLEDGMENTS

The authors wish to thank the Irrigation Training and Research Center and Kings River Conservation District for sharing the Irrigation Evaluation data that was used in this analysis.

Trade names mentioned are for the benefit of the reader and do not imply endorsement or preferential treatment by USDA.

SENSITIVITY OF SURFACE IRRIGATION TO INFILTRATION PARAMETERS: IMPLICATIONS FOR MANAGEMENT

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Theodor S. Strelkoff
Albert J. Clemmens

ABSTRACT

Infiltration characteristics are a major source of uncertainty in the design and management of surface irrigation systems. Understanding the sensitivity of the design to errors or variation in the design inputs is needed to develop management recommendations that account for this uncertainty. This paper further analyzes the sensitivity of the level basin design procedure proposed by Clemmens (1998). Results show that the recommended management approach, cutting off inflow when the water advances a fixed distance relative to the field length, works best when actual advance time is more than predicted. If actual advance time is the same or less than predicted, then cutoff based on time may be a better approach, independent from variations due to differences in infiltration, roughness, inflow, or all of these factors combined.

INTRODUCTION

There are three main sources of uncertainty in infiltration predictions for surface irrigation design and management. First is the mathematical formulation of the process. Second is the variability of infiltration and the determination of representative parameters required by the selected infiltration formulation. Last are soil changes, with implications to infiltration characteristics, occurring not only during the course of the irrigation season but even during one irrigation event (consolidation, aggregate dispersal, sealing of cracks, sediment deposition, etc.). Irrigation specialists recognize these uncertainties but tools do not exist yet that can be used to systematically analyze their consequences. Design and management recommendations are made, therefore, under the assumption that the farmer will experiment with the irrigation system and gradually reach reasonable levels of performance (Bautista et al., 2001).

Rather than relying on trial-and-error, irrigation specialists need to provide farmers with measures of how the system will react to differences in the design specifications. Such measures should help develop a framework for identifying likely reasons for the differences between the anticipated and actual performance

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and, consequently, develop a management strategy that effectively compensates for variations in the suspect parameters.

An example of this type of framework was provided by Clemmens (1998), who proposed a design procedure for level basins based on a distance-based cutoff criteria. The sensitivity of the design to changes in design parameters was compared for two management options, with cutoff determined based on distance and cutoff based on time. Clemmens' analysis considered only the effect of individual input parameters, such as infiltration, but not their interaction, and only one infiltration parameter was analyzed. For furrow irrigation, Zerihun et al. (1996) concluded that sensitivity measures for surface irrigation design are difficult to obtain because changes in one parameter aggravates or mitigates the impact of changes in other parameters.

The objective of this study is to further examine Clemmens' (1998) design approach vis-à-vis the uncertainty in the design inputs. The analysis considers infiltration conditions not considered in the original study and the interaction between infiltration and other input parameters. The study falls within the scope of research activities being promoted by ASCE/EWRI Task Committee on Soil and Crop Hydraulic Properties for Surface Irrigation (Strelkoff et al., 2000).

INFILTRATION UNCERTAINTY

Infiltration equations for surface irrigation modeling

A variety of infiltration equations have been used in conjunction with surface irrigation models. Most of those equations are empirical (explicit relationships between cumulative infiltration and opportunity time) but also semi-physical approaches have been tested (Green-Ampt type formulations). When properly calibrated for an individual irrigated unit, i.e., a border, basin, or furrow, these equations can result in reasonable agreement between predicted and field-measured advance, recession, and runoff (Clemmens, 1982; Bautista and Wallender, 1993; Fonteh and Podmore, 1989). The diversity of infiltration formulations reflects the difficulty in modeling the infiltration process. In irrigation modeling research, selection of an infiltration formulation is frequently based on the perceived ability of the equation to fit field-measured infiltration data (Clemmens, 1983; Tarboton and Wallender, 1989; Childs et al., 1993). In irrigation design practice, choice of a formula is more frequently based on familiarity or availability of data, particularly for the SCS equation (Bautista et al., 2001). The use of physical or semi-physical formulations is not common, partly because of their mathematical complexity, but also because of the ability of empirical equations to compensate for measurement errors and to reproduce the initial stage of the infiltration process (which is often dominated by effects not accounted for in the governing equations of porous media flow). A key difficulty in the use of any formulation is that for many field data sets, several equations may fit the data equally well; however predictions for times longer than the duration of the infiltration test can be significantly different (Clemmens, 1982; Tarboton and Wallender, 1989). Thus, infiltration parameters need to be calibrated taking into account the typical duration of irrigation events.

Comparison of irrigation predictions with alternative infiltration formulations for the same field conditions is lacking. In the case of border irrigation, Clemmens et al. (2001) found that predictions are similar if the infiltrations equations have a similar infiltration characteristic time, i.e., similar intake opportunity time for the required application depth. These findings are based on empirical infiltration equations only and, thus, comparisons with semi-physical approaches are needed. These authors also pointed out the limitations of using this characteristic infiltration time for design, as performance degrades with changes in the irrigation target, which can occur during the course of the irrigation season.

Variability of infiltration

Point-measured cumulative infiltration for fixed opportunity time and infiltration rates can vary by an order of magnitude over an irrigated field and over an irrigation season, with reported coefficients of variation (CV) ranging from 35% up to 90% for both variables (Bautista and Wallender, 1985; Jaynes and Hunsaker, 1989; Childs et al, 1993; Hunsaker et al., 1999). The implication is that many point measurements are needed to characterize infiltration with a high degree of certainty. Infiltration variability estimates are affected by the measurement method and the scale of measurements (Bautista and Wallender, 1985; Jaynes and Hunsaker, 1989). Use of entire irrigated units as infiltrometers (borders, basins, furrows), can reduce the influence of these systematic errors. Still, variability of infiltration measured on larger units can be significant. For example, a CV of 24% for cumulative infiltration (Tarboton and Wallender, 1989) has been measured within furrow sets while a 12% CV has been measured for the time to infiltrate 100 mm of water for a group of borders on a 32 hectare field (Clemmens, 1992).

An alternative approach for assessing infiltration variability is through the distribution of parameters of infiltration functions. Results of studies of this type have been mostly inconclusive because of the correlation between fitted parameters (Jaynes and Hunsaker, 1989; Hunsaker et al., 1999). Studies have shown, however, that the mean and variance of the distribution of advance times, infiltrated depths, distribution uniformity, etc., can be predicted if the mean and variance of infiltration parameter distribution is known and is random (Jaynes and Clemmens, 1986). Since a wide variation in system performance (advance times, infiltrated depths) can result from a given distribution of infiltration characteristics, a conservative approach must be used in formulating design and management recommendations because the performance of individual irrigated units may depart substantially from the average.

SIMULATION PROCEDURES

Clemmens (1998) used the following design specifications to test his design procedure: the target application depth d_{req} is 100 mm, the characteristic infiltration time (for the required application depth) τ_{100} is 210 min. A Kostiakov relationship is assumed for the infiltration function with the exponent $a = 0.5$. The value of K is then determined from τ_{100} and a. The Manning's roughness n is equal to 0.15 and the available discharge is 230 l/s. The proposed design approach requires the designer to specify the distance at which flow will be cut off relative to the field length. Recommended values for this distance-based cutoff ratio, R, are between 0.85 and 1, in accordance with results presented by Clemmens and earlier work by Clemmens and Dedrick (1982). With the given design data and by requiring an R of 0.9 and a potential application efficiency (PAE) of 80%, Clemmens used the BASIN program (Clemmens et al., 1995) to compute the basin dimensions, L= 199 m and W = 85 m (length and width, respectively). The resulting cutoff time t_{co} is 153 minutes.

The sensitivity of the design can be tested through simulation. The SRFR program (Strelkoff et al., 1998) was used in the analysis. Clemmens (1998) studied the impact of infiltration variation by varying the Kostiakov K by ±20 % and ±50%. The corresponding variation in τ_{100} is between -44% and +400%. This study limits the variation in infiltration based on τ_{100} (±25%, ±50%) because it is closer to the CV of cumulative infiltration for a specified time measured in field experiments, as was discussed earlier. The effect of infiltration function form is studied, in a limited way, by varying the exponent a of the Kostiakov relationship. SRFR allows the user to specify the characteristic infiltration time and the value of the Kostiakov a, from which the program internally then determines K. The combined effects of a and τ_{100}, and the impact of these two parameters in combination with Manning n and inflow rate Q, are also considered here. These parameters are likely sources of uncertainty in actual irrigation events. Land leveling precision effects are ignored here as well as changes in d_{req}.

RESULTS

A smaller than assumed τ_{100} slows down the advance because more water infiltrates in a given time relative to the design conditions. In this situation, operating the system based on a specified cutoff time is inappropriate because water would not reach the end of the field, even with just a 25% decrease in τ_{100} (Table 1). In the table, t_L represents the final advance time and L_{max} the maximum advance distance. If cutoff is based on R, then water will reach the end of the field and the minimum depth (d_{min}) will be close to d_{req} (100 mm). If, on the other hand τ_{100} is greater than specified in the design, there is still an advantage in using R as the cutoff criteria because water advances more rapidly than originally anticipated, forcing an earlier cutoff time. This reduces d_{avg} (average application depth) and improves application efficiency (AE). Under these conditions, cutting off at the target t_{co} has no impact on AE but low quarter distribution uniformity (DU_{lq}) improves, so performance will still be close to the design specifications. Thus, if flow rate and roughness are expected to agree with the design specifications but there is uncertainty about τ_{100} (though not about the form of the infiltration function), managing cutoff based on R should provide reasonable performance for likely variations in τ_{100}.

Table 1. Sensitivity of level basin design to □$_{100}$

Variable	τ_{100} (min)									
	Cutoff based on R					Cutoff based on t_{co}				
	-50%	-25%	Design	+25%	+50%	-50%	-25%	Design	+25%	+50%
	105	158	210	263	315	105	158	210	263	315
t_{co} (min)	193	166	153	143	137	153	153	153	153	153
t_L (min)	238	198	179	168	160	∞	∞	179	167	160
R	0.9	0.9	0.9	0.9	0.9	0.77	0.85	0.9	0.94	0.97
AE (%)	63.2	73.7	80.1	85.6	89.5	68.8	77.5	80.1	80.1	80.1
DU_{lq}	0.72	0.82	0.87	0.89	0.90	0.35	0.71	0.87	0.9	0.92
d_{min} (mm)	82.9	98.6	102	98.5	96.8	0	0	102	108	119
DP (mm)	57.4	35.5	24.8	16.7	11.7	38.8	27.6	24.8	24.8	24.8
d_{ave} (mm)	157	136	125	117	112	125	124	125	125	125
L_{max} (m)	199	199	199	199	199	174	199	199	199	199

For a given value of τ_{100}, a reduction in the infiltration exponent a causes more water to infiltrate during the initial wetting; however, infiltration rates decrease more quickly than with the original value. Advance, therefore, should slow down[1]. This is in fact what happens when simulations are carried out with a design based on $a = 0.5$, but with actual $a = 0.3$ (Table 2). These results are in contrast with the findings of Clemmens et al. (2001), who reported little effect of functional form on border design and performance. With cutoff based on R, the slower advance translates into more water being applied to the basin than needed, even when actual τ_{100} is much greater than the design value. The time-based cutoff provides better results than R-based cutoff if actual τ_{100} is greater than in the design. AE remains constant in such case. However, if the actual τ_{100} is less than in the design, advance doesn't reach the field's end. In practice, one would not be able to determine if the problem is in the estimation of τ_{100}, a, or both. Therefore, the best approach would be to cut off based on advance distance.

Table 2. Sensitivity of level basin design to τ_{100} and infiltration function form (a = 0.3)

Variable	τ_{100} (min)									
	Cutoff based on R					Cutoff based on t_{co}				
	-50%	-25%	Design	+25%	+50%	-50%	-25%	Design	+25%	+50%
	105	158	210	263	315	105	158	210	263	315
t_{co} (min)	197	183	174	167	163	153	153	153	153	153
t_L (min)	231	213	202	194	189	∞	∞	229	204	192
R	0.9	0.9	0.9	0.9	0.9	0.74	0.78	0.81	0.83	0.86
AE (%)	62	67	71	73	75	72	78	80	80	80
DU_{lq}	0.93	0.95	0.96	0.96	0.96	0.51	0.78	0.92	0.95	0.96
d_{min} (mm)	146	139	134	130	127	0	0	107	115	118
DP (mm)	61	49.1	41.4	36.4	33.1	35.1	27.6	24.7	24.8	24.8
d_{ave} (mm)	161	149	142	136	133	125	125	125	125	125
L_{max} (m)	199	199	199	199	199	178	193	199	199	199

Table 3. Sensitivity of level basin design to τ_{100} and infiltration function form (a = 0.7)

Variable	τ_{100} (min)									
	Cutoff based on R					Cutoff based on t_{co}				
	-50%	-25%	Design	+25%	+50%	-50%	-25%	Design	+25%	+50%
	105	158	210	263	315	105	158	210	263	315
T_{co} (min)	196	151	133	123	117	153	153	153	153	153
T_L (min)	∞	185	159	146	138	∞	183	158	145	137
R	0.9	0.9	0.9	0.9	0.9	0.79	0.9	0.98	>1	>1
AE (%)	58	78	89	95	98	66	77	80	80	80
DU_{lq}	0.49	0.66	0.79	0.83	0.85	0.23	0.68	0.83	0.88	0.90
d_{min} (mm)	0	59.3	74.7	75.8	76	0	65.3	93.6	103	108
DP (mm)	66	27.3	12	5.5	2.1	42.2	28.4	25.1	24.8	24.8
d_{ave} (mm)	159	123	109	101	95.8	125	125	125	125	125
L_{max} (m)	192	199	199	199	199	171	199	199	199	199

Increasing the exponent a from 0.5 to 0.7 has quite different implications (Table 3). In this case, initial infiltration is less than in the original design so advance is faster (except in a very long basin). In this case, severe underestimation of τ_{100} (-50%) will cause water not to reach the end of the basin with either cutoff criterion. If τ_{100} is underestimated by only 25%, the R-based cutoff time is close to the design value, so results are similar. For all other conditions (conditions under which the advance time to the end of the field is less than specified in the design) the time-based criterion provides clearly better results.

Clemmens (1998) showed that if the only source of uncertainty in the operation of the irrigation system is in the determination of Manning n (with n varying by ± 50%), then determining cutoff based on time provides better performance than based on R. Changes in roughness and infiltration characteristics can have equal or opposing effects on the advance rate of the surface flow. Their interaction is examined next.

A Manning n of 0.15 is the NRCS recommended value for alfalfa or broadcast small grains. A value of 0.10 is recommended for drilled grains, while a value of 0.2 is recommended for a dense alfalfa. Thus differences of this magnitude would not be unlikely between the design and the field values. If the actual n = 0.20, advance will be slower, but both the time- and distance-based cutoff criteria will result in reasonable performance (Table 4). If cutoff is time-based, there will be some underirrigation near the end of the field, but with distance-based cutoff there will be greater deep percolation and lower AE. The choice of criteria for cutoff would depend on the sensitivity of the crop to a water deficit, cost of water, and possibly other factors.

Table 4. Combined effect of change in n, τ_{100}, a and Q with n = 0.2.

Variable	n = 0.2		□$_{100}$ = 158 min		a = 0.3		Q = 207 l/s	
	Cutoff criteria							
	t_{co}	R	t_{co}	R	t_{co}	R	t_{co}	R
t_{co} (min)	153	167	153	182	153	198	153	217
t_L (min)	206	197	∞	216	∞	229	∞	253
R	0.8	0.9	0.80	0.9	0.73	0.9	0.68	0.9
AE (%)	80	73	75	67	75	62	77	63
DU_{lq}	0.83	0.88	0.6	0.84	0.68	0.96	0.48	0.95
d_{min} (mm)	92.1	113.6	0	113	0	152.2	0	149.2
DP (mm)	25	36.4	31.1	48.2	30.9	61.2	26.1	59.5
d_{ave} (mm)	125	136	125	148	125	161	112	160
L_{max} (m)	199	199	190	199	187	199	171	199

If in addition to n being in error, τ_{100} is also in error by -25%, then the time based criteria no longer works because the water does not reach the end of the field (Table 4). While the alternative criterion (R) allows irrigation of the entire field, the drop-off in AE is noticeable.

Two other scenarios are presented in Table 4, one in which in addition to the previously noted changes, the exponent a is also in error ($a = 0.3$). The last scenario considers a flow rate reduction of 10%. All of these factors cause the advance to be slower. As a result, the time-based criteria is inadequate in all of these cases. If using distance to determine cutoff, water will reach the end of the field in all cases, but AE will be low.

An alternative set of scenarios is presented in Table 5. Actual Manning n is 0.10 (faster advance) but then τ_{100}, a, and Q are varied as in the previous example so the changes in these variables have opposing effects on advance. Results show that if n, τ_{100}, and a are uncertain, but less hydraulic resistance is anticipated than specified in the design, then cutting off based on time will be a better alternative than if based on distance. This assumes that the design Q is accurately delivered. If Q cannot be accurately measured or not measured at all, but the operator has reasons to believe that it is less than the design value, then a better approach is to manage the system based on distance. It should be noted, however, that if the operator has knowledge of the difference in discharge with respect to the design value and adjusts the cutoff time accordingly, then the time based approach will still result in the expected AE and near the expected uniformity.

Table 5. Combined effect of change in n, τ_{100}, a and Q with $n = 0.1$.

Variable	n = 0.1		τ_{100} = 158 min		$a = 0.3$		Q = 207 l/s	
	Cutoff criteria							
	t_{co}	R	t_{co}	R	t_{co}	R	t_{co}	R
t_{co} (min)	153	133	153	147	153	165	153	182
t_L (min)	157	158	174	177	210	193	∞	214
R	0.98	0.9	0.92	0.9	0.85	0.9	0.78	0.9
AE (%)	80	90	80	82	80	74	83	75
DU_{lq}	0.89	0.83	0.81	0.78	0.89	0.93	0.64	0.92
d_{min} (mm)	104.9	82.3	89.4	79.2	102.5	122	0	119.2
DP (mm)	24.9	11	25.2	21.6	24.7	34.3	19.4	33.8
d_{ave} (mm)	125	109	125	120	125	134	112	134
L_{max} (m)	199	199	199	199	199	199	186	199

DISCUSSION

In general, results indicate that, for the indicated scenario, if water advances nearly as fast or faster than anticipated in the design to the cutoff point, then the system should be managed based on time. In such cases, final infiltrated depth distribution depends mostly on the total volume of water applied, independent from various sources of error. If advance is significantly lower than expected, then a safe approach is to irrigate based on cutoff distance, although performance could be poor, especially if the design underestimated roughness and infiltration characteristics, and flow rate is less than expected. An important consideration in the management of the system under conditions where advance is much slower and performance potentially much poorer, is determining which inputs are most different from the design specifications.

Given that level basin systems represent a significant investment, especially for large basins typical in the U.S., one could argue that reasonable flow rate measurement should be required with those systems and that irrigators need to be familiar with the effect of inflow rates different that the design value on advance (a readily available performance measure). If actual advance behaves in accordance with predictions for the actual inflow rate, then other design inputs are likely in agreement with field values. In such cases, use of either cutoff criterion should yield levels of performance close to the target, even for significant differences in flow rate relative to the design value (Clemmens, 1998). The hydraulic roughness characteristics of bare and cropped field have been analyzed (Gilley and Finkner, 1991; Gilley and Kottwitz, 1994) and these analyses suggest that the recommended NRCS Manning *n* values are reasonable. Designing the system for mid-season roughness conditions, when resistance to flow is highest, provides a way to partially compensate for higher than anticipated infiltration conditions (Table 5) and assures that slower than predicted advance will be the result of differences in infiltration characteristics.

The results of Tables 2 and 3 show that the infiltration function form impacts performance, even when τ_{100} is accurately known. Again, the consequences seem minor if actual advance time is less than expected but the interaction between τ_{100} and a seems significant if advance is slower than expected. It is important to note that this part of the analysis assumes that τ_{100} and a are estimated independently. In situations where τ_{100} is calculated based on an assumed a, underestimating a would likely result in an overestimation of τ_{100} while the opposite would be true if a is overestimated. This has implications for the results presented in Tables 2 and 3. In the former case (Table 2), an actual τ_{100} □ 210 min would be less likely to occur and, therefore, the recommendation would always be to cutoff based on distance. In the latter case (Table 3), an actual τ_{100} □ 210 min would be less likely to occur and time-base cutoff would always be the preferred management choice.

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[1] For a very long field, if near steady-state infiltration rates are reached in the upstream portions of the field during the course of the irrigation, actual advance may be initially slower than predicted but later in the irrigation actual advance may be faster than predicted.

A LEADING-EDGE IRRIGATION DEMAND MODEL FOR ASSESSING IRRIGATION EXPANSION WITH FINITE WATER SUPPLIES

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ABSTRACT

Like many jurisdictions in North America, the irrigation industry in Alberta, Canada has found it necessary to intensively examine its future state of development, in view of substantially increased competition for a finite supply of available water. In order to do so, it was recognized that available technical science and assessment tools needed to be up-dated and expanded. Specifically, the opportunity and ability to utilize state-of-the-art computer modelling techniques could allow much more detailed and varied analyses to be carried out. As part of a broad scope basin water management planning review, the development of a complex irrigation demand model was undertaken. After several years of detailed and intensive software development, a suite of data input, irrigation simulation and analysis tools has been derived. The application of the irrigation demand model component provides for very detailed projections of daily water requirements, consumptive use, conveyance and application losses, as well as return flows. Annual and multi-year irrigation demands can be determined in conjunction with water supply conditions that reflect both the inter-relationship with the vagaries of climate as well as varying scenarios of development within the industry. In particular, output from the application of the whole suite of tools indicates both the projected level of water supply deficits as well as the potential impacts of those shortages.

BACKGROUND

In the province of Alberta, Canada, with its current irrigated land base in excess of 600,000 hectares, the industry is assessing the opportunities and risks associated with expanding that base, within the confines of currently licensed water allocations. Irrigation efficiency is defined, for Alberta conditions, as the ratio between the net amount of diverted irrigation water available for plant consumption and the total amount of water diverted from a natural water source

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for irrigation use. It has been projected that the irrigation water use efficiency gains that have been made over the last 10 to 20 years have resulted in substantial reductions in water consumption per unit of irrigation area. This "freed-up" water has the potential to extend existing licensed volume allocations over an expanded irrigation area. The question then was, just how much of an irrigated area increase can be sustained by available water supplies, and variabilities thereof, and at what level of risk and impact to the irrigation farm economy?

The assessment carried out in this respect has been an extensive five-year partnership project, beginning in 1996, between the 13 local producer-owned irrigation districts, the Alberta Government and the Government of Canada. This multi-million dollar project not only developed the necessary simulation and analytical software but also carried-out detailed unit-by-unit inventories of irrigation systems, crops and distribution infrastructure. These inventories as well as a variety of complementary field research were all carried-out to calibrate and drive the modelling components. There were three primary objectives of the Irrigation Water Management Study (Irrigation Water Management Study Committee 2002), as it related to Alberta.

- Quantify the extent of improvement in irrigation water use efficiencies over the previous 10 to 20 years.
- Accurately determine current and future water requirements and water management operations for sustainable irrigation development.
- Quantify the potential opportunities for irrigation expansion, at reasonable risks to irrigation producers.

Available Modelling Software

Up until the mid-1990s, irrigation demands were modelled as a basic component within the Water Resources Management Model (WRMM), a long-standing river basin planning tool originally developed for application by the Alberta Government's Department of Environment, as a part of their water management, protection and regulation mandates. Previous irrigation demand modelling employed in this Alberta river basin planning did not fully recognize the large extent of variability in irrigation water requirements from irrigation block to irrigation block or from day-to-day or year-to-year. As a result, only a general regional demand input was generated, with no capability to also carry-out sensitivity analyses for projected future irrigation conditions. Further, the weekly time step process incorporated did not provide the level of time sequence detail required to fully recognize potential daily occurrences of moisture events that could affect how irrigations are managed or demands adjusted.

Consequently, it was recognized, early in the study process, that a new state-of-the-art irrigation demand model would be required to more effectively determine irrigation opportunity potentials, yet retain the interface where demand output could be input to the basin planning model that controlled water supply conditions. A search was carried out to determine the availability of existing software that could satisfy the requirements of generating and managing irrigation demands throughout the complex networks of today's irrigation district systems. Despite a review of both North American and internationally recognized basin planning and water routing software, it was determined that what was available was designed to simulate hydrologic functions of "aggregating" tributary water from several sources into common collectors, the opposite direction from that required in deriving and meeting field-by-field irrigation demands.

DEVELOPING THE MODELLING PROCESS

The principal objective of using modelling techniques was to derive variable irrigation demands on available, but limited water supplies, quantifying and qualifying any deficits that may occur, and then quantifying the impacts of those potential deficits on the financial viability of various types of farm enterprises. Although the primary focus for the discussion in this paper is on the development and application of the Irrigation District Model (IDM) software (Baker et al. 2000), the modelling process employed in the overall Irrigation Water Management Study actually involved several different simulation, data capture, data manipulation and analytical tools, most of which were new developments.

These tools included an on-farm irrigation demand generator, referred to as the Irrigation Requirements Module (IRM); a conveyance network demand quantifier and routing solver, referred to as the Network Management Module (NMM); the WRMM water supply manager; and an economic impact and risk assessment application, referred to as the Farm Financial Impact & Risk Model (FFIRM). Figure 1 illustrates this modelling and analysis cycle with the primary model components displayed within the assessment sequence. As can be seen, the IDM is actually made up of two distinct but integrated modules; the IRM and the NMM. Other support tools involved both the data capture and conversion processes, merging the required data into the Local Operating Database (LOD).

First and foremost in supporting any modelling application and the integrity of its output, is the data upon which a model executes its functions. In the case of this particular study and model development, an inordinate amount of data were collected in the field or extrapolated from existing databases. The primary data complex and GIS shape files driving the application of the suite of modelling tools were assembled with 1999 as the base reference year. The fundamental data components included daily agro-climatic data, on-farm crop mix and irrigation system information, as well as conveyance infrastructure component and network details.

The IDM suite of tools is designed to run on a Microsoft Windows NT Server platform, referencing MS Access database tables and a Microsoft SQL local operating database. The software is primarily developed in MS Visual Basic and

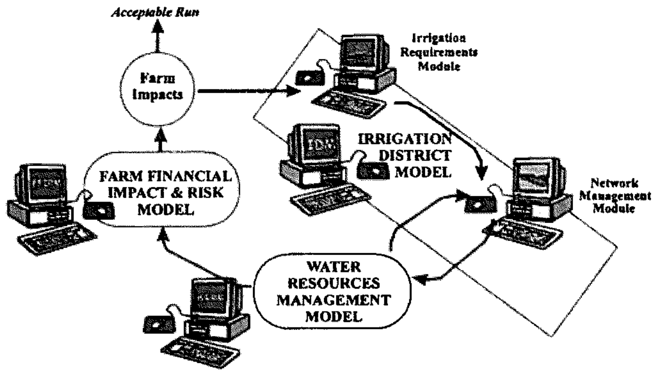

Figure 1. The irrigation demand, supply and impact modelling analysis cycle.

MS Visual C++, with a graphic user interface displaying simulated network components as a GIS map using ESRI MapObjects (AAFRD 2002).

Data Collection and Warehousing

The agro-climate database, fundamental to driving irrigation demands, was developed on the architecture of the existing Gridded Prairie Climate Database (GRIPCD) (Riewe et al. 2001). The GRIPCD is a synthesized database of daily climate information, including such parameters as precipitation, maximum and minimum temperatures, relative humidity, wind and solar radiation. The database was developed to reference climate data on a 50-kilometre by 50-kilometre grid across the Canadian prairies, covering a period from 1920 through 1995. This extensive database was enhanced for the region covering the irrigated areas in Alberta by pre-determining daily potential evapotranspiration (PE) values for each of the associated climate grid points (Riewe et al. 2001).

Much of the required data collection was achieved through an intensive field-by-field inventory process, conducted by each irrigation district. In compiling this extensive database, each and every irrigation field within a district was catalogued as to the type of crop being grown and the type of in-field irrigation system that was used to apply irrigation water on to that field. A total of 56 different crop or crop production-types and 18 different irrigation systems or system configurations were identified for the inventory process. The final inventory consisted of more than 10,000 individual irrigation fields, each linked to individual infrastructure delivery points identified in a GIS shape file, with each being simulated on a daily basis for each day in an irrigation year, for as many years as were to be included in a model run.

Finally, in cooperation with the irrigation districts, a GIS database (ESRI ArcInfo) was developed that provided detailed linework and specific attributes for all of the irrigation district conveyance and drainage infrastructure, an estimated 8,000 kilometres or more, including reservoirs.

In order to examine the potential effect on irrigation demand through expanding the irrigation land base, or of on-going changes in on-farm irrigation technology, or of crop mix shifts, the Scenario Builder software application allows the user to modify any model dataset to reflect those types of changes, either as wholesale adjustments to a project or to specific component areas within a project.

The Irrigation Requirements Module (IRM)

In essence, this module monitors, through its simulations, the soil moisture status of each irrigation field within any defined irrigation block or project, for each day of 365 days in each and every year of a simulation period. The layering and compounding of irrigation water demands and losses is illustrated in Figure 2. The diagram defines the progression of IDM (IRM + NMM) derivations down to integration with the WRMM. Some of the defining parameters are fixed or have default values, but many are variables that the user can set to depict a particular operating condition or else a projected situation for a sensitivity analysis. Variable settings can be critical in affecting the derived overall demand. The ultimate effect on demand derivations rests with the accuracy and functionality of the model algorithms, ones that have been developed, defined and incorporated, based on actual field research or long-time experience (AAFRD 2002).

Besides driving the daily crop consumptive use, the agro-meteorological data file also supports derivation of evaporation components, start date of plant growth each year, time periods for application of alternative out-of-season soil moisture accumulation or loss algorithms, as well as magnitude of rainfall intensities for run-off determinations.

The crop-type attributes provide an extensive level of detail to variable crop-type water use determinations. Such crop-specific parameters as maximum root zone depth, rate of root depth development, daily consumptive use coefficients, crop growing season and harvesting date(s), randomized irrigation threshold values and randomized eligibility for fall irrigation applications are some of the main attributes that extend the diversity of the modelling regime.

An identification of soil textures for each irrigation field was derived through the AGRASID (CAESA 1998) soils database for Alberta. From that soil texture information, a range of water holding capacities was defined for each texture group, values that then quantified available moisture and irrigation threshold values. In addition, through soil texture polygons overlaid on the GIS infrastructure base, channel seepage potentials were derived (Iqbal et al. 2000).

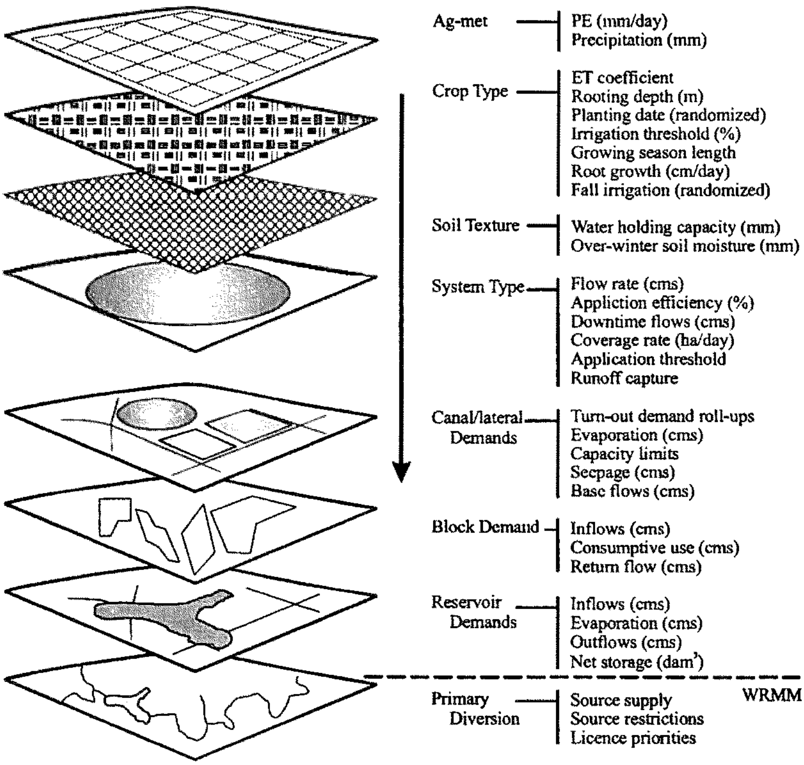

Figure 2. IDM "layers" of water demand and routing variables.

In similar fashion to the crops detail, the diversity of on-farm system types and various operating parameters provided considerable variability in how water was demanded, in how much by-passed through system shut-down or set moves, in how it was applied, by rate and by amount, in how much of a field was covered each day, and in how much was lost through the application process. Several additional system-specific operating criteria were also attached, such as reduced application depths at the beginning of the year when crop rooting was in its early stages.

The Network Management Module (NNM)

This module accumulates all system demands for respective turn-out deliveries into respective conveyance works, routing those demands back through the network to up-stream reservoirs and initial diversion points off of the basin water supply system. The NMM is component object based (Baker et al. 2000) and is built to conform to the Microsoft Component Object Model (COM). The NMM requires few algorithms or parameter settings, as it is basically an arithmetic accumulator of downstream demands as it moves "up" the network. ILOG CPLEX linear programming has been incorporated into NMM to solve for optimal routing of water distribution.

The conveyance and drainage network of each and every irrigation block has been inventoried in detail and captured within a GIS shape file. This defines, on a reach-by-reach basis, the type of works in place, the design capacity thereof, the seepage and evaporation rate potentials, as well as the projected base flow. The latter, for example, is derived for each network, based on previously recorded return flows and is tagged at the most downstream end of associated return flow channels as a Base Flow Object. These shape files and associated attributes are merged with IRM demands into the LOD, linked by individual delivery turnouts.

As specified by each district's operational pattern, NMM initiates the start-up date for each district, including a time period for canal flushing. All canal and lateral demands are rolled up to produce required block inflows and resulting block outflows. Outflows can, in part or in whole, become inflows to adjacent blocks or can simply be directed out of the block as return flow or to an outflow sink.

As block demands are rolled-up, outflow requirements from storage facilities react according to reservoir operating rules. These in turn roll-up demand requirements in successive fashion to the primary point(s) of water source diversion.

MODEL CALIBRATION AND VALIDATION

To achieve confidence in the modelling output, it was mandatory that validation of results with actual recorded events be performed. To that end several individual validation projects were undertaken, where accurate historical recorded data were available, or where water audits had been previously carried out.

On average, for the test cases analyzed, simulated total water demand for the season, as compared to actual demand, was within ±1.5%. Figure 3, for the Lethbridge Northern Irrigation District (LNID) (67,000 hectares), illustrates a comparative graphical plot of time-series IDM simulated demand and return flow for 1999 (an average year), in comparison with actual recorded inflows and outflows. Simulations were found to be within 1.7% for inflow volumes, within 3.2% for return flows and within 3.1% for total consumptive use. One notable irregularity between the two profiles is at system start-up in the spring. Due to early spring conditions that were drier than normal, particularly on perennial forages, an irrigation demand threshold was triggered within the IDM, whereas in reality, irrigators were slower to react, for various cultural reasons.

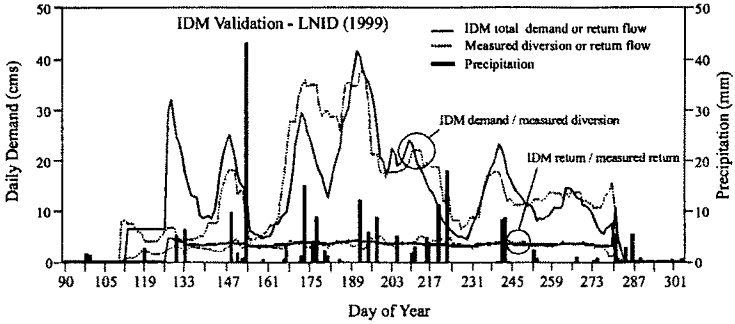

Figure 3. Comparison of profiles of modelled and actual water demand and return flows.

MODEL OUTPUT AND RESULTS ANALYSIS

The IDM provides a considerable amount of water demand detail, whether it is examining individual parcel demands, rolled-up block demands or overall project diversion requirements. Output, at whatever desired level, can be displayed in both volume amounts or in a depth equivalent per irrigation unit area, for the quantifiers summarized below.

- Gross diversion demand.
- Total consumption at the farm gate.
- Net irrigation application as that amount of water available to crop roots.
- Total consumption at the farm gate and through the distribution system.
- Total return flow.
- On-farm losses.
- Distribution system losses.
- Reservoir evaporation losses.
- Other system losses not returned to the basin hydrology.

More extensive analysis of the output data can be extracted or derived. For example, conveyance works' design capacity is not used to restrict flow routing. Rather, an "exceptions log" is produced by NMM through each model run that allows the user to determine where, in the network, demands exceed current capacities, by how much those capacities are exceeded and over how many days these exceptional demands occur. This helps the user to verify whether limitations in meeting demands are a function of a water supply deficiency or a conveyance restriction.

With the demand data output having been entered into the WRMM process, output from the latter provides a direct weekly roll-up comparison between the ideal IDM demand and the WRMM simulated supply, including quantified potential supply deficit conditions. These include when and where in the system operation deficits can occur. With both IDM and WRMM output, further analysis of the frequency, magnitude and duration of deficits can be carried-out to determine the potential financial impact of water supply shortfalls on the agriculture sector, or where alternative operational strategies could be employed.

Figure 4 illustrates the variable area-weighted-average irrigation demands and projected water supply deficits for a typical expansion scenario modelled for nine irrigation districts diverting their water from the Oldman River Basin. The scenario conditions applied include:

- 68 years of variable climate, 1928 to 1995 (with 1927 as a seed year);
- a 20% expansion in irrigation area beyond 1999 levels;
- a shift in crop mix to higher value, higher water-demand crops;
- a shift to more efficient on-farm systems;
- a higher level of farm water management with irrigators meeting 90% of optimum crop water requirements, and
- on-going improvements to distribution works and water control systems.

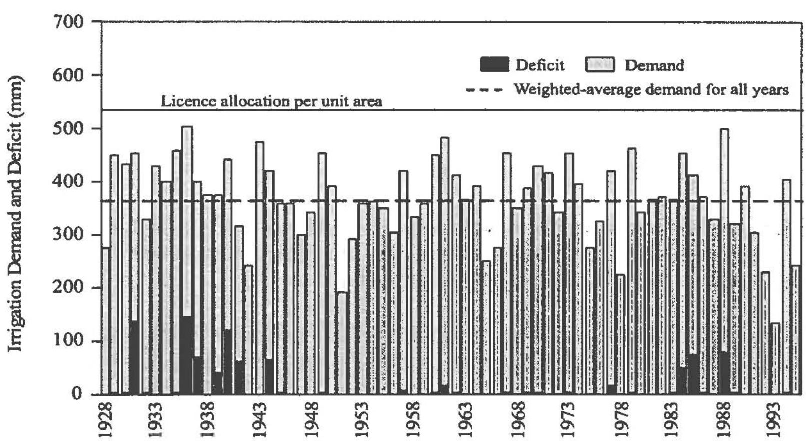

Figure 4. IDM-modelled variable irrigation demands and projected deficits.

Despite the 20% expansion and higher levels of crop irrigation requirements, the results in Figure 4 still indicate water demands each year being within licensed water allocations. Subsequent economic analyses through the FFIRM software indicated that that the revealed deficits were manageable, for the most part, particularly as they appeared to any extent on a 40 to 50-year cycle.

CONCLUSIONS

As reported in the conclusions of the Irrigation Water Management Study, "The simulation models in this study are excellent tools for evaluating the effects of changing water management variables on water demand and supply within irrigation districts." The extensive detail within the modelling output allows for very specific and concentrated analyses that can examine a variety of localized effects and impacts.

With the data structure in place to characterize the physical works of the irrigation industry in Alberta, and with the suite of data processing and decision-making tools developed, water managers, planners, and operations personnel are now much better equipped to implement critical irrigation water management and development initiatives that utilize and impact a finite resource.

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CANAL AUTOMATION SYSTEM DEMONSTRATION AT MSIDD

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ABSTRACT

A new canal automation system, known as SACMAN (Software for Automated Canal MANagement), has been developed at the U.S. Water Conservation Laboratory in cooperation with Automata, Inc. through a Cooperative Research and Development Agreement. SACMAN works with a commercial Supervisory Control And Data Acquisition (SCADA) system. It allows canal operators to automatically route scheduled changes in demand through their canal system utilizing volume-compensation and time delay calculations. SACMAN can automatically maintain constant water levels on the upstream side of check structures with either downstream or upstream control logic. SACMAN is also capable of automatically maintaining constant gate flows and making incremental gate flow changes. The operator can also make manual changes to the system without turning the automation off. The SACMAN system is currently being tested in real time on the WM canal, a lateral canal of the Maricopa Stanfield Irrigation and Drainage District (MSIDD) in Central Arizona. At the control center, the SACMAN system uses a standard personal computer and commercial SCADA package. Each gate is operated with limitorque motors (not part of this package), which are controlled with Automata's "Mini" Remote Terminal Unit (RTU). Control is based on water level and gate position sensors. A new gate-position sensor was developed that includes both absolute (potentiometer) and very fine relative (optical encoder) position. Communication between the personal computer and RTUs is accomplished with spread-spectrum radios and MODBUS communication protocol. The entire system is available through Automata, Inc. The paper includes a brief description of the software, hardware, and field-test results.

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INTRODUCTION

The objective of the canal operator is to provide the correct amount of water to each user, which requires the correct divisions in flow at bifurcations. A manual, local canal operator uses water level deviations to judge whether conditions have changed and whether adjustments are needed. A local canal operator cannot see the entire canal at once and does not know what changes will be felt at that location in the future. A supervisory canal operator can see the entire canal at once and thus can see how flow mismatches vary throughout the system, as reflected by deviations in water levels. Most supervisory canal operators make changes based on these mismatches and then wait for conditions to stabilize, often three or four hours. Any new changes in demand (or inflow outflow difference) during that time are hard to detect since the impact of prior control changes have not yet been felt. Automatic supervisory control overcomes this limitation by making more-or-less continuous adjustments (for large canals, maybe every half hour or so).

In most canal systems, only large main canals are controlled by supervisory control systems, with the rest of the distribution system handled locally, manually. Local automatic upstream water level controllers have shown good performance in controlling water levels, but at the expense of fluctuating downstream flows. Simulation studies of centralized water level control suggest that automatic control can improve water level control on the Arizona Canal compared to manual supervisory control (Clemmens et al 1997, 2001). This has not been demonstrated on the real canal. Even so, improved control of main canals does not solve all water distribution problems. Water users are demanding more flexibility in the timing of water delivery, more constant flows, and more flexibility in adjusting flow rates during irrigation events. Conversion from surface irrigation to pressurized irrigation also requires better control of lateral canals. Districts are facing the need to add field staff to accommodate these user demands.

In the past, SCADA systems have been relatively expensive to install and operate. Costs have come down by nearly an order of magnitude over the last decade, making the automation of lateral canals more feasible. There are many good SCADA packages available for personal computers. Also, remote terminal units (RTUs) can be purchased for 10% of what was available a decade ago. Spread-spectrum radios have opened up communications without the need for FCC licensing. Transducers have seen similar price reductions. Thus the cost and complexity of automation has come down at the same time water delivery demands have increased.

In this paper, we introduce a canal automation system, SACMAN (Software for Automated Canal MANagement). This system interfaces directly to a commercial SCADA package (currently iFIX Dynamics by Intellution, Inc., but other packages can also be used). Communication and control to RTUs is done by the SCADA package. The SACMAN software runs in parallel with the SCADA system and interfaces with the SCADA database. SACMAN hardware includes low-cost RTUs with spread-spectrum radios, pressure transducers, gate position sensors, and gate control relays, all available through Automata, Inc. This paper describes the system and the results of field trials at the Maricopa Stanfield Irrigation and Drainage District (MSIDD).

CONTROL OBJECTIVES

The SACMAN control system has been developed in a flexible manner so that a variety of control objectives can be attained. It is recognized that sloping canal systems cannot automatically respond to large demand changes regardless of the control logic (i.e., open canals cannot perform like closed pipelines). Major flow changes need to be routed through the canal. With SACMAN, this can be done manually by the operator or automatically by SACMAN itself. For feedback control of water level errors, SACMAN examines water level errors within each pool and the history of prior control actions. From this SACMAN determines the control actions needed to correct the errors according to user-defined objectives. These objectives can vary from centralized control (all gates adjusted based on all water level errors) to local downstream control or local upstream control, with many options in between. More details on the control approach can be found in Clemmens et al. (1997) and Clemmens and Schuurmans (2002).

CONTROL SYSTEM IMPLEMENTATION

Hardware

The hardware for this system consists of water level and gate position sensors, RTUs, gate motor drivers, gate motors, spread-spectrum radios, and a personal computer, as shown in Figure 1.

The Automata "Mini" is used as the RTU, which has a 10-bit processor for analog to digital conversion. For this application, it is set up for 4 digital inputs and 4 digital outputs. Any commercially available water level sensor can be used: however, its range (e.g., 4-20 ma) must be compatible with the digital input of the "Mini" (as ordered).

We developed a new gate position sensor that includes two sensors, one for absolute position and one for relative position. When originally developed, we had only an 8-bit processor and could not get adequate resolution with an absolute position sensor. A rigid gear rack, attached to the gate along its centerline, passes through the gate position sensor enclosure. The gear rack rotates a gear that drives two position sensors: a potentiometer that gives absolute gate position to within 0.004 ft or 1.2 mm (based on a 4 ft span divided into 2^{10} or 1024 parts) and an optical encoder that gives relative gate position to within 0.003 ft or 0.9 mm regardless of span (based on diameter of gear). This interval can be cut in half

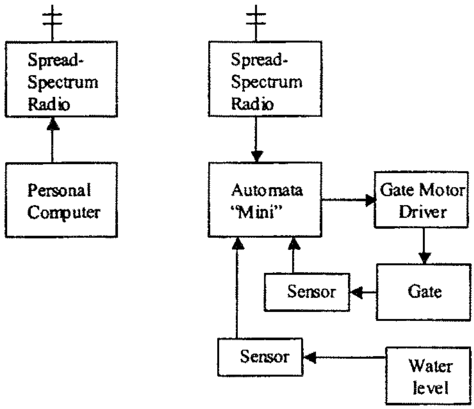

Figure 1. Hardware for SACMAN canal automation control system.

with additional programming, but this does not seem needed at this time. In principle, any gate position sensor can be used.

Automata has standard circuits for controlling gate motors. The circuit boards generally need to be set up to fit the particular gate motor housing being used, or packaged separately.

Communication between the RTUs and the computer is through 900 MHz spread-spectrum radios with MODBUS protocol.

Software

iFIX Dynamics by Intellution, Inc. is the SCADA package currently being used. The canal is set up for supervisory control in a standard manner. iFIX is set up to monitor canal water levels every minute and to store these values in a database. Standard iFIX displays are used to graph the current water levels, flow rates, and gate positions for each check structure. In addition, the water level and flow setpoints are added to the display. The canal operator can still manipulate gates manually.

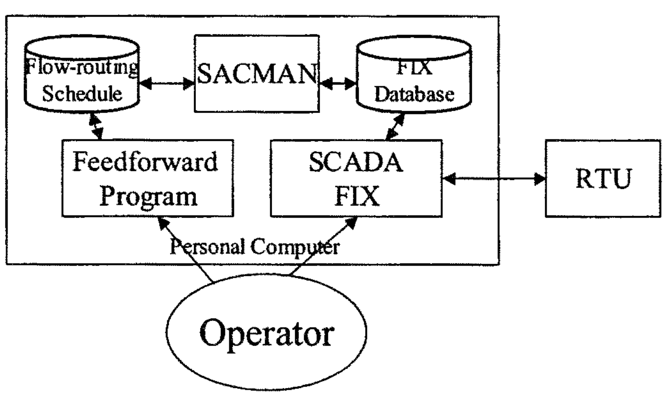

Figure 2. Layout of SACMAN canal automation control system software.

The operator interfaces with the iFIX Dynamics SCADA system to monitor the canal through the iFIX output screens. SACMAN also monitors the canal by reading the iFIX database, as shown in Figure 2. Based on the information in this database, it determines whether control actions are needed. The operator also interfaces with the program for routing demand changes through the canal (feedforward routing). A schedule of offtake flow changes is input by the operator. The program then determines the timing and magnitude of flow changes at all check structures that are required to implement the operator's schedule. This schedule is influenced by the current conditions in the canal, thus information about those conditions must be entered by the users. In the future, this information will be transferred to the feedforward program from the FIX database by SACMAN. The schedule of flow changes is written to a database for SACMAN to read.

SACMAN includes logic: 1) to route intended flow changes through the canal, 2) to adjust gate flow rates in response to downstream water level errors, and 3) to determine gate positions required to provide the desire (setpoint) flow rates at each gate. New values for the flow rate setpoint are determined based on both feedforward routing of intended flow changes and feedback control of downstream water levels. SACMAN currently computes the necessary changes in gate positions for each gate based on the current flow rate, the new flow rate setpoint, and the current gate position. This could also be accomplished at the RTUs by sending just the requested change in flow rate, but this has not yet been implemented. Gate flow rate control uses incremental control, where a change in gate position is determined for the desired change in gate flow. This avoids problems associated with not knowing the flow rates and gate positions exactly. Flow control is accomplished in SACMAN by inverting the gate equations. SACMAN instructs FIX to send information to the RTUs by changing values in the FIX process database.

Feedback control of canal water levels is accomplished with the control logic developed by Clemmens and Schuurmans (2002). It uses a state-space form to compute new check-structure flow rates based on water level errors and prior flow rate changes. Within SACMAN, this amounts to a matrix calculation, with the current states multiplied by a gain matrix to arrive at new flow set points. The coefficients in this matrix are determined with control engineering logic, as described by Clemmens and Schuurmans (2002).

Feedforward routing of intended flow changes is accomplished with software developed by Bautista et al. (2002) based on volume compensation. This requires information for each pool on the relationship between flow rate, water level set point and canal pool volume.

SACMAN uses a constant time step for downstream feedback control of water levels. If it performs the flow control function, the time step for that must be a multiple of the feedback control time step. The raw feedforward schedule is not linked to these time intervals, but should be adjusted by the operator to match either time interval. Eventually, this will be done automatically.

Firmware

The "Mini" uses a 10-bit PIK microprocessor. Codes sent from FIX are used to request sensor information, to change register values, and to cause functions to be performed. The "Mini" is programmed to accept signals in MODBUS protocol. In the current application, a request for a change in gate position is sent as a binary signal. The first bit is a sign bit, which indicates up or down movement. The other seven bits represent the amount of gate movement. This value is placed in a register. Then the relays are set to start moving the gate in the proper direction. For each count on the pulse counter, the register is decremented by one. When it reads zero, the gate motor is stopped. Run-on has never been more than one pulse. A timer limits overrun in the event of sensor failure. The absolute position sensor provides a check, and a backup if the optical encoder fails. Use of this sensor for gate control has not been programmed yet.

APPLICATION AT MSIDD

The SACMAN control system has been implemented on the WM canal at the Maricopa Stanfield Irrigation and Drainage District (MSIDD). The WM canal is a lateral canal with a capacity of 90 cfs (2.5 m^3/s). It was originally supplied with motorized gates. Relay boards, built by Automata, were installed in each gate motor. "Level-tel" water level sensors were installed in existing stilling wells along the upstream side of the gate frame. Automata's new gate position sensors were also installed.

The control logic used in this application is described by Clemmens and Schuurmans (2002). Application to ASCE test canal 1, which is based on the WM canal, is described in Clemmens and Wahlin (2002). The control logic converts water level errors into flow rate changes at each gate. SACMAN determines the gate position change needed to achieve that flow control change and sends a gate position change to FIX. We plan to be able to send flow control commands directly to the RTU, where the change in gate position would be determined.

The current control system determines new flow setpoints for each check structure every 10 minutes. Gate position changes to achieve that flow rate at each check structure are performed every 2 minutes. If a large number of sites are being controlled, the flow control function may best be accomplished locally, depending on the complexity of the flow calculations.

Field Testing

Initial testing of the system was performed in the fall of 1999. Since then, we have converted from Automata's older RTU to the "Mini," the MODBUS protocol was programmed into the "Mini" and Automata's base station firmware, we switched from FM to spread-spectrum radios, and the SACMAN software was totally reworked. These conversions were completed in the summer of 2001. Field studies were limited by infrequent water deliveries along the WM canal. Two successful tests are reported below.

On September 25, 2001, the control system was turned on after a flow increase had been manually routed through the canal via the SCADA system. Only the first 4 pools were under automatic control. Downstream water level control was implemented as a series of simple proportional-integral (PI) controllers (see Clemmens and Schuurmans 2002 for details). A tuning coefficient of $R_1 = 5$ was used to design the controller. The feedback control time step was 10 minutes and the flow control at each check structure was done every 2 minutes. The system was not at steady state when the control system was started and this caused the water levels to oscillate, as shown in Figure 4. The first two pools show considerable oscillations, while the downstream two pools remain nearly constant. Clemmens and Wahlin (2002) determined that R_1=5 is too low for this canal, and suggest R_1 =20 to avoid these oscillations. At 16:45, flow to turnout M4 was shut off (6.5 cfs or 184 l/s). This shut off was routed through the canal manually, with data generated from the scheduling software (Bautista et al. 2002). The increase in water depth at this time in pools 3 and 4 resulted from improper timing of the flow changes relative to the offtake shutoff.

On October 16, 2001, a similar test was run. In this case, a PI^{+1}_{-1} controller was implemented, tuned with R_1 = 5 (Clemmens and Schuurmans 2002). In addition to the standard PI controller, this controller sends control signals to one additional gate upstream and one additional gate downstream. Simulation studies by Clemmens and Wahlin (2002) for the ASCE test canal, which is a simplification of this canal, suggest that this controller was a good compromise between complexity and performance. (For this test, the controller was designed for a ten-minute feedback control interval, but the test was inadvertently run with a two-minute feedback control interval.) Between 11:00 and 12:00, the pressure transducers were recalibrated, creating a disturbance in the controller (Figure 4). This started oscillations in pool 1 which did not dampen quickly. The improvement in controller performance over the simple PI controller in Figure 3 is obvious.

In the spring of 2002, additional testing was done with this control system on the entire canal (8 pools) and for as long as 72 hours continuously. The results of these tests is still being analyzed.

Simulation comparison

The SOBEK unsteady flow simulation model was used to simulate these field tests (Delft Hydraulics 2000). SOBEK was chosen because this canal has many reaches with supercritical flow and few simulation models can handle both supercritical flow and user-defined canal control algorithms. Based on the conditions assumed for controller tuning, the simulation did not match the field measured conditions for the test run on September 25 (not shown). In the simulation, the water levels in pools 1 and 2 did not oscillate as much as the real canal and the jumps in the water levels in pools 3 and 4, when the off take was shut off, do not show up. Clearly, the conditions in the canal are not exactly as assumed. We changed the Manning n values in the simulation from 0.014 to 0.018 to provide untuned conditions. The results are shown in Figure 3. First, the water-level oscillations in pools 1 and 2 now more closely match the observed water level oscillations. Second, the rise in water levels in pools 3 and 4 at roughly 16:45 also show up. The oscillation patterns as observed and as simulated are similar, although there are also differences. In reality, we do not know exactly how the real canal differs from the assumed canal. Our rough guess of what adjustments to make gave us the right kind of response, even though it did not match exactly.

In Figure 4, we show the results of SOBEK simulation for the conditions of October 16, 2001. In this case, our assumed conditions seem to match the observed conditions fairly well, although more oscillations were observed in the water level of pool 1 than what was simulated. This could also be due to gate hydraulic conditions that we are not accurately representing. By using untuned conditions (not shown), we found more oscillations in all pools under simulation than as observed, again except pool 1 late in the run.

DISCUSSION

We have demonstrated that the SACMAN control system is capable of controlling water levels in an irrigation canal. The basic components are working satisfactorily within a commercial SCADA package. The Automata hardware and firmware in the field is also performing as expected. Refinements are needed to make this system more failsafe so that it can run essentially unsupervised.

Initial tests with simple controllers suggest that simple PI controllers need to be very damped for this type of canal to avoid oscillations. These oscillations are caused by inaccurate estimates of the delay times for the pools that were used for controller tuning. Since canal pool properties change over time, one cannot expect to know these delays accurately. Much better control was observed for the PI^{+1}_{-1} controller that passed control actions to one additional gate upstream and one additional gate downstream. Further research needs to be done to document the performance of these controllers under a wider variety of conditions.

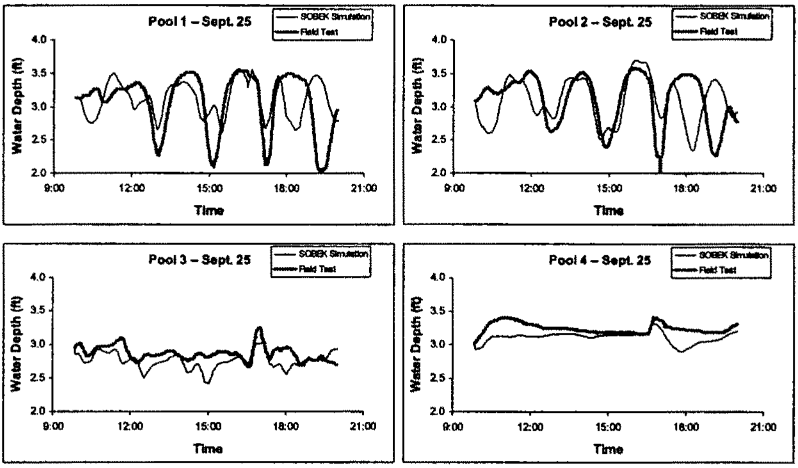

Figure 3. Field data and Sobek simulation results under untuned conditions for tests run on September 25, 2001.

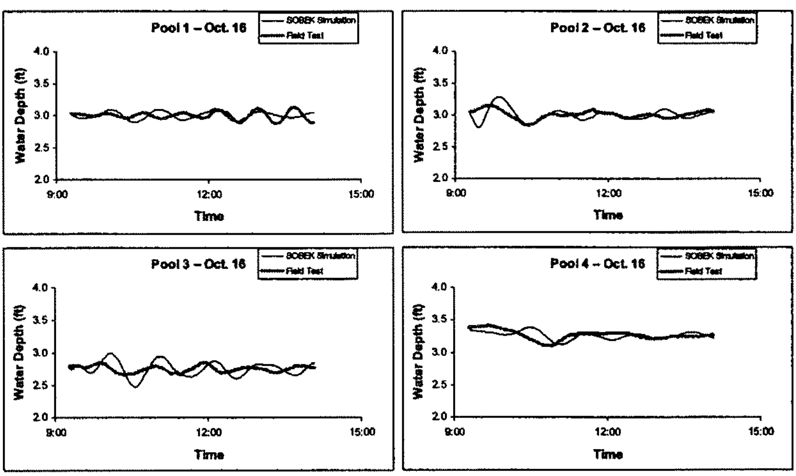

Figure 4. Field data and Sobek simulation results under tuned conditions for tests run on October 16, 2001.

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SUBSURFACE DRIP IRRIGATION APPLICATIONS IN THE HUMID REGION: STATUS OF THE TASK COMMITTEE ACTIVITIES

Michael D. Dukes[1]
Daniel L. Thomas[2]
Members of the SDI task committee

ABSTRACT

The SDI applications in the humid region task committee was formed to coordinate expertise in SDI from the humid areas of the United States. These experts have met on several occasions and formed subgroups to draft working documents focusing on SDI applications in the humid region. Draft documents are currently being developed in the areas of site selection, design, installation, and management. As these documents are created, some areas will be identified as "lacking" good information to make reasonable recommendations. A separate "research needs" document will be created to help with future research directions.

INTRODUCTION

The Task Committee on "Subsurface Drip Irrigation (SDI) Applications in the Humid Region" is organized under the Environmental and Water Resource Institute (EWRI) of the American Society of Civil Engineers (ASCE) Water Quality and Drainage Technical Committee within the Irrigation and Drainage Technical Council. The purpose of the task committee is to bring together SDI system experts, determine unique characteristics of such systems when used in the humid region, and draft documents targeted toward potential SDI users in the humid region. The primary documents will address the site selection, design, installation, and management of SDI systems in the United States. Participants involved range from university personnel and government agencies to industry and farmers. Partnering organizations include the American Society of Agricultural Engineers (ASAE) and the Irrigation Association (IA).

ACTIVITIES TO DATE

1. November 30 – December 2, 1999, Augusta, GA. Preliminary planning meeting on initial concepts and partitioning of responsibilities for workshop development team.

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[2] Professor, University of Georgia, Tifton, GA

2. August 9 – 10, 2000, Florence, SC. Planning meeting for the local arrangements team for the workshop. A visit was made to the Pee Dee Research and Education Center to determine facilities available for the workshop. An initial workshop agenda and schedule was developed.

3. November 15, 2000, Phoenix, AZ. Meeting of all interested in the SDI project effort. This meeting was used to solicit input on topic ideas, facility requirements, and travel arrangements. Outlines in major topic areas were developed. The meeting coincided with the annual Irrigation Association meeting and the Irrigation Symposium. See Table 1 for participants.

4. February 13-15, 2001, Florence, SC. Meeting of the SDI task committee at the Pee Dee Research and Education Center. Groups were formed based on site selection, design and installation, and management topic areas. Outlines initially developed at the previous meeting were further developed and specific sections were assigned to individuals for completion. The goal was to have a draft document to present at the Environmental Water Resources Institute meeting in Orlando, FL in May 2001. Funding for this activity was provided by ASCE and outside sources to reimburse participants for travel expenses. See Table 1 for participants.

5. May 20-21, 2001, Orlando, FL. Meeting of the American Society of Civil Engineers (ASCE) Irrigation and Drainage committee. Panelists from each of the four SDI subgroups presented the status of their respective documents. Feedback from the audience was solicited.

6. July 10-13, 2002, San Luis Obispo, CA. Present a paper and poster outlining the current status of the SDI in the humid region task committee.

Table 1. Participants and affiliated organizations in the SDI task committee activities.

Last Name	First Name	Organization	Meeting Attended Phoenix, AZ Nov., 2000	Meeting Attended Florence, SC Feb., 2001
Alam	Mahbub	Kansas State University		
Ayars	James	USDA, ARS	Yes	
Benham	Brian	University of Nebraska		
Bliesner	Ron	Keller and Bliesner		
Boman	Brian	University of Florida		
Boswell	John	Farmer		
Buchanan	John	University of Tennessee		Yes
Burgess	Mark	Roberts Ro-Drip		
Camp	Carl	USDA, ARS	Yes	Yes
Clark	Gary	Kansas State University	Yes	
Curtis	Larry	Auburn University	Yes	
Darrell	Virginia	Washington State Dept. Health		
Dukes	Michael	University of Florida		Yes
Edling	Bob	Louisiana State University		
Evans	Robert G.	Washington State University	Yes	
Evans	Robert O.	North Carolina State University	Yes	Yes
Fipps	Guy	Texas A&M University	Yes	
Gervais	Ken	Coastal Irrigation		
Grabow	Garry	North Carolina State University		Yes
Haman	Dorota	University of Florida		
Hanson	Blaine	University of California, Davis	Yes	
Harrison	Kerry	University of Georgia		Yes
Henggeler	Joe	University of Missouri	Yes	
Hobbs	Bryan	B. B. Hobbs Irrigation		Yes
Hook	Jim	University of Georgia		
Huffman	Rod	North Carolina State University		
Jester	Ronald	University of Delaware	Yes	
Johnson	Henry	North Carolina		
Khalilian	Ahmad	Clemson University		Yes
Lamb	Marshall	USDA, ARS		Yes
Lamm	Freddie	Kansas State University	Yes	Yes
Law	W. P.	Farmer		

Parsons	Larry	University of Florida		
Patterson	Randall	North Carolina Farmer		
Powell	Norris	Virginia Tech		Yes
Roberts	Mike	Virginia Tech	Yes	
Rochester	Gene	The Rochester Group		
Rogers	Dan	Kansas State University	Yes	
Ross	David	University of Maryland	Yes	Yes
Ruskin	Rodney	Geoflow, Inc.		
Schwankl	Larry	University of California, Davis	Yes	
Smith	Bryan	Clemson University	Yes	Yes
Smith	Vernon	USDA, NRCS		
Sorensen	Ron	USDA, ARS		Yes
Spofford	Tom	USDA, NRCS	Yes	Yes
Tacker	Phil	University of Arkansas	Yes	Yes
Thomas	Dan	University of Georgia	Yes	Yes
Thomas	Jim	Mississippi State University		
Trooien	Todd	Kansas State University	Yes	
Tyson	Ted	Auburn University		
Tyson	Tony	University of Georgia		
Vories	Earl	University of Arkansas	Yes	Yes
Warner	Richard	University of Kentucky		
Wright	F. Scott	USDA, ARS		
Yoder	Ron	University of Tennessee		
Zazueta	Fedro	University of Florida		
Total			21	18

STATUS OF THE DOCUMENT

Currently, there are four main documents being prepared concurrently. The documents focus on the following areas with the coordinator for each group,

- site selection – Carl Camp,
- design – Garry Grabow,
- installation – Kerry Harrison,
- and management – Ron Sorenson.

All four groups have developed draft documents; however, several sections are incomplete in the site selection, design, and installation areas. Once developed, these documents will lead to another document that details the research needs in SDI for the humid region.

The site selection group is working on topics in four general areas:

1. site conditions and assessment,
2. crop,
3. system operation, management, and maintenance,
4. and economic considerations.

These sections consider planning topics ranging from site topography, soils, water supply, and environmental factors. Some topics specific to the humid region are soils, water supply, and environmental factors such as off site impacts. Since SDI is relatively new to the humid region other factors that must be considered by users is availability of technical support and materials to install and maintain the system. Economics of SDI systems must be studied prior to implementation in the humid region. It is anticipated that SDI systems will exceed the cost of conventional systems (e.g. sprinkler); however, potential benefits such as lower water costs and increased yields will be explored.

The design group is working on design topics similar to other irrigation systems such as maximum daily evapotranspiration (ET) rate, pumping systems, and pipelines. Factors that must be considered in the humid region include crop water requirements, topography, crop, and location of lateral lines to avoid damage from tillage operations.

The installation group is addressing those characteristics that are most important in the actual installation of subsurface drip irrigation systems, especially as they relate to the humid region. Time of year, working with excess moisture conditions in the soil (plow pans, etc.), infield and long term documentation of buried lateral line locations (considering the impacts of adjacent tree lines on typical positioning technologies like global positioning, GPS), and equipment (available and build your own) used in the installation process.

The management group has developed a document that discusses typical management issues found in all irrigation systems such as when to irrigate and how much to irrigate. One topic identified that needs to be answered in the humid region is irrigation in one event versus irrigating several times to supply the same volume of irrigation water. The combination of sandy soils (typical of many humid area agricultural fields) with significant rainfall can lead to chemical leaching problems and trafficability impacts.

An interactive presentation was developed and copied to compact disc (CD) for distribution at the February 2001 meeting/workshop in Florence, SC. The presentation includes a database of SDI reference materials and also links to websites containing SDI information.

FUTURE WORK

Future deadlines will include further revision of existing documents and development of sections missing in current documents. In addition, people with expertise in the various areas are being solicited to participate and review the documents once they are completed. Finally, the document outlining research needs will be developed based on areas defined by development of the current documents. It is anticipated that the final documents will be published via ASCE publications and on the world wide web.

CONTACT INFORMATION

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SIMPLIFICATION OF PLANNING TO MEET FUTURE DEMANDS FOR FOOD AND WATER

George H. Hargreaves[1]
Donald T. Jensen[2]

ABSTRACT

There is now insufficient food for half of the world's population and in many areas the population is increasing faster than the food supply. Part of the current food supply is produced by depleting groundwater, overgrazing, degrading agricultural lands, and slash and burn agriculture. In order to conserve resources for future generations, these practices need to be eliminated. Countries with a good food supply also have good population stability. If food production from irrigated alluvial lands can be increased fast enough, then irrigated agriculture can stabilize population growth and greatly reduce the depletion of resources. Slash and burn agriculture results in deforestation, flooding of alluvial lands, and erosion. High priority needs to be given to eliminating this practice. This paper present methods and equations for simplifying water resource development planning and management. The World Water and Climate Atlas is briefly described. Development of a surface water Atlas is proposed. Some of the benefits that have resulted from the construction of large dams and the irrigation of alluvial lands are described.

INTRODUCTION

There is increasing concern relative to the world's future supplies of food and water. Within a 30-year period (1970 to 2000) the world population increased 62 percent while grain cropland per person declined 36 percent. Storm damage increased 40 fold. (See World•Watch, March/April, 2000.)

Rangelands are overgrazed and degraded. Some large areas of grain lands have gone out of production due to erosion. Grain comprises 80 percent of the world's food chain. Food production from slash and burn agriculture needs to be decreased and gradually eliminated. Increased production from irrigated agriculture can reduce resource degradation.

Unfortunately, perhaps 20 to 25 percent of presently irrigated lands are irrigated by overpumping ground water. As water tables decline, much of these irrigated

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areas will go out of production. An estimated 29 percent of irrigated lands are subject to salinization, (Suarez 2002). Some of these lands may be reclaimed but others will be taken out of production permanently.

In many developing countries, increasing use is made of slash and burn agriculture. This practice is very destructive of many natural resources. Erosion and flood damage are increased. Valuable timber is destroyed. The smoke from the fires interrupts transportation and commerce and increases health problems.

Prescott-Allen (2001) indicates that, based on 1996-97 data, 50 percent of the world's population have insufficient food and only 10 percent are indicated to have a good food supply. Food sufficiency is rated based on the World Health Organization targets. There should be less than 20 percent prevalence in the stunting of children and no more than 10 percent for the prevalence of low birth rates when food is adequate.

Prescott-Allen (2001) proposes the objective of a stable population. Of 180 countries rated for stability of population, 60 are rated as good and 42 as bad. Most countries with a good or fair food sufficiency are also rated as having good or fair population stability. Afghanistan has the lowest rating of food sufficiency (27 percent) and has poor population stability.

Almost one fifth of the world's population lives on less than $1.00 a day, (World Bank, 2001). Almost half of the world's population lives on less than $2.00 a day and millions, mainly children, go to bed hungry at night (Wolfensohn 2000). In China, India, and 54 other countries there is not enough material goods and income to secure basic needs and decent livelihoods, (Prescott-Allen, 2001).

There is a need to conserve resources for use by future generations. Slash and burn agriculture, overgrazing, overpumping of groundwater, and the degrading of agricultural lands should be reduced and if possible eliminated. It is proposed that the rate of irrigation development be accelerated to facilitate the reduction of production from sources that are non renewable or damaging to the environment. This will require an increase in water storage facilities and in transbasin diversions. In a period of rapid technological advances it would be irresponsible to fail to meet these needs. Good policies for resource use and conservation will also reduce violent conflicts (Renner, 2002).

The ecosystem wellbeing index is not rated as good for any country but is rated as bad or poor for 40 countries. Land and water use are not good in any country. Land use is bad or poor in 58 percent of the countries and water use is bad in 47 percent (Prescott-Allen, 2001).

The low-income countries with more than 40 percent of the world's population are attempting to industrialize. If they were to now have CO_2 emissions per capita equal to that of the United States it would more than double the worlds CO_2 emissions. Considering projections of population increases and increases in Co_2 emissions from the other countries, the potential harmful effects are cause for serious concern. There is urgent need for a careful evaluation of these effects. Also the high-income nations should assume an obligation to use their wealth and technology to help low income countries develop without depleting or polluting resources and without doing harm to the world's ecosystem wellbeing.

The objective of this paper is to call attention to an approaching food and ecosystem crisis and to present suggestions of methods that are designed to simplify and facilitate planning for the future sustainable development of land and water resources.

METHODS AND EQUATIONS

A moisture adequacy index (MAI) was developed for use by the Inter American Geodetic Survey (IAGS) in natural resource inventories. Reference crop evapotranspiration (ET_o as defined later) is used as the index of water requirements. The 75 percent probable rainfall amounts (P_{75}) are compared with ET_o. The equation for moisture adequacy is:

$$MAI = P_{75}/ET_o \qquad (1)$$

P_{75} can be computed from the mean rainfall (Pm) and its standard deviation (SD). The relationship is:

$$P_{75} = Pm - 0.74\ SD \qquad (2)$$

Values of MAI have been used for studies of rainfall adequacy for many countries. The usual time step has been monthly, although some studies have also used 10-day and shorter time steps.

Hargreaves (1975) related relative water adequacy (X) to relative yield (Y) where Y = 1 for the maximum yield under prevailing conditions and X = 1 for the water required for maximum yield. The equation is:

$$Y = 0.8\ X + 1.3\ X^2 - 1.1\ X^3 \qquad (3)$$

Reference crop evapotranspiration (ET_o) as used in this paper is as defined in FAO Irrigation and Drainage Paper 56 (Allen et al. 1998). Hargreaves (1977) found that ET_o could be calculated from mean temperature (T_m) and solar radiation (R_s). The equation in degrees Celsius is:

$$ET_o = 0.0135\ R_s\ (T_m + 17.8) \qquad (4)$$

Equation 4 was used in various country-wide studies and by Utah State University in two worldwide studies. Due to the paucity of reliable measured data for R_s, data from the Atlas by Lof et al. (1966) were used. Wu (1997) evaluated (4) and recommended that this equation be used for irrigation scheduling.

R_s can be computed from the temperature range (TR) and extraterrestrial radiation (R_a) where TR is mean maximum minus mean minimum temperature. By combining the equation for R_s with (4) the 1985 Hargreaves ET_o equation was derived. The equation is:

$$ET_o = 0.0023\ R_a\ (T_m + 17.8)\ TR^{0.50} \tag{5}$$

where T_m is in degrees Celsius and ET_o and R_s are in the same units of equivalent water evaporation.

Equation 5 has been evaluated by comparisons with the FAO Penman-Monteith ET_o method with data from various well watered sites. For a time step of five or more days the comparisons are very acceptable. Equation 5 has been recommended for use by ASCE and FAO (Allen et al. 1996 and Allen et al. 1998). Unfortunately, various attempts to evaluate (5) have been made using data from various non-irrigated sites. Some of these have included data from several dry months. Comparisons using data from both irrigated and non-irrigated sites in the same climate indicates that computed values for the dry months are biased by aridity. The FAO-Penman-Monteith values may be biased as much as 30 percent. Those from (5) are biased also but only about half as much. Therefore, data from non-irrigated arid and semi-arid sites should not be used for calculating ET_o by means of the FAO Penman-Monteith method, unless adjustments of the data can be made to correct for site aridity.

Hargreaves (1988) reviewed the literature on extreme rainfall distributions. Various researchers have reported a worldwide similarity of distributions. Extreme rainfall events vary in depth (D) with the fourth root of the duration (t) and the return period (T). In equation form:

$$D = K\ (t \times T)^{0.25} \tag{6}$$

As a result of numerous comparisons with the Gamma probability distribution, it was found that the 20 year return period rainfall amount (P_{05}) can be calculated from the mean (M) and the standard deviation (SD). The equation is:

$$P_{05} = M + 2.05\ SD \tag{7}$$

Equation 7 can be used for events other than rainfall.

WORLD WATER AND CLIMATE ATLAS

The Atlas was developed jointly by the International Water Management Institute (IWMI) and the Utah Climate Center at Utah State University with funding from the Government of Japan and with cooperation from other agencies and institutions. Global coverage is provided at 2.5-minute grid spacings. The data used are for the period 1961-1990. Annual, monthly, and 10-day summaries are given. The temperature parameters are: average, mean daily maximum, and mean daily minimum. The precipitation parameters are: total (P), 75 percent probability (P_{75}), number of days with precipitation (DWR), and the standard deviation (SD). The agricultural parameters are: ET_o (from 5), MAI, and NET ($NET = ET_o - P_{75}$).

Information on how to use the Atlas can be obtained from IIMI@cgnet.com and the Atlas is available on the Internet at http://www.iwmi.org, http://www.cgiar.org/iimi, or http://climate.usu.edu. Hargreaves and Merkley (1998) describe the Atlas and some of its uses.

A digital elevation model (DEM) is available for use with the Atlas. For most of the world the interval is 30 arc seconds. Also, most of the world has been mapped at a scale of 1:50,000 with a 20 m contour interval. Sometimes use of the topographic maps is restricted because of the military value of these maps. However, the maps are usually available to responsible individuals provided the benefits from their use are properly documented. The DEM is available from the U.S. Geological Survey as described by Lacroix et al. (2000).

CLASSIFICATION OF CLIMATE

Hargreaves and Merkley (1998) prepared a summary of optimum and operative temperatures for five crop groups. Most crops are in two groups. One has a minimum operative temperature of 5°C and the other of 10°C. Hargreaves (1977) proposed two classifications of climate. One is based on temperature and the other on the Moisture Availability Index (MAI). The MAI classification is presented as Table 1.

Some judgment is required in using Table 1 since adequate moisture at low temperatures does not promote plant growth, except when stored in the soil and used later.

The annual depth of runoff for each of the seven MAI classifications was calculated from long-term averages of surface runoff for the United States. The values shown in Table 1 are averages. There was significant variation in runoff for each classification. This may result from differences in temperature, land use, soil depth texture and permeability, vegetation, and rainfall distribution and intensity.

Hargreaves et al. (2001) made some comparisons using runoff records from tropical countries. Where comparisons were made, the runoff from tropical watersheds was more than the averages obtained for the runoff data from the United States.

Table 1. MAI-Based Climate Classification and Average Annual Runoff

Climate Classification	MAI Criteria	Water Constraints on Productivity	Average Runoff in mm
Very Arid	All months with MAI □ 0.33	Not suited for rainfed agriculture	15
Arid	1 or 2 months with MAI □ 0.34	Limited suitability for rainfed agriculture	35
Semi-Arid	3 or 4 months with MAI □ 0.34	Suitable for crops requiring a 3 to 4 month growing season	120
Wet-Dry	5 or more consecutive months with MAI □ 0.34	Suitable for crops requiring a 5 or more month growing season	200
Somewhat Wet	1 or 2 months with MAI > 1.33	Natural or artificial drainage required	290
Moderately Wet	3 to 5 months with MAI > 1.33	Good drainage required	440
Very Wet	6 or more months with MAI > 1.33	Very good drainage required	935

A STREAM FLOW ATLAS

A world Atlas of runoff from watersheds would be very useful for purposes of development planning. A summary should be made of existing records. This should include 10-day, monthly, and annual means and standard deviations. Various methods for estimating runoff from climate could be tested and evaluated. Estimates could be made for unmeasured watersheds. IWMI has indicated interest in finding ways to estimate runoff from climate data.

DAMS AND DAMSITES

Almost half (three billion) of the world population live on less than $2.0 a day and millions, especially children, go to sleep hungry every night (Wolfensohn, 2000). Half of the world population has insufficient food (Prescott-Allen, 2001). What would the world be like without food production made possible by existing dams? Some become emotional about the beauty of wild and scenic rivers. Sometimes the benefits of dams to the environment are overlooked.

In Greece, the Truman Doctrine (American Aid for Greece and Turkey) provided assistance for the construction of large dams and appurtenances for flood control. These activities lead to rural electrification and a rate of development that increased the economy four-fold in 20 years. Herding of goats, cutting of firewood, and farming on steep slopes became uneconomical. This made possible the reforestation of hundreds of thousands of hectares of eroding mountains.

In Brazil, during the 1960's and 70's the air in the large cities became very polluted. This is also a problem in many of the world's large cities. Then in Brazil large dams were constructed and large areas were planted to sugar cane. Sugar cane was used to produce alcohol and alcohol was substituted for gasoline. The change from fossil fuels to energy from alcohol and hydropower cleared the pollution in the cities. This improved the environment in Brazil and reduced the emissions of greenhouse gasses thereby providing worldwide benefits.

In California, as dams and flood control facilities were constructed and valley lands were irrigated, there was a resulting rapid decline in the non-irrigated farming of steep hillsides and mountainsides. Dams facilitated soil conservation and erosion control.

Slash and burn agriculture is increasing in many countries. Data are scarce on the extent and the harmful effects of this practice. A careful evaluation is needed. Dams for flood control and the irrigation of alluvial lands could provide a valuable alternative to slash and burn agriculture.

There is evidence that not to reduce carbon dioxide emissions would truly damage the global economy. The nuclear option is of highly questionable viability. Of all other non-carbon alternative energy sources, low-head waterpower from small dams is the least expensive. The next least expensive is wind (Ayres, 2001). Wind is now becoming competitive with coal and oil. It is not only a clean energy source but is a rich source of new employment (Renner, 2001).

In Brazil, during the 1960's, USAID provided assistance by arranging for studies made by the US Bureau of Reclamation. Dam sites and resources were investigated. These studies resulted in significant development. Fourteen dams were constructed in the State of Bahia. For these USAID furnished the services of an engineer. Half of the labor was paid in food from the Food for Peace Program. The U.S. Army donated the construction equipment from surplus and the Brazilian Army operated the equipment as a training exercise. This type of development could be included in the planning for food sufficiency. It would seem desirable for use in countries critically short of food such as Afghanistan.

The DEM and available topographic maps can be used to identify potential damsites. Priority should be given to locating economically feasible off stream sites and to locating suitable transbasin diversions form wet areas to arid areas. Sites for dams should be selected so as to minimize flooding of alluvial lands and displacement of people. A world inventory of suitable dam sites is recommended.

SOME ADAPTATIONS OF METHODS

The moisture availability index (MAI) has been used to determine the level of water adequacy required for the use of fertilizer to be economical. IWMI (1999) mapped MAI for South Asia and related monthly values to the relative yield equation (Eq. 3). MAI has been used in Iran and El Salvador to determine when and where it is economical to apply fertilizer.

Equations 6 and 7 can be used with the Atlas values for 10-day rainfall amounts. Twenty-year return period extreme depths of precipitation for a duration of 10-days can be converted to other durations and return periods by means of Eq. 6.

The Atlas and Table 1 have been used for determining areas of excessive rainfall that may be used in transbasin transfer to irrigate arid and semi-arid lands. The Atlas can be also used for determining irrigation requirements, needs for drainage, probable flood risks and for many other uses.

SUMMARY AND CONCLUSIONS

The world's population is increasing. About half of the present population lack sufficient food. Rangelands are overgrazed. Cropland is being degraded. Most of the required increase in food production must come from irrigated lands. However, due to overpumping of groundwater and salinity irrigation is not sustainable on a significant portion of the presently irrigated lands.

Methods and equations are given in order to facilitate planning for future increases in irrigation and in food production. These include a rainfall adequacy index (MAI), a water-related relative yield equation, and equations for extreme rainfall depths of fall.

Use of a MAI-based classification of climate is proposed. This classification is recommended over other classifications because it relates to the potential for food production and to water availability from rainfall for crop production. It can be used to estimate values of annual runoff and to determine when and where it is profitable to fertilize non-irrigated crops.

A world streamflow Atlas is proposed. This Atlas would be a summary of available long-term runoff records and would include correlations and estimations based on the relationships between climate and runoff.

About half of the world's population lives on less than $2.0 per day and millions, especially children, go to bed hungry every night. The population of many developing areas is increasing faster than the food supply. Groundwater resources are being depleted. Various farming practices are destructive of resources. Most of the required increase in food production must come from increases in irrigated area. When consideration is given to the competing demands for water, there seems to be no viable alternative to storing and transporting more water.

A world inventory of possible new damsites is proposed. There has been far too much emotion relative to the construction of dams. Increasing effort should be made to mitigate possible adverse effects of dams. However, dams now have a very important role in food production, environmental protection, and in the production of clean energy. There have been large benefits to the environment from the positive effects from dam construction. If priority can be given to minimizing the flooding of alluvial lands and the displacement of people and to finding off-stream sites and more efficient means of allowing fish to pass through or around dams, there will be less opposition to their construction.

The present food shortage is to an important degree due to the increase in population. However, a good food supply stabilizes population. By increasing the area under irrigation and increasing the food supply the world's population can be stabilized.

In Brazil, renewable energy was developed to replace much of the use of fossil fuels. A worldwide emphasis on research and development of renewable energy is needed. The world's leading economics should provide leadership in this effort. This effort could have a large positive effect on the future environment.

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IMPACT OF SCAVENGER WELLS ON CROPS AND GROUNDWATER TABLE

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Abdul Khalique Ansari [2]

ABSTRACT

Scavenger well extracts fresh and saline water through dual pumping, resulting in lowering the water table and maintaining the equilibrium between both aquifers.

This study was under taken to analyze the impact of scavenger wells on extent of water table control, cropping pattern, crop yield and land utilization in Irrigation subdivision Khadro Sanghar Sindh, Pakistan.

The results show that the water table has decreased at least 132 cm below the surface in most of the study area. Field survey results show that crop yield has increased 50% for cotton. Abandoned land has dropped from 78% to 28% of Culture able Command Area (CCA), which are under trenches, uneven topography and saline drains. As a result the land value has increased by six to ten times at many places.

INTRODUCTION

Irrigation is considered to be the lifeblood of agriculture, as 90% of agricultural production is being produced from irrigated lands in Pakistan (1).

Along the major canals in the Nawabshah and Sanghar areas, there are extensive areas where a layer of fresh groundwater overlies deeper saline water. The problem of extracting the fresh water is not new. It is conventionally tackled by using skimming wells. The drawback of extraction in this way is that only a very small capacity well should be used in this way. A second method is to draw normal discharge and accept that when the well becomes saline, it must be abandoned and a new well drilled. This has been done by farmers in area and requires drilling a new well after three years or so (2).

Scavenger wells can be effective in handling this problem. Scavenger well abstracts groundwater from both fresh and saline aquifers as shown in Figure 1.

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This results in lowering of groundwater levels. Fresh water can be used to rehabilitate saline land by providing the fresh water for leaching of salts from soil profile and Irrigation for later cropping. Where as saline water is discharged to saline drains. Abstraction from both aquifers prevents the excessive up conning of the interface and maintaining equilibrium.

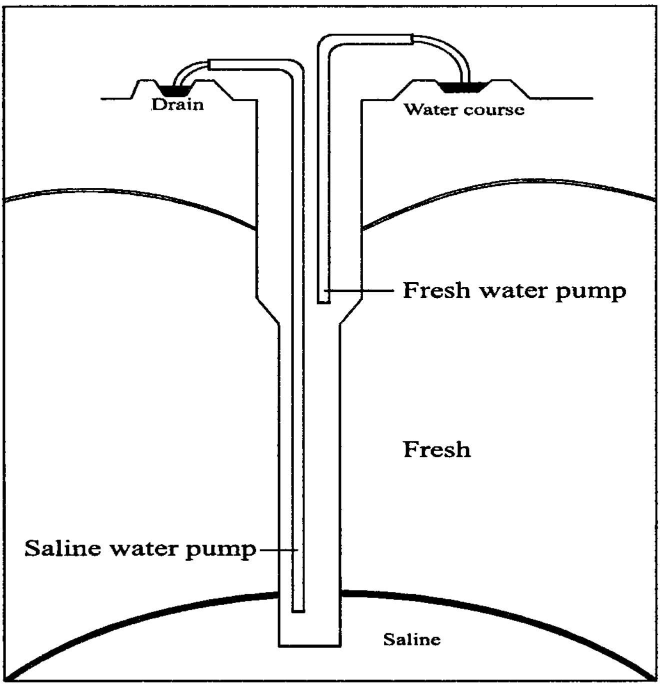

Fig. 1: Schematic scavenger well

The main environmental impact of scavenger wells is recovery of abandoned land. Early monitoring of scavenger wells has indicated a significant lowering of groundwater levels and recovery of about 30,000 ha of land in the Nawabshah District (3).

A series of 189 scavenger wells were installed in Nawabshah while construction of 175 scavenger wells in Sanghar is in progress.

METHODOLOGY

Eight scavenger wells starting from JRS-43 to JRS-50 as shown in Figure 2 were studied in Irrigation subdivision Khadro. Except JRS-47, all studied scavenger

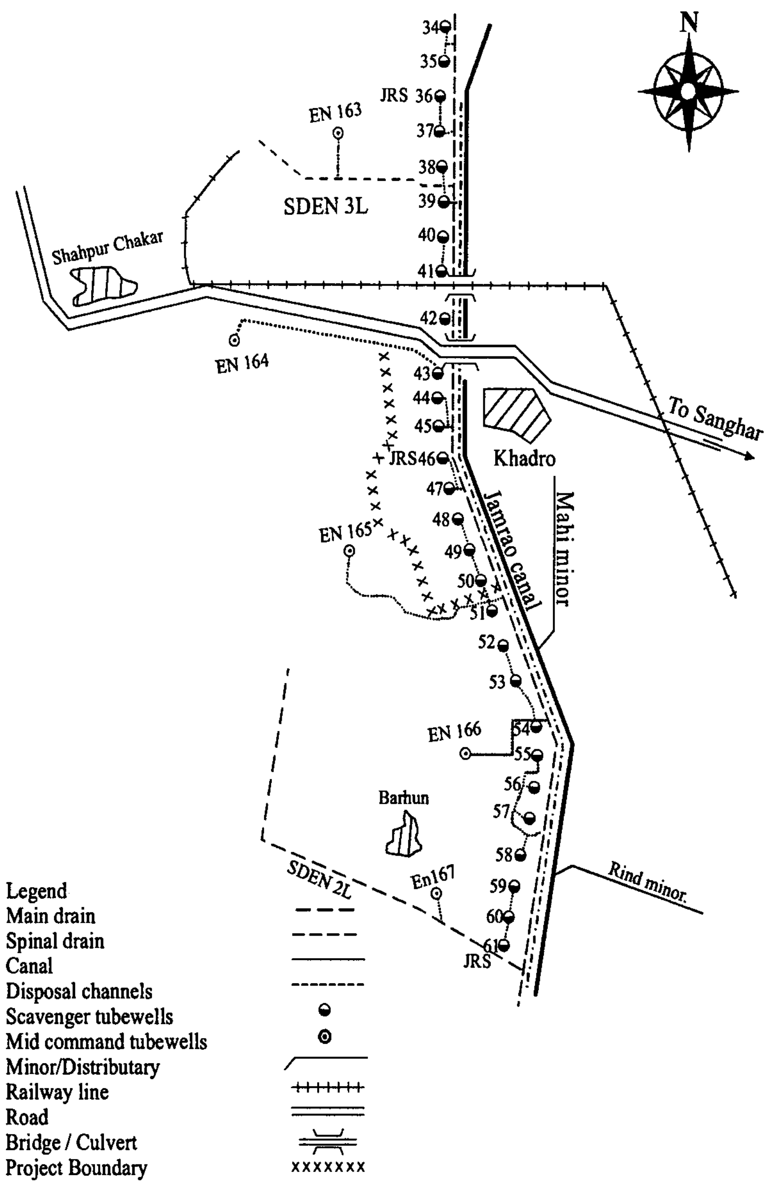

Fig. 2: Location map showing project area under investigation.

wells had both fresh and saline water outlets. The following methodology was used to study the impact of scavenger wells on cropping pattern, crop yield, land used and elevation of groundwater table in the study area. Methodology included as under

- Experimental work
- Field data analysis
- Project area survey (through questionnaire)

EXPERIMENTAL WORK

An auger was used to drill observation holes to measure the elevation of the water table. The groundwater table depth was recorded in equilibrium condition. Observation holes were drilled with the help of a pushing type auger; on the second day the water table depths were measured. Measurements were recorded on month basis from July 1998 to January 1999.

RESULT AND DISCUSSION

Under Stage 1 of the Left Bank Outfall Drain (LBOD) program, 189 scavenger wells were installed in Nawabshah component (Nawabshah and Sanghar) from 1993 to 1995. Out of these, 79 scavenger wells were installed on the right bank of Jamrao called Jamrao Right Scavengers (JRS). In this study the eight scavenger wells starting from JRS-43 to JRS-50 as shown in Figure 2 were studied in Irrigation subdivision Khadro.

Table 1. Water table Depth (cm) in Project Study Area

S. No	JRS No.	Distance of well from canal(m)	July 1998	Aug 1998	Sept 1998	Oct 1998	Nov 1998	Jan 1999	Aver-age
1.	JRS-43	370	132	134	130	133	137	140	134.33
2.	JRS-44	160	137	134	135	136	138	141	136.66
3.	JRS-45	430	167.5	164.75	166	162	165	168	165.54
4.	JRS-46	480	142.25	143	141	146	143	147	143.71
5.	JRS-47	220	147	146	144	142	149	170	149.66
6.	JRS-48	500	167	164	162	165	164	170	165.33
7.	JRS-49	310	114	116	113	115	112	116	114.33
8.	JRS-50	350	127	125	129	128	130	132	128.50

Water table Depth (cm)

The data pertaining to post project water table depth recorded from various points during this study (July to November 1998 and January 1999), are shown in Table 1.

It can be seen that the water table depth is deepest at JRS-45 and 48, while highest in the JRS-49 area. Water table depths in JRS-45, 47 and 48 areas are in safe range, as required for major crops (Table 2). Although the water table depths in areas surrounding JRS-43, 44 and 46 are decreasing steadily towards safe range. Areas around JRS-49 and 50 may take one year or more to reach the desirable depth, even if meteorological conditions go in favour.

Table 2. Effect of different water table depths on crop yields

Water table depths [cm]	Crop yield decrease (%)					
	Cotton	Sugarcane	Wheat	Barseem	Summer Fodder	Mango
0-25	98	91	79	77	80	100
25-50	57	66	49	45	27	100
50-75	35	46	28	24	0	100
75-100	21	29	13	9	0	87
100-125	12	10	5	2	0	63
125-150	5	5	1	0	0	38
150-175	0	0	0	0	0	0

Source: Water Sector Investment Planning Study, Vol 1 [4]

Only one piezometer was located in the project study area (JRS-43 to JRS-50). This piezometer was near JRS-49 area. It is reported that during the preproject period (1988-1995), the water table measured in October was as high as 78 cm in 1994 and as low as 146 cm in 1988. In the postproject period (1997) the Scarp Monitoring Organization (SMO) found the water table at 220 cm, reflecting that the water table depth has drastically dropped from 1994 position i.e. 78 cm. However the water table was found 115 cm in October1998, which shows that the water table has not lowered as much due to scavenger wells as found by SMO. This may be due to the fact that SMO piezometer was located in between scavenger well and spinal drain and it does not represent the area.

FIELD DATA ANALYSIS:

Impact of Scavenger Wells on Land Use

The land use of the study area at Khadro has been found for the total command area of two watercourses HB (Hamzo Baghrani)/2R (Right) and LS (Laski)/1R (Right) in deh Hamzo Baghrani, Laski and Kulan. Which were 432 ha. Due to the rising groundwater table and associated salinization of non-cropped areas, the amount of abandoned land was increasing and cultivated land was decreasing. As a result farmers were forced to grow crops on their best land. Table 3 indicates the land use of pre and postproject period. Land cultivation data were obtained from Revenue and Irrigation Departments record for the period of 1989-1998. The Kharif (Summer) crop for 1998 was observed in the present study.

Table 3. Land use (ha) in Project Study Area from 1989 –1998

S.No	Year	Land Under cultivation		Land Abandoned	
		ha	%age of CCA	ha	%age of CCA
1	1989*	89.62	21.96	318.38	78.04
2	1991**	83.50	20.46	324.50	79.54
3	1995**	163.84	40.16	244.16	59.84
4	1996**	211.84	51.92	196.16	48.08
5	1997**	252.50	61.89	155.50	38.11
6	1998**	283.12	69.40	124.88	30.60
7	1998***	292.58	71.71	115.42	28.29

* Revenue Data
** Irrigation Department Data
*** Present study

Before the installation of scavenger wells (1989), only 22% of CCA was under cultivation and about 78% of the land was abandoned due to waterlogging and salinity. The deterioration of land was under way. After the installation of scavenger wells, groundwater table lowered to a significant depth, which results in recovery of abandoned land. Results show that cultivated land area is increasing and abandoned land area is decreasing. By the end of 1998 cultivated land had increased from 22% to 72%. Table 3 shows that abandoned land were only 28% in 1998. This 28% of the project study area was under trenches, saline drains, and irregular topography. Some landlords in the areas had sold their land to contractors for dredging of soil for embankment of Jamrao and spinal drain.

Impact of Scavenger Wells on Cropping Pattern

A crop survey of Kharif (1998-1999) was made in the project study area; Figure 3 shows the preproject (1989) and post project (1995-1998) cropping pattern. In the preproject period (1989), cotton, rice, fodder, garden and other crops were cultivated on 22.10, 48.47, 10.11, 5.74 and 3.20 hectares, respectively. While in the postproject period cotton, rice, fodder, garden, chilies, and other crops were cultivated on 219.30, 5.60, 41.65, 14.87, 1.65 and 9.51 hectares, respectively.

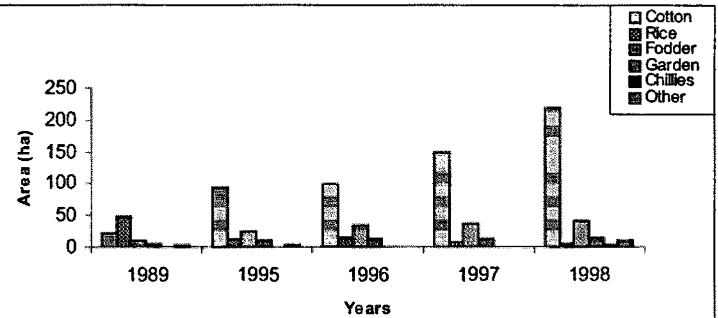

Figure 3. Cropping pattern of Kharif in project study area.

Figure 4 shows that the preproject (1990-1991) and post project (1997-1998) crops of Rabi (Winter). This shows that in preproject status, wheat, oil seeds, fodder and garden were cultivated on 56.29, 5.86, 15.97, 5.38 and 2.33 hectares, respectively.

It can also be seen in this Figure that in the postproject of Rabi season (1997-1998), wheat, oilseeds, fodder, garden, onion and other crops were cultivated on 204.17, 33.65, 25.55, 13.34, 3.17 and 3.24 hectares, respectively. It is obvious from the results that scavenger wells had improved the land for cultivation by dropping the water table to a desirable depth and reducing the soil salinity to safer levels. It was further observed that cropped areas in post project period were increasing compared to preproject level.

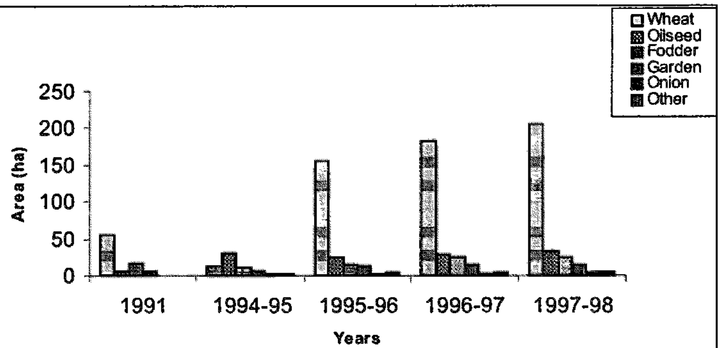

Figure 4. Cropping pattern of Rabi of project study area.

PROJECT AREA SURVEY RESULTS

Abandoned Land

The project area survey results show that percentage abandoned land of individual holdings was 25% or more in most cases. Previously reported value of abandoned land, as 28% seems reasonable.

Scavenger Wells Operation and Responsibility

Respondents reported most of Scavenger wells functional and operational responsibility was shared by Water and Power Development Authority (WAPDA) and Irrigation Department. Fresh water from scavenger wells is only used for Irrigation purposes. There is no awareness or willingness to use this precious source of fresh water for drinking purpose although it meets the quality standards. In some cases this water is drained out in to the saline drains. The problem of weeds was commonly reported by respondents, which cause restriction in the flow of saline water and hence flooding of the nearby land.

Land value

The field survey data shows that the minimum preproject land value was in the range Rs.500 to Rs.2500 rupees per hectare. The majority of respondents however reported that land value was high and fluctuated between Rs.2500 to Rs.7500 per hectare in preproject period. The lowering of water table and leaching of salts at deeper depths due to scavenger wells and subsequent Irrigation have increased the land value in the postproject period. The majority of respondents reported that, at present, land value was about Rs.87, 500 to Rs150, 000 per hectare, while maximum land value reported was Rs.200, 000 or more per hectare.

Impact of Scavenger Wells on Crop Yield

The field survey results show pre and postproject crop yield of project study area. Before the installation of scavenger wells crop yield was very low. The majority of respondents reported that yield of the cotton crop was between 0.48-0.8 tons per hectare, yield of rice was 1.0-1.5 tons per hectare, and yield of wheat was 0.5-1.5 tons per hectare. After functioning of scavenger wells, landlords had cultivated cash crops on most of the cropland. The crop yield had also improved, up to 1.0-2.0 tons for cotton. These results seem realistic against Agriculture Extension Department data shown in Table 4.

Table: 4 Crop yield

S.No.	Year	Crops (Ton/ha)		
		Wheat	Rice	Cotton
1.	1990-91	2.27	1.53	0.31
2.	1991-92	2.41	1.54	0.43
3.	1992-93	2.19	0.89	0.32
4.	1993-94	2.11	1.89	0.45
5.	1994-95	2.11	1.56	0.52
6.	1995-96	2.07	2.27	0.59
7.	1996-97	2.08	2.35	0.65
8.	1997-98	2.56	2.13	0.72
9.	1998-99	2.58	2.03	2.63

Source: Sindh Agriculture Extension Department, Pakistan.

CONCLUSION

The study resulted in following conclusions:

1. Generally scavenger wells have lowered the water table below 132 cm, except for the area around JRS-49, where the water table has dropped to115 cm (not as much as 220cm as claimed by WAPDA).

2. It has been estimated that the crop area in the project study area has increased from 22% to 72%.

3. Survey results showed that crop yield of cotton and wheat in the project study area has also increased significantly. As a result the land value has increased at least by six to ten times.

ACKNOWLEDGEMENT

The authors wish to express the sincere thanks to Miss Saira Syed Senior School Teacher Army Public School Chhor cantonment for her help in preparing Figures and graphs.

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CONSUMPTIVE USE PROGRAM (CUP) MODEL

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ABSTRACT

For many years, published crop efficient (K_c) values have been used to estimate crop evapotranspiration (ET_c) from reference evapotranspiration (ET_o). The Consumptive Use Program (CUP) was developed to improve estimates of K_c and ET_c values to aid in California water planning. CUP computes ET_o from mean monthly values for solar radiation, maximum, minimum, and dew point temperature, and wind speed. From this, the program uses a curve-fitting technique to produce daily ET_o and rainfall data for a year. Bare soil evaporation is used to estimate K_c values for the off-season and as a baseline for early season K_c calculations. One improvement is to account for the influence of rainfall and/or irrigation frequency on K_c and ET_c during initial growth. For tree crops, it is important to account for cover crops, which has not been done in the past. Another improvement is to compute and apply all ET_o and K_c values on a daily basis to determine crop water requirements. Using new information on midseason K_c values and bare soil evaporation, a user-friendly Excel program, CUP, was developed to improve long-term estimates that account for rainfall, cover crop, and immaturity effects. This paper presents the advantages of CUP.

INTRODUCTION

A user-friendly Microsoft Excel computer program, CUP was developed to help growers and water agencies determine crop coefficient (K_c) values and crop evapotranspiration (ET_c). California needs long-term estimates of ET_c for water planning. CUP, written by M. N. Orang with assistance from R. L. Snyder and J. S. Matyac, was designed to account for factors affecting K_c that are generally ignored in other water requirement programs. For example, the program provides estimates of nearly bare soil evaporation during initial growth of crops based on ET_o rates and irrigation or rainfall frequency. Because California growers do not

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use metric units, CUP's input and output data are in English units. However, in the near future, the program will be converted to metric and translated into Spanish.

WORKSHEETS

CUP has 14 Excel worksheets. The first six worksheets are "Disclaimer," "HelpAbout," "HELP," "ET_o Zones Map," "ET_o Zones," and "Weather Input." "HelpAbout" provides information about the program. "HELP" explains the components of the program and provides step-by-step instructions for inputting data into the program. "ET_o Zones" contains a map showing 18 zones of similar ET_o rates for California. The map was developed by D. Jones, R.L. Snyder, S. Eching, and H. Gomez-MacPherson. The "Weather Input" worksheet is used to input monthly mean weather data for calculating ET_o.

ET_o and crop data are entered into the "Input_Output" worksheet, which then displays the summary of inputs and monthly and seasonal outputs. The "Calculations" worksheet shows all of the growth dates and K_c values as well as the daily calculations of ET_o, K_c, and ET_c for each of the growth periods. The "K_c Chart" shows a plot of the calculated seasonal crop coefficients with colored lines representing each growth period. Charts "RainInitGrow" and "IrrigInitGrow" show plots of K_c versus mean ET_o rates. The chart "RainInitGrow" shows the nearly bare soil K_c corresponding to the mean initial growth period ET_o rate and soil wetting frequency by rainfall. The chart "IrrigInitGrow" shows the nearly bare soil K_c corresponding to mean ET_o rate and soil wetting frequency by irrigation.

Rainfall or irrigation wetting frequencies are entered into the "Input_Output" worksheet, and the charts are automatically displayed. If the K_c resulting from surface wetting is bigger than the table value during early crop growth, the larger K_c value is used. There are also summary worksheets for K_c values, ET_o, and ET_c. After data entry, the current crop information and calculated K_c data in the "Input_Output" worksheet can be printed to one row in the "Summary of K_c" worksheet. ET_o data are printed to "Summary of ET_o," and ET_c data are printed to :Summary of ET_c." The chart "ET_o_ET_c" shows a bar graph of ET_o and ET_c totals by month for the current crop information. The "Crop References" worksheet contains estimated growth date and K_c information, which are used as default values in the program.

INPUT_OUTPUT WORKSHEET

Crop information is entered into cells on the left-hand side of the 'Input_Output' worksheet. To enter pan data, 77 is input into the ET_o Zone number cell; to use monthly weather data, 88 is entered into the cell; and to enter raw ET_o data, 99 is input. Next a crop number is entered into the Crop Number cell. CUP provides a list of crops and crop numbers in the 'Crop References' worksheet. That worksheet also contains the percentage of the season to various growth dates (explained later), K_c values at critical growth points, and sample start and end dates for the season.

Note that the crop numbers have one digit to the left and two digits to the right of a decimal point. The single digit identifies the crop type, and the double digit identifies the crop. When a crop is selected, the growth, K_c value, and default start-end information is automatically used for the calculations. The start date corresponds to planting for field and row crops and to leaf-out date for tree and vine crops. Nondeciduous trees, turfgrass, and pasture crops start on January 1 and end on December 31. If different from the default values, the start and end dates can be changed in the "Input_Output" worksheet.

Initial K_c value for most crops is affected by and, thus, depends on wetting frequency from rainfall and/or irrigation. As canopy shading increases, the contribution of soil evaporation to ET_c decreases while the contribution of transpiration increases. In the "Input_Output" worksheet, the rainfall frequency during early growth is entered to determine a K_c value for nearly bare soil evaporation. Similarly, irrigation frequency is entered and a K_c value determined for nearly bare soil evaporation during initial growth of field and row crops. CUP compares K_c values from the "Crop References" worksheet with those based on rainfall and irrigation frequency and selects the largest of the three for use in calculating ET_c. If no rainfall or irrigation frequency is entered, the K_c value from the A-B column in the "Crop References" worksheet is used as the initial growth K_c value. The starting K_c value for type-2 crops (for example, turfgrass and pasture) and for type-4 crops (for example, subtropical orchards) is not affected by irrigation or rainfall frequency entries.

Cover crops affect ET_c rates, and CUP accounts for the effects. Cover crop start and end dates are entered into cells under "Enter 1st Cover Crop (day/mon)." Because some crops have cover crops in spring and fall but not in the summer, a second set of cover crop dates can be input under "Enter 2nd Cover Crop (day/mon)." During a period with a cover crop, the value 0.35 is added to the "clean cultivated" K_c value. However, the K_c value is not allowed to exceed 1.15 or to fall below 0.90.

The right-hand side of the "Input_Output" worksheet shows the weighted mean K_c, ET_o, ET_c, and seasonal ET_c values by month for the selected crop and input information. The daily mean ET_o rates by month are also shown below the other data. Below that set of cells, there are "Copy/Paste" and "Delete" buttons. When the "Copy/Paste" button is pressed, results of the calculations are sent to "Summary ET_o," "Summary K_c," and "Summary ET_c" worksheets. The "Delete" button clears all entries from the summary worksheets. To retain all of the data entries, save the CUP file as an Excel workbook with a different name. To save only the summary sheets, display the summary sheet and save as a tab or comma delimited file. After saving the desired output data, click the "Delete" button to erase data from the summary worksheets.

CALCULATION

The "Calculation" worksheet shows the selected and input data as well as critical dates for growth and cover crops and the daily calculations of ET_o, K_c, and ET_c values by the growth stages. The main factors affecting the difference between ET_c and ET_o are (1) light absorption by the canopy, (2) canopy roughness, which affects turbulence, (3) crop physiology, (4) leaf age, and (5) surface wetness. When not limited by water availability, both transpiration and evaporation are limited by the availability of energy to vaporize water. Therefore, for unstressed crops, solar radiation (or light) interception by the foliage and soil mainly affect the ET_c rate.

As field and row crops grow, the canopy cover, light interception, and the ratio of transpiration (T) to ET increases until most of the ET comes from T and evaporation (E) is a minor component. The K_c value increases with canopy cover until reaching about 75% cover. For tree and vine crops the peak K_c value is reached when the canopy has reached about 70% ground cover. The difference between the crop types is because light interception is higher for the taller crops.

FIELD AND ROW CROP K_C VALUES

Field and row crop K_c values are calculated using a method similar to that described by Doorenbos and Pruitt (1977). A generalized curve is shown in Figure 1. In their method, the season is separated into initial (date A-B), rapid (date B-C), midseason (date C-D), and late season (date D-E) growth periods. K_c values are denoted K_cA, K_cB, K_cC, K_cD, and K_cE at the ends of the A, B, C, D, and E growth dates, respectively. During initial growth, the K_c values are a constant value, so K_cA= K_cB. During the rapid growth period, when the canopy increases from about 10% to 75% ground cover, the K_c value increases linearly from K_cB to K_cC. The K_c values are also at a constant value during midseason, so K_cC= K_cD. During late-season, the K_c values decrease linearly from K_cD to K_cE at the end of the season.

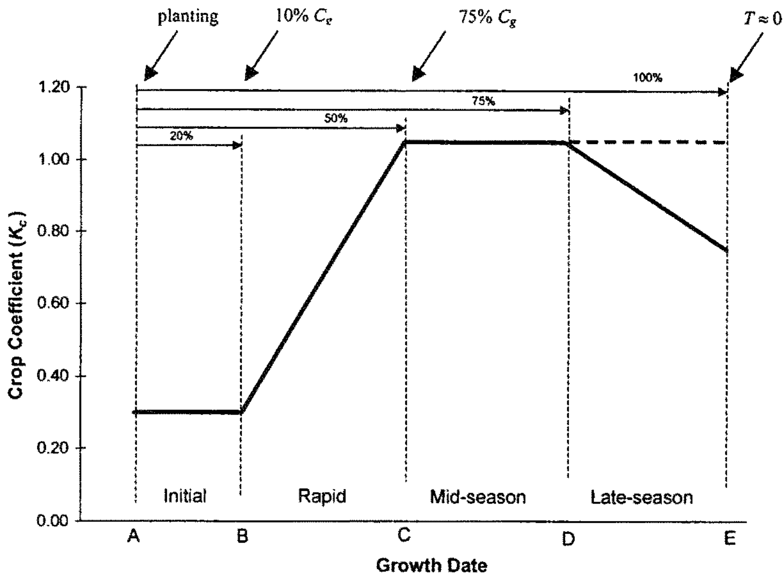

Figure 1. Hypothetical crop coefficient curve for field and row crops using percentage of the season to delineate growth dates. The dashed line is for fresh market crops with no late-season K_c value drop (that is, there is no date D).

Doorenbos and Pruitt (1977) provide estimated number of days for each of the four growth periods to help identify the end dates of growth periods. However, because there are climate and varietal differences and because it is difficult for growers to know when the inflection points occur, irrigators often find this confusing. To simplify this problem, CUP uses percentages of the season from planting to each inflection point rather than days in growth periods (see Figure 1). Irrigation planners need only enter the planting and end dates. The intermediate dates are determined from the percentages, which are easily stored in a computer program.

During initial growth of field and row crops, the default K_c value (K_c1) is used for K_cA and K_cB unless it is overridden by a K_c value based on rainfall or irrigation frequency. If a soil wetting-based K_c1 is desired, the irrigation or rainfall frequency is entered in the "Input_Output" worksheet. Then, a graph showing the K_c values curve corresponding to input wetting frequency versus mean ET_o rate is shown in the chart "IrrigInitGrow" or "RainInitGrow," respectively, for irrigation or rainfall entries.

The values for K_cC= K_cD depend on (1) light interception differences, (2) crop morphology effects on turbulence, and (3) physiological differences between the crop and reference crop. Some field crops are harvested before senescence, and there is no late season drop in K_c values (for example, silage corn and fresh market tomatoes). Relatively constant annual K_c values are possible for some crops (for example, turfgrass and pasture) with little loss in accuracy.

DECIDUOUS TREE AND VINE CROP K_C VALUES

Deciduous tree and vine crops, without a cover crop, have K_c value curves that are similar to those for field and row crops but without the initial growth period (Figure 2). Default K_cB, K_cC= K_cD= K_c2 and K_cE= K_c3 values are given in the "Crop References" worksheet of the CUP. The season begins with rapid growth at leaf out when the K_c value increases from K_cB to K_cC. The midseason period begins at approximately 70% ground cover. Then, unless the crop is immature, the K_c value is relatively fixed between dates C and D until the onset of senescence. For immature crops, canopy cover may be less than 70% during the midseason period. If so, the K_c value will increase from K_cC up to the K_cD as the canopy cover increases. The CUP program does account for K_c value changes of immature tree and vine crops. During late season, the K_c value decreases from K_cD to K_cE, which occurs when the transpiration is near zero.

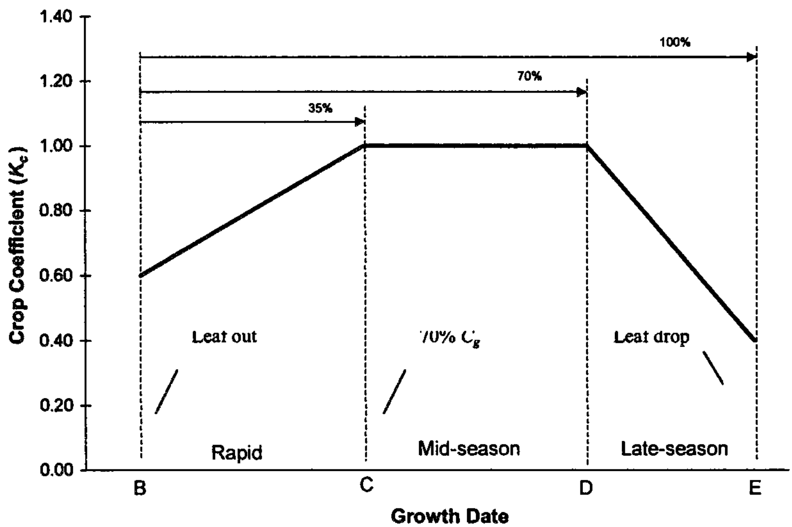

Figure 2. Hypothetical crop coefficient curve for deciduous tree and vine crops using percentage of the season to delineate growth dates. There is no initial growth period, so the season starts at leaf out on date B.

Correcting K_cB for soil evaporation

Initially, the K_c value for deciduous trees and vines (K_cB) is selected from a table of default values. However, ET is mainly soil evaporation at leaf out, so CUP contains the methodology to determine a corrected K_cB, based on the bare soil evaporation.

Correcting for cover crops

With a cover crop, the K_c values for deciduous trees and vines are higher. When a cover crop is present, 0.35 is added to the clean-cultivated K_c. However, the K_c value is not allowed to exceed 1.15 or to fall below 0.90. CUP allows beginning and end dates to be entered for two periods when a cover crop is present in an orchard or vineyard.

Immature trees and vines

Immature deciduous tree and vine crops use less water than mature crops. The following equation is used to adjust the mature K_c values (K_{cm}) as a function of percentage of ground cover (C_g).

$$\text{If } \sin\left(\frac{C_g}{70}\frac{\pi}{2}\right) \geq 1.0 \text{ then } K_c = K_{cm} \text{ else } K_c = K_{cm}\left[\sin\left(\frac{C_g}{70}\frac{\pi}{2}\right)\right] \tag{1}$$

Subtropical orchards

For mature subtropical orchards (for example, citrus), using a fixed K_c value during the season provides acceptable ET_c estimates. However, if higher, the bare soil K_c value is used for the orchard K_c value. For an immature orchard, the mature K_c values (K_{cm}) are adjusted for percentage of ground cover (C_g) using the following criteria.

$$\text{If } \sqrt{\sin\left(\frac{C_g}{70}\frac{\pi}{2}\right)} \geq 1.0 \text{ then } K_c = K_{cm} \text{ or else } K_c = K_{cm}\sqrt{\sin\left(\frac{C_g}{70}\frac{\pi}{2}\right)} \tag{2}$$

Field crops and landscape covers with fixed K_c values

Some field crops and landscape plants (type-2 crops) have a fixed K_c value all year. However, if the significant rainfall frequency is sufficient to have a higher K_c value for bare soil than for the selected crop, then the higher bare soil K_c should be used. CUP permits entry of monthly mean rainfall frequency data. If entered, daily K_c values for bare soil evaporation are computed for the entire year. The higher of the fixed crop K_c value or the bare soil K_c value is used to estimate

ET_c for the crop. If no rainfall frequency data are entered, then the fixed crop K_c value is used.

ESTIMATING BARE SOIL K_C VALUES

A soil evaporation K_c value, based on ET_o and rainfall frequency is needed as a minimum (baseline) for estimating ET_c. It is also useful to determine the K_c value during initial growth for field and row crops ($K_c1= K_cA= K_cB$) and the starting K_c for deciduous tree and vine crops ($K_c1= K_cB$) based on irrigation frequency. The K_c values used to estimate bare soil evaporation are based on a two-stage soil evaporation method reported by Stroonsnjider (1987) and refined by Snyder et al. (2000). The method provides a K_c values as a function of ET_o rate and wetting frequency that are similar to those published in Doorenbos and Pruitt (1977).

When mean monthly weather and ET_o data are entered into the "Weather Input" worksheet, including the number of significant rainy days per month, CUP calculates a baseline soil evaporation curve. Daily precipitation is considered significant when $P_s > 2 \times ET_o$. Whenever, the K_c value for bare soil evaporation is higher than the K_c value based on table or calculated K_c values, the higher K_c value is used.

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GLENN-COLUSA IRRIGATION DISTRICT FLOW MONITORING PROGRAM

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ABSTRACT

In 1996, the United States Bureau of Reclamation (USBR), Mid-Pacific Region, Northern California Area Office began the Willows Flow Monitoring Program. The program objective was to monitor, in near real-time, 85% of the Sacramento River Diversions from Shasta Dam to Sacramento. Several site visits were conducted to the Districts and their diversion points along the Sacramento River by USBR and Irrigation Training and Research Center (ITRC) staff from California Polytechnic State University (Cal Poly), San Luis Obispo. One of the sites identified for flow monitoring was the Glenn-Colusa Irrigation District (GCID) canal at the Stoney Creek Siphon. At question was the application and functionality of the Acoustic Doppler Flow Meters (ADFM) inside the siphon channels. Data were relayed through a Supervisory Control and Data Acquisition (SCADA) system provided by Sierra Controls of Carson City, Nevada.

The maximum volume of flow in the GCID canal siphon (3200 cfs) called for three (3), 12'×14' rectangular tubes over 200 feet in length. Each tube required an individual flow meter. Three ADFMs were installed to measure the flow of the water through the siphons. The sensors were placed far enough into the downstream end of the siphon to insure constant submergence. Final installation appeared to work best with the sensor mounted on the side of the rectangular tubes. A structure was constructed for the SCADA equipment that was used for data management and access. As a result of program efforts, the USBR and GCID were set up with the capabilities to poll the GCID Stoney Creek Siphons site and monitor the real-time flow data at their respective office computers.

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BACKGROUND

The new flow measurement equipment installed at the GCID Stoney Creek Siphon is a state-of-the-art implementation of several infant technologies that show great promise for the future of flow measurement for irrigation districts. Three ADFMs were connected to a single computer where the information would be logged into a newly developed SCADA System. The data developed from the site could then be downloaded to the USBR office in Willows or to the GCID main office. This makes a very useful tool for monitoring and recording flow rates into the GCID canal system, providing the Bureau with river diversion data and the district with canal management data.

Several designs were reviewed for the new structure, but the early estimates for the more common acoustic velocity metering in a lined section rapidly grew from $350,000 to over $1.1 million. The USBR, Mid-Pacific Region, Northern Area Office, opted to try a new design that resulted in a cost saving of some $900,000.

SACRAMENTO RIVER FLOW MONITORING PROGRAM

The Sacramento River Flow Monitoring Program was developed to monitor real-time flow and river level data at as many districts as practical on the Sacramento River between Shasta Dam and Sacramento. Presently, the program has succeeded in monitoring real-time flows of 80% of the diversions, with a target of 85% of the river's diverted flow. The USBR continuously works to improve the flow measurement with each of the districts involved and with several more districts becoming involved. Diversions from the Sacramento River are important not only for the USBR, but for each of the districts as well.

The program allows the Willows office to call each of the sites and collect the flow data. The sites use different types of meters to monitor the flow. The most common are propeller meters fitted with electronic heads; but the older technology is giving way to new acoustic and acoustic Doppler technologies. All of the metering equipment produces either a 4-20 mA analog signal or a Digital pulse for reporting flow. The SCADA equipment receives, accumulates, records and provides either a workstation or a reporting station for polling.

Starting in 1996, a base (office) system and one or two remote diversion sites were installed at a couple of districts and at the USBR Willows office. Annually, as funding allowed, programs at these districts have grown and the USBR program has expanded to additional districts. Since neither the districts nor the USBR could bear the burden of "turn key" systems, the program has been a cost-share, joint effort between the districts and USBR. It has been designed to expand as each district recognizes opportunities for applications and essential funding.

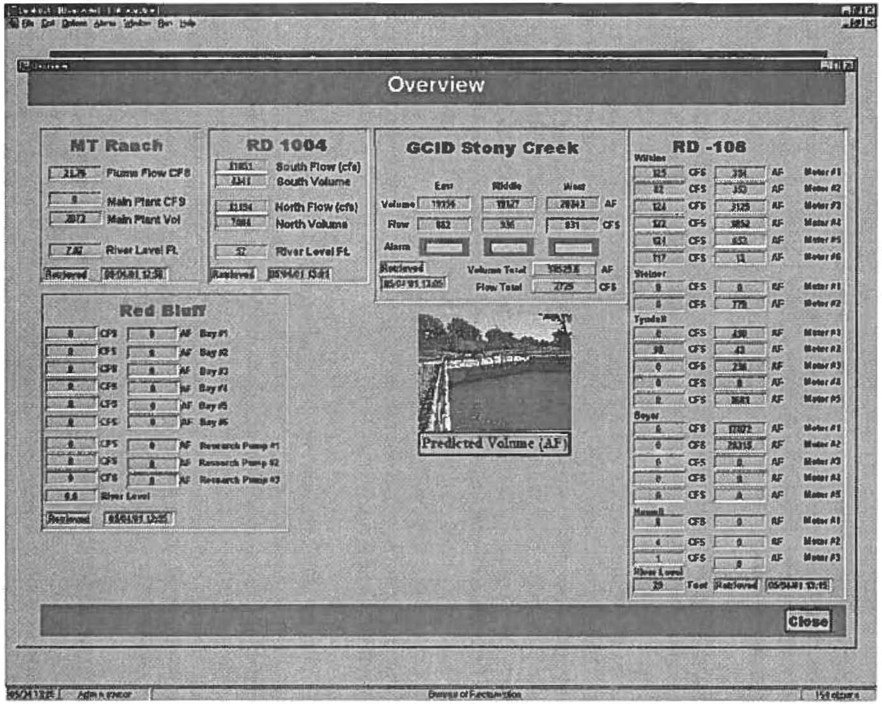

Figure 1. Willows Flow Monitoring "Overview" screen

To utilize the communications available in the system, a flow prediction screen was developed. This screen allows the districts to project diversions seven to ten days in advance and to make updates, as additional information is available. The information is collected at the Willows USBR office as a flow-polling activity, with polling frequency (hourly) and updates being shown on separate predictions screens. The information is then available to the river operations office for use in river flow management. Presently, the information is only requested in years of reduced (drought) flows.

Typically, an integrator works with the districts to program and install the equipment and to train district staff. The integrators being utilized in the Sacramento Valley have been willing to provide services cooperatively with the districts (i.e., allow the districts to bury pipe and run wires, etc.) to keep costs at a minimum. Many hours have been required to get a coordinated program up and running to draw flow data from the many pumps and districts along the river. Annual site visits have been made to most of the districts involved to make adjustments to fix communication problems that have been encountered. Each year more districts and water users are in the process of connecting into the Sacramento River Flow Monitoring Program. The success of this program will be visible at many levels: first, at the river management level; second, at the diverter's management level; and third, at the intra-district canal and water release (turnout) management level.

The GCID Stoney Creek Siphon site is unique because it is a relatively new application of the Acoustic Doppler technology. A housing structure was designed and built to enclose all of the flow monitoring equipment needed. The housing structure includes an air conditioning system to offset the hot summer weather environment of the Sacramento Valley. All of the computer equipment, the ADFM equipment, and the remote monitoring equipment are housed in the housing structure. The structure can be seen in Figure 2.

The housing structure allows the equipment to be stored in a safe place and in an environment ideal for the computer equipment. Any work necessary, from programming the meters to records back up and communications, can be conducted on the computer equipment within this structure. Figure 3 shows the inside of the new structure, along with the computer equipment.

Using the SCADA System, the USBR and GCID can monitor and record real-time flow values at the Stoney Creek Siphon site. The GCID structure houses the computer equipment needed to communicate with the USBR Willows office and with the GCID office. Also housed in the new structure are the ADFM digital electronics and the analog converters for the inputs to the SCADAPack. Figure 4 shows the screen on the Willows computer for the GCID Stoney Creek Siphons.

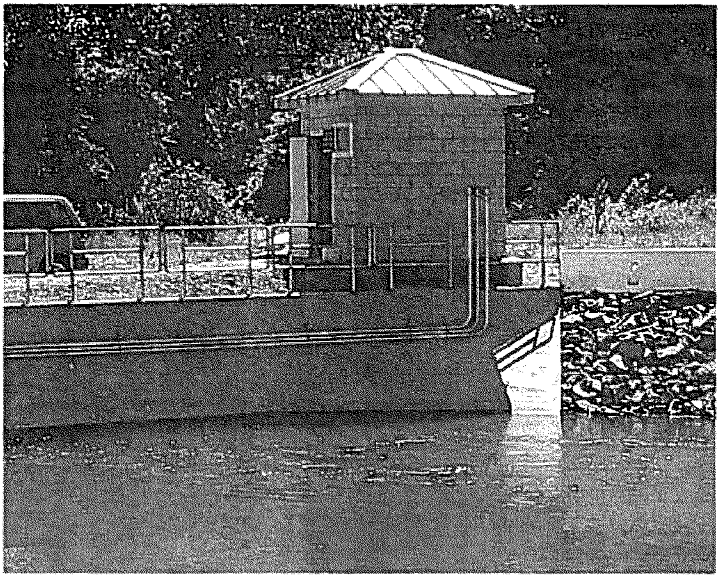

Figure 2. New flow-measurement structure at GCID Stoney Creek Siphons site

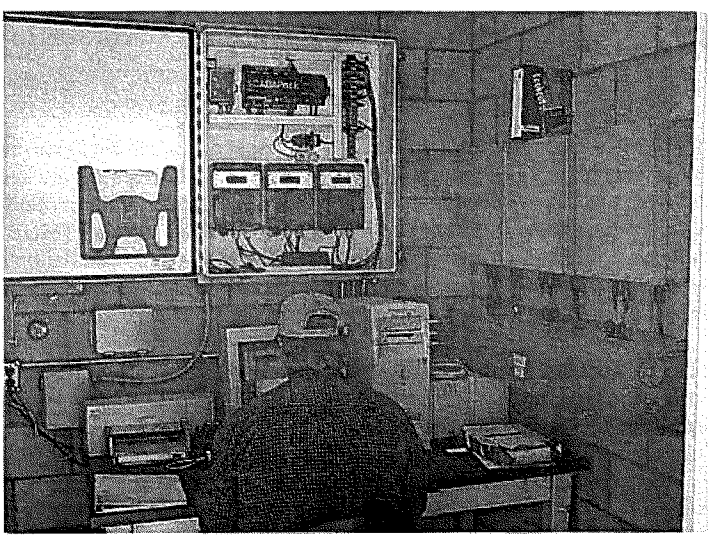

Figure 3. Housing structure at GCID Stoney Creek Siphons site

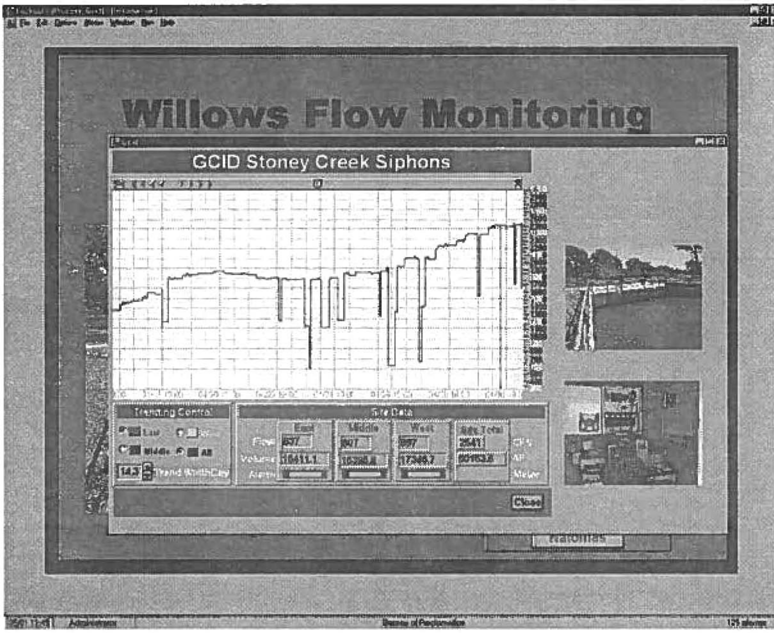

Figure 4. Willows flow-monitoring screen for GCID Stoney Creek Siphons site

MGD ADFM

The ADFM has four piezoelectric ceramics that emit short pulses along narrow acoustic beams pointing in different directions. Echoes of these pulses are back scattered from material suspended in the flow. Since this material has motion relative to the transducer, the echoes have a Doppler shift in frequency (MGD Technologies, 2002). Measurement of this frequency enables the calculation of the flow speed. A fifth ceramic mounted in the center of the transducer assembly, and aimed vertically, is used to measure the depth.

The ADFM divides the return signal into discrete regular intervals that correspond to different depths in the flow. Velocity is calculated from the frequency shift measured in each interval. The result is a profile, or linear distribution of velocities, along the direction of the beam. The profiles are generated from velocity data measured by an upstream and downstream beam pair. Data from one beam pair are averaged to generate Profile #1, and a beam pair on the opposite side of the transducer assembly generates Profile #2 (MGD Technologies, 2002).

Velocity data from the two profiles are entered into an algorithm to determine a mathematical description of the flow velocities throughout the entire cross-section of the flow. The algorithm fits the basis functions of a parametric model to the actual data. The results, which predict flow velocities at all points throughout the flow, are integrated over the cross-sectional area to determine the discharge.

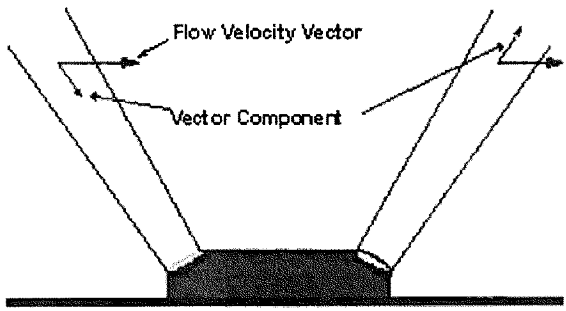

Figure 5. ADFM beam geometry

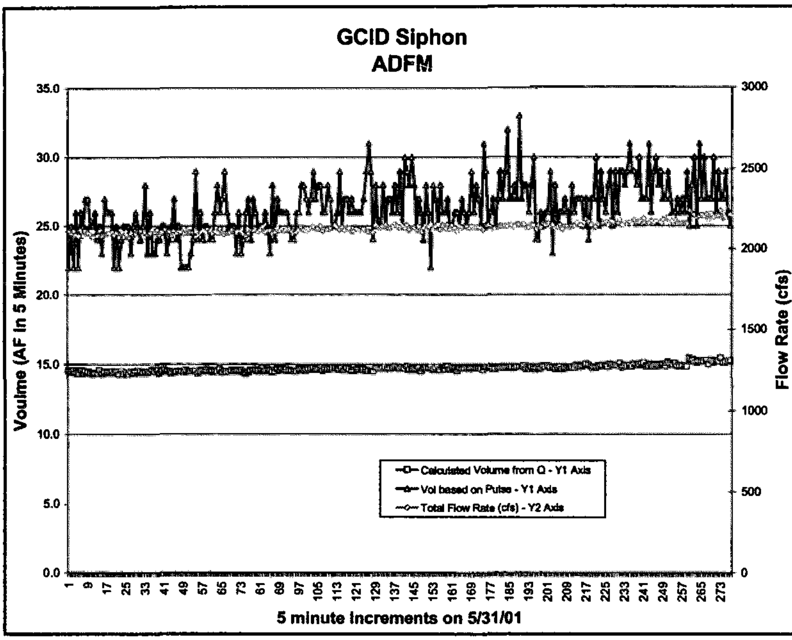

Figure 6. ADFM data at GCID Siphons site on May 31, 2001

KEY ISSUES AT THE SITE

The new flow meters encountered a few problems when first installed. The meters were installed on the top of the cross-section, facing down, for water to flow by. With this configuration, debris that floats on top of the water's surface caught on the ADFM and interfered with the velocity profile measurements. To help alleviate this problem, the USBR hired a diver to clean off any debris that may catch on the meters. In the end, to eliminate the problem with floating debris, the meters were moved from the top to the side of the cross-section.

Another problem that had to be fixed was encountered on one of the meters. While the flow rate could be obtained, the totalizer pulsing was not working correctly. A consulting SCADA integrator (Concepts in Controls, CIC) reprogrammed the code for the totalizer, and it is currently working correctly. Two of the three digital electronics sensor controllers for the MGD device have required repair by the manufacturer (over a two-year period). MGD has replaced the units via overnight response, and the units have been quick and easy to trade in and out.

There have been times when the telecommunications between the Northern California Area Office and the Siphons site have become disabled. The modem has had to be replaced and programming reestablished. The experience in this program effort is that radio communication has been more reliable than telephone. Selection of either depends on availability and functionality.

Use of non-proprietary SCADA equipment has facilitated quick and easy access to maintenance personnel. The use of modular equipment makes it easy for someone with limited electronics background to make many of the replacements necessary for malfunctioning equipment.

SUMMARY

The Sacramento River Flow Monitoring Program began with a concept that real-time information related to flow diversions and corresponding river levels would be beneficial to USBR's management of the Sacramento River. Additionally, the diverters of river water would benefit from having real-time pump operation, flow data and operation automation, as well as having monitoring capabilities at control sites such as their office or homes.

The Water Conservation program can see the relative benefit of improved water management at nearly all levels due to those control and automation capabilities. The program started with a few districts in 1996, adding a couple of districts each year. The Sacramento River Flow Monitoring Program has expanded to include the nine districts that are the major water diverters from the Sacramento River. These nine districts represent over 80% of the water diverted from the river between Shasta Dam and Sacramento. The program has continued to generate interest from other districts with a desire to become involved in this new state-of-the-art flow measurement program.

The new structure at the GCID Stoney Creek Siphons site has been a key factor in the expansion of the program to other districts. This new site allowed the USBR to evaluate an emerging flow measurement technology. Some modifications have been made, such as moving the sensors to the side of the siphon instead of on top. Knowledge gained from this site has encouraged the USBR to move forward in its efforts to include electronic water measurement and SCADA programs at other districts.

Because of the Sacramento River Flow Monitoring Program, diversions from the Sacramento River will be monitored more closely in the years to come. As more districts come on-line to the program, the program approaches the target of 85% monitoring that was originally considered financially feasible, and the ability to accurately measure and monitor the diversions will increase. With these improvements, the foundation is provided for improved water management both on the larger scale of river management and at the district and farm levels.

SELENIUM LOADING IN THE LOWER GUNNISON RIVER BASIN

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Delbert Smith[2]

ABSTRACT

The Gunnison River is the largest tributary to the Colorado River that is completely within the State of Colorado. The river has been altered to meet demands for uses such as irrigation, hydropower, and recreation. These alterations have created river water quality issues that create challenges for aquatic life.

A water quality issue that is being investigated is elevated levels of selenium in the River below the Uncompaghre Project Area. Concentrations of selenium in this reach of the Gunnison River commonly exceed 8 ppb (parts per billion), higher than the EPA established cronic criteria. These concentrations may be adversely affecting reproduction in endangered razorback sucker and Colorado pikeminnow.

The Bureau of Reclamation, U.S. Fish and Wildlife Service and U.S. Geological Survey have been cooperatively collecting and analyzing water quality data and developing remediation programs in the lower Gunnison River Basin (below Crystal Reservoir) as part of the National Irrigation Water Quality Program in conjunction with the Gunnison River Basin Selenium Task Force. This report focuses on the basin-wide data that have been collected and the alternatives being considered to reduce selenium loading to the lower Gunnison River Basin.

INTRODUCTION

The Gunnison River Basin is located in western Colorado. The Gunnison River and its major tributaries (North Fork of the Gunnison and Uncompahgre River) have been developed to meet both agricultural and municipal needs. Federal Reclamation projects include the Wayne N. Aspinall Unit (Blue Mesa, Morrow Point, and Crystal Reservoirs), the Uncompahgre Project (Taylor Reservoir), Smith Fork Project (Crawford Reservoir), Paonia Project (Paonia Reservoir), Bostwick Park (Silver Jack Reservoir), Dallas Creek (Ridgway Reservoir), and Fruitgrower's Project (Fruitgrower's Reservoir) (See Figure 1).

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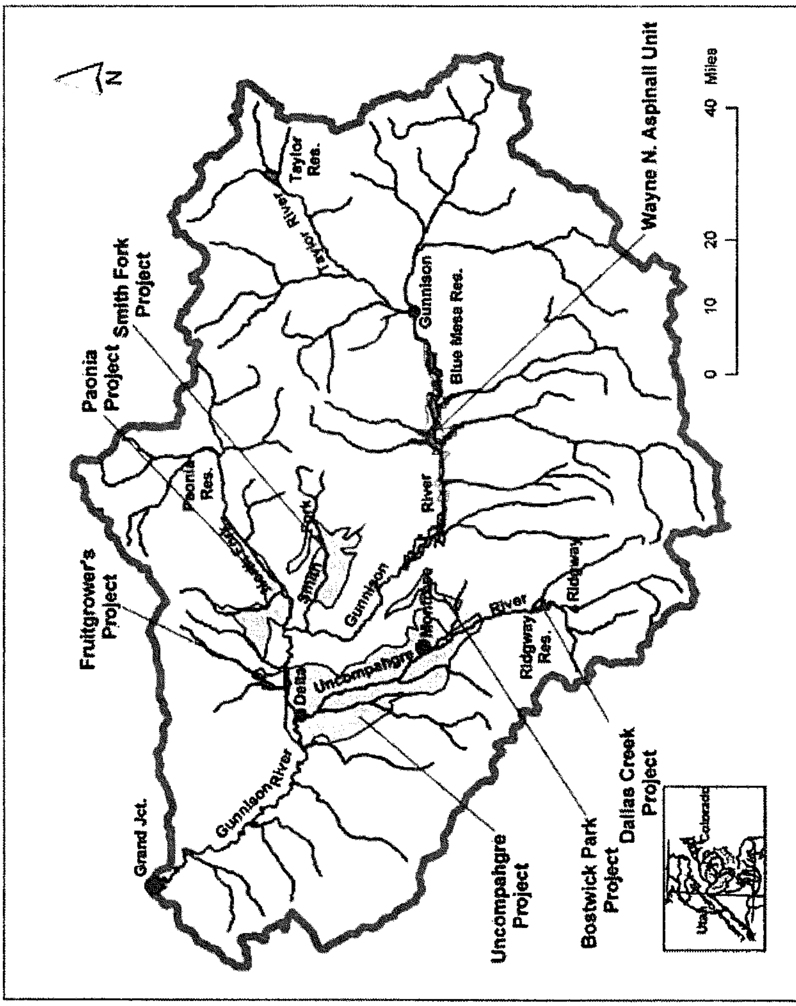

Figure 1. Gunnison River Basin

In 1985, the U.S. Department of the Interior (DOI) began the National Irrigation Water -Quality Program (NIWQP) to determine if irrigation drainage from DOI-constructed or managed projects was having adverse effects on water quality and on fish and wildlife in seventeen western states. The program has five phases: 1) site identification, 2) reconnaissance investigation, 3) detailed studies, 4) planning, and 5) remediation (Deason, 1986).

A comprehensive survey (Phase 1) of about 600 irrigation project areas and federal and state wildlife refuges was completed in 1989. A total of 35 sites in 15 states were identified as having high or medium potential for irrigation induced contamination problems. Reconnaissance investigations (Phase 2) were completed for 37 sites.

Detailed studies (Phase 3) as a result of the reconnaissance investigations were conducted on eight sites in California, Colorado, Montana, New Mexico, Nevada, Oregon, Utah and Wyoming. Among the five sites determined to have adverse impacts to fish and wildlife resources from federal irrigation projects was the Gunnison/Grand Valley, Colorado. The next step, phase IV remediation planning for the Gunnison Basin is governed by a core team consisting of representatives from Bureau of Reclamation, the U.S. Geological Survey, and the U.S. Fish and Wildlife Service. Under the direction of the core team, a technical team evaluates various issues related to the proposed remediation alternatives.

Remediation implementation (Phase 5) activities have been initiated on the Gunnison/Grand Valley in Colorado.

UNCOMPAGHRE PROJECT

In 1987B89, a reconnaissance-level study identified potentially adverse concentrations of selenium in some water, bottom sediment, and biota samples in the Uncompahgre Project Area (Butler et al, 1991). Additional physical, chemical and biological data for a detailed study of irrigation drainage in the Uncompaghre Project were collected in 1991-92 (Bulter et al, 1994). A detailed study of selenium associated with irrigation drainage in the Uncompaghre Project area was completed in 1991-93 (Bulter et al, 1996) (Figure 2). Additional tributary data were collected using funds provided by the State of Colorado through Section 319 of the Clean Water Act. The 1991-1993 detailed study concluded that the Uncompaghre Project and the Grand Valley may account for as much as 75 percent of the selenium load in the Colorado River near the Colorado-Utah State line. The Uncompaghre River accounts for approximately 40 percent of the selenium loading of the Gunnison River while only comprising about 20 percent of the total acreage of the overall Gunnison River Basin (Figure 3).

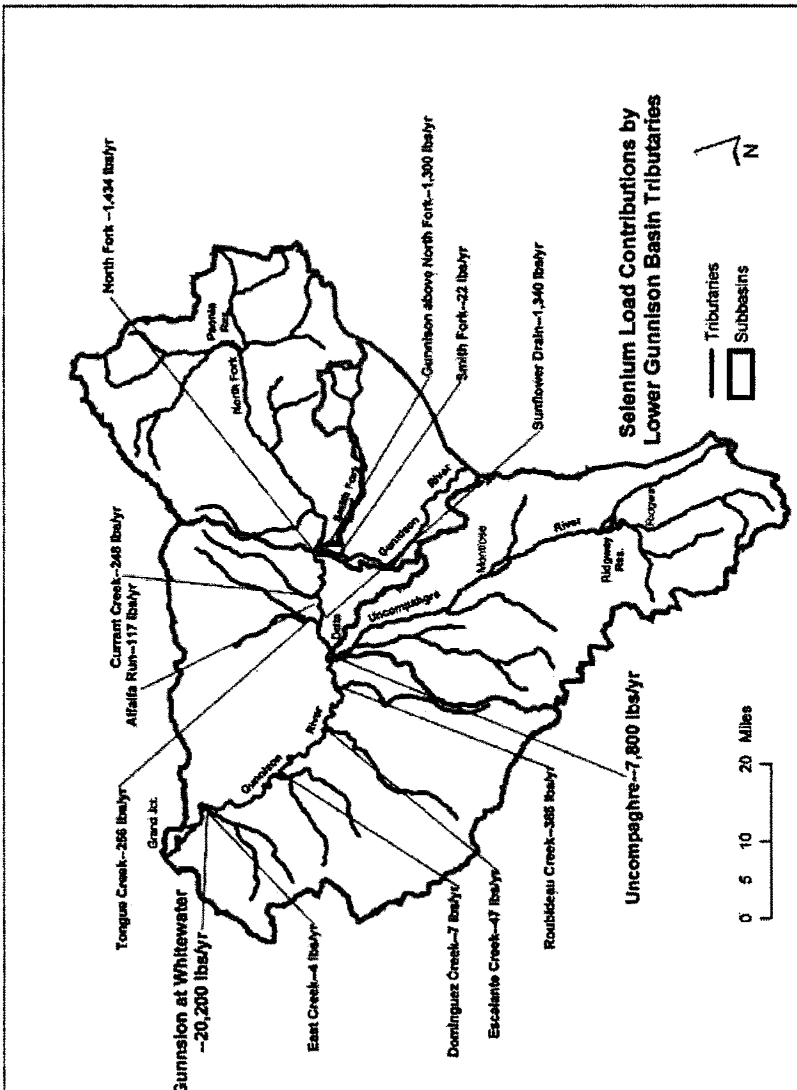

Figure 2. Selenium Load Contributions by Lower Gunnison Basin Tributaries

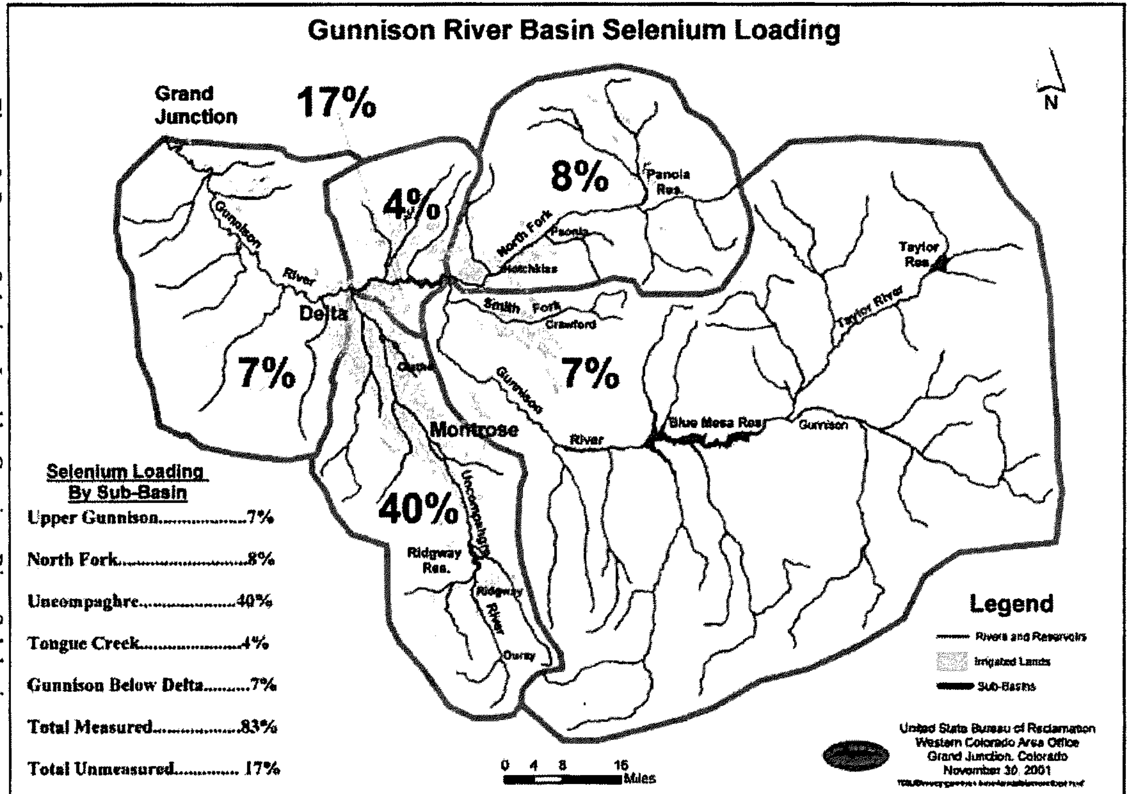

Figure 3. Percent Selenium Load by Gunnison River Sub-basin

A majority of the irrigation water used in the Uncompaghre Valley is diverted from the Gunnison River and transported via the Gunnison Tunnel to irrigate lands in the Montrose and Delta areas. Irrigation return flows drain into the Uncompahgre and Gunnsion Rivers. Twelve sample sites were established in the Uncompahgre Project area to measure and estimate annual selenium load contributions. The Loutzenhizer Arroyo was identified as contributing about 60 percent (4,900 lbs/year) of the selenium loading in the Uncompaghre River at Delta, Colorado (7,811 lbs/year)(See Figure 4). Selenium concentrations typically range from 16 ppb in the summer to 250 ppb in the winter for the Loutzenhizer Arroyo, and 4 ppb in the summer to 26 ppb in the winter for the Uncompahgre River at Delta, Colorado. Cedar Creek was also identified as a major selenium contributor with an annul load of 1,990 lbs/year with concentrations from 4.9 ppb in the summer to 45 ppb in the winter.

PLANNING REMEDIATION

Under the Phase IV Planning Remediation process, the goal is to develop remediation programs to reduce selenium hazards to benefit the endangered fishes. In July 1997, the Colorado Water Quality Control Commission adopted a 5 ppb aquatic life standard for selenium in the Gunnison River Basin. In addition, Colorado Water Quality Control Division is in the process of developing total maximum daily loads (TMDL) for river reaches out of compliance including the Lower Gunnison River.

As indicated previously, concentrations of selenium in the lower reach of the Gunnison River commonly exceeds 8ppb, thus exceeding the State of Colorado standard. These concentrations may be adversely affecting reproduction in wildlife including the endangered razorback sucker and Colorado pikeminnow.

Measured at the Uncompaghre River at Delta and at Whitewater, near Grand Junction, estimated selenium loads of the Gunnison River are approximately 7,800 lbs/yr. and 20,200 lbs/yr. respectively. Projects are being proposed to reduce selenium loading by 6,500 lbs/yr. at the Uncompaghre River at Delta and 8,400 lbs/yr at Whitewater to meet the state standard of 5 ppb . Because the Uncompaghre River is the largest contributor of selenium, remediation efforts have been focused in this area.

Lining canals, piping laterals, pond linings, and phyto-remediation have all been considered as possible alternatives to assist in reducing selenium in the Gunnison River Basin. One promising alternative involves the application of polyacrylamide (PAM) in an aqueous solution to the dry canal prism prior to the irrigation season. PAM bonds with soil particles, which settle and seal the canal bottom and reduce seepage. Initial test applications to 13-miles of laterals in the

Sunflower Drain in 2000 and 2001(See Figure 3) have resulted in an approximate 20 percent reduction in selenium load. Additional investigations in the Lotzenhizer Arroyo area are planned to begin in 2002 to document the fate (chemical stability) and transport of PAM. If successful, PAM could be applied to a considerable portion of the Uncompaghre Project, and provide a cost effective measure to assist in the reduction of selenium in the Gunnison River.

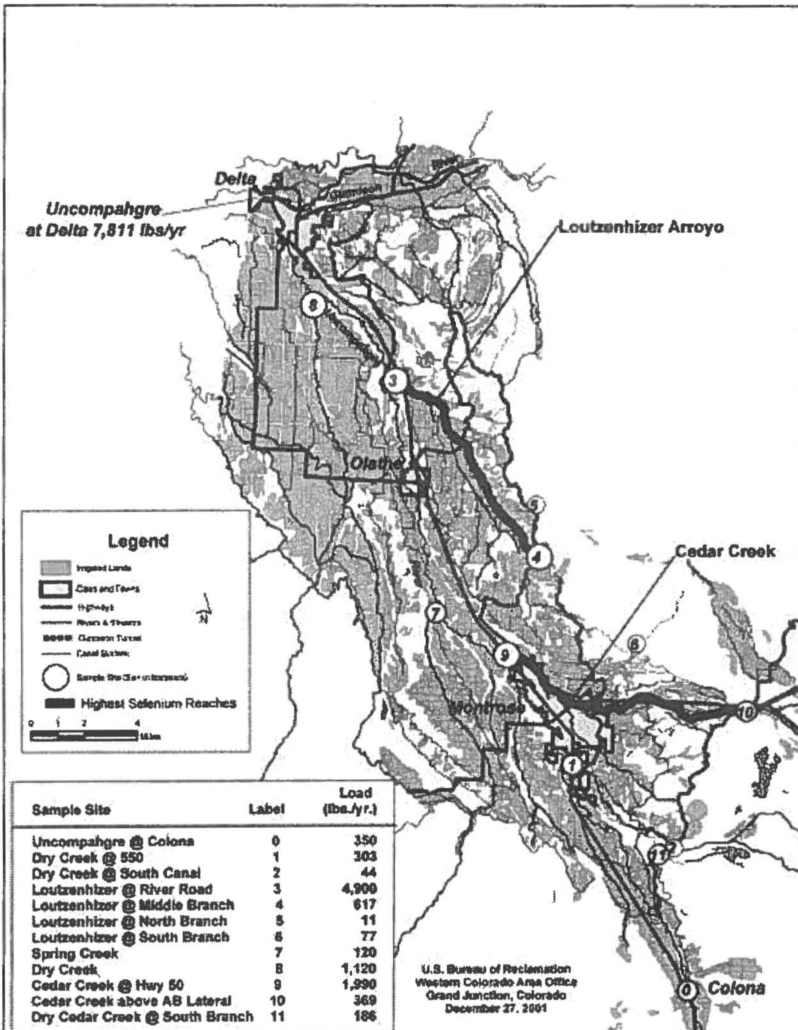

Figure 4. Uncompaghre River Selenium Loading

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AUTOMATION IN DRIP IRRIGATION SYSTEM FOR COTTON GROWING ON LARGE SCALE - A CASE STUDY

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ABSTRACT

Micro Irrigation technology concept is now accepted by most of the farmers in Maharashtra. It's adoption started in grapes a decade back and has since then spread in to a large number of crops. Drip irrigation however had not been adopted in cotton crop until as recently as four to five years ago. This was primarily because cotton has a large plant population. Adequate research recommendations are not available regarding net water requirement, plant population, plant to plant and row to row spacing, distance between emitters, emitters discharge in black cotton and light soils etc. However, a progressive farmers tried drip irrigation in cotton with remarkable success. Fertigation through drip is yet to be given a genunine attempt. An integrated pest management approach to plant protection in cotton is a recent attempt in organized scale cluster demonstration and farmer's field school. Drip irrigation offers additional advantages in cotton growing such as application of controlled quantity of water at the initial stages of plant propogation in order to increase flowering, earl in flowering and thus higher yields and at source fertigation. Water saving of 60 percent increase in the irrigated area, and reduces the cost of fertigation, weeding, etc. This paper deals with the irrigation system equipment, design layout, computer control, field communication systems and water requirements.

INTRODUCTION

India is a major cotton producing country in the world with more than 7.5 million hectare in production. Maharashtra with 3 million hectares, accounts for 40 percent of the total cotton area in the country. In terms of per hectare productivity Maharashtra ranks eight in the country with 180 kg lint per hectare compared to the national average of 248 kg by comparison. Punjab has a

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productivity of more than 500 kg lint per hectare. However, Israel with the help of drip irrigation and fertigation through drip has been able to harvest 1900 kg lint per hectare. Therefore, they have attracted world wide attention for development and adoption of modern technology to overcome natural constraints to successful agriculture with limited water resources. Adoption of micro irrigation technology has helped them overcome this constraint successfully. Replication of this technology in cotton in Maharashtra would positively help in increasing yield. Therefore, cotton growing on a large scale with fully computerized drip irrigation systems has been implemented.

Installation Details of the Most Extensive Cotton Growing Project

The installation of the drip irrigation system spread over an area of 200 hectares (80 ha. at Central Research Station and 120 ha at Central Demonstration Farm) was a challenging task which involved detailed planning, procurement, co-ordination and management. The size of the project can be realised from the amount of overall material utilised in the project. Pressure compensating Drippers, numbered 22,000,00 (Twenty two lakh) and there was 11,000,00 (Eleven lakh) meters of LLDPE lateral with a 16 mm diameter. Twenty six thousand meters of PVC pipe line ranging from 63 mm to 280 mm diameter with 6 kg/cm2 pressure were used. About 18 kms of trenching was done to lay these pipes. The EBS Filter with a 400 m^3 /hr flow rate was electrically at operated. One hundred ten Bermad hydraulic valves were installed. Plastro in India (Pune) supplied the drip line with preinserted-drippers at specified spacings of 0.5 m .

Layout of the Irrigation System

The area of the project has been subdivided into plots with an average size of 2 ha. Each plot has a length of 200 m - 250 m and a width of 80 - 100 m according to the nature of the area. Each group of eight plots will have a secondary filtration head. The area has been so sub-divided so as to facilitate the work from the aspect of plot cultivation and the work of the irrigation operators and to facilitate flexible control over the water supply and irrigation system. The automation field layouts for drip irrigation systems for Central Research Station, (CRS), Akola and of Central Demonstration Farm (CDF), are given in Fig. 1 and 2 respectively.

The water from the main control head is delivered by a PVC pipeline to the secondary filter heads in the plots and to the operating heads. The PVC pipe having a diameter ranging from 280 mm to 90 mm and to a 75 mm and a working pressure rating of 6 Kg/cm^2. The PVC pipes has been laid in a trench, with the submain at 80 cm depth and the main pipeline at 1.2 m depth. Concrete anchors were constructed as necessary.

The Operating Heads

Each field plot have an operating head commanded by the control computer according to the operating program. The average application is 4 mm/day on the basis of six irrigation days in a week. The daily amount of water to be returned to the soil is 4.7 mm. Approximately 2 hours irrigation is required daily to apply this dosage. Therefore according to the operating program of eight sectors per day, the irrigation system is operating about 16 hours per day. The design and other details are given in Table 1, and water requirements of cotton during May to November is given in Table 2.

The drip laterals are connected to the sub main by means of saddles installed with 25 mm flexible pipes and the other end rises above ground level. A six way fitting is connected to the riser above the ground which enables connection of six drip laterals, three to each side. The outlets are spaced along the sub main at a distance of 5.8 meter. The drip laterals are laid out in the plots at the beginning of each season and rolled up at the end of each irrigation season.

Control and Automation

The irrigation systems at both places (CRS and CDF) are commanded and controlled by a central computer. Single electrical NYY cable is connected to the remote terminal units (R.T.U.) which are fixed on every valves in the field. The electronic coding to the valves is possible due to circuits cards and numbering of every individual valve. Accordingly these valves were operated with the help of computer and the data of operation is recorded. Both the irrigation systems at CRS and CDF for cotton growing are identical and have the same operating regime they have operated satisfactorily since installation in the year 1997.

Single Cable Field Communication System Overview

The single cable (SC) module is a microcontroller based module. The module operates up to 126 remote terminal units divided into two lines with each line controlling 63 RTU's. Each SC line is based on a communication cable, which transfers about 40 V A.C. feed power and command / report communication to the RTU's on site. Each RTU is comprised of an electronic circuit board and latching solenoid. When an RTU receives a command signal, its circuit board decodes the command signal and energizes the latching solenoid, thus controlling a hydraulic valve. The RTU can be connected to two back indication sensors (watermeter, pressure gauge and other type of dry contact connections). One single cable module is capable of serving up to 126 output and 252 input stations and the system can be protected against lightning surges by line protection units connected to the communication cables.

Operation

When the IRRINET unit is reporting to a central irrigation computer, an alarm report will be sent to the irrigation computer in case of failure of the 40 V mode and CLK. The physical connection table in the system defines the in and out connections to the logical stations and inputs. For each connection module / remote number, the input / output number is specified . The single cable module has 126 output and 252 input positions, which are divided into two communications lines with 63 RTU's on each line Therefore, when you need to physically connect a logical output or input to a SC module, you have to refer to one of the 126 output or 252 inputs. The RTU's can be checked through software by performing general operation diagnostics using the manual operation and the input status tables. Such unique field communication systems are installed on the large farms of the university. On 120 ha area there are 79 output positions with 43 output positions in module no. 1 and 36 in module no. 2 and 4 positions. Out of 4 input positions, 2 input positions are for main valves (hydrometers) and 2 are for fertigation pumps at Central Demonstration Farm (C.D.F.) at Wani Rambhapur and on a 80 ha area with 40 output positions in module one and 2 input positions in module no. 3 at Central Research Station, Akola (Table 3).

Maintenance

In order to ensure proper operation of the single cable system , periodic checking as mentioned below is necessary.

i. Proper electrical connection of the communications cables and the IRRINET unit. Also remove all the RTU's and LPU's cylinder covers and check for proper wire connections, remove dump residues and check the condition of the LPU's fuses.
ii. All grounding rods should be inserted properly in the ground and have a good cable connection
iii. The communication cables should be in the ground or properly routed indoors.

Remedies for Malfunctioning.

The following steps should be taken when malfunctions are detected.

When SC line fails to function

i. Check main power
ii. Check line connector on the SC module front panel for a good connection and no loose wires
iii. Inspect the SC module front panel for the illumination of the 40 V, MOD and CLK.
iv. Check all software implementation regarding specific line operation.

v. Check fuses F_1 and F_2 in the LPU's.
vi. Check all electrical wires on the RTU board are connected properly.
vii. Only use the fuses of specified type and ratings to avoid risk of fire and thereby damaging to equipment or other property.

When a single RTU fails to function.

i. Check all software implementation regarding the specific RTU operation.
ii. Recheck the address jumper setting of the RTU board and electrical wire connections
iii. Check for proper mechanical function of the RTU by using the manual operation knob.

Transfer of technology

About 1 lakh farmers have visited the project at the various stages of the cotton crop during last year and demonstrated the technology with effective and efficient water management practices. Training was provided to the Government Extension, Village Development, and Agricultural Officers, as well as to the social workers of non government organisations and farmers to improve the cotton growing technology in the State. Department of Agricultural Govt. of Maharashtra and Agricultural Universities have jointly under taken the need base technology demonstration programme for small farmers on 1, 2 3 and 4 hectares area at various Government and University farms in the region. This will transfer the technology at root level with proper soil, water and fertilizer management practices through Drip Irrigation.

CONCLUSIONS

1. The system works efficiently if logical monitoring, careful operation and timely and wise maintenance is carried out.
2. Frequent failures of the RTU's circuit cards are due to the humidity and mishandling.
3. Entry of water or moist conditions inside the RTU box is strictly prohibited.
4. Position of the RTU box in the field or at the bank of the stream should be sufficiently above the surface heading during rainy season.
5. At the time of installation of the cable, care should be taken to avoid cable damage due to the soil clods or stones etc. during filling of the trenches.
6. Cable jointing should be fully water tight.

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LIST OF ABBREVIATIONS

cm	centimeter
Doz	Dozen
ha.	hectare
Irri.	Irrigation
Kg.	Kilograms
l	liter
LDPE	Low Density Polyethylene
m	meter
MS	Maharashtra State
No.	Number
Q	Quintal
Rs.	Rupees
Spr.	Sprinkler
Sur.	Surface
T	Tones
UAS	University of Agricultural Sciences

Table 1. Design Details And Irrigation Data

Design / irrigation parameters	Units	C.D.F.	C.R.S.
Crop		Cotton	Cotton
Irrigation area (net)	Ha	120	80
Water source type		Tank	Tank
Crop spacing	m	1.96	1.96
Type of irrigation system		Drip	Drip
Emitter type		Katif	Katif
Emitter flow rate	l/h	2.3	2.3
Emitter spacing	m	0.5	0.5
Lateral spacing	m	1.93	1.93
Number of laterals per bed	-	1.00	1.00
Irrigation rate	mm/h	2.4	2.4
Average peak consumptive use	mm	4.66	4.66
Irrigation interval	days	6 days out of 7	6 days out of 7
Water application per irrigation	mm	4.66	4.66
Time of irrigation per operation	h	1.94	1.94
Number of operations per cycle		8	8
Max. discharge per operation	m^3/h	376	395
Min. discharge per operation	m^3/h	352	280
Max. total irrigation time per cycle	h	16	16
Min. required emitter pressure head	m	8	8
Elevation (up+, down -)	m	-	-
Required pressure head at water source (outlet)	m	60	60

Table 2. Water Requirements of Cotton During May to November

Item	May	June	July	Aug	Sept.	Oct.	Nov.	Total
Class A pan evaporation, mm/day	17	7-14	5.7	4.3	5.2	5.6	5.0	
Crop evapotrans-piration factor			0.4	0.6	0.8	0.4		
Monthly water demand, mm	35		88	80	125	140	60	508
Rainfall, mm/month	15	183	251	179	101	53	9	791
Balance, mm/month	-20	183	163	99	-24	-87	-51	283

Table 3. Input and output connections for the field communication system.

Sr. No	System valve details	Output connections		Input connections	
		Module	Out put	Module	Input
Central Demonstration Farm Wani-Rambhapur (120 ha)					
1	Line I control valve no. 1 to 43	1	1 to 43	-	-
2	Main control valve, no. 44	3	1	3	1
3	Fertilizer injection pump, no. 45	3	2	3	2
4	Line II control valve no. 46 to 81	2	1 to 36	-	-
5	Main control valve, no. 82	3	3	3	3
6	Fertilizer injection, valve no. 83	3	4	3	4
Central Research Station, Akola (80 ha)					
1	Control valve no. 1 to 40	1	1 to 40	-	-
2	Main control valve, No. 41	3	1	3	1
3	Fertilizer Injection pump valve no. 42	3	2	3	2

Figure 1. Automation field layout of Drip Irrigation System for Center Research Station (CRS), Akola

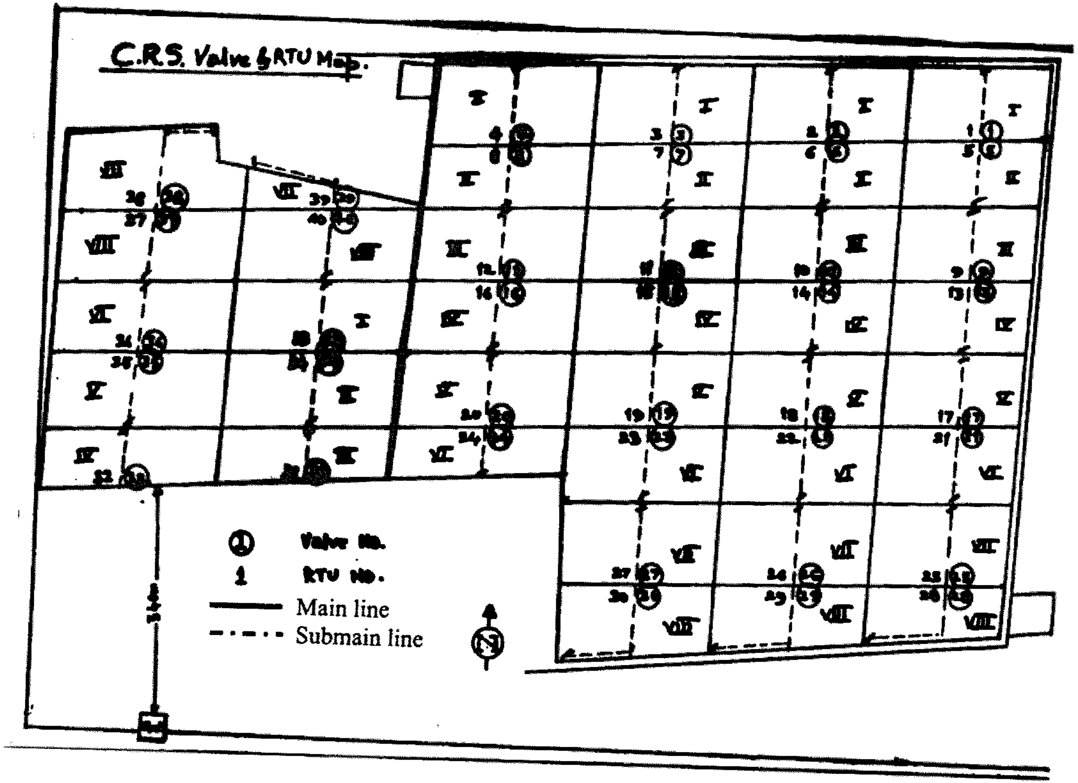
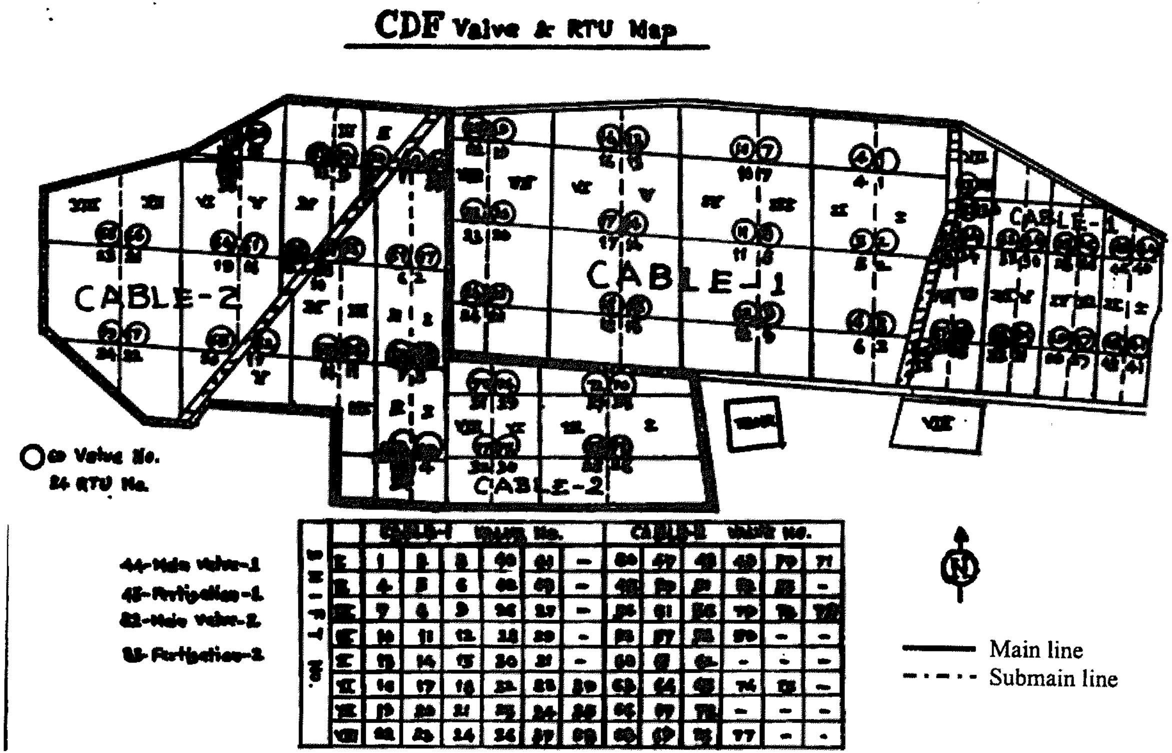


Figure 2. Automation field layout of Drip Irrigation System for Central Demonstration Farm (CDF) Wanirambhapur



ON-FARM INCREASED PRODUCTION, INCOME AND WATER-USE EFFICIENCY THROUGH MICRO-IRRIGATION IN INDIA

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ABSTRACT

On-farm production, income and water use efficiency for horticultural and other agronomical crops in terms of the benefit cost ratio micro-irrigation systems were calculated, and it is observed that B.C. ratio of drip irrigation was greater than conventional methods of irrigation. The water use efficiency was also higher than for conventional methods of irrigation. The monetary gains from micro-irrigation systems were larger than for conventional methods of irrigation for all crops. Information on water use efficiency, productivity and advantages of micro-irrigation compared to conventional irrigation methods is compiled and presented.

INTRODUCTION

Agriculture is the principal occupation of a major portion of the Indian population. Timely availability of an adequate and assured supply of irrigation water is crucial for the agricultural productivity. A large variation in the amount and distribution of rainfall in the recent past has resulted in inadequate availability of irrigation water and declines in the ground water table in most parts of the country. While the demand for water will continue to rise rapidly in agricultural, industrial and domestic sectors, the availability of water will be increasingly tightened because of the limited potential. It is also observed that presently agriculture alone utilized about 83 percent of the exploited water resources. In the future agriculture may receive a smaller share of water, because of the vital demand for drinking water and competition for industrial use. Therefore, agriculture will have to use less water. Simultaneously the high cost of development of water resources is making major irrigation projects economically unsustainable. The cost of irrigation per hectare has gone up from Rs. 1000/ha(US $24) in 1956 to nearly Rs. 65,000/ha (US $ 1475) in 1992, and still shows an increasing trend. Hence, it is clear that a major hurdle to increasing production and productivity, especially of high value commercial crops, may be inadequate water supplies. This hurdle can mainly be overcome through enhancing water use efficiency.

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Need for Study

Farmers are not yet fully convinced about the economics and advantages of micro-irrigation. Some farmers are convinced, as they are rich and can afford to install these systems. But they have not come forward to adopt these systems. Why? The simple and obvious answer is, what is the use of keeping one cow when milk is available freely? In light of this psychology, The authors have interviewed sixty two farmers in the region and on the basis of that efforts were able to calculate the on-farm increased production and income in horticultural crops in terms of the benefit- cost ratio through micro-irrigation systems on papaya, pomegranate, banana, grapes, strawberry as well as for tomato and roses. By going through the economics, The authors hope that a majority of the farmers shall be convinced to adopt micro-irrigation systems. Information as regards cereals, oilseeds, cash crops, and vegetables about water use efficiency, productivity and advantages as compared to conventional irrigation methods at the different places in country is compiled and presented in this paper.

RESULT AND DISCUSSION

Papaya

The cost of a drip irrigation system for papaya is about Rs. 40,000 per ha. The latex extracted and the yield of the fruit in drip fields is about 50% more from control fields. The payback period for installation of drip is less than one year. The B-C ratio of drip is about 4.0 compared to 3.0 in surface method. The extra income due to drip irrigation over conventional method is about Rs. 52,550 per ha. The economic details for drip irrigation for papaya are given in Table 1, results indicates that drip irrigation has good potential for papaya crops in coming years.

Pomegranate

The cost of a drip irrigation system for pomegranate is about Rs. 30,000 per ha. The economics for drip irrigation of pomegranate are given in Table 1.Extra income due to drip irrigation is about Rs.1,11,600 per ha. Hence pomegranate is very suitable for drip irrigation, and can be popularized in large scale in all parts of the country.

Banana

The variety of banana grown was Basarai which takes about 17-18 months to mature. After introducing drip irrigation, farmers are able to harvest two crops (main and one ratoon) in 24 months. The cost for drip irrigation is about Rs.

45,000 per ha. The amount of water applied by the farmer is usually about 16-20 liters/day/plant in the summer and 8 to 10 liters/day/plant in the winter. The yield of crop is about 75 Tons/ha compared to 50 Tons/ha in surface method of irrigation. The net per ha extra income due to drip irrigation over conventional irrigation is about to Rs.42,400 (Table 1). Micro-irrigation techniques have proved to be advantageous. The reason drip irrigation is not more popular is due to capital costs, and the farmers imagine that it is very cumbersome. It is difficult to get subsidies and bank loans, and the farmers believe in the traditional agricultural and irrigation methods.

Grape

The cost of the drip system is about Rs. 43,750 per ha. (average) for a spacing of 3 x 1.8 m. The amount of water applied by the farmer is about 20 liters/plant/day. The yield of grapes using micro-irrigation is 40 to 50 Tons/ha. Further it was reported that the quality of the fruit is uniformly good with berry elongation. Price of the fruits grown by drip is invariably Rs.1-2 more per kg than conventionally irrigated fruits. Further, it is possible to get the fruits 2 to 3 weeks earlier in the market, maximum extra yield and price for the fruits, under micro-irrigation system. The net extra income due to micro-irrigation comes to Rs. 1,90,250/ha (Table 2). The payback periods for drip come is less than one year and more than 60 percent of the area in Maharashtra has come under drip. Data indicate that there is an imperative need to bring the entire grape area under drip to get more yield, more income and more foreign exchange for the country apart from saving water especially in water scarcity areas. Further, more area should be brought under grapes, which will give more income to the farmers and bring us more foreign exchange as there is a large scope for exporting grapes from India to other foreign countries.

Strawberry

It has been reported that the yield is about 5 Tons/ha. Daily about 2 to 3 liters/plant water is applied. Drippers are provided every 30 cm on lateral tubes. The total population of plants per ha is about 100,000 when irrigated by drip since no bed is required for drip. The per ha cost of cultivation by drip is about Rs. 60,000, and fruits are obtained 15 days earlier than under conventional irrigation methods with average yield of 7.5 Tons/ha. The net per ha extra income due to drip irrigation over conventional methods is about Rs. 92,500 and the B-C ratio is 2.34. Micro-irrigation of strawberries gives more production and profit to the farmers

Tomato

Introducing a drip system into tomato production requires the spacing of the crop modified to reduce the cost. Normally the spacing is 90 x 60 cm. Some farmers have changed to a pair row layout and use as spacing of 45 cm x 165 cm. LDPE Lateral / Drip Tape can be used as a lateral to give water. One hole every 30 cm is provided in the drip tape and about 50 m^3 / ha of water are given daily, i.e., about 5 mm/day. Yields with drip irrigation systems varies from 75 to 125 Tons/ha while 45 to 62.5 Tons/ha in conventional irrigation. Cultivation costs and crop duration can be reduced by Rs. 10,000/ha and 20 days, respectively, in drip as compared to conventional irrigation methods. Net extra income from micro irrigation over conventional irrigation is about Rs. 71,600/ha. The payback period for drip is one season for tomatoes. In view of increased yield, profit, and high in B-C ratios, micro-irrigation technology such as drip irrigation, is recommended for tomatoes.

Rose

The daily water requirement of well grown rose plants varies from 5 to 10 liters/day/plant. The initial cost of the system can be repaid in the first year since the profit with drip irrigation is very high (Table 3). It is advised to grow the rose crop on a large scale to meet the market demand.

For cereal crops, vegetables, relative performance of the crops grown in different locations of the country with micro-irrigation systems were better than for conventional irrigation. Performance was reflected in increased yield (Table 4). The water use efficiency in cereals like wheat, bajra, sorghum, barley, gram and maize were similar to those from groundnut, sunflower, cotton, sugarcane and vegetables. As a result the area under drip irrigation has increased from, 1000 ha in 1985 to 60,000 ha in 1993 and 2,54,000 ha in 1998. In another five years, projected area is estimated to increase about one million hectares which is 1% of the irrigated area, and about ten million hectares by 2020-2025 AD.

CONCLUSION

The net monetary returns, the benefit-cost ratio and other advantages have been show to be superior for micro-irrigation techniques compared to conventional irrigation. Farmer in the region should adopt micro-irrigation to increase on-farm production, income and water use efficiency for horticultural as well as agronomic crops.

Motivating farmers to adopt the technology will enhance water use efficiency, save irrigation water and increase production and productivity. Efforts should be made to bring down the costs of the system. Supply of standard material and equipment conforming to BIS standards need to be made available. Prompt customer services for maintenance and repair are essential. Farmers should develop conviction and confidence in the systems.

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Table 1. Economics of Drip Irrigation for Papaya, Pomegranate, and Banana

Particulars	Papaya		Pomegranate		Banana	
	Drip	Surface	Drip	Surface	Drip	Surface
Spacing (Meter)	2.4x1.8	2.4x1.8	4.2x4.2	4.2x4.2	1.5x1.5	1.5x1.5
Cost of drip system (Rs/ha.)*	40,000	-	30,000	-	45,000	-
a) Life 5 yrs for lateral/ drippers and 10 yrs for main, submain and filter	-	-	-	-	-	-
b) Depreciation (Rs/ha)	6800	-	5100	-	7650	-
c) Interest 13%(Rs/ha)	2400	-	1800	-	2700	-
d) Repair and maintenance(Rs/ha)	2000	-	1500	-	2250	-
e) Total (Rs/ha)	11200	-	8400	-	12600	-
Cost of cultivation (Rs/ha)	30,000	37,500	40,000	50,000	37,500	42,500
Seasonal total cost (3+2 e)(Rs/ha)	41,200	37,500	48,400	50,000	50,100	42,500
Water used (L/day/plant)	15	25	10	17.5	10	25
Yield of produce						
a) Latex (Kg/ha)	1,375	1,250	-	-	-	-
b) Fruits(Tons/ha)	150	100	23	17.5	75	50
Selling price						
a) Latex (Rs/kg)	50	50				
b) Fruits(Rs/ton)	500	500	10000	8000	2000	2000
Income from produce (6x7)(Rs/ha)	168750	112000	250000	140000	150000	100000
Net seasonal income (8-4)(Rs/ha)	127550	75000	201600	90000	99900	57500
Additional area cultivated due to saving of water (ha)	1	-	1	-	1	-
Additional expenditure due to additional area (4x10)(Rs/ha)	41200	-	48400	-	50100	-
Additional income due to additional area (8 x 10)(Rs/ha)	168750	-	250000	-	150000	-
Additional net income (12-11)(Rs/ha)	127550	-	201600	-	99900	-
Gross cost of production (4+11)/2 (Rs/ha)	41200	37500	48400	50000	50100	42500
Gross income (8+12)/2 (Rs/ha)	168750	112500	250000	140000	150000	100000
B.C. ratio (15/14)	4.09	3.00	5.17	2.80	3.00	2.35
Net extra income due to drip irrigation over conventional method Rs/ha [(13+9Drip)-9Surf]	52550 $1072	-	111600 $2277	-	42400 $865	-

* Approved by Government of Maharashtra State (India).

Table 2. Economics of Drip Irrigation for Grapes and Strawberry

Particulars	Grapes		Strawberry	
	Drip	Surface	Drip	Surface
Spacing(Meter)	3.0x 1.8	3.0x 1.8	3.6x2.7	2.4x1.5
Cost of drip system (Rs/ha.)	43,750	-	1,87,500	-
a) Life 5 yrs for lateral/ drippers and 10 yrs for main, sub-main and filter (Rs/ha.)	-	-	-	-
b) Depreciation(Rs/ha.)	7437.50	-	31875	-
c) Interest 13%(Rs/ha.)	2625	-	11250	-
d) Repair and maintenance(Rs/ha.)	2187.50	-	9375	-
e) Total(Rs/ha.)	12250	-	52500	-
Cost of cultivation(Rs/ha.)	125000	162500	60000	92500
Total Seasonal cost (Rs/ha.) (3+2 e)	137250	162500	112500	92500
Water used in (L/day/plant)	20	40	2	-
Yield of produce (Tons/ha)	45	30	7.5	5
Selling price (Rs/ton)	9000	8000	35000	30000
Income from produce (6x7) (Rs/ha.)	405000	240000	262500	150000
Net seasonal income (8-4) (Rs/ha.)	267750	77500	150000	57500
Additional area cultivated due to saving of water (ha)	1	-	0.5	-
Additional expenditure due to additional area (4x10) (Rs/ha.)	137250	-	112500	-
Additional income due to additional area (8 x 10) (Rs/ha.)	405000	-	262500	-
Additional net income (12-11) (Rs/ha.)	267750	-	150000	-
Gross cost of production (4+11)/2 (Rs/ha.)	137250	162500	112500	92500
Gross income (8+12)/2 (Rs/ha.)	405000	240000	262500	150000
B.C. ratio (15/14)	2.95	1.48	2.34	1.62
Net extra income due to drip irrigation over conventional method Rs/ha [(13+9Drip)-9Surf]	190250 $3883	-	92500 $1888	-

Table 3. Economics of Drip Irrigation for Tomato and Rose

Particulars	Tomato		Roses	
	Drip	Surface	Drip	Surface
Spacing(Meter)	1.65x0.45	0.9x 0.6	0.9x0.6	0.9x0.6
Cost of drip system. (Rs/ha.)	30,000	-	25,000	-
a) Life 5 yrs for lateral/ drippers and 10 yrs for main, sub-main and filter	-	-	-	-
b) Depreciation(Rs/ha.)	5100	-	4250	-
c) Interest 13%(Rs/ha.)	1800	-	1500	-
d) Repair and maintenance(Rs/ha.)	1500	-	1250	-
e) Total(Rs/ha.)	8400	-	7000	-
Cost of cultivation(Rs/ha.)	70000	80000	175000	225000
Total seasonal cost (Rs/ha.) (3+2 e)	78400	80000	182000	225000
Water used in(L/day/plant)	5	10	5	10
Yield of produce (Tons/ha) (Dozens/ha)	75	50	76500	76000
Selling price (Rs/ton) (Rs/doz)	2000	2000	6.00	4.80
Income from produce (6x7) (Rs/ha.)	150000	100000	459000	362200
Net seasonal income (8-4) (Rs/ha.)	71600	20000	277000	137200
Additional area cultivated due to saving of water (ha)	1	-	0.5	-
Additional expenditure due to additional area (4x10) (Rs/ha.)	78400	-	182000	-
Additional income due to additional area (8 x 10) (Rs/ha.)	150000	-	459000	-
Additional net income (12-11) (Rs/ha.)	71600	-	277000	-
Gross cost of production (4+11) (Rs/ha.)	156800	80000	182000	225000
Gross income (8+12) (Rs/ha.)	300000	100000	459000	367200
B.C. ratio (15/14)	1.91	1.25	2.52	1.63
Net extra income due to drip irrigation over conventional method [(13+9Drip)-9Surf]	71600 $1461	-	277000 $5653	-

Table 4. Relative Performance of Crops Grown Under Micro-Irrigation(Sprinkler) and Traditional Surface Irrigation Methods

Crop	Location	Yield (Q/ha)		Irrigation Water (cm)		Water Use Efficiency (Q/ha-cm)		Advantage of Sprinkler	
		Sur. Irri.	Spr Irri.	Sur. Irri.	Spr Irri.	Sur. Irri.	Spr Irri.	Water saving (cm)	Yield increase (Q/ha)
Cereals									
Wheat	Rahuri	32.41	36.39	35.0	20.25	0.93	1.79	14.75	3.98
	Udaipur	26.61	33.02	33.02	14.52	0.81	2.27	18.50	6.41
	Hissar	44.80	48.70	33.94	32.68	1.32	1.49	1.26	3.90
Bajra	Rahuri	6.97	8.33	17.78	7.82	0.39	1.07	9.96	1.36
Jowar	Rahuri	4.92	6.62	25.40	11.27	0.19	0.59	14.13	1.70
Barley	Bikaner	24.09	28.15	17.78	7.82	1.35	1.59	9.96	4.06
	Hissar	34.80	35.10	23.87	21.88	1.47	1.27	1.99	0.30
Gram	Hissar	6.55	9.91	17.78	7.82	0.37	1.27	9.96	3.36
Sorghum	Rahuri	44.12	54.97	18.00	12.00	2.45	4.58	6.00	10.85
Maize (Kharif)	Udaipur	15.62	18.10	12.80	9.00	1.22	2.01	3.80	2.48
Oil Seeds									
G.Nut (Summer)	Rahuri	23.24	28.98	90.00	62.00	0.26	0.47	28.00	5.74
	Junagarh	13.00	16.00	91.00	65.00	0.14	0.25	26.00	3.00
	Dharwad	33.96	39.86	76.30	63.60	0.45	0.63	12.70	5.90
	Punjab	5.50	11.90	68.60	50.20	0.08	0.24	18.40	6.40
	Navsari	30.00	31.00	56.00	44.00	0.55	0.68	12.00	1.00
(Kharif)	NCPA	8.33	9.34	60.00	30.00	0.14	0.31	30.00	1.01
	Rahuri	18.31	22.15	21.00	14.00	0.87	1.58	7.00	3.84
Sunflower (Rabi)	Rahuri	16.02	19.19	30.00	20.00	0.53	0.96	10.00	3.17
Cash Crops									
Cotton	Navsari	6.99	7.04	40.64	29.65	0.17	0.24	10.99	0.05
	Punjab	10.00	15.00	91.10	58.60	0.12	0.26	12.50	5.00
Sugarcane	Rahuri	792.1	866.3	245.0	188.0	3.23	4.61	57.00	74.20
	Dharwad	48.00	55.70	51.40	43.50	1.08	1.10	7.90	7.70
Vegetables									
Garlic	Rahuri	69.99	73.99	84.00	60.00	0.83	1.23	24.00	4.00
Chillies (Kharif)	Pune	17.41	21.52	36.00	24.00	0.48	0.89	12.00	4.11
	Rahuri	17.15	20.91	39.00	26.00	0.44	0.80	13.00	3.76
Onion (Summer)	Rahuri	334.9	412.7	78.00	52.00	4.29	7.94	26.00	77.80

SPRINKLER IRRIGATION SCHEDULING IN HUMID AREAS WITH THE EASY PAN

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ABSTRACT

The UGA EASY Pan Sprinkler Irrigation Scheduler is introduced as a simple, yet effective, method of scheduling irrigations in humid area row crop production. The device uses a wash tub, a toilet bowl float unit, and a covering screen. The unit can respond to both evaporation (as related to crop water use) and precipitation. The EASY Pan can be read from a distance allowing irrigation management personnel to determine crop water status without exiting a field vehicle. Preliminary field tests in South Georgia on peanuts and cotton indicate reasonable effectiveness when compared to standard irrigation scheduling approaches. Additional data is required to fully verify the pan's effectiveness. The simple nature of the unit allows in-field tests by farmers or other persons for their crop, soil and climate conditions.

INTRODUCTION

Irrigation scheduling for efficient water management in humid areas remains as one of the critical needs in humid area row crop production under sprinkler irrigation. Many techniques have been recommended for use by farmers: checkbook, moisture blocks, tensiometers, computer models, etc. (Yoder et al., 1998). The vast literature on conventional irrigation scheduling methods will not be reviewed in this paper. Suffice it to say, if soil water is managed properly, i.e., it is replenished within an appropriate time period before plants are stressed, irrigation has a positive benefit toward crop production. In the humid area, irrigation historically was designed to "supplement" rainfall. If rainfall was satisfactory, irrigation applications could be postponed until water needs increased. Therefore, irrigation scheduling in the humid regions is a prime target for "neglect". If records and instrumentation are not maintained consistently, even through periods of rainfall, schedules are disrupted, and irrigation applications are poorly timed with inappropriate amounts of water being applied.

The relationship between pan evaporation and evapotranspiration under a crop

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canopy has been investigated and used extensively as an alternative to installed soil water sensors for irrigation scheduling (Wolfe and Evans, 1964;). A relationship does exist between surface evaporation and consumptive use of crops, if certain conditions are met. A free water surface exposed to excessive wind and solar radiation may not respond the same as a plant canopy. If some form of "covering" or wind protection is provided for the pan surface, the relationship improves.

The typical Class A evaporation pan found in many weather station applications is the standard in evaporation measurements, but they are rather expensive (most are stainless steel) for individual farmer applications. The use of alternative, less expensive, evaporation measuring devices provided the foundation for the UGA EASY Pan.

The use of alternative evaporation-based devices is not new. Torres (1998) used a plastic bucket as a visual device for irrigation scheduling of sugarcane in Colombia. The device was designed to work as a rain gage and as a measuring device for evaporation. The bucket approach had two settings, one for irrigation of sugarcane younger than four months, and one for older than four months. The depth of the bucket was checked periodically to determine the need for irrigation. An overflow "hole" allowed excess rainfall to leave the bucket as the soil profile was returned to field capacity. The need to consistently check the plastic bucket elevation, eliminate algae growth and animal impacts, and maintain good working conditions over the long sugarcane growing season makes this particular approach ripe for damage and neglect. In addition, the user was still required to go to the bucket to obtain a water depth reading.

Westesen and Hanson (1981) reported results of irrigation scheduling using a wash tub as the evaporation scheduling device. Their approach used a no. 1 washtub with a wire screen on top to keep animals out. Their approach to managing water included a separate rainfall measurement to take into account effective rainfall conditions. The elevation of the pan and the raingage had to be read consistently while maintaining good bookkeeping records. Recommendations for "how much" rainfall was considered effective had to be calculated as well. The user was also required to go to the wash tub to get a water level reading. This particular approach worked reasonably well, but was poorly accepted by local farmers in Montana.

The problem with most of these techniques is the maintenance required, the additional time spent in determining the status of the water level in the tub, and whether the pan level truly represents soil water conditions. Experience with row crop farmers in South Georgia indicates that new technologies will be considered as long as it does not take too much time (or none of their time), does not interfere with field operations, is reliable, makes good recommendations, and is not too expensive.

A prime example of technology adoption for irrigation scheduling is the implementation of model-based systems like Irrigator Pro©[5] (a development of the USDA, ARS, National Peanut Laboratory, that is currently marketed through the Peanut Foundation, Inc., Davidson et. al, 2001). This particular system is currently available for peanut irrigation scheduling only. Other crops are anticipated to be added to the system, but the expert-system based recommendations will require significant development for some other crops. No widely used irrigation scheduling model techniques are currently being used for humid area applications for many of the potential crops that are being grown (cotton, corn, soybean, vegetables, etc.).

This paper describes the working characteristics of the UGA EASY Pan (Evaporation-based Accumulator for Sprinkler-enhanced Yield). How the pan works, its relationship to other irrigation scheduling technologies, and relative costs will be discussed. For additional information on the UGA EASY Pan (including construction schematics), the reader is referred to the Cooperative Extension Service Publications of the University of Georgia, Biological and Agricultural Engineering Department (Thomas et al., 2001).

Figure 1. The EASY Pan Irrigation Scheduler in operation on cotton research plots.

MATERIALS AND METHODS

The EASY Pan Irrigation Scheduler includes a no. 3 galvanized washtub, a screen cover, a float system, and an indicator back plate (Fig. 1). The washtub has at least two holes drilled at 90 mm (3.5 inches) from the top of the tub. These holes allow excess rainfall water to be removed from the pan (as is similar with soil). The covering screen can include a variety of different materials.

[5]The use of tradenames, etc. in this publication does not imply endorsement by the University of Georgia of the product named nor criticism of similar products not mentioned.

The primary purpose for the screen cover is to keep animals out and provide some level of limitation on the evaporation rate (Fig. 2). In tests in South Georgia, 25 mm (1 in.) to 50 mm (2 in.) mesh chicken wire has performed well with peanut irrigation. Standard window screen has worked well with cotton (see field test section). Since both these crops have very different relationships to pan evaporation, the limitation on air contact and the reduction in temperature for the free water surface reduces the evaporation rate when using the window screen.

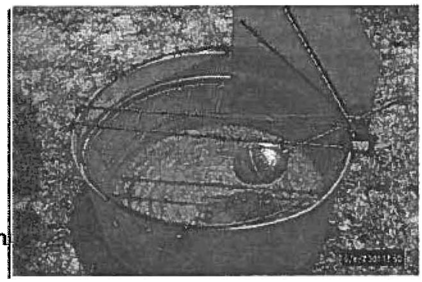

Figure 1. Entire system. When the soil water is full, the white indicator arm covers the black line. As water is removed, the indicator arm moves toward the red line (irrigate).

The unique operating characteristic of the unit is the float and indicator arm. The float is mounted on an aluminum rod that can be lengthened or shortened based on the soil water holding characteristics, the rooting depth of the crop, and the management allowed depletion (MAD) for the crop. Since free surface evaporation is more available and consistent than soil water removal by plants, the relationship is strong, but is not always "one to one". However, on the average, which is the benefit of this type device, the relationship is good.

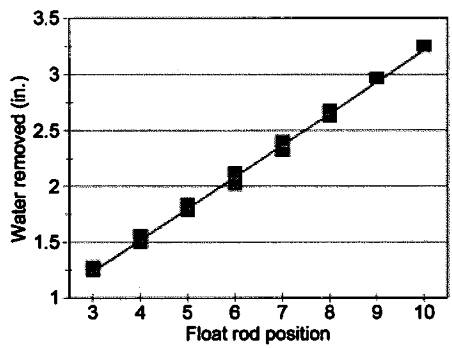

Figure 3. The relationship between water removed and the float rod position based on the indicator arm moving from field capacity, black line on the back board, to irrigation required, red line on the back board (15 degrees).

Field Tests

Preliminary irrigation scheduling tests were conducted over a two year period to test the EASY Pan recommendations for irrigation scheduling. Two years of tests were performed on cotton and peanuts. The first application of the EASY pan was in 1995. The 1996 season was used to test the response of different screen materials on the evaporation from the EASY Pan. The peanut irrigation season in 1997 resulted in some logistics problems with the irrigation application system.

Those data are not included in this discussion. The following results are preliminary, but are indicative of irrigation scheduling trends between the different approaches. All field management, planting date, chemical application, and treatment information for the cotton and peanut field tests are available on the following web site:
http://nespal.cpes.peachnet.edu/agwateruse/research/default.asp.

1995 Cotton: Cotton (Georgia King) was grown on the Gibbs Research farm near Tifton, GA on a Tifton loamy sand in a randomized complete block design with five replications, using solid set sprinkler irrigation (Rainbird 35A PJ-ADJ-TNT sprinklers with #11, 11/64 in., nozzles for an 8 mm/hr application rate with all sprinklers operating full circle). Each irrigation plot was 14.6 m x 11 m. The treatments were: 1) soil water content scheduling: if the average soil water content in the 10 to 45 cm depth exceeds 35 kPa, schedule an irrigation for the next day, 2) irrigation schedule based on the Neogen Envirocaster® recommendations for cotton production, 3) the EASY Pan with 50 mm (2 in.) mesh wire screen and the float rod set at the 5 in. position, and 4) no irrigation.

1997 Cotton: Cotton (DP5415) was grown at the same research site and plot design as in 1995 with treatments (that are important to this discussion) of: 1) same as treatment 1 from 1995, 2) water applications based on the Extension Service checkbook scheduling method: replace needed water once each week, 3) the EASY Pan with standard wire window screen and the float rod set at the 5 in. position, and 4) no irrigation.

1995 Peanut: Peanut (Georgia Runner) was grown at the Gin House Field site in Tifton, GA, on a Tifton loamy sand in a randomized complete block design with four replications, using the same sprinklers as in the cotton trials on a 12.2 m x 12.2 m plot spacing. The treatments important to this discussion were 1-4, the same as the 1995 trial for cotton production: 1) soil water content scheduling, 2) Neogen sensors, 3) EASY Pan with 50 mm mesh wire screen, and 4) no irrigation.

RESULTS

For each of the following results, some irrigation water was actually applied to the "no irrigation" treatment at the beginning of the season to get the crop up.

The 1995 cotton season resulted in the following yields (Fig. 4) and water applications (Fig. 5). The 1995 growing season was relatively dry. The EASY Pan had the highest recommended irrigation application, which was not reflected in the yield. This particular result was the primary reason for switching the screen material for the 1997 season.

The 1997 cotton season resulted in the following yields (Fig. 6) and water applications (Fig. 7). Irrigations scheduled with the EASY pan were slightly less (151.4 mm) than those applied by the control treatment (156.4). Very little difference in yield was reflected by those two treatments. The current Extension Service recommendations for cotton irrigation resulted in a relatively large application amount with no direct yield benefit for this trial. The EASY Pan irrigation recommendations appeared to be very similar to those based on soil moisture management.

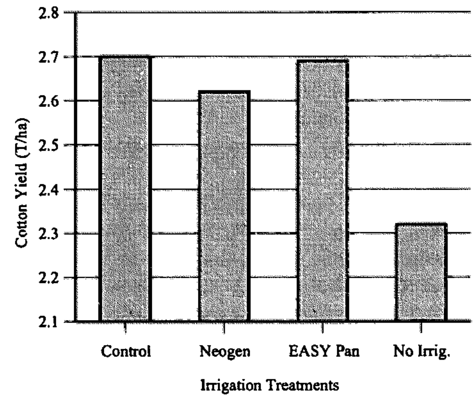

Figure 4. Cotton yield response for the 1995 season for the indicated irrigation treatments.

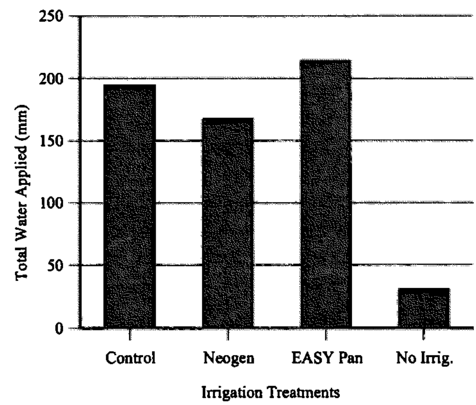

Figure 5. Total irrigation water applied during the 1995 cotton growing season.

During the 1997 cotton season, the position of the float arm was measured three times per week to determine the response characteristics due to evaporation and rainfall. Fig. 8 illustrates the degree change over time. The "0" degree value represents "field capacity" for the soil. Obviously, the pan is much more responsive to water application by precipitation or irrigation than would be expected with the soil. One characteristic of interest, is the EASY Pan response to "deficit irrigation". Irrigation events on 7/16 and 9/10 did not return the pan (or soil) to the field capacity (0 degree) position. Therefore, some of the rainfall following the 7/16 irrigation was effective toward replenishment of the soil water. The ability to manage irrigations while taking advantage of expected rainfall provides a more efficient use of available water resources in humid areas.

The 1995 peanut season resulted in very similar irrigation water applications between the control and EASY pan treatments (Fig. 9). Yield results for peanut under the three irrigation treatment scenarios were not different. Water required by both the Control treatment and the EASY Pan were essentially the same. This result indicates good agreement between the two methods in scheduling irrigations under the conditions tested.

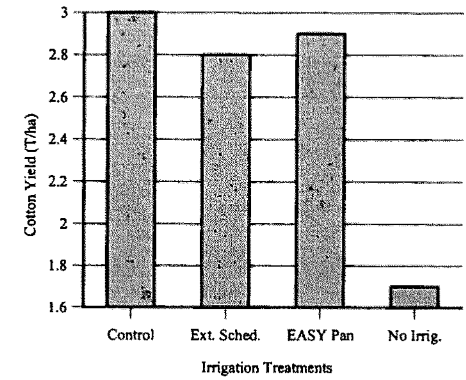

Figure 6. Cotton yield response for the 1997 season for the indicated irrigation treatments.

Limited data were presented to indicate the relative response of the UGA EASY Pan for scheduling sprinkler irrigation on both cotton and peanuts. The EASY pan responded reasonably well as compared to other accepted techniques for irrigation scheduling. The screen material and the float rod position need to be selected to most accurately reflect the characteristics of the soil and crop. The few parameters that can be adjusted on this unit makes it easy to understand and apply to sprinkler irrigated crop production in the humid region.

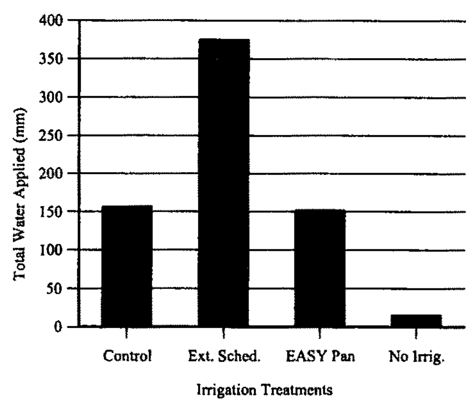

Figure 7. Total irrigation water applied during the 1997 cotton growing season.

The ability to determine the need for irrigation in the field without leaving a field vehicle is considered desirable by many farmers in South Georgia. The relative low cost (currently about $100, in 2002 $, if all components are purchased), and the available construction schematics allows farmers to create their own EASY Pan units. The potential for farmers to use the EASY Pan on other crops, following their own field testing, is also a reasonable assumption. The EASY Pan results indicate that it is a reasonable alternative for scheduling irrigations.

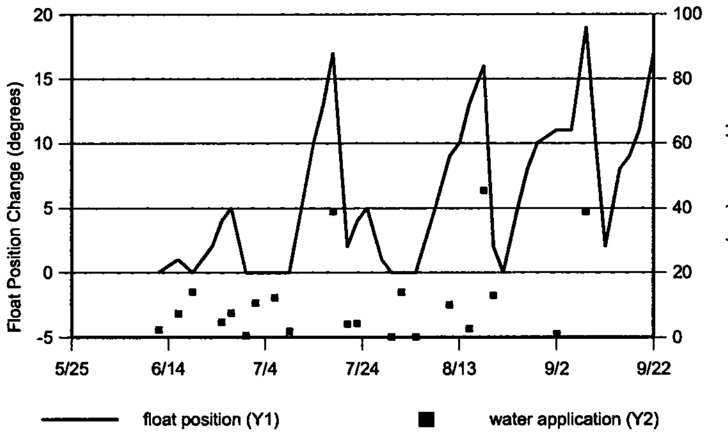

Figure 8. Response of the EASY Pan float indicator over the 1997 cotton irrigation season with rainfall/irrigation applications. Irrigation events were initiated after the float rod indicator reached 15 degrees away from the starting float position.

As with any other scheduling technologies, the EASY Pan does have limitations. It will not indicate when to stop irrigating. The unit works best if the surface of the pan is near the crop canopy height, so some method of raising the pan (recommendations include using cinder/concrete blocks) during the season is necessary. The EASY Pan does require some maintenance over a season to reduce algae growth and remove any foreign material that may have collected in the pan.

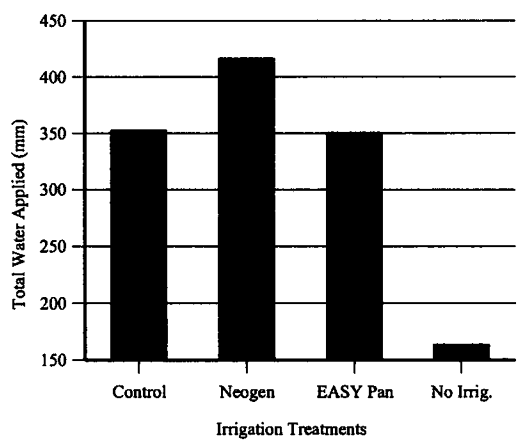

Figure 9. Total irrigation water applied during the 1995 peanut growing season.

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USE OF HYDROLOGICAL MODELS IN WATER DESIGN PROJECT IN TURKMENISTAN

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ABSTRACT

In 1998 irrigated area of Turkmenistan was 1750 thousand ha and total water intake was 22.8 km^3 which was enough to meet total agricultural water requirements. Pursuant to the programs of the President of Turkmenistan, the irrigated area must be brought to 2250 thousand ha by 2010, but the volume of water taken from the Amudarya river must be kept as 22 km^3. Thus in the future irrigation water requirements including the necessity of irrigated area extension can be met only under specific conditions. The given situation demands much from the justification and quality of the projects. In this connection, at present, in the course of designing and renovation of major water projects (reservoirs, main canals and drainage systems) of Turkmenistan, hydrological models and engineering calculation schemes are being used. Sources of initial hydrological data and methods of their processing (depending upon the sphere of their utilization, results of hydrological studies and the problems which solution they will be used for) are under consideration in the Report.

INTRODUCTION

Turkmenistan has very limited natural water resources. Its principal surface waters arise outside the country and are, as a consequence, trans-boundary resources. Annual and longer-term river flow rates are erratic. They are characterized by high turbidities, especially during flood periods. These cause flood control and sediment flow problems in water conduit sections of all of the

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major canals. Water intake from the Amudarya River, the major surface source, is limited to 22 km^3 - in accordance with inter-Governmental agreements. As a consequence of this, irrigated agriculture - which is the basis of agriculture in Turkmenistan – has developed under conditions of water resource deficiency. Water conduit and drainage sections of the canal (from dozens up to 1000 km in length) are usually open and flow through sands under complex hydro-geological conditions. Drainage waters are highly saline, and soils along the canal routes have different degrees of salinity and their seepage capacities also differ. All major hydraulic structures are located in highly seismic zones. These require that specific hydrologic and technical problems be solved in relation to their safety. In this connection, hydrological models (amongst others) have recently been used in the design and renovation of major water projects (reservoirs, main canals and drainage systems) in Turkmenistan. These models have been implemented as independent program modules and as engineering calculation schemes in MS Excel medium for the determination of the following:

1. Hydrologic characteristics of river design (annual, flood, maximum daily average discharge, and pre-set excess probability flow volumes).

2. Flood hydrograph design, according to various scenarios.

3. Catastrophic flood control through reservoirs.

4. Optional parameters (total and effective storage capacities, excess water volumes, and water intake diagrams) for reservoir and flow control modes over a period of several years.

5. Parameters and hydropower plant operating modes. Sediment discharge volumes.

6. Seepage losses (natural seepage, bank backwater seepage in dam bodies and their foundations) and evaporation losses.

7. Breakthrough wave parameters under conditions of storage dam failure, and analysis of their subsequent economic damage.

8. Expedience of economic reservoir construction for alternative reservoir capacities, intake within annual and long-term periods, and agricultural specialization on irrigated lands.

9. Salt balances in reservoirs, irrigation, drainage and water canals.

ACTUAL AND DESIGN HYDROLOGIC CHARACTERISTICS

A series of actual observations over 60 – 90 years are available for daily average discharges and water levels for the three main rivers (Amudarya, Murgab and Tejen) that contribute to the Turkmenistan water balance. Usually the design hydrologic characteristics of the latter two rivers are rarely altered (not more often than once in 10 years). When they are it has been in connection with the construction of large reservoirs or after great floods. From the point of view of hydrology, the Tejen River is the most interesting. It is characterized by extremely irregular annual and intra-annual flows. The average flow of the river near the Pulihatune gauge station over many years (64 years of continuous observations) is 1066 Mm^3. The minimum recorded value is 92.1 Mm^3 and maximum 4145 Mm^3, including respectively 84.1 Mm^3 and 3855.7 Mm^3 for the limiting period (February – June). The average discharge over many years is 35.3 m^3/s. The minimum recorded value is 2.9 m^3/s and maximum is 131.5 m^3/s, including respectively 6.5 m^3/s and 297.5 m^3/s for the limiting period (February – June). The Tejen River's intra-annual flow distribution is unfavorable for irrigation: 80 - 85 % of the flow passes during the period March - May. The river actually dries up in the July - August period when demand for irrigation water is greatest. The maximum observed daily average discharge during the flood period is 1090 m^3/s. In one year, the maximum value was only 26 m^3/s. Full-scale hydrologic calculations were made in 1996 – 1998 when the Pulihatune reservoir was designed on the Tejen (Herirude) River between Turkmenistan and Iran. Hydrologic and water economy calculation data was used in all the above models. Although it was not actualized in a single program, it did in actual fact form a single calculation system. A brief description of the basic calculation blocks (models) of the calculation system is given below.

INITIAL HYDROLOGIC DATA

Data from the Pulihatune, Turkmen gauge station has been used for daily and monthly average flows for the period 1915 – 1978 (the station was subsequently ruined by flooding and has so far not been restored). Data from a similar Iranian gauge station has been used for the period of 1968 – 1994. After checking the series for coinciding periods, values were reduced to a homogeneous row on the basis of correlation dependence. Samples from the 69-year series of observations were used for all subsequent calculations.

Empirical distribution curves are smoothed and extrapolated by means of SNiP (Construction norms and specifications) 2.01.14-83 «Determination of Design for Hydrologic Characteristics». These provide for the use of three-parameter gamma distributions that are generally being used in Turkmenistan up to the present day. Our experience of hydrologic data processing shows that for practical calculations Pearson's Type III Analytical Curve is more convenient to use. We used a computer actualization of the design algorithm developed by one of the authors of the Report. This actualization involved the statistical processing of a series of daily observations of direct and total solar radiation taken over many years (using a modified equation of Pearson's curve expressed through complete and incomplete gamma functions). Analytical distribution curve parameters (average, coefficient of variation, asymmetry coefficient, etc.) forming the equation were determined by the moment method. Output data were presented in the form of diagrams and tables (through 1% of probability, and 99.99%, 99.9%, 0.1%, 0.01% on the edges of the distribution respectively) – average value, upper and lower limit of the confidence level (p=0.95%) for the corresponding probability value.

In view of the great spread of maximum daily average discharge values during the flood period, a 58-year truncated sample was used to calculate analytical curves instead of using all 69-year observation data. This means that the dry year data were excluded from the sample. When judged from the formation of the flow, a real flood - as such - did not take place. A complete sample calculation would require using a rather long series of observations and other analytical distribution curves (which would result in lowering the calculation accuracy in the most critical part of the curve - 0.1, 0.01% of probability). From the calculation results:

> Flow volume P = 0.01% probability for one year is 8351 Mm^3; for the flood period (February – June) it is 7559 Mm^3;
>
> Maximum daily average discharge P = 0.01% probability for the flood period is 2192 m^3/s.

Flood Hydrograph Models

Design parameters are accepted on the basis of upper limit of confidence probability. First class safety structures whose damage could result in disaster and great losses are designed allowing for emergency operation, i.e. for P = 0.1% exceedence probability (basic calculation) catastrophic flood control and for P = 0.01% exceedence probability (check calculation). Flood volume and maximum discharge of pre-set exceedence probability have been determined for the February – June period, which includes the beginning and end of the flood period in a number of the observed years. Based on the observation data for the flood period, design probability flows and discharges were determined by mathematical statistics methods. They are 7559 Mm^3 and 2192 m^3/s when P = 0.01% and 5843 Mm^3 and 1759 m^3/s when P = 0.1% respectively. Design hydrographs were made up on the basis of real hydrographs of the greatest floods occurring during the available observation period. Those years with the highest annual flow value, maximum discharge peaks and typical time flow distributions (1939, 1991, 1992 and 1993) were chosen as model years on which scenarios of design flood hydrograph were made. A typical year was chosen from a real year for which the basic statistical parameters were close to the average over many years. Ordinates of design hydrograph sections (150-day period from 1 February to 30 June) were determined by equidistant transformation of ordinates of the observed flood daily average discharges. The design hydrographs are then used to design a spillway structure and the alternatives for floodwater transportation through it.

SEEPAGE LOSSES FROM THE RESERVOIR

Total seepage losses from the reservoir consist of temporary seepage losses during saturation of the aeration zone, backwater bank seepage, annual full saturation of the reservoir basin and edges, evaporation losses and seepage from the dam body and its foundation. To analyze seepage losses, so-called geo-seepage model of the reservoir basin was developed. It consists of nine geological and hydro-geological cross sections; the distance between them is from 1 – 2 km up to 4 – 5 km. They reflect geological and lithological structure, initial hydro-geological conditions and rock seepage properties. Based on the results of the filtration profile solutions annual seepage losses from the reservoir were estimated, taking into account the reservoir design operation mode and the special features of the Herirude River's hydrology. As the length of the Report is limited, a detailed description of the models used for the calculation of the losses is not given. It should be mentioned however that they were developed in different research institutes of the former USSR and actualized as computer programs in FORTRAN in the 80's and 90's.

Sediment Discharge

The annual average turbidity of the Tejen River is 13.0 kg/m^3 according to the Pulihatune gauge station data (fluctuations are 2.5 kg/m^3 – 190 kg/m^3). The average discharge of suspended sediments below the confluence of the Herirude and Keshifrude rivers (Pulihatune gauge station data) is 360 kg/s for the design number of years, i.e. 1929 - 74. When the liquid discharge is 30.8 m^3/s, the turbidity value is 11.7 kg/m^3 for the same period. The annual volume of silt accumulation is 8.1Mm^3 when the (many-year) average volume of liquid flow is 970 Mm^3 (for 1929 - 74 years) and the volumetric weight of sediments is 1.39 kg/m^3. In view of the fact that sediment flow values in the Keshifrud River, the Tejen's tributary, have a great influence on the suspended sediment value in the Pulihatune gauge line, the annual volume of silt accumulation is slightly less than 7.5Mm^3. The annual reservoir dead storage capacity is accepted as being 300 Mm^3. This value must be enough for the 45-50 year period of the reservoir's operation.

MODE OF RESERVOIR OPERATION IN ACTUAL YEARS

Discharge control calculations have been made in real years to determine the reservoir's rational capacity - within the range of 600 Mm^3 – 1500 Mm^3. Under the preset intake diagram for 420 Mm^3 – 920 Mm^3 per year, the flow alternatives in the Pulihatune reservoir line are as follows:

1. Under many-year average flow S = 1070.8 Mm^3;
2. Under many-year average flow considering lower limit of confidence level S = 867 Mm^3;
3. If 600 Mm^3 Salma reservoir (Afghanistan) with 300- 400 Mm^3 intake is available.

Calculations for river flow control under varied reservoir preset capacities and irrigation volumes contain the following:

1. Many-year average values for evaporation and seepage losses from the reservoir.

2. Discharges, actual intakes, many-year average reservoir volumes, water levels and filling respectively.

3. Many-year average filling coefficients (the ratio of many-year average reservoir capacity to planned reservoir capacity).

4. Probability of intake assuming water supply continuity (a number of years with 100 % efficiency to a number of years of a designed series ratio – year continuity water supply coefficient).

5. Volume intake probability coefficient – ratio of many-year average actual intake to planned intake volumes.

6. Coefficients of river flow control (ratio of actual intake to many-year average river flow).

A 59-year continuous sample (from 1936 to 1994) of actual monthly average discharges from the Tejen River was used as the initial data. It was assumed that the sample was stationary and homogeneous, and that its statistical characteristics within accidental error would be maintained for the future period of the reservoir's life. In this case one of the calculation alternatives assumed the evaluation of the reservoir operational mode under the forecast of many-year average flow in accordance with the lower confidence level. Water balance calculation schemes were actualized using the MS Excel medium. The value for reservoir capacity and intake at a guaranteed efficiency is chosen on the basis of calculated data and from general considerations. Basic characteristics for long-term flow control are the guarantee of many-year fail-safe flow transportation and the guarantee of the adequate volume of water supply.

P = 0.01 % PROBABILITY FLOOD CONTROL AND DETERMINATION OF SURFACE DISCHARGE VALUES

A design hydrograph made upon the scenario of 1992, as the most unfavorable year from the point of view of flood transportation dynamics, was used for the flood calculation and control. Calculation conditions are as follows:

A constant discharge, 30 m^3/s, is carried out through the dam's bottom outlet in accordance with the schedule of irrigation demand and water supply.

Calculation of regulation is made in several versions with specific reservoir capacity from 850 to 1250 Mm^3 - total reservoir capacity at surcharged level, from 700 to 1000 Mm^3 actual capacity at full reservoir level and intake from 420 to 920 Mm^3.

By the flood beginning (1 February) the reservoir is filled up to the full reservoir level.

Flood water is started to be discharged when the water level in the reservoir is higher than the full reservoir level.

Flood water transportation starts when the water level is higher than the structure's crest. Volume of the discharged water increases as the reservoir volume increases when the flood reservoir is filled and reaches its maximum value under the surcharged reservoir level. Scheme of water balance calculation (together with the hydraulic calculation of surface discharge) was actualized on the basis of the catastrophic flood design hydrographs.

STABILITY OF DAM SLOPES

Calculations determining cross-section of the rockfill dam with a loamy core were made with the help of the "STAB" program under various combinations of loads and exposure to various actions for the dam 1st safety class. The following load and effect combinations have been considered:

Static version with level of water in reservoir at mark of surcharged reservoir level = 475.00 m, without earthquake effect and with earthquake effect ($K_{horiz} = 0.15$ g);

Dynamic version with fast drawdown of reservoir from mark of Full Supply Level = 470.00 m to mark of Dead-Storage Level = 445.50 m in 63 days (discharge $Q = 120$ m^3/sec; $\Delta h = 0.4$ m/day, at $K_{horiz} = 0.15$g);

Dynamic version with fast drawdown of reservoir from mark of Full Supply Level = 470.00 m to mark of Dead-Storage Level = 445.50 m in 6 days (discharge $Q = 1200$ m^3/sec; $\Delta h = 4$ m/day, at $K_{horiz} = 0.15$ g).

AGROECONOMIC CALCULATIONS

Basis for the agroeconomic calculations was the determination of the reservoir construction economic feasibility and its optimum capacity, which were calculated for various alternatives varying the reservoir capacity from 600 to 1500 Mm^3, intake from 600 to 1500 Mm^3 flow, structure of irrigated lands, cost indexes, engineering solutions and water division between Turkmenistan and Iran. The economic figures on irrigated area of Turkmenistan were calculated under the following structure:

Version of a cotton specialization: wheat - 25 %, barley - 5.5 %, cotton - 25 %, alfalfa - 32.5 %, orchards - 4 %, vegetables - 8 %, other - 4 %.

Version of a vinicultural specialization: wheat - 4 %, corn - 6 %, vineyard - 67 %, lucerne - 6 %, gardens - 5 %, vegetables - 8 %, other - 4 %.

All economic calculations for various alternatives were done on the basis of the calculation of net present value (NPV) and internal rate of return (IRR). In order to determine an economically beneficial alternative of the reservoir capacity and the intake volume, elements of optimization calculation were used. While carrying out optimization calculations for the actual hydrological series the irrigated area was assumed in accordance with the value of planned water intake (420 ... 920 Mm^3/year). Income from agricultural activity was taken into account only for the lands, which had a guaranteed supply with water. Wasteful expenditure on the lands not irrigated in some specific years were referred to the costs.

BREAKTHROUGH WAVE

In case of the dam failure under force majeure conditions, a breakthrough wave is formed. Calculations were carried out with the help of the "WAVE" program (developed by Scientific and Industrial Enterprise SANIIRI, Uzbekistan) that has been widely used in the CAR countries for more than 10 years. As the result of the calculation the following have been determined: time of wavefront travel to section line involved in calculation starting from the moment of dam breakthrough, wavefront width, maximum height, velocity of wave travel, and discharges at discharge sites. Forty design section lines were determined in the section from the dam site to the Garagum canal section line which is 166 km in length. Two zones can be selected in this section of the Tejen (Herirude) river, they are as follows: wave front zone and flooding zone. Calculation of damage resulting from the breakthrough wave has been done considering the cost of economic enterprises located in the zones and rehabilitation work costs.

MODERN LARGE WATER PROJECTS OF TURKMENISTAN AS PROJECTS UNDER HYDROLOGICAL STUDIES

First it is Turkmen Lake which projects' construction has started recently in Turkmenistan. As far as this project is concerned the following should be mentioned. Social and economic reforms reflected in **"Strategy of social and economic reforms in Turkmenistan for the period of up to 2010"**, new program of President Saparmurat Turkmenbashy, provide a stable development of all branches of national economy, including gaining of food independence of the country. In this connection water resources will be mainly used for crop production on the irrigated areas in the nearest future. Nowadays total volume of drainage waters formed in the irrigated areas of Turkmenistan is estimated as 6.0 km^3 and including drainage waters coming from the territory of neighboring country (Republic of Uzbekistan), this figure can become more than 11.0 km^3 (1998 data). In Lebap velayat (velayat means region) most part of the drainage waters is drained to the Amudarya and the smaller part of them is transported to the natural depressions of the locality. Waters from the territory of Turkmenistan drained to the Amudarya and drainage waters from Uzbekistan cause low quality of the river water resulting in high salinity in the middle and lower parts of the river and negatively influencing social, economical and environmental situation in Dashoguz velayat (Turkmenistan), Karakalpakistan (Uzbekistan) and Aral Sea region of Kazakhstan. Drainage waters formed within Mary, Ahal and Balkan velayats are transported to the natural depressions of the Karakum desert and on their ways to the depressions they flood pastures, causing decrease of pasture areas or decline in their efficiency. Under these conditions quick salinization of soils makes desert species of flora and fauna poor. For the purpose of efficient use of drainage waters, creation of water reserve fund and environment improvement in the country, President of Turkmenistan decided to construct a unique 132 km^3 capacity Turkmen lake in the center of the Karakum desert (in Karashor natural depression) in April 2000. Proceed from all above mentioned, it is quiet obvious that the implementation of this project will require carrying-out of the whole complex of research works on different technical disciplines of water management sphere. It is planned to select the following characteristics under study: stationary and dynamic water-salt balance of drainage system and water reservoirs; dynamics of hydraulic regime through transit drainage canal lengths taking into consideration irregularity of supply, canal annual flow; expert estimation of canal bed processes. It is planned to use modern computer technologies widely while preparing this work (computer models, GIS, time-tested program complexes, search for information through Internet, etc.).

It is expected that the fulfillment of the given work will allow to give the fist appraisal of the possible solution of the problems regarding the Turkmen lake project necessary for the further reasoning for design, site investigation and construction works. It is also expected that the work will help to initiate a number of joint studies in the sphere of priority trends of the Turkmen lake project with the participation of foreign companies (experts) within the frameworks of international programs.

It should be taking into consideration that there are two more groups of water structures in Turkmenistan. From the point of view of financing the priority should be given to these two groups as compare to Turkmen lake project, as they also influence the stability of economic and social development of the country. These two groups of water structures are as follow - the Karakum canal and existing water reservoirs. Despite the fact that the problems of these groups is not a subject of this work, one can say for sure that they require close attention and taking of urgent measures not in the future, but now. Delay in the solving of this problem can lead to great negative results not only for water and agricultural spheres, but also for the country's economy on the whole.

ECONOMIC AND MATHEMATICAL MODEL FOR OPTIMIZATIONAL USE OF WATER AND LAND RESOURCES

We would like to draw the specialists' attention to one more of the aspects that relates to hydrology indirectly – the sphere of the hydrological study result utilization and the problems which solution they will be used for. Water activity of the Central Asian Region (CAR) countries in the Aral Sea basin is carried out on the background of developing ecological crisis under conditions of water deficit, state different interests in the water strategy, and operational water management. After split of the USSR and establishment of independence states there is a new political and economic situation, including water management, which has formed in the region. The situation needs a review of all problems at the national level and then their interconnecting and coordination on the regional level, where Aral Sea should be counted as the separate main consumer. Solution of these multilevel problems, related by common water resources and by their lack, requires an accounting of all level priorities and their dynamics in the temporal scale. The main purpose of optimization and simulation mathematical models development is the creation of the instrument for water planning and water management under conditions of water deficiency, limited financial resources of the CAR countries and the approved commitment of guarantee a water supply to the Aral Sea. In this context, complex models which were prepared not only considering hydrological features of the region main water sources but also considering the requirements of real economic and political situations in the countries of the region will be of great interest for the CAR countries according to our opinion. One of the authors' works aimed at identifying the expediency and potential for utilizing economic and mathematical models to solve basic problems in CAR countries by integrating and specifying agricultural production in the Aral Sea basin countries. In regions where one of the main purposes of the project is the solution of environmental problems, special attention must be paid to "a virtual redistribution of water resources" and not to a direct redistribution of water resources - although this issue has already been touched on by some politicians and scientists. The distribution of "virtual water resources" means the import of foodstuff from one part of the region to another, since the imported foodstuff contains water that was used in its production. Based on such an approach, it is possible to consider and compute optimal alternatives for the location of agricultural production that will provide maximum profit and minimum environmental impact in the region. It is obvious that no country will approve a policy that will lead to heavy dependence on food imports. However, a policy of full self-sufficiency in basic types of agricultural production - which has been implemented in CAR countries (based on the available water resources) - pays more attention to political and social criteria than to economic and environmental ones. Implementation of this policy for a long period of time, set against the background of the actual demographic and environmental situation, will lead to the aggravation of negative events.

TOOLS FOR DESIGN, CALIBRATION, CONSTRUCTION AND USE OF LONG-THROATED FLUMES AND BROAD-CRESTED WEIRS

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ABSTRACT

Long-throated flumes and broad-crested weirs provide a practical, low-cost, flexible means of measuring open-channel flows in new and existing irrigation systems and have distinct advantages over other flume and weir devices. Application of these flumes and weirs has been greatly facilitated by the 1999 release of the WinFlume software used to design and calibrate these structures, and the recent publication of *Water Measurement with Flumes and Weirs*, a text providing comprehensive information on design, calibration, construction, and operation issues. The primary advantages of these flumes and weirs are that they can be custom-designed to satisfy unique operational and site requirements, and they can be computer calibrated without the need for laboratory testing. In addition, these devices are easily and economically constructed, and a number of commercially built, pre-calibrated devices are available. This paper and accompanying poster describe the use of the WinFlume software and present examples to illustrate application to a range of situations, including various flow rates, channel types, and construction techniques.

INTRODUCTION

The term *long-throated flume* describes a broad family of critical-flow flumes and broad-crested weirs used to measure open-channel flows. A variety of specific configurations are possible (Fig. 1). Bos et al. (1984) described the theory for determining discharge through these flumes. WinFlume is the most recent in a

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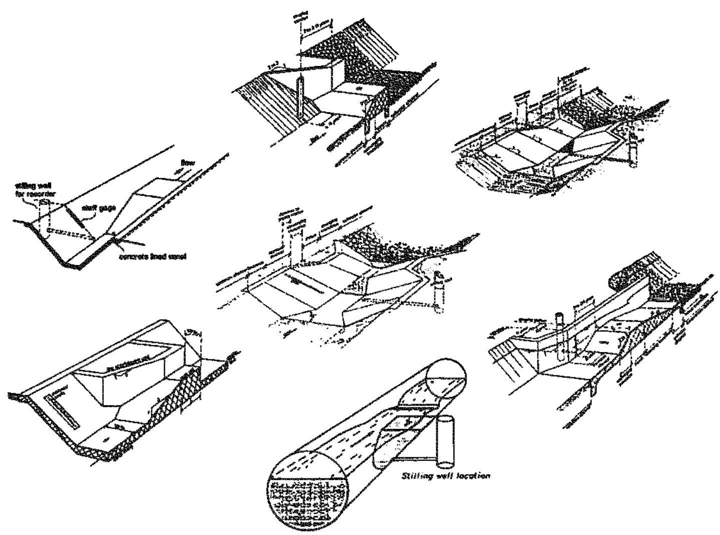

Figure 1. Examples of Long-Throated Flumes and Broad-Crested Weirs.

series of computer programs used to design and calibrate these structures. *Water Measurement with Flumes and Weirs* (Clemmens et al. 2001) presents updated hydraulic theory, design procedures, application and construction examples, and detailed instructions for using the WinFlume software.

Long-throated flumes are the measurement device of choice for most open-channel applications, having significant advantages over Parshall flumes and other traditional devices. These older devices were laboratory-calibrated, because the flow through their control sections is curvilinear. In contrast, streamlines are essentially parallel in the control sections of long-throated flumes, making them ratable using straightforward hydraulic theory. Significant advantages of long-throated flumes include:

- Rating table uncertainty of ±2% or better in the computed discharge.
- Choice of throat shapes allows a wide range of discharges to be measured with good precision.
- Minimal head loss needed to maintain critical flow conditions in the throat of the flume.
- Ability to make field modifications and perform computer calibrations using as-built dimensions.
- Economical construction and adaptability to varying site conditions.

Adaptability of these structures is a primary advantage, with the range of applications demonstrated by design and construction examples provided in Clemmens et al. (2001). Structures meeting the criteria for analysis as long-throated flumes can be small or large, permanent or temporary, fixed or portable, and may serve as either simple flow measurement devices or as combined measurement and control or flow division structures.

PERMANENT STRUCTURES FOR FLOW MEASUREMENT

Permanent flumes and weirs form the backbone of many water measurement networks for open-channel irrigation and drainage systems. In concrete-lined canal sections the most common structures are of the broad-crested weir type, consisting essentially of a sill that spans the full width of the existing lined channel. The resulting shape of the throat section of the flume (the control section) may be trapezoidal, rectangular, or one of several complex shapes such as a sill in a circle or sill in a parabola. The side walls of the existing lined channel form the side walls of the throat section, so construction is generally straightforward.

Figures 2-4 illustrate several possible construction methods. In Figure 2 an earthfill core is constructed, and a concrete shell is then placed over the earthfill to form the converging ramp and the control section. Drain pipes through the structure eliminate undesirable standing water upstream from the flume during the off-season. When flow is initiated, the underlying earthfill is washed out, leaving a concrete shell structure.

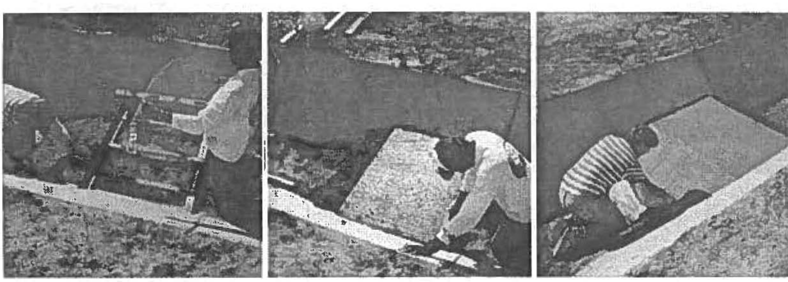

Figure 2. Construction of a Broad-Crested Weir in a Concrete-Lined Trapezoidal Channel.

Figure 3 shows the installation of pre-cast concrete sections of a broad-crested weir for a lined trapezoidal channel. This construction technique is useful when many weirs of exactly the same design are needed for installation at several locations in a canal system. Similar weirs can also be constructed with pre-fabricated metal ramp and sill sections.

Figure 3. Placing Pre-Cast Weir Sections in a Lined Trapezoidal Canal.

Figure 4 shows the construction of a large permanent flume with a trapezoidal throat section. In this case, a length of the existing earth canal was fully lined to create the sidewalls of the control section. The most important aspect of construction is maintaining a level sill in the throat section (level in the flow direction is most important).

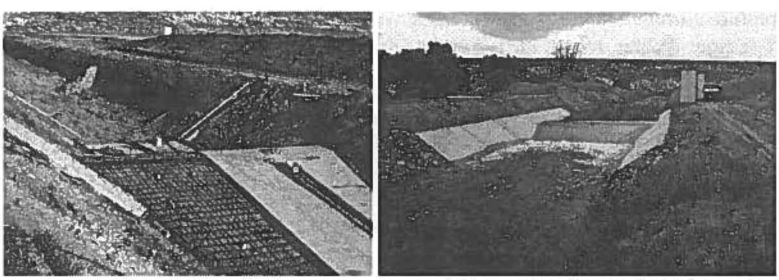

Figure 4. Construction of a Large Broad-Crested Weir in an Earthen Channel.

A common flume design used for earth-lined canals is illustrated in Figure 5. The throat section is rectangular, which often simplifies construction, since sloped side walls do not have to be constructed in the throat section. The vertical side walls are constructed from concrete block and mortar. Erosion protection is provided downstream from the flume in the channel bottom and on the side walls.

In natural channels where a very large range of flows may need to be measured, flumes with a triangular-shaped control section are often used. These flumes offer good measurement accuracy at low flows, while passing flood flows without an unacceptably large flow depth upstream from the structure. Figure 6 shows a triangular-shaped flume constructed in Florida.

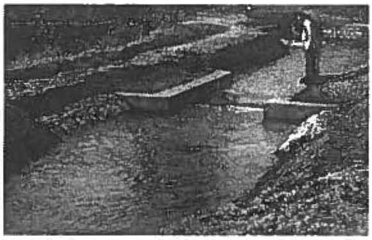

Figure 5. A Rectangular-Throated Flume in India.

Figure 6. Triangular-Throated Flume in Florida.

PORTABLE AND TEMPORARY FLUMES

Long-throated flumes and broad-crested weirs are very useful for portable and temporary applications. Such devices can be very economical to build, while still providing highly accurate flow measurement capability. The RBC family of 5 small trapezoidal-throated flumes is suitable for flow survey work. These flumes can be constructed from sheet metal using drawings provided in Clemmens et al. (2001), or commercially pre-fabricated flumes are also available, such as the fiberglass models shown in the right hand photograph in Figure 7.

Figure 7. Portable RBC Flumes Measure Flow in Drains and Furrows.

Portable flumes with rectangular throat sections are shown in Figure 8. These can be constructed from wood, metal, or fiberglass. Temporary flumes for lined trapezoidal canals can be constructed from plywood (Figure 9).

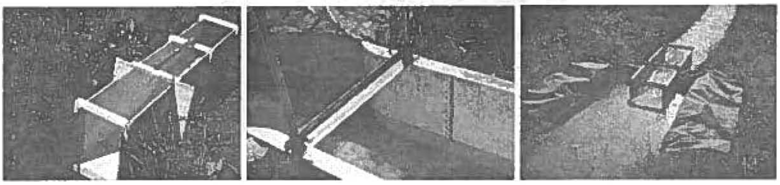

Figure 8. Rectangular-Throated Portable Flumes.

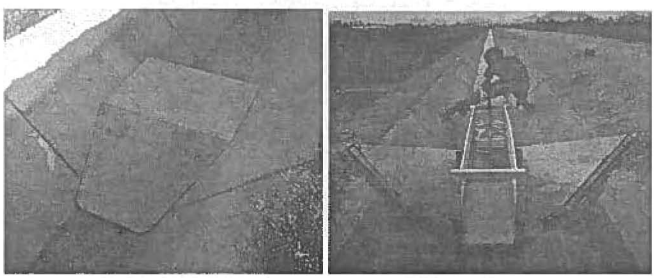

Figure 9. Temporary Flumes for Lined Trapezoidal Channels.

Portable flumes can also be constructed by installing a sill into a circular or semicircular pipe section. If the pipe section is small enough, these flumes can be easily transported to different measurement sites. Leveling bubbles shown on the flume in Figure 10 help the user to achieve a good installation.

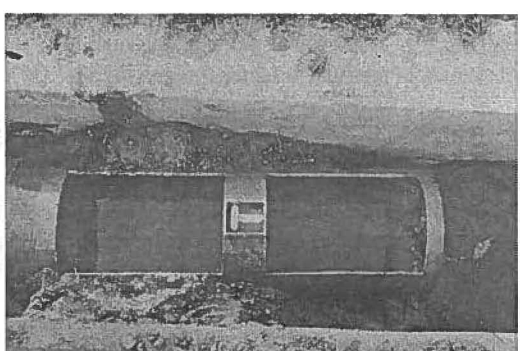

Figure 10. Portable Flume Constructed in a Section of Circular Pipe.

FLUMES AND WEIRS FOR FLOW DIVISION AND CONTROL

In addition to flow measurement, flumes and weirs can facilitate regulation of deliveries and accurate division of flows. For regulation of deliveries, weirs with a vertically movable crest are used. Such weirs can be constructed in sizes up to about 4 m wide. These weirs are installed at offtakes where the supply water level is steady, and the weir elevation is varied to change the delivered flow rate. Figure 11 shows a pair of vertically movable weirs and the flow registration system used to directly indicate the flow rate.

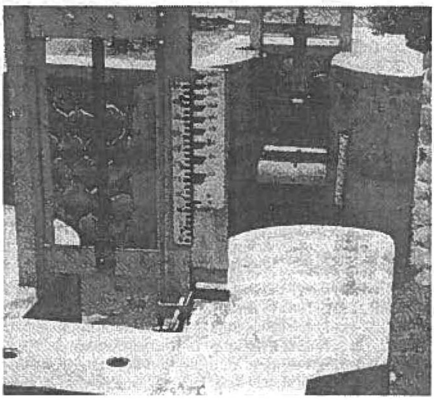

Figure 11. Movable-Crest Weirs and Flow Registration Scales.

Broad-crested weirs with a rectangular throat section can be used to accurately divide flows among two downstream users or districts. A vertical division board is installed just downstream from the control section. The division board may be fixed or movable. Such structures are useful on projects where different users are entitled to a percentage of the available water supply, rather than a fixed quantity of water. Figure 12 shows an adjustable flow division structure in Argentina.

PRECOMPUTED FLUME DESIGNS AND RATINGS

Two basic approaches can be taken to the problem of flume design. Custom designs may be developed with the WinFlume software described later, or precomputed flume designs can be selected from tables provided in Clemmens et al. (2001). Precomputed designs are available for trapezoidal-throated structures to be installed in lined canals, rectangular-throated structures for installation in lined or earthen channels, triangular-throated structures for natural channels and drains experiencing a wide range of flows, and for sills installed in circular conduits. Tables provide discharge and head ranges, minimum head loss requirements, structure dimensions, and rating equation parameters. Precomputed designs are available for typical small to medium-size channels; for large structures, analysis with WinFlume is recommended to achieve an optimum design. Each precomputed design has been developed to satisfy criteria related to approach channel Froude number and flow measurement accuracy. The user is responsible for verifying that a selected design will satisfy upstream freeboard requirements and operate in a free flow condition (i.e., that enough head loss is available at the site to avoid submergence of the flume by the existing tailwater conditions). Clemmens et al. (2001) provides numerous design examples to illustrate the use of the tables.

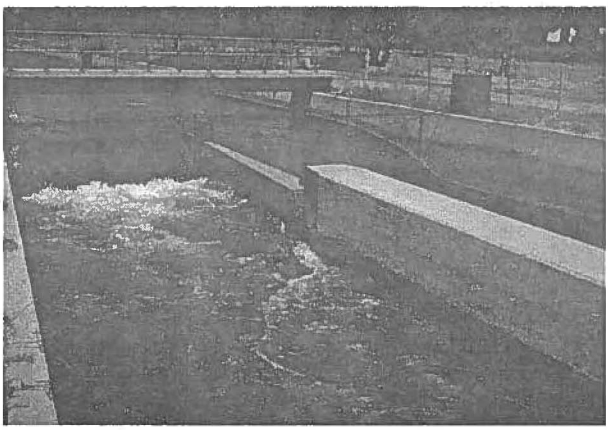

Figure 12. Flow Division Structure with a Movable Divisor.

WINFLUME SOFTWARE

Custom flume ratings and designs to satisfy nearly any application can be developed with the WinFlume software (Figure 13), available for free download on the Internet at www.usbr.gov/wrrl/winflume/. The software operates on all Windows-based computers. An online help system is integrated into the program, and a printable user's manual is distributed electronically with the software. A helpful flume wizard is available to assist new users with basic data entry.

Flume ratings for existing structures can be developed from as-built dimensions, provided that the throat section is horizontal in the flow direction and there is a smooth transition from the approach channel into the throat section. The software will generate head-discharge tables, rating curves, rating equations, and flow-graduated wall gages. Wall gages can be printed at full scale on Windows-compatible plotters and printers and provided to a fabricator for construction of durable field-quality staff gages (Figure 14). Additional optional parameters provided in WinFlume's rating tables include the required head loss and modular limit (i.e., maximum allowable submergence ratio).

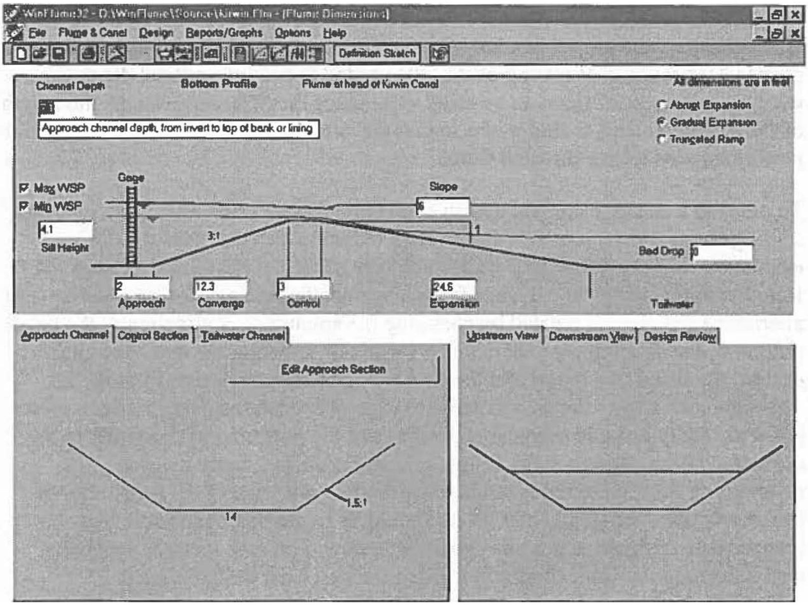

Figure 13. Entering Flume Dimensions into the WinFlume Software.

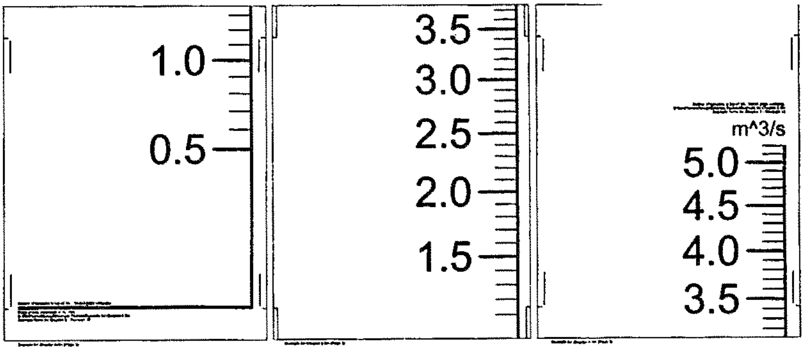

Figure 14. A 3-Section Flume Wall Gage Produced by WinFlume.

Design of new flumes can be accomplished either by trial or with the assistance of WinFlume's automated design evaluation routines. To develop a design by trial, the user enters the flume dimensions and design requirements (e.g., desired freeboard, discharge range, tailwater conditions, accuracy requirements), and then performs a design review. The design review report will indicate deficiencies in the design and suggest alternatives to improve the design. The initial design task is to determine the proper amount of contraction in the throat section (narrowing of the channel width or raising of the sill) needed to produce critical depth flow over the full range of flows to be measured. Once this is accomplished, the length of flume components is fine-tuned so that the structure meets the requirements for performance as a long-throated flume.

To develop a design using the design evaluation routines, the user again provides starting flume dimensions and design requirements, then executes the design evaluation routine. WinFlume builds a family of virtual flume designs derived from the initial design and evaluates each against the primary design criteria. The alternative designs are created by changing the amount of contraction in the throat section of the structure, for example by raising or lowering the sill. Acceptable designs are listed in a report and the user can then select a design from the list. This approach allows the user to see the range of acceptable designs (since there is not normally just one acceptable design) and the performance tradeoffs to be considered in selecting a final design. These tradeoffs usually involve maintaining acceptable upstream freeboard while minimizing the possibility of flume submergence if tailwater levels should be higher than expected. An intermediate design that provides acceptable freeboard and operates somewhat above the modular limit condition is often the best final design choice.

CONCLUSION

The release of the WinFlume software and publication of *Water Measurement with Flumes and Weirs* has greatly advanced the application of long-throated flumes and broad-crested weirs for open-channel flow measurement. The flexibility of these structures and the simplicity of developing, calibrating, and constructing effective, economical designs make them the best structure for most open-channel flow measurement applications.

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POLICIES TO REDUCE ENVIRONMENTAL DEGRADATION AND ENHANCE IRRIGATION AND DRAINAGE IN THE INDUS BASIN OF PAKISTAN

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ABSTRACT

Growth rates in the yields of rice and wheat in the Indus Basin of Pakistan have been declining in recent years, for reasons that include the degradation of soil and water resources. Policies regarding irrigation water, technology, and energy prices have contributed to the resource degradation. This paper describes how agricultural policies influence farm-level irrigation and drainage decisions, while suggesting how those policies might be modified to promote improvements in resource quality and to restore positive rates of growth in crop yields. The paper includes a review of recent literature and data describing irrigation and drainage in the Indus Basin, recent estimates of productivity in the region, and an economic perspective regarding opportunities for enhancing the sustainability of irrigation and drainage activities. Successful implementation of appropriate policies may enhance food security in the region, while improving rural incomes and supporting economic growth and development in Pakistan.

INTRODUCTION

Pakistan is largely an agricultural country in which 70% of the population is supported directly or indirectly by agriculture (Gill et al., 1999). While the proportion of gross national product contributed by agriculture has declined over time, its role in providing jobs for more than 50% of the country's work force remains substantial. In addition, agriculture accounts for 70% of export revenues, directly or indirectly (Faruqee, 1999). Cotton, rice, and textile manufactures, taken together, account for more than half of the total value of Pakistan's exports (Govt. of Pakistan, 2000; EIU, 2001).

Agriculture and other sectors in Pakistan must grow rapidly in future to support economic development, enhance food security, and create jobs for the nation's rapidly expanding population. With an estimated annual growth rate of 2.7%, the population of Pakistan is expected to increase from 140 million in 1996 to 208 million in 2025 and 357 million in 2050 (Siddiqui, 1998; Chaudhry, 2000). Such growth would give Pakistan the third largest population in the world in 2050,

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placing it ahead of the United States, Nigeria, and Indonesia.

Aggregate production of food crops and cotton in Pakistan has increased substantially since the 1950s. Sugarcane and cotton production have increased more than six-fold since 1950, while wheat production has increased by a factor of 4.5 and rice production has increased by a factor of 5.7. Pakistan's population has increased by about four-fold since 1951 (Govt. of Pakistan, 1999). Hence, per capita production of food grains and cotton has remained constant or increased when viewed from this long-term perspective. However, the rates of growth in total production began declining in 1980 for rice and in 1990 for cotton and wheat. Since then, the rates of growth in rice and wheat production have not kept pace with the steadily increasing demand for food in Pakistan.

In 1998-99, Pakistan's farmers produced more than 17 million tonnes (mt) of wheat, 1.5 mt of cotton, 55 mt of sugarcane, and 4.7 mt of rice. Wheat is the primary food crop in Pakistan, accounting for 50% of total calories and 85% of protein intake (Ahmad and Muhammad, 1998). More than 70% of Pakistan's wheat is produced in Punjab Province, largely with irrigation (Byerlee, 1993). The nation came close to achieving self-sufficiency in wheat production during the early 1980s (Ahmed and Siddiqui, 1995). However, the population has increased faster than wheat production since that time, and the country has had to import from 0.6 to 4.1 mt of wheat annually to match the increasing demand (Kurosaki, 1996; Ahmad and Muhammad, 1998; Govt. of Pakistan, 2000).

Value added in the agricultural sector of Pakistan has grown by a factor of four since 1950 but annual growth rates in crop production have fluctuated substantially. Growth rates in crop production increased from less than 2% per year in the 1950s to as high as 4.74% and 8.18% in the 1960s (Table 1). Much of the gain in the 1960s was due to improvements in productivity per hectare, made possible by the adoption of higher yielding varieties of wheat and rice and rapid increases in the use of fertilizer and irrigation water (Ahmed, 1987; Chaudhry et al., 1996). Moderate gains in productivity per hectare of about 2.5% per year were sustained from the late 1970s through the early 1990s, while the average increase in cropland area fluctuated between 0.15% and 1.58% per year (Table 1). The rate of growth in aggregate inputs has exceeded the rate of growth in crop production during two of the five most recent five-year periods shown in Table 1. The rate of growth in total factor productivity was negative in those periods and less than 1.5% in other periods, since 1970.

Further increases in cropland area in Pakistan will be limited by the amount of water available for irrigation. Hence, future gains in agricultural production will require improvements in the average productivity of land and water resources within the

Table 1. Annual Rates of Growth in Agricultural Inputs, Output, and Total

Factor Productivity in Pakistan, 1950 through 1995

Time Period	Crop Production	Crop Land	Productivity Per Hectare	Aggregate Inputs	Total Factor Productivity
			(percent)		
1950-55	0.33	1.24	-0.91	1.64	-1.31
1955-60	1.91	2.05	-0.14	2.40	-0.49
1960-65	4.74	2.03	2.73	2.30	2.56
1965-70	8.18	0.63	7.55	2.26	5.82
1970-75	0.48	0.70	-0.22	2.59	-2.11
1975-80	4.15	1.58	2.57	3.16	0.99
1980-85	2.63	0.15	2.48	3.32	-0.69
1985-90	3.70	1.50	2.10	2.83	0.87
1990-95	3.17	0.62	2.55	1.70	1.47
1949-95	3.02	1.31	1.71	2.54	0.48

Sources: Government of Pakistan (1990, 1996) and Kemal and Ahmad (1992), as reported in Chaudhry et al., 1996.

Indus Basin and in rainfed areas outside the Basin. The improvements will require substantial efforts to restore the rates of growth in crop yields achieved in the 1960s and 1970s, particularly with respect to rice and wheat. The declining growth rates observed since 1980 are due in part to the degradation of land and water resources caused by inappropriate management of resources on farms and throughout irrigated areas (Mustafa and Pingali, 1995; Pingali and Shah, 1999; Murgai et al., 2001). Repairing the damage done by waterlogging, salinization, and poor management of soil fertility will be costly and time consuming. Changes in public policies that have encouraged inefficient use of land and water resources will be helpful in restoring lost productivity.

IRRIGATION IN THE INDUS BASIN

Pakistan's canal irrigation system is the largest contiguous irrigation system in the world, with 60,000 km of canals and more than 80,000 watercourses, channels, and ditches (Qureshi et al., 1994). An estimated 130 billion m^3 of water enter the canal system each year, but only 60% of that water reaches farm gates due to inefficiencies in the delivery system (Faruqee, 1996). The canal water supply is augmented by extensive use of public and private tubewells that extract water from shallow aquifers. An estimated 29 billion m^3 of groundwater with a salinity level less than 1500 mg/l are available in the Indus Basin each year (Kijne and Kuper, 1995), but current extractions likely exceed that amount. A national survey reports that in 1991, 46 billion m^3 of groundwater were used for irrigation in the Indus Basin (Kijne, 2001a).

Water is the input that limits the intensification of agriculture in Pakistan, where more than 16 million ha of land are irrigated (Ahmad and Kutcher, 1992). Nearly 90% of the irrigated area is within the canal command area of the Indus Basin irrigation system. Irrigated area increased substantially during the 1970s and 1980s following completion of the Mangla, Chashma, and Tarbela dams (Afzal, 1996) and with expansion of the area irrigated with private tubewells. Crop production has not increased at the same rate as irrigated area, due largely to inefficiencies and inequities in the water delivery system that have contributed to structural and environmental degradation (Mellor, 1996). Large seepage losses from canals and excessive irrigation with brackish shallow groundwater contribute to waterlogging and salinization, while the misallocation of water among regions and farmers reduces economic returns (Ahmad and Sampath, 1994; Qureshi et al, 1994; Mellor, 1996; Qureshi and Barrett-Lennard, 1998). Rising water tables and groundwater salinity are considered by some to be among the most important issues affecting agricultural productivity and sustainability in the Indus Basin (Shah et al., 2001).

The limited volume of water available in canals and the inherent rigidity of Pakistan's rotational irrigation system have motivated farmers to install private tubewells. The pace of installation has been enhanced by credit subsidies and flat-rate pricing of electricity beginning in the 1960s. By the early 1980s, 182,000 tubewells had been installed in Punjab, while 19,300 tubewells had been installed in Sindh, 7,850 in Balochistan, and 5,400 in the NWFP (Chaudhry, 1990). By the late 1980s more than 30% of farm-gate available water in Pakistan was supplied by private tubewells (Mustafa and Pingali, 1995). By the middle 1990s more than 300,000 private tubewells were supplying about 40% of total irrigation water in Pakistan (Ahmad and Faruqee, 1999). Tubewells enhance agricultural production and land quality in regions where shallow groundwater is not saline (Mustafa and Pingali, 1995), but they contribute to salinization in regions with brackish shallow groundwater (Faruqee, 1996).

GOVERNMENT POLICIES REGARDING AGRICULTURE

The Government of Pakistan has subsidized the purchase of key inputs such as irrigation water, fertilizer, electricity, pesticides, and seed for many years. The subsidy on pesticides was removed in the early 1980s and subsidies on fertilizer have been reduced substantially in recent years. Subsidies remain in place for diesel and electric tubewells, purchased seeds, canal water, and credit (Chaudhry and Sahibzada, 1995; Faruqee and Carey, 1995). The reduction and removal of input price subsidies in Pakistan during the 1980s caused substantial increases in farm-level input prices and reductions in farm-level net returns (Ahmad and

Chaudhry, 1987; Looney, 1999).

The Government also is involved in the importation, production, and distribution of selected farm inputs. For example, the Government imports and distributes phosphorus fertilizer and it produces a large portion of the seed required by farmers each year. Distortions caused by government intervention limit the supply of those inputs available to farmers (Faruqee and Carey, 1995). Farm-level difficulties in acquiring the seeds of modern crop varieties and obtaining sufficient amounts of phosphorus fertilizer for timely application likely have contributed to the declining rates of growth in crop yields.

The Government of Pakistan supports the farm-level prices of all major crops through guaranteed minimum prices or other price support programs (Faruqee and Carey, 1995; Khan, 1997). The government procures about 30% of the wheat produced each year at a pre-determined support price and it releases wheat flour to consumers through government-owned utility stores and through private markets (Kurosaki, 1996). The price of Basmati rice also is supported, although much of the crop is exported. Private sector participation in the exporting of rice has increased in the 1990s.

The government discourages cotton production by imposing an export tax that prevents farmers from receiving the world price for their output. Removal of the export tax would stimulate cotton production, raise rural incomes, and generate a more diverse set of rural, non-agricultural employment opportunities (Mellor, 1993). Cotton is a relatively profitable crop in Pakistan and requires less irrigation water than sugarcane. Reducing the price support level for sugar, while at the same time reducing cotton export taxes would generate greater net benefits and may reduce some of the pressure on land and water resources in the Indus Basin (Mellor, 1993).

CURRENT STATUS OF AGRICULTURAL PRODUCTIVITY

Average crop yields in Pakistan are much lower than the potential yields observed on experiment stations and the average yields observed in some other countries with similar production conditions. Estimated yield gaps for cereals in Pakistan range from 72% for wheat to 88% for maize and 95% for tomatoes (Table 2). Yield gaps exist in all countries because crop management and resource endowments on experiment stations are not the same as those on farms. However, the average yields of some crops in Pakistan are considerably smaller

Table 2. Estimated Yield Gaps for Selected Crops in Pakistan, 1990-91

Crop	Potential Yield	Average Yield	Estimated Yield Gap

	(kg/ha)	(kg/ha)	(%)
Wheat	6,415	1,773	72
Rice (Paddy)	9,849	1,600	83
Maize	6,944	840	88
Sorghum	3,500	600	83
Sugarcane	18,300	4,070	78
Chickpea	3,000	440	85
Cotton	1,400	544	61
Potatoes	3,128	1,040	67
Tomatoes	21,000	1,024	95

Source: Agriculture Statistics of Pakistan 1990-1991, published by the Government of Pakistan, and reported in Mellor, 1996.

than average yields reported for other countries. For example, the average production of wheat in Pakistan during 1996 through 1998 ranged from 2,018 to 2,238 kg/ha, while the world average production was greater than 2,500 kg/ha (Table 3). Average wheat production in Pakistan compared favorably with average wheat production in Turkey during those years, but was lower than average productivity in India and Canada.

The average production of maize in Pakistan was slightly lower than average production in India during 1996 through 1998, but productivity in both countries was substantially less than the world average productivity in those years (Table 3). Average rice productivity in Pakistan exceeded that in Thailand during 1996 through 1998 and it compared favorably with average productivity in India and Bangladesh. However, the average productivity in all of those countries was less than the world average productivity. Seed cotton productivity in Pakistan was higher than average productivity in India and it compared favorably with world average production during 1996 through 1998, but it was lower than average productivity in Egypt and Turkey during those years.

The relatively low average productivity observed in Pakistan is caused in part by the nation's limited water supply and the design of its irrigation system. In particular, the Indus Basin system was designed to provide 'protective' rather than 'full' irrigation potential (Jurriens and Mollinga, 1996). The goal was to prevent famine by maximizing the area served by the irrigation system, so that a large

Table 3. Estimated Average Productivity of Land for Major Crops Grown in Pakistan, 1996 through 1998

Crop and Country	1996	1997	1998
	(kilograms per hectare)		

Wheat				
	Pakistan	2,018	2,053	2,238
	World	2,523	2,676	2,624
	India	2,472	2,671	2,578
	Canada	2,430	2,121	2,266
	Turkey	1,980	1,997	2,234
Maize				
	Pakistan	1,445	1,440	1,473
	World	4,176	4,096	4,395
	India	1,596	1,712	1,613
	Canada	6,919	6,870	7,969
	Turkey	3,636	3,817	4,182
Rice (Paddy)				
	Pakistan	2,868	2,805	2,892
	World	3,786	3,823	3,747
	India	2,819	2,906	2,890
	Bangladesh	2,813	2,769	2,757
	Thailand	2,410	2,350	2,324
Seed Cotton				
	Pakistan	1,518	1,584	1,536
	World	1,582	1,688	1,561
	India	794	894	851
	Egypt	2,481	2,632	2,100
	Turkey	2,800	2,917	2,864

Source: Agricultural Statistics of Pakistan 1998-1999, published by the Government of Pakistan.

number of households would receive at least a partial irrigation supply (Johnson et al., 1978; Johnson, 1982; Chohan, 1989; Mustafa, 2001). It was known at the time of system design and development that the total water supply would not be sufficient to provide full irrigation potential throughout the irrigated area. The system was designed to support subsistence agriculture at cropping intensities ranging from 50% to 75% (Ul-Haq and Shahid, 1997). However, cropping intensities have risen beyond those levels over time, as farmers have attempted to maximize economic returns to their limited land and water resources.

Profit-maximizing farmers who receive a partial irrigation supply that is not sufficient to generate maximum yield on all of their land will apply irrigation water to maximize the net return per unit of water received (Upton, 1994). That will occur when the incremental productivity of water is the same on all land parcels. Hence, farmers will attempt to spread a limited water supply across a larger land area than project planners may have considered when designing the irrigation system. Farmers also will augment surface water with groundwater if it is available at reasonable cost, enabling them to diversify cropping patterns and increase cropping intensities. The extensive use of tubewells has enabled farmers in the Punjab to increase the cropping intensities of cotton-wheat and rice-wheat rotations from about 100% in 1960 to more than 150% in 1990.

Irrigation with tubewells will enhance productivity in regions with high quality groundwater. In regions with brackish or saline groundwater, however, productivity may be degraded over time if the supply of higher quality surface water is not sufficient to leach salts from the root zone. The combination of higher cropping intensities and increased use of saline groundwater may substantially increase the rate at which salts accumulate in arid zone soils.

The productivity of land and water in lower portions of the Indus River basin is limited by problems of water scarcity, waterlogging and salinity, inefficient water delivery and use, inequitable distribution, and inadequate maintenance of the irrigation and drainage system (Wescoat, 1991; Afzal, 1996; Ul-Haq and Shahid, 1997; Wambia, 2000; Kijne, 2001b). Salinization, alone, may be reducing productivity by 25% to 70% on moderately affected soils and by nearly 100% on severely affected soils (Ahmad et al., 1998). Qureshi and Barrett-Lennard (1998) suggest that 6.3 million ha are affected by waterlogging and salinization in Pakistan and that the livelihoods of 16 million people are affected directly by those conditions.

POLICY RECOMMENDATIONS

Improvements in agricultural productivity are needed in Pakistan to increase the production of food and fiber for domestic consumption, to enhance opportunities for international trade, and to provide employment for a rapidly expanding labor force. Labor-intensive improvements in agricultural production will generate new jobs directly, while greater output of agricultural products can provide the raw materials for expanding employment opportunities in processing and marketing. For example, improvements in cotton production may generate greater employment in Pakistan's textile industry.

Much has been learned about improving agricultural productivity since Green Revolution technologies were introduced in south Asia in the 1960s. Much has been learned also about the impacts of public policies on farm-level decisions regarding inputs and outputs, and subsequent impacts on the natural resources that support agricultural production. The Green Revolution was driven by rapid increases in the use of water, energy, and chemical fertilizers. The public policies that were implemented to support farm-level adoption of new techniques were very successful in promoting rapid increases in crop yields, irrigated area, and aggregate output. However, over time, those same policies have encouraged the degradation of soil and water resources that has contributed to the declining rates of increase in crop yields observed on the Indo-Gangetic Plains.

Both economic theory and observations regarding farm-level input use suggest that farmers will not account sufficiently for the long-term impacts of their decisions on soil and water resources if the prices of key inputs do not reflect those long-term costs. Farmers receiving fertilizer and electricity at heavily subsidized prices will tend to use those inputs excessively. Flat-rate pricing for electricity has encouraged farmers to pump greater volumes of groundwater with tubewells than would have been pumped if electricity had been priced in accordance with the amount that farmers use. Similarly, fertilizer subsidies have encouraged farmers to increase the intensity of agricultural production, but the relative amounts of nutrients applied each year may not be consistent with maintaining soil fertility over time.

Government policies that modify the farm-level returns from crop production also have influenced input choices and the evolution of cropping patterns. Implicit taxation of some crops, subsidies for producing others, and mandatory procurement schemes have had significant impacts on farm-level decisions in Pakistan and in many other countries. Government supply of key inputs, such as fertilizer, electricity, and credit can influence the degree to which farmers can obtain and apply those inputs in a timely fashion. Restrictions or delays regarding the availability of seeds and fertilizer can reduce crop yields substantially. Persistent difficulties in obtaining key inputs may cause farmers to modify cropping patterns to include less profitable or less appropriate crops.

There is a pressing need in Pakistan to address the degradation of soil and water resources that has been developing over a sustained period of time. The challenge is to develop and implement policy reforms that encourage farmers to use limited resources efficiently, while not causing substantial disruption in current levels of economic activity and employment. A comprehensive program that allows input and output prices to reflect true market values and opportunity costs should include aggressive efforts to enhance the financial and human capital components of farm operations.

Research has shown that labor and fertilizer are complementary inputs on farms in the Punjab, and that fertilizer use is greater among farmers with higher levels of education. Fertilizer use also is correlated with tubewell installation, as a reliable source of irrigation water enhances the productivity of land and fertilizer. Farmers in Sindh Province with better access to canal water invest greater effort in land preparation, apply more fertilizer, and achieve higher yields than do farmers with a less reliable water supply. Research also has shown that farmers respond to input and output prices when making decisions regarding fertilizer use and that farm-level credit constraints limit the use of fertilizer in some areas.

Public programs that enhance farm-level educational opportunities and provide credit at affordable rates may be helpful in restoring soil productivity in the Indus River Basin and in motivating farmers to choose input levels that are consistent with sustainability goals. Those programs would complement efforts to adjust input prices to levels that reflect long-term costs. Similar educational and credit programs can be implemented to encourage wiser use of irrigation water from both surface and groundwater sources, so that the rate of increase in waterlogged and salinized areas might be reduced.

In summary, policies that establish appropriate prices for agricultural inputs and outputs, and enhance the human and financial capital available to farmers will enable them to respond optimally to new production and marketing opportunities that will arise in future. Those opportunities, and the optimal farm-level responses to them, will enhance the likelihood that food security will be achieved and maintained in Pakistan and throughout the developing world.

ACKNOWLEDGMENT

This paper is Rhode Island Agricultural Experiment Station Contribution Number 3927.

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