Contemporary Challenges for Irrigation and Drainage

Proceedings from the USCID 14th Technical Conference on Irrigation, Drainage and Flood Control

Phoenix, Arizona

June 3-6, 1998

Sponsored by

U.S. Committee on Irrigation and Drainage

Edited by

Joseph I. Burns
Consultant

Susan S. Anderson
U.S. Committee on Irrigation and Drainage

Published by

U.S. Committee on Irrigation and Drainage
1616 Seventeenth Street, Suite 483
Denver, CO 80202
Telephone: 303-628-5430
Fax: 303-628-5431
E-mail: stephens@uscid.org
Internet: www.uscid.org/~uscid

USCID

The U.S. Committee on Irrigation and Drainage (USCID) is a nonprofit professional society whose members share an interest in the planning, design, construction, operation and maintenance of irrigation, drainage and flood control works; agricultural economics; water law; and environmental and social issues affecting irrigated agriculture.

USCID publishes the *USCID Newsletter*, proceedings of USCID meetings, and special reports; organizes and sponsors periodic technical meetings and conferences; and distributes publications of the International Commission on Irrigation and Drainage. ICID* publications include the biannual *ICID Journal, Irrigation and Drainage in the World*, and the *Multilingual Technical Dictionary on Irrigation and Drainage*.

For additional information about USCID, its publications and membership, contact:

U.S. Committee on Irrigation and Drainage
1616 Seventeenth Street, Suite 483
Denver, CO 80202

Telephone: 303-628-5430
Fax 303-628-5430
E-mail: stephens@uscid.org.
Internet: www.uscid.org/~uscid.

Printed in the United States of America

Library of Congress Number 98-61731

ISBN 1-887903-06-2

Preface

The papers included in these proceedings were prepared for the **14th Technical Conference on Irrigation and Drainage**, held June 3-6, 1998, in Phoenix, Arizona. The theme of the Conference was *Contemporary Challenges for Irrigation and Drainage.* The Conference, sponsored by the U.S. Committee on Irrigation and Drainage, was a multidisciplinary forum which provided an opportunity for more than 100 irrigation and drainage specialists to share and discuss problems and solutions.

Nineteen papers presented at the Conference are included in the Proceedings. Also included are a lunch presentation by Maurice Roos, Chief Hydrologist, California Department of Water Resources and a dinner address by Rita Pearson, Director of the Arizona Department of Water Resources.

The papers presented during the first technical session featured the 1988 Water Conservation Agreement between the **Imperial Irrigation District** and the **Metropolitan Water District of Southern California.** The Agreement called for a **Water Conservation Program** to modernize and rehabilitate Imperial's water distribution system, with the goal of conserving water that could be used to help meet MWD's urban water needs. The papers in this session offered an introduction to the physical facilities and technical aspects of this decade-long effort to conserve irrigation water, and reviewed the Program components and the irrigation management techniques used. The final paper of the session provided an update on Program costs and results.

Professional papers presented during three additional Technical Sessions and the Poster Session addressed other current irrigation and drainage issues:

- Improving Irrigation Management and Modernizing Irrigation and Drainage Systems
- Drainage and Water Quality
- Institutional, Environmental and Economic Issues Affecting Irrigated Agriculture

The U.S. Committee on Irrigation and Drainage and the Conference Chairman express gratitude to the speakers, authors, session moderators and participants for their contributions to a successful Conference.

Joseph I. Burns
Conference Chairman
Sacramento, California

Table of Contents

IID/MWD WATER CONSERVATION PROGRAM – IMPROVED IRRIGATION WATER MANAGEMENT THROUGH SYSTEM AUTOMATION

John H. Korinetz[1] Carlos Z. Villalón[2]

ABSTRACT

Imperial Irrigation District (IID) services 450,000 acres in Imperial Valley, California, USA. The sole source of supply is the Colorado River. Imperial Dam diverts water from the Colorado River into the All American Canal (AAC) and subsequently into three main canals. Each main canal is approximately 40 miles in length and can divert 1,200 to 2,200 cfs. Lateral canals are then serviced by the main canals. There are 240 laterals that vary in length from 1 to 10 miles and from 40 to 160 cfs in capacity. Farm deliveries are made through laterals via 5,500 individual user gates. Each has a 20 cfs minimum capacity. On average each delivery services 80 acre parcels. Based on operational considerations, the geographic distribution of control sites, the high inertia of the system and the harsh desert environment a distributed control design was implemented. Commercially available industrial control components were integrated into a SCADA system in a non-traditional manner. Three subsystems were developed; field site automation, communication network, water control center. The level of automation implemented at field sites allows more flexible main canal operation. This allowed the various projects developed under the IID/MWD Water Conservation Agreement to be optimally managed and have water savings verified.

DESCRIPTION OF IMPERIAL IRRIGATION DISTRICT

General

The Imperial Irrigation District (IID) is a special district formed under California's Water Code. A five-

[1] Manager of Water Resources and Industrial Automation, UMA Engineering Inc., 17762 Cowan Street, Irvine, CA, 92714 jkorinetz@umagroup.com

[2] General Superintendent, Water Operations, Imperial Irrigation District, 333 Barioni Blvd., Imperial, CA 92251 czvillalon@iid.com

member board is elected using the general election process. The board develops policy that guides long term and day to day activities of the IID.

IID provides agricultural irrigation water and drainage services within an identified area of the Imperial Valley referred to as the Imperial Unit. Only areas within the Imperial Unit can be serviced with Colorado River water.

Electric power services are also provided by the IID. Power generation, transmission and distribution are supplied to a service area that includes all of Imperial County and parts of Riverside County.

Colorado River Watershed

IID is located in southeastern California within the Colorado River Watershed. The Colorado River originates in the Rocky Mountains being fed by melting snow from these mountains. It is the third longest river in America, winding 1,700 miles through deep canyons of seven southwest States toward the Gulf of California in Mexico. Its drainage basin covers approximately 245,000 square miles, one-twelfth the area of the contiguous United States. Although the River has many tributaries in the upper basin, there are few in the lower basin. It is the sole source of water for the IID.

Imperial Dam

Serving as a diversion structure, Imperial Dam facilities include the All American Canal Headworks for diversion and desilting works to eliminate the sediment problem from Colorado River water. It is located on the California-Arizona border.

Various facilities are located at Imperial Dam:

- All American Canal Heading and desilting works
- California Waste Way
- Gila Gravity Main Canal Headworks

All American Canal

At the California end of the Imperial Dam is located the intake for the All-American Canal (AAC). Four 75-ft. (22.9 m) roller-dam-type gates supply separate channels. Each of the first three channels leads to a pair of desilting basins, and the fourth leads through a bypass directly to the AAC. The water from each of the first three channels enters a gradually contracting influent channel feeding a pair of desilting basins. Clarified water is distributed uniformly by openings in the side walls of the basins into the AAC. Along the AAC flows are diverted through a series of drops, where hydropower has been developed. It then serves the main canals of the IID system along the south side of the service area.

The AAC is the link between the Colorado River and the Imperial Valley. It is 80 miles (49.7 km) in length and varies in width from 230-ft. (70.1 m) at the headworks to a width of 100-ft. (30.48 m) at its point of termination. The water surface elevation of the lake above Imperial Dam, the point of beginning, is 180.00 feet above mean sea level (msl). At the end of the AAC, the water surface elevation is 5.8 feet (1.8 m) below msl. Flow capacity is 15,500 cfs at the heading.

Structures along the All-American Canal include:

- AAC Headworks
- Desilting Basins
- Station 48+50 Check
- Station 60+00 Flow Metering Station
- Station 1035+00 Level Metering Station
- Pilot Knob Check and Spillway
- Drop No.1 Check
- Drops No. 2, 3, 4 and 5
- East Highline Check and T.O.
- Allison Check
- Alamo River Check and Spillway
- Central Main Check T.O.
- New River Check and Spillway
- Wistaria Check
- Woodvine Check
- West Side Main Heading

Irrigation System

IID operates an open-channel-gravity system exclusively. There are three main canals, three supply canals and over 450 laterals and sub-lateral canals.

Main Canals: East Highline Canal is 45 (28 km) miles in length, 120 ft. (36.58 m) wide at the heading with a gradual decreasing width to 15 ft. (4.57 m) near the end. It has a capacity of 2600-cfs (73.63 m^3/s) at the head and can carry approximately 120 cfs (3.40 m^3/s) near the end. The diversions from this canal consist of 72 lateral canals, two supply canal systems and numerous direct farm deliveries. The water surface elevation of the pond upstream of the headworks gates is 42.50 ft. (12.95 m) above sea level and at the end the elevation is 57 ft. below see level.

The Central Main Canal (CM) is 26 miles (41.8 km) in length, 85 ft. (25.5 m) wide at the heading and 45 ft. (13.7 km) wide at the end. It has a capacity of 1,300 cfs (36.8 m^3/s) at the head and can carry approximately 500 cfs (14.2 m^3/s) at the end. The diversions from this canal consist of 14 lateral canals and numerous direct farm deliveries. The water surface elevation of the pond above the head gates is 13.50-ft. (4.1 m) above sea level and at the end the elevation is 97.7-ft. (29.8 m) below sea level.

Westside Main Canal is 45.5 miles (72 km) long to the Trifolium Extension Canal Heading. This canal is 90-ft. (27 m) wide at the heading with a gradual decreasing width to 25 ft. at the end. It has a capacity of 1,300 cfs (36.8 m^3/s) at the head and approximately 200 cfs (5.66 m^3/s). The water surface elevation of the pond upstream of the head gates is 994.20-ft. (287.8 m); the pond elevation at the Trifolium Extension Canal is 836.4 ft. (254.9 m).

The three supply canals are the:

- Rositas Supply Canal
- Briar Supply Canal
- Vail Supply Canal

These are sub-main canals supplying a limited number of laterals for hydraulic purposes.

Reservoirs: At the present time there are six main-canal reservoirs in operation:

- Sperber
- Fudge
- Sheldon
- Singh
- Carter
- Galleano

The primary use for main canal reservoirs is for flow management. Long-term storage capacity is not available because of the flow capacity of the adjacent main canal system. Typically reservoirs have a maximum capacity of 300 acre-feet.

There are also three interceptor reservoirs serving intercepted laterals systems:

- Bevins on the Plum-Oasis Interceptor system
- Young on the Mulberry-D Interceptor system
- Russel on the Mulberry-D Interceptor system
- Wiley on the Trifolium Interceptor system

Drainage System

IID operates an extensive agricultural drainage system. It is made up of open channel lateral drains, main drains and a collector basin.

Drains: There are more than 1,450 miles(2,300 km) of drains in the IID that drain into the New River, Alamo River and Salton Sea. Drains collect tailwater and leach water from farm fields. Drains are generally parallel to irrigation canals and laterals. Where the drains cannot be constructed deep enough to receive farm discharge sumps and pumps are operated and maintained by the IID.

Rivers: Two rivers are used as drainage collection channels. New River and Alamo River were formed originally when the Colorado River breached its banks near Yuma, Arizona, and flowed naturally into the Salton Sea. The river channels are now used as drains for agricultural and municipal discharges.

Salton Sea: Salton Sea lies in the depression known historically as Lake Cauhilla. It was filled early this century when the Colorado River breached its banks twice. Subsequently, its main source of water has been agricultural and municipal runoff from Imperial Valley, Coachella Valley and the Mexicali Valley in Mexico.

SYSTEM OPERATION

Hydraulics of the Irrigation System

IID operates a network of open channel gravity canals and minimal pumping in the system. No ground water is used due to its saline content.

Upstream Supply Based: Water is scheduled in advance from the Colorado River. No changes can be made without a three day notice. All deliveries to IID are coordinated by the USBR and strictly adhered to.

Flexible Deliveries: Within the IID the water users can order water for either 24-hour or 12-hour periods. With a three-hour notice these deliveries can be increased, decreased or cancelled.

Upstream Level Control: The canal ponds are maintained as close to steady state in upstream level control using check structures. Lateral headings located upstream of checks can then be set manually at the required flow.

Downstream Flow Control: All lateral headings are in downstream flow control. Aggregate delivery orders within each lateral plus operational flow requirements are maintained through the required period.

Annual Water Order: The annual volume of water required for operation is provided to USBR in October for delivery the following year. An estimate is prepared using all information which is available at the time; crop patterns, federal crop programs, etc. Data is usually very scarce, as crop patterns have not been formulated for the year. The best source for crop information is the County Agricultural Commissioner's Office. Various activities can affect water use; for example government subsidy programs which can cause a significant change in water use for the year.

Weekly Master Schedule of Orders: In addition to the Annual Water Order, weekly requirements are supplied to USBR. Every Wednesday an order for the following week, defined as Monday through Sunday, is supplied by Water Control to the River Division. USBR accumulates the order from all water users of the lower Colorado River and prepares a Master Schedule of Flows. The amount of water scheduled on the Master Schedule of flows is the quantity of water the IID is entitled to unless it is revised by the Watermaster at least 72 hours in advance. It is a common occurrence for the Watermaster to ask for and receive extra water above the Master Schedule of flows allotment. Normally excess flows are not scheduled as it could be counted against the IID's allotment in years when the water supply is low.

Daily Operation: Water orders originate with the water users and are accumulated by the three operating divisions. The divisions stop accepting water delivery orders at 12:00 noon daily. A summary of these orders is called in to the Water Control Section by each division, stating the amount lined up to run and amount to be carried over. Carry-overs are caused by water orders exceeding the capacity of the system or the available water. Water Control personnel then must allot available amounts to each division making sure that the percentage of carry-overs is balanced throughout the divisions. By 1:00 p.m. River Division is notified by Water Control to place a firm order for the following day and to make any change in the Master Schedule for the fourth day following. As soon as this order is confirmed by USBR, the Water Control Office allots all available water back to the three divisions in amounts keeping carry-overs balanced.

SUPERVISORY CONTROL AND DATA ACQUISITION (SCADA)

The objective of the System automation Program is to improve water management utilizing modern control technology. Through the use of automation the maximum benefits of the water conservation program were achieved. The SCADA system discussion that follows includes Field Sites, Communications network and a new Water Control Center, maintenance and benefits.

Field Sites

The water conservation program impacted on all aspects of the district from the All American Canal, to the Lateral Canals and on-farm projects. The SCADA system integrates all of these different needs into one system. The IID system includes three types of field sites: remote-monitoring sites, small canal sites and major sites.

Remote Monitoring Sites: These installations provide level/flow information via radio telemetry. The monitoring sites are solar powered and consist of a level sensor, a remote terminal unit (RTU) and a radio. The units were designed to be easily relocated, as warranted by the verification program. The number on units in service varies depending on the needs of the verification program and may be up to forty units.

The RTU is programmed to provide analog signal averaging; daily minimum and maximum with time stamp and storage of readings at a selected time interval. In addition to multiple level sensors the system battery voltage is also monitored. The controller retains several days of readings and in the event of a loss of communications the data can be retrieved locally from the RTU.

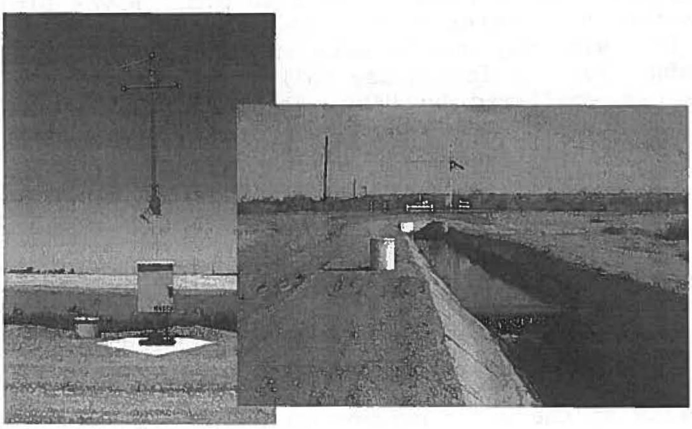


Fig. 1 Remote Monitoring Site

Small Canal Sites: These sites consist of solar powered single gate structures and a level sensor, a programmable logic control (PLC) and a motorized gate. These sites are designed to provide stand-alone automatic control and provide either a constant upstream water level or a downstream flow.

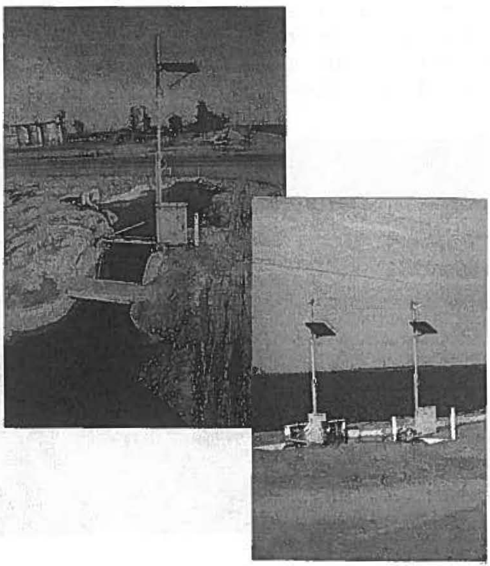


Fig. 2 Small Canal Sites providing automatic level control and flow control.

In addition to providing constant upstream level control, the Lateral Interceptor "interface gates" monitor the water level in the Lateral Interceptor canal and automatically switch to flow control when the Lateral Interceptor canal reach it's maximum capacity. In this event, the adjacent spillway gate which is programmed to maintain a higher level set point, takes over upstream level control.

There are a total of one hundred and ten of these sites. Some of sites include radio telemetry for data collection as part of the verification program.

Major Sites: These sites are also designed to operate as a stand-alone automatic sites, but to further enhance the control of the main canal system, radio telemetry provides real time monitoring of the system and provides the ability to remotely change set points. The installation consists of a level sensor, often one upstream and one downstream, a PLC and several motorized gates or pumps or a combination of both. The sites provide automatic upstream level control or downstream flow control and may also automatically switch between level and flow control.

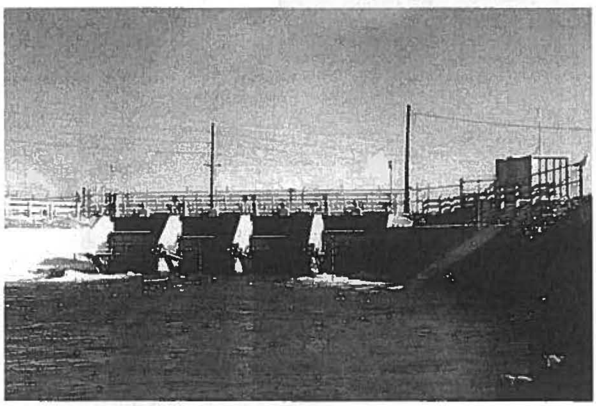


Fig. 3 Main Canal Check Structure

Some sites consist of a combination of gate control structures and pumping plants that provide different functions depending on whether there is a shortage or access flow in the system. The flow control sites include the provision to set a flow setpoint with a time for the new setpoint to take effect.

The major sites are powered by utility power but the PLC and all sensors operate on 24 Volts DC which is supplied by a battery back up system. Many of the sites also include a standby generator. The system was designed so that the PLC can control the standby generator and provides monitoring of generator status including fuel level. The PLC is programmed to automatic test the backup systems once a week and the PLC control also provides the ability to remotely control the generator to conserve fuel and extend operation in a power outage.

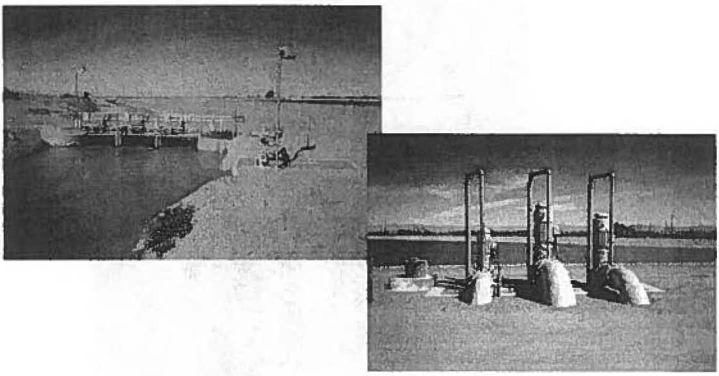


Fig. 4 Interceptor Reservoir with pumps

<u>Construction</u>: The remote monitoring units were pre-assembled and the only site work required was the installation of a level sensor stilling well. The small canal installations were also pre-assembled and required minimal site work. In addition to installing a level sensor stilling well the existing gate was removed so that the new gate could be installed in the same guides of the concrete structure. The main canal sites included existing electrically operated gates which were previously operated remotely by a tone telemetry system.

These sites were upgraded with new gate hoists and new electrical wiring. To minimize the disruption of service during the transition of a site from the old control equipment to the new equipment a pre-fabricated control building was used in the design. Working with the manufacture of sea-going cargo containers, the specially constructed units were designed to require very little maintenance and also to be bullet resistant. The units were delivered to the IID's yard where the control equipment was installed and tested.

At the site a concrete pad is constructed and conduits installed. The control unit was then delivered to the site and the field wiring is completed.

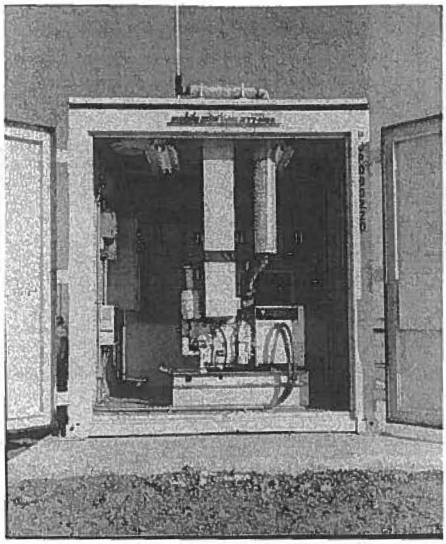


Fig. 5 Standby Generator

To better prepare district staff to maintain the system it was decided that IID staff should be involved in the actual construction of the new control system. To facilitate this approach the design drawings were prepared to "shop drawing" details. The staff was also involved in the site checkout and commissioning.

Hardware: The equipment used at the field sites is industrial grade and of very high quality. For example the PLCs have a Mean Time Between Failure (MTBF) of almost 1 million hours. Over 40,000 units of this brand of PLC are produced annually and they are in use and supported in over 130 countries around the world. They were first used in 1968 in the automobile manufacturing and are now used by power utilities, in water treatment plants and all types of manufacturing processes.

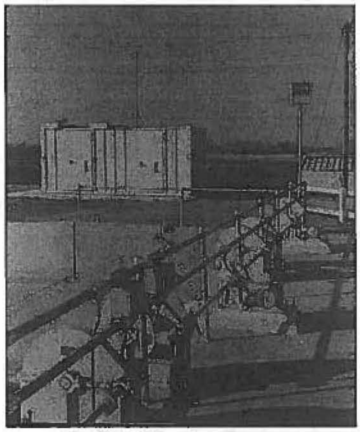


Fig. 6 Prefabricated control building

Several PLC manufactures were evaluated and this specific brand of PLC was chosen because of its robust communications protocol and its highly desirable feature of being able to make program changes remotely and without stopping the program.

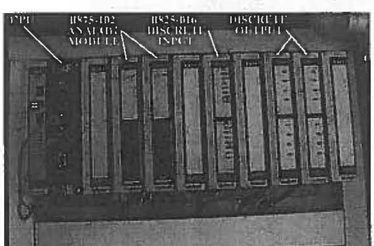


Fig. 7 Programmable Logic Controller (PLC)

The RTU used for remote monitoring applications was a single board controller. It supported the same communications protocol as the PLCs but provided additional data storage capability over the PLC, as well as being lower in cost and using less power.

All sensors and transmitters used produced an industry standard 4-20 mA which is much less effected by electrical noise than a voltage signal. The analog inputs are protected by surge protection devices which provide additional protection to the PLC input modules. If the sensors are considerable distance away from the control building then surge protection devices are installed at the field devices as well.

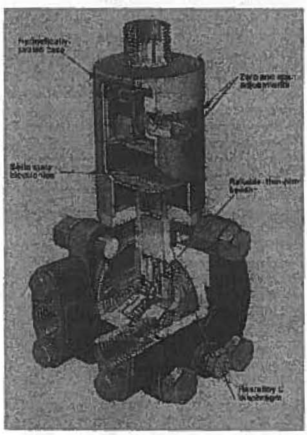


Fig. 8 Differential Pressure Transmitter

The level sensor used water levels greater than 1200mm (4ft) was a differential pressure transmitter. These units have an accuracy of 0.1% as compared to the more common accuracy of 0.25% for such devices.

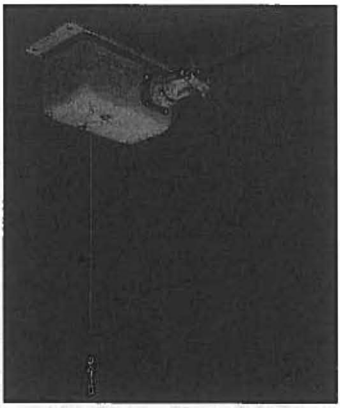


Fig. 9 Level sensor (shown without float)

For water level spans of less than 1200mm (4ft) a low cost level sensor was used. It consisted of a potentiometer driven by a float connected to a cable which is under tension by a torsion spring. Both sensors provide an adjustable span, which further enhances the level measurement accuracy.

New gate hoists were installed at most of the major sites. The new hoists include over torque protection, limit switches and gate position transmitters. A special gate transmitter enclosure was designed for the project which was required because of the limited working space between the gate hoists in these multi-gate installations.

Accurate flow measurement was an important consideration not only for the operation of the system but as well for accurate data collection for the verification program. To obtain flow measurement in the All American Canal a design was developed for an acoustical velocity meter that could be installed with the canal flowing.

The design involved installing power poles in the canal and using them to mount the transducers. Divers accomplished the final alignment of the transducers. A simpler installation was also used in a concrete canal where there was no head available for a weir or other head measurement device.

Fig. 10 Open Channel Acoustical Flow Meter

Acoustic flow meters were used to obtain pumping plant flows on the Lateral Interceptor projects. This meter allowed monitoring of the pumped flow but also provided reverse gravity flow, which was possible in some applications.

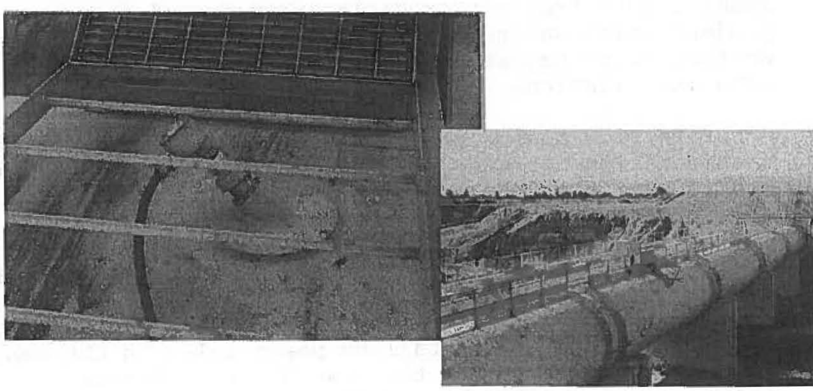


Fig. 11 Acoustical Pipe Flow Meter

This type of unit was also used to obtain flow measurement through a pair of seventeen and one half-foot diameter siphons. The manufacturer modified their electronics to handle the longer signal paths.

Software: A large portion of the PLC software provides for "safety and order" which is critical for the large control sites, including major points on the All American Canal. This was also important for the smaller sites since they operate automatically without remote monitoring and use less expensive components.

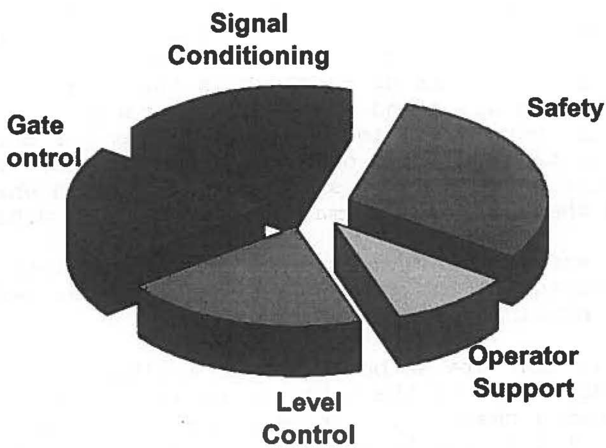


Fig. 12 Software Components

The software functions in a cascade fashion. The lowest level provides communications support. The next level functions to condition and verify all analog inputs.

Since the level sensor is the primary device it is constantly monitored for abnormal readings; too low, too high or too fast a change. If the level sensor is determined to be unreliable the sensor is failed and automatic operation is halted.

The next level is the gate control logic. The gate position sensor is monitored to detect drifting when the gate is not being moved, when the gate is moving; the rate of change in the gate position is monitored for too slow or too fast a change in readings. If the gate position is determined to be unreliable then the gate is failed and in a single gate site automatic operation is halted. In multiple gate sites, automatic control would continue with the remaining available gates. The logic also looks at the Hand/Off/Auto switch and if a gate is not in Auto then the gate cannot be moved by the PLC and gate control is failed, and again automatic operation is halted.

The gate control logic includes a routine to always position a gate from a lower position to a higher position. If a gate is being lowered it is driven past setpoint then raised to the desired setpoint. This ensures there is no backlash in the hoist gear reduction units and provides repeatable gate positioning. The logic also provides for multiple re-tries to obtain the desired gate position. The gate position setpoint is shown in engineering units (feet) and the gates are normally positioned to 0.01 feet.

The software includes soft limits that limit the operation of the gate hoists and provides redundancy to the physical limit switches.

Sites that are AC powered also include power monitoring to determine if the power is ok which can include checking phases of a three-phase service and/or checking to see if AC power is available. If the AC power is determined to not be ok or is unavailable, then gate control is failed and automatic operation is halted.

For multiple gate structures, the gate control logic includes a staging sequence, which allows for opening the gates in a sequence that best suits the canal and structure hydraulics. This logic also handles failed gates in the staging logic.

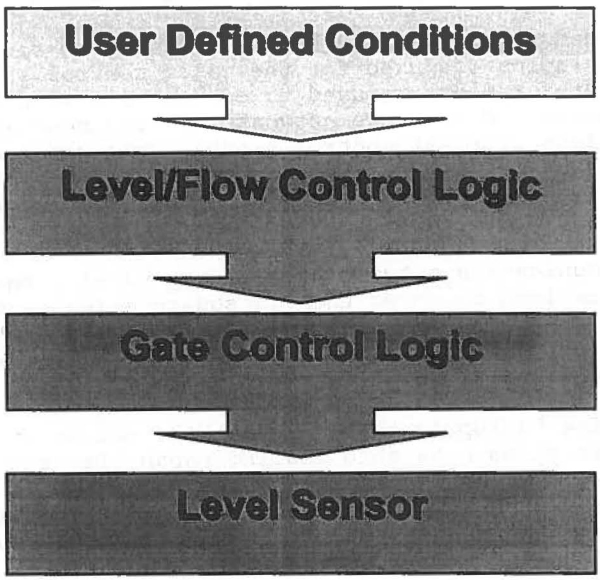
The pump control logic also includes staging sequences for adding and removing pumps similar to the gate stager. The pump control also incorporates logic to control the sequencing of the pumps to provide equal running time. Several of the pumping plants include

Variable Frequency Drives (VFD) which provide more exact control and reduce energy costs.

The final level of logic is level/flow control. Both of these modes of control use Proportional Integral and Derivative (PID) control logic. IID Lateral canals are on slopes greater than 0.002 ,which results in the flow in the concrete canals to be nearing critical velocity. This combined with farm deliveries of 0.6cms (20 cfs), results in dramatic flow changes in these small canals.

The proportional part of the logic adjusts the gate set point proportional to the deviation in level or flow. Integral logic works to bring the level or flow back to setpoint within a time period. Derivative logic looks at how fast the deviation is changing and adds to the other two terms to speed up the return to set point.

For sites requiring maximum accuracy control is tuned to maintain the water level within +/- 3mm (0.01 feet) and flows are maintained to the equivalent flow for +/- 3mm (0.01 feet) of level or +/- 3mm (0.01 feet) of gate opening, which ever flow is larger. Less critical sites may be tuned to maintain the water level to +/- 10mm to reduce the amount of wear on equipment.



As mentioned earlier, flow control sites often include the provision to set an "automatic flow up date" with a time for the new setpoint to take effect. The level setpoint is not often changed at sites that are maintaining a constant upstream level setpoint, however the software includes logic to control the rate at which a new level setpoint is achieved. This was specifically designed to reduce bank instability due to de-watering an unlined canal to rapidly.

Communications

To meet the IID's different SCADA requirements, three radio systems are incorporated and function as one system. The three radio systems provide different levels of service in terms of reliability and speed. Each of the radio systems include several master radios which are connected by a microwave system and a combination of high-speed digital and low-speed analog modem links to Water Control.

The RTUs, which are used to collect historical information, are polled using lower frequency (450 MHz); lower cost radios and lower speed modems (1,200 baud). At present, two master radios handle the current data traffic on the low speed network.

High-speed network (9,600 baud) using higher frequency 960 MHz radios are used for the major control sites. Four master radios are used to communicate with the major sites. A midrange network has also been added to handle less critical control sites. This radio highway uses low cost 450 MHz, 9600 baud radio modems. In total nearly two hundred field units are polled through the communications network.

The communication network uses radio and microwave communications to cover the 700 square miles of the district. It has been found that a microwave dish may shifted off alignment as a result of an earthquake and communications could be lost.

To provide backup in such an event the modems at the microwave sites have auto dialing capability and use special high priority emergency phone lines. The equipment has been configured so that if the microwave link is lost the backup system will automatically connect and restore communications.

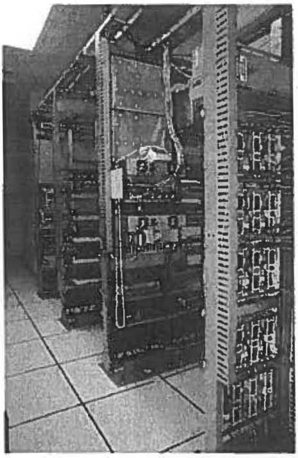


Fig. 13 Communication Rack

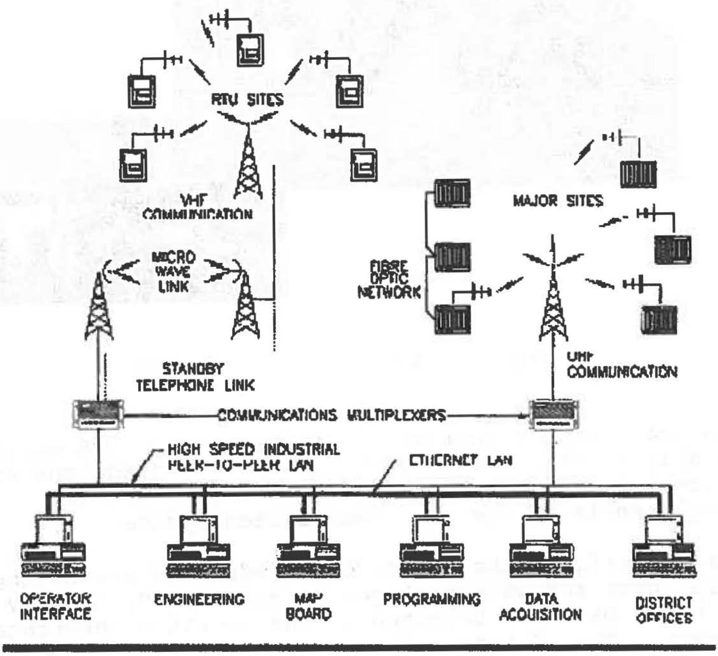


Fig. 14 Communication Network

New Water Control Center

The 10,000 square foot Water Control Center is designed around a large control room which has several operator stations and three 67" rear projection screens. The IID's main canal system is displayed graphically on the rear projection screens with key information such as flow rates, reservoir storage, gate positions and water levels displayed in real time. This approach provides software-configurable mapboards and better accommodates future expansion.

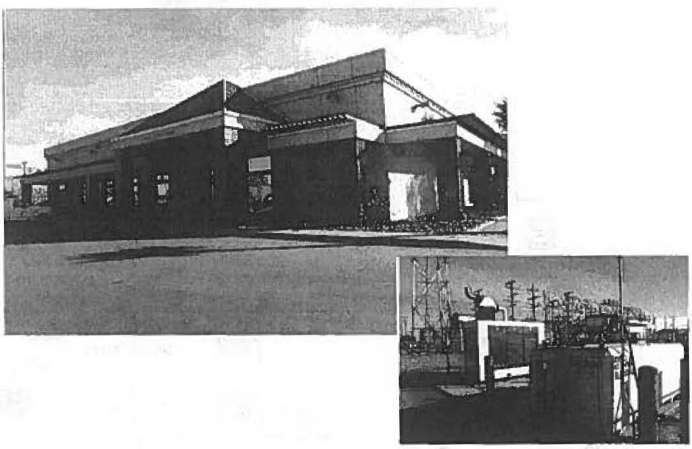


Fig. 15 Water Control Center

A low cost graphic operator interface system was setup initially to help identify the District's needs and to provide the staff with a gradual move from the old control panels to the new computerized system.

After identifying the District's needs, and evaluating several host software packages in early 1990, Factory Link by US Data was selected as the operator interface software. The US Data software was offered for several operating systems; DOS, Windows, OS2 and Unix with an easy upgrade path. For the IID application, OS2 was chosen mainly as a more reliable operating platform than Windows while not being as expensive as Unix.

IID staff were trained on the configuration of the operator screens at the new Water Control center and as sites were commissioned in the field they added them to the operator screens.

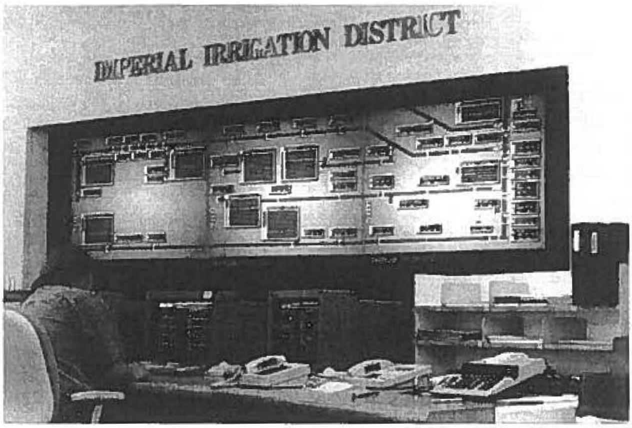


Fig. 16 Computer Generated Map Board

The new Control Center is also backed up by standby generator, which is also controlled by a PLC with the same functionality as the field sites. The control center is designed to be an emergency operation center in a disaster.

Maintenance Program

The district's maintenance program has been developed using a maintenance scheduling computer program. The software allows for tracking all tasks and costs and scheduling of maintenance. This has helped refine the interval between maintenance schedules and has shown areas where preventative maintenance could be replaced by predictive maintenance.

Documentation for maintenance has been developed using a web browser. It provides complete documentation in one package and includes site description, site drawings, calibration information and even site photos.

The maintenance manual is written to a CD so that it is available to the maintenance staff in the field.

OPERATIONAL BENEFITS

Equipment Independence

Site operation is fully automated. This provides the system operator with the ability to concentrate on system wide water management. Automatic system override is available to the operator all times at various levels.

Level Control: The operator can select to have a level setpoint maintained and be assured of continuous monitoring and control by the PLC. Level setpoints can alternatively be raised or lowered to a target setpoint at a selected rate.

Flow Control: Continuous flow monitoring and control is maintained by the PLC. The operator can preset updates that match the dispatching schedule and the actual arrival of water.

Reservoir Operation: Site operation is fully automated. This provides the system operator with the ability to concentrate with system on water management. Automatic system override is available to the operator all times at various levels. Normally reservoirs maintain level at adjacent upstream ponds. The software allows the reservoir to be placed offline and have an adjacent structure resume upstream level control.

Field Operators: Local site operation can be conducted by field personnel, at various levels of control. In fully automatic mode field staff can change setpoints and monitor operation using a man-machine-interface (MMI)device. Local-supervisory control is also available through the MMI. In this mode the field operator takes responsibility of the site through various cascaded fallback modes.

System Management

With the various levels of automatic to supervisory control provided by the software operations staff is able to concentrate efforts on managing the system instead of operating equipment. This has allowed to fine tuned their own skills at another level of operation.

Reliability: Site operation was made more reliable due to various factors. The use of industrial grade ruggedized equipment reduces the failure rate. This allows more up time for operation and less down time maintenance and repair.

Fall back features were then incorporated into the system so that if a piece of equipment fails a strategy is in place that allows the control system, a remote operator or a local operator continue with the process.

Accuracy: Increasing the accuracy of the measurement devices and the operating equipment increases the ability to achieve target setpoints with minimal operating variations.

This allows the operator to reduce the operating fraction from the total amount of water being requested from the source upstream. Each point in the system thus reduces the operating fraction of water required. In a large system such as the IID this can be a considerable amount of water, but it has not been quantified.

Uniform Operation: Variations in system operation can be caused the person scheduled to work, and the flow season of the year. This can result in more or less fluctuations in the levels and flows.

Invariably staff developed skills to varying degrees based on individual skill and interest in the operation. Previously the IID operated 22 remotely controlled sites and 38 field operated sites. Each was manned 24 hours a day with rotating shift staff. The individual abilities of each operator were reflected in the amount of fluctuation that developed at each site and the overall flow balance in the system.

The SCADA system reduces the need to operate individual sites and equipment at each site. Sites are controlled independent of the operator with consistent system-wide criteria and operating parameters providing uniform operation.

Timeliness: Prior to the development of the SCADA system on the main canals, the upstream level at each site was checked and modified once an hour at remotely controlled sites, or as soon as a hydrographer could return, at field operated sites.

With the new system level control sites benefit from the continuous operation of the level control logic operated at each site. Fluctuations are managed by the system as they arrive at the site.

At flow control sites flow changes were managed by the hydrographer based on his standard schedule. Flow fluctuations were checked and adjusted when the hydrographer had time during his work period.

The new system allows for multiple flows and their appropriate diversion time to be entered remotely. At the scheduled time the flow change is made and continuously maintained with feed back from a measurement system.

Flexibility: At key sites dual functionality is provided by the software; level or flow control. Some locations allow the fluctuation of the level and serve as inline reservoir. These sites are normally operated in flow control with high and low level overrides that convert the site to level control if the system edges towards either extreme.

This dual mode is also provided at sites with multiple structures. Each structure can be the primary level control point depending on equipment availability, seasonal operational requirements or emergency operations.

WATER CONSERVATION PROGRAM IMPACTS

Integration Of Projects

Flexible service to the water user was one of the main goals of the water conservation program. Various projects were implemented to achieve this. This created main canal fluctuations because of the size and number of change orders made by the water users. The new SCADA system was able to provide the system operators with the ability to manage the system.

12-hour deliveries were devised to match on-farm irrigation needs. Normally water deliveries are made in 24-hour periods. The 12-hour program allowed the water user to irrigate more accurately by allowing a finish order to be placed on the system. At the end of the 12-hour period the water is returned to the system for use elsewhere.

Interceptor systems permit a water user operating within the area to have the water order to his farm cutoff by IID personnel when the irrigation has been finished. This can occur any time of the day and creates large numbers of returned orders.

Pump-back systems capture water that is reaching the lower end of an irrigated field and re-circulate it to the upper part for reuse. At the point in time that water is re-circulated the order is reduced by the same amount. The reduced portion of the order is returned to the system and can occur at various times of the day.

All returned flows are returned to the main canal system and managed via the SCADA System by providing accurate measurement and timely control.

Program Verification

One of the main aspects of the IID/MWD agreement was the Verification Program. In order to quantify the amount of water being conserved by the various projects an extensive monitoring program was developed. Without the ability to identify "wet" water, the Water Conservation Program would not have been accepted by the parties involved.

The Verification Program required installation of flow measurement sites, continuous monitoring and data storage. The SCADA system provides for data retrieval and storage for the quality control system that is used to verify water conservation projects. It also allowed the integration of small monitoring sites with the numerous large control sites within one operating environment.

ON-FARM WATER MEASUREMENT AND EVALUATION

David Bradshaw[1] Tim O'Halloran[2]

ABSTRACT

Metering of farm water deliveries in the Imperial Irrigation District has always been a costly and difficult procedure. Due to existing structural and environmental conditions, many of the traditional methods of metering deliveries had in the past proved cumbersome or unsuccessful. With funding provided by the IID/MWD Water Conservation Program, a method for utilizing ultrasonic transducers for metering farm water deliveries under orifice flow conditions has been developed. These on-farm water level sensors were designed to be portable, environmentally rugged, solar powered, simple to operate and maintain, and visually unobtrusive to minimize vandalism. This paper describes the construction of the on-farm water level sensors and their function as a useful tool in providing rapid and accurate irrigation evaluations to farmers.

INTRODUCTION

The Imperial Irrigation District is currently involved in the IID/MWD Water Conservation Program both on-farm and system-wide. The Irrigation Management Unit is responsible for implementing the on-farm programs. Some of the current projects the Irrigation Management Unit is involved with include; tailwater-return systems, linear move systems, and the use of CIMIS to help with irrigation scheduling.

In 1995 the Irrigation Management Unit completed development of 15 portable meters. The meters record the amount of water entering a field through the delivery gate and the amount of tailwater leaving the field through the tailwater box. Fifteen fields can be monitored at any one time. The sensors remain in the field for an entire irrigation event. When the irrigation event is complete, they are moved to another field. Quality control hand readings are taken and compared to the sensors. This information is then processed and an irrigation evaluation is created.

---

[1]Acting Supervisor, Irrigation Management Unit,
Imperial Irrigation District, Imperial, CA 92251

[2]Assistant Water Master, Kings River Water Association,
4888 East Jensen, Fresno, CA 93725
Formerly Supervisor, Irrigation Management Unit
Imperial Irrigation District, Imperial, CA 92251

HISTORY

The water users in the Imperial Irrigation District order water one day in advance in 12-hour or 24-hour increments. The ditch tender arrives in the morning to "set" the order. The ditch tender takes three measurements: 1) water level upstream of the gate, 2) water level downstream of the gate, and 3) inches of gate opening. These measurements are input into a table based on an orifice flow equation. The ditch tender checks this gate two more times during the day to make sure the water user is "on-order." This existing method works well for general billing purposes and total water use accounting, but does not tell the whole story as it relates to water flow fluctuations throughout the day and night.

METERING DEVICE DEVELOPMENT

The Imperial Irrigation District needed a method of metering water deliveries that was reliable, accurate, affordable, and able to withstand environmental extremes of the Imperial Valley. Among the various meters considered, propeller meters were looked at first. Due to the high silt loads and the amount of moss in the irrigation water, propeller meters were ruled out as a reliable device for this type of application.

The Imperial Irrigation District had done extensive work in the past with broad-crested weirs to measure delivery flows into individual fields. They however had some problems. Because of our very low delivery pressure situation they were not widely applicable throughout the district. The engineering and construction costs were also prohibitive. With over 5,000 delivery points being measured with orifice flow equations, it was disturbing to measure water on a small subset of the gates using a different measurement method and then call it representative.

Also considered and discarded were pressure transducers and acoustic velocity meters. The pressure transducers do not lend themselves to moving between locations every few days. The membrane on certain types would dry up and then fail when the water reached the sensor. Sometimes an air bubble would form on the tip of the sensor and give a false reading. Also, the inlet to the membrane would often plug with silt and prevent water from reaching the sensor.

What was finally arrived at is a portable device that is easily moved between pre-calibrated sites. The major components of this metering device are ultrasonic level sensors for reading both upstream and downstream water levels and a linear transducer attached to the gate stem that measures gate position. These three sensors as well as ambient air temperature and battery voltage are input into a data-logger that logs a reading on a 10-minute interval. This data is later retrieved for graphing and analysis.

ADVANTAGES AND DISADVANTAGES

An advantage of this device is its ability to accurately measure flow without touching the water. The ultrasonic level sensors send a sound pulse to the water that determines the distance within 1/8-inch accuracy. This non-intrusive approach avoids many moss and silt problems inherent in the other devices. Environmental concerns were met by enclosing the meter in a waterproof and dust-proof container. This container has a tight seal and insulation on all sides. The harsh desert environment does not affect the equipment located inside. This device is portable and can be set up in less than one minute. This portability allows the $3200 value to be amortized over several sites. Meter brackets easily attach to existing gate structures without any gate modification. This bracketing also allows the meter to be locked to the gate structure.

Perhaps the greatest advantage of this type of device is the way in which it exactly duplicates the measurements of a Zanjero. Flow changes can be attributed directly to either pressure through the structure or gate position. This device gives us a more accurate picture of delivery flow behavior.

Some of the disadvantages encountered include floating debris below the sensors causing inaccurate readings. For this reason and others the Irrigation Management Unit has engaged in a quality control program to assure better evaluations. Technicians go to the gate operating during an irrigation event to measure by hand the two levels and gate position. These levels are recorded on the irrigation evaluation. From here analysts can compare readings and check for any inaccuracies.

IRRIGATION EVALUATIONS

Ultimately the delivery data coupled with tailwater data allows the Irrigation Management Unit to consider the total irrigation event and provide timely feedback and suggestions to water users. There are three groups that currently use the irrigation evaluations:

1. Farmer, Irrigation Foreman, Irrigator
2. IID Operations Staff
3. Irrigation Management Unit
4. Water Resources and Planners

Delivery and tailwater flow is recorded in 10-minute intervals and the water user is provided a hydro-graph of an Irrigation Event. This hydro-graph provides a dynamic picture of an irrigation event by measuring the total amount of water onto and off a particular field.

1) By examining an irrigation evaluation chart the water user can see the amount of water delivered and the amount of tailwater spilled and at what times this took place. From here the water user can decide if his irrigation practices are optimum and what options are available to conserve water delivered or tailwater if necessary.

2) By examining an irrigation evaluation chart the IID Operations and Divisions can see the amount of water delivered as calculated by the sensors and compare the flow to their own measurements. The sensors measure the same exact three components that the ditch rider measures, and in the same way. The three measurements; upstream level, downstream level, and gate position are measured directly in inches and charted. Fluctuations in flow are easily traceable to one of these three components shown on the evaluation chart. From here IID Operations can target certain laterals that fluctuate more than others can and determine if any measures need to be taken.

3) By examining an irrigation evaluation chart the IMU Unit can see the amount of water delivered and the amount of tailwater spilled for educational purposes. Depending on the demand for information, fields with certain crops, soil types, or special irrigation practices can now be accurately monitored. Ultimately, when enough irrigation evaluations have been completed this data may enhance or replace data currently used for IID delivery and tailwater averages.

4) Water Resources staff and District planners can see areas of the District that have unacceptable fluctuations and plan for mid-lateral reservoirs, automated gates, and lateral interceptors.

HYDRO-GRAPHS

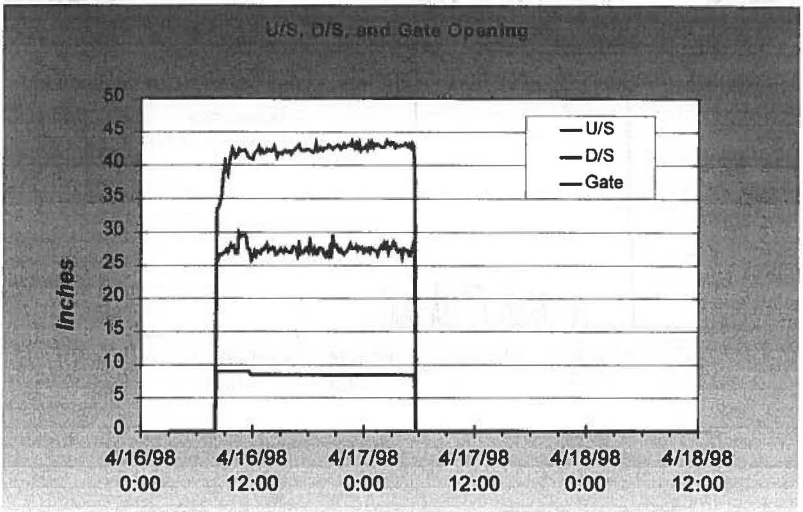


Figure 1. Water Measurement Components

Figure No. 1 shows the three components the meter is required to measure in order to get a flow measurement through a delivery structure. The top line (between 40 inches and 45 inches) is the upstream water level with respect to the gate, the second line (between 25 inches and 30 inches) is the downstream water level with respect to the gate, and the third line (between 5 inches and 10 inches) is the slide gate opening. The length of this irrigation event can also be determined. The gate was opened at 08:00 on 4/16/98 and closed at 06:00 on 4/17/98. These readings are taken at 10-minute intervals and stored for later retrieval.

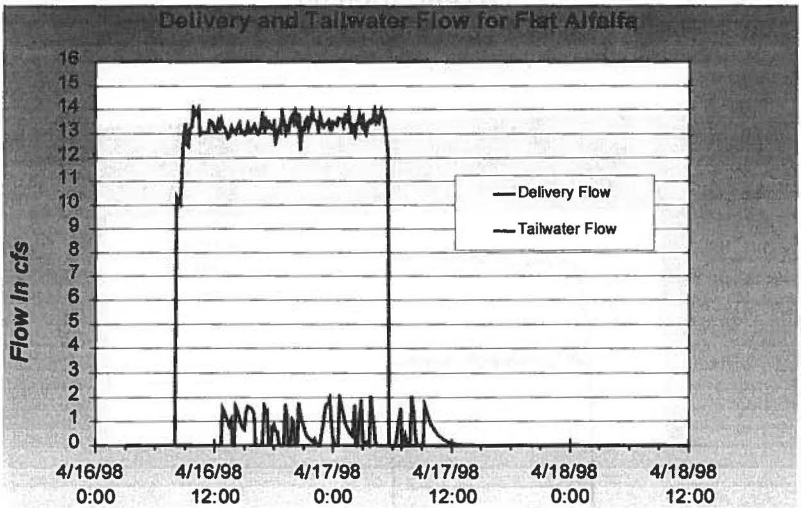
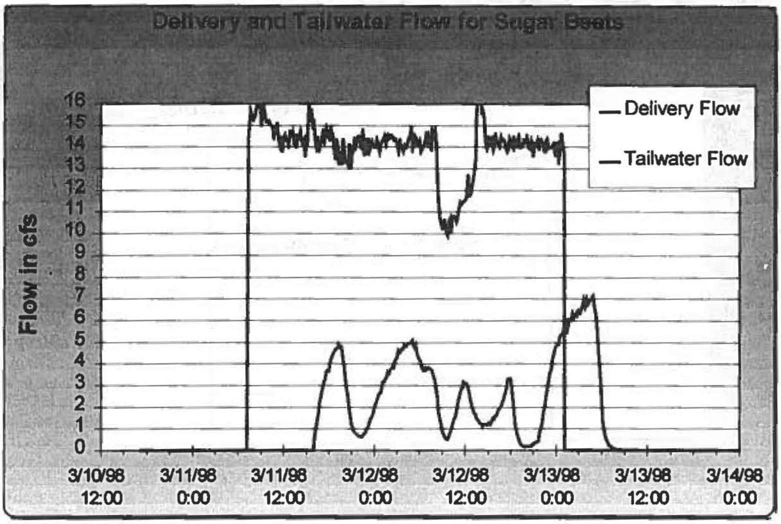


Figure 2. Flat Crop Irrigation Event

Figure No. 2 is an example of a flat crop irrigation event. This is a hydro-graph built from the previous three individual components; upstream level, downstream level, and gate position. These components are input into an orifice flow formula and flow can now be read directly in cubic feet per second (CFS) between 13 CFS and 14 CFS. The tailwater is also calculated with a weir formula and input into the hydrograph. The evenness of the delivery flow and the tailwater flow (below 2 CFS) are evident in this example.



Delivery Volume (AF) = 48.3 | **% TW= 20%** | **Acres = 70**
Tailwater Volume (AF)= 9.5 | **Order = 2d-15'** | **In. App.= 8.3**

Figure 3. Row Crop Irrigation Event

Figure No. 3 is an example of a row crop irrigation event. By calculating the acre-feet of water applied and the acre-feet (AF) of tailwater runoff, a more complete picture is formed. With this irrigation event there is some unevenness in the tailwater flow. The last set of tailwater is flowing more than the previous and may be combining with other sets causing a larger amount of tailwater near the end of the irrigation event. One way to prevent this is to order a "cut" in the water order amount that better matches the individual field's needs. An example of a cutback irrigation event is shown and explained in figure No. 6.

Included in Figure No. 3 are the original water order and the number of acres in the field. Analysts calculate inches applied from the acres and actual measured water amount. This information is taken to the field and discussed with the water user. The evaluation is left with the water user to serve as a tool to help with the next irrigation event.

By looking at the line representing the flow in CFS, a dip is noticeable. The flow dropped at 08:00 on 3/12/98. By looking at the individual components of this flow it will be possible to determine the cause of the flow.

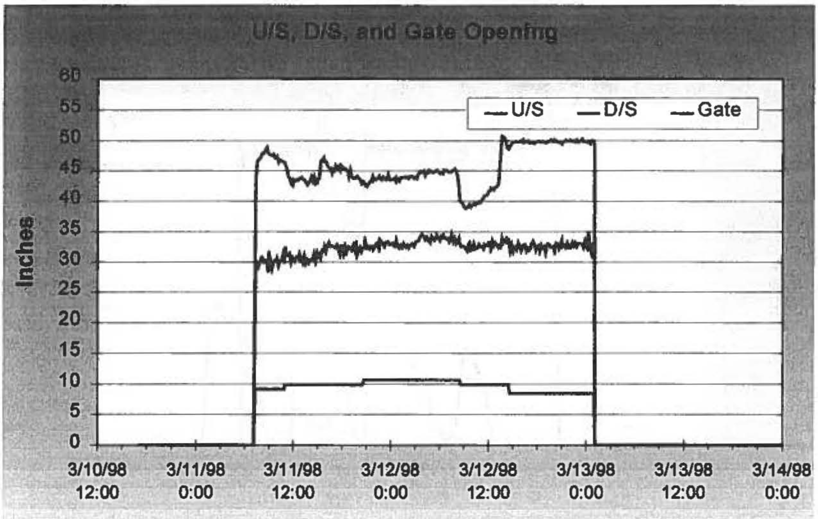


Figure 4. Row Crop Water Measurement Components

By looking at Figure No. 4 it is possible to see how the upstream level drops from 45 inches to 40 inches at 08:00 on 3/12/98. The pressure through the structure drops from 10 inches to 5 inches. This drop was reflected in the flow as shown in Figure No. 3. The flow at the same time dropped from 14 CSF to 10 CFS. Eventually the water level moves up (to 50 inches) and ordered flow is attained once again.

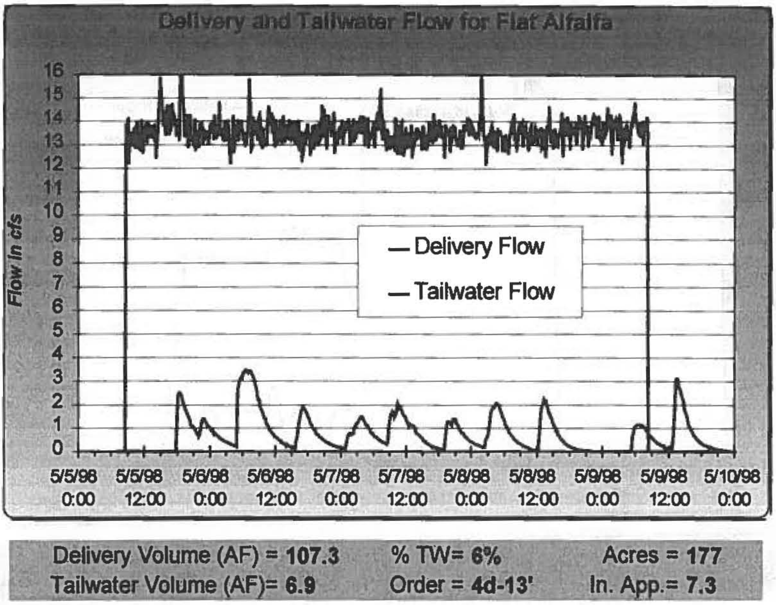


Figure 5. Flat Crop Irrigation Event

This is an example of a flat crop irrigation event. The flow and tail-water is even with very few larger peaks. This was a four-day irrigation event for a 177-acre field.

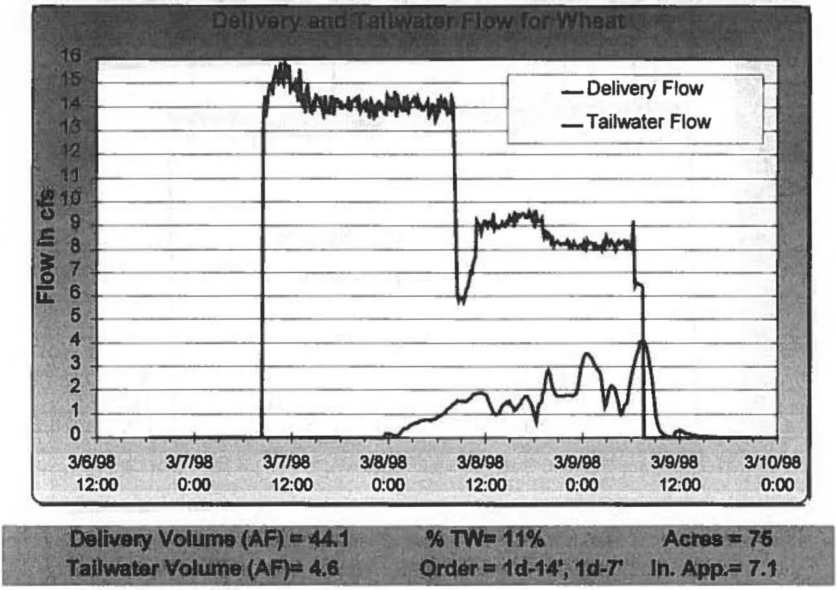


Figure 6. Flat Crop Irrigation Event

This is an example of a cutback irrigation. Anticipating the tailwater to build up, the water user requested a cutback irrigation. The water user ordered a 5 CFS cut. This caused the flow to drop from 14 CFS to 9 CFS. This allowed the water user to better manage the amount of tailwater.

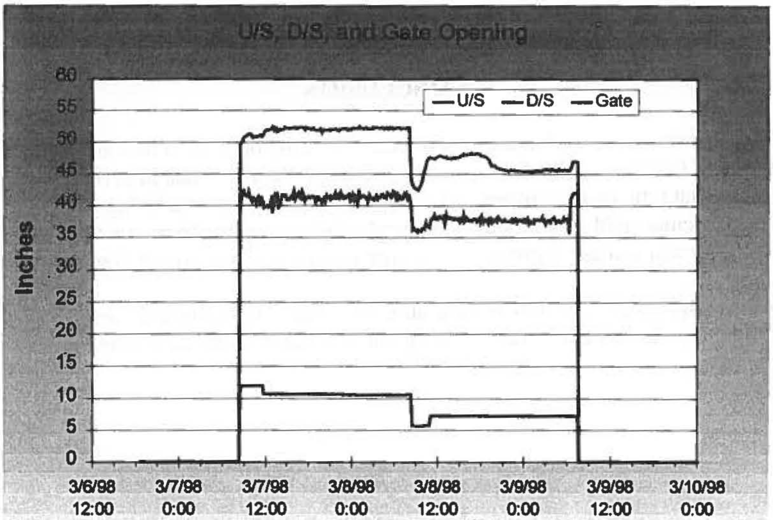


Figure 7. Flat Crop Components

By examining the flow components the reason the flow reduced from 15 CFS to 9 CFS can be determined. Note the pressure (or difference between upstream and downstream water level) stays about the same. The pressure stays around 10 inches but the gate has been lowered from 11 inches to 7 inches. This 4- inch change in gate opening caused the flow to drop.

CONCLUSION

The on-farm water level sensors have been and will continue to be a successful program. Our water users in this program benefit from this data in several ways. For the first time many growers can at a glance understand what is happening on their particular field. By looking at several irrigation events from one field the water user can recognize problems and true water conservation can begin.

New measuring devices may be available soon that are simpler and more affordable. This would be the next step into a broader water measurement program.

IID WATER INFORMATION SYSTEM -- IRRIGATION DATABASE

PRINCIPLES AND MANAGEMENT

Bryan Thoreson[1] Anisa Divine[2]

Mike Archer[3]

ABSTRACT

The Imperial Irrigation District (IID) and the Metropolitan Water District of Southern California (MWD) entered into a pioneering water transfer agreement in 1988. MWD was to finance water conservation programs in the IID in return for the transfer of the conserved water volume each year for 35 years. A requirement of the agreement was that the water conserved be verified.

In 1993, the authors spearheaded collection of conservation verification data using the WCC SCADA system. As measurement structures and data collection equipment were installed and calibrated, data collection began. Initial examination of the data showed that certain types of data errors occurred repeatedly. A Fortran program, developed to incorporate these checks, was used in conjunction with spreadsheets to achieve a high level of consistent, automated data quality control. Meanwhile, other IID departments were collecting data using Stevens Charts and data loggers, with the logger data being processed in a spreadsheet. Manual quality control was performed on both types of data.

The IID/MWD program began developing a Water Information System (WIS) incorporating daily quality control operations and a data storage warehouse function for site-specific, time-series data related to the flow of water through IID's irrigation and drainage system. The WIS also provides an audit trail of the data elements as they flow through the quality control operation. Once a month, graphs of the data are printed, checked for final quality control, and archived as hard copy. A *Processed Flow Data* document is published annually, and a *Users Manual* is updated regularly to be current with procedural changes.

In this paper, the authors describe the principles they have incorporated and the lessons they have learned during the process of setting up the irrigation flow database. Primary among these insights are: 1) the value of keeping all data in a centralized database eliminating duplication and ensuring that all analysts use the same data, and 2) keeping all data entry and manual data quality control decentralized so that data quality control is done by those who know the data best.

---

[1] Project Engineer, Davids Engineering, 1105 Kennedy Place, Suite 5, Davis, CA 95616
[2] Water Resources Planner, Imperial Irrigation District, P.O. Box 937, Imperial, CA 92251
[3] Senior Engineer, Murray, Burns and Kienlen Consulting Civil Engineers, 1616 29th Street, Suite 300, Sacramento, CA 95816

INTRODUCTION

The Imperial Irrigation District's (IID) data collection program has grown greatly in the last 20 years. Data collection has tended to be the result of specific programs often with little or no coordination between new and existing programs. Data collection programs have proliferated as water supplies remain more or less constant, while increasing demand for water results in demand for better water management. In addition to the often overlapping data needs of various programs, similar data are also needed for operations. A common result is the collection of duplicate data, likely processed differently, resulting in slightly different historical records for the same site stored with different units.

During the same time span, data collection technology has undergone significant change. The preferred method has shifted from analog Stevens Recorder Charts to digital data loggers and is moving to radio telemetry. The advent of personal computers and office networks has provided the technology to easily share data and allows many people in disparate locations access to a single data source.

At the Imperial Irrigation District, the verification data requirement of the IID/MWD water conservation program has been the catalyst bringing the new technology together with the need for more data. Water conservation verification requires many types of data from disparate sources, and provides needed funds to drive data integration. Thus, the IID's Water Information System (WIS), an integrated irrigation database, was born. The WIS contains information important not only for the conservation verification program, but also for improved operation, maintenance and management of IID's irrigation distribution system and for future planning activities. For enhanced IID on-farm water management assistance to farmers, the WIS may one day include weather, evapotranspiration (ET), and tailwater return system (TRS) operational data.

BACKGROUND

IID's first Stevens Recorder was installed in 1914, soon followed by others, until by 1980, some 75 Recorders were in service. The data collected thereby was archived on Stevens Recorder Charts in analog format. In the mid-1980's, IID also began using data loggers to collect data in electronic format, with the data stored on a PC in ASCII text file and spreadsheet format. In addition, the IID has a mainframe, AS400 system which was developed to handle IID's accounting and water order functions. The responsibility for each of these systems lies with a different organizational unit in IID, with data developed to serve a variety of functions.

Staff processed Stevens Charts weekly. Notes were entered to indicate site conditions that affected the data. These entries indicated times when the site was out of operation, when a storm had occurred, when the pen had run out of ink or the chart slipped off the cylinder, and so forth. Often the zanjero (ditch tender) left notes regarding changes at the site, particularly at sites where grade boards were used as the measurement structure. Sometimes these movements could be determined by staff, even without notes from the zanjero. Finally, a flow was recorded on the chart for each day based on each day's average water surface level (head). As of this writing, only a few Stevens Recorders remain in operation in the IID, being replaced by data loggers or SCADA.

Data loggers are in use throughout the IID. Staff collects the packs on a regular weekly or biweekly schedule. Before the development of the WIS, the data were transferred to a PC, and processed in spreadsheets. Processing mainly consisted of adjustment for grade boards, based on notes left at the site by the zanjero and the judgement of the processor, and entry of zeros for any period of missing record (gap). Logger output consisted of hourly flow calculated as an average of four 15-minute water surface levels. Plots of the water levels and flow were printed and distributed weekly.

The AS400, in addition to accounting functions, is used to record water transactions, including orders, changes in orders, and deliveries to each farm delivery gate; cropping patterns and acreage; and other data. These entries are based on IID's accounting procedures as well as zanjero field reports.

Thus, by the late-1980's, IID had data sets in three different formats with each set collected for different purposes, processed by different persons using different methods, and stored in different locations.

ANALYSIS POTENTIAL

Organizational History

IID's data collection, processing, and management system worked well for operational (water movement) and financial (water accounting and payment) needs. However, with the advent of an elevated water conservation ethic, other needs arose, for example planning and implementing water conservation programs, and verifying their effectiveness. At the same time, technological advances allowed the District to implement systems that would facilitate these new needs.

One project developed under the IID/MWD program, the automation of many of IID's main canal structures with a SCADA system, uses radio telemetry from the new Water Control Center (WCC). The SCADA system, which depends on the transmission of data, allows IID to collect data from many sites, both those designed to conserve water in IID for transfer to MWD and those which IID developed on its own.

Two types of IID/MWD water conservation projects have large data collection and processing requirements. System-level projects (mainly interceptors) are under the jurisdiction of the Water Resources Unit (WRU), while the Irrigation Management Unit (IMU) is in charge of farm-level programs (mainly tailwater return systems).[1] Once the data collection began, the data was reviewed and the need for quality control (QC) and data processing was evident. The WRU staff, under the direction of the CVC, developed a data QC and processing procedure.[2] Meanwhile, the IMU staff developed a method of data QC and processing appropriate for the on-farm systems.

Furthermore, processing Stevens Charts to develop a historic record compatible with the level of accuracy available from the electronic data, led to the realization that having data in electronic form greatly increases its usefulness for analysis. Electronic form facilitates many different analyses with a minimum of additional processing time. This was a major factor driving the replacement of nearly all Stevens Recorders throughout the District with data loggers. This logger data is processed by the Hydrography Unit staff, which has recently been placed under the direction of the WRU.

Thus, IID has largely moved from an analog, paper-based data collection, processing and warehousing procedure, to one where the system is almost entirely electronic, with charts printed monthly to maintain a written data archive. However, daily log sheets, which record operational flows as reported by the zanjeros, are still in use, with entries hand-written by dispatchers in the WCC.

Nearly all WRU data is SCADA-based, while both SCADA and logger data are used by IMU. In addition, due to the water ordering and accounting systems of the mainframe AS400, a large amount of farm-level data are available. The result is the availability of a lot of data which may or may not have been recorded, processed, or calculated using the same procedure; and which may or may not be in compatible format for analytical processing. In 1996, with the advent of data QC, processing and warehousing in IID's WIS, this disparity began to change; and a process of data integration began.

Transaction Processing versus Analytical Processing

[1] These programs are described in the accompanying paper: On Farm Irrigation Water Measurement and Evaluation by David Bradshaw, Imperial Irrigation District, Imperial, CA.
[2] The CVC mandate and the QC and data processing procedure are described in the accompanying paper: Irrigation Flow Data Collection, Quality Control and Site Monitoring by Michael C. Archer, Anisa Joy Divine and Bryan P. Thoreson.

Many irrigation districts, including IID, have had an on-line transaction processing (OLTP) system to process water orders and billing. IID's system uses a mainframe computer (AS400) to store data in "flat files." This system has almost 12 years worth of information about water ordered and delivered along with crop acreage and other data. Although this historical data is valuable for planning and analysis, the structure of the OLTP system does not allow optimum performance for analysis. This, along with the fact that the peak use of the OLTP system for order processing (late morning to late afternoon) coincides with the peak use time for analysis, makes planning and analysis functions difficult using this system.

These problems have led to emergence of on-line analytical processing (OLAP) and data warehousing. In these systems, historical data from the OLTP system is stored on another system dedicated to planning and analysis and structured as a relational data warehouse for optimum query performance. Historical data can be moved from the OLTP system into a different structure on the OLAP system in batch routines scheduled to run during the night at predefined intervals, i.e. daily, weekly, monthly. The interval selected depends on the need for current data. OLAP systems are designed to store millions and millions of rows of historical information and still allow users to run queries that return results in a few minutes.

DATA INTEGRATION

IID is collecting, processing and warehousing 15-minute time series data at over 150 SCADA sites. Another 70 or so logger sites provide hourly time series data. The SCADA and logger data are processed to provide hourly and daily flow data for accounting and analysis. The hourly and daily flow data are available in their processed form on the WIS. In addition, the raw data are stored and can be viewed, but not modified, by staff. Figure 1 shows the main data categories that are being incorporated into the WIS.

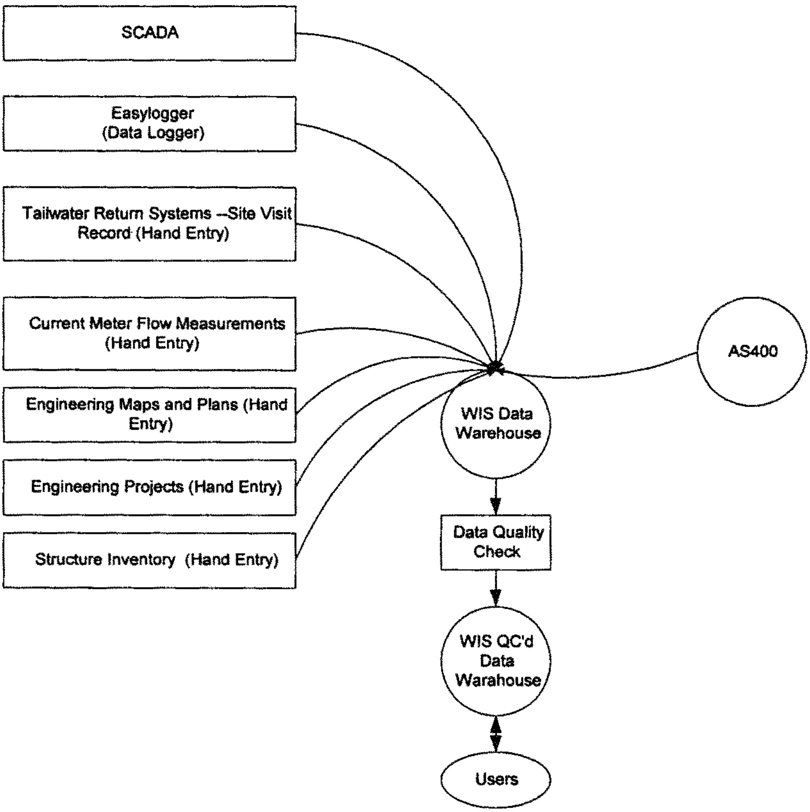


Fig. 1. WIS Conceptual Data Flow Model

Given that the WIS is a LAN-based system, manual quality control has remained decentralized in the areas of responsibility and interest: WRU, IMU, and Hydrography. Other collectors of data may join the system, with the ability to develop their own data QC and processing procedures. Each unit collaborates with WIS programmers to develop a QC processing routine for its data. Next, forms and sometimes graphs are developed that allow the final manual QC processing step to be completed by those who know the data best. Thus, even though the data is processed, warehoused and accessible from a central location, the final QC processing step remains decentralized. These final QC activities are usually carried out once a month before the summary process calculating hourly and daily averages for the previous month is run. The process for SCADA and logger sites is similar. Figure 2 diagrams the quality control process for data from SCADA sites.

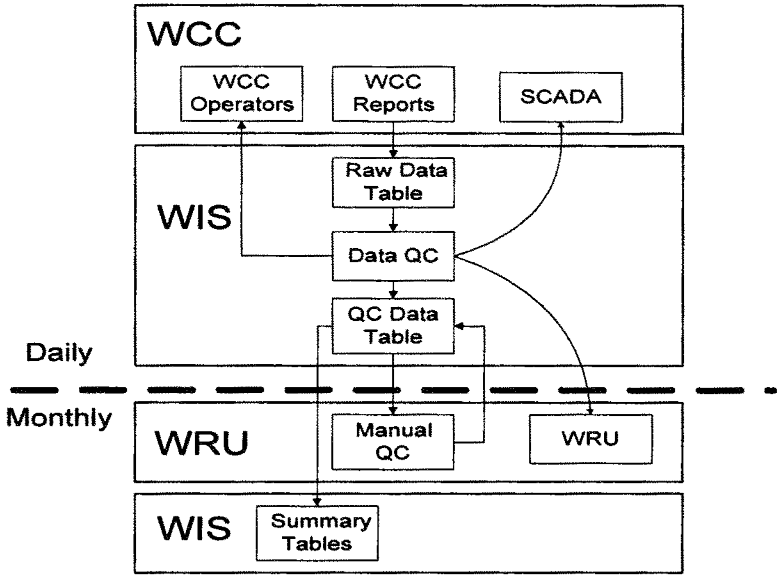


Fig. 2. SCADA Data Quality Control Process

For SCADA sites, the raw 15-minute time series data are processed daily, shortly after midnight. Only the current year of processed 15-minute data remain available on the WIS. Previous years' data are archived to CD-ROM for permanent storage. For logger sites, the processing occurs after the packs have been collected and entered into the system by Hydrography staff.

Several advantages accrue from this integration. One is that collection and processing of duplicate data is eliminated. In addition, the level of data QC and type of processing is well documented. Finally, analysts and planners throughout the District have access to the same data – which are becoming available at their PCs in forms that are increasingly easy to use.

FLOW DATABASE DEVELOPMENT PRINCIPLES

During the design, development and evolution of the WIS, a list of important principles to consider during design and development of irrigation databases that will store many years of data was collected. These principles will help ensure that data can be accessed quickly and easily for analysis and planning purposes. These principles are listed and discussed below.

1. Store all data in a centralized database to eliminate duplication and ensure that all analysts are using the same data.
2. Maintain decentralized data entry and manual QC. Thus, those most familiar with the data are responsible for QC.
3. One flow equals one unique physical location. This allows for integration with a geographic information system (GIS) and facilitates changes in data collection.
4. Classify flow sites according to type of flow measurement at the site for ease of flow calculations. Calculating flow as part of the quality control procedure has the advantage of maintaining a history of site rating curves and parameters.
5. For use in water balance calculations, sites can be classified as: a) spill--flow from canals to drains, b) discharge or interface--flow from canals to reservoirs or other canals, c) headings--flow from main to lateral canals, and d) deliveries--flow from canals to farmers.
6. Classify sites according to "destination" and "origin" pools of water to facilitate volume balance calculations. These classifications can be made at both micro and marco levels.
7. Apply QC codes in a defined priority order, thus a "bad" record receives only the first QC code that applies to it. The remaining QC checks are skipped, once a record has been declared "bad."
8. Classify QC codes according to general conditions. For example, three possible QC code classifications result from: a) QC program checks, b) manual data review and c) variables affecting the flow calculation.
9. Make QC codes general, not referencing specific sites or values. Memos can be written and stored in the database, or site books to describe specific situations that require more explanation. If QC codes are written referring to specific sites or values, the number of QC codes quickly proliferates when the database covers many years.
10. Users who manually enter QC codes have their user identification codes and the date entered in fields on each record they modify. This provides a complete audit trail of the QC process.
11. Store data in only one location, and enter data using lists of values (LOVs) as much as possible to prevent spelling errors. LOVs allow users to enter data based on data already stored in the database so users do not have to type in values.
12. Use a constant flow data collection interval. The interval depends on how often the flow changes. Base the interval on the sites with the greatest data variance – most likely flows from the end of lateral to drains, for which a 15-minute interval has been chosen. Using a constant time step allows accurate hourly, daily and monthly averages to calculated using relational database functions. Whereas, direct use of the average function to determine monthly

and daily averages gives incorrect results when a variable time step is used. A constant time step also makes it easier to predict how much storage space will be needed. Variable time steps may decrease, or increase, storage requirements depending on the parameters used to define when a new value is recorded.

13. Create summary tables with daily and hourly flow averages to increase the speed of access to daily and hourly data that users will need more frequently.
14. Estimate all missing flows using carefully defined procedures depending on the amount of missing data and the timeliness of the required data (flow with estimates required immediately, after one week, or after one year).
15. Use data warehousing principles (Kimball, 1996) and table structures to keep the number of tables to a minimum and relationships between tables simple. Time series flow and level data are stored in a fact table linked to dimensional tables with many, many fewer records that describe the sites, codes and time periods that cannot be easily calculated with standard date functions. The links to these smaller tables are used in queries to limit the number of records that must be accessed in the larger time series data table. These techniques increase query performance.

Documentation

Two volumes have been developed documenting the system. One volume, WIS Warehouse Project System Reference Manual, documents the database structures and related procedures. The second volume, WIS Warehouse Project System Users Manual, documents the user menus. These documents have the added advantage of documenting the reports produced and various procedures used to account for water.

APPLICATIONS

At present, five applications are available to Water Resources Unit staff. These are QC Management, QC Update, QCD Reports, a New Site Form and an Inventory Form. These applications allow any user with rights to perform QC management and update functions, to enter new sites, to access the data in report form, and to enter IID operational site inventory data. The screen for the QC Update menu is shown in Fig. 3 as an example.

A logger application is available to Hydrography Unit staff for loading logger data and performing QC procedures, plotting graphs, and running an annual report. The Current Metering Menu can be accessed by the Chief Hydrographer to enter data and print graphs of rating curves along with the metered points.

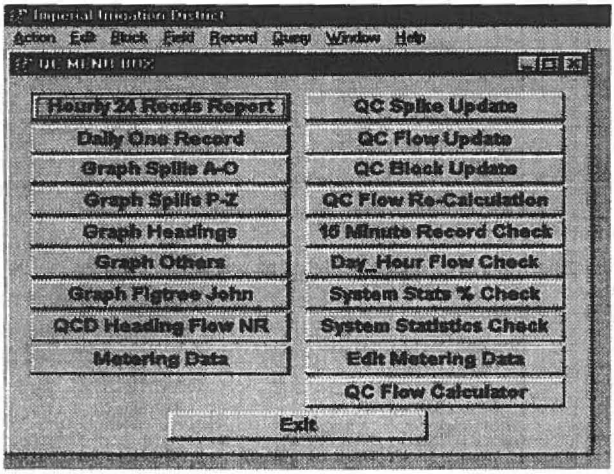


Fig. 3. The QC Update Menu

Daily System, Site and Data Monitoring

WRU staff checks for WIS messages each day to ensure that the data QC procedure has run and that the sites and SCADA systems are operating as expected. In the event that a problem is found, the proper individual is notified so that the problem can be rectified. In addition, WRU staff can run weekly reports to determine the amount of spill along each interceptor and the amount of water discharged into the interceptor reservoirs.

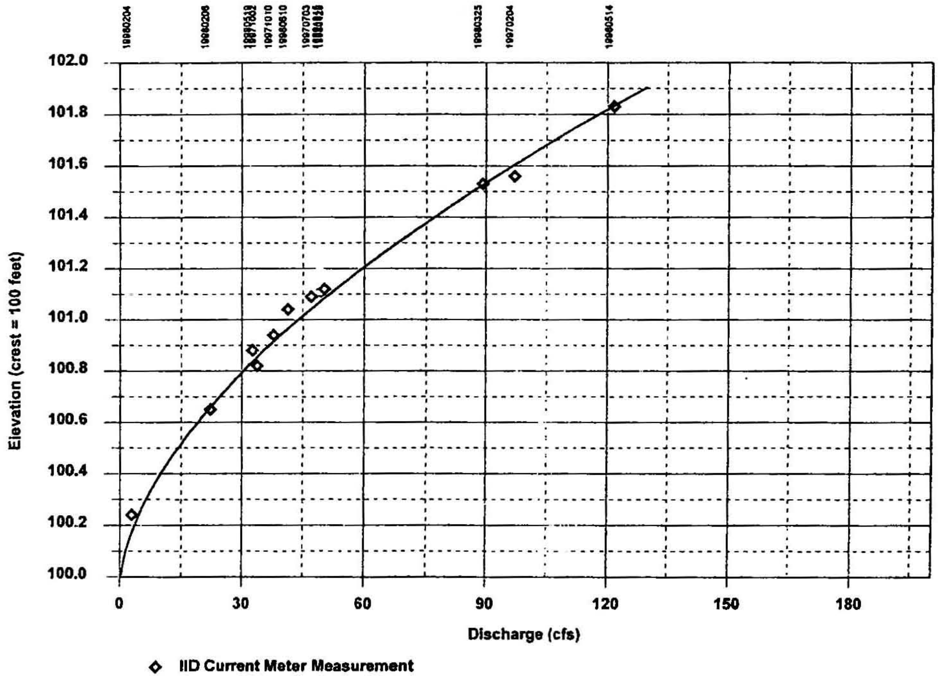
The current metering application allows WRU to calibrate the rating curves for sites where this is needed. This application allows for current meter measurements to be accessed quickly and in a useful format for rating curve development. A graphical application was developed to allow staff to quickly evaluate rating curves versus the current meter measurements (Fig. 4).

Monthly Quality Control

Once a month, the graph applications are used to print a graph for each site. Each graph is inspected for possible quality control requirements, including such things as sudden, large flow rate changes; erroneous flow data associated with a flat overshot gate; and gaps in the heading data. In addition, the record is checked to ensure that the correct number of records are contained in the data set for the month. The data is then manipulated using various other applications to complete the quality control of the 15-minute time series data before the running of the summary process on the second Sunday of the month to calculate hourly and daily flow averages.

Redwood Canal Heading BCW

Rating Curve No. 19951103 (WIS Active Date)



Date Graphed: 07/17/98

Prepared by: WIS, IID

Fig. 4. Rating Curve and Current Meter Measurements Graph

Finally, each graph is archived in a "Site Book." This is a book for each site consisting of pertinent information about the site, and the monthly graphs. In this form, IID is able to keep a "hard" copy of the data.

Processed Flow Reports

Mean Daily Flow reports (Fig. 5) can be printed for all WIS sites, SCADA and logger. In addition, daily averages of digitized Stevens Recorder Chart data, which has been loaded into the system, can also be printed in this form. Tables of Mean Monthly Flows in CFS and Monthly Flow Volumes in AF can also be printed. In addition, once a year the WIS programmer prints a set of plots of the Monthly Volumes.

A Processed Flow Data document is compiled annually. This report contains the data used for verification of water conserved by six IID/MWD projects. Included in this report are an introduction, including site name conventions, abbreviations and acronyms, and tables which graphically indicate the period of processed record.

The next section of the Processed Flow Data document contains a detailed site summary table to assist the reader to locate original files, as well as identify the nature of the site and any notes that might affect the data. An alphabetical index is provided so the reader can easily locate the data tables and plots contained in the report. Finally, for each project, Notes, a Project Area Map, the Annual Mean Monthly Flow and Monthly Volume tables, Monthly Flow in Acre-Feet Plots, and Mean Daily Flow sheets are presented.

This report was initially developed using databases and spreadsheets. This procedure contained the possibility of error as the data was transferred from place to place. Using the relational database, these functions are not only much more easily performed, but the chance of error has been reduced, as well.

FUTURE

Many applications have already been developed for the WIS, and many more are envisioned as additional data are incorporated into the system and development continues. Some future uses include: automated water balances and regular reports.

Automated Water Balance

Addition of a few sites and minor further development work is required to allow monthly and yearly water balances to be calculated as a WIS report. This will allow regular analysis and tracking of system performance ratios and trends.

Imperial Irrigation District
Oat Lateral Spill
03OAT___031_S
Mean Daily Flow In Cubic Feet per Second

YEAR: 1997

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
04	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0e	0.0	0.0	0.0	0.0
05	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
06	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
07	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
08	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
09	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0e	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3 *	0.0	0.0
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3e	0.0	0.0
19	0.0e	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.3e	0.0	0.0
20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2 *	0.0	0.0
21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0
22	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0e	0.0	0.2	0.0	0.0
23	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0e	0.0	2.0	0.0	0.0
24	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0e	0.0	0.0	0.0	0.0
25	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0	0.0	0.0	0.0
26	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0
27	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0e	0.0
28	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	0.0		0.0		0.0		0.0	0.0		0.0		0.0
Total	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	1.0	3.4	0.0	0.0
Mean	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0
Min	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Max	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	1.0	2.0	0.0	0.0
AC-FT	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	2.0	6.7	0.0	0.0

e - 100% of daily volume estimated
* - 50% or more of daily volume estimated

Mean Flow = 0.0 cfs
Total Volume = 9.1 ac-ft

Notes: Day begins at midnight (0000 hrs).
Estimated flow for a missing record gap is computed as the average flow of the records preceding and following the gap equal in number to the missing records in the gap.

Date Printed: 03/11/98

Prepared by: WIS/IID

Fig. 5. Example Mean Daily Flow Site Report

Quality Data Available for Regular Reports

Additional regular reports could be run from the WIS. Only one or two sites remain to be added to the WIS to complete many of these reports.
Even if no more sites are added to the WIS, it is much easier to produce reports and has improved documentation of data collection and quality control.

CONCLUSION

The IID WIS is currently loading and quality controlling over 150 SCADA sites, a total of 14,880 data records (rows) each day. Following quality control, a detailed report is generated listing the problems at any site with greater than five percent of the total records for the day being "bad" is provided to the WRU. Next, a report is written to the operators in WCC indicating the daily flow and level averages at over 30 sites.

Data for around 70 logger sites are loaded into the WIS. Quality control is then performed, and the WIS-generated reports are distributed to IID staff. Once a year, a processed flow report is published reporting daily and monthly summaries of selected flow sites. The current meter graph is used regularly to check the rating curves at various sites.

The WIS has improved the ability of IID staff to access data, perform data quality control and assurance, and account for water in the IID's distribution system. The WIS has also increased the speed of data access and improved the confidence in the data by standardizing QC functions.

REFERENCES

Kimball, R. 1996. The Data Warehouse Toolkit: Practical Techniques for Building Dimensional Data Warehouses. John Wiley & Sons, Inc. New York, NY.

IMPROVED IRRIGATION WATER MANAGEMENT–A DIRECT BENEFIT OF A WATER CONSERVATION PROGRAM

Arnold K. Dimmitt, P.E. [1]

ABSTRACT

Implementation of the 15 projects in the Water Conservation Program (Program) identified in the landmark December 1988 Water Conservation Agreement (Agreement) between Metropolitan Water District of Southern California (Metropolitan) and Imperial Irrigation District (Imperial) and in the December 1989 Approval Agreement among Metropolitan, Imperial, Palo Verde Irrigation District, and Coachella Valley Water District began in January 1990. The last major construction work was completed in December 1997. While the Program has focused primarily on modernizing and rehabilitating Imperial's irrigation distribution system, it has included on-farm water management projects that permit greater water management flexibility for the farmers and opportunities for farmers to apply water more effectively. In actuality, both distribution system and on-farm management improvements are, in some cases, interrelated such that one without the other would reduce the effectiveness of any individual project, of the Program, by itself. The level of the Program's effectiveness has been demonstrated through a process of verifying each project's accomplishments. This paper will review the various projects completed to improve Imperial's overall irrigation system and use of water and how the projects were planned, managed, and the conserved water verified. Additionally, an update on the Program's costs and resulting conserved water volume will be presented.

INTRODUCTION

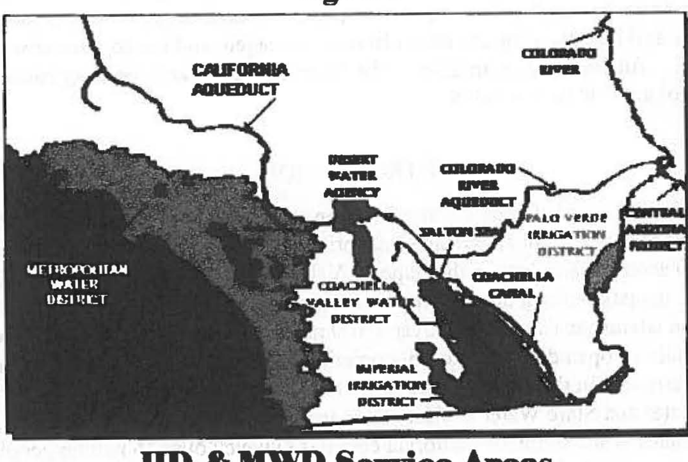
Imperial distributes between 2.5 and 3 million acre-feet of Colorado River water annually through the All American Canal primarily for gravity irrigation of nearly 500,000 acres of farm land in the Imperial Valley in southeastern California. The Imperial irrigation distribution system consists of approximately 1,600 miles of main and lateral canals, of which over 1,100 miles are concrete lined, and some 1,400 miles of open drains that carry primarily agricultural runoff to the Salton Sea. Metropolitan distributes between 1.6 and 2.5 million acre-feet of Colorado River water and State Water Project water annually to a service area of over 5,100 square miles in six Southern California counties in which over 16 million people reside. Faced with the possibility of water supply shortages in its service area in the future, Metropolitan has been aggressively pursuing various programs aimed at

[1] Principal Engineer, Metropolitan Water District of Southern California, 107 So. 5th Street, Suite 200, El Centro, California 92243

improving the adequacy and reliability of its water supplies. Such programs include the Imperial/Metropolitan Water Conservation Program, Palo Verde Irrigation District/ Metropolitan Test Land Fallowing Program, and water banking/exchange programs with the Coachella Valley Water District, Desert Water Agency, Semitropic Water Storage District, and Arvin-Edison Water Storage District.

The landmark Agreement between Imperial and Metropolitan became effective December 1989 and provided for the implementation by Imperial of 17 projects, two augmentation projects constructed by IID and 15 new projects, estimated to conserve 106,110 acre-feet of water annually upon completion of construction and placing into operation the last project. Metropolitan has funded all the costs of 15 projects of the Program and in return will have available additional water from the Colorado River for diversion through its Colorado River Aqueduct. Figure 1 shows the service areas of Imperial and Metropolitan, the Colorado River, and the Metropolitan Colorado River Aqueduct.

Figure 1



IID & MWD Service Areas

PROGRAM ORGANIZATION

As a means of managing the Program to provide prompt and orderly review and approval of budgeting, planning, design, and construction activities the Agreement called for the establishment of a Program Coordinating Committee (PCC) consisting of three professional engineers competent and experienced in the agricultural and civil engineering fields. The PCC is composed of one representative from Imperial, one representative from Metropolitan, and one representative selected by both parties to the Agreement.

Additionally, to oversee and direct the activities to verify the quantity of water conserved by the individual projects as well as for the total Program, a Water Conservation Measurement Committee (WCMC) was established. The WCMC is composed of the three PCC members plus one representative each from the Coachella Valley Water District (CVWD) and the Palo Verde Irrigation District (PVID). CVWD and PVID hold intervening priorities to use of Colorado River water in California, hence their interest in verifying the amount of water conserved by the Program. The WCMC is assisted in carrying out its responsibilities by the Conservation Verification Consultants (CVC) consisting of three consultants specialized in water resources engineering.

The primary budgeting, planning, design, and construction activities were carried out by the Imperial staff supported, as required, by consultants and contractors. The on-going operation and maintenance activities, for the next 35 years, will be conducted and managed by Imperial staff and, when required, with consultant support.

IRRIGATION WATER MANAGEMENT

Originally the Program's projects targeted operational spill as the primary water to be conserved. However, as monitoring equipment was installed throughout the district, analysis of the considerable data gathered established the baseline operational spills to be captured and also provided detailed insight into the interaction among the various projects to include substantial water savings from improved irrigation water management at the farm level. While certain on-farm water savings resulted from providing the farmers improved tools such as 12-hour deliveries (ordering water for a 12 hour period) versus the normal 24-hour deliveries and tailwater return systems it became evident that additional water savings resulted due to the availability of other system facilities such as reservoirs, lateral interceptors, and system automation. In other words, by having improved system facilities the potential on-farm savings were maximized and these facilities afforded the farmers greater flexibility in managing the water ordered resulting in more effective application of water to the crop. Prior to describing this project interaction which results in improved irrigation water management a brief description of each of the projects involved follows:

12-Hour Delivery--This project permits the farmer to order water for a 12-hour period rather than the standard 24-hour period. This allows the irrigation application rate to more closely match the rate required by the soil and crop rather than the less flexible 24-hour basis which, even if the irrigation event was completed, had to run the total 24-hour period, resulting in substantially more tailwater runoff discharged to the drains. This project was made available, through the Program, to the farmers in February 1990.

Reservoirs--One regulating reservoir, the Galleano (425 acre-feet (AF)), was constructed to capture the operational spill occurring at the "Z" spill at the end of the East Highline Canal. Additionally, improvements (construction of a pumping plant in 1998 to allow stored water to be discharged to the East Highline Canal) to the existing IID constructed Singh Reservoir (enhancing the 12-Hour Delivery and System Automation projects water savings opportunities) will make it a fully regulating reservoir of 323 AF capacity.

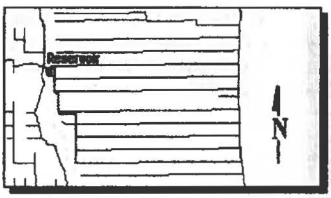
Lateral Interceptors--Three lateral interceptors-the Plum-Oasis, Mulberry-D, and Trifolium-were constructed including the Bevins Reservoir (253 AF); Young (275 AF) and Russell (200 AF) reservoirs, and Willey Reservoir (300 AF) and a pipeline respectively. A lateral interceptor consists of an open concrete lined canal which collects and transports operational discharge and farm delivery water which remains in the distribution system when a turnout is closed (returned water) from the ends of several laterals to a storage reservoir for use in another part of the distribution system. (See Figures 2, 3, & 4) All of the reservoir facilities were automated and the flow from the intercepted laterals controlled by automated drop-leaf gates. The three lateral interceptor projects cover a service area of some 83,436 acres, approximately 18 percent of Imperial's service area covered by its distribution system.

System Automation--In addition to the automation of the five lateral interceptor reservoirs and the Singh Reservoir some 57 major and minor main canal flow control structures were automated either by modernizing existing facilities or installation of new automation equipment at existing sites including upgrading the existing communication system. Major sites include complete communications, monitoring and control facilities such as equipment building, generator, a Programmable Logic Controller (PLC) while the minor sites do not have the building or generator. Certain minor sites included the installation of automated drop-leaf gates at 13 Westside Main Canal and 9 East Highline Canal sites. This extensive system automation project including a new Water Control Center (WCC) provides for better overall system control, more water user flexibility, and improved water delivery.

Throughout the implementation of the Program a considerable number of automated sites (currently over 100 sites), some of which were installed to monitor

Figure 2

- **Plum/Oasis Lateral Interceptor**
  - Lateral interceptor canal intercepts 8 laterals.
  - Service area = 24,000 acres
  - Operational in 1992.
  - Area (Bevins Reservoir) = 37 acres.
  - Capacity (Bevins Reservoir) = 253 acre feet.
  - Gravity Inlet with a pump outlet into the Redwood Canal system



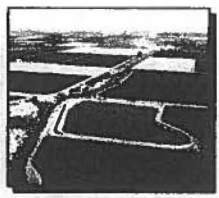
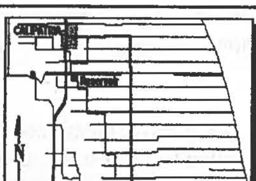
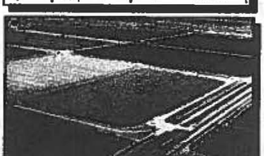


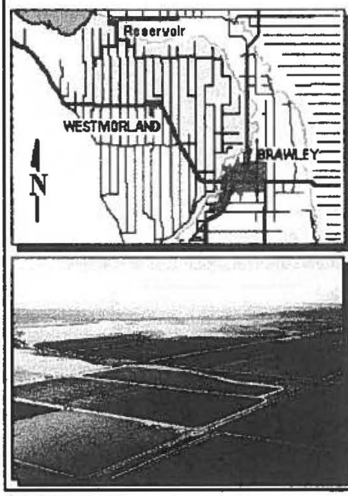
Figure 3





- **Mulberry - "D" Interceptor**
  - Lateral interceptor canal intercepts 11 laterals.
  - Service area = 31,000 acres
- **Area:**
  - Young Reservoir = 47 acres.
  - Russell Reservoir = 29 acres.
- **Capacity:**
  - Young Reservoir = 275 acre feet
  - Russell Reservoir = 200 acre feet
- Gravity flow on inlet and outlet for Young Reservoir.
- Gravity flow on inlet and pumps on outlet for Russell Reservoir

Figure 4



- **Trifolium Interceptor**
  - Lateral interceptor canal intercepts 15 laterals.
  - Service area = 30,190 acres
  - Operational in December, 1997.
  - Reservoir Area = 61 acres.
  - Reservoir Capacity = 300 acre feet.
  - Gravity flow on inlet and pumps on outlet with 21,900 feet of 45" diameter pipeline.

main/lateral canal and drain flows, resulted in a substantial amount of data being gathered. Most of this data gathering will continue for the 35-year period. This data was analyzed and the resulting information used in the planning, design, and verification activities of the Program. This permitted the PCC to take maximum advantage of all the overlapping operational opportunities resulting in improved water conservation and cost savings to the Program.

To illustrate this overlapping operational functionality of the above mentioned projects an example using the 12-Hour Delivery Project will be presented. As a 12-Hour water delivery is being shut off, which is affording greater water management flexibility and improved water use effectiveness to the farmer, the backing out of this water into the distribution system can cause operational difficulties for Imperial and result in spillage. IID orders water from the USBR four days in advance based on past year's usage, current weather conditions, crops being grown in the valley, and current level of farmer's orders (both 12-Hour and 24-Hour orders). Hence, once the water is released by USBR into the Colorado River any returned water that cannot be stored or used at another location must be spilled. However, Imperial now has a greater array of management tools at its disposal and therefore an increased number of options to manage (in many cases directly from the WCC) the flows backed out in the system as follows:

A. If the water delivery is shut off within an interceptor system service area Imperial can:

1. Have the Zanjero (ditch rider) make a gate adjustment to accommodate the flow if the flow can be used at another turnout along the lateral,

2. If action 1 is not possible the flow can be conveyed to the interceptor canal and transported, for storage, to the interceptor reservoir for later use.

3. Depending on the location of the returned flow(s) along the lateral an adjustment to the lateral heading gate (called a shut down) can be made by the Hydrogapher (main canal operator) effectively "backing the water out" into the main canal for use in another part of the distribution system or transported to a reservoir to be stored for later use.

B. For service areas outside an interceptor system Imperial can:

1. Carry out gate adjustments as stated in A, 1 above.

2. Absent the ability to implement action B, 1 the Zanjero must manually reduce the flows along the lateral, by adjusting the lateral check gates, as the Hydrographer shuts down the lateral heading gate to "back the water out" into the main canal. Once in the main canal system via the upgraded and new automated facilities this water can be transported to another part of the distribution system for immediate use or to a regulating reservoir, such as the Singh and/or Galleano, for storage and use later or to other lateral interceptor reservoirs for use in those respective systems.

VERIFICATION

It was and remains critical that all Program conserved water, including that resulting from improved on-farm water management, be verified as having been conserved. As an integral part of the Program a verification process was developed and put into place to identify consequential effects for each project and the most accurate method for establishing the volume of conserved water. This process becomes even more critical given the fact that IID deliveries have increased since the implementation of the Program and has verified that water is being conserved even with IID's increased deliveries. A very important element of all this verification effort was the gathering, archiving, and analysis of accurate data. The details of this process have been detailed in other papers and presentations at this conference and won't be repeated in this paper. It was essential, from the start, that a set of Conservation Verification Principles and Guidelines be established for use as a guide in developing the process and methods which have been used to establish the conserved water volumes for each project. The final result has been the documentation of the conserved water verification for each project through a Verification Summary Report plus detailed documentation (Annexes, etc.). This will institutionalize the verification process, for each project, for the Agreement term of 35 years. While the process and methods can be modified with the acquisition of new data and the resulting analysis of such, this documentation will set the stage for the Program's conserved water estimates in the future.

During the course of developing the verification strategies and processes it became evident that verification activities, for any water conservation program, should be one of the first activities initiated. It is important to gather pre-project data to establish a baseline of water use against which use after implementation of a conservation program can be compared. Additionally, this will permit validation checks to be developed which will support and assist in establishing verification processes for the long haul as well as for the immediate planning and design of specific projects. It is important to be flexible and make adjustments as field data dictates. An example would be the planning and design of the lateral interceptors. As additional field data was gathered on lateral spills for each of the interceptor projects, subsequent to the construction of the first project, the Plum-Oasis Lateral Interceptor, analysis of such indicated that the size of the interceptor lateral canal could be reduced. This revised design of the canal capacity provided for handling of all potential flows plus reduced the capital costs of the subsequent two lateral interceptor projects.

Another aspect of the verification program was the establishment of the Systemwide Monitoring (SWM) program. It is inevitable that other water conservation programs will be carried out in the Imperial Valley in the future. Based on our knowledge that any conservation project can have a negative, positive, or neutral impact on another project it was important to be able to monitor the overall Imperial distribution system. The SWM program will allow the WCMC to monitor trends, changes, etc. in the overall distribution system, both physically and operationally, to alert them to review certain projects or areas in the system for potential and/or actual changes that may affect the Imperial/Metropolitan Program conserved water volume. Adjustments can then be made, if required, to the verification process and/or the conserved water volume.

CURRENT STATUS OF THE PROGRAM

As of December 31, 1997, with the exception of pumping plant construction at the Singh Reservoir, all of the major construction work implemented under the Program has been completed. It is expected that the Singh improvement work will be completed in 1998. For the calendar year 1998 the estimated volume of conserved water is 107,160 acre-feet which is available for use by Metropolitan. It is important to note that of this total approximately 52 per cent of the volume has the verification process and analysis procedures finalized with the balance of 48 per cent being of a provisional status but expected to be finalized during 1998.

Table 1 shows the amount of water conserved each year and Table 2 shows the volume of water conserved by each of the projects of the Program.

Table 1
Total Water Conserved

	Water Conserved (Acre-Feet)	Water Available for Diversion by MWD (Acre-Feet)
1989 (Augmentation)	6,110	-
1990	20,590	6,110
1991	7,229	26,700
1992	20,901	33,929
1993	18,040	54,830
1994	1,700	72,870
1995	16,310	74,570
1996	6,860	90,880
1997	9,800	97,740
1998	9,420	107,160

Table 2
Project Conservation Summary

Projects	Water Conserved (Acre Feet)
• Reservoirs	9,700
• Concrete Lining	26,060
• 12 Hour Delivery	22,290
• Irrigation Water Management	5,180
• Non-Leak Gates	630
• System Automation	13,490
• Lateral Interceptors	29,810
Total	107,610

Through December 31, 1997 a total of $110,142,125 in capital expenditures have been made which, in 1988 dollars, is estimated to be $94,828,297. O&M costs totaled $24,111,142 over an eight-year period and the one time indirect costs totaled $23,000,000. This has resulted in a total actual cost of $157 million. a portion of which has been paid from interest earned on funds advance to Imperial by Metropolitan. Table 3 provides a cost breakdown by year.

Table 3
Total Expenditures

	Capital	Annual Direct	Indirect	Total
1990 (Actual)	$ 15,225,804	$ 980,514	$ 4,600,000	$ 20,806,319
1991 (Actual)	$ 26,879,778	$ 1,822,537	$ 4,600,000	$ 33,302,315
1992 (Actual)	$ 18,847,594	$ 2,522,125	$ 4,600,000	$ 25,969,719
1993 (Actual)	$ 17,219,098	$ 2,634,128	$ 4,600,000	$ 24,453,225
1994 (Actual)	$ 7,489,396	$ 4,077,599	$ 4,600,000	$ 16,166,995
1995 (Actual)	$ 7,731,874	$ 3,505,880	$ -	$ 11,237,754
1996 (Actual)	$ 6,725,418	$ 4,184,736	$ -	$ 10,910,154
1997 (Actual)	$ 10,023,163	$ 4,383,624	$ -	$ 14,406,787
Total	$ 110,142,125	$ 24,111,142	$23,000,000	$ 157,253,267

Based on the costs, to be paid in 1998 for work performed in 1997, on the Trifolium Lateral Interceptor plus the Singh Reservoir improvements being constructed in 1998, we expect the total capital expenditures to come in under budget, when measured in 1988 dollars. Upon completion of the Singh improvements the Program will enter a total operations and maintenance phase for the next 35 years.

SUMMARY

It has taken some eight plus years to bring this landmark Program to a successful conclusion. The Program organization managed through the PCC along with the WCMC's verification work has functioned exceedingly well being very effective in responding to the Program's technical requirements as well as budgetary needs and constraints. Even with the delay, caused by the Regional Water Quality Control Board, Colorado River Basin Region requirement to prepare an Environmental

Impact Report in 1993 and 1994, the Program anticipates completing construction under the original capital estimate of $97,758,000 in 1988 dollars.

One of the major successes of the Program has been the overlapping functionality of various projects such as 12-Hour Delivery, Reservoirs, Lateral Interceptors, and System Automation which has resulted in substantial water savings from improved irrigation water management on-farm as well as within the distribution system. This has afforded the farmers greater flexibility in managing the water they order which translates into more effective application of the water to the crop and for Imperial's more efficient transporting and delivery of water to the farm turnout. The verification process and procedures have played a major role in shaping the planning, design, and operation of the projects.

As a final but important note, the success of this Program must be attributed to a dedicated, professional, and hard working Imperial/Metropolitan team effort. With all of the posturing that exists and negotiating that takes place between agricultural and urban areas with respect to further conservation agreements this success says much and hopefully can be built upon in the future.

MCCLUSKY CANAL IMPROVEMENTS

Jerry Schaack[1] Warren Jamison[2]

ABSTRACT

The McClusky Canal is a 74-mile (119 kilometers) long channel. It was constructed from 1969 to 1976 for transporting water from the Missouri River Basin to the Red River Basin of the north, which is in the Hudson Bay Drainage Basin. The canal is one of the main features of the Garrison Diversion Unit (GDU), which was authorized by the Flood Control Act of 1944, or more commonly called the Pick-Sloan Act. The McClusky Canal was designed with a capacity of about 2,000 cubic feet per second (56.6 cubic meters per second) to provide water for the Garrison Diversion Project to irrigate 250,000 acres (100,000 hectares) and other purposes in the state of North Dakota in north central United States. The primary water supply for North Dakota is the Missouri River, therefore, water must be transported into the Red River basin to fully develop the water resource in that area.

To transport water by gravity from the regulating reservoir (Lake Audubon) across the continental divide, it was necessary for the McClusky Canal to follow a meandering course and, at times, through over 100 foot (32.7 meters) deep cuts. Some design and construction deficiencies were also not rectified, and the Garrison Diversion project has never been completed nor operated to near its capacity or maintained properly, except during the past five years. Recent efforts to introduce legislation for project completion have renewed the need for rehabilitation and proper maintenance of the canal.

The conditions mentioned above have contributed significantly to a general deterioration of the canal and have necessitated the need for major improvements and an upgraded O&M program. Some of the major problems which are being worked on are summarized briefly below:

- Several miles have cuts as deep as 50 feet (16.4 meters) and one 2½ mile (4 kilometers) length has an average cut of 110 feet (36.1 meters). These factors, along with high ground water conditions

---

1

District Engineer, Garrison Diversion Conservancy District, PO Box 140, Carrington, North Dakota

2

Manager, Garrison Diversion Conservancy District, PO Box 140, Carrington, North Dakota

and poor surface drainage, have contributed largely to severe sliding in some portions of the canal.

- Much of the canal was not constructed with adequate side slope protection, resulting in severe erosion of some of the canal banks particularly at bends and areas susceptible to wind erosion (wind frequency and velocity is high in North Dakota).

- Inadequate drainage of the upper berm slopes and on the O&M roads have resulted in erosion, puddling of water, and deterioration and of the O&M roads.

- The O&M problems are significant: 10,000 acres (4,000 hectares) of right-of-way, 150 (240 kilometers) miles each of fence and O&M roads, five recreational lakes, numerous fish and wildlife areas and public access areas to O&M.

This paper will describe and discuss the improvements and O&M which has been conducted during the past five years to upgrade this canal to satisfactory operating conditions. The uniqueness of this canal (very deep cuts, multiple uses, design deficiencies, inactivity, minimum O&M, and general deterioration) has required innovative and unique measures not normally needed for canal improvements and O&M.

INTRODUCTION

The Garrison Diversion Project was originally authorized under the Flood Control Act of 1944 and planned to irrigate one million acres in the state of North Dakota; this was scaled back to 250,000 acres (100,000 hectares) in 1964 and to 130,000 acres (50,000 hectares) in 1986. Legislation was introduced in 1997, which will further reduce the proposed irrigation area by approximately 50 percent and change the emphasis of the project to municipal, rural, and industrial water supply. Figure 1 shows the McClusky Canal and surrounding area.

The long delay in the completion of the project and changed emphasis has caused significant operation & maintenance (O&M), and improvement problems. The McClusky Canal was designed with a capacity of about 2,000 cubic feet per second (56.6 cubic meters per second) to primarily provide irrigation water to 250,000 acres (100,000 hectares) in North Dakota located in north central United States. In addition, right-of-way for the canal was purchased for future expansion of the project to one million acres (400,000 hectares), further complicating the O&M issues.

The McClusky Canal is one of the main features of the GDU designed and constructed to transport water from the Missouri River Basin to the Red River Basin. To transport water from Lake Audubon (regulating reservoir) across the continental divide by gravity, it was necessary for the canal to follow a

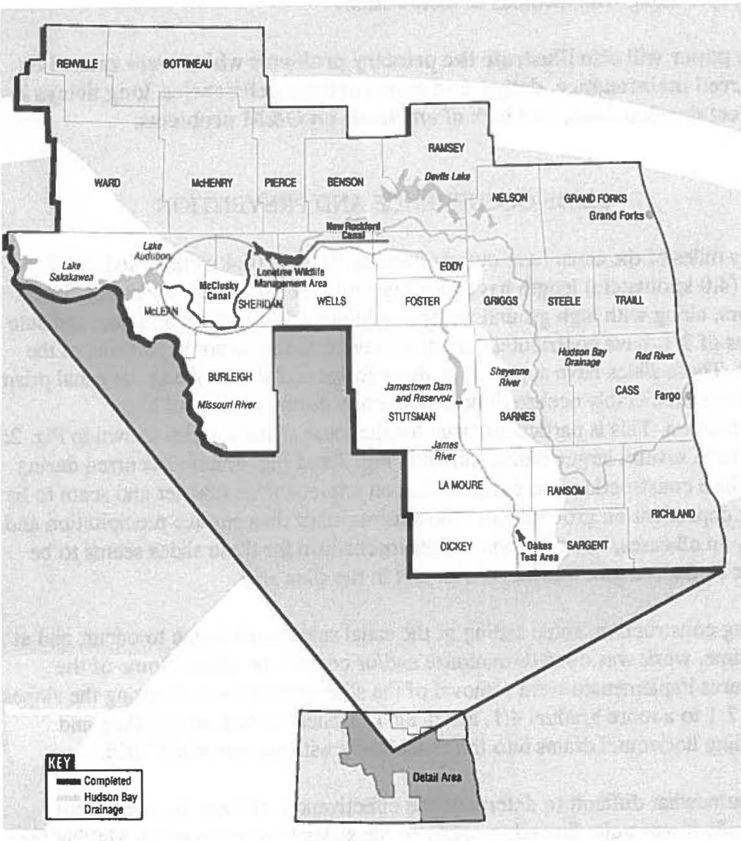


Fig. 1. An overall view of the McClusky Canal and surrounding area.

meandering course and at times through cuts in excess of 100 feet (32.8 meters). In addition, some design and construction deficiencies were not rectified or did not properly address the actual field conditions. The canal has not been operated as intended, and except for the last five years, it has not been maintained adequately. The Garrison Diversion Conservancy District assumed O&M of the McClusky Canal and other project features in 1992 under a cooperative agreement with the U.S. Bureau of Reclamation. Under this agreement, Reclamation funds the O&M of the canal and other facilities.

Recent efforts to pass legislation for project completion have reemphasized the need for initiating canal improvements and proper maintenance in anticipation of future canal operational needs. This improvement work and renewed O&M activity is being done at the present time and will be discussed in this paper along with methods of rectification.

This paper will also illustrate the primary problems which were caused by deferred maintenance, design and constructions deficiencies, long delays in project development, and lack of emphasis on O&M problems.

SLIDE OCCURRENCE AND PREVENTION

Many miles of the canal have cuts as deep as 50 feet (16.4 meters) and one 2 ½-mile (4.0 kilometers) length has an average cut of 110 feet (36.1 meters). These factors, along with high ground water conditions, poor surface drainage and side slopes of 2:1, have contributed largely to severe sliding in some portions of the canal. These slides have appeared in many forms and shapes along the canal prism and have predictably occurred more frequently during periods of high precipitation. This is particularly true for the more shallow slides shown in Fig. 2. The more severe, larger slides, shown in Fig. 3 and Fig. 4, have occurred during and since construction was completed in an unpredictable manner and seem to be more dependent on ground water movement rather than surface precipitation and flow. In all cases, however, the trigger mechanism for these slides seems to be either surface or groundwater movement in the slide area.

During construction, some sliding of the canal side slopes began to occur, and at that time, work was done to minimize and/or correct the slides. Some of the measures implemented were removal of the slide material and changing the slopes from 2:1 to a more gradual 4:1, installing T "french" drains, and drilling and installing horizontal drains into the canal banks with an "aardvark" drill.

It is somewhat difficult to determine the effectiveness of these measures, but generally it was quite limited, as many of the slides have reoccurred. Making the slope less steep helps to a limited degree; however, sliding does not appear to be as dependent on slope as it is on ground water movement in the area. In areas where ground water surfaces on the canal banks, it is likely that sliding will occur regardless of other conditions. The soils through which the canal is constructed are a glaciated nonhomogeneous-type, which is generally slowly permeable but can contain pockets of sand and gravel which transmit water readily. These soils often become unstable when wet.

Fig. 2. The two slides in the center and right side of the picture are examples of shallow slides which have occurred on the McClusky Canal.

The installation of T "french" drains (trench filled with permeable material and/or pipe drain) generally helped in slide reduction and prevention when they were properly located and intercepted water flowing onto the surface of the side slopes; however, they are quite costly, and it is sometimes difficult to locate these drains properly for effective performance. This is a viable method of slide prevention and specific applications of this method, which were effective, will be discussed later in this section.

The installation of horizontal drains drilled into the canal bank at a 90 degree angle to the longitudinal axis using the "aardvark" drilling machine resulted in very limited slide prevention. It appears that the area of influence from these drains is very small since water is moving downslope parallel to drains; thus, resulting in very little interception of the water.

Slide prevention methods during the past four years on the McClusky Canal have been intensified and quite successful. The primary methods implemented during this time are the removal of the slide material to unload the slope, along with installing a gravel "french" drain or conventional plastic pipe drain with a gravel envelope. These methods have been generally successful. In cases where flowing water is not apparent, drains are installed anyway based on observed conditions in case water movement occurs in the future.

Fig. 3. This is an example of some of the severe deep slides which have occurred on the McClusky Canal (note movement of pipe at mid right of picture)

Figure 4 shows the area where a slide occurred at a tunnel outlet and threatened a railroad track and state highway above. This was the first slide improvement completed by the District, and it proved to be successful, as the sliding has completely stopped. The procedure used was to remove some of the slide material and dig three trenches up the slope as shown in Fig. 4 to be used primarily as outlet drains to the perimeter interceptor drain also shown in Fig. 4. A corrugated plastic pipe was installed in the perimeter and outlet drains and they were filled to the surface with a gravel envelope material. As mentioned above, this method has proven to be quite successful, especially when the flowing water is evident, which makes it easier to locate the drains to achieve maximum effectiveness.

Similar work has been completed in other areas of the canal, which included unloading the slopes and installing a T "french" drain with gravel envelope.

Fig. 4. Completed french drain work on a slide which jeopardized a state highway and railroad track above the slide.

Another successful method used has been the placement of a conduit in the channel where sliding has occurred. An illustration of this work is shown in Fig. 5 displaying installation and Fig. 6 showing preparation for seeding with grass. The channel, in this case, was a 40 cfs outlet channel where sliding became apparent in 1994. The first method tried was to unload the canal banks of the slide material and change the slope to about 4:1 without installation of drains; the slide reoccurred in about two weeks. Since the slide was encroaching on the channel right-of-way, it was decided to install a 3' by 5' (0.97 meter by 1.61 meter) box culvert to eliminate the slide problem over a length of about 450 feet (145.2 meters). This work was quite costly (U.S. $250,000); however, it has been successful and will not likely cause future problems. This method may be cost prohibitive in a large, long channel; however, it should be considered for shorter, smaller channels.

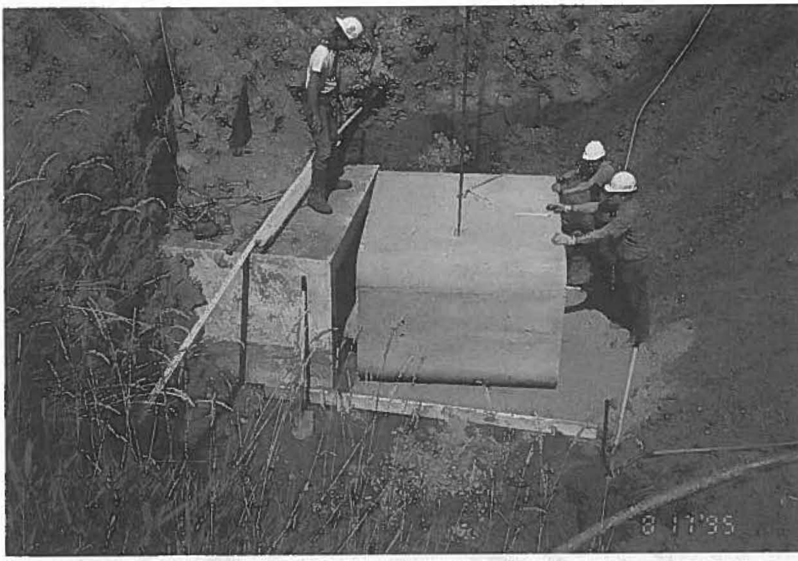


Fig. 5. Placement of box culvert to repair slide area where right-of-way width was limited.

V DRAIN IMPROVEMENTS

Shallow, open V drains were dug at the intersection of the berm side slope and the outside of the O&M road during construction. The purpose of these drains was to provide drainage for the O&M roads and for the berm areas. However, snow and water accumulates and freezes in these drains in the late fall and spring and thaws very slowly, making them ineffective when they are most needed. It should be noted that these conditions are unique to northern latitudes in the United States, which experience freezing temperatures during much of the October to March time period. Weed growth and sediment also collect in these drains, which accentuates the problem. This inadequate drainage has caused the O&M roads to deteriorate rapidly, making them unusable for long periods of time.

The purpose of this drainage improvement work is to protect and improve the O&M roads and make them usable on a more timely basis, especially during the spring and after heavy rains by providing drainage for the O&M roads and intercepting surface and subsurface water moving down the berm banks. This drainage problem is a design and construction deficiency which requires correction.

Fig. 6. An overall view of the completed work shown in Fig. 5. The area has been seeded and a mat installed to prevent erosion.

This deficiency is being corrected by the installation of a six-inch (15 cm) diameter slotted corrugated pipe tubing into the open drain, which is excavated to the desired grade at an average depth of about four feet (1.29 meters). A graded gravel envelope material, ranging in size from 0.75 inches (1.9 centimeters) to a 200 screen size, is placed in the trench so that the drain tubing is enveloped in at least four inches (10.2 centimeters) of the material. The graded envelope material is brought to about 1.5 feet (0.48 meters) above normal ground surface. The purpose of extending the gravel envelope above ground surface is to provide a catch basin for sediment accumulation and removal. Figure 7 illustrates the installation of these drains on the McClusky Canal.

This improvement program has been very effective in providing drainage for the O&M roads and intercepting and draining surface and groundwater from the berm area. Many of the O&M roads are now passable in a timely manner throughout much of the year, and the general condition of the roads and berm areas have improved dramatically, resulting in a significant savings of time and money. Prior to the installation of these drains, sloughing and sometimes total collapse of the O&M roads were experienced; this problem is now minimal.

Fig. 7. V drain installation at the outside of the O&M road. Note the drain tubing at the bottom center of the picture.

These drains are functioning properly; however, a program for drain evaluation is planned for this year to determine drain effectiveness and possible changes that should be made in drain construction and operational activities.

All of the drainage improvement work has been done with District forces and equipment, which is shown in Fig. 8; an average of about 600 feet (193.5 meters) of drain can be installed in an ten-hour day with a six-person crew. This drainage program has been in effect since 1992, and about 40 miles (64 kilometers) of the drains have been improved in the manner described above at about a cost of U.S. $50,000 per mile.

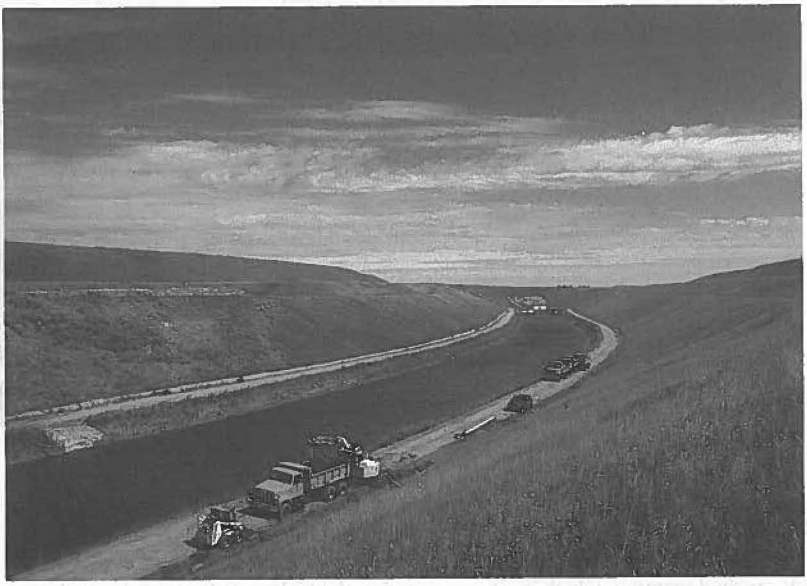


Fig. 8. A bird's eye view of the V drain installation showing the equipment used. The canal cut is about 110 feet, one of the deepest cut areas of the McClusky Canal.

CANAL SIDE SLOPE PROTECTION

Much of the McClusky Canal was constructed with inadequate side slope (bank) protection. This has resulted in severe erosion and undercutting of the canal banks, particularly at bends and areas susceptible to wind erosion, which is quite intensive in this area. This erosion has also caused concern for losing the integrity of the canal, particularly in some of the high fill sections. The canal is lined in some sections, and the erosion has encroached on the lining under certain conditions. Most of the McClusky Canal has been constructed in glacial till soils, which are quite variable and mixed and often susceptible to sloughing. These soils have accentuated the erosion problem and emphasized the need for timely correction. Figure 9 shows a typical example of erosion that has taken place on the unprotected canal banks.

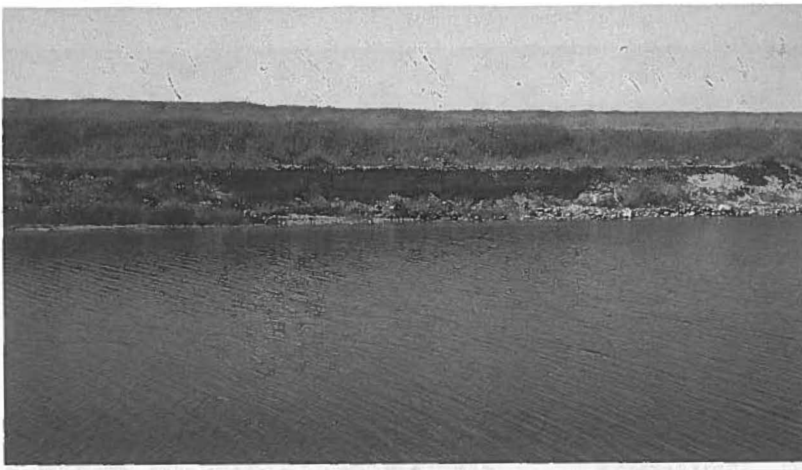


Fig. 9. A typical example of erosion and sloughing of the canal banks (side slopes).

The lack of canal bank protection is being corrected by placing a graded material (often referred to as beachbelting) on the side slopes of the canal. The gradation of this crushed material, which is normally used in the highly eroded areas and where the integrity of the canal may be in jeopardy, such as a high fill area, is given below.

Screen No.	% by weight passing
4 inch (10.16 cm)	100
1 ½ inch (3.81 cm)	40-60
3/4 inch (1.91 cm)	5-15
3/8 inch (0.95 cm)	less than .5

In the cut portions and other lengths of the canal which are less susceptible to erosion and loss of canal integrity, other types and gradations of beachbelting has been used. This material has varied from pit run gravel to a rounded-type material that is available on the project. These types of materials have been primarily used on a test basis; however, they appear to provide adequate bank protection at a much lower cost than the crushed graded material described above, which is relatively expensive.

The installation of the graded crushed material has been primarily done under contract, and the cost has varied from US $17-20 per linear foot (0.32 meters) of canal. A procedure for the installation of the beach belting material, which is quite effective and efficient has been developed. It consists of clearing the bank of vegetation and other material about ten feet between stipulated elevations and hauling in and placing selected material to obtain a smooth side slope. The bank is then watered and compacted with a roller attached to a backhoe. About a 12-inch (30.5 cm) keyway is cut into the bank at the bottom elevation and sloped upward to prevent slippage of the beachbelting material. A permeable geotextile material is then placed on the canal side slope to hold the soil in place. A front-end loader or conveyor attached to the rear end of a dump truck distributes the beachbelting material evenly on the geotextile and canal bank.

A limited amount of this work has been performed by District forces, and we anticipate more will be done in the future as most of the critical areas are complete. Depending on the results of the test sections, which to date look very satisfactory, it is anticipated that a significant amount of this work will be completed using pit run or a less restrictive graded material resulting in optimum utilization of District forces and equipment and cost savings.

Figures 10 and 11 illustrate the installation of the beachbelting material on the canal side slopes and Fig. 12 shows the completed work.

Fig. 10. Organic and other foreign material is removed followed by placement of material on the bank. The bank is then smoothed, compacted and a keyway (upper right) installed to prevent slippage of material.

Fig. 11. The beach belting material being placed on the geotextile on the canal bank.

Fig. 12. The beach belting placement is complete.

OPERATION AND MAINTENANCE ACTIVITIES

General

The operation and maintenance (O&M) activities on the McClusky Canal are significant: 10,000 acres (4000 hectares) of right-of-way, 150 miles (240 kilometers) each of fence and O&M roads, five recreational lakes, public access and use and numerous wildlife development and habitat areas. The problems normally anticipated in the O&M of an irrigation canal have been magnified by the factors stated above along with deferred maintenance, and design and construction deficiencies. The different aspects of the O&M program activities are described below.

The annual expenditures for normal O&M activities is approximately US $1 million, and the average cost of the canal improvement work is approximately US $1.4 million. This improvement work includes repairing of slides, installation of pipe drains, and beachbelting work as discussed above. At the peak of the season, about 20 personnel, who are stationed at the McClusky O&M Office, perform work on the canal. These workers are employed as permanent, permanent seasonal, and temporary employees at salaries commensurate with similar jobs in the regional area. Health, retirement, vacation and other benefits are also provided to all but the temporary workers.

Right-of-Way Operation and Maintenance (ROW)

The ROW for the canal is much larger than is presently needed because it was acquired in anticipation of expanding the project to about four times its original size. The ROW is managed for optimizing wildlife habitat, noxious weed control, recreational activities, and public use such as camping, hunting and hiking. The District has developed and implemented an integrated pest management (IPM) program to enhance the environment and minimize chemical usage for vegetative control. Some of the methods used include mowing the ROW on a five-year rotation, grazing, burning, fish and insects.

The mowing and grazing programs are done in cooperation with area farmers on a competitive bidding process. The Garrison Diversion Conservancy District is presently seeking approval for the use of grass carp in the canals for aquatic weed control, and insects have been released in certain areas for leafy spurge control.

The IPM program has been very effective from the standpoint of cost savings, environmental enhancement, safety and improved control and additional emphasis will be placed on this program in the future.

Operation and Maintenance Roads and Fence Repair

There are about 150 miles (240 kilometers) each of O&M roads and fence to maintain on the canal and ROW. The roads are bladed at least two times per year, depending on rainfall and usage. In the public access areas where use is high, they are sometimes maintained more frequently. All of the ROW was originally fenced; however, in areas where fencing is not needed, it is removed and permanent ROW markers constructed.

Recreational Lakes and Public Access

There are at least seven major lakes on or adjacent to the canal which are used for fishing, boating, swimming and other water recreational activities. The water levels and quality are maintained in these lakes in cooperation with fish and wildlife and park personnel to optimize all recreational activities. Minimum flows are also maintained in downstream creeks to enhance aquatic life and water quality and the environment. Most of these lakes are excellent fisheries and receive high usage throughout the year. Some portions of the canal are also fished quite heavily and hunters frequently use the O&M roads to gain access to their favorite hunting spots, which are often in the vicinity of the lakes and canal. Public use of the canal ROW is allowed, as long as long as it is reasonable and safe.

The McClusky Canal ROW has recently been designated as part of the North Country Trail, which is a non-motorized national trail extending through the northern United States from the state of New York through North Dakota.

Wildlife Areas

There are about 20 wildlife management and development areas along the McClusky Canal, and these are managed in cooperation with the North Dakota State Game and Fish Department and the U.S. Fish and Wildlife Service to obtain optimum benefits. An excellent example of cooperative and conjunctive benefits is the enhancement and development of Lakes Brekken and Holmes as recreational lakes in conjunction with the development of six wildlife areas and stream enhancement. Lakes Brekken and Holmes were initially saline lakes dependent on precipitation for their water supply. Water from the McClusky Canal is now used to improve their water quality and stabilize levels. The poorer quality water is drained and used for the development and management of six wildlife areas downstream. The water is ultimately released further downstream for stream enhancement before it flows into the Missouri River. A win-win situation.

SUMMARY AND CONCLUSIONS

This paper illustrates several problems and conditions which were encountered in the planning design, authorization, construction, operation and maintenance of a major water project. The history of the Garrison Diversion Project began in 1944, when the original authorization was passed and continues on today with the recent introduction of legislation, which will dramatically change the focus of the project. Most of the main supply works were constructed in the time frame of 1968-1976 for a 250,000 acre (100,000 hectare) transbasin irrigation project. The legislation introduced in 1997 would provide funding for a comprehensive water project providing municipal, rural and industrial water development, irrigation development of 70,000 acres (28,000 hectares), fish and wildlife, and other interests. When constructed, it will provide a badly needed affordable and reliable water supply to develop the water resources of North Dakota.

The major problems which have been encountered during the development and construction of this project are discussed in this paper and are summarized below.

<u>Design and Construction Deficiencies</u>: The design did not take into consideration the susceptibility of the canal side slopes and berms to major sliding. The cause(s) of slide occurrence during construction was not adequately addressed. The canal side slopes were also not properly protected, and drainage of the O&M roads and berms was inadequate for the conditions normally encountered. These conditions have caused major problems in the O&M and improvement work on this canal.

<u>Deferred Operation and Maintenance</u>: Much of the needed maintenance and improvement work on the canal was deferred for a relatively long period of time, resulting in significant deterioration of the canal due to erosion, general inattention, inadequate drainage, and sliding.

<u>Delays and Changing Objectives</u>: Due to the extremely long delays in final approval and funding, project objectives changed significantly. For example, the primary benefits changed from irrigation to a more comprehensive water resources project. This, in turn, dramatically reduced the water requirements for the project. Other project needs are also changing, which will likely result in changed development and operational requirements.

Approximately 550,000 acres (220,000 acres) of prime North Dakota farmland was inundated by Oahe and Garrison Reservoirs, which provide benefits to all Missouri River Basin states under development of the Pick-Sloan Act. North Dakota has received fewer benefits than most states and has given more in the form of lost farms and economic development. This is an irritant and constant frustration to project proponents.

Some of the problems encountered in the history of this project, such as design and construction deficiencies and deferred O&M were probably preventable: however, it is always easier to look back and be wiser. Hopefully, this history will provide a more efficient process for the future. However, the history of water project authorization, funding, and development is an arduous one and is sometimes inherent in our legislative and political process even though it is costly, frustrating, and time consuming. The GDU project has been no exception to this process.

IMPLEMENTATION OF A DISTRICT MANAGEMENT SYSTEM IN THE LOWER RIO GRANDE VALLEY OF TEXAS

Guy Fipps[1] Craig Pope[2]

ABSTRACT

The Lower Rio Grande Valley of Texas is undergoing rapid population growth and industrial development. No additional water rights are available in the lower Rio Grande River Basin, and future development will depend on water transfers from agriculture. The potential for saving water in irrigation districts is being studied as part of a regional water resources planning project. An Irrigation District Management System (DMS) is under development to aid in this analysis. The DMS is built upon GIS-based maps and databases for organizing and displaying district information on water accounts, fields, and distribution systems. Various other components are being linked to the DMS or are under development to enhance its capabilities, including a crop growth and irrigation scheduling model for determining water use under various water supply scenarios, and a routing model for determining the ability of the distribution systems to deliver the volumes of water needed for each scenario. The implementation of the DMS in the Valley and its use in regional water planning is described.

INTRODUCTION

The Lower Rio Grande Valley is a four-county area along the Mexican border located at the south-most tip of Texas (Fig. 1). While usually referred to as the "Valley," the area is actually a delta of the Rio Grand River. It is known for mild winters, excellent hunting and fishing, rare and endangered wildlife, the unique "Tex-Mex" culture of the border region, and South Padre Island. Manufacturing is rapidly expanding on both sides of the border, and the area is among the fastest growing regions in the U.S. and Mexico.

The Valley is also an intensively irrigated region. Just two counties account for the bulk of the region's 740,000 irrigated acres. An irrigated area of similar size

[1]Associate Professor and Extension Agricultural Engineer, Department of Agricultural Engineering, Texas A&M University, College Station, TX 77843-2117.

[2] Graduate Research Assistant, Department of Agricultural Engineering, Texas A&M University, College Station, TX 77843-2117.

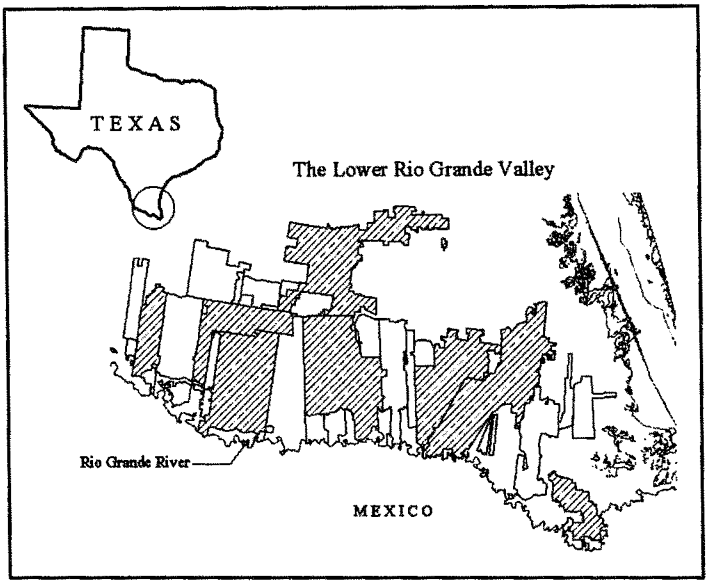


Figure 1. The 8 irrigation districts in the Lower Rio Grande Valley that have initiated GIS-based management systems.

is located just across the border in Mexico. Ninety-eight percent of all the water used in the border region is from the Rio Grande (called the Rio Bravo in Mexico). Cotton and sorghum account for the most acreage, but the semi-tropical climate of the region supports a wide range of crops including citrus, sugar cane, vegetables, aloe vera and other specialities.

Irrigation development began in the late 1800s by land development companies chartered by the state. Water conflicts among Texas growers, and between Texas and Mexico were common throughout the first half of the century. In the 1940s, treaties were signed between the U.S. and Mexico for the construction of two dams on the Rio Grande: Falcon and Amistad. Inflows into this reservoir system are divided between Texas and Mexico, with about 55 percent of inflows allocated to Texas and about 45 percent allocated to Mexico. The International Boundary and Water Commission was created to oversee and maintain the reservoir and river system.

Texas began to judicate water rights in the 1950s, a process that took to the mid-1960s to complete. Texas established the Rio Grande Watermaster to authorize water releases from the reservoirs according to account balances of water rights holders. A few growers along the river have water rights and pump their own water. Additionally, 28 irrigation districts hold the agricultural water rights, pump the water from the Rio Grande, and deliver it to individual farms and municipalities through gravity-flow canals and underground pipelines.

There is very little water in the Rio Grande River which makes its way past the El Paso area. Much of the inflow for the lower Rio Grande comes from the Rio Conchos, which intersects the Rio Grande just north of Presidio. Some additional recharge occurs from the Pecos River and a number of streams, most of which are in Mexico. Thus, water supply in the Valley depends on rainfall in watersheds hundred of miles away.

DISTRICT MANAGEMENT SYSTEM

The development of an irrigation district management system (DMS) at Texas A&M University began in 1992. The vision was to create a decision support system for scheduling, water management and conservation planning. The DMS would incorporate field-level analysis of water demand with management of the overall distribution system. The format and components of the DMS have evolved significantly, due to advances in software, availability of digitized information, and input from Texas' irrigation districts. The three main components of the DMS are the Visual System, IRRDESS, and Distribution System Routing.

Visual System

In Texas, irrigation districts are units of government with locally elected directors. State regulations govern the organization of districts, election of directors, and certain financial matters. Otherwise, districts set their own policies and procedures for allocation of water to individual growers. In most districts, property owners are assessed a "flat fee" each year, and growers are charged for each irrigation. To order water, growers are required to turn in a "water ticket" form which includes information on the field, water account number, tenant's or owner's name, crops planted, etc. Most districts enter this information into a computer database. The purpose of the visual system is to allow easy access, analysis, and display of this information.

The basic design and functions of the visual system were developed cooperatively with the Harlingen Irrigation District during the period 1995-1997. A small portion of the District was selected for analysis. An aerial photograph of this portion of the district was digitized and geo-referenced, that is latitude and longitude coordinates were provided for each pixel comprising the photograph. Thus, each point of the photograph has exact coordinates which allows for easy integration with other maps and databases. Next, the individual water account or field boundaries were drawn using the photograph as a guide. This was done with the software *ArcInfo* which runs on a Unix platform. In *ArcInfo* terminology, a "coverage" was created of the water account boundaries (Note: recent releases of *Windows*-based *ArcView* now support the drawing of maps with this software, and such maps are referred to as "themes").

This coverage (i.e., map of field boundaries) was linked to the district's database, allowing all information in the database to be displayed using *ArcView*. By clicking on a field, database information is shown in a list box. Information can also be displayed using shading or color coding. For example, Figure 2 shows the number of times each field was irrigated in 1997. Also visible in this figure are the underlying aerial photograph and the "polygons" comprising the boundaries of each water account. Next, the distribution system in this portion of the district was mapped. A database of all information available on the distribution system was developed and linked to this coverage which can be displayed by *ArcView* in a list box.

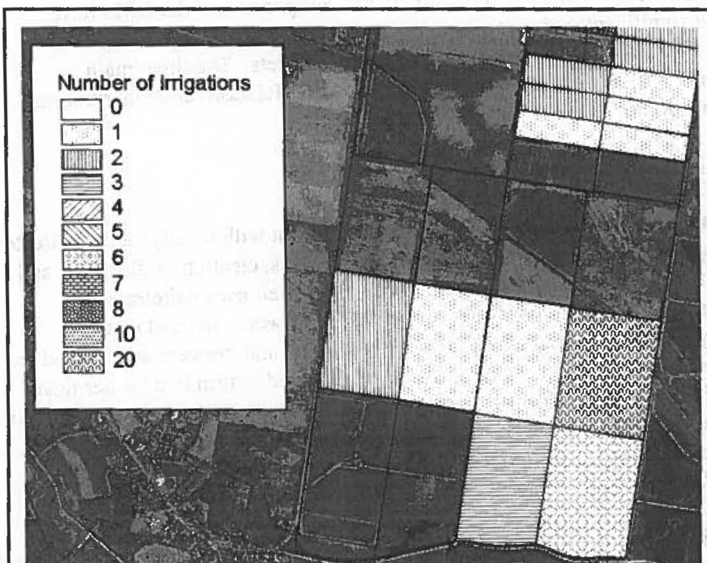


Figure 2. Water account boundaries drawn using an areal photograph as a guide and linked to the district's data base to display the number of irrigations for each account during 1997.

IRDDESS

IRDDESS (Irrigation District Decision Support System) is a crop growth and irrigation district simulation model developed at Texas A&M University during the period 1992-1995. As a crop growth model, IRDDESS is similar to the family of models "WOFOST" developed at the Center for World Food Studies, the Netherlands (Driessen and van Diepen, 1986; Driessen and Konijn, 1992). Only a brief description is given here; for more information see Endale (1995) and Endale and Fipps (1996).

IRDDESS allows for the simulation of different crops on individual fields within an irrigation district. Soil type and related properties, and irrigation method and scheduling can vary from field to field. Daily crop growth is simulated based on climate, water availability, planting date, etc. Various water supply scenarios can be considered such as supplying full crop requirement, following a pre-established schedule, and irrigating according to soil moisture levels or depletion. The model tracks water demand at each field and in the distribution system, which may consist of primary, secondary and tertiary canals. Once daily water demand is determined at the field level, IRRDESS sums the required flows in each segment of the distribution system and checks the demand against the capacity.

Distribution System Routing and Accounting

We found that the original routing module of IRDDESS was too simple to handle the complicated distribution systems of real districts, and the code too cumbersome for incorporating into a GIS format. A more sophisticated procedure is under development. For the current analysis, we programmed a distribution system routing algorithm directly in *ArcView* using the script language *Avenue*. Fields are linked to a specific gate or valve. The distribution network serving the field is created by linking from gate to gate, back to the main diversion point.

THE LOWER RIO GRANDE VALLEY INTEGRATED WATER RESOURCES PLAN PROJECT

The lower Rio Grande River is over appropriated; that is, there are more water rights than firm yield. At the first of each year, the Rio Grande Watermaster allocates the available water to rights holders (following state regulations), with municipal and industrial water rights having priority over agriculture. As discussed earlier, the region is undergoing rapid population growth and industrialization. The Texas Water Development Board (TWDB, 1997) projects that by the year 2010, municipal water demand will increase by 66% and industrial water use by 19%. By 2050, municipal demand is expected to increase 171% and industrial 48% over current usage (note: these numbers do not included expected water demand increases on the Mexican side of the border).

The Lower Rio Grande Integrated Water Resources Plan - Phase II Project (IWRP) is a 1-year intensive study of multiple proposals for providing the additional water which will be needed. These proposals include a municipal supply pipeline from Falcon, a channel dam at Brownsville, conservation programs, as well as water savings in agriculture which would lead to the transfer of water rights. The project is scheduled to be completed in November 1998.

Potential water savings in agriculture is being examined at all levels. At the farm level, these include water savings from metering, improvements in on-farm water management practices, conversion to sprinkler and drip irrigation, and changes in crop mix. At the district level, the analysis covers changes in irrigation district operation, management and infrastructure, which include lining canals, installing pipelines, sharing main canal systems between districts, and instituting various water pricing programs. IWRP includes technical feasibility and economic analyses.

For such water planning projects, a fully developed and implemented DMS could be used in several ways, including:

- allowing access to the districts' databases for analyzing past and current water use, trends in cropping patterns, and changes in district efficiency from past improvements;
- determining water demand under proposed irrigation technology, water management and crop mix scenarios;
- determining whether the existing distribution system can supply the volume of water needed under various scenarios; and
- analyzing the potential increases in conveyance losses if canals have to remain fully charged for longer periods to meet expected water demand.

IMPLEMENTATION OF DMS

We have taken a dual approach in implementing the DMS. We are working directly with eight districts (Fig. 1) to map their systems and interface the GIS-based maps with the districts' water accounting databases. We are also assembling a "Regional GIS" for analysis of the water saving potential from improvements in conveyance system efficiencies. In constructing the GIS maps of the districts, we are using DOQQs (digital orthographic quarter quads), which are obtainable from the USGS. We are using DOQQs with a scale of 1:12000 which provide a resolution of 1 m. These are aerial photographs taken in 1995, corrected for the earth's curvature, geo-referenced, and digitized.

To participate in the program, we required each district to provide a personal computer (equal to or faster than a Pentium II 233 MHZ, with 128 mb of RAM), purchase *ArcView* version 3.0, select or hire a person for GIS mapping, and obtain copies of the DOQQs of their districts. Two workshops were conducted to provide basic instruction on GIS mapping and theme construction in *ArcView*. Each district was instructed to begin by drawing their water accounts and/or field boundaries (Fig. 2). We choose to begin with water account boundaries in order to produce a useful tool that would immediately improve district management and bookkeeping.

Mapping distribution systems with *ArcView*, particularly canals, is relatively easy with the DOQQs as a guide, since all but very small canals (less than 1 m wide) are clearly visible. Even underground pipelines can be drawn using the stand pipes as a guide, which are also visible. However, assembling the attributes of the systems requires very detailed technical information as outlined in Table 1. Only two of the 24 major districts have complete sets of data assembled in a format that is easy to access. For the others, assembling this information will be a major task. One example is Cameron County ID#2 which had no technical information available on their canal system. The district had its canal riders assemble the data, which took approximately 3 weeks of full-time work by 7 individuals.

DMS IN WATER RESOURCES PLANNING

In the current phase of the regional water study, the DMS is being used primarily to develop maps and the attributes of the irrigation distribution systems. These resources will assist us in determining the potential water savings from lining and pipeline replacement of earthen canals, and from the elimination of canals expected due to urban growth and expansion. For example, Figure 3 shows the main distribution system for the Mercedes Irrigation District, the total extent of unlined canals, and the sizes (top widths) of the canals. Overlaying this information on a soils map will help identify unlined canals which maybe candidates for field reconnaissance and further analysis.

Figures 4 and 5 show the effects of urban growth and expansion on the irrigation distribution systems. The total urbanized area is overlaid onto the map of the main irrigation distribution systems. Large areas in the Western portion of the Valley will be covered up by urbanization, and a number of districts will effectively be separated into northen and southern portions. To date, although we have mapped the main distribution systems of all the districts (Figure 4), we only have complete information (canal sizes, etc.) on about half of the distribution system (Tables 2 and 3). Maps of the secondary distribution system and the database of attributes for both the primary and secondary systems must be completed before more detailed analysis can be conducted.

Table 1. Examples of the Type of Data Needed for the Distribution System in Creating a GIS-based District Management System.

Water Delivery Systems

- Segment ID
- Category (primary, secondary, tertiary)
- Segment Length
- Starting Elevation
- Ending Elevation

Additional Data for Canal Segments

- Canal Shape
- Top Width
- Bottom Width (where applicable)
- Side Slope (where applicable)
- Depth
- Lining
- Normal Operating Depth
- Normal Operating Capacity
- Maximum Capacity
- Elevation of Top of Canal in Relation to Ground Level
- Condition, Maintenance, etc.

Additional Data for Pipeline Segments

- Diameter
- Capacity
- Material
- Condition

Main Pumping Plant and Relift Pumps

- Pump ID
- Maximum Pump Capacity
- Normal Operating Capacity
- Condition, Technical Specifications, etc.

Location and dimensions of Structures

- Gates
- Siphons/Culverts
- Valves/Outlets

Table 2. Miles of Canals, Pipelines and Resacas of the Primary Irrigation Distribution Systems in the Lower Rio Grande Valley.

canals (miles)	pipelines (miles)	resacas (miles)	unknown (miles)	total (miles)
641.9	9.7	44.6	0.1	696.3

Table 3. Canal Sizes, Extent, and Lining Classification for the Primary Irrigation Distribution Systems in the Lower Rio Grande Valley.

Top Width (feet)	**Canal Type (or lining material) (miles)**	
	concrete	**earth**
< 10	41.6	1.0
10 - 20	98.0	11.9
20 - 30	25.2	52.2
30 - 40	3.8	35.1
40 - 50	1.1	60.1
50 - 75	1.4	30.9
75 - 100	0	11.1
> 100	0	9.7
Unknown Widths	99	134.5
Total Miles[1]	270.1	346.4

[1] no size or lining information is available on an additional 25.4 miles of canals

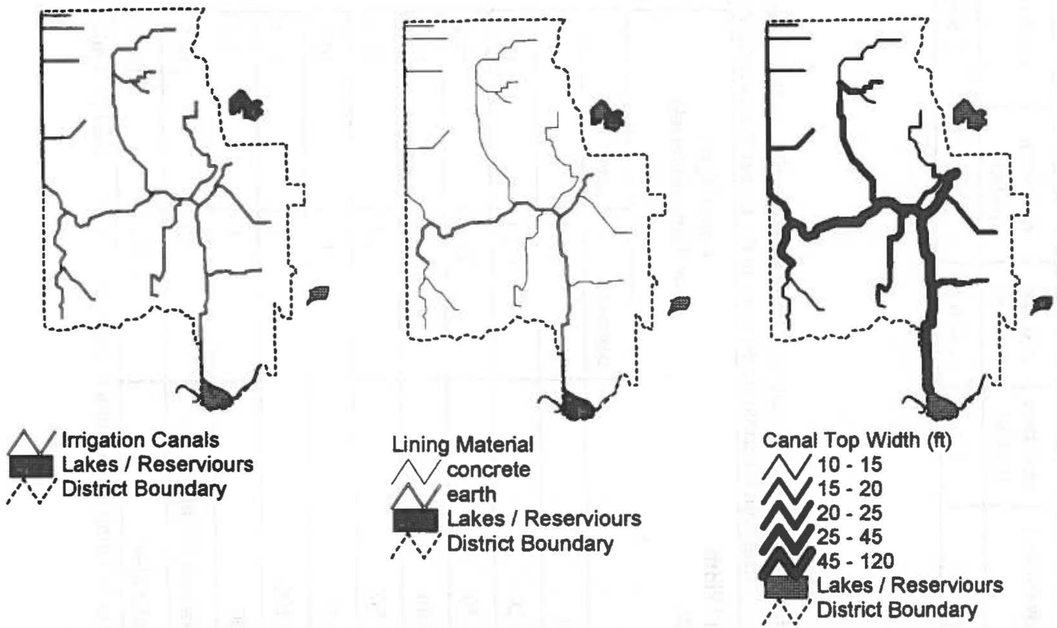


Figure 3. The main distribution system of the Mercedes Irrigation District and various ways of displaying its attributes for analysis, including the lining classification and canal size.

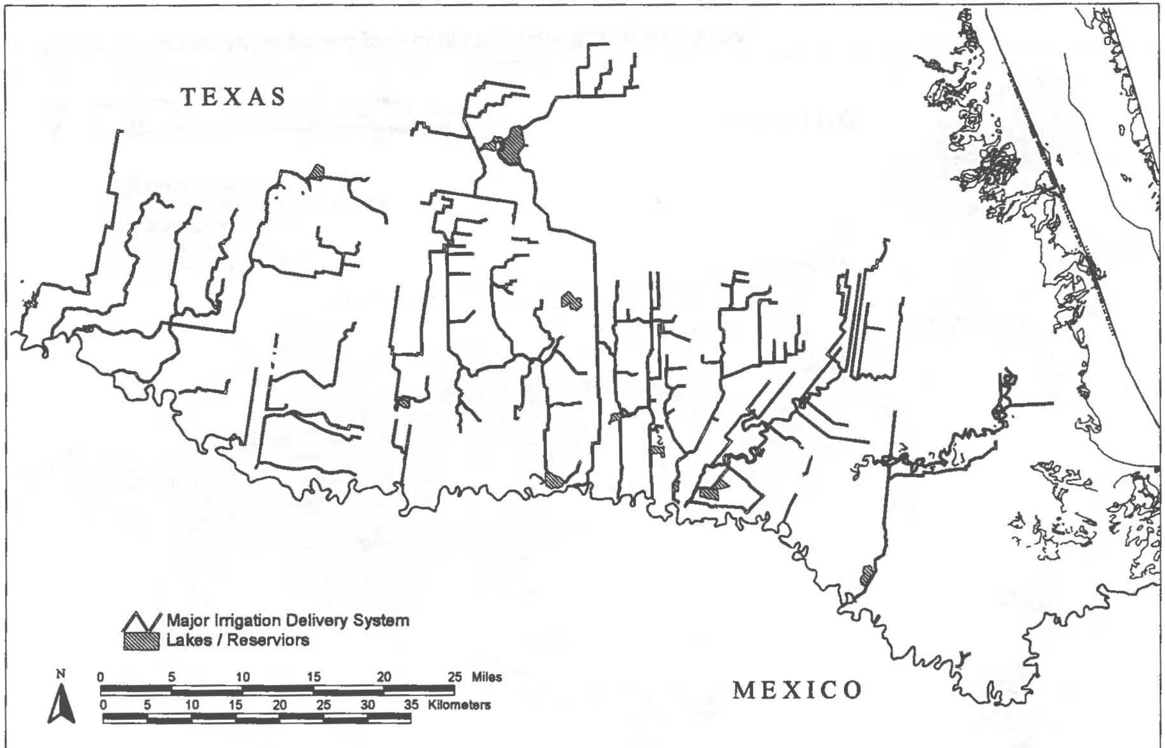


Figure 4. Main irrigation distribution systems in the Lower Rio Grande Valley.

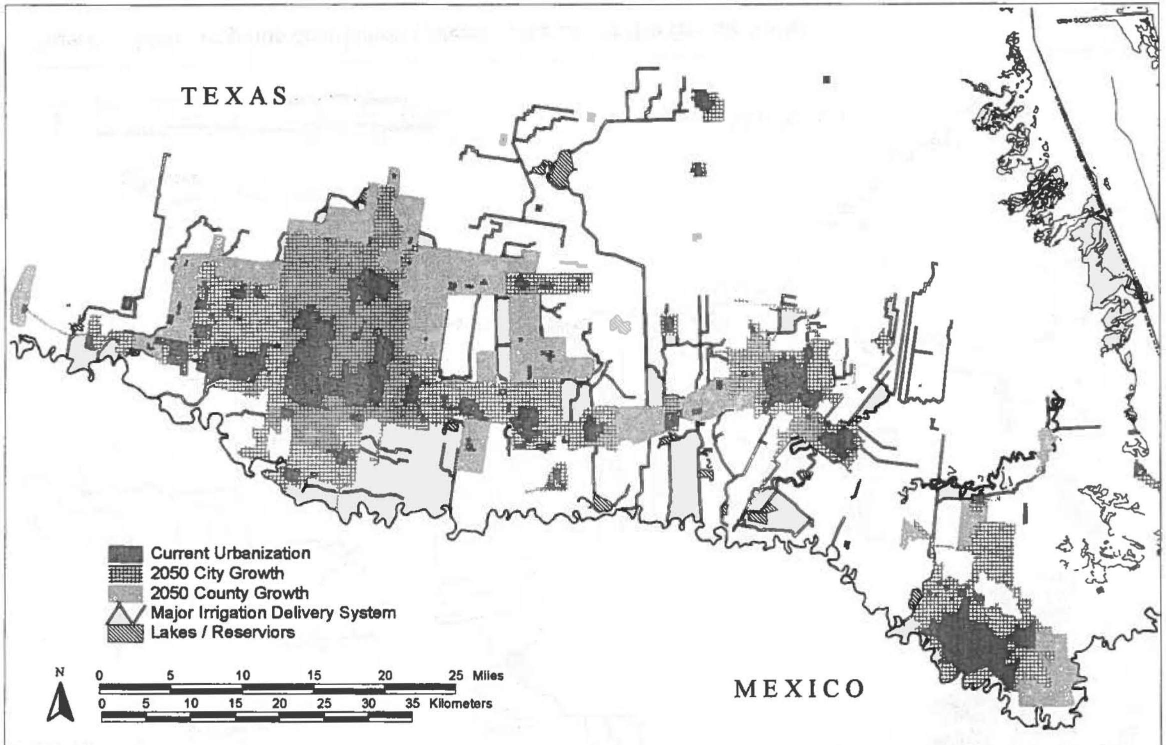


Figure 5. Current and expected growth in the Lower Rio Grande Valley.

REFERENCES

Driessen, P.M. and N.T. Konijn. 1992. Land Use System Analysis. Wageningen: Department of Soil Science and Geology, Wageningen University.

Driessen, P.M. and C.A. van Diepen. 1986. WOFOST, a procedure for estimating the production possibilities of land use systems. CABO, Wageningen, The Netherlands.

Endale, D.M. 1995. Irrigation district decision support system (IRDDESS). Ph.D. thesis, Texas A&M University, College Station, TX.

Endale, D.M and G. Fipps. 1996. Modeling Irrigation Strategies and Scheduling in Irrigation Districts. Proceedings of the International Conference on Evapotranspiration and Irrigation Schedule, November. American Society of Agricultural Engineers.

TWDB. 1997. Water for Texas. Texas Water Development Board, Austin, TX.

USE AND PRODUCTIVITY OF EGYPT'S NILE WATER

David J. Molden[1] Mona el Kady[2]
Zhongping Zhu[3]

ABSTRACT

Many irrigated areas worldwide are facing increasing competition from agricultural, municipal, industrial, environmental and other uses of water. In water basins, changes in water use in one area often affect how water is used in another area. It is therefore vital to understand how water resources are presently used, and how changes may affect future use of water. A water accounting methodology is presented to show the use and productivity of water. The methodology was applied to Egypt's Nile River system to evaluate the present status of water use and productivity. It was shown that there has been a trend of increasing consumption of water by agriculture and an increase in the productivity of water available to agriculture. There is little water remaining to be saved, and increases in productivity must focus on gains in productivity per unit of water consumed by evapotranspiration. The example from Egypt demonstrates the use and utility of the water accounting methodology in describing water use patterns by different sectors. It is envisaged that this methodology will be further developed to be useful in a wide range of situations.

INTRODUCTION

There is a need for irrigated agriculture to perform better to meet food production and food security needs of a growing population. In many situations water available for irrigation is decreasing because industrial, urban, and environmental uses require an increased share of water resources to meet their growing demands. We must become more productive with our water resources in a manner that can be sustained, and that meets equity, environmental, and other goals of society. A common objective within irrigated agriculture is to increase the productivity of water devoted to agriculture in light of increased competition from other sectors.

Decisions about water require a clear presentation and understanding of the present uses of water. Often times the situation of water use is quite complex due

[1] Research Leader, Performance and Impact Assessment Program, International Water Management Institute (IWMI), PO Box 2075, Colombo, Sri Lanka.
[2] Chairperson, National Water Research Center, Ministry of Public Works and Water Resources, P.O. Box 74, Fum Ismailiya Canal, Shoubra El-Kheima 1314, Cairo, Egypt.
[3] Advisor, Winrock International, Suite 600, 1611 North Kent Street, Arlington, VA 22209, USA.

to problems of scale, interactions between users, and use and re-use of water as it flows down a river basin. With knowledge of field level practices, it is difficult to upscale to basin-wide water use. Likewise, basin-wide information does not necessarily yield information on how water can be better used locally. With the many uses and interactions between water uses, a basin level understanding is required to better understand the effects of a change in water use. There is a need to clearly and simply present information on water use and its productivity, so the results of possible water-related actions can be understood.

To help to reveal information about water use at different levels of analysis, a water accounting methodology was developed (Molden, 1997) for evaluating water management within and among all sectors. This water accounting approach is influenced by work of Willardson (1985), Seckler (1993 and 1996), Willardson, Allen, and Frederiksen (1994), Jensen (1993), Keller and Keller (1995), Keller, Keller, and El Kady (1995), Keller, Keller, and Seckler (1996), Frederiksen (1997), and others. The purpose of this paper is to provide a detailed example of water accounting at the basin level in order to demonstrate its applicability. The example is taken from Egypt's Nile River where somewhat detailed information is available on water use and productivity. Egypt's Nile River serves as an example of increased competition for a limited supply of water. Through this and similar work, it is hoped that a common, robust methodology will be developed.

TERMINOLOGY

Many approaches, choices, and potential conflicts exist to manage basin-wide, inter-sectoral water use for better performance. Tradeoffs often considered are: municipal and industrial use versus agricultural use, water markets versus subsidies and fixed allocations, efficient on-farm practices versus reuse of water and centralized versus decentralized management. While there is much debate about the best possible strategies, it is important to be able to measure or predict the end result of combinations of actions. Water accounting provides a means for stating end results of various actions so that present performance can be understood and better strategies formulated.

Water accounting relies on a water balance approach, where balance terms are found for a *domain of interest* bounded in time and space. A domain could be an irrigation system bounded by its headworks and command area, and bounded in time for a particular growing season. Conservation of mass requires that for the domain over the time period of interest, inflows are equal to outflows plus any change of storage within the domain. A domain could be defined at the level of a certain use such as on-farm irrigation, at a service level such as an irrigation system, or at a sub-basin or basin level. The domain of interest for this study is the Nile River and its irrigated area from the High Aswan Dam to the Mediterranean Sea. A time step of 1 year is considered.

Gross inflow is the total amount of water flowing into the domain from precipitation, and surface and subsurface sources. Here the gross inflow is the flow from the High Aswan Dam plus precipitation falling into the area. It is assumed that there are no lateral sub-surface inflows.

Net inflow is the gross inflow plus any changes in storage. It is assumed that over a year's period of time there are no changes in storage in the Nile system. This ignores the effect of seawater intrusion where groundwater storage of freshwater is being decreased and replaced by saline waters originating from the sea.

Water Depletion is a use or removal of water from a water basin that renders it unavailable for further use. Water depletion is a key concept for water accounting, as it is often the productivity and the derived benefits per unit of water depleted we are interested in. It is extremely important to distinguish water depletion from water diverted to a use, because not all water diverted to a use is depleted. Water is depleted by four generic processes (Seckler 1996, Keller and Keller 1995, and Molden 1997): 1) *Evaporation:* water is vaporized from surfaces or transpired by plants; 2) *Flows to sinks:* water flows into a sea, saline groundwater, or other location where it is not readily or economically recovered for reuse; 3) *Pollution:* water quality gets degraded to an extent that it is unfit for certain uses; and 4) *Incorporation into a product* by a process such as incorporation of irrigation water into plant tissues.

Process consumption is that amount of water diverted and depleted to produce an intended good. In industry, this includes the amount of water vaporized by cooling, or converted into a product. For agriculture, it is water transpired by crops plus that amount incorporated into plant tissues.

Non-process depletion occurs when diverted water is depleted, but not by the process it was intended for. For example, part of water diverted for irrigation is consumed by transpiration (process), but also depleted by evaporation from soil and free water surfaces (non-process). Drainage outflow from coastal irrigation systems and coastal cities to the sea is considered non-process depletion. Deep percolation flows to a saline aquifer may constitute a non-process depletion if the groundwater is not readily or economically utilizable. Non-process depletion can be further classified as *beneficial or non-beneficial.* For example, a community may place beneficial value on trees that consume irrigation water. In this case, the water depletion may be considered beneficial, but depletion by these trees is not the main reason why water was diverted.

Committed water is that part of outflow that is committed to other uses. For example, downstream water rights or needs may require that a certain amount of outflow be realized from an irrigated area. Or, water may be committed to environmental uses such as minimum stream flows, or outflows to sea to maintain fisheries. In the case of Egypt, there is a need to release water to the northern lakes or the Mediterranean sea either through the Nile itself or through drains in order to flush out salts and pollution, and to maintain the environment.

Uncommitted outflow is water that is not depleted, not committed, and is thus available for a use within a basin or for export to other basins, but flows out due to lack of storage or operational measures. For example, waters flowing to a sea in excess of requirements for fisheries or environmental or other beneficial uses are uncommitted outflows. With additional storage, this uncommitted outflow can be transferred to a process use such as irrigation or urban uses.

Available water is the net inflow less the amount of water set aside for committed uses and represents the amount of water available for various uses. Available water includes process and non-process depletion, plus uncommitted water.

Non-depletive uses of water are uses where benefits are derived from an intended use without depleting water. In certain circumstances, hydropower can be considered a non-depletive user of water if water diverted for another use such as irrigation passes through a hydropower plant. Often, a major part of instream environmental objectives can be non-depletive when outflows from these uses do not enter the sea.

ACCOUNTING FOR NILE WATER USE

In this section, water accounts are given for the agricultural year 1993 to 1994. The accounts are based on water balance computations by Zhu et al. (1995). This water balance approach considered measured inflows and outflows, and assumed no change in storage and calculated evaporation and transpiration as a residual. Crop evapotranspiration was calculated after making estimates of other sources of non-crop evapotranspiration. The values for the balance are summarized in Fig. 1. The different terms are explained in some detail below for illustration purposes.

Inflows

The gross inflow into the Nile system is 56.2 km^3, which is equivalent to 55.2 km^3 of releases from the High Aswan Dam (HAD) plus 1.0 km^3 of precipitation. It is assumed that over the one year time period there are no storage changes, so gross inflow is equal to net inflow. Water released from HAD flows down the river, where most of it is diverted to agriculture. Diverted water either leaves the domain through evaporation or transpiration, or returns back into the system where it is diverted for use again. The value of 65.3 km^3 in Fig. 1 represents the reported diversions, and is greater than the dam releases. It should be noted that actual diversions are much greater than this due to considerable reuse of return flows at scales smaller than main canals. At the tail end, most water that is not depleted by an evaporative use flows out to the Mediterranean Sea.

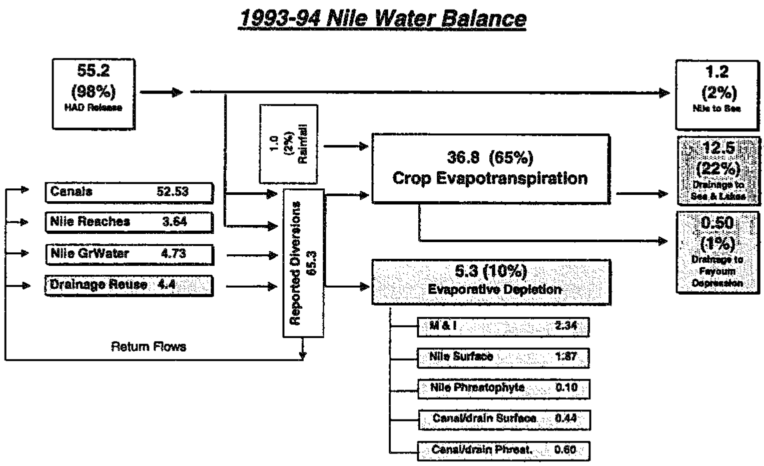


Fig. 1. Water balance components for Egypt's Nile River below the High Aswan dam for the 1993 to 1994 agricultural year. Values are in cubic kilometers (1 km^3 = 10^9 m^3).

Process Uses

Major process uses of Nile water are municipal, industrial, agricultural, and navigation uses. A total of 56.1 km^3 is diverted from the Nile directly to use into canals for irrigation, municipal and industrial (M&I) uses. There is considerable return flow into drains, groundwater, canals and rivers where it is available for use again before leaving the system. A recorded amount of 12.7 km^3 is reused from drains or from groundwater. There is, in additions a considerable amount of unrecorded reuse from these sources.

The total water depleted by crop evapotranspiration is estimated at 36.8 km^3, while process depletion by M&I uses[4] is estimated at 2.3 km^3. During much of January, the Nile irrigation system is closed for maintenance. During this time, some water is released in the Nile to keep levels high enough to support navigation. During this period there was outflow of 1.2 km^3 from the Nile mouth, which can be charged as a process use to navigation. Agriculture is by far the largest process user of water.

Committed Water

Some water is required to flow out of the Nile system for environmental needs, such as, to drain out salts, to carry away pollutants that would otherwise concentrate in the Nile waters, and to maintain coastal estuaries for fishing. A drastic reduction in drainage outflow would cause pollutants to concentrate, which would result in detrimental environmental effects, sicknesses and losses to the fisheries industry. Based on environmental concerns, some would argue for more discharge flowing to the northern lakes and sea than what presently exists. The minimum outflow requirement is an important value that deserves much more research attention.

With our present knowledge it is difficult to give an estimate for the volume of outflow required, but there are indicative values (Emam et al. 1996, SRP et al. 1996, and Zhu et al. 1996). A major consideration is the salinity of the outgoing water. The Drainage Research Institute (1993) reports that in 1992, 3.1 km of the 12.3 km^3 of drainage outflow had salinity levels of less than 1500 ppm, which is reasonably good quality water for irrigation. The remaining 9.2 km^3 is of marginal and poor quality water in terms of salinity. At present, water carries a heavy load of pollution that needs to be washed out. With these considerations, a first estimate of minimum outflow on the order of 8 km^3 can be made.

Non-Process Depletion

The majority of the outflow is through the drainage system. Some of this water can be considered the environmental commitment discussed above. The remainder of the water is considered a non-process, non-beneficial drainage outflow. In 1994/95, the amount of drainage outflow to the Mediterranean Sea, the northern lakes, and the Fayoum Depression was 13.0 km^3. Subtracting 8 km^3 of committed outflow from both the outflows yields 5.0 km^3 of non-process depleted water leaving the domain. This amount represents an estimate of the

[4] The depleted fraction for M&I uses was assumed to be 30% for the Nile Valley and 20% for the Nile Delta. That is, in the Nile Valley, 30% of the water diverted for M&I use is depleted through evaporative consumption, or through disposal outside of the domain.

amount of water that can be saved and converted to a process or other beneficial use.

Other types of non-process depletion are evaporation from fallow land, evaporation from free water surfaces, and evaporative use by phreatophytes and other non-agricultural vegetation. Certainly, some of this depletion is beneficial as it leads to the desirable green belt along the Nile. There may be other subsurface outflow into sinks, such as flow from the Nile Delta to the Qatara depression (Bastiaanssen et al. 1990) but here the value is assumed to be negligible. It was estimated that there was 3 km^3 of non-process, evaporative depletion during the time period of interest.

WATER ACCOUNTING

The water accounting components are summarized in Table 1, and visualized in Fig. 2. Process depletion includes ET, M&I uses and navigation. Non-process depletion includes evaporation from free surfaces, fallow land, phreatophytes, and drainage to the sea in excess of environmental requirements. Available water is the calculated volume after subtracting that water committed for environmental uses from the gross inflow. There are no uncommitted utilizable outflows remaining; thus all the water available is depleted.

Accounting Indicators

Accounting indicators characterize the use of water in the Nile. They are based on ratios of various depleted amounts to gross, net, total depleted, and available water. Following Willardson et al. (1994), they are presented as fractions as opposed to efficiency terms to avoid placing erroneous value judgments on the terms (i.e. bigger is not always better). The indicators are defined below, and Table 2 summarizes the accounting indicators.

Depleted Fraction (DF) is that part of the inflow that is depleted by both process and non-process uses. Depleted fraction can be defined in terms of net, gross, and available water. Table 2 summarizes accounting indicators. In this case, it is assumed that there is no change in storage, therefore net inflow = gross inflow, and

$$DF_{net} = DF_{gross} = \frac{Depleted}{Gross\ Inflow} = \frac{48.2}{56.2} = 0.86 \quad (1)$$

Subtracting committed water from net inflow leads to the available water, and the depleted fraction of available water is:

$$DF_{available\ water} = \frac{Depleted}{Available\ Water} = \frac{48.2}{48.2} = 1.00 \qquad (2)$$

Table 1. Water accounting components of Egypt's Nile River.

		Total	Component
Inflow		km^3	km^3
Gross Inflow		56.2	
	Surface diversions from High Aswan Dam		55.2
	Precipitation		1.0
	Subsurface sources from outside subbasin		0.0
	Surface drainage sources from outside subbasin		0.0
Storage change		0.0	
	Surface		0.0
	Subsurface		0.0
Net Inflow		56.2	
Outflow			
Total Outflow		14.2	
	Surface outflow from rivers		1.2
	Surface outflow from drains		12.5
	Subsurface outflow		0.0
	Flow to Fayoum depression		0.5
Committed Water		8.0	
	Environment maintenance (assumed)		8.0
Available Water			
	Total (gross inflow less committed water)	48.2	
Depletive use			
Process depletion		40.3	
	Evapotranspiration		36.8
	M&I Uses		2.3
	Navigation		1.2
Non-process depletion		8.0	
	Outflows in excess of environmental requirements		5.0
	Other evaporation (phreatophytes, free water surface)		2.9
TOTAL DEPLETION		48.2	

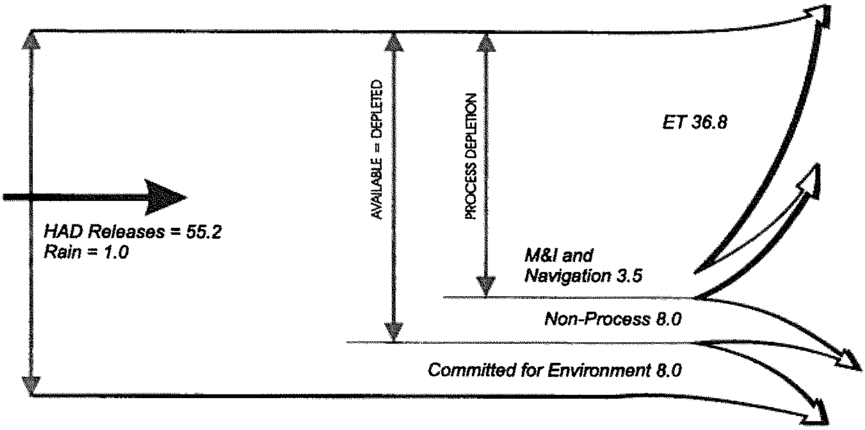


Fig. 2. Water accounting for Egypt's Nile River.

This shows that all available water is being depleted. There are no uncommitted outflows to the sea which can be further exploited, there is only savings from drainage outflows (a non-process depletion). What percent of depleted water goes to intended processes?

Process Fraction (PF) relates process depletion to inflow, total depletion, or available water and is useful to identifying opportunities for water savings. Process fraction is analogous to the effective efficiency concept forwarded by Keller and Keller (1995) and is particularly useful in identifying water savings opportunities when a basin in fully or near fully committed.

$$PF_{depleted} = \frac{Process\ Depletion}{Total\ Depletion} = \frac{ETc + Navigation + M\ \&\ I}{TotalDepletion}$$
$$= \frac{36.8 + 1.2 + 2.3}{48.2} = 0.84 \quad (3)$$

This shows that 84% is of total depletion is depleted by intended process uses, and 16% of the depleted water goes to non-process depletion. Converting the non-beneficial part of the 16% to process use represents a means to increase productivity of water.

It is possible to define a process fraction for irrigated agriculture alone. First, the amount of water available for irrigation is set at the gross inflow less committed water less M&I and navigation depletion (56.2-8-2.3-1.2=44.7).

$$PF_{available} = \frac{Process\ Depletion}{Available\ Water\ for\ Irrigation} = \frac{36.8}{44.7} = 0.82 \qquad (4)$$

By doing this, evaporative depletion through phreatophytes and free water surfaces is all charged to irrigation. It could be argued that some could be charged to M&I uses as the water is conveyed through the same river and canals. The intended process of crop evapotranspiration depletes a very large percent of the water available for irrigation, and it will be difficult to increase this process consumption. In the future, the water available for irrigation is likely to decrease as domestic and industrial uses with take from this share of water.

Table 2. Water accounting indicators.

Accounting Indicators		Value
Depleted Fraction		
	of Gross Inflow	0.86
	of Available Water	1.00
Process Fraction - All Uses		
	of Gross Inflow	0.72
	of Available Water	0.84
Process Fraction for Irrigated Agriculture		
	Available for Agriculture in km^3	44.7
	Irrigated Agricultural Process Fraction	0.82

Agricultural Productivity of Nile Water

Productivity of Water (PW) in agriculture can either be related to the physical mass of production or the economic value of produce per unit volume of water. Productivity of water is similar to a water use efficiency (Viets 1962, Howell et al. 1990) term which relates mass of production to evapotranspiration or transpiration. Here productivity of water is used in a basin-wide sense and can be defined as production in terms of net inflow, depleted water, and process depletion.

Productivity of agriculture can be measured in several manners, for example mass of production, gross value of production, net value of production, or calories derived from the produce. With the multiple food and non-food crops existing in Egypt's Nile sub-basin it is intuitive to use a monetary value for production, and here both gross and net value of production are considered. An ideal expression of basin water productivity should consider the sum of net benefits obtained from agriculture, fisheries, navigation, environment, industrial, municipal and other uses. Valuing the productivity of all the uses can be quite difficult, especially in environmental and municipal sectors, and falls out of the scope of this paper.

Here the concentration will be on productivity of the irrigated agricultural sector.

In 1993/94, the total gross value of production of the irrigated agricultural sector was estimated at 24.8 billion Egyptian pounds, equivalent to 7.5 billion US$ at 1993 prices (exchange rate of 3.3 Egyptian Pounds (LE) per US$). The productivity of water defined as the gross value of production per gross inflow is:

$$PW_{gross\ inflow} = \frac{Gross\ Value\ of\ Production}{Gross\ Inflow} = \frac{7.5BUS\$}{56.2km3} = 0.13\frac{US\$}{m^3} \qquad (5)$$

A base measurement for productivity of water is to consider the value of production related to crop evapotranspiration, the process depletion for agriculture. An important means of gaining more productivity out of the water resource is to get more output per unit of evapotranspiration. Growing higher value crops or less water consumptive crops with equal value, or improving agronomic and water management practices may lead to a higher water productivity of water. The productivity of process water for agriculture is:

$$PW_{process} = \frac{Gross\ Value\ of\ Production}{Process\ Depletion\ (ET)} = \frac{7.5BUS\$}{36.8km^3} = 0.20\frac{US\$}{m^3} \qquad (6)$$

The value of productivity of water consumed by evapotranspiration compares quite favorably with other irrigated (Molden et al. 1998).

Table 3. Water Productivity Indicators.

Gross Value of Production 1993 in billions of 1993 US$	7.5
Gross Value of Production per Unit of	US$/m^3
Gross Inflow	0.13
Water Available for Agriculture	0.16
Crop Evapotranspiration	0.20

TRENDS IN WATER USE AND PRODUCTIVITY

Supplies and utilization can be understood by tracking certain key variables as presented in the water balance by Zhu et al (1995). Figure 3 focuses on the major inflow to the domain -- releases from the High Aswan Dam (HAD), and the most significant outflows from the domain -- through crop evapotranspiration, drainage outflow to the sea, and outflows from the Nile surface system to the sea.

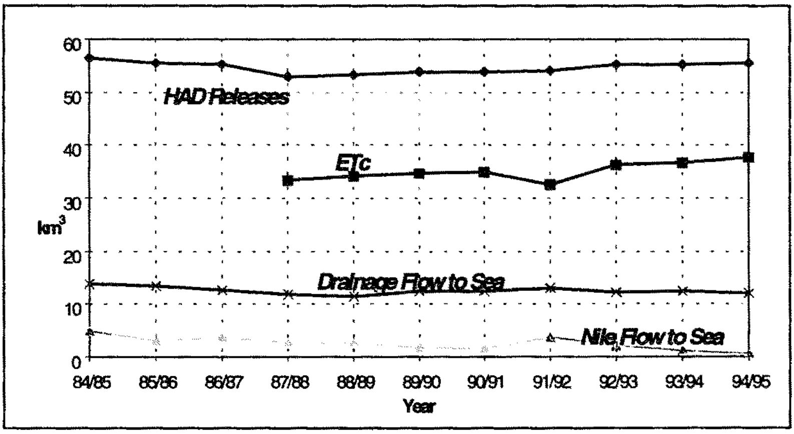


Fig. 3. Water Supply and Use of the Nile River below the High Aswan Dam.

A clear trend of Nile outflow to the sea in recent years is apparent as shown in Fig. 3. From 1974 to 1980 the average annual Nile River flow to the sea was 6.2 km^3, from 1981 to 1990 it was 3.9 km^3, while from 1991 to 1996 it was reduced to 1.7 km^3. For the 1995/96 irrigation season, the Nile River outflow to the sea was reduced to 0.26 km^3. In order to save water for use in agriculture and other sectors, managers have been devising ways to close the route of water to the sea. Future potential from obtaining more savings of this Nile River water is quite small.

Data for drainage outflow to the northern lakes and sea is available from 1984 to present and is maintained by the Drainage Research Institute (DRI 1992) in their annual yearbooks. Changes in drainage outflow to the sea have been much less dramatic than for the Nile outflow as shown in Fig. 3. Notable is the change that took place between 1987 and 1989, the period of drought when Lake Nasser reached its minimum level. Starting from 1988/89, drainage outflow was reduced from 14 km^3 to about 12 km^3, and has stayed at around this level until the present. Unlike the situation with Nile outflow, reduction in drainage outflow to the sea has been difficult to achieve. One possible major factor for the lack of reduction in drainage outflow in spite of conservation efforts is that rice irrigation in the northern delta area contributes heavily to drainage outflow, and the rice area has been increasing recently.

Crop evapotranspiration has shown a steady increase. As shown on Fig. 3, reductions in Nile discharge to the sea and drainage outflow to the sea have been balanced by an increase in crop evapotranspiration. Given the ongoing expansion of agricultural area, plus the increase in yields on existing land, this trend is expected to remain on course for the next few years, until it becomes very difficult to replace drainage water flowing to the sea with crop evapotranspiration.

Over time, the production value of Egyptian agriculture has shown an upward trend as shown in Fig. 4. The increase in gross value of crop production has been due to several factors including increased productivity, the growing of higher valued crops, and the expansion of irrigated land.

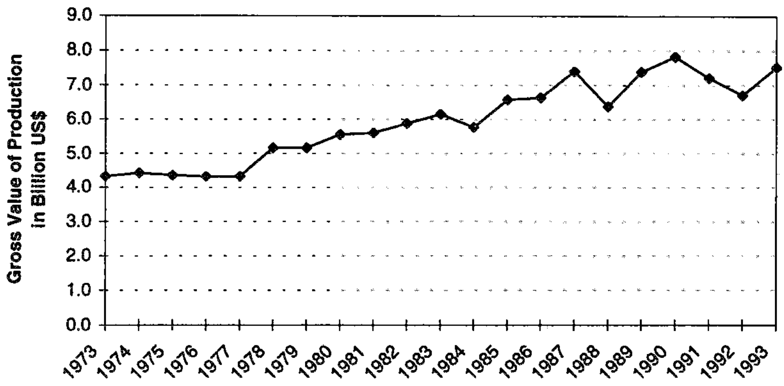


Fig. 4. Value of Production over time (Data Sources: Ministry of Agriculture and Land Reclamation, 1973 to 1992 and Agricultural Economics Research Institute, 1993. Currency is adjusted to 1993 prices as per the World Bank Tables, 1994).

Agricultural productivity per cubic meter of gross inflow provides a means of tracking performance of Egyptian irrigated agriculture (Fig. 5). From the 1970s to the 1990s, this value has nearly doubled[5]. This trend is expected to increase in the near future with expansion of irrigated land, economic liberalization, the shift to higher valued crops, and increasing productivity. It is possible however that the trend could reverse, if old lands are taken out of production through salinization or lack of water, in order to serve new facilities. Data on gross value of production per unit of ET is available for the period of the water balance analysis. During this time, the value varies around US$0.20

[5] The gross value of output reported here includes crops produced by non-Nile water, which is small in comparison to that produced by Nile water.

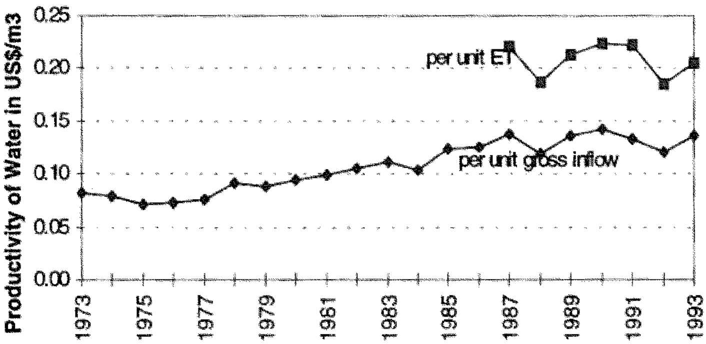


Fig. 5. Trend of gross value of production per cubic meter of water for agricultural crops. Values are in constant 1993 US$.

IMPROVING THE PRODUCTIVITY OF WATER

A picture of water use and productivity emerges from the above analysis. Agriculture dominates as the major user, with depletive use by municipal and industrial uses a small but growing factor. A very high percentage of inflowing Nile water is being depleted (depleted fraction = 0.85), and most of the water depleted is being put to productive use (process fraction = 0.82). The productivity of water in agriculture is quite good, with the productivity of gross inflow estimated at US$0.13 per cubic meter and increasing and the productivity per unit ET at US$0.20. However, opportunities for water savings are becoming more difficult in spite of increasing demand in agriculture, environmental, urban, and industrial sectors.

One opportunity to increase productivity of Nile water is to capture more of the drainage outflow and convert it into a productive use. However, initial estimates based on downstream environmental outflow requirements show only 4 to 5 km^3 can be captured with conservation efforts (El Kady and Molden 1995, WRSR reports 1 and 2, 1996). Only well-conceived projects to increase efficiency on a local level may achieve slight increases in the depleted fraction when these reduce drainage discharge to the sea. Other efforts are required including storage and further reuse of water.

Outflow into the northern lakes and the sea is a significant, but not well understood factor in determining how much water still remains to be captured and placed in productive use. Much of the existing outflow is required to flush salts and other pollutants out of the system, and to maintain an economically viable fisheries industry. There is very limited opportunity to save water for use in agriculture without adversely affecting the environment.

The Nile river below the High Aswan Dam is an example of a "closing" (Seckler 1996) water basin, where there is little opportunity for water resource development or water savings. To obtain additional water for a depletive use, water reallocation within and between sectors becomes more common. It is likely that water will be reallocated from agriculture to municipal, industrial, and environmental uses. The response to this "closing basin" situation has been in many ways remarkable. Water savings have been achieved, in particular through increased reuse. Water productivity has shown a gradual rise per unit of inflow and remains relatively high per unit of evapotranspiration.

The challenge is to sustain the water productivity increase to match the needs of a rising population in light of increased competition from between sectors. Even within the agricultural sector, there is a desire both to expand irrigated agriculture to new lands, and to increase productivity of existing lands. The most important means of increasing water productivity will be to find means to increase the returns per unit of crop evapotranspiration. This is possible through switching to higher value crops or using crops or agronomic practices that produce more but consume less. Identifying technologies, management, strategies and policies to obtain this will be an important area of strategic research for Egypt.

CONCLUSIONS

A water accounting procedure was defined and applied to Egypt's Nile River system. The procedure allows evaluation of the present status of water use and productivity, trends over time, and gives an indication of where gains can be made. While the procedure sufficiently described the Nile system, it has yet to be demonstrated in situations with more rainfall and with more variability in climate.

Worldwide, many water basins are facing perceived water shortages due to increasing demands on water from all sectors. It is vital that key information be presented in a way that can benefit policy makers. Choices, tradeoffs, and evaluation of use must be described as clearly as possible. This is the role that water accounting plays. This example represents an application of water accounting that we feel is applicable to many other situations. Over time, this methodology will be refined to be useful in many situations found in the world.

ACKNOWLEDGMENTS

This paper draws from the experience gained from the Government of Egypt and USAID supported Water Resources Strategic Research Activity carried out by the National Water Research Center and Winrock International and from the System Wide Initiative on Water Management (SWIM) of the Consultative Group on International Agricultural Research (CGIAR).

REFERENCES

Agricultural Economics Research Institute, "National Agricultural Income Estimates", Ministry of Agriculture and Land Reclamation, Cairo, 1993.

Bastiaanssen, W. G. M., and M. Menenti, 1990. Mapping Groundwater Losses in the Western Desert of Egypt with Satellite Measurements of Surface Reflectance and Surface Temperature, in J. C. Hooghard (ed.), Water Management and Remote Sensing, TNO-Proc. and Inform. No. 42, The Hague, the Netherlands.

Drainage Research Institute, 1991 through 1993, Drainage Water in the Nile Delta, Reuse Monitoring Program Yearbooks, Ministry of Public Works and Water Resources, National Water Research Center.

El Kady, M and D. J. Molden, Conservation Potential for Egypt's Nile Waters: the Strategic research Program, WRSR Report 22, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Emam, E. and K. Ibrahim, 1996, Minimum Nile Drainage Needs for Sustainable Estuarine Ecosystems, Water Resources Strategic Research Activity Report 16, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Frederiksen, H. D., 1997. Personal communications.

Jensen, M. E., 1993. The Impacts of Irrigation and Drainage on the Environment. the 5th N. D. Gulhati Memorial Lecture, Presented at ICID Conference, The Hague, The Netherlands.

Keller, A., and J. Keller. 1995. Effective efficiency: A water use concept for allocating freshwater resources. Water Resources and Irrigation Division Discussion Paper 22. Arlington, VA, USA: Winrock International.

Keller, A., J. Keller, and M. El Kady, 1995. Effective Irrigation Efficiency Applied to Egypt's Nile System, Strategic Research Program, Working Paper

Series, no. 5-1, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Keller, A., J. Keller, and D. Seckler. 1996. Integrated water resource systems: Theory and policy implications. Research Report 3. Colombo, Sri Lanka: International Irrigation Management Institute.

Howell, T.A., R. H. Cuenca, and K. H. Soloman, 1990. Crop Yield Response. In: Management of Farm Irrigation Systems. G. J. Hoffman, T. A. Howell and K. H. Soloman (eds). American Society of Agricultural Engineers Monograph. ASAE, St. Joseph, MI, USA.

Land Expansion Sector, Ministry of Agriculture and Land Reclamation, "The Annual Bulletin of Agricultural Economics," 1970 through 1992.

Molden, Accounting for Water Use and Productivity. SWIM Paper 1. Colombo, Sri Lanka. International Irrigation Management Institute.

Molden, D. J., R. Sakthivadivel., C. J. Perry, C. de Fraiture, W. Kloezen, Indicators for Comparing Performance of Irrigated Agricultural Systems. Research Report 20. Colombo, Sri Lanka: International Water Management Institute.

Seckler, D. 1996. The new era of water resources management: From "dry" to "wet" water savings. Research Report 1. Colombo, Sri Lanka: International Irrigation Management Institute.

Seckler, D. 1993. Designing water resources strategies for the twenty-first century. Water Resources and Irrigation Division Discussion Paper 16. Arlington, Virginia, USA: Winrock International.

Strategic Research Program, Planning Sector and Irrigation Sector, Maximizing the Global Efficiency of Egypt's Nile Irrigation System, Water Resources Strategic Research Activity Report 16, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Water Resources Strategic Research Activity, 1996, National Level Strategies and Policies for Utilizing Egypt's Nile Resource, WRSR Report 1, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Water Resources Strategic Research Activity, 1996, Beheira Pilot Study Area Integrated Water Conservation and Utilization Program, WRSR Report 2, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Willardson, L. S. 1985. Basin-wide impacts of irrigation efficiency. Journal of Irrigation and Drainage Engineering 111 (3), (September 1985).

Willardson, L.S., R. G. Allen, and H. D. Frederiksen. 1994. Universal fractions and the elimination of irrigation efficiencies. Paper presented at the 13th Technical Conference, USCID, Denver, Colorado, October 19-22, 1994. Duplicated.

Willardson, L.S., R. G. Allen, and H. D. Frederiksen. 1994. Universal fractions and the elimination of irrigation efficiencies. Paper presented at the 13th Technical Conference, USCID, Denver, Colorado, October 19-22, 1994. Duplicated.

Zhu, Z., L. Abed, and N. G. Yaqoub, 1995. Revised Nile River Water Balance - Years 1987-1993, Strategic Research Program working paper series, no. 5-1. Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

Zhu Z., Mona el Kady and David Molden, 1996, Drainage Reuse and Land Expansion, Water Resources Strategic Research Activity Report 16, Ministry of Public Works and Water Resources, National Water Research Center and Winrock International Institute for Agricultural Development.

World Bank, World Tables, 1993.

DEFINITIVE BASIN WATER MANAGEMENT

Lyman S. Willardson[1] Richard G. Allen[2]

ABSTRACT

The required level of management of the fresh water supply practiced within a given watershed is defined by all of the physical, chemical, economic, environmental, and sociological factors involved. Efficiency of water use, consumptive versus non-consumptive utilization of water, diversion requirements, and environmental requirements all need to be understood and balanced to optimize use of the available water. Where watersheds span states and sovereign nations, treaties and agreements are required for orderly use of the fresh water resource. Understanding of the nature of water use and the hydrology of the water resource system is a key element in rational utilization of the resource. Elevation, water quality, and temporal availability are some of the parameters that must be considered. Ground water and surface water need to be treated as a single resource for effective management.

INTRODUCTION

The shortest path in the hydrologic cycle occurs over the ocean. Water evaporates from the surface of the sea, rises into the atmosphere, condenses into clouds and falls directly back into the ocean as rain. The total hydrologic cycle time for a given drop of water over the ocean may be only a few hours. If the clouds move over a land mass before the rain falls, the hydrologic cycle may take years to complete. When precipitation occurs over a land mass, the hydrologic cycle is modified by many physical processes such as reservoir storage, aquifer storage, interception, evapotranspiration, and travel time in a stream or an aquifer. The quality of the water will be naturally degraded as the water moves toward an outlet. Part of the water will return to the atmosphere without ever entering an ocean. When rainwater falls on the land, the raindrops coalesce and become controllable liquid water. An area of land that contributes water to an identifiable outlet or measuring point is called a basin. The percent of the precipitation that arrives at an outlet or measuring point as liquid water is the yield of the basin and can generally be predicted statistically by correlating rainfall and runoff, using existing measured data.

[1]Professor, Dept. Biol & Irrig Engr, Utah State Univ. Logan, Utah 84322-4105

[2]Professor, Dept. Biol & Irrig Engr, Utah State Univ. Logan, Utah 84322-4105

Water management (meaning management of the liquid water) in a hydrologic basin requires an understanding of the hydraulics of water movement in aquifers and open channels, and an understanding of the evaporative processes that occur. Evaporation is the only process that subtracts water from the hydrologic system, so identification and quantification of water uses should be made on the basis of whether a particular use has an evaporative component. For example, hydroelectric power generation on a river does not consume water as a result of its being passed through a turbine, but retaining the water in a reservoir until it can be routed through the turbine will result in evaporation that decreases the quantity of water remaining for downstream users.

Wherever a water use occurs, the quality of the remaining water will be reduced. Hydroelectric power generation generally consumes little water and does not significantly affect the chemical quality of the remaining water, however, elevation or hydraulic head is lost. Commercial production of salt will result in a total loss of the liquid water and no water will be available for subsequent reuse. Municipal use of water has a relatively low evaporative component, when lawn irrigation is not practiced, but the biological quality of the remaining water may be seriously degraded. Agriculture evaporates a relatively large proportion of the water it uses, and decreases the chemical quality of the remaining water by a natural concentration process. Therefore, definitive water management in a hydrologic basin requires that quantity, quality, elevation, timing, and volume all be understood in the context of evaporative and non-evaporative uses. The paths followed by the water on its way to an outlet or measuring point must be understood and the effect of the path on water quality and temporal availability must be included in the basin water management analysis. If wrong assumptions are made, serious errors will occur in prognostications.

One of the terms used to evaluate the appropriate use of water is the word efficiency. Efficiency was applied to on-farm use of water for irrigation by O.W. Israelsen (1932). He measured the amount of water applied to a field and determined the proportion of the applied water that was stored in the root zone. Water that left the field as deep percolation below the bottom of the root zone and water that ran off the surface of the field was considered to be "lost" in the calculation of irrigation efficiency. On a single field basis, an irrigation efficiency thus calculated can give an indication of how a given supply of water might be more effectively utilized. Irrigation efficiency analyses only gave recognition of the part of the water applied to the field that was evaporated and not to the unevaporated quantity that was still part of the basin supply. There was also no recognition of the difference in change of water quality between deep percolated water and surface runoff water. Irrigation efficiency is a term that can be used to evaluate the efficacy of a single irrigation application, but the term does not have applicability in definitive water management in a basin. Irrigation efficiency can also be used to compare irrigation application methods or practices on a field or a farm, but cannot be used to either analyze or manage a basin water supply.

It has been suggested (Willardson, Allen & Fredricksen, 1994 and Allen, Willardson and Fredricksen, 1997) that water basin management analyses can be made more definitive by the use of decimal fractions or percentages to better identify the disposition of water for any use within a basin. For example, if 60 percent of the water applied to a field is stored in the active root zone of the plants, all of that water will be eventually evaporated by the plants and the remaining 40 percent of the water applied will remain in the basin's liquid water supply. Any surface runoff water will have approximately the same chemical quality as the water that was applied, not considering any sediment picked up by the water. The deep percolation water, however, will have a salt concentration that may be measurably higher than that of the applied irrigation water because of the natural concentrating effect of evapotranspiration. Dissolution of any existing salt in the soil profile will further increase the salt concentration of the deep percolation water. If the surface runoff water and the deep percolation water happen to be remixed at some downstream point, the resulting water will always have a salt concentration that is higher than that of the original upstream water. This effect occurs naturally, even in the absence of irrigation diversions. In many basins, the salinity of the original water supply is low enough that concentration of salts through natural evaporation and evapotranspiration and by diversions for irrigation still leaves water that is quite acceptable for other uses. To definitively manage water in a basin, both the quality and quantity of the water before and after a given use must be evaluated.

Watershed management has legal and political components as well as physical and chemical components. Decisions on water management are sometimes taken in courts and legislatures that are not founded on the defined physical reality of the watershed involved. In the newsletter, Resource Law Notes, published by the Natural Resources Law Center of the University of Colorado at Boulder (Number 42, Winter Issue, February 1998) is a story entitled "The Watershed Approach," that contains the following statement:

> "Among those elements opened-up for scrutiny are: the determination of who should be involved in making management decisions, at what geographic locations should these decisions be based and what should be the evaluation criteria utilized to determine appropriate water uses and management philosophies."

The statement implies that watershed management can be based on philosophies and evaluation criteria, in the absence of the physical facts that define the hydraulic and hydrologic characteristics of the watershed. Decisions that are taken in the absence of the defined limiting physical parameters can have serious negative results, if they are not correlated closely with the hydrologic realities of the watershed. The physical facts must be known before effective management criteria are established.

The position of a given water use in a basin is another important factor that has an effect on the type and intensity of water management that must be provided to get an eventual full utilization of a given water supply. The following sections describe how water management differs from the top to the bottom of a watershed or basin.

HIGH WATERSHED AREAS

Extensive use of water in the upper parts of a watershed should not ordinarily be a concern from an efficiency standpoint. The use of water by plants is a function of the available evapotranspiration energy and is not a function of the amount of water applied. Research in the high mountain meadows of Colorado (Kruse, E.G. and H.R. Haise, 1974) found that water applications of ten times the amount of water consumed by the plants did not affect downstream water quality or quantity beyond the effect of the consumptive use. The water supply was only reduced by the fraction of the water that was consumed by evapotranspiration. The full unconsumed fraction of the water applied quickly made its way back to the stream either as surface runoff or as part of the ground water flow entering the stream. The flow paths were relatively short so that the temporal availability of the water was not modified significantly.

The studies by Kruse and Haise (1974) also showed that yields could be increased by reducing the amount of water applied and by planting better varieties of grasses that did not have to have tolerance to continuous flooding. Therefore, reducing the quantity of diversions and flow through the soil in the upper watersheds would have increased evapotranspiration losses to some degree, due to higher plant vigor and leafiness, as reflected in the higher yields. The result of "improved" water management in this case would be increased depletion of the downstream basin water supply.

The quality of the water in the upper part of the watershed is high because of the high natural leaching fraction. The salts that are generated naturally by the soil weathering process are leached away at low concentrations and the natural stream flows have a large dilution capacity. Higher watershed areas tend to have short growing seasons and limited land areas, and produce most of the water that is available.

Trans-Basin Diversions: Diversion of water high in one watershed into another basin not only removes the full amount of water transferred and some salt load, but it reduces the dilution capacity of the stream that normally maintains downstream water quality. If half of the water generated in a basin is removed by an upstream diversion, the runoff per unit area of the basin will be seriously decreased. The diverted water has a relatively low salt load and therefore will not be available downstream to improve downstream water quality. It is well known that decreasing the leaching fraction in an irrigated soil profile will reduce the quality of the total amount water that seeps below the root zone. The same principle applies to a watershed or hydrologic basin.

Every basin, as a result of the generation of soluble salts by soil weathering processes, has a potential salt load that is removed in the outflow from the basin (Drever, 1988). The salt load is a function of the area of the basin. Transporting that salt load in a smaller volume of water at the exit from the basin means a higher salt concentration in the water leaving the basin.

The change in salt concentration in a stream from the upper to the lower parts of a watershed is illustrated in Table 1 which was taken from river basin simulations published by the Utah Water Research Laboratory (1968). Table 1 shows the area in square miles, the annual runoff in Acre Feet, the runoff per unit area and the estimated electrical conductivity of the outflow in a downstream direction for the Bear and the Sevier river basins of Utah. The electrical conductivity of the water was estimated by assuming that new salt was generated by soil weathering at a rate of 300 kg/ha/year and that it was removed uniformly with the outflow. Table 1 shows that the annual outflow in acre feet per square mile decreases naturally as the area of the watershed increases in a downstream direction. This is primarily due to lower precipitation amounts at lower elevations. Since salt is generated on an area basis, the salt concentration in the water also increases in a downstream direction. The electrical conductivity values are theoretical calculated natural values and have not been compared with measured values that would take into account leaching of residual salts and diversion of water for irrigation. Actual values are known to be higher than those shown.

To illustrate the effect of diversion of water from a basin, it can be assumed that 100,000 AF are diverted from the Sevier Basin watershed in the first 10 percent of the area. The diverted water would carry some of the dissolved salt out of the basin and would reduce the outflow from the basin by 100,000 Acre Feet. The salt load in the river would be reduced by the amount carried in the diverted water, and the electrical conductivity (theoretical) of the basin outflow would therefore change from 1.03 dS/m to 1.12 dS/m. This occurs because the balance of the salt generated in the basin must be carried by the remaining 974,000 Ac Ft (1.12 = (1,074,000(1.03)-100,000(0.17))/974,000). If, instead of a complete diversion away from the upper basin, the same 100,000 AF of water were completely consumed by irrigation in the bottom 80 percent of the watershed, the water would be lost, but the full salt load would remain. Use of the water for irrigation would change the electrical conductivity of the basin outflow from 1.03 dS/m to 1.14 dS/m (1.14 = 1,074,000(1.03)/974,000). This example demonstrates that salt concentration naturally increases in a downstream direction, just due to basin hydrology and that any consumptive use of water within the basin or diversion of water from the basin will increase the downstream salt concentration by decreasing the basin leaching fraction.

Table 1. Increasing salt concentration in the downstream direction for the Bear and Sevier Rivers of Utah. Utah Water Research Laboratory 1968.

	Percent of Total Area					
	10	20	40	60	80	100
Bear River						
Area, Sq. Mi.	328	655	1,310	1,966	2,621	3,276
Runoff, Ac-Ft	342,900	561,100	812,600	943,500	1,017,300	1,039,100
Ac-Ft/Sq.-Mi.	1045	857	620	480	388	317
dS/m (est)	0.09	0.11	0.16	0.21	0.25	0.31
Sevier River						
Area, Sq. Mi.	1,129	2,259	4,518	6,776	9,035	11,294
Runoff, Ac-Ft	655,200	929,100	1,050,400	1,063,300	1,069,800	1,074,000
Ac-Ft/Sq.-Mi	580	411	232	157	118	95
dS/m (est)	0.17	0.24	0.42	0.63	0.83	1.03

Trans-basin diversions also reduce downstream potential for hydro-electric power generation. Irrigation diversions reduce downstream flow volumes, but only by an amount equal to the consumptive use that ensues. If a large proportion of the return flows travel back to streams via an aquifer, the temporal availability of the water supply downstream may be affected, but the total remaining volume discharged is relatively unaffected, since there are few water losses from a groundwater storage system. In some situations, especially where most of the water is derived from snowmelt, large diversions during large streamflows can reduce downstream flooding potential and the subsequent return flows may return to the surface water supply when they will augment low late-season stream flows, thereby improving fish habitat and streambed environments. The defining water management factors in the upper parts of a basin deal primarily with volume and timing of flows and not with efficiency of use and local water quality. The stream bed environment is generally not seriously affected by local water uses in the upper parts of the watershed. Biological pollution may occur if heavy human recreational use of the upper watershed area occurs, but the quantity and chemical quality of the water will generally remain high.

The definition of water uses in the upper portion of a watershed requires evaluation of the effect of any water uses or diversions on downstream water quality and quantity, with emphasis on the quality and quantity of the fraction not consumed or diverted outside the basin.

MIDDLE WATERSHED AREAS

Most water uses, natural and man-related, occur in the middle areas of watersheds. The land is relatively flat and readily adapted to irrigated agriculture and urbanization. Middle watersheds have less precipitation than high watersheds and their effect on the basin water supply is greater. Large amounts of water can be diverted for irrigation and all of the water not consumed by crops still returns to the groundwater and to the surface water supply, sometimes beyond any point of possible local reuse. When a large proportion of the applied irrigation water is consumed, the quality of the deep percolation fraction of the returning unconsumed water may be significantly reduced if the initial salinity of the water is high. The water diverted in the middle watershed usually has a higher salt content than water diverted higher in the watershed. However, the drainage water (return flow) from irrigation is almost always reusable downstream. In some extreme situations, such as the irrigation of saline soils, the drainage water may become too saline for reuse without dilution.

Water diverted for municipal use usually has a low consumed fraction so that most of it returns to the downstream water supply, although it may have a high biological oxygen demand because of the organic matter it contains. Municipalities, on average, consume from 10 to 15 percent of the water they divert (Fredricksen, 1992) so that 85 to 90 percent of the water remains available downstream in the basin from a mid-basin diversion. Tertiary treatment of municipal waste water usually makes municipal waste water safe for return to a natural stream if there is sufficient dilution and travel time for further natural biological purification to take place. Downstream reuse by other humans is then possible. Measurements of water diversions for municipal purposes does not define the hydrologic impact of such diversions. The consumption and dilution requirements for the recovered water need to be considered as well as the chemical and biological water quality.

The water supply in the middle part of a watershed should be considered to be a conjunctive use system, that includes both surface and groundwater, where quantity, quality, timing, and elevation are parameters of use. Any use of water for any purpose will affect one or all of the important middle watershed parameters. Again, only the fraction of the water supply that is actually consumed is no longer available downstream. All other water will eventually appear above or at the outlet of the system. Examining the fractional disposition of the water will define what is actually happening to the supply and what the final effect of use in the middle watershed will be on the quantity and quality of water downstream.

Environmental considerations of water management become important in the middle watershed. If diversions are too high, the stream may actually disappear for some distance along the natural stream course until surface and subsurface return flows replenish the stream. If wildlife preservation requires the maintenance of a minimum streamflow, then diversions may have to be restricted even though there is excess water available at the downstream end of the middle watershed. Definition of all water needs in the middle watershed, in terms of water consumed, water quality, required stream flow, and temporal availability are required for good management. Conjunctive use of groundwater must managed to prevent an undesirable loss of streamflow (due to seepage losses to groundwater or reduced inflow to the stream from groundwater) caused by lowered water tables. Control of groundwater extraction or recharge of the local aquifer will be required to prevent overuse of the conjunctively managed water supply of the basin.

LOW WATERSHED AREAS

Even under natural conditions, the quality of water in the lower part of a watershed may be relatively low. A natural stream that is undisturbed has a progressive decrease in chemical water quality along its length (Table 1). The flow rate may increase in a downstream direction, but the chemical quality will decrease because the basin leaching fraction decreases. The runoff rate per unit area is highest in the upper watershed and decreases in a downstream direction. The area of the watershed increases in a downstream direction, so that the apparent leaching fraction, defined as the outflow per unit area, decreases. Downstream water quality will always be lower than upstream water quality. In river basin management, the minimum downstream water quality must be set to some standard and the upstream water uses must be managed and controlled to preserve that standard. Any management practice followed in the lower watershed cannot have a physical effect on the watershed anywhere upstream, but it may have a defining effect on management of the upstream water. For example, downstream water rights may have priority over upstream water rights, in which case, upstream uses are restricted. Depending on the water needs (quality and quantity) and water rights in the lower watershed, careful definition of the hydrologic and hydraulic system for the entire watershed is required before management is undertaken. The fractions of water taken from the supply and the fractions returned must be known in terms of quantity, quality, elevation, and timing in order to define the kind of management required for a given system. If the groundwater part of the equation is not included, serious losses in surface-related investments can occur. The planning should take place in a manner that will make the water resource sustainable and usable for all interested parties and for the public good.

The hydrologic position of the lower watershed changes the management and costs (in terms of water volumes) so that they are different from those in the upper and middle parts of a watershed. In the upper watershed, quality is high and all of the water that is not consumed is available for downstream use. In contrast, a lower watershed may border on a saline sink into which any non-recoverable water from the watershed discharges and is, therefore, not reusable. Such locations may be a city or an irrigation project that borders on an ocean, such as the cities of San Diego, Los Angeles, and the San Francisco Bay area, or an irrigation project such as the

Culicacan project in Mexico, the Imperial Irrigation District in California, and the Sevier River system in Utah. All of the diverted water that is not consumed can readily enter the sink to be evaporated and eventually returned to the atmosphere but cannot be directly reused. In the lower watershed, every economically feasible effort should be made to productively consume as much of the water as possible before it is lost to a saline sink, unless there are important environmental reasons for not doing so.

Defining the quality of water that makes it no longer economically and physically useful and which therefore must be discharged into a sink is an important defining management parameter. Large cities that border the ocean have large diversion requirements, but consume only a small fraction of the water and often discharge their treated municipal wastewater directly into the sea. Los Angeles has made an attempt to recover some of their treated sewage water by recharging the groundwater between the ocean and the inland freshwater aquifers that were being pumped down to elevations below sea level. In the Imperial Valley, the groundwater is too saline for recovery, so that pumping groundwater for reuse as a means of recovery is not feasible. In the Imperial Valley, only irrigation water management that results in a high consumed fraction of the water diverted will minimize the loss of the water resource. This occurs because a high consumed fraction, given the relatively constant rate of consumptive use, translates into reduced diversions. This would require reducing the deep percolation to groundwater and reducing surface runoff that is otherwise destined for the Salton Sea. Any water transferred from the Imperial Valley, which currently consumes more than 70 percent of the water diverted, and that is used instead for municipal supplies in sea coast cities, which consume less than 20 percent of the diverted water, will result in net reduction in productively consumed fresh water. In the Salt River Project of Arizona, a middle watershed location, all of the deep percolation from irrigation and municipal treatment recharge facilities is recoverable with deep wells, a management alternative that is not available in a lower watershed.

CONCLUSIONS

Defining the hydraulic and hydrologic system of a given watershed is an important first step in the management of a basin water supply. The fractions of diverted water and the paths that they follow will enable rational decisions to be made concerning the importance of investments in infrastructure to manage water. High efficiencies of water use in an upper watershed are generally unimportant because water not consumed is not actually lost. High efficiencies in lower watersheds are imperative because any water not consumed is not recoverable. A true conjunctive use approach in the examination of the disposition of the total water supply from the top to the bottom of a watershed is necessary to define the management parameters needed to guide the disposition of a limited fresh water supply.

REFERENCES

Allen, R.G., L.S. Willardson and H.D. Frederiksen. 1997. Water Use Definitions and Their Use for Assessing the Impacts of Water Conservation. Proc. ICID Workshop: Sustainable Irrigation in Areas of Water Scarcity and Drought. Oxford, England. Sept. 11-12. 1997. pp.72-80.

Drever, J.I. 1988. The Geochemistry of Natural Waters. 2nd Ed. Prentice-Hall. Englewood Cliffs, New Jersey. 437 pp.

Frederiksen, H.D. 1992. Drought Planning and Water Efficiency Implication in Water Resource Management. World Bank Technical Paper 185. World Bank. Washington, D.C. 28 pp.

Israelsen, O.W. 1932. Irrigation Principles and Practices. John Wiley and Sons. New York p. 310

Kruse, E.G. and H.R. Haise. 1974. Water Use by Native Grasses in High Altitude Colorado Meadows. ARS-W-6. Agricultural Research Service. USDA. Washington, D.C. 60 pp.

Utah Water Research Laboratory. 1968. Hydrologic Atlas of Utah. UWRL. Utah State University. Logan, Utah. November 1968 p. 79.

Willardson, L.S., R.G. Allen, and H.D. Frederiksen. 1994. Universal Fractions for the Elimination of Irrigation Efficiency. 13th Technical Conference. USCID. Denver, Colorado. Oct. 19-22, 1994

AGRICULTURAL WATER RESOURCES DECISION SUPPORT(AWARDS)

L. Albert Brower[1]   Curtis L. Hartzell[2]

ABSTRACT

The Bureau of Reclamation (Reclamation)is the largest wholesale supplier of water in the United States and serves more than 31 million people in the 17 contiguous Western States, providing more than 9.3 trillion gallons of water each year. Accurate, timely hydrometeorological information is essential for efficient water management. The National Weather Service (NWS), in partnership with other agencies, has installed a network of around 160 radar systems throughout the United States and at selected overseas sites, known as the NEXt generation weather RADar (NEXRAD) system. The NEXRAD system provides precipitation information that is readily available to the general public (TV weather, Internet). Great potential exists for agencies such as Reclamation to apply enhanced NEXRAD precipitation products for improving the efficiency of water resource operations and reducing risk of loss from extreme precipitation events. Reclamation's initial work to make operational use of NEXRAD rainfall estimates was the development of an automated information system to assist water users. The result, called the Agricultural WAter Resources Decision Support(AWARDS)system, provides easy, timely access to rainfall and daily crop water use estimates for improving the efficiency of water management and irrigation scheduling. An Early Warning System(EWS) component will provide enhanced public safety to populations at risk downstream from dam structures.

1

Civil Engineer, Bureau of Reclamation, Denver Federal Center, P.O. Box 25007, D-8550, Denver, CO 80225-0007

2

Meteorologist, Bureau of Reclamation, Denver Federal Center, P.O. Box 25007, D-8510, Denver, CO 80225-0007

INTRODUCTION

The AWARDS system is based on modern remote sensing, communication, computer, and Internet technologies. The current AWARDS system uses NEXt generation weather RADar (NEXRAD) and automated surface weather station near real-time data, available from remote computers, to automatically prepare rainfall image and evapotranspiration (ET) chart products for Internet access. Reservoir operators, water managers, and on-farm water users access the AWARDS system products via the Internet to make their operational decisions.

Various Reclamation and other users can benefit from near real-time rainfall or snowfall estimates customized for their particular area of interest. Agricultural water districts can conserve water, and individual irrigators can improve their on-farm operations, when NEXRAD rainfall estimates are coupled with evapotranspiration models to provide better estimates of water need. NEXRAD can be used to provide improved rainfall estimates over watersheds draining into reservoirs with flash flood potential. Because NEXRAD provides continuous spatial and temporal coverage of most of the 17 Western States (mountain blockage is a problem for some areas), many water managers can benefit from radar-estimated precipitation used as input to practical rainfall-runoff models. Such models are often linked to water resource operation models and decision support systems.

AWARDS SYSTEM

The purpose of the AWARDS system is to improve the efficiency of water management and irrigation scheduling by providing guidance on when and where to deliver water and how much to apply. The current AWARDS system works as summarized below and shown in figure 1.

- NEXRAD Doppler radar systems measure equivalent reflectivity factor *(Z_e)* data as input to the Precipitation Processing System(PPS. The PPS produces the Hourly Digital Precipitation(HDP) array product for each radar system, which is identified as Level III, Stage I.
- Real-time surface weather stations collect data.

- Radar and weather data are transmitted and input to central computers for processing; the National Weather Service (NWS) River Forecast Centers produce NEXRAD Level III, Stage II and III products, as defined later in the paper.
- An AWARDS system computer, a UNIX workstation, automatically collects digital format data files of Level III, Stage III radar rainfall estimates, weather station data, and NWS precipitation forecasts from the central computers.
- The AWARDS system computer prepares the rainfall image and chart products, making them available in near real-time for Internet access.
- Reservoir operators, water managers, and on-farm water users access the AWARDS system products via the Internet.
- Reservoir operators, water district staff, and on-farm irrigators make operational decisions based upon the information provided by the AWARDS system.

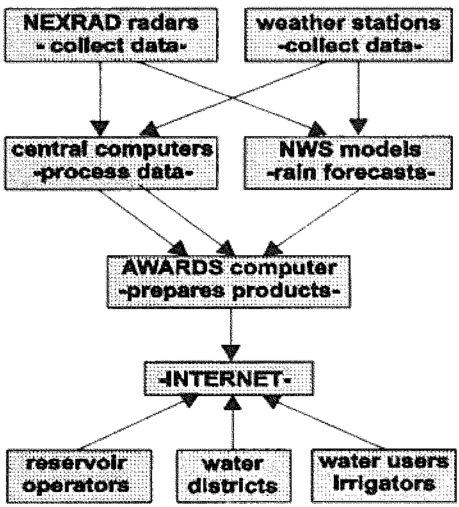


Fig. 1. Schematic Diagram of the AWARDS System.

The AWARDS system automatically integrates 1-hour and 24-hour NEXRAD rainfall estimates

with 24-hour surface weather station data:
- mean temperature
- mean relative humidity
- mean wind speed
- point rainfall accumulation
- total solar radiation

and uses the NEXRAD radar rainfall estimates and surface weather station data with:
- crop evapotranspiration (ET) equations
- local terrain and soil information
- effective rainfall estimation procedures
- local daily maximum and minimum normals
- quantitative precipitation forecasts
- watershed/reservoir systems
- irrigation water distribution systems

to provide the water managers and users with:
- NEXRAD rainfall and watershed rainfall water volume estimates
- effective rainfall estimates
- ET estimates for determining crop water use requirements

The above information is available from Reclamation's NEXRAD Web page (Internet site) at **http://www.usbr.gov/rsmg/nexrad.**

DATA ACQUISITION

NEXRAD Data

NEXRAD precipitation estimates are derived products produced by the NWS Radar Product Generators (RPGs). The radar reflectivity data are converted to rainfall rates using a *Z-R* relationship, and precipitation accumulations are then calculated (Crum et al., 1993; Klazura and Imy, 1993). Level I data are the analog signals from the Radar Data Acquisition (RDA) site, Level II data are the digital base data output from the RDA signal processor, and Level III data are the base and derived products/algorithm output produced by the NWS NEXRAD RPGs. Following are descriptions of the Level III HDP products.

Stage I: Stage I precipitation processing, also referred to as the NEXRAD Precipitation Processing Subsystem (PPS), runs on the NEXRAD computers (RPGs) located at the NWS local Weather Forecast Offices. The PPS generates the Hourly Digital Precipitation (HDP) accumulation product that uses the Hydrologic Rainfall Analysis Project (HRAP) grid cells, sized at about 4- by 4-kilometer(km).

Stage II: Stage II precipitation processing creates hourly precipitation estimates (HDP) using Stage I output in combination with rain gage data. Rain gage data are used to adjust the radar data, using an objective analysis procedure, to create a multi-sensor hourly precipitation estimated accumulation analysis. At present, the Stage I output data are passed to the NWS River Forecast Centers (RFC) for follow-up Stage II and Stage III precipitation processing.

Stage III: Stage III processing mosaics(merges)the Stage II analyses from individual radars,using tools that allow the forecaster to analyze and edit the individual multi-sensor analysis to create an HDP product for the entire RFC's area of responsibility. These data are generated into Network Common Data Format (NetCDF) or xmrg (binary file format) files.

The digital hourly NEXRAD precipitation estimates are automatically collected into the AWARDS computer via File Transfer Protocol (FTP) from the RFCs within 45-minutes of the next hour. Once a full 24 hours are accumulated, computer programs produce 24-hour summaries and make them available on the Internet site maps (images).

Quantitative Precipitation Forecast (QPF)values for the current 24-hour period are derived from algorithms run at the NWS Weather Forecast Offices, then ported to the RFC, where hydrometeorologists create composite products in NetCDF, similar to the Stage III product.

Weather Data

Automated weather stations in operational and developing AWARDS areas transmit surface weather data via radio signal, phone line, cellular phone, or satellite to local computer systems. These data are then automatically collected from the sources, via

FTP, into the AWARDS computer. In some cases, 5-minute data are acquired; in others, only full 24-hour accumulations are available. These data are normally accessible in the early morning, allowing sufficient time to prepare ET charts for the Internet site maps.

Mapping Data

Various Geographic Information System(GIS) data resources are used, such as watershed, hydrologic, political boundary, irrigation district conveyance system, and other features, for developing the base maps for the AWARDS system. Digitizing these features from maps is required when no GIS sources are available. These data are transferred to longitude-latitude coordinates for input to a graphics program available from the National Center for Atmospheric Research, called NCAR Graphics. The HRAP grid cells are plotted and overlaid with the NEXRAD precipitation estimates and weather station rain gage measurements. Lastly, the crop water use and weather data are integrated into the maps via pop-up charts.

Fig. 2. Example of interactive image showing the 24-hour NEXRAD STAGE III rainfall estimates.

ALTUS, OK MESONET SITE - ESTIMATED CROP WATER USE - AUGUST 23, 1997

CROP	START DATE	DAILY CROP WATER USE-(IN) PENMAN ET - AUG 19	20	21	22	FORE-CAST AUG 23	COVER DATE	TERM DATE	SUM ET	7 DAY USE	14 DAY USE
COTTON	501	0.21	0.18	0.15	0.12	0.13	801	1001	14.7	1.5	3.1
COTTON	507	0.23	0.22	0.19	0.16	0.17	805	1001	14.4	1.7	3.4
COTTON	514	0.26	0.25	0.23	0.21	0.21	810	1001	14.1	1.9	3.6
COTTON	521	0.29	0.30	0.28	0.26	0.27	815	1001	13.1	2.1	3.8
COTTON	528	0.29	0.30	0.30	0.29	0.29	820	1001	11.8	2.1	3.8

NEXRAD HRS AVAIL	24	24	24	24	QPF
TOTAL RAIN	0.04	0.04	0.00	1.62	0.00
EFFECTIVE RAIN	0.04	0.04	0.00	1.46	0.00

NEXRAD MONTHLY TOTAL RAIN:	
MAY	4.13
JUNE	5.87
JULY	1.38
AUGUST	5.18

(164x32)

Fig. 4. Example of a pop-up estimated 24-hour Crop Water Use chart that includes NEXRAD rainfall, effective rainfall, and QPF, for cell number 164x32.

Weather station data from the Oklahoma Mesonet system are downloaded to the AWARDS computer using FTP every day at 6:40 a.m. These data include climate values for the prior day. Crop water use for the northern half of the district is determined from the Mangum station and the southern half from the Altus station. If necessary, an averaging algorithm could be implemented to better establish the ET for each cell as calculated by the two weather stations. Averaging was not used in the Lugert-Altus Irrigation District because minor variations exist in the weather data between the stations.

The primary crop in the Lugert-Altus Irrigation District is cotton, which was planted during the month of May in 1997. The daily crop water use is calculated for each of the five planting date ranges using the Penman-Monteith combination reference ET method (ASCE,1990). A crop curve based on Growing Degree Days is implemented as suggested by researchers (New, 1997) at the Texas Agricultural Extension

Service. In this example, the quantities, in inches, are shown for the past 4 days, August 19 through 22. A forecasted ET for August 23 is estimated by averaging the past 3 days use. Data from the National Centers for Environmental Prediction (NCEP) Early ETA model will be used for improving this forecast value and to extend the forecast up to 5 days (development in progress). The cover and terminate dates are shown for reference, and the summation of ET since planting and for the past 7 and 14 days is presented. The NEXRAD total daily rainfall quantities for the specified number of hours of data availability and an estimate of the effective rainfall are also presented. Further work will enhance the effective rainfall quantities using soil, slope, and vegetation data for each specific 4- by-4 km cell. NEXRAD rainfall totals for this cell are also displayed.

OTHER DEVELOPMENTS

New Mexico

In addition to implementation in southwestern Oklahoma, the AWARDS system is being developed in the Rio Grande River Basin to support the Upper Rio Grande Water Operations Model (URGWOM). The URGWOM is a multi-agency surface water modeling effort capable of simulating water storage and delivery operations for the Rio Grande from its headwaters and San Juan-Chama Diversion to Fort Quitman, Texas. This numerical computer model will be used in flood control operations, water accounting, and evaluating water operation alternatives. Results from the AWARDS system will be ported to an ET toolbox component of URGWOM for estimating consumptive use demands on a daily basis for both agricultural and riparian lands.

Two irrigation districts, Middle Rio Grande Conservancy District (MRGCD) and Elephant Butte Irrigation District (EBID), are assisting with installation of weather stations and defining conveyance and delivery systems. Reclamation and district GIS data are used to define map features. NEXRAD precipitation quantities are mapped in the URGWOM study area using Level III Stage III data from radar sites located near Albuquerque and Alamogordo, New Mexico, and El Paso, Texas. These data are available to the AWARDS computer via FTP from the Western Gulf River Forecast Center. Cooperative efforts with governments and universities in both states allow acquisition of daily weather station data and vegetation indexes. The weather station values are transmitted via phone lines and cellular phone to computers at the New Mexico Climate Center at the New Mexico State University (NMSU) and then ported via FTP to the AWARDS computer. The daily crop curves used for determining ET in this AWARDS system were developed by NMSU staff (King, 1998), and crop phenology data were generated by staff at the MRGCD and EBID.

The Biodiversity Assessment Group research facility at NMSU has an agreement with the White Sands Missile Range to receive daily images from Advanced Very High Resolution Radiometer satellites. These data are corrected for atmospheric effects, referenced to vegetation indexes, and processed to calculate acreage of both riparian and agricultural vegetation in each 4- by 4-km cell. The riparian growth includes Cottonwood, Russian Olive, and Salt Cedar trees, and Salt Grass and other species.

In addition to the ET toolbox effort, weather station charts and cell-by-cell crop water use charts for the specific acreage (agricultural and riparian) are mapped with the NEXRAD rainfall estimates and displayed on an Internet site. This 4- by 4-km grid map provides a near real-time decision support tool for water managers and users within the Basin.

Oregon

In southwestern Oregon, near Medford, NEXRAD precipitation quantities from the Northwest River Forecast Center are similarly mapped on an Internet site for the Rogue River Valley watersheds. An important element of this program will be an early warning component, which will be implemented to provide increased awareness of flood danger, as discussed later in the paper.

Mapped data for this project were acquired from the Rogue Valley Council of Governments, United States Geological Survey, and Reclamation GIS sources.
For each HRAP cell within the boundaries of the three irrigation districts in the area, crop water use

charts from Reclamation's Agricultural Meteorology (AgriMet) program are transferred, via FTP, to the AWARDS computer and displayed on the maps. AgriMet weather station values are transmitted via a satellite downlink site in Boise, Idaho, and are then also transferred to the AWARDS computer. Weather station data from other organizations will be accessed in the future. The AgriMet program uses the Kimberly-Penman method for calculating reference evapotranspiration for crops grown in Reclamation's Pacific Northwest Region.

EARLY WARNING SYSTEM (EWS) COMPONENT

Similar to the AWARDS system application, use of near real-time (< 1 hour) NEXRAD data to estimate rainfall over watersheds should enhance existing reservoir and dam EWS. The EWS designs consist of the following elements: (1) a method for detecting flash flood events; (2) a decision making process; (3) a means for communicating warnings between operating personnel and local public safety officials; and (4) a method for local public safety officials to effectively communicate the warnings to the public and carry out a successful evacuation of the threatened population at risk (Fisher, 1993). NEXRAD should be able to pinpoint the cores of heavy convective storms. The HDP data are available about 45 minutes past each sampled hour. Such data should, in most cases, provide alerts before the runoff is measured by a stream gage.

However, Reclamation is working on a NEXRAD precipitation accumulation alogrithm to improve the temporal resolution (Hartzell et al., 1998). A source for these data is a NEXRAD Information Dissemination Service, that provides nearly instantaneous reflectivity data at a spacial resolution of 1 degree by 1 kilometer. The increased accuracy, reliability, and alert lead time gained by incorporating these NEXRAD rainfall estimates can provide enhanced public safety to populations downstream from dam structures, reducing the risk of loss of property and life.

SUMMARY

- The AWARDS system demonstrates a methodology that integrates NEXt generation weather RADar (NEXRAD) Level III, Stage III rainfall estimates with modern computer, communication, and Internet technologies for improved water resources management.
- Daily crop water use estimates for improving the efficiency of on-farm water management are easily available for the crops grown in each 4- by 4-km area.
- Water delivery system managers, in near real-time, can observe hourly precipitation quantities for improved operations.
- Determination of evapotranspiration (ET) for the riparian zone and mapping at these 4- by 4-km resolutions provides an additional decision support tool for river system management.
- Porting consumptive use data to the multi-agency Upper Rio Grande Water Operations Model system via an ET toolbox component contributes daily estimates of ET for both agricultural and riparian lands.
- Reclamation is designing an Early Warning System to provide enhanced public safety to populations at risk downstream from dam structures during potential flash-flood events using the technologies available through the AWARDS system.

REFERENCES

American Society of Civil Engineers, 1990, *Manual 70, Evapotranspiration and Irrigation Water Requirements.*

Crawford, K.C., F. Brock, R. Elliot, G. Cuperus, S. Stadler, H. Johnson, and C. Doswell III, 1992: *The Oklahoma Mesonet: A 21st century project. Preprints, Eight International Conference On Interactive Information and Processing Systems for Meteorology, Oceanography, and Hydrology,* Atlanta, GA, American Meteorological Society, pp. 27-33.

Crum, T.D., Alberty, R.L., and Burgess, D.W., 1993: Recording, Archiving, and Using WSR-88D Data. Bulletin American Meteorological Society, 74, pp. 645-653.

Fisher, D.B., 1993: Reclamation's design process of Early Warning Systems for dam safety. *Hydraulic Engineering '93, Vol. 1.* ASCE, 690-695.

Hartzell, C.L., Super, A.B., Fisher, D.B., Brower, L.A., 1998: *Nexrad-Enhanced Early Warning Systems, Preprints, Special Symposium on Hydrology,* American Meteorological Society, Phoenix, AZ, January 11-16, 1998, pp. 109-112.

King, P., 1998: New Mexico State University, personal communications.

Klazura, G.E. and Imy, D.A., 1993: A Description of the Initial Set of Analysis Products Available from the NEXRAD WSR-88D System. Bulletin American Meteorological Society, 74, pp. 1293-1311.

New, Leon, 1997: Texas Agricultural Extension Service, The Texas A&M University, personal communications.

PROGRESS ON CANAL AUTOMATION

FOR WATER DELIVERY MODERNIZATION

A. J. Clemmens[1]

ABSTRACT

Modernization implies not just rebuilding but rather improving to meet new performance criteria. For irrigation water delivery systems, better customer service is high on the list of priorities. Agricultural customers are facing increasing competition, increasing water costs, and increasing production costs. Improvements in water deliveries can facilitate improved farm irrigation systems management. Canal automation is potentially one piece of the puzzle in trying to modernize and improve overall project performance. Canal automation theory has advanced substantially over the last decade. However, few of these advances have been implemented on operating projects because the theory has not been easy to apply. This paper presents the results of ongoing research to make canal automation more affordable and to integrate it with water delivery operations.

INTRODUCTION

The need to modernize irrigation water delivery systems is well recognized. Modernization is often justified to reduce maintenance costs, but more importantly, modernization is needed to improve water delivery flexibility and delivery service. Burt et al. (1997) report that many districts in the Mid-Pacific Region of the Bureau of Reclamation already have high levels of flexibility. However, there are many systems throughout the world where both flexibility and delivery service are very poor. Yet, the level of service required is relative to the perceived needs of users. So even systems at a relatively high level of service may still need some form of modernization to meet the current needs, particularly where water supplies are limited.

Modernization suggests improvements in the measurement and control of water supplied to and delivered by a project. Increasing the level of control implies the ability to provide more flexible and better service. The level of control that can be achieved for a given project is dictated by physical constraints (e.g., canal properties, structures, etc.), water supply constraints (e.g., storage and

---

[1]Director, U.S. Water Conservation Laboratory, USDA-ARS, 4331 E. Broadway, Phoenix, AZ 85040 bclemmens@uswcl.ars.ag.gov

availability), and operating procedures and methods. Some of these constraints can be minimized with improvements in hardware and operating procedures. In this paper, we discuss the potential role of canal automation in the modernization of irrigation water delivery systems.

WATER CONTROL

In simple terms, ***the control of water within a delivery system centers on control of flow rate and control of volume*** at various points within the system, particularly at delivery points. For any part of the system, inflow equals outflow plus change in storage volume over time. Most canal operating schemes focus on these two concepts of flow and volume balances in one form or another. While these concepts are simple in theory, they are often difficult to apply in practice.

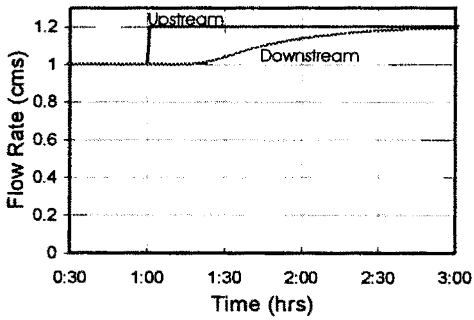


Fig. 1. Flow rate at downstream control structure for a step change in canal inflow rate.

For open-channel systems, application of flow-volume concepts is complicated by lag times — the time required for changes in flow to travel through the system. Further, sudden flow changes made upstream tend to arrive gradually at downstream locations due to wave dispersion, as shown in Figure 1 and described in detail by Strelkoff et al. (1998).

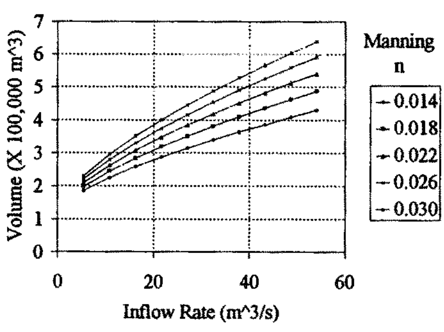


Fig. 2. Pool volume variation with inflow rate, setpoint depth, and Manning *n*.

For sloping canals, changes in flow rate and/or resistance to flow result in changes in pool volume that may not be considered by operators (Figure 2). Changes in pool water levels upstream from control structures also change pool volumes. ***Operators are easily fooled by the time delays, wave dispersion, and pool volume changes that occur within a system.***

While many operators have a heuristic understanding of these concepts and operate to take these into account, canal automation provides a more systematic way of dealing with these issues.

Flow rates set at check and offtake structures are never exactly correct. These errors may be large or small depending on the sophistication of the technology and operating staff. But these ***flow-rate errors will tend to accumulate within the system***. For effective, modern operations, some form of feedback, either manual or automatic, is needed to remove these errors. If inflow is inaccurately set for a given canal (or pipeline) and storage changes are not taken into account, outflow (i.e., deliveries) will never be as intended.

Check and offtake structure properties influence how flow changes are divided at a bifurcation. They can also influence pool volume (e.g., if the downstream level changes) and the speed at which upstream changes are felt downstream (see Strelkoff et al. 1998 for examples). Thus, ***structure hydraulics also influence the response of the system*** and have an influence on the effectiveness of both manual and automatic controls.

Modernization may also include better accounting for water diverted. Improved measurement and control can also help provide better estimates of water delivered, or help determine where in the system losses are occurring. Developing the hardware and operational procedures for good internal auditing of water volumes over time can provide a good impetus for further modernization efforts. If you don't know where the inefficiencies of the system are, it is hard to prioritize potential improvements. ***"To Measure is to Know!"***

MANUAL OPERATIONS

Gate Settings

A vast majority of canal systems are operated manually, with varying degrees of success. A common concept for local-manual control is to divide the flow at bifurcations by establishing a target water level. Gates are set so that if the water level is "close" to the target, the proper flow will go to each offtake or continuing canal. Because the zanjero (ditch rider or canal operator) cannot see all control structures at once, control actions must be made based on judgment and observations from traveling up and down the canal. (See Johnston and Robertson 1990 for further details). Errors in gate settings can increase the amount of time required by the operator to stabilize flow in the canal. Changes in gate hydraulics can cause the relationship between flow, level, and gate opening to change over time. Without separate flow measuring devices, additional zanjero judgment is required.

Scheduling Deliveries

There are a variety of water delivery schedules that determine when an offtake will receive water. The main ones being categorized as rotation, arranged or demand schedules (See Johnston and Robertson 1990 for further details). For all but pure demand systems, the schedule of water delivery changes can be established each day at all points within the system. For manually operated systems, there are several different methods for determining the timing of gate actions. Often, the changes are made at the head of the canal at some point in time (e.g., at the start of the morning shift). Then changes to downstream offtakes are made as the wave travels downstream. Based on experience, the zanjeros can estimate the time of the change at a particular offtake downstream based on when the heading was changed and the travel time for the wave to reach that offtake. Alternatively, a new delivery is made to correspond to the completion of another delivery, and the heading flow is not changed (e.g., delivery is rotated based on demand). For more flexible schedules, the change at the canal head is made to correspond to the requested time of change at the offtake. For example, if the offtake flow is to start at 10 am and the travel time is 3 hours, then the change at the heading must be scheduled for 7 am. The scheduling of deliveries thus depends upon the rules of delivery service and the amount of delivery flexibility provided to the water users. Generally, increased flexibility requires a better control system, both in terms of personnel (e.g., skills, number of employees, etc.) and hardware.

Manual Routing of Flow Changes

The main job of zanjeros is to route flow changes through the canal system. This is a time-consuming, tedious task. Water in open canals flows according to the laws of physics and not the desire of zanjeros. The work involves considerable judgment and experience. This judgment can be improved with a better understanding of canal hydraulics — i.e., "training".

For manually operated systems with gates (or combined weirs and gates) as control structures, increases in flow are nearly always routed from the canal head to the offtake being changed. The operator starts flow into the canal and travels to the next gate downstream. There, (s)he waits for the change in flow to arrive. Since the wave arrives gradually, (s)he must wait until a sufficient portion of the flow increase arrives before transmitting it downstream by opening the gate. Here the gate opening is judged by making the same change in flow as at the previous upstream gate (less seepage losses if significant) for the target water level. The water level at the time of the change may be different from the target level, but should eventually return to it. Figure 3 shows what happens to the flow rate to the offtake and downstream canal while the water level stabilizes. This type of offtake hydrograph is not uncommon (Palmer et al 1991).

The operator proceeds downstream changing each gate in turn until the offtake is finally opened. Now the operator must return to the canal head and repeat the setting of gates with the assumption that flows have stabilized. Now adjustments are made to correct for errors made during the first pass. If the errors in the first pass are large, then the second pass may not be sufficient to bring the canal into balance. In particular, if inflow to the canal is set wrong, the actual canal inflow and the desired outflow cannot balance. To achieve a balance, the headgate must be adjusted and the process starts over.

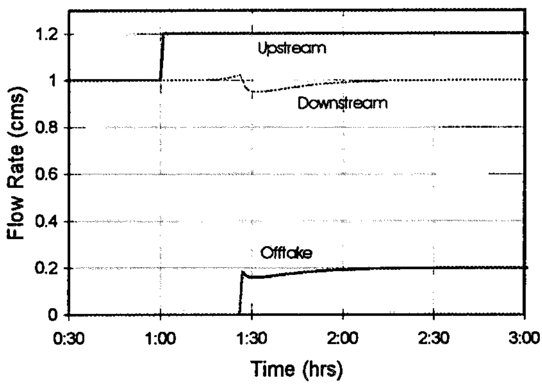


Fig. 3. Typical response in canal outflow for a change in offtake flow resulting from dispersion of step inflow upstream.

This correction of headgate inflow based on mismatches constitutes manual downstream control. Making flow changes on a canal usually requires a minimum of two passes (the second to confirm everything is okay) and a maximum of four or five passes. Flow measurement devices which give an accurate flow rate can help the zanjero minimize trips up and down the canal.

A common practice in some areas is to deliver a greater flow change than needed to satisfy changes in demand. For example, if a 150 lps change is needed, 200 lps more may be added. Experience suggests that this "carriage" water is needed to help move the change through the canal faster. This carriage water is useful for supplying the pool volume changes associated with the change in flow rate. Unfortunately, this carriage water is often left in the canal long after the transients have died out, resulting in wasted water.

We can model these unsteady-flow processes and determine the results of different zanjero operating rules, and the potential improvement of automation. As an example, Lamacq (1997) found through simulation a 10 to 20% variation (standard deviation) in delivery flow rates with respect to their targets using an unsteady flow model and knowledge of zanjero operating rules. Her finding were supported by district records. These variations in flow occurred even though this district has modern equipment and strives to provide excellent service.

Supervisory (Manual-Remote) Control

A single operator has difficulty controlling a canal where changes are taking place at many locations at once. For large canals, it has become practical to control gates from a centralized location. Supervisory control and data acquisition (SCADA) systems are remote manual control systems that replace local canal operators with supervisory control operators. Such systems have been in use for irrigation projects since the 1960's. One irrigation district who recently switched from manual-local control to supervisory control reduced their operating staff by seven people on about 80 km of main canals (Clemmens et al. 1994).

One of the main advantages of supervisory control operations is that the operator can see what is occurring on the entire canal simultaneously. Spills from upstream control of main canals are undesirable. Downstream control requires control over the water source, which may not be feasible for large main canals, particularly where transmission distances are long. Most SCADA systems provide water levels to operators. However, most are operated to adjust volume. For some systems, pool volume errors are actually computed, and flow rate changes needed to compensate for these errors are suggested. Operators may chose to let water levels deviate from the target so that they can use the canals for storage, for example when they do not have complete control of canal inflow.

If one pool is gaining while another is losing volume, gates are adjusted to shift volumes between pools. This method of operation has proven to be effective in many cases. However, most supervisory control operators have difficulty dealing with canal transients. Most control decisions are made after flows have stabilized -- a change and wait approach. Automated systems, discussed below, can be designed to take transients into account so that the operators do not have to wait for the flows to stabilize. They can act continuously. Brouwer (1997) compared an automated control scheme with manual SCADA control for a large main canal, through simulation. The results suggest that automatic control could provide significant improvement. While simulation results of automatic control are encouraging (Clemmens et al. 1997), they have yet to be proven in the field.

Water Accounting

Water accounting methods should determine the destination of all water diverted or pumped into the canal system. One level of water accounting is to compare water delivered to users with that entering the canal. For some districts, records of delivered water are not sufficiently accurate to make these assessments. Water changes tend to reflect ordered volume more than delivered volume. Such water accounting often shows that a significant amount of the diverted water is not accounted for, even when seepage and evaporation are taken into account. Charging for water based on cropped acreage discourages water conservation.

Few districts with canals have volumetric water meters. Mostly, rate is determined from head on a gate or weir. Such measurements may or may not give an accurate picture of water delivery. Careful monitoring of one water district showed that the average flow rate varied by 30% from that based on a one time a day measurement or by 40% from that ordered or intended (Palmer et al. 1991).

Proper water accounting takes this one step further to include water balances on lateral canals. Often, lateral canals near the head of the system receive more water relative to demand than those downstream. Proper water accounting can be used to document and correct this inequity. Good flow rate measurements with sufficient reading frequency or totalizing meters are required to make this water accounting reliable. However, good water accounting should be the first step toward solving water delivery and distribution problems.

AUTOMATIC CONTROLS

The position taken here is that there is a place for some type of automatic control of various delivery-system operational functions. The type and extent of automatic control that is appropriate depends upon the specifics of the system and its management. However, canal automation has much more potential than is currently being exploited.

There are a wide variety of automatic control systems in the literature. Most of these are classified and discussed by Malaterre et al. (1998). Those in use are summarized by Rogers and Goussard (1998). In reality, effective control, whether manual or automatic, must control the distribution of volume and must control flow rates at bifurcations. Control of water levels is typically secondary in order to control flow rates.

The application of canal automation is really in its infancy. There are basically only two types of automatic controls currently in use;

1. automation of single devices or single functions, with more system-wide functions done manually and
2. automation of some global decisions, with local functions done manually or through simple structures (e.g., where storage volumes are large).

The net result is that automatic controls are primarily implemented in a piecemeal fashion. A more ***systematic approach to the development of control "strategies" is needed***, as opposed to control "devices" or control "algorithms." Local control devices will continue to be one component of canal modernization. But, they can not deal with overall regulation issues and will not be discussed here. Also, I will not attempt to cover all the control methods proposed or in use. Several of the more important ones will be discussed. First, however, it is important to distinguish several different types of control.

Open-loop control occurs when the variable that is being controlled is not measured. For example, routing of flow changes through a canal can be done without regard to existing water levels (e.g., by determining needed changes in gate openings based on assumed conditions). If the controlled variable is water level and you change the upstream flow based on a measured offtake flow change, this is still open-loop control. This is often called feedforward control. Closed loop control exists when the variable to be controlled is measured and control actions are taken (e.g., changes in gate flows or opening) based on that variable. Automatic control of water levels immediately upstream or downstream from a gate is a form of closed-loop (feedback) control.

Centralized Automatic Control

There are only a few canals in the world which use centralized automatic control. Utilizing centralized automatic control logic has become a research project for every canal to which it has been applied. At this point in time, it is not off-the-shelf technology. However, a necessary and important first step in such automation is development of the hardware and communication systems needed for supervisory control (i.e., SCADA).

One of the more significant approaches to canal automation is dynamic regulation, developed for the Canal de Provence in southern France. The scheme estimates future demands, observes water levels within the system and determines changes in flow rate at the head of canal needed to restore volumes if those demands are realized. Pool volumes as a function of flow rate and stage are known. Flows between pools are adjusted by automatic gates that try to maintain a constant differential in water levels between pools. Water is pumped from the canals into water towers for pressurizing sprinkler irrigation systems. Thus the canals really serve as reservoir, and are quite different from gravity flow systems typical of much of the Western U.S. Other systems built by the French and operated in a similar way also tend to have large storage volumes – i.e., canals are not designed as efficient sections for transmission of water, as is typical in most irrigation projects. This is primarily an open-loop control system, with some local feedback components.

Another significant approach is that used to control the Central Arizona Project. Their control approach is to determine the desired conditions for some future time, and then changes the gate settings so that when the transients die down, the system will be at the desired steady state. The system seems to work well, and is useful considering the constraints imposed by lift station pumps. However, it is not responsive to changes in demand and the staff has to continuously calibrate gate coefficients and canal roughness parameters. There is no real feedback. Gate stroking was originally proposed (discussed below), but proved too difficult to implement.

Gate Stroking

Wiley (1969) first proposed a method for numerically computing, with the method of characteristics, the timing and amount of upstream flow changes to satisfy downstream changes in demand. This method has come to be known as gate stroking. It is a form of open-loop feedforward control. The U.S. Bureau of Reclamation (Falvey and Luning 1979) developed software to implement Wiley's method. Several attempts have been made to implement this in practice and they have all been unsuccessful. Several finite-difference approaches (e.g., Preissman scheme) to the gate stroking problem have been attempted. Bautista et al. (1997) summarize these methods and present an improved method. However, further research with this method suggests it will always be difficult to use because of the hydraulic constraints being imposed. With gate-stroking, the downstream water level is fixed exactly, and the desired discharge is forced to make abrupt changes. Because of the dispersive nature of waves, sharp changes in discharge and a constant water level are essentially a physical impossibility. Thus, the numerical procedures often produce upstream inflow hydrographs that oscillate significantly or are not physically possible. This water-level constraint is actually not critical, since water levels can change a small amount with little negative influence on delivery performance. A further complication is that unsteady flow is not linear. As a result, the inflow hydrograph for the sum of several individual changes does not equal the inflow hydrograph for the combined changes. Thus, this technique would require recalculation for every combination of changes – these hydrographs have to be computed essentially in real time. A much simpler and still effective alternative is discussed below.

Integrator-Delay Model

Schuurmans et al. (1995) propose an approximate model of canal response (integrator-delay model) based on two simple canal pool properties: the disturbance wave time delay and the water surface area of the pool portion influenced by backwater from the control structure. These two properties, delay time and backwater pool area, can be computed with their model, determined from observation of canal properties, or computed from unsteady-flow simulation. This canal response model assumes that downstream structures use constant flow rate control. Thus a step change in inflow would cause, once the wave arrives, a constant rate of change of backwater pool volume. Assuming that the backwater area is constant for a given set-point depth, the rate of rise of the water level is then related to the mismatch in flow rate (e.g., difference between inflow and outflow), which is used to guide the development of controller constants. The properties of the integrator-delay model can be used to develop both feedforward and feedback control methods.

For pools affected by backwater over their entire length, the above-described model assumes no time delays. The backwater pool area is the only controller design variable. However, reflection waves may be present for these types of pools. Most pools have either a significant time delay, or reflection waves, but seldom will they have both. Examples of the response of downstream water level to a step change in pool inflow and constant pool outflow for two pools of one canal are shown in Figures 4 and 5 (Clemmens et al. 1997).

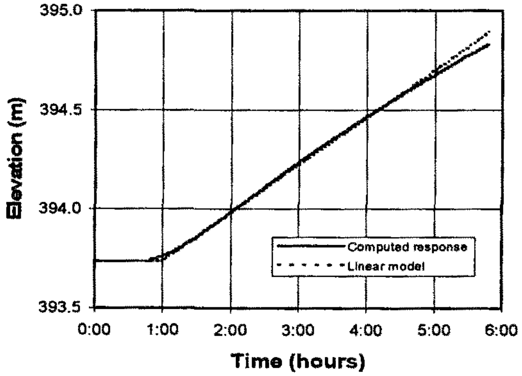


Fig. 4. Response in water level at the downstream end of pool 1 for a step change in inflow from 43 to 47.3 m^3/s and no change in outflow (from Clemmens et al., 1997).

Figure 4 shows a linear integrator-delay model fitted to water-level response data for a pool primarily flowing at normal depth. The fit is reasonably good initially, but the actual response deviates from the model at large depths because the actual surface area changes as the depth rises, which is ignored in this approximate model.

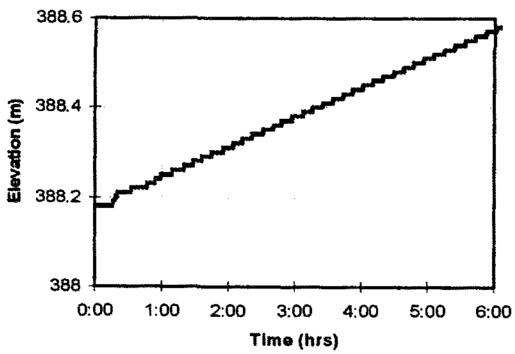


Fig. 5. Response in water level at the downstream end of pool 3 for a step change in inflow from 9 to 10.8 m^3/s and no change in outflow (from Clemmens et al, 1997).

Figure 5 shows the response for a pool entirely under backwater. Note that changes in downstream water level occur with several cycles of delays followed by rapid changes. These are the result of oscillation waves within the pool that are reflecting off the boundaries. Wave celerity governs the period of these cycles.

The waves dampen quickly, and at long times, the change in water level over time is essentially linear. A straight line fit to the data shown in Figure 5 intersects the initial water level at approximately time zero, suggesting that the approximate model by Schuurmans et al (1995) is reasonable.

Simple Open-Loop Routing Method

Bautista and Clemmens (1998) developed a simplified routing scheme based on required volume changes and kinematic and dynamic wave velocities. The approach is outlined in Figure 6. It starts with determining the change in volume required, ΔV, to go from the initial steady flow rate, Q_i, to the final steady flow rate, Q_f, resulting from a requested flow change, Δq. Next the travel time for a wave to go from the upstream end to the downstream end of the pool is determined, $\Delta\tau$. The initial change in flow rate upstream, $\Delta Q(t_1)$, is computed as the needed change in volume, ΔV, divided by the travel time, $\Delta\tau$. This change in flow rate may be different from the requested flow change. In this case, a second flow change, $\Delta Q(t_2)$, is made upstream so that inflow and outflow balance. The assumption behind this method is that if the correct volume is applied and inflow matches outflow, the pool will eventually stabilize itself with the correct volume and flow rates. Simulation studies performed suggest that this is the case.

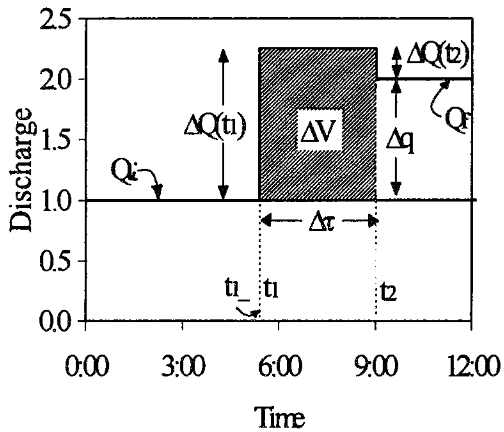


Fig. 6. Check flow change schedule from volume compensation and time delay.

For multiple pools, the volumes and delay times are summed from downstream to upstream. The cumulative volume is divided by the cumulative delay time to arrive at the flow change for each structure. If multiple flow changes are desired, check flow changes are computed for each. These incremental changes are then overlapped. For example, one requested flow change may compute a change of +100 l/s at 12:45 pm and -10 l/s at 3:00 pm. Another requested flow change may compute a change of +200 l/s at 2:00 pm and -40 l/s at 3:30 pm. An example of multiple flow changes is given in Figures 7 and 8.

In order to implement this method, pool volumes must be determined for various combinations of 1) flow rate, 2) downstream set-point level, and 3) Manning n. Volume as a function of these variables can be determined from computed backwater curves and canal geometry, or from simulation with steady hydraulic

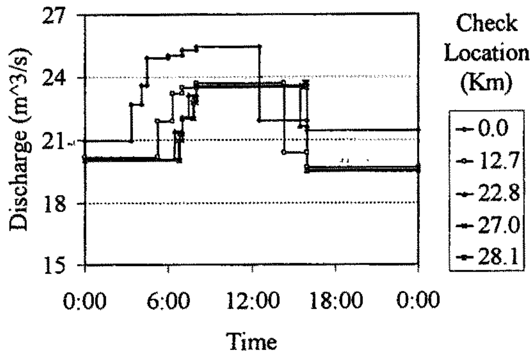


Fig. 7. Computed check flow schedules.

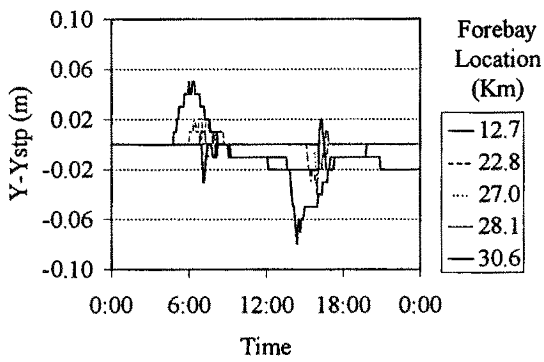


Fig. 8. Simulated forebay water level variations.

models (e.g., HEC-RAS). This scheme can also be used to implement changes in water level set-point.

The integrator-delay model of Schuurmans et al. (1995) suggests using a delay or travel time of zero for pools under backwater. This is supported by the long-term pool response. However, there is a finite delay in these pool, at least initially. Our experience with simulation of this procedure is that it seems to work best when we use a delay time equal to ½ that determined from the speed of a celerity wave for pools or portions of pools under backwater. For these pool sections, celerity should be computed with an "average" depth over the portion under backwater.

Downstream Feedback Control of Water Levels

Without some form of downstream control, there is no way of controlling the water delivery to users. Local manual and supervisory control systems use some form of manual downstream control to make adjustments when the system is "out-of-balance." Automatic downstream control systems serve the same purpose. They adjust the system for mismatches in inflow and outflow. They do this is such a way that the proper volumes are added to the system. Downstream control is useful even if all demand changes are prescheduled.

Many of the older automatic downstream control systems have been developed under the assumption that all demanded flow changes downstream control can handled. For most canals, this is simply not possible. Pool delay times preclude large demand changes from being implemented from the downstream end without anticipation and routing. The problem is that many of these older downstream control scheme are not set up to handle simultaneous routing of demand changes. This is a major weakness and has resulted in these systems being shut off frequently, for example when demand changes are large. The ability to combine open-loop routing of flow changes with feedback control of downstream water levels is essential for the effective wide-scale implementation of canal automation.

Feedback control of downstream water levels on canals with many pools and long delay times is usually required to be relatively damped to insure stability, and to reduce unnecessary oscillations in canal inflow and water levels. Disturbances behave differently in normal-depth pools than in backwater pools and must be handled differently in the feedback control system design (just as they are handled differently in open-loop routing). In the normal-depth sections, disturbances essentially travel only in the downstream direction. While in backwater pools, disturbances can travel in both directions and reflect at the boundaries. If improperly designed, the feedback can produce oscillations or even instability.

An important issue for water level feedback on canals with many pools is whether to use local feedback controllers (e.g., ELFLO or BIVAL) or more centralized controllers. Simple, local feedback controllers may have very limited performance for some canals (Schuurmans, 1992). However, centralized controllers are often too complex and too much like a black box, such that controller performance may be somewhat unpredictable. There is a strong reluctance to actually implement some of these controllers because of their complexity. A new downstream-water-level-control method with an intermediate level of complexity has been proposed (Clemmens et al. 1997, based on Schuurmans, 1995). It has been combined with other control features into an overall scheme, discussed below. Research is ongoing in this area.

USWCL AUTOMATIC CONTROL SCHEME

The staff at the U.S. Water Conservation Laboratory (USWCL) has developed a control scheme that is based on the integration of automatic controls with existing, manual operations. It allows one to take a more systematic approach to canal automation. It uses optimization to develop feedback controller components and attempts to maintain simplicity and understandability. It has three components:

1. open-loop control of flow rate and volume based on hydraulic routing,
2. closed-loop control of (distant) downstream water levels, and
3. local closed-loop control of check-structure flow rate based on 1 and 2.

Routing of flow changes is required because many canals have insufficient storage to provide adequate control with downstream feedback control alone. Feedback control of downstream water levels is necessary, even when demand changes are made by routing, since flow rates set at check structures always contain errors and since routing is never perfect. Check structure flow rate control provides some advantages for both open-loop routing and downstream feedback control.

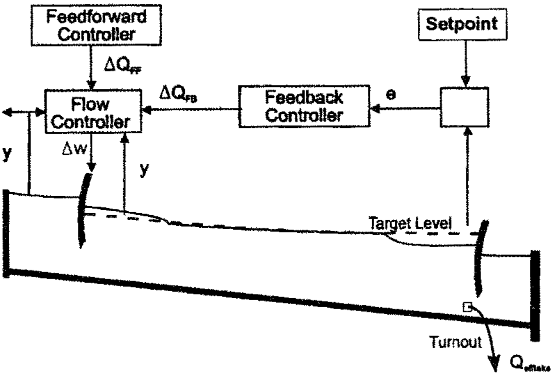


Fig. 9. USWCL canal control scheme.

It also allows the two to be easily combined. The general scheme for one pool is shown in Figure 9. Manual controls can also be done simultaneously with automatic control — that is, the automatic control does not have to be shut off to make manual changes.

Implementation

The wide-spread implementation of canal automation depends upon its being integrated with the overall operation of the district. Several research projects are ongoing to provide the needed integration. A pilot project on canal automation was initiated by the Salt River Project (SRP). Under this pilot project, the USWCL canal automation scheme will be tested in real time. During Phase I, completed in March 1997, simulation tests were run to determine whether the automatic control system could handle typical SRP control situations. This phase was very successful and the control system is now being implemented on SRP's SCADA system during Phase II, scheduled for completion in December 1998. Real time testing will begin in 1999 under Phase III. If successful, the system will be expanded.

A cooperative research and development agreement was established with Automata, Inc. to jointly develop a canal automation product line based on the USWCL control scheme. The intent is to try to make this system *Plug-and-Play*. Initial testing of Automata's system is being done on Maricopa Stanfield Irrigation and Drainage District's WM canal. This system should also be ready for real-time testing in late 1998 or early 1999. If these two efforts are successful, canal automation may quickly become a useful and powerful tool for modernization of irrigation water delivery systems.

REFERENCES

Bautista, E., A.J. Clemmens and T.S. Strelkoff. 1997 Comparison of numerical procedures for gate stroking. J. Irrig. And Drain. Eng 123(2):129-136.

Bautista, E. and Clemmens, A.J., 1998. An open-loop control system for open-channel flows. Proceedings, 1998 Intern. Water Res. Engr Conf. ASCE, Memphis, TN, Aug. 3-7.

Brouwer, M. 1997. *Performance analysis of Automatic Control of the Upper Arizona Canal.* M.S. Thesis, Delft University of Technology, Delft, The Netherlands.

Burt, C.M., Styles, S.W., Fidell, M. and Reisfsnider, E. 1997. Irrigation district modernization for the western U.S. p.677-682 Proceedings of Theme A, 27th IAHR Congress, San Frans., CA

Clemmens, A.J., Bautista, E. and Strand, R.J. 1997. Canal Automation Pilot Project. Phase I Report prepared for the Salt River Project. WCL Report #22, U.S. Water Cons. Lab., Phx, AZ.

Clemmens, A.J., G. Sloan and J. Schuurmans. 1994. Canal control needs: example. *J. Irrig. and Drain. Engr.* 120:6:1067-1085.

Favley, H.T. and Lunig, P.C. 1979. Gate Stroking. Report REC-ERC 79-7, U.S. Bureau of Reclamation, Denver, CO.

Johnston, W.R. and Robertson, J.B. (ed.) 1990. Management, Operation and Maintenance of Irrigation and Drainage Systems. Manual #57. ASCE, Reston, VI

Lamacq, S. 1997. *Coordination between Water Supply and Demands in Irrigated Systems. Scenarios, Systems and People.* PhD thesis, ENGREF, Montpelier, France.

Malaterre, P.O, Rogers, D.C., and J. Schuurmans. 1998. Classification of Canal Control Algorithms, J. Irrigation and Drainage Engineering, 124(1):3-10.

Palmer, J.D., A.J. Clemmens, A.R. Dedrick, J.A. Replogle, and W. Clyma. 1991. Delivery system performance case study: Wellton-Mohawk Irrigation and Drainage District, USA. Irrig. & Drain. Syst. 5:89-109.

Rogers, D.C. and Goussard, J. 1998. Canal Control Algorithms Currently in Use. J. Irrigation and Drainage Engineering, 124(1):11-15.

Schuurmans, J. 1992. Controller design for a regional downstream controlled canal. Report No. A668, Lab. For Measurement and Control, Delft Univ. of Technology. Delft, The Netherlands.

Schuurmans, J. 1995. Control of Open-Channels -- Concepts. Delft Univ. of Technology. Delft, The Netherlands. Unpublished.

Schuurmans, J., Bosgra, O.H., and Brouwer. 1995. Open-channel flow model for controller design. Appl. Math. Modeling. 19: 525-530.

Strelkoff, T.S., Deltour, J.L., Burt, C.M., Clemmens, A.J. and Baume, J. P. 1998. Influence of canal geometry and dynamics on controllability. J. Irrig. and Drain. Engineering, 124(1):16-22.

Wiley, E.B. 1969. Control of transient free-surface flow. J. Hydraulics Div., ASCE, 95(HYD1):347-361.

PHYSICAL AND OPERATIONAL IMPROVEMENTS THAT AID MODERNIZATION OF IRRIGATION DELIVERY SYSTEMS

John A. Replogle[1]

ABSTRACT

Because many irrigation systems throughout the world are well into the late years of their economic design life, efforts presently are toward their rehabilitation and modernization. Modernization is widely recommended over simply rehabilitating to the original status. Several goals of modernization include providing flexible water delivery service that is responsive to modern, on-farm irrigation systems, such as drip, sprinkler, level basin, and surge. To adopt many of these technologies implies that the farm unit can control the water supply by being able to start and stop the delivery at will, or at least negotiate or specify start and stop times. This often is well served by canal automation, a tool that will play a prominent role in improving the operation of delivery systems. Automation techniques are still under development and may still be prohibitively costly for many applications. However, there are a number of structural and management changes that can be considered that require only limited automation, or no automation, and can still provide significant flexibility of irrigation delivery to the farm unit. These measures include combinations of canal level-control structures, field outlet structures, strategically placed off-line reservoirs, low-cost measurement devices, and canal operating procedures. How to retrofit these useable features into an existing system as part of the rehabilitation and modernization scheme is the major emphasis of the paper. These features usually improve the convenience of irrigation applications and can encourage better irrigation efficiency.

INTRODUCTION

Many of our world irrigation systems are well into the late years of their economic design life. Project reconstruction efforts are currently emphasizing rehabilitation and modernization. Modernization (upgrading to new performance criteria) is widely recommended over simply rehabilitating to the original operating condition (Burt, et al., 1997). The possible meaning for modernization of irrigation delivery systems covers a wide spectrum, ranging from simple mechanization to full automation of system operations. The specific form that modernization activity should encompass appears to be very site-specific. The decision is based, among other things, on a combination of economic, social and educational structures.

---

[1]Research Hydraulic Engineer, U.S. Water Conservation Laboratory, 4331 E. Broadway Rd., Phoenix AZ 85040.

Most system modernization and rehabilitation projects involve canal delivery systems, and less frequently involve pipeline distribution systems. Infrequently, the project may involve changing from a canal distribution system to a pipeline distribution system. Sometimes the work will involve either a hybrid system where parts of the old project are converted to pipelines and the remaining canals are refurbished in some manner. By far the majority of projects will involve upgrading canal distribution systems without any type of conversion to pipelines.

BACKGROUND AND SCOPE

Several goals of modernization include providing flexible water delivery policies that are responsive to modern, on-farm irrigation systems, such as drip, sprinkler, level basin, and surge. The needs of these systems obviously differ. Drip systems benefit from a low delivery rate on almost a continuous basis. Sprinklers operate with medium, steady flow rates over extended periods. Level basins work well with large flow rates that may last a few minutes to a few hours. Surge irrigation systems need special variations of flow rate at the field level, but, as with most all systems, benefit from nearly constant delivery rates furnished to the farm. Serving these various farm needs simultaneously usually exceeds the capabilities of the canal delivery system.

Providing tools to projects that would enable improved delivery capability and thus the ability to offer flexible delivery policies is challenging. It usually implies that the farm unit can control the water supply by being able to start and stop the delivery at will, or at least negotiate or specify start and stop times. This in turn often requires automation of the delivery system for physical implementation. Efforts are growing to make automation economical, reliable, and practical. Canal automation is a tool that will play a prominent role in improving the operation of delivery systems. Several of these automation techniques are still under development and may still be prohibitively costly for many applications, (Clemmens, et al., 1997).

There is, however, a middle ground between simple rehabilitation and full automation. That middle ground involves the structural and operational changes that might be beneficially imposed during a partial rehabilitation that would serve the purposes of modernization. Often these changes can be economically implemented to offer immediate operational benefits and still allow further mechanization and automation. Here, mechanization is defined as the mechanical follow-through of a stimulus initiated manually. Automation, on the other hand, may use the same mechanical follow-through, but is self initiating, that is, it requires no manual initiation to function, once installed. Currently, canal automation of more than individual structures, for the most part, implies computer control.

Supporting the idea that a manual system should be capable of later automation is desirable in the reverse sense. That is, it should be operational, in some fashion, if the automation fails. The inconvenience of no planned manual backup to an automated system was brought painfully to the forefront by the failure of the Denver Airport's train that stranded thousands in the Spring of 1998. No manual backup existed, not even a pedestrian walkway. Thus, automation of an irrigation project should include planned manual operation as backup insurance. This backup will probably mean temporary reversion to a non-flexible delivery service because some delivery techniques are too difficult to do without computer control.

There are a number of structural and management changes that can be considered that require only limited automation, or no automation, and can still provide significant flexibility of irrigation delivery to the farm unit. Retrofitting these changes into an existing system as part of the rehabilitation-modernization scheme is a major emphasis of this discussion. Most of the items to be discussed are known to veteran irrigation technologists. However, the lack of general application indicates a gap between general knowledge and field practice. It is this gap that this paper also addresses.

PREPARATION FOR AUTOMATION

What to Do until Automation Arrives

Several questions come to mind. What can be done when limited funding efforts allow only small amounts of annual work? What can be done with manually operated systems when automation is not forthcoming in the immediate future? How can these changes be implemented without jeopardizing future automation? A partial list of the control concepts, structural changes, and operational changes that need addressing includes:

- Flow measurement
- Priority for flow measurements
- Control concepts
- Isolating canal subsystems
- Selection of appropriate hydraulic control for the delivery function
- Improved operating function of check structures, or cross-regulators
- Operational management of check structures
- Application of canal level-control structures
- Use of reservoir and other storage possibilities
- Sediment transport considerations

Flow Measurement

The importance of flow measurement as a means for achieving flow control in canal systems has been generally supported by irrigation technologists and was specifically documented in a case study by Palmer, et al. (1991). Instilling a local interest in the flow measurements already available appeared to have a positive effect on water control, even without new structures. That study reinforced the premise that modernization and efforts to improve water control, even before automation arrives, depends heavily on water measurement.

For canal systems, long-throated flumes have emerged as the device of choice because of their accuracy, ease of construction and related costs, flexibility in size and channel-matching shapes, and small required-head drop, which allows their use in relatively flat canals. Design and selection procedures are described in Bos, et al. (1991); Replogle, et al. (1990); and USBR (1997). Computer modeling to obtain accurate calibration equations for almost any geometric shape of a long-throated flume is given in Clemmens, et al., 1993. This includes rectangular, triangular, trapezoidal, circular, parabolic, or complex cross-sections. More recently, small rectangular flumes with adjustable throats in a variety of sizes with flow rates up to about 1 m^3/s have become commercially available (Replogle, 1998). Because water measurement is well treated by these other sources, no attempt is made here to repeat or even summerize those efforts.

Priority for Flow Measurements

For most irrigation delivery systems, management is most convenient when canals can be controlled and monitored at all desired points. This is usually impractical. Thus, priority measurement and control points should be determined. High priority is usually at the headings of all canals, but these headings also have a priority. The main canal can usually be operated to meet criteria based on maintaining a certain flow depth, (e.g., full). If a constant and known delivery is provided at the head of secondary canals, it may be practical to subdivide the flow among two or more tertiary canals and sometimes practical to subdivide on down to the farm units with sufficient accuracy. Thus, secondary canals are first priority for flow measurement. Farm deliveries are usually considered the next priority, followed by measurements at the heading of the main canal and finally at intermediate points in the main canal. Automation schemes may alter these priorities.

Control Concepts

Canal operators throughout the world use several different hydraulic procedures and concepts to operate their systems. We will review some of these operational methods in terms of hydraulic theory and accuracy. The discussion is primarily directed to canal systems where the delivery to farms is through adjustable orifice gates that may be intentionally varied from irrigation to irrigation. Although this discussion is mostly qualitative, numerical examples are presented to illustrate limitations of some concepts. Specific limitations and actual operating procedures for selected canal systems are frequently developed through experience, or by using one of the newer canal modeling techniques (Clemmens, et al., 1997).

Two general canal control concepts available to a canal operator are *upstream control* and *downstream control* (Clemmens and Replogle, 1989). These designations are based on whether for a particular control structure or gate, the upstream or the downstream flow level is used to determine a response to adjust the structure or gate. Upstream control is the method most commonly used by canal operators. Upstream control can be implemented with manual, remote, or automatic gates; with oblique weirs and duckbill weirs; or with combinations of these (Walker, 1987). Most canals contain a series of check structures that the canal operator manipulates to control the water level at specific places in the canal. Alternately, rate of flow could be used if an appropriate measuring method is available. For most deliveries through irrigation structures, maintaining the level of water also maintains the rate of flow. Canal offtakes (turnouts) to laterals or smaller canals are generally located immediately upstream from the structure and usually depend on this assumption. Downstream control is less common than upstream control because it is more difficult to implement due to the usual need to monitor and control a flow level far downstream from the supply gate.

Isolating Canal Subsystems

It is desirable to isolate and localize backwater interactions between canal subsystems, such as the farm operations on a secondary canal, and the secondary canal operations on the main canal. Passive methods to achieve hydraulic isolation of system components include reservoirs, free overfalls or critical flow devices, long-sloping canals, and high-head orifices. Among active methods of isolation are float, electronic-based, and hydraulic-based control structures.

For example, farm operations far from the head of a secondary canal may not affect the flow rate through the lateral gate, but later when operations are nearer to the gate they may sometimes back water onto the gate and reduce flow rate. A simple overfall structure placed downstream of the gate that slightly raises the water surface so that it exceeds the expected changes in the backwater from farm operations will thus stabilize the discharge rate for all operations on that farm. Figure 1 illustrates a broad-crested weir serving this purpose when it is installed between the field and the gate and causes enough overfall to absorb field operational changes. This type of weir is particularly useful because it has a high submergence limit, which means it will remain insensitive to downstream changes in water level as long as it has more than one or two cm drop in head through it for the usual farm-sized delivery. This helps stabilize the flow in the supply canal and in turns provides more constant delivery to the farm. Note that sharp-crested weirs and high head orifices can be used if a large head drop can be tolerated.

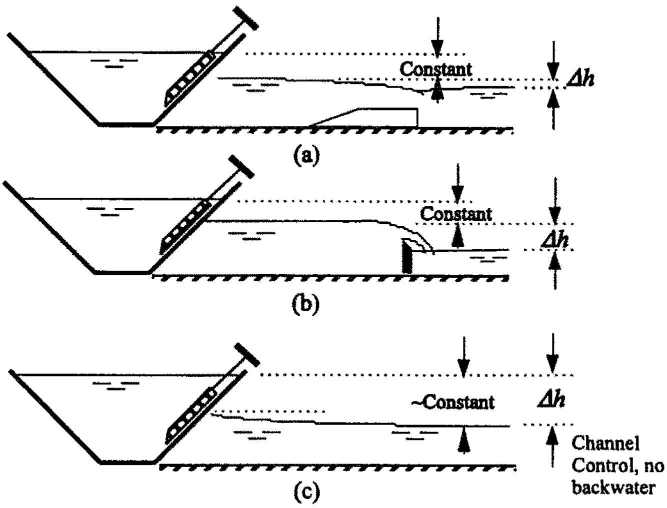


Fig. 1. A Long-throated Control Such as the Broad-crested Weir (A) Can Isolate the Small Canal from its Source Canal with Significantly Smaller Head Loss, Δh, than Can a Sharp-crested Weir (B) or an Orifice with a Large Differential Head (C).

Selection of Appropriate Hydraulic Control for the Delivery Function

Canal operations are greatly enhanced when the canal structure is appropriate to the needed function. In general, deliveries to secondary canals and to farms are served well with an orifice type of delivery. This takes advantage of the hydraulics of orifices and allows head fluctuations in the canal to have a small influence on the delivery rate. Structures in a canal that deliver flows further down the canal benefit by using overshot gates (Wahlin and Replogle, 1996) or duckbill weirs (Walker, 1987), to maintain a constant head on the farm delivery orifice. A discussion of the effects of overshot and undershot gates on achieving equilibrium after a change in flow at the upstream end is discussed by Strelkoff, et al., 1998.

Functions that can sometimes be combined, such as measurement and control, usually are best served by separate structures. For example, vertically moveable weirs (Bos, et al., 1991) can control a flow rate and serve to measure that rate. However, it is subject to errors caused by fluctuations in the delivery canal and is best suited to outlets from large reservoirs where the head does not change rapidly. Sometimes flow needs to be divided accurately, such as when it has been measured upstream with a flume at the heading of a secondary canal and must later be proportioned to two farmers. Proportional dividers can be fashioned from overfall or critical-depth weirs with a dividing board (Bos, 1989).

Exploiting Gate Hydraulics: Canal systems are usually operated to maintain a relatively constant level upstream from a cross-regulator, so that there is a constant differential head, Δh, on the delivery gate to the secondary canal or farm unit. Thus, the deliveries are constant. Most canal operators recognize that when the head drop, or differential head, through a rectangular slide gate remains constant for its range of opening (e.g., a free-discharging gate to a small canal from a large canal) each equal increment of gate opening, such as that caused by each revolution of the handle on a screw gate mechanism, adds approximately an equal increment to the discharge. Consider the simple orifice equation,

$$Q = C_d A_o \sqrt{2g\Delta h} \tag{1}$$

in which:

Q	=	Discharge rate
C_d	=	A discharge coefficient
A_o	=	Area of gate opening
g	=	Gravitational constant
Δh	=	Differential head.

If Δh remains constant, each complete revolution of the screw handle opens the gate an increment, ΔA. For each turn, the increment ΔQ is added to the discharge without the need to know the total discharge rate. In practice, if the operator knows from calibration procedures that each turn of the screw handle changes the flow by 10 l/s, then for a flow change of 60 l/s, the screw handle must be turned six revolutions to add the desired discharge increment. This same response is required whether the original opening provided 100 l/s or 1000 l/s.

Ignoring small changes in discharge coefficient, C_d, and changes in downstream head, the slide gate, under these special circumstances of constant head, responds linearly to gate opening. If the gate is submerged (backwater on the downstream face) then the gate is likely to respond nonlinearly. If the gate is not free flowing (submerged), then the flow per unit area of opening, is no longer linear. It is now a function of both gate opening and the square-root of differential head, Δh, which may vary with time and discharge.

Improved Operating Function of Check Structures

Large canals are often constructed with rather flat slopes of 0.0001 m/m. Because of their size, the velocities may still exceed 1 m/s, which is usually sufficient to transport moderate amounts of sediment. Check structures, sometimes called cross-regulators, or simply canal gates, interrupt the flow of bed load sediments, and can cause sediment aggradation. Some of the recommendations discussed here will need evaluating in terms of this problem if significant amounts of sediment must be transported.

These check structures can be sluice gates that discharge as an orifice. These and the related radial gates are sometimes referred to as *undershot* gates and their hydraulic response is that of an orifice, albeit with a variable coefficient. Other check structures include vertically adjusted leaf gates, which are basically hinged plates placed across the flow that may have a cable mechanism to lift the downstream edge. These are sometimes referred to as *overshot* gates (Wahlin and Replogle, 1996). These gates respond hydraulically as weirs, again with a variable coefficient. They can be manually operated or automated. If canal levels are to be maintained within a narrow margin, fixed structures that employ the overshot concept, such as long-crested, or duckbill, weirs, can be used (Walker, 1987).

Canal Water Level Control: Electronic-based and hydraulic-based canal water level controllers can provide constant discharge into downstream laterals from a source canal or reservoir with fluctuating flow depth. Many of these are described by Burt (1987), and by Burt and Plusquellec (1990). Float balanced gates, such as the Neyrtec Automatic gates are discussed by Goussard (1987), and the Danadian and DACL controlled-leak (hydraulic-based) controllers, by Clemmens and Replogle (1987), are specific examples of level controllers. Some level of automation can be attained by using these devices, but the response can be very slow because of hydraulic interaction between pools (Rogers and Goussard, 1998).

If a canal lateral is long enough to require several check structures then there are several ways that a canal operator can manipulate these structures. Some of these methods are best suited to "remote-monitored-and-remote-operated" techniques or to full canal automation. Others are also suited to manual control. Also, the delivery policy influences the operating procedure. For example, an arranged policy (Replogle, et al., 1980), where the flow rate and the delivery timing are negotiated between the irrigator and the delivery authority, generally allows a new flow delivery to be requested for a farm at any location from the canal head. This new flow rate must be added to the existing flow rate and passed through, or over, all intermediate check structures. Likewise, when honoring a request to stop flow at a far downstream location, the intermediate gates, as well as the head gate and the specific farm gate may need multiple adjustments.

Operational Management of Check Structures

Primary problems facing the canal operator include starting and stopping deliveries to farms along the canal without significantly interfering with deliveries to other farms, and achieving balance between the inflow into the canal and those deliveries out of the canal, because tail-end spills usually are to be avoided.

There are at least three methods of operations recognized in practice. These are:

Spilling over the gate or adjacent weirs: The lateral canals are constructed so that flows can be passed over weir walls on either side of the slide gate, or over the gate itself. In the overspill case the gate is adjusted so that a portion of the flow passes over the weirs. These overfall weirs can be oblique weirs, duckbill weirs, or when several are in parallel, labyrinth weirs.

The advantage of adjusting the gates to overspill some of the flow is that changes in demand for a downstream destination can be passed through the system with small changes in head at each gate because of the hydraulic characteristics of overspill weirs. This is illustrated later in Numerical Examples 1 and 2.

Operation at Several Centimeters Below Weir Crest: Operating below weir crest elevation is a method suited to automation or to remote operations. In this process the pools being used to make deliveries would be held as constant as practical by frequent gate adjustments, but the pools without deliveries are allowed to fluctuate to provide flow-rate buffering. It thus becomes possible to quickly start or stop a farm delivery anywhere in the entire length of the lateral. It appears practical to be able to start or stop a delivery about once every 3 or 4 hours, depending on the flow travel time for the canal. Several delivery changes, simultaneously, or in close time proximity, would be difficult to handle in this manner. Any new start or stop would require all intermediate gates to be adjusted one or more times. This may not be a problem for an automatic or remotely operated system, but would generate considerable canal-bank travel for a manually operated system. It has the advantage of being able to absorb both immediate shutoff and immediate turn-on conditions within limits of ability to absorb the storage. The system may fluctuate and require much practice for an operator to handle manually.

The pool mismatches can be corrected by changing the discharge from the lateral head at a prescribed rate. For example, it takes a flow rate increase of 10 l/s per meter of canal-surface width to raise the level of a kilometer of canal 1 cm in 1000 s (16.67 min). Thus, replacing 20 cm of water depth in a 1 km reach 3 meters wide, with a flow-rate increase of 30 l/s would require 20,000 s (5.5 hours). Conversely, 30 l/s could be drawn from the same pool for a period of 5.5 hours resulting in a drawdown of 20 cm.

Near-Spill Method: The near-spill method of operating the canal, i.e., operating the canal at nearly the brink of the weir, has been observed. The primary advantage claimed is that it allows the operator to quickly notice small changes in pool level and then make minor gate adjustments. It is possible for the operator to make an immediate start by borrowing from the full pools but does not allow immediate shutoff because the pools are already full. This method requires many gate adjustments for each flow change but does not provide for quick shutoff. Thus, it is more labor intensive than the overspill method and provides little added benefit.

Hybrid Method: The hybrid method has not been observed by the author. The hybrid method would include important canal reaches (pools with active farm deliveries) operated as over-fall pools, and reaches that have no active deliveries operated as buffering pools. Thus, the buffering pools could give up or absorb flow volume to accommodate short notice requests for either delivery or for shut-off. If enough pools exist, the shutoff at the farm and the shutoff at the head of the lateral can be separated in time far enough to allow travel of an operator between the two points. The several intermediate pools would provide the needed buffer.

When canals are steep, ponded storage is limited. Under these conditions, trying to begin or end a delivery to a farm by the operations suggested above may be difficult. Special computations have not been made to quantitatively evaluate the specific limitation for a selected canal. Qualitatively, the flow velocity in these canals is usually high and the elapsed time needed to get a flow delivered to the end of the lateral can be reasonably short, allowing relatively rapid response to requests to start or stop flows by only using upstream control concepts.

Application of Canal Level-Control Structures

Some practical considerations concerning the use of spill weirs as described above are illustrated in the following examples:

Numerical Example 1: Spill weirs might be practical when changing the overspill part of the canal discharge from about 15 l/s to 30 l/s for each meter of weir width. If we assume that these are usually concrete walls about 20 cm thick with 45°, 2.5 cm by 2.5 cm chamfered edges, then for crest depths of less than about 0.2 m, the discharge may be estimated per meter width as a broad-crested weir with a rounded nose, and as a sharp-crested weir for higher flows. The equation for unit-width discharge, q, for the lower flow range is given by Bos (1989) as

$$q = \frac{Q}{b_c} = C_d C_v \frac{2}{3}\sqrt{\frac{2}{3} g}\; h_1^{1.5} \qquad (2)$$

in which:

- Q = Discharge, m³/s
- b_c = Width of weir crest, m
- C_d = Discharge coefficient, assume to be approximately 0.95
- C_v = Velocity Coefficient, assume to be 1.01
- g = Gravitational constant, 9.8 m/s²
- h_1 = Head, referenced to elevation of the crest, m (for this example, h_1 is between 0.01 and 0.1 m)

Figure 2 shows an estimate of expected discharges throughout the broad-crested weir range and into the sharp-crested weir range (above about h_1 = 0.2 m). The upper end dashed curve is estimated using a sharp-crested weir equation and is used for freeboard estimates. Dotted vertical and horizontal lines indicate the part of the curve that is used in the Numerical Example.

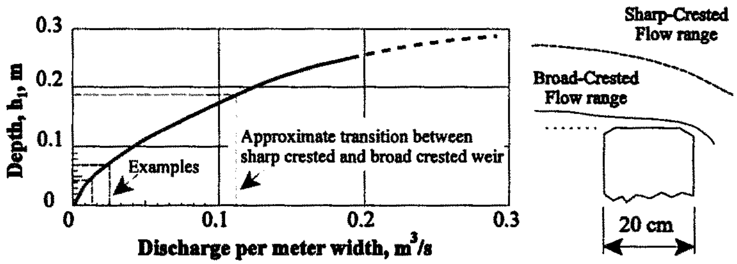


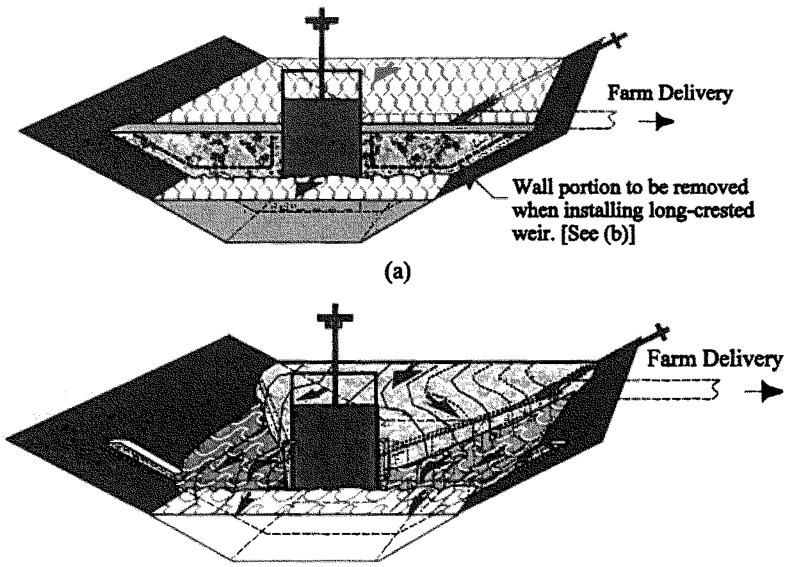
Fig. 2. Estimate of unit-width discharge rates for a weir wall 20 cm thick.

Doubling a flow rate from 15 to 30 l/s per meter of weir width would change the weir spill depth from about 4.5 cm to 7.0 cm (depth increase: 2.5 cm). The question then is whether this increase will significantly affect deliveries made through orifice gates to farms in the vicinity. For example, if it was desired that the 2.5 cm head change would cause less than a 5% change in flow rate, then the total head of the orifice needs to be greater than 25 cm. This is because orifices characteristically require 10% increase in head to cause 5% increase in a flow rate. If this differential head prevails, an effort to accommodate another farm delivery farther down the lateral can be accomplished without changes to the intermediate gates, depending on the widths of the overspill weirs and the flow change desired. This example illustrates that duckbill weirs, or oblique weirs that have extensive crest length, such as shown in Fig. 2, may be desirable if smaller changes in head and larger flow rates are needed.

Numerical Example 2: Assume that an operational goal is to have no more than 5% change in farm delivery discharge due to passing a new flow past the farm gate. Assume that multiple deliveries are in a canal with a maximum capacity of 4.0 m^3/s. What weir width would be needed to bypass another 300 l/s, if the sluice gate is adjusted to cause an overspill depth of about 1.5 cm, and the head drop through the farm gate is 20 cm? What if the over-fall depth is initially 10 cm?

The allowable head change to protect this farm delivery at the ±5% level is 10% of 20 cm, or 2 cm. Thus, the entire flow change must be passed with an increase in head from 1.5 cm to 3.5 cm. From Eq. (2) we find the increase in discharge per meter width to be about 7.7 l/s (Increasing from 3.0 l/s to 10.7 l/s). Thus, we need about 40 meters of oblique weir to accommodate these requirements. This length can be obtained as shown in Figure 3b.

If the overflow depth were initially 10 cm, again with an increase of 2 cm, the increased flow rate per meter width, by Eq. (2) is 16.3 l/s (51.7 l/s to 68.0 l/s), decreasing the required weir width to about 20 meters. This option significantly invades the canal freeboard, and thus should be carefully examined for safety considerations, unless it is deliberately planned to construct the weirs somewhat lower than the original weir walls. This might be an option if the desired flow capacity through the farm gates can be maintained during times when through-flow may be too small to match the original canal flow depth.



(b)

Fig. 3. A proposed modification to a canal check structure (a) to incorporate a long-crested weir, as in (b). The bypass weirs on either side of the gate are cut out and long weir crests are built at a diagonal upstream. Part of the flow is still passed under the original gate structure. New weir crests are made the same height, or slightly lower, than original crests.

This example illustrates some of the design parameters that may be considered to select options that may not otherwise produce desired results except through automation. Figure 3 shows one option of installing long oblique weirs in an existing canal. Each of the two oblique weirs would be 9.5 meters long, if we choose the deeper overfall-depth option of the above example. The cut-out notches made in the original cross wall need to pass about 625 l/s each (65.9 l/s per meter, 9.5 meters long). This would require each notch to be about 1 m wide and 0.5 m deep. Vertical free fall clearances for the diagonal-weir overfalls of about 2 cm to 5 cm should be added to this cut-out depth. Backwater from downstream structures may control the shape and amount of cut-out needed. Except for considerations of structural integrity, over sizing of the cut-out has no special consequence.

Use of Modified Gates for Canal Operations: Freeboard is frequently specified as 20% of maximum canal flow depth for sub-critical velocities. For a canal that flows 1 m deep, the constructed freeboard would likely be 20 cm. In the above example one option was to increase the flow depth over the weir from 10 cm to 12 cm. This would leave only 8 cm of freeboard, unless the weir crest is moved down. With no lowering, and because the depth is controlled by something as assured as a weir overfall, calculations of the increase in discharge needed to overflow the canal sidewalls would show that at 20 cm of head, the discharge is about 146 l/s per meter width, a total overspill of 2.9 m^3/s, plus any flow through the sluice gate. This is more than two times the flow rate used in the example. A major concern would be that the sluice gate is closed and the canal is presented with a flow of over 2.9 m^3/s to pass. Assuming a canal flowing 1.0 m deep 1:1.5 side slopes, and 1.0 m bottom width, the velocity would be about 1.16 m/s, representing a Froude Number of 0.47. By compromising, and moving the weir crest down half of the overflow depth, or 6 cm, we now can pass a depth of 26 cm, 217 l/s per meter width, or about 4.3 m^3/s, (velocity of 1.7 m/s, with a Froude Number of 0.7). Even though this is near, or in, the unstable region of canal Froude Numbers, it is not likely to occur except possibly for storm flooding.

Thus, a downstream delivery would involve opening the supply gate a prescribed amount, leave intermediate gates alone, and at an appropriate delay for flow travel time, open the gate at the downstream point of delivery. Shut-down would likewise involve closing the supply gate a prescribed amount and after a time delay, close the delivery gate at the farm.

Note that the entire process is upstream controlled and that the farm location in the system determines the time delay. This delay is also a function of total canal flow rate. There is no place near the delivery point to store an emergency shutoff. Emergencies will usually produce an operational spill. For manual operation by a ditch rider, this procedure requires at least one trip from the head gate to the delivery point for each change in flow rate (on or off), but would not require stopping at each gate along the way.

Use of Reservoir and Other Storage Possibilities

Main Canals as Buffer Reservoirs: When lateral canal inflows are automatically controlled, it is feasible to operate the main canal as an extension of the source reservoir. Due to the possibility of trapping water behind canal linings that can cause canal lining failure when the canal is rapidly emptied, most large canals are operated to decrease flow depth by less than 2 cm per hour, but they can be safely filled more rapidly. This change in storage capacity can often be used in lieu of off-line reservoirs. Additional check structures can often increase this reservoir storage. This storage can then facilitate manual operation and enhance future automation when it arrives.

Strategically Placed Reservoirs: Reservoirs can be strategically placed within the system to buffer operational flow mismatches (Marum and Styles, 1997). Usually these reservoirs should be close to the end of the project, but still command enough area to use collected flows. In areas of the world where night irrigation is not desired, these reservoirs would be more numerous and would collect enough flow overnight to irrigate the command area during daylight hours. Farm reservoirs can be used to collect the water delivered by a rigid supply schedule, and then the water can be regulated according to farm needs. An alternative is to place the regulating reservoir further upstream on the canal system, e.g., at the head of a secondary canal, which would isolate this canal and its several users from the rest of the larger system. While installing critical-depth, flow-measuring devices (weirs, flumes) at the head of secondary canals or downstream of all offtake gates on farm canals, can provide sufficient free overfall to isolate the source canal from the receiving canal, they only isolate flow influences in one direction. Automatic gates are particularly useful for maintaining rates at the headings of secondary canals (Clemmens and Replogle, 1987).

Sediment Transport Considerations

Lined canals with a water source from a reservoir usually do not have significant sediment problems. Direct diversions from rivers often have heavy sediment loads that usually are passed down to the farm. Flow control structures usually cause severe maintenance problems for these canals. Before changes are implemented, sediment handling considerations must be addressed.

CONCLUSIONS

Canal rehabilitation and modernization are usually done by a massive reconstruction effort. However, there is merit to including modernization as part of an annual maintenance program. Some of the things that can be accomplished include adding more check structures to shorten canal reaches, adding flow measuring flumes at the heading of each secondary canal, and changing gate handling operations to provide more equitable deliveries to farms far from the main canal. Automation of an irrigation project should include planned manual operation as backup insurance.

REFERENCES

Bos, M.G. (Editor). 1989. Discharge measurement structures. Third Revised Edition. Publication 20. International Institute for Land Reclamation and Improvement/ILRI, P.O. Box 45, 6700 AA Wageningen, The Netherlands. 401 p.

Bos, M.G., J.A. Replogle, and A.J. Clemmens. 1991. Flow Measuring Flumes for Open Channel Systems. 2nd Edition. American Society of Agricultural Engineers, St. Joseph, Michigan. 321 p.

Burt, C.M., S.W. Styles, M. Fidell, and E. Reifsnider. 1997. Irrigation District Modernization for the Western U.S. In: *Managing Water: Coping with Scarcity and Abundance.* Proceedings of Theme A (Marshall English, Ed.), The 27th Congress of the International Association for Hydraulic Research, 10-15 August, San Francisco, CA. Published by the American Society of Civil Engineers, New York. pp. 677-682.

Burt, Charles M. 1987. Overview of Canal Control Concepts. Proc. Symposium on Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems. (Zimbelman, D.D., Editor) sponsored by the American Society of Civil Engineers. Portland, OR. 28-30 July. pp. 81-109.

Burt, Charles M. and H.L. Plusquellec. 1990. Water Delivery Control. Chapter 11. In: Management of Farm Irrigation Systems (Hoffman, G., T. A. Howell, and K. H. Solomon, Editors). American Society of Agricultural Engineers, Monograph. pp. 373-423.

Clemmens, A.J., E. Bautista, and R.J. Strand. 1997. Implementation of canal automation in central Arizona. In: *Managing Water: Coping with Scarcity and Abundance.* Proceedings of Theme A (Marshall English, Ed.), The 27th Congress of the International Association for Hydraulic Research, 10-15 August, San Francisco, CA. Published by the American Society of Civil Engineers, New York. pp. 302-307.

Clemmens, A.J. and J.A. Replogle. 1987. Controlled-Leak Methods for Water Level Control. Proc. Symposium on Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, (Zimbelman, D.D., Editor) sponsored by the American Society of Civil Engineers. Portland, OR. 28-30 July. pp. 133-147.

Clemmens, A.J. and J.A. Replogle. 1989. Control of irrigation canal networks. Journ. of Irrigation and Drainage Engineering. American Society of Civil Engineers. 115(1):96-110.

Goussard, Jean. 1987. Neyrtec Automatic Equipment for Irrigation Canals. Proc. Symposium on Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, (Zimbelman, D.D., Editor), American Society of Civil Engineers. Portland, OR. 28-30 July. pp. 121-132.

Merriam, John L. and Stuart Styles. 1997. Use of Reservoirs and Large Capacity Distribution Systems to Simplify Flexible Operations. Part II-Reservoir Capacity and Case Studies. In: *Managing Water: Coping with Scarcity and Abundance*. Proceedings of Theme A (Marshall English, Ed.), The 27th Congress of the International Association for Hydraulic Research, 10-15 August, San Francisco, CA. Published by the American Society of Civil Engineers, New York. pp. 706-710.

Palmer, J.D., A.J. Clemmens, and A.R. Dedrick. 1991. Field study on irrigation delivery performance. Journal of Irrigation and Drainage Engineering, American Society of Civil Engineers. 117(4):567-577.

Replogle, J. A., Clemmens, A. J., and Bos, M. G. 1990. Measuring irrigation water. Chapter 10, p. 313-370. In: Management of Farm Irrigation Systems. T. A. Howell and K. H. Solomon G. Hoffman (editors). American Society of Agricultural Engineers. St. Joseph, MI. 1040 p.

Replogle, J.A. 1998. Measuring flows in earthen canals and irrigation wells. *Irrig. J.* 48(2):14-16.

Rogers, David C., and Jean Goussard. 1998. Canal Control Algorithms Currently in Use. Journal of Irrigation and Drainage Engineering, American Society of Civil Engineers. 124(1):11-15.

Strelkoff, T.S. and J.L. Deltour, C.M. Burt, A.J. Clemmens, and J.P. Baume. 1998. Influence of Canal Geometry and Dynamics on Controllability. Journal of Irrigation and Drainage Engineering, American Society of Civil Engineers, Water Resources Engineering Division. Vol. 124 (No. 1) pp.16-22.

USGS. 1980. National Handbook of Recommended Methods of Water-Data Acquisition. U.S. Geological Survey, Office of Water Data Coordination. Prepared Cooperatively by agencies of the United States Government. Government Printing Office , Washington DC.

Wahlin, B.T., and J.A. Replogle. 1996. Discharge characteristics of overshot gates. Proc. Int. Conf. on Water Resources Engineering, American Society of Civil Engineers. Session C-168. North American Water and Environment Congress, 24-28 June. Unpaginated (CD-ROM). 6 Pages.

Walker, Robert E. 1987. Long Crested Weirs. Proc. *Symposium on Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems*, (Zimbelman, D.D., Editor) sponsored by the American Society of Civil Engineers. Portland, Oregon. 28-30 July. pp. 110-120.

ESTIMATING THE COSTS OF FARM-LEVEL AND DISTRICT EFFORTS TO ACHIEVE SELENIUM LOAD TARGETS

Dennis Wichelns[1]
David Cone[2]
Shannon Fretwell[3]
Kristen Ward[4]

ABSTRACT

Seven irrigation and drainage districts in the California's San Joaquin Valley are participating in a regional program to reduce the load of selenium entering the San Joaquin River. Farmers have improved their irrigation methods to reduce surface runoff and deep percolation, district staff have improved their operations to support farm-level water management efforts, and a regional association has been formed to operate regional drainage facilities and coordinate efforts to achieve monthly and annual selenium load targets. The costs of these efforts include farm-level expenditures for new irrigation systems and for increases in irrigation labor and other water management inputs, district-level expenditures for new facilities and staff to manage water deliveries and drainage water reduction efforts more aggressively, and the fixed and variable costs of operating and maintaining regional drainage facilities. Empirical estimates of these costs for the Broadview Water District in 1997 include $93 per acre for farm-level irrigation improvements, $11 per acre for district-level efforts, and $14 per acre for supporting regional drainage activities.

INTRODUCTION

Much of the subsurface drain water collected beneath farmland on the west side of the San Joaquin Valley contains selenium, boron, and other elements that occur naturally in local soils and are leached from the profile during irrigation and drainage activities (Letey et al., 1986; Deverel and Gallanthine, 1988; Gilliom, 1991). The U.S. Environmental Protection Agency has established a national water quality criterion for selenium of 5 parts per billion (ppb), when measured as a 5-day moving average concentration. The California Regional Water Quality Control Board for the Central Valley Region has adopted a Basin Plan Amendment designed to achieve the national selenium concentration standard, over time.

[1] Assoc. Professor, Univ. of Rhode Island, Kingston, Rhode Island
[2] Manager, Broadview Water District, Firebaugh, California
[3] Water Management Specialist, Broadview Water District
[4] Graduate Research Assistant, Ag. Econ., Univ. of Calif., Davis

That amendment includes a set of monthly and annual selenium load targets that are expected to generate acceptable selenium concentrations in the near term, while farm-level and regional efforts are implemented, over time, to achieve the national water quality standard.

The Regional Water Quality Control Board has also established a 2-ppb selenium water quality objective for sloughs and channels in a wetland habitat located between an agricultural production area and the San Joaquin River. As a result, it became necessary to remove agricultural drainage water from wetland waterways, as selenium concentrations in drainage water are often in the range of 20 to 100 ppb (Deverel et al., 1984; Presser and Barnes, 1985). Agricultural districts were encouraged by water quality agencies to form a regional drainage organization and to develop a program for achieving the selenium load targets and removing drainage water from the wetland waterways.

Seven irrigation and drainage districts in the region have formed the Grassland Basin Drainage Activity Agreement to construct and operate regional drainage facilities and coordinate efforts to achieve the selenium load targets. The group has constructed a new channel that carries drainage water from all seven districts around the wetland area to an existing portion of the San Luis Drain, which carries the water to a tributary of the San Joaquin River. That program, known locally as the Grassland Bypass Project, began operations in September of 1996.

The monthly and annual selenium load targets, which became effective in October of 1996, are substantially lower than estimates of selenium discharges in recent years. For example, the monthly load targets are equivalent to less than 50% of the estimated discharges from the region during several months of the 1995 and 1996 crop years (please see the figure). The estimated annual load of selenium discharged in each of those years is almost twice the annual load target.

The monthly selenium load estimates for 1995 and 1996 reflect improvements in irrigation practices and drainage water management efforts that were implemented by farmers and districts in the early 1990s, when water supplies were reduced due to persistent drought conditions and changes in public policies regarding the allocation of water among urban, agricultural, and environmental uses (Wichelns and Cone, 1992a; Wichelns et al., 1996a). Further reductions in selenium loads may be possible, but the incremental cost of achieving those reductions will likely be substantial, as the most affordable improvements in water management practices have already been implemented.

FARM-LEVEL COSTS

The potentially restrictive selenium load targets, described above, have caused irrigation and drainage districts to implement additional programs to motivate further improvements in farm-level irrigation practices that will reduce surface runoff and deep percolation, and to increase the blending and re-use of drainage water with fresh water supplies. Innovations in district policies have included tiered water pricing, low-interest loans for purchasing new irrigation systems, and restrictions regarding surface runoff discharged into drainage ditches. Farmers have responded by replacing traditional surface irrigation methods with improved furrow methods, gated pipe, and sprinkler systems. They have also increased the labor and management components of irrigation activities.

The farm-level costs of using improved irrigation methods have been estimated by interviewing farmers in the 9000-acre Broadview Water District. Broadview implemented a tiered water pricing program in 1989 and has adopted additional incentive programs in recent years (Wichelns 1991; Wichelns and Cone 1992a and 1992b; Wichelns et al. 1996a). As a result, Broadview farmers have gained considerable experience in modifying irrigation methods to reduce surface runoff and deep percolation. Farmers have provided detailed estimates of the capital costs and variable costs of operating and maintaining siphon tube, gated pipe, and sprinkler systems (Wichelns et al., 1996b).

We compare the estimated annual costs of alternative irrigation methods for the four major crops grown in Broadview in Table 1. The estimated cost of using traditional siphon tube methods ranges from $148 per acre on seed alfalfa to $200 per acre on cotton, which requires more irrigations per season. Improved siphon tube methods include shorter irrigation run lengths and the use of a night irrigator to manage irrigation events carefully. The costs for gated pipe and sprinkler methods include a capital cost component and a larger labor requirement. Gated pipe must be carried into a field and removed at the end of an irrigation season, while portable sprinkler systems must be moved several times during each irrigation event. Detailed information regarding each cost component is provided in Wichelns et al. (1996b).

Prior to the 1990s, Broadview farmers used traditional siphon tube methods to irrigate most crops. They began improving irrigation methods in the early 1990s, in response to the district's tiered water pricing program and in an effort to maximize the value of surface water supplies that were limited by persistent drought conditions and by changes in public water allocation policies. Farmers have continued to use the improved methods as selenium load targets and uncertainty regarding their water supply have gained importance in recent years. It is unlikely that Broadview farmers will ever resume using traditional siphon tube methods.

We estimate the increased costs of irrigation by comparing the cost of irrigation improvements with the cost of using traditional siphon tube methods. For example, Broadview farmers irrigated 3,877 acres of cotton (63% of the total cotton area) with a combination of sprinklers and siphon tubes in 1997, while using improved siphon tube methods on 1,372 acres (Table 2). The estimated cost of using sprinklers and siphon tubes on those 3,877 acres exceeds the estimated cost of using traditional siphon tube methods by $321,217, while the estimated cost increase for the 1,372 acres irrigated with improved siphon tube methods is $50,193 (Table 3). The estimated increase in irrigation costs for all cotton fields is $494,090, or an average cost of $81 per acre.

Similar analysis of estimated irrigation costs for cantaloupes, processing tomatoes, and seed alfalfa generates a total estimated increase in irrigation costs of $824,566 for the Broadview Water District in 1997, or an average cost of $93 per acre. This result represents the estimated increase in farm-level irrigation costs made necessary by selenium load targets and other public policies regarding agricultural water supplies. While it is not possible to determine the effects of individual policies or water quality programs on farm-level irrigation costs, this estimate provides insight regarding the magnitude of cost increases that have occurred on farms on the west side of the San Joaquin Valley in recent years.

DISTRICT-LEVEL AND REGIONAL COSTS

Irrigation and drainage districts have implemented several programs to support farm-level irrigation improvement efforts. These programs raise the annual cost of district operations, resulting in higher farm-level assessments, service fees, or water prices. For example, several districts have implemented tiered water pricing programs that require more accurate measurement of water deliveries to farm turnouts than were necessary when all water was sold at the same price per unit. Districts have also increased the flexibility with which farmers may begin or terminate water deliveries, to accommodate sprinkler systems and shorter duration irrigation sets. The quality of water deliveries has also been improved to minimize potential problems with clogged sprinkler nozzles.

Most districts in the region have begun or expanded a recycling program to blend commingled drainage water with fresh water supplies, to reduce the volume of drainage water discharged into the regional drainage system. These programs require careful monitoring of delivered water salinity to ensure that crops are not damaged, particularly when farmers are using sprinklers during early-season irrigations. Several districts have modified their water delivery and drainage systems to optimize recycling opportunities. Additional staff have been hired to monitor water quality at several locations and to manage the blending of drainage water from many sources within a district.

All districts in the region have implemented water quality monitoring programs to estimate the loads of salt and selenium in drainage water at frequent intervals, to improve the information available to districts regarding short-term blending opportunities and long-term drainage water reduction strategies. The annual cost of collecting and analyzing water quality samples has increased substantially for all districts in the region.

Several districts have taken a leadership role in seeking a method to remove selenium from drainage water. The cost of innovative research efforts is often shared by all districts in the region, using a formula that accounts for each district's size and area served by subsurface drainage systems. Districts have also shared the costs of conducting numerous irrigation workshops and field demonstrations of farm-level techniques for reducing surface runoff and subsurface drain water.

We have estimated the annual district-level costs of these programs using data collected from the Broadview Water District. The estimated annual cost of the district manager's time in performing administrative tasks and attending meetings regarding district-level and regional drainage issues is $23,000. The estimated cost of staff time required to manage the district's 25 subsurface drainage systems, operate the drainage water recycling system, and monitor progress regarding monthly and annual selenium load targets is $44,000 per year. The estimated annual cost of collecting and analyzing water quality samples is $24,000. Broadview and other water districts contribute funds each year to support regional research projects regarding selenium removal from drainage water. The average annual contribution from Broadview is $10,000. The estimated total cost of these activities is $101,000 per year, or an average cost of $11 per acre (Table 4).

Broadview is one of seven irrigation and drainage districts participating in the Grassland Basin Drainage Activity Agreement, the regional authority that operates and maintains drainage facilities serving an area of 94,000 acres. The authority coordinates district-level efforts to achieve monthly and annual selenium load targets. It also conducts a regional water quality and toxicity monitoring program, and interacts frequently with state and federal agency staff regarding compliance with water quality objectives. The annual budget of the drainage authority is $865,575 or an average cost of about $9 per acre, though the average cost for each district varies according to the area served by drainage systems and other district characteristics.

SUMMARY

All of the expenditures required to improve water management practices and achieve regional water quality objectives are eventually paid by farmers in the region, either directly or through district fees and assessments. A summary of the estimated annual costs of these efforts for farmers in the Broadview Water District is presented in Table 5. The estimated annual cost of irrigation improvements in Broadview is $824,566, or an average cost of $93 per acre. Farmers in the six other districts participating in the Grassland Bypass Project have implemented similar improvements in irrigation practices. Therefore, an estimate of the regional cost of irrigation improvements is obtained by multiplying the average cost of $93 per acre in Broadview by the 94,000 acres in the region. This generates an estimated regional cost of irrigation improvements of $8.7 million per year (Table 5).

The estimated annual cost of district efforts to reduce drainage water volume and selenium loads is determined by multiplying the average cost of $11 per acre in Broadview by the 94,000 acres in the region, resulting in an estimated regional cost of $1,054,680 (Table 5). The annual budget of the regional drainage authority is $865,575, resulting in a estimated total regional cost of farm-level, district, and regional efforts of $10.6 million (Table 5), or an average cost of about $113 per acre.

The district-level and regional costs appearing in Table 5 are largely attributable to efforts to reduce drainage water volume and selenium loads, in response to the imposition of monthly and annual selenium load targets. However, some of the farm-level costs of irrigation improvements would likely be observed even in the absence of selenium load targets. Farmers in Broadview and other districts have improved irrigation practices for several reasons in the 1990s including drainage water reduction efforts, greater uncertainty regarding water supplies, and efforts to improve the efficiency of water use.

Many farmers in the San Joaquin Valley have developed innovative combinations of irrigation methods that generate better crop yields, while reducing the volume of water delivered to farm turnouts. The farm-level cost of irrigation has increased as a result of these improvements, while the yield per unit of water delivered has increased. Given the outlook for future water demand and supply in California, it is likely that farmers will continue using improved irrigation methods, provided that water supplies are sufficient to sustain profitable crop production activities and to maintain salt balance in the soil profile.

The estimated costs presented in Table 5 pertain to current activities implemented by farmers and districts to improve water management practices and achieve selenium load targets. A portion of those costs can be attributed to general irrigation improvement efforts made necessary by the changing allocation of water among competing uses in California. However, most of the improvements in irrigation practices and district operations are helpful in reducing surface runoff, deep percolation, and the load of selenium discharged to the San Joaquin River.

Estimates of long-term costs that may result from changes in irrigation and drainage management practices are not included in Table 5. For example, all of the participating districts have increased their recycling of saline drainage water to reduce the volume discharged into the regional drainage system. As a result, the average salinity of water deliveries has increased in the region, resulting in greater loads of salt deposited on farm fields. Soil salinity and boron concentrations will increase, over time, if farmers are unable to leach their fields with a frequency and volume sufficient to maintain salt balance.

ACKNOWLEDGMENTS

This paper is Rhode Island Agricultural Experiment Station Publication 3690. Support from the Water Conservation Office in the California Department of Water Resources is appreciated.

REFERENCES

Deverel, S.J., R.J. Gilliom, R. Fujii, J.A. Izbicki, and J.C. Fields, 1984. Areal distribution of selenium and other inorganic constituents in shallow ground water of the San Luis Drain Service Area, San Joaquin Valley, California - a preliminary study. U.S. Geological Survey Water-Resources Investigations Report 84-4319.

Deverel, S.J. and S.K. Gallanthine, 1988. Relation of salinity and selenium in shallow groundwater to hydrologic and geochemical processes, Western San Joaquin Valley, California. Open-File Report 88-336, U.S. Geological Survey, Sacramento, California, 23 pp.

Gilliom, R.J., 1991. Overview of sources, distribution, and mobility of selenium in the San Joaquin Valley, California. In: Dinar, A. and D. Zilberman (eds.), The Economics and Management of Water and Drainage in Agriculture. Kluwer Academic Publishers, Boston.

Letey, J., C. Roberts, M. Penberth, and C. Vasek, 1986. An agricultural dilemma: drainage water and toxics disposal in the San Joaquin Valley. Special Publication 3319, Division of Agriculture and Natural Resources, University of California, 56 pp.

Presser, T.S. and I. Barnes, 1985. Dissolved constituents including selenium in waters in the vicinity of Kesterson National Wildlife Refuge and the West Grassland, Fresno and Merced Counties, California. U.S. Geological Survey Water-Resources Investigations Report 85-4220.

Wichelns, D., 1991. Motivating reductions in drain water with block-rate prices for irrigation water. Water Resources Bulletin, 27(4):585-592.

Wichelns, D. and D. Cone, 1992a. Farm-level and district efforts to improve water management during drought. Irrigation and Drainage Systems, 6:189-199.

Wichelns, D. and D. Cone, 1992b. Tiered pricing motivates Californians to conserve water. Journal of Soil and Water Conservation, 47(2):139-144.

Wichelns, D., L. Houston, and D. Cone, 1996a. Economic incentives reduce irrigation deliveries and drain water volume. Irrigation and Drainage Systems, 10:131-141.

Wichelns, D., L. Houston, D. Cone, Q. Zhu, and J. Wilen, 1996b. Labor costs may offset water savings of sprinkler systems. California Agriculture 50(1):11-18.

Wichelns, D., L. Houston, and D. Cone, 1997. Economic analysis of sprinkler and siphon tube irrigation systems, with implications for public policies. Agricultural Water Management 32:259-273.

Table 1. Estimated annual irrigation costs for selected crops in California's San Joaquin Valley

Irrigation Method	Cotton	Cantaloupes	Processing Tomatoes	Seed Alfalfa
		(Dollars per Acre)		
Siphon tubes, traditional	200	162	189	148
Siphon tubes, improved	237	178	226	162
Gated pipe	310	255	301	225
Sprinklers and siphon tubes	283	223	332	204
Sprinklers and gated pipe	342	285	399	255
Sprinklers	370	n/a	n/a	n/a

Source: Wichelns, et al., 1996b. Original cost data have been converted from 1993 dollars to 1997 dollars using the producer price index for intermediate materials excluding foods. The appropriate conversion factor is 1.076.

Notes: Total annual costs include annual capital and maintenance costs, labor, and the cost of water and energy. Traditional siphon tube irrigation includes 1/2-mile furrows (cotton, seed alfalfa, and tomatoes) or 1/4-mile furrows (cantaloupes), with no night irrigator. Improved siphon tube irrigation includes 1/4-mile furrows (cotton and seed alfalfa) or 1/6-mile furrows (cantaloupes and tomatoes) and a night irrigator.

Labor costs are estimated using $7.80 per hour for irrigators and $26 per line for line movers, including wages and payroll taxes. The cost of water is $40 per acre-foot and the cost of energy is $10 per acre-foot of water delivered.

Sprinklers cannot be used throughout the season on cantaloupes, tomatoes, or seed alfalfa.

Table 2. Summary of irrigation methods used by farmers in the Broadview Water District, 1997

Irrigation Method	Cotton	Cantaloupes	Processing Tomatoes	Seed Alfalfa
		(Acres)		
Siphon tubes, improved	1,372	303	0	313
Gated pipe	150	146	0	150
Sprinklers and siphon tubes	3,877	0	73	0
Sprinklers and gated pipe	524	760	799	237
Sprinklers	187	0	0	0
Total Area	6,110	1,209	872	700

Notes: Combinations of irrigation systems are used in sequence. For example, farmers use sprinklers for early irrigations of cotton and tomatoes, before switching to siphon tubes or gated pipe for later irrigations. In addition, many farmers who use sprinklers to pre-irrigate cantaloupe and cotton fields will use surface methods for seasonal irrigations.

Table 3. Estimated increases in the annual costs of irrigation, by crop and irrigation method, in the Broadview Water District, 1997

Irrigation Method	Cotton	Cantaloupes	Processing Tomatoes	Seed Alfalfa
		(Dollars)		
Siphon tubes, improved	50,193	4,564	0	4,378
Gated pipe	16,463	13,510	0	11,459
Sprinklers and siphon tubes	321,217	0	10,447	0
Sprinklers and gated pipe	74,425	93,225	167,646	25,246
Sprinklers	31,791	0	0	0
Total Increase	494,090	111,299	178,093	41,084

Notes: Cost increases are estimated by subtracting the annual, per-acre cost of using siphon tubes, traditional (Table 1) from the annual per-acre cost of using each alternative irrigation method (Table 1), and multiplying the per-acre difference by the number of acres planted in each crop (Table 2). The sum of estimated total costs for these four crops is $824,566, or an average of $92.74 per acre.

Table 4. Estimated annual costs of district efforts to reduce selenium loads in the Broadview Water District, 1997

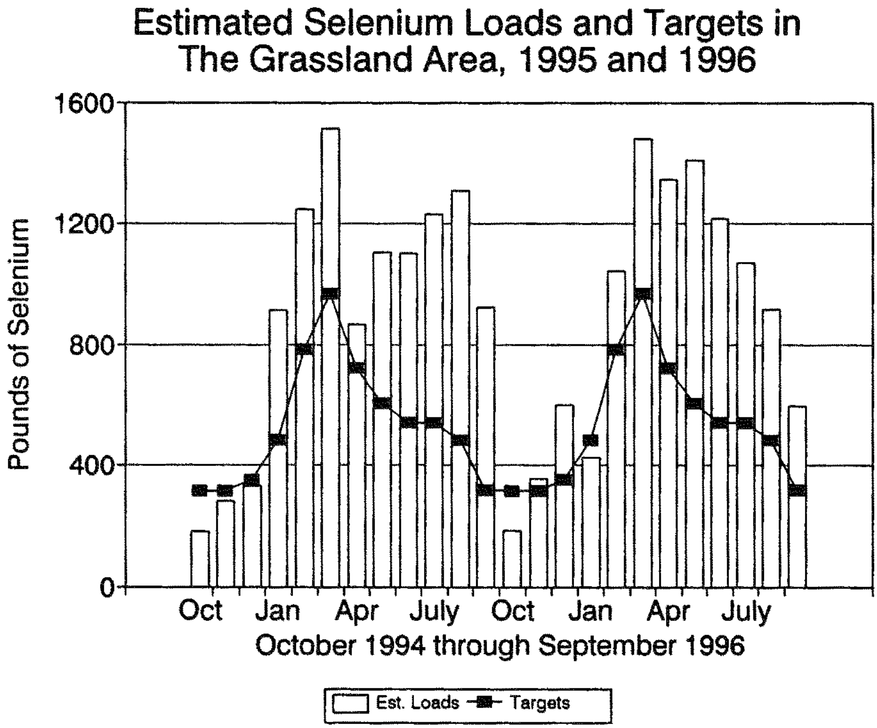
Item	Description	Estimated Annual Cost
		(Dollars)
Manager's time	Administrative tasks regarding drainage issues	23,000
Staff time	Re-design and operation of sumps and recycling system	44,000
Water Quality Program	Frequent sample collection and laboratory analysis	24,000
Selenium Removal Research Support	Contributions to regional research program	10,000
Total Cost		101,000

Notes: The estimated cost of support for regional research projects varies, over time and among districts, with changes in the research program.

Table 5. Summary of estimated annual farm-level, district, and regional costs to reduce selenium loads entering the San Joaquin River, 1997

Item	Broadview Water District Total Cost	Broadview Water District Average	Grassland Bypass Project Area
	(Dollars)	($/Acre)	(Dollars)
Farm-level irrigation improvements	824,566	92.74	8,717,560
District efforts	101,000	11.22	1,054,680
Regional Drainage Authority	124,338	13.82	865,575
Total Cost	1,049,904	117.78	10,637,815

Notes: Estimated annual costs of farm-level and district efforts in the Grassland Bypass Project area are determined by multiplying the estimated average annual cost for Broadview Water District by the 94,000 acres participating in the Bypass Project. The estimated cost of the Regional Drainage Authority is obtained from the annual budget of that organization. The Broadview portion of that budget is determined by a formula that allocates costs among the seven participating districts according to the drainage characteristics of each district.


Estimated Selenium Loads and Targets in
The Grassland Area, 1995 and 1996
1600
1200
800
400
0
Pounds of Selenium
Oct
Jan
Apr
July
Oct
Jan
Apr
July
October 1994 through September 1996
Est. Loads
Targets

STRATEGIES FOR UTILIZING SHALLOW GROUND WATER IN ARID AREAS

James E. Ayars[1] Richard A. Schoneman[2]

Richard W. Soppe[3]

ABSTRACT

Shallow ground water in arid irrigated areas has generally been treated as a waste product of irrigation which was to be discharged into an available water course for ultimate disposal in an ocean. This practice is no longer environmentally acceptable and means need to be developed to minimize the environmental impact of uncontrolled discharge of drainage water from irrigated lands. This paper presents the results of field and theoretical studies which demonstrate methods to reduce and minimize the volume of drainage water for disposal. The field studies demonstrated the use of subsurface drip irrigation with modified crop coefficients to increase the water use from shallow ground water, and the use of control structures on drainage systems to control the depth to shallow ground water to improve the water use by the crop from shallow ground water. Application of these techniques resulted in significant use of ground water by cotton and tomato. The theoretical studies demonstrated that using new drainage design criteria will result in less drainage discharge and lower salt loads. Improved irrigation efficiency will have the largest impact on reducing total drainage discharge.

INTRODUCTION

[1]Research Agricultural Engineer, USDA-ARS, Water Management Research Lab. 2021 South Peach Avenue, Fresno, CA 93727

[2]Agricultural Engineer, USDA-ARS, Water Management Research Laboratory, 2021 South Peach Avenue, Fresno, CA 93727

[3]Visiting Scientist, USDA-ARS, Water Management Research Laboratory, 2021 South Peach Avenue, Fresno, CA 93727

Shallow ground water in arid irrigated areas is generally considered a problem which is corrected using a subsurface drainage system. The conventional wisdom in irrigation and drainage system design for arid areas is that a drainage system is necessary to prevent water logging and to provide leaching of salt from the soil profile. The salt originates from sources such as the irrigation water, fertilizers, and the parent soil materials (Ayars et al, 1987). Disposal of the drainage water containing salt and other naturally occurring elements is one of the major problems facing irrigated agriculture throughout the world. (Ayars and Meek, 1994).

The San Joaquin Valley Drainage Program report (1990) identified the following methods to manage subsurface drainage water; source control, supplemental irrigation with collected drainage water, in-situ use of shallow ground water by salt tolerant crops, and land retirement.

Source control consists of reducing the deep percolation losses from irrigation by conversion of irrigation systems, improved irrigation scheduling, and improved system management. Conversion of irrigation systems includes using gated pipe instead of siphons, using sprinklers instead of surface irrigation methods, or using micro-irrigation instead of surface or sprinklers. In each case the adoption of a new irrigation method should allow for better control of the applied water and reduced deep percolation.

It also comes with a price. The purchase cost of the new irrigation system is the most obvious, but there is also a price associated with learning to manage a new irrigation system. The techniques for scheduling and controlling the water are different than with the old system. If this is ignored then nothing will be gained. An extreme example would be to manage a drip system in the same fashion as a surface irrigation system. It would be difficult to get the water applied in a timely manner with the drip system, and there would be deep percolation associated with the excess application of water at a point.

Improved irrigation scheduling will result in application of the required amount of water at the appropriate time. This is most effective when implemented in conjunction with improvements in the irrigation system management. Irrigation at the beginning of the growing season is the most difficult time to manage water to reduce deep percolation. It is often the case with annual crops that only 1 to 2 inches of water are needed for the first irrigation. Surface systems such as furrow and flood generally can not apply this small an amount of water and the excess water results in deep percolation. Studies in the San Joaquin Valley have shown that the first irrigation of the season has the poorest efficiency and the largest deep percolation values (Ayars and Schoneman, 1991). It is difficult to use a water balance method to schedule irrigation in the presence of shallow ground water because the ground water contribution is unknown.

Supplemental irrigation of salt tolerant crops has been touted a method to reduce the total volume of drainage water for disposal. Studies by Ayars et al. (1993) and Rhoades et al. (1989) have demonstrated the feasibility of using saline water on salt tolerant crops such as cotton and sugar beet and in a rotation with salt sensitive and moderately tolerant crops (Rhoades et al, 1989). In a five year study Ayars et al. (1993) estimated that nearly 70 inches of saline water were used as supplemental irrigation on salt tolerant crops. However, the use resulted in an increase in the salinity and boron concentrations in the soil profile such that it would require nearly 70 inches of good quality water to return the boron concentration to the previous levels. The negative impact of using saline water is the application of salt and other potentially toxic elements to the soil surface and subsequently to the soil profile.

In-situ use of saline water has the potential to lower the volume of water for disposal while minimizing the environmental impact of accumulation of salt, boron, and other elements in the upper part of the soil profile. This technique is not as effective as the surface application of irrigation water since there is a time lag before the plant has developed a large enough root system to begin extracting water from the shallow ground water. The total extraction from ground water depends on the depth to the water table, the ground water quality, and the crop salt tolerance. Cotton potentially can extract up to 45% of its water requirement from ground water with an electrical conductivity of 7 mmho/cm at a depth of 4 feet. Tomato will extract from 30 to 45% of its water requirement from ground water with an electrical conductivity of 0.3 to 5 mmho/cm at a depth of 4 feet. The challenge is to manage the irrigation system to achieve these levels of use. This compares to supplemental irrigation where nearly all the water requirement after crop emergence can be met with saline irrigation water.

All of these techniques either have been or are currently being investigated as methods to enable irrigated agriculture to survive while long term sustainable solutions are developed. This paper will discuss both field and theoretical studies which have applied new concepts to the integrated design and management of irrigation and drainage systems.

STUDIES

Britz Farms

A field study was conducted in the San Joaquin Valley on 320 acres which are underlain by ground water at a depth of 4 to 8 feet depending on the time of year. The shallowest depth to ground water occurs during the winter and spring following rainfall and pre-plant irrigation which occurs during the winter and early spring in this area. A subsurface drip irrigation system (SDI) was installed on two

30 acre parcels, one on each of the quarter sections used in the research. The SDI system was made up of 5 individual systems each of approximately 6 acres and each containing a different type of drip tubing (Roberts, Ram, Chapin, Typhoon, T-System)[4]. The lateral spacing in one 30 acre block was 80 inches and 66 inches in the second block. All drip tubing was installed at a depth of approximately 15 inches below the soil surface. The crop rotations were tomato, cotton, tomato on the block with the 66 inch spacing , and cotton, cotton, cotton on the block with the 80 inch spacing.

The remainder of each quarter section was irrigated using furrow irrigation from gated aluminum pipe. Sprinkler irrigation was used to germinate the tomato crop each year and was followed by furrow irrigation for the remainder of the season. Irrigation scheduling on the furrow plots was the responsibility of the cooperator and the management of the SDI system was the responsibility of the Water Management Research Laboratory. The drip systems were scheduling using evaporation data from an on-site evaporation pan, a pan coefficient developed using weather data collected from a California Irrigation Management Information System (CIMIS) weather station located approximately 3 miles from the site, and a crop coefficient. The cotton crop coefficient was developed by Ayars and Hutmacher (1994) to account for the use of ground water by cotton. Irrigation was scheduled to be applied when approximately 0.16 inches of evapotranspiration had accumulated. The system was run up to twice a day. A locally developed crop coefficient was used for the tomato crop scheduling.

The water balance data of the Britz site for 1992 and 1993 are given in tables 1 and 2, respectively. The Et_c was estimated using both total dry matter (TDM) (Davis, 1983) and with climate data taken from a CIMIS weather station. In 1992, the total water applied by the furrow system was slightly greater than the crop Et_c and resulted in some deep percolation. In the drip plots the high frequency irrigation coupled with the modified crop coefficient resulted in less total applied water than the furrow irrigated plots and resulted in a ground water contribution to the crop water use. In 1993 both the drip and furrow irrigated plots were under irrigated which resulted in substantial use of shallow ground water by the cotton crop.

The yield data for each of the three years is given in table 3. The data show that the yields were improving in the drip irrigated plots during the three years of the project. The average yields for the drip plots were 1230, 1590 and 1830 lb/ac in 1991, 1992, and 1993, respectively. The yields in the furrow irrigated plot

---

4

Product names are given for the benefit of the reader and do not imply endorsement by the USDA-ARS.

Table 1. Water balance for the Britz Shallow Groundwater Management Demonstration Project (1992) for subsurface drip irrigation (Roberts, Ram, Chapin, Typhoon, T-Systems) and furrow irrigation.

Irrigation system	Soil Water Depletion (in)	Effective Rain (in)	Applied Water (in)	Total Dry Matter (t/ac)	Cotton Et_c TDM (in)	Groundwater contribution (%)
Furrow	1.40	0.12	18.7	4.3	17.2	-17
Roberts	-0.28	0.12	11.8	4.6	17.9	35
Ram	-0.28	0.12	14.8	6.2	22.7	35
Chapin	-0.28	0.12	15.0	5.9	22.3	33
Typhoon	-0.28	0.12	14.9	6.6	24.1	39
T-System	-0.28	0.12	15.3	6.4	21.0	28

Table 2. Water balance for the Britz Shallow Groundwater Management Demonstration Project (1993) for subsurface drip irrigation (Roberts, Ram, Chapin, Typhoon, T-Systems) and furrow irrigation.

	Seasonal					
Irrigation system	Applied Water (mm)	Effective Rain (mm)	Soil Water Depletion (mm)	Et_c CIMIS (mm)	Et_c TDM (mm)	Groundwater Contribution (%)
Furrow	13.2	0.0	0.55	22.7	25.4	40
Roberts	8.31	0.0	0.67	22.7	23.3	61
Ram	13.4	0.0	0.90	22.7	32.4	37
Chapin	14.4	0.0	0.94	22.7	29.5	33
Typhoon	12.1	0.0	0.79	22.7	25.9	43
T-System	11.5	0.0	0.63	22.7	24.8	47

remained constant during this time. The cotton yields in the furrow irrigated plots were typical of the previous production levels on this field.

The highest yield on all plots was obtained in 1993 on the SDI plot receiving the smallest amount of irrigation water during the irrigation season. The furrow plot yield in 1993 was comparable to that of 1991 in a situation with apparent under-irrigation and significant contribution from the ground water.

Table 3. Cotton yield of Britz Shallow Groundwater Management Demonstration Project in 1991, 1992, and 1993 for subsurface drip irrigation (Roberts, Ram, Chapin, Typhoon, T-Systems) and furrow irrigation .

Irrigation system	1991 (lb/ac)	1992 (lb/ac)	1993 (lb/ac)
Roberts	1250	1430	2100
Ram	1160	1520	1700
Chapin	1340	1600	2300
Typhoon	1160	1700	1520
T-Systems	1250	1700	1520
Furrow	1340	1250	1340

<u>Broadview Site</u>

A subsurface drain system of corrugated plastic tubing, which had previously been installed on 160 ac of land located in the Broadview Water District, CA, was modified to test concepts for water table control (1996). The system is laid out in a gridiron pattern with a total of seven laterals spaced approximately 400 ft apart. The lateral length is 2200 ft and the depth of installation is 7.8 ft. Butterfly valves were installed on each lateral at the juncture of the lateral and main collector line. Manholes with weir structures were installed at three locations along the main collector line. Schematic drawings of these control structures are shown in Fig. 1a and 1b and the field layout is shown in Fig. 2.

The installation of the control system was completed in April 1994. The site was sprinkle irrigated on 2/1, 3/1, and 3/14/94 and was planted to processing tomatoes (*Lycopersicon esculentum* var. APEX 1000) on February 14-16, 1994. Subsequent irrigation was by furrows with water delivered by gated pipe on 4/17, 5/25, 6/9, 6/17, and 6/25/94. Water was applied in every furrow which had a run length of 655 ft.

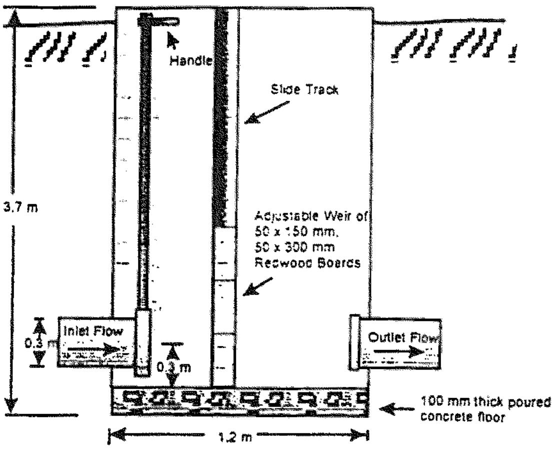


Fig. 1a. Schematic drawing of manhole and weir structure used on Broadview Shallow Ground Water Management Demonstration Project.

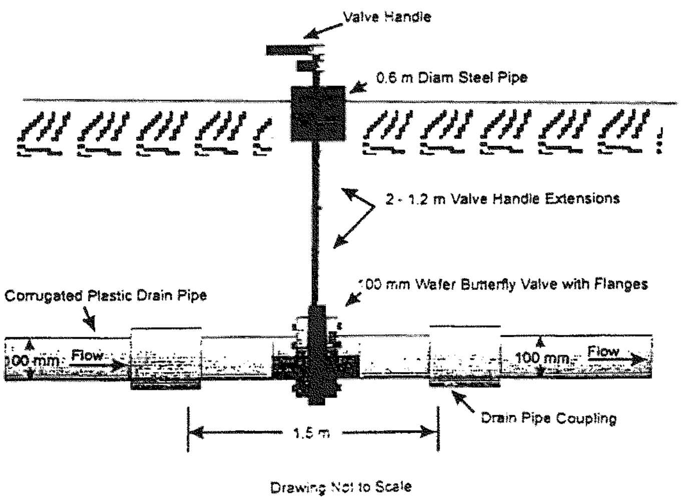


Fig. 1b. Schematic drawing of control valve used on subsurface drainage laterals in Broadview Shallow Ground Water Management Demonstration Project.

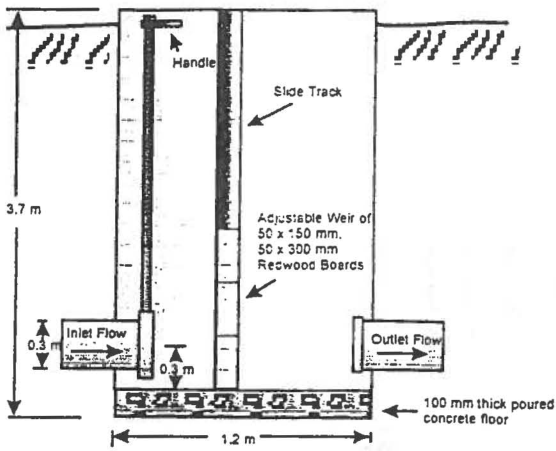


Fig. 1a. Schematic drawing of manhole and weir structure used on Broadview Shallow Ground Water Management Demonstration Project.

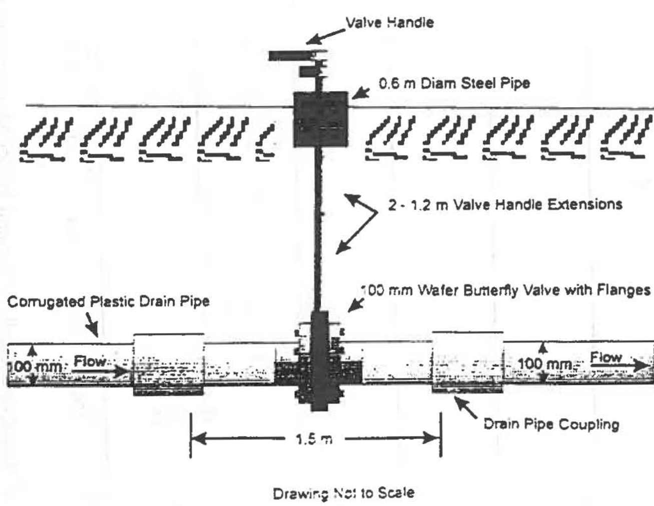


Fig. 1b. Schematic drawing of control valve used on subsurface drainage laterals in Broadview Shallow Ground Water Management Demonstration Project.

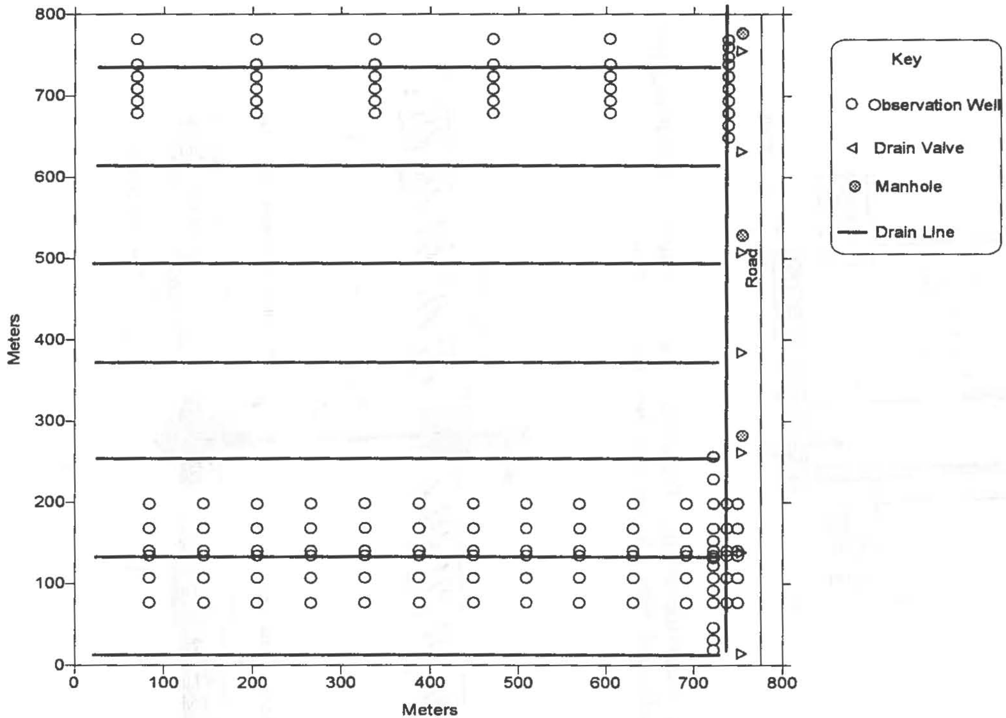


Fig. 2. Layout of Broadview Shallow Ground Water Management Demonstration Project showing the locations of the subsurface drainage laterals, drain control valves, manhole, and observation wells.

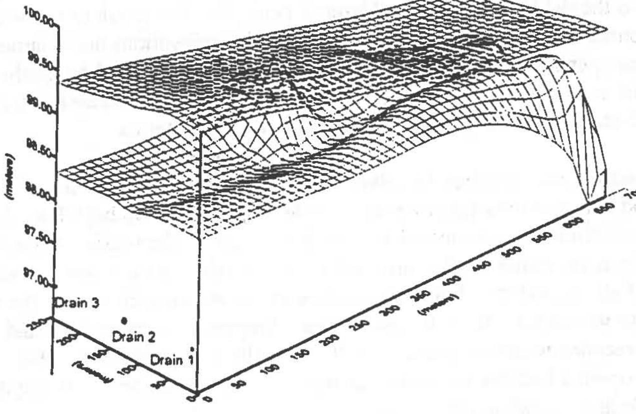


Fig. 3a. Water table position on May 17, 1994 in the Broadview Shallow Ground Water Management Demonstration Project.

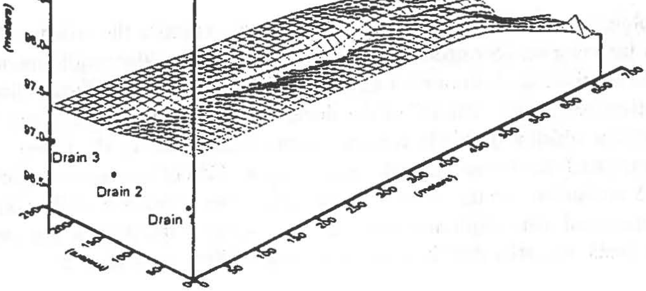


Fig. 3b. Water table position on June 25, 1994 in the Broadview Shallow Ground Water Management Demonstration Project.

Observation wells constructed of 10 ft long 1.5 inch diameter PVC pipe, which had slits cut into the bottom 3 feet, were installed at each valve installation and across the field between several laterals (Fig. 2). The depth to the water table was measured weekly and used to plot water surface elevations and responses to the valves opening and closing. The drain laterals were installed on grade from west to east with the outlet on the east side of the field. The tomato rows were in a north-south orientation, perpendicular to the drain laterals.

The water table response to valve operation is shown in Fig. 3a and 3b for the period between the irrigations on 4/17/94 and 5/25/94. In both Fig. 3a and 3b, the control structures are located at 2200 ft (670 m) on the x-axis. The soil surface is shown as the upper surface grid and the water table as the lower surface grid in both Fig. 3a and 3b. After the valves were closed on each lateral, the water table rose to within a 3.3 ft of the soil surface. The valves were opened and the water level receded to approximately 6.6 ft below the soil surface (Fig. 3b). The valves were opened because the ranch manager was concerned about drying the soil profile in preparation for harvest.

The shallow area close to the control structures had a water table fluctuation from 4.9 to 7.2 ft below the soil surface. The medium depth area had a water table depth of 5.9 to 8.5 ft during the experimental period and the deep area had a water table fluctuation of 7.2 to 8.5 ft. during the experimental period. The hand harvest yields and the component breakdown are shown in Fig. 4 for each of the test areas. The yields in the shallow and medium areas were larger than in the deep area. It appears that the largest difference in yield component occurred in the large red fruit. This value in the deep area was considerably smaller than found in the other two areas. There was also a larger percentage of limited use tomatoes in the deep area than in the other areas; the vines in the deep area did not hold up as well as the vines in the other areas and there was more damage to the fruit from the sun. The machine harvest yields were similar to the values shown for the hand harvest (data not shown).

The objectives of the drain control project were to reduce the volume of drain water by using shallow groundwater to meet the crop water requirement and reduce depth of applications for each irrigation. The results indicate that these objectives were met. The EC of the shallow groundwater ranged from 3 to 8 mmho /cm which is usable by a tomato crop. Hutmacher et al. (1989) demonstrated that tomatoes could extract up to 45% of the water requirement from 5 mmho cm^{-1} water when the water table was within 4 ft of the soil surface. The improved plant vigor and reduced stress levels in the shallow and medium depth areas indicated that the crop was using shallow groundwater.

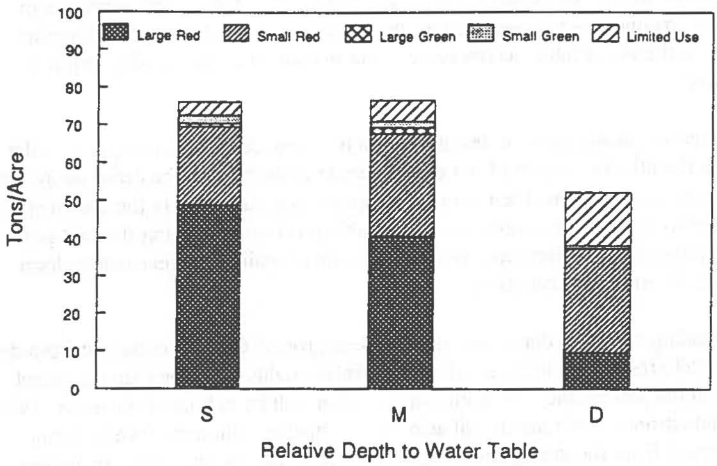


Fig. 4. Tomato yield components from 1994 tomato crop on the Broadview Shallow Ground Water Demonstration Project.

Maintaining the shallow groundwater reduced the crop water requirement by 5.6 in. A companion field which did not have water table control required 32 in of irrigation and the test field needed only 27 in. This resulted in a savings of 73 ac-ft of water. This was significant since the water allocation this year to the district was 35% of the normal supply. In areas with shallow groundwater the irrigation set time was reduced from 12 to 2 hrs.

Subsurface Drainage System Design for Integrated Water Management

Design of drainage systems to integrate irrigation and shallow ground water management will require the adoption of new criteria for the depth and placement of the drains and the depth to water table at the mid-point between the drains (Doering et al, 1982), (Ayars, et al, 1995). Both the drain depth and the allowable mid-point depth need to be reduced from the current recommendation of 8 ft for drain depth and 4ft for mid-point water table depth (U.S. Department of Interior, 1993). Changes in the design which relax current depth and spacing criteria will require additional management criteria to prevent salinization of the soil profile,

The first proposed subsurface drainage design change is to set the recommended mid-point water table depth to approximately 3 ft for all situations. The value of 3 ft was selected as a compromise to permit use of shallower drain depth installation while maintaining a reasonably wide lateral spacing. It was observed in a previous study (Ayars and McWhorter, 1985), that when crop water use of shallow ground water is included in the drainage system design, the minimum depth to the water table occurs early in the season when the rooting depth is shallow.

The second change in drain design criteria is to reduce the drain depth in order to reduce the effective depth of the ground water collection by the drainage system. However, reducing the drain depth also results in a reduction in the lateral spacing in order to adequately control the water table position. Relaxing the mid-point water table depth requirement will compensate to maintain a reasonable drain spacing for irrigated conditions.

By reducing the drain depth and spacing, less ground water is collected from deep in the soil profile, and in cases where the water quality declines with increased depth in the soil profile, less poor quality water will be extracted (Grismer, 1990). The reduction in drain depth will also lead to smaller volumes of water being discharged from the drains and more water being used by the crop. Irrigation scheduling cognizant of salinity stresses at seed germination, and later upward flow from the water table for meeting consumptive use needs of the crop will become part of salinity management in the root zone required in the overall irrigation/drainage management system.

Drainage - No Drainage Cycle

A new concept called drainage - no-drainage was developed as a means to reduce the total drainage flow and induce uptake from shallow ground water (Manguerra and Garcia, 1997) . This proposed operational method starts with a leached profile and then eliminates any drainage flow until the shallow ground water rises to a level which negatively impacts plant growth or the salinity levels in the soil have a negative impact. At this time the drains are opened and a leaching event takes place. The effectiveness of this concept is dependent on the depth of installation of the drains, the configuration of the drain laterals, the salinity of the ground water, the salt tolerance of the crops, and the irrigation efficiency.

The drains should be at least 8 feet deep with the laterals installed perpendicular to the surface grade of the field. This is a configuration similar to that found in the Broadview study. This configuration gives maximum control over the water table over the entire area of the field as was demonstrated in the Broadview Study.

The interaction of the crop salt tolerance and the ground water salinity will determine the potential uptake by the crop. The total ground water utilization will be affected by the age of the crop and the depth to the water table as previously discussed.

As the irrigation efficiency increases the total deep percolation losses will be reduced and the interval between drainage cycles will be increased. This is demonstrated with an example cotton crop with a water requirement of 26 inches, a drain depth of 7 feet, a minimum water table depth of 3 feet, soil porosity of 0.5, and assuming no capillary fringe. The time to store 20 inches of deep percolation was calculated based on percentage uptake by a cotton crop and irrigation efficiency.

Case 1 assumed an irrigation efficiency of .7 and a 20% ground water contribution to the crop water requirement. Case 2 assumed an irrigation efficiency of 0.8 and a ground water contribution of 10%. Case 3 used an irrigation efficiency of 0.7 and a ground water contribution of 10% while case 4 assumed an irrigation efficiency of 0.9 and a ground water contribution of 5%.

The results of this study indicated that there is a 6 year cycle for case 1 with 40 inches of water being extracted from the ground water. Case 2 had a 7 year cycle of operation with only 21 inches of water being extracted from the ground water. Case 3 was the shortest cycle with the estimate being every 2 years with only 9 inches of water coming from ground water. The final example had a 13 year cycle with 13 inches of water coming from ground water. The study demonstrates that the most significant impact will be derived by improving the irrigation efficiency.

SUMMARY

Field and theoretical studies demonstrated new concepts for using and managing shallow ground water in arid irrigated agriculture. The objective of these techniques is to reduce the total volume of drainage water for disposal by maximizing the use of available water supplies. Using subsurface drip irrigation in conjunction with a modified crop coefficient resulted in the maximum uptake of water by cotton from the ground water with the minimum application of water without a yield reduction. Use of controls on a subsurface drainage system resulted in a saving of 5 inches of water applied to a tomato crop without a negative impact on yield. Changing the drainage design criteria for new drainage system design will result in reduced drainage volume and reduce salt load. Adopting a cycle of drainage and no-drainage with improved irrigation efficiency will result reductions of drainage discharge.

REFERENCES

Ayars, J.E. 1996. Managing irrigation and drainage systems in arid areas in the presence of shallow groundwater:case studies. Irrigation and Drainage Systems, 10:227-244.

Ayars, J.E., and R.B. Hutmacher. 1994. Crop Coefficients for Irrigation Cotton in the Presence of Groundwater. Irrigation Science, 15(1):45-52.

Ayars, J.E., and D.B. McWhorter. 1985. Incorporating crop water use in drainage design in arid areas. p. 380-389 *IN* C.G. Keyes and T.J. Ward (ed.) Proceedings, Specialty Conference, Development and Management Aspects of Irrigation and Drainage Systems, Irrigation and Drainage Division, ASCE. American Society of Civil Engineers, New York, New York.

Ayars, J.E., and D.W. Meek. 1994. Drainage Load-Flow Relationships in Arid Irrigated Areas. Transactions of the ASAE, 37:431-437.

Ayars, J.E., and R.A. Schoneman, 1991, Interaction of irrigation water and drainage flow, Proceedings, Irrigation and Drainage Division, IR DIV/ASCE, Honolulu, HI, 22-26 July, pp 275-282.

Ayars, J.E., S.H. Patton, and R.A. Schoneman. 1987. Drain water quality from arid irrigated lands. p. 220-230 *IN* American Society of Agricultural Engineers (ed.) Proceedings of Fifth National Drainage Symposium. American Society of Agricultural Engineers, St. Joseph Michigan.

Ayars, J.E., R.B. Hutmacher, R.A. Schoneman, S.S. Vail, and T. Pflaum. 1993. Long Term Use of Saline Water For Irrigation. Irrigation Science, 14:27-34.

Davis, K.R. 1983. Trickle irrigation of cotton in California. Proceedings of Western Cotton Production Conference, Las Cruces, NM, 9-11, August, pp 34-38.

Doering, E.J., L.C. Benz, and G.A. Reichman. 1982. Shallow-water-table concept for drainage design in semiarid and sub-humid regions. p. 34-41 *IN* American Society of Agricultural Engineers (ed.) Advances in Drainage, Proceedings of the Fourth National Drainage Symposium. Vol. ASAE Publication 12-82, American Society of Agricultural Engineers, St Joseph, Mich.

Grismer, M.E. 1990. Subsurface Drainage System Design and Drain Water Quality. Journal of Irrigation and Drainage Engineering, 119:537-543.

Hutmacher, R.B., J.E. Ayars, S.S. Vail, and R.A. Schoneman, 1989, Measurements in Small Weighing Lysimeters: Effects of Groundwater Salinity on

ET Rates and Water Uptake from Shallow Groundwater of Processing Tomatoes. Water Management Research Laboratory Annual Report, page 18 -19.

Manguerra, H.B., and L.A. Garcia. 1997. Field strategy for agricultural drainage and water-quality management. Journal of Irrigation and Drainage Engineering, 123:37-44.

Rhoades, J.D., F.T. Bingham, J. Letey, G.J. Hoffman, A.R. Dedrick, P.J. Pinter, and J.A. Replogle. 1989. Use of saline drainage water for irrigation: imperial valley study. Agricultural Water Management, 16:25-36.

San Joaquin Valley Drainage Program. 1990. The problem. p. 183 *IN* U.S. Department of Interior and California Resources Agency (ed.) A Management Plan for Agricultural Subsurface Drainage and Related Problems on the westside San Joaquin Valley. California Department of Water Resources, Sacramento, California.

CONJUNCTIVE AND EXCLUSIVE USE OF SHALLOW GROUNDWATER FOR IRRIGATION OF SPRING WHEAT

C. K. McCormick[1] J. C. Guitjens[2]

ABSTRACT

The use of drainwater for irrigation is a viable technology both for improving overall irrigation efficiency and for protecting water quality by reducing the mass output of salts and trace elements from irrigated areas. This was demonstrated in a field study at Newlands Agricultural Research Center in Fallon, NV by growing spring wheat (*Triticum aestivum*) under four irrigation water treatments. The four treatments were: 1) the exclusive use of canal water applied during the day; 2) the exclusive use of drainwater applied during the day; 3) the exclusive use of drainwater applied during the night; and 4) the conjunctive use of drainwater and canal water beginning with a day-time application of drainwater and finishing with canal water. The drainwater came from a shallow aquifer which had elevated levels of salinity and boron. The effects on crop yield of boron and salts applied with drainwater treatments were of primary interest. The field was divided into four blocks representing different soil conditions. Each block was divided into four plots and each plot was randomly assigned one of the four treatments. The growth response to these water qualities was evaluated by weighing plant samples harvested four times during the growing season. The hypothesis that daytime irrigation with drainwater would significantly reduce growth of spring wheat was rejected. The use of drainwater for irrigation appears technically feasible and offers opportunities for improving irrigation efficiency and for reducing the mass output of salts and trace elements from the Newlands Project.

INTRODUCTION

The Newlands Project in the Fallon area of Nevada was among the first irrigation projects authorized under the Federal Reclamation Act of 1902 (Warne 1973). After water deliveries began, the water table rose as much as 18 m. Drainage ditches were needed to alleviate waterlogging and salinization problems. For decades surface and subsurface drainage waters have discharged into endangered wetlands, including the Stillwater Wildlife Management Area (SWMA), to the

[1]Research Support Specialist, Kansas Geological Survey, 1930 Constant Ave., Lawrence, KS 66044

[2]Professor of Irrigation Engineering, University of Nevada, 1000 Valley Rd., Reno, NV 89512

south, east, and north of the irrigated area. Prior to the creation of the Newlands Project, the Carson River provided a safe water supply to these wetlands.

Water in the SWMA can be of poor quality, adversely affecting aquatic life, and trace element concentrations exceed Federal and State criteria (Lico 1992; Hoffman 1992). A program is underway to acquire irrigation water rights for exclusive use in the wetlands (USDI 1993). The use of drainwater for irrigation was demonstrated as an emerging technology for drainage reduction and water quality protection (Faulkner 1996) and is herein extended to the conjunctive and exclusive uses of canal water and drainwater.

In the dispute over water for wetlands, preoccupation with water quantity ignores the quality issues. River water is undiminished in quality whereas drainwater contains the accumulating salts from evapoconcentration following irrigation and from dissolution of salts and trace elements from minerals in the shallow aquifer. The use of drainwater for irrigation has the potential to exchange drainwater for river water and reduce the contamination of wetlands.

The objective is to demonstrate the hypothesis that day-time irrigation with drainwater will significantly reduce growth of spring wheat.

METHOD

Statistical Design

The experiment was designed as a completely randomized design with 4 replications and 3 subsamples per plot (4 replications * 4 treatments * 3 subsamples/plot = 48 samples). The field was divided into 4 blocks with one wheel line per block containing 4 plots. Statistical analysis indicated that there was not a significant block effect and later analyses considered blocks as replications. The single treatment factor was 4 qualities of irrigation water (Cd, Dd, Dn, DdCd). The experiment was designed to test:

H_0: $\mu_{Cd} = \mu_{Dd} = \mu_{Dn} = \mu_{DdCd}$
$H_{alt.}$: At least one mean is significantly different.

Treatments: The study used the following four irrigation treatments:

Cd: A control treatment which was irrigation with canal water applied during the daytime.
Dn: Drainwater applied during the night.
Dd: Drainwater applied during the day.
DdCd: A conjunctive use of drainwater and canal water where the drainwater was applied for one-half the time of an irrigation

event and finished with canal water.

See Fig. 1 for the treatment plot arrangement.

The hypothesis was the Dd treatment would have the lowest yield because of boron concentration and salinity of the water. Dn was included to reduce the chance of leaf burn. The DdCd treatment was included to rinse the plant foliage with canal water after using drainwater to reduce the possibility of leaf burn. The Cd component filled the upper half of the soil column with the less saline canal water. This is especially advantageous because plants preferentially draw water from the upper portion of the root zone.

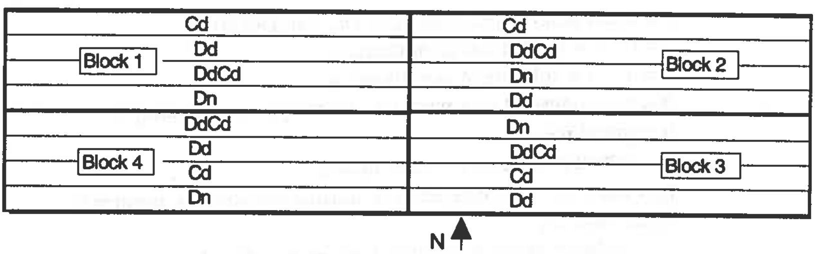


Fig. 1: Arrangement of Treatment Plots

In keeping with Rhoades (1989) recommendations to keep drainwater and canal water separate so high quality water can be preserved for situations where blended water would not be of adequate quality, drainwater was not blended with canal water.

<u>Sampling</u>: A sample site was randomly chosen along a transect running through the center of a plot, parallel to the wheel lines and crop rows. The first 26 m (80 feet) adjacent to the north-south mainline separating blocks 1 and 4 from blocks 2 and 3 were excluded to reduce effects of overlapping spray from a different treatment. The last 13 m (40 feet) farthest from the north-south mainline were also excluded because the application zones did not overlap in the same manner as they did in the rest of the plot. The remaining length available for sampling was 129 m (400 feet) within experimental plots measuring 168 m (520 feet) in length. Subsamples were taken at 3 randomly selected locations along the transect on each sampling date. A sample site consisted of a 1-m length of 1 row of wheat, clipped at ground level. The entire aerial portion was collected, dried, and weighed.

Samples were collected at 4 growth stages. The first samples were collected after all plants had been established using only canal water and before applying any drainwater. This made it possible to test for variability in the field other than the variability caused by the treatments. The second sampling was done 4 weeks later when the plants were in the leaf sheath stage, a time when plants are more sensitive to boron. The third sampling occurred 13 weeks after planting when most plants were in the mid-milk to early-dough stage. The fourth sampling was at harvest time, 17.5 weeks after planting. An additional (fifth) sample was obtained of the grain contained in the fourth sample.

Statistical Model: The experiment used the following statistical model:

$$Y_{ijk} = \mu + A_i + \varepsilon_{ij} + s_{ijk}$$

where:

i = water quality treatment (Cd, Dd, Dn, DdCd)
$j = 1\ldots4$ = replication identification
$k = 1\ldots3$ = subsample identification
Y_{ijk} = response of treatment i on k^{th} sample in replicate j
μ = grand mean
A_i = effect of i^{th} level of water quality
ε_{ij} = random error, independent, normal distribution, mean = 0, equal variance
s_{ijk} = subsample error, independent, normal distribution, mean = 0, equal variance.

The data were analyzed using the SAS® (Statistical Analysis Systems version 6.11) macro MXANOVA (Fernandez 1997). MXANOVA tested the treatment effects and computed the corrected mean-square error for treatment effects using the treatment effects within replications (REP(TRT)) as the correct error term. MXANOVA then produced a box plot of each treatment response, checked the ANOVA (analysis of variance) normality assumption by producing a normal probability plot; and using the D'Agostino-Pearson Omnibus Test, checked for outliers and influential observations within a plot and the Cooks D statistic, and checked the equal variance assumption with Levene's test. The treatment arrangements were randomly chosen to ensure their independence. The ANOVA assumptions were verified using PROC GLM (standard or robust) and the ANOVA was performed using PROC MIXED (or ROBMIX). The robust analysis iteratively weights influential observations to remove outliers. MXANOVA also produced tabular output and a plot of treatment mean differences and confidence intervals as well as output showing the treatment mean differences and the confidence intervals for a given confidence level. Also included in the output were the least-square treatment means, standard errors, and pair and or group comparisons.

Field Preparation

The study area was the central 4.5 ha (146 m x 315 m) of a 9 ha field The entire field had 15 parallel, corrugated plastic drains buried 2 m deep and spaced 37 m apart. Drainwater was collected in a sump and pumped into a 680 m^3 surface reservoir as the source of water for plots receiving drainwater treatments. A centrifugal pump supplied the sprinklers with pressurized drainwater and another pump at the northwest corner pumped canal water.

On March 31, 1996 the study area was planted with spring wheat variety Penewawa at a seeding density of 124 kg/ha (110 lbs/ac). This was the second consecutive season for wheat to be planted on the site. Local and NARC practices, typically dictate that alfalfa is cropped for approximately 7 years followed by 1 - 2 years of small grains before reverting to alfalfa. The seeding density and choice of Penewawa were based on recommendations from the County Cooperative Extension. At planting, 98 kg/ha (87 lbs/ac) of urea was applied based on recommendations derived from soil sample analysis from the Helena Co., a private agricultural service company with a representative in the Fallon area. The same firm performed a tissue analysis of samples 8 weeks after planting and recommended the application of Bayfolan® Plus, a crop mix fertilizer, (N:P:K=11-8-5 plus micronutrients) at 4.7 L/ha (1/2 gal/ac). The application was performed a week later together with Weedestroy®, a 2,4-D based herbicide. The harvest was on 31 July, 17.5 weeks after planting.

Irrigation Scheduling

A new irrigation cycle began when estimated D_i reached 71 mm or after 7 days, whichever occurred first. The amount to apply was based on the estimated ET by employing the FAO Class A pan method of Doorenbos and Pruitt (1977):

$$ET_0 = K_{pan} * E_{pan}$$

where:

ET_0 = reference evapotranspiration (mm)
K_{pan} = pan coefficient (function of wind speed, relative humidity, pan location, and upwind fetch)
E_{pan} = depth of water (mm) evaporated from a Class A pan over 7 days

and ET_{crop} is:

$$ET_{crop} = K_{crop} * ET_0$$

where:

K_{crop} = the crop coefficient (Guitjens, 1990) and
ET_{crop} = crop evapotranspiration (mm) over 7 days

The weekly irrigation demand was calculated as:

$$D_i = (ET_{crop}/0.70) - D_{rain}$$

where:

D_i = depth of irrigation demand (mm)
0.70 = the assumed irrigation efficiency of the system and
D_{rain} = depth of rain (mm).

Estimated soil water used by the crop during one week was replaced during the following week's irrigation cycle.

The root zone water holding capacity was estimated at 10 cm (4 in) (Guitjens, 1992). Meyer and Green (1980) report a significant reduction in leaf growth when the available soil water dropped below 45% of the holding capacity. They concluded that the general practice of allowing 50% depletion before the next irrigation is a safe practice. The irrigation scheduling was based on this 50% depletion. A new irrigation cycle began each week or when the evaporation pan model estimated ET_{crop} to have reached 5 cm (2 in). There were 18½ irrigation cycles in 16 weeks of irrigation. The shortest cycle was 4 days during the period of warm weather and high crop water demand. The crop was allowed to dry during the last 1½ weeks before harvesting. During the last half cycle the entire field received half the estimated D_i.

Sprinkler System

Sprinkler irrigation was used during the 1996 season. Four sprinkler lines were used , 1 per block, spaced 18.2 m (60 ft) between line positions and 12.1 m (40 ft) between nozzles. The nozzles were set for semi-circle application. After the D_i was determined for a cycle, one-half was applied with the sprinkler lines stationed on the south edge, and spraying north into the plots. After all plots received one-half the D_i this way, nozzles were reversed to face south and plots were irrigated with the lines on the north edge and spraying south. This allowed plots to go a maximum of 4 days without irrigation. The water application rate was 2 cm/hr (0.8 in./hr) with the pumping rate of 2,270 l/min. (600 gpm).

Water Chemistry

Samples of both canal water and drainwater were taken at 12 weeks after planting and at harvest for analysis of Ca^{2+}, Mg^{2+}, Na^+, K^+, SO_4^{2-}, Cl^-, NO_3^--N, HCO_3^-, As, Fe, Cu, B, Se, and total P by the EPA approved Nevada State Health Laboratory in Reno. Dissolved oxygen, EC and pH were measured on site biweekly during the latter portion of the experiment.

RESULTS AND DISCUSSION

Statistical Analysis

Box plots indicate that the median value of the Dd treatment response is greater than those of all other treatments during all samplings when using both standard and robust analysis. Only Sample 1 had an outlier. However, the Cooks D test identified an additional outlier in Sample 2 and two outliers in Sample 3. Huber's Iteratively Weighted Robust ANOVA portion of SAS reduced the influence of outliers in Samples 1 and 2. Sample 3, however, retained one outlier.

ANOVA Assumptions: According to Levene's test for equal variance and standard analysis, the equal variance assumption was not met for Sample 1 ($P = 0.0028$) and Sample 3 ($P = 0.040$). The validity of Levene's test has been ascertained only for single factor analysis and it remains to be seen if it is valid for mixed data. Therefore, plots of the residuals (observed value - mean value) are used to show variance among treatments. The absence of a fan or diamond trend in the plots indicates that equal variance may still be assumed. The large variation with standard analysis is resolved by the robust analysis. Robust analysis allowed Sample 3 to meet the criterion ($P = 0.052$) but Sample 1 still did not improve, in fact the problem worsened ($P = 0.00074$). This can be corrected by transforming the data but when working with mixed data, such as in this study, data transformation is not appropriate. The D'Agostino-Pearson Omnibus Normality Test verified the normality assumption and checked the skewness and kurtosis. Standard analysis found the normality assumption was not met only for Sample 3 ($P = 0.000068$). Sample 3 was significantly skewed ($P = 0.018$) and the kurtosis was significant ($P = 0.00022$). Robust analysis rectified the normality assumption ($P = 0.076$) and the skewness ($P = 0.42$) and the kurtosis was greatly improved ($P = 0.034$).

ANOVA Results: The treatment effect when using robust analysis was not significant ($P > 0.05$) for any sampling date except for Sample 4 ($P = 0.0215$); although it was borderline for Sample 3 ($P = 0.0560$) and Sample 5 ($P = 0.0640$). Standard analysis revealed the treatment effect was significant for Sample 4 ($P = 0.0144$) and Sample 5 ($P = 0.0316$). The experimental error was significant for all cases except the standard analysis of Sample 4 ($P = 0.1733$).

When comparing pairs of treatments using the differences of least-square means, there were no significant differences among treatment responses for Samples 1 and 2. Means and their upper and lower confidence intervals are listed in Table 1. Response to treatment Dd was significantly greater than DdCd at Samples 3, 4, and 5 for both robust and standard analysis. Response to treatment response Dd was also significantly greater than Dn at Sample 3 according to both methods of analysis. Also for Sample 3, Dd was significantly greater than Cd with robust analysis but the significance was not present with standard analysis. The reverse was true for Sample 4, the Dd was significantly greater than Cd with standard analysis but not with robust analysis. Both robust and standard analysis found significantly greater yields for Dd than Cd for Sample 5. Only standard analysis found that Dd had a significantly higher yield than Dn. Of most interest for all of these comparisons is that only the Dd treatment was at any time significantly greater than any other treatment.

Table 1: Treatment Response Means, Standard Errors and Confidence Intervals

Treatment	Sample	Lsmean	Standard Error	95% Confidence Interval		P-value
				Lower	Upper	
Cd	1	4.8333	0.6956	3.3178	6.3488	0.0001
Dd	1	4.8333	0.6956	3.3178	3.3488	0.0001
DdCd	1	3.4167	0.6956	1.9012	4.8322	0.0004
Dn	1	3.0833	0.6956	1.5678	4.5988	0.0008
Cd	2	36.167	7.1054	20.685	51.648	0.0003
Dd	2	39.083	7.1054	23.602	54.564	0.0001
DdCd	2	33.333	7.1054	17.852	48.812	0.0005
Dn	2	36.750	7.1054	21.269	52.231	0.0002
Cd	3	60.667	11.175	36.318	85.015	0.0002
Dd	3	88.167	11.175	63.818	112.52	0.0001
DdCd	3	52.667	11.175	28.318	77.015	0.0005
Dn	3	52.583	11.175	28.235	76.932	0.0005
Cd	4	89.583	9.7867	68.260	110.91	0.0001
Dd	4	138.92	9.7867	117.59	160.24	0.0001
DdCd	4	95.583	9.7867	74.260	116.91	0.0001
Dn	4	118.75	9.7867	97.427	140.07	0.0001
Cd	5	25.042	4.724	14.749	35.335	0.0002
Dd	5	45.158	4.724	35.165	55.751	0.0001
DdCd	5	25.375	4.724	15.082	35.668	0.0002
Dn	5	29.958	4.724	19.665	40.251	0.0001

Note: Mean values (g) are for raw subsamples, to convert to g/m^2 multiply mean weights by 6.56

Figure 2 shows the mean weights of the four treatment responses at each sampling. Sample 5 shows average grain yields of 1.6-3.0 Mg/ha depending on the treatment.

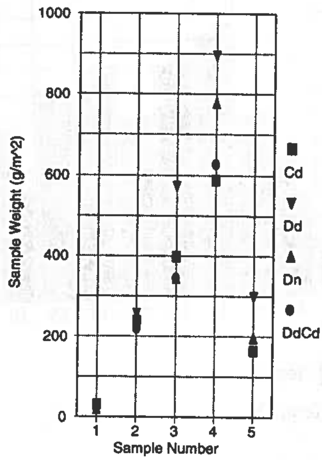


Fig. 2: Mean Weights by Treatment at Each Sampling

Yields

World-wide the mean yield for wheat is approximately 4 - 6 Mg/ha (Doorenbos and Kassam, 1979). The Nevada Agricultural Statistics Service quoted the mean yield for the Fallon area in 1996 as 4.0 Mg/ha. This study, using the results of Sample 5, had average yields ranging from 1.64 to 2.98 Mg/ha depending on treatment. Grain yields ranged from a low of 29.5 g/m^2 (0.295 Mg/ha) for a Cd treatment subsample to a high of 476 g/m^2 (4.76 Mg/ha) for a Dd subsample. Guitjens (1992) showed that the variability within fields contributes to lower average yields. Crop variability can be caused in part by soil heterogeneity which for example resulted from shifting river channels and lake levels.

Causes of the low yield in 1996 are unclear. Although the type sample for Sagouspe loamy sand was obtained from NARC, and the Churchill County soil specialist considered NARC to have uniform soils, the north portion of the field is sandier than the south. Water often ponded on the southwest plots (block 4) for several consecutive days. The yield of Sample 5 was greatly affected by a severe weed infestation despite the herbicide spraying program.

Before irrigating under different treatments, a benchmark was established with Sample 1. Despite the fact that Sample 2 was taken at a lifestage when wheat is sensitive to boron and those effects might be expected to be seen in the results of Sample 2 or Sample 3, the response to treatment Dd was greater than the response to treatments DdCd and Dn, contrary to the hypothesis that irrigation with drainwater would significantly reduce growth of spring wheat.

Irrigation Scheduling

The water applications as compared to estimated irrigation demand can be seen in Fig. 3. The mean D_a for the season was 130 mm (15%) greater than the seasonal D_i. Nearly all of this excess water was applied during the four weeks labeled 0 through 3 when the soil profile was being filled and excess water could leach salts accumulated from the previous season. Week 8 and Week 9 represent one irrigation cycle and the apparent underirrigation of Week 9 was nullified by

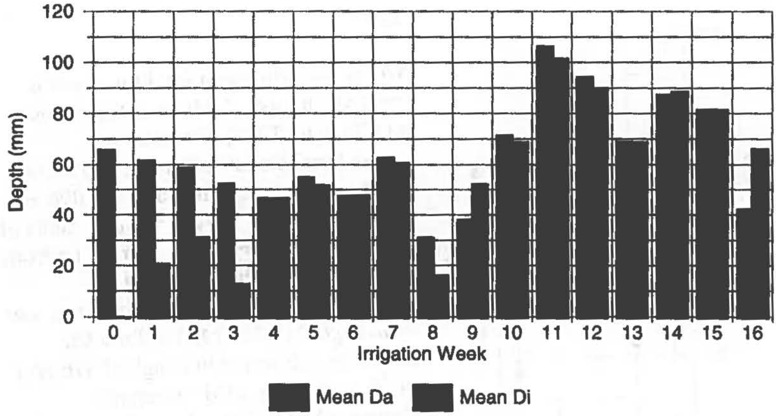


Fig. 3: Mean D_a and Mean D_i

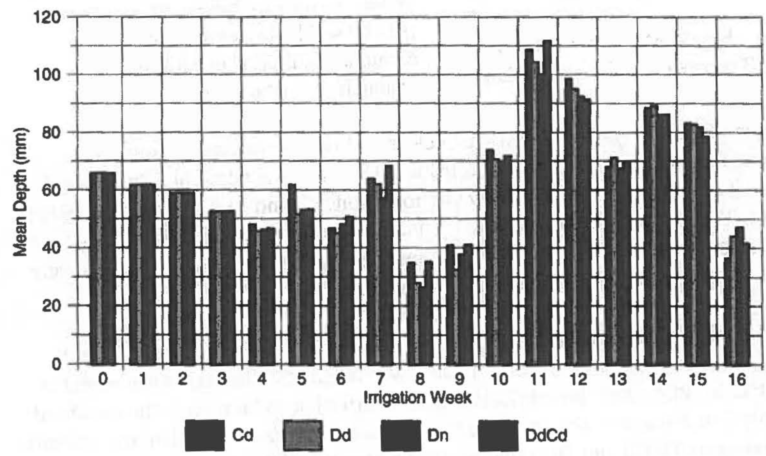


Fig. 4: Mean D_a per Treatment

crediting Week 8 with a greater D_a. The same variations which caused the D_a to deviate from D_i can be seen to cause slight differences in the amount of water applied to each treatment (Fig. 4).

The Truckee-Carson Irrigation District (TCID) has been mandated by the federal government to improve its efficiency of operations including water delivery.

Current methods to improve efficiency focus on the canal delivery system. Irrigators are not allowed access to irrigation water again until 2 weeks after irrigating, leaving lateral canals empty until a minimum number of farmers along a canal are ready to irrigate. This is an attempt to minimize canal losses.

On the other hand, very little is being done to improve on-farm irrigation efficiency. One way to improve on-farm irrigation efficiency is to use drainwater for irrigation. Drainwater can be collected, stored, and applied to the same field as was done at NARC, or it can be pumped back into delivery canals to blend with canal water. Increasing efficiency in Fallon reduces irrigation demand for canal water and salt loading in the SNWR and other wetlands. Drainwater transports salts evapoconcentrated in the root zone and mobilizes and transports additional salts and trace elements during percolation to the wetlands.

Improved efficiency involves water conservation which requires producing the same yield with less canal water. Drainwater can be used in place of canal water for irrigation thus conserving canal water. Gould (1992) demonstrated that the mass flux of salt is somewhat proportional to the flux of drainwater. When drainwater is used for irrigation, there is less off-farm drainage and hence less mass flux of salts from irrigated lands. Irrigation with drainwater is a practicable BMP for the control of nonpoint source flow.

Doorenbos and Kassam (1979) list worldwide water use efficiency (WUE) (Equation 1) for irrigated wheat (whether spring or winter wheat is not clear) as 0.8-1.0 kg/m^3 when using actual ET. Musick and Porter (1990) quote 1.0 - 1.2 kg/m^3 for fall- planted spring wheat although it may be as high as1.9 kg/m^3, but

$$\mathrm{WUE} = \left(\frac{\mathrm{yield(kg/m^2)}}{\mathrm{ET(m)}}\right) = \mathrm{kg/m^3} \qquad (1)$$

tests in the Great Plains, Washington and Utah agree with the values given by Doorenbos and Kassam. Using the modeled ET of 0.66 m and an average yield of 0.298 kg/m^2 for treatment Dd, this study had a WUE of 0.45 kg/m^2. The subsample with the highest grain yield (0.476 kg/m^2) was in the Dd treatment. That subsample had a WUE of 0.72 kg/m^3. The subsample with the lowest yield (0.059 kg/m^2) received the Cd treatment and had a WUE of 0.08 kg/m^3. Statistical testing at a field scale rather than on small plots appears to have a confounding influence by creating large variability.

Water Chemistry

Table 2 reports the water chemistry of canal water and drainwater. All the elements and ions listed contribute to salinity. Boron at 1.5 mg/L, which is highly soluble and leaches readily, is at a potentially problematic concentration for wheat in the drainwater. Maas (1990) lists tolerance limits for wheat and alfalfa as 0.75 -

Table 2: Water Chemistry

	22 July, 1996 Canal		22 July, 1996 Drain		2 Aug. 1996 Canal		2 Aug. 1996 Drain	
Major Ions	mg/L	meq/L	mg/L	meq/L	mg/L	meq/L	mg/L	meq/L
Na^+	17	0.739	284	12.3	17	0.739	284	12.3
Ca^{2+}	18	0.9	47	2.35	18	0.9	43	2.15
Mg^{2+}	5	0.413	9	0.744	5	0.413	9	0.744
K^+	2	0.051	9	0.231	3	0.077	12	0.308
Total P	0.14	0.23	0.54	0.087	0.08	0.013	0.24	0.039
SO_4^{2-}	26	0.578	160	3.56	25	0.56	159	3.53
NO_3^--N	0.2	0.071	12.4	4.43	0.3	0.107	10.7	3.82
HCO_3^-	83	1.36	664	10.9	73	1.20	605	9.92
Cl^-	5	0.26	39	2.05	5	0.26	33	1.74
Trace Elements	(mg/L)		(mg/L)		(mg/L)		(mg/L)	
Fe	0.29		0.10		0.78		0.07	
Mn	0.03		0.03		0.04		0.01	
B	0.1		1.5		0.1		1.5	
Se	NA		0.003		NA		0.006	
As	0.007		0.250		0.008		0.260	
Hardness	66		155		66		145	
Alk.	68		544		68		496	

	1 Jn	16 Jn	28 Jn	6 Jy	13 Jy	20 Jy	1 Ag.	Mean
<u>CANAL</u>								
Temp (°C)	11.4	15	15	16	15.5	16	17.5	15.2
DO (ppm)	9.5	6.6	9.3	8.8	7.3	8.9	9.1	8.5
EC (dS/m)	0.26	0.24	0.24	0.22	0.23	0.22	0.22	0.23
<u>DRAIN</u>								
Temp (°C)	11	16	14.2	13.5	14.5	14	14.5	14.0
DO (ppm)	9.5	5.6	3.6	4.4	3.2	3.0	5.2	4.9
EC (dS/m)	1.53	1.4	1.57	1.5	1.37	1.57	1.44	1.48

1.0 mg/L and 4.0 - 6.0 mg/L respectively, but notes that tolerance varies with climate, soil conditions, and crop variety and makes similar qualifications regarding EC tolerance limits. Boron in the canal water at 0.1 mg/L is not of concern. The EC of the drainwater is also well below the 2.0 dS/m tolerance limit for alfalfa and the 6.0 dS/m limit for wheat (Maas, 1990). HCO_3^- is the dominant ion in canal water followed by Ca^{2+}, Na^+, SO^{2-}_4, and Mg^{2+}. In drainwater, Na is dominant followed by HCO_3^-, NO_3^--N, SO_4^{2-}, Ca^{2+}, Cl^-, and Mg^{2+}. The Cl^- concentration should not be harmful to alfalfa or wheat (Maas, 1990). Ayers and Westcot (1985) recommend using irrigation water with less than 0.10 ppm As. Drainwater concentrations of As were 0.25 and 0.26 ppm. Fe and Mn are both adsorbed to the soil matrix. Their concentrations are safe for irrigation. The presence of any selenium could be a problem if it evapoconcentrates or biomagnifies in wetlands or if selenium assimilating plants are established in irrigated pasture. Overall, the drainwater should not cause any problems when used as irrigation water.

Once plants are established using canal water, irrigation with drainwater is viable for spring wheat. The drainwater was not detrimental to crop yield. Since wheat is moderately sensitive to B and alfalfa is more tolerant, the B concentration in drainwater will not reduce yields in alfalfa. The high variability within the field Once plants are established using canal water, irrigation with drainwater is viable for spring wheat. This variability is common in the Fallon area and can be taken as representative of local conditions.

CONCLUSION

Irrigation with drainwater did not significantly reduce the yield of spring wheat, contrary to the hypothesis. Of the four treatments, only irrigation exclusively with drainwater during the day produced yields significantly greater than any other treatment. The Dd treatment response was significantly greater than DdCd and Dn at Sample 3, DdCd at Sample 4, and DdCd at Sample 5 according to ANOVA using SAS® standard and robust analysis procedures. Only the standard analysis found that Dd was significantly greater than Cd in Sample 3 and Dn in Sample 5 whereas the robust procedure found a significant difference from Cd in Sample 4.

The use of drainwater for irrigation of spring wheat was shown to be technically feasible. Grains yields based on individual subsamples obtained during Sample 5 ranged from a low of 29 g/m^2 for a Dn subsample to a high of 475 g/m^2 for a Dd subsample. By using drainwater to replace an equal depth of canal water, this study reduced the canal water demand. This method can be sustained over many years since drainwater is not available early in the irrigation season and any accumulated salts are flushed from the soil by using canal water. If this leaching is insufficient, the area receiving drainwater may be changed each year when the accumulated salt concentration necessitates greater leaching than that provided by the early season irrigations alone. The mass flux of salts from irrigated areas is somewhat proportional to the flux of drainwater. Irrigation with drainwater serves as a BMP to reduce non-point source flow of drainwater and a proportional mass flux of salts.

REFERENCES

Ayers, R.S., and Westcot, D.W. 1985. Water Quality for Agriculture. *Irrigation and Drainage Paper No. 29.* FAO, United Nations, Rome, 174 p.

Chambers, E.P. and Guitjens, J.C. (1995). "Shallow Ground-Water Influence on Salt Budgets for Newlands Project." *ASCE J.. of Irr. and Drain. Engr.*, 121(6), 436-441.

Doorenbos, J. and Pruitt, W.O. (1977). "Crop Water Requirements." *FAO Irrigation and Drainage Paper No. 29*, FAO, United Nations, Rome.

Doorenbos, J. and Kassam, A.H. 1979. Yield Response to Water. *Irrigation and Drainage Paper No. 33*, FAO United Nations, Rome, 193 p.

Faulkner, B.F. (1996). Soil Chemistry and Hydrodynamics of Irrigation with Drainwater. University of Nevada, Reno M. S. Thesis. 78 p.

Faulkner, B. and Guitjens, J.C. (1996). "Soil Water Chemistry of Irrigation with Drainwater." Proc. ASCE North American Water and Environment Congress, Anaheim, CA.

Fernandez, G.C.J. 1997. MXANOVA, SAS Macro for Analyzing Mixed Factor ANOVA and Checking for the Violations of Assumptions. APST: Design and Analysis of Experiments Course, Dept. of Applied Economics and Statistics. University of Nevada- Reno. Reno, NV 89557.

Gould, J.C., 1992. Influence of Irrigation on Subsurface Drainage. M.S. Thesis, University of Nevada-Reno, 83 p.

Guitjens, J.C. (1990). "Crop Coefficients for Estimating Evapotranspiration of Spring Wheat." *ASAE Applied Engr. in Agr.*, 6(3), 301-304.

Guitjens, J.C. 1992. Interpreting Spatial Yield Variability of Irrigated Spring Wheat. *Transactions of the ASAE*, 35(1):91-95.

Hoffman, R.J. (1992. "Detailed Study of Irrigation Drainage in and near Wildlife Management Area, West-Central Nevada,1987-1990. Part C. Summary of Irrigation-Drainage Effects on Water Quality, Bottom Sediment, and Biota."*USGS Water Resources Investigations Report 92-4024C.* USGS, Carson City, NV.

Lico, M.S. (1992). "Detailed Study of Irrigation Drainage in and near Wildlife Management Areas, West-Central Nevada, 1987-1990. Part A. Water Quality, Sediment Composition, and Hydrogeochemical Processes in Stillwater and Fernley Wildlife Management Areas." *USGS Water Resources Investigations Report 92-4024A.* USGS, Carson City, NV.

Maas, E.V. (1990). "Crop Salt Tolerance." *Agricultural Salinity Assessment and Management, ASCE Manuals and Reports on Engineering Practice; no. 71*, ASCE, New York, NY.

Meyer, W.S., and Green, G.C. 1980. Water Use by Wheat and Plant Indicators of Available Soil Water. *Agronomy Journal.* 72(2):253-257.

Musick, J.T., and Porter, K.B. 1990. Wheat. (in) Stewart, B.A. and Nielsen, D.R. (eds.) Irrigation of Agricultural Crops. *Agronomy No. 30.* ASA, CSSA, SSSA. Madison, WI, pp. 597-638.

Rhoades, J.D. 1989. Intercepting, Isolating and Reusing Drainage Waters for Irrigation to Conserve Water and Protect Water Quality. *Agricultural Water Management.* 16:37-52.

USDI, (1993). *Water Rights Acquisition Program for Pyramid Lake and Lahontan Valley Wetlands, Nevada. Report to the United States Congress.* USDI Fish and Wildlife Service.

Warne, W. E. (1973). *The Bureau of Reclamation.* Praeger, New York, N.Y.

RESTRUCTURING A NATIONAL WATER RESOURCE DEVELOPMENT AGENCY: THE CASE OF THE U S BUREAU OF RECLAMATION

Mark Svendsen[1]

ABSTRACT

The United States Bureau of Reclamation has a long history of successful large-scale water resource development in the Western US. In the 1980s that mandate changed abruptly, and the Bureau of Reclamation was instructed to shift its focus from resource development to resource management. The organization then began a 10-year period of adjustment, downsizing, and restructuring. This process resulted in a 25% reduction in staff levels, a new flatter organizational structure, and simplified administrative and review processes. The paper describes the forces which led to this abrupt change in orientation, the processes employed in carrying out the restructuring, and the results. It concludes with lessons learned which may help other countries facing similar pressures to make necessary adjustments in their public irrigation institutions.

BACKGROUND: THE BUREAU OF RECLAMATION

The reclamation and settlement of arid lands will enrich every portion of our country and our people as a whole will profit — Theodore Roosevelt, 26th President of the United States

Creation

In 1902 the U S congress passed the Reclamation Act, establishing the United States Reclamation Service, later renamed the Bureau of Reclamation. Its mission was to stimulate Western growth and development by constructing a system of irrigation works for the storage, diversion, and development of Western[2] water resources. By this means, the arid west would be "reclaimed" or made useful for settlement and habitation and would yield livelihoods for the new settlers. Much of the early experience with irrigation development in the Western U S was through the investment of private capital, and by the 1890s, 1.4 million hectares of land had been brought under irrigation. At the turn of the century, however, accumulating financial failures and the large scale of many of the potential development sites were making it increasingly clear that a federal presence would ultimately be required. At that time, British-ruled India was the most experienced nation in the world with the construction and operation of very large hydraulic structures. In 1898, several years prior to the passage of the Reclamation Act, two civil engineers from the

---

[1] Consultant in Water Resource Management, 24144 Price Creek Road, Philomath, OR 97370, (541)929-4855, msvendsen@compuserve.com

[2] The West is defined as the 17 continental states lying largely west of the 100th meridian and is the Bureau's assigned area of responsibility. This corresponds roughly with the portion of the United States in which agriculture is risky or impossible without the benefit of irrigation.

United States Geologic Survey were sent to India study that experience. Their report was important in demonstrating the possibilities for very large scale hydraulic works development and helped set the stage for the creation of the USBR several years later.

Glory Days

The economic depression of the 1930s was the stimulus for a huge acceleration in the pace of Western water development. The New Deal programs of President Franklin Roosevelt, introduced soon after his taking office in 1933, led to the day, in 1936, when five of the largest dams in the world — Hoover, Bonneville, Fort Peck, Shasta, and Grand Coulee — were all going up at the same time (Reisner and Bates, 1990). During World War II, projects with power potential were given priority over irrigation projects and rushed to completion. In the late 1940s and early 1950s the USBR program again accelerated before declining somewhat in the mid-1950s. The last increase in Bureau construction activity began in the 1960s and lasted until the mid-1970s (Guy, 1995). By the end of that decade, area irrigated by Bureau facilities had grown to nearly 5 million hectares.

The Bureau reached its symbolic apex under the leadership of Floyd Dominy, Commissioner between 1959 and 1969. Dominy was colorful, charismatic, combative, ambitious, and politically astute. He was an economist by training, holding a masters degree from the University of Wyoming, and a political operator by instinct. Dominy taught himself to build small dams as a county extension agent in Wyoming during the great depression, found he liked it, and never looked back. He rose quickly through the ranks of the Bureau following the second world war and became Commissioner in 1959, serving under Presidents Eisenhower, Kennedy, and Johnson, and briefly under President Nixon. Having established strong symbiotic relationships with key congressional leaders, he was, in some ways more powerful than his immediate superior, the Assistant Secretary, and even the Secretary himself. Together with his congressional allies, he presided over the Bureau's last great construction boom. Even during this period, however, public attitudes were changing, and Dominy's self-assured and single-focused dedication to initiating new dam construction helped to feed the growing reaction against such projects.

Organizational Structure

The Bureau of Reclamation is an organizational unit of the Department of the Interior. The United States federal government is organized into *Departments*, equivalent to what are called ministries in many countries. A Department is presided over by a *Secretary*, who is appointed by the President but must be confirmed by the Senate. The Secretaries of the Departments comprise the President's cabinet. A Secretary is not an elected official nor a member of the national legislature, but is usually a member of the President's political party. Below the secretary are *Under Secretaries* and below them *Assistant Secretaries*.

The Bureau is responsible to the Assistant Secretary for Water and Science, who reports to the Secretary of the Interior. It is headed by a *Commissioner*, who is appointed by the White House and , since 1982, subject to Senate confirmation.

In terms of geographical structure, prior to the reorganizations there were four levels — the Washington DC headquarters, the Denver Reclamation Service Center, regional offices, and project offices located near the sites of individual projects. Technical support services were located in Denver. About three-quarters of the Bureau staff, in 1986, were located in regional or project offices, with another 21 percent in Denver, and the remainder in Washington.

Funding for the Bureau is contained in the annual appropriations act passed by the Congress. A budget is proposed by the President and then taken up by the appropriations committees in the House and Senate. Because the appropriations bill includes the funding for both studying new projects and constructing approved projects, the chairmanship of a sub-committee dealing with water projects is an extremely powerful post, with the ability to channel federal funds to the various congressional districts of the country in which projects are, or potentially could be, located. Because of the control which these committees and sub-committees have over the construction of water resource projects, the relationship between the Bureau and the Congress has historically been a very close one.

Mode of Operation

For the first 80 years of its existence, the Bureau of Reclamation was an organization focused tightly on planning, designing, and constructing irrigation projects and multi-purpose projects with major irrigation components. So concentrated was this focus, that from the beginning, reclamation policy has been to create farmer-controlled irrigation districts prior to system construction and to transfer operation and maintenance responsibility to the districts soon after completing construction. The Bureau has thus never been extensively involved in managing irrigation systems, as have the irrigation agencies in many countries. The Bureau has often continued to manage major impoundment and primary conveyance facilities, however, delivering bulk water supplies to irrigation districts and other users.

THE REFORMS

The era of constructing large federally financed water projects is drawing to a close — Assessment '87...a New Direction for the Bureau of Reclamation

Driving Forces

Three major forces can be identified, at least in retrospect, which brought the Bureau to the point where drastic change was virtually inevitable.

The Closing Water Frontier: The first and most fundamental of these was the closing Western water frontier. Although undeveloped Western water resources were abundant at the turn of the century when the Bureau was created, 75 years later this was no longer the case. In part, this was due to the Bureau's own achievements in harnessing the major river systems of the West — the Colorado, the Columbia, the Missouri — and the massive California Water Project, constructed by both Federal and State governments, which routes the bountiful snowmelt from the northern California mountains to the central and southern deserts and the urban concentration of Los Angeles. The water resources of these river systems were thus nearly fully allocated to the various consumptive uses to which water was put — irrigation, navigation, and municipal use in coastal cities — leaving little water for new allocations.

In addition, there are Native American claims for water related to land grant treaties signed by the government with the Native American Tribes in the 19th century. The water rights associated with these land grants have never been fully specified, but the courts have affirmed that, in principle, water rights are attached to these lands[3]. The amounts of water involved are potentially large and would often compete directly with existing uses of available water resources.

Because the best projects had already been built, and the choicest damsites utilized, economic justifications for remaining projects were becoming increasingly unattractive. One controversial project in Colorado, which was developed by the Bureau and lobbied for by congressional backers in the 1970s and 1980s, for example, had a benefit/cost ratio of well under 0.5.

At the same time, demands for additional water, especially from non-agricultural users, are expanding. These competing demands include municipal water supplies for growing urban concentrations, recreational in-stream uses of water, and environmental demands. The combination of static supply and growing demand places a premium on improved management of water resources, on development new technology to facilitate better management and reduce consumptive use, and on creative linkages among users to develop more efficient use patterns through cooperative action.

Environmental Concerns: Another powerful force acting on Western water resource development, and the Bureau of Reclamation, was rising concern over changes, most of them negative, in the natural environment of the West. These concerns extend far beyond dams and irrigation projects and encompass a wide range of resource exploitation activities (logging, fishing, mining, irrigation) as well as the indirect effects of rapid population growth, extensive motor vehicle use, urban sprawl[4], thermal power generation, and so on. Dams and irrigation, however, have become closely associated with a particular set of negative environmental impacts, to which they undoubtedly contribute. The list of these includes destruction of wetlands and other wildlife habitat; creation of "artificial[5]" settlement patterns, submergence of scenic areas; reduced biodiversity; altered temperature and flow patterns in rivers; reduced riverine recreational opportunities; extinction of regional populations of anadromous fish[6]; altered salinity levels in estuaries; changes in

---

[3]The Winters Doctrine of 1912.

[4]The emergence of low-density urban concentrations occupying large areas and oriented toward automobile travel.

[5]Los Angeles is "artificial" in the sense that it far exceeds the size that could be supported by local water resources and can exist only by importing water from locations hundreds of miles away through vast civil works.

[6]Fish which periodically migrate from fresh to salt water and back, such as salmon.

sediment deposition; soil salinization; induced drainage problems; creation of saline and toxic lakes and wetlands by irrigation return flows; and groundwater contamination.

In the main, these problems were a result not of single decisions or single projects, but the cumulative effects of numbers of projects on the same river or in the same basin[7]. When only a small fraction of total river discharge was being controlled and exploited, negative impacts on the environment were small relative to the undisturbed area remaining. As the degree of water control and exploitation expanded, the **cumulative** effects became an increasingly large share of the total, giving each new project a larger relative negative impact on the remaining environmental asset, (species population, forest area, etc.). This made the negative environmental consequences of each new project more serious and led to growing public opposition where there had previously been unquestioning support.

Intensifying these concerns and infusing them with political power was an increasingly urbanized population which valued rural areas not a places to generate a livelihood through farming, logging, or mining, but as scenic and recreational destinations and as repositories of complex natural ecosystems. Anxious to preserve the dwindling stock of natural environment, people supported and acted through environmental lobbying groups, such as the Sierra Club, the Audubon Society, the Environmental Defense Fund and others, to express these values in state legislatures and in Washington, DC. And increasingly, this meant opposition to new dam construction. Urban dwellers also backed these values with increasing expenditures on outdoor recreation, creating a burgeoning new industry with interests in **not** damming rivers and flooding canyons, thus adding economic clout to direct political influence.

These new values found powerful political expression in the passage of the National Environmental Policy Act (NEPA) in 1970. This act created a new agency, the Environmental Protection Agency (EPA), responsible directly to the President and charged to safeguard the nation's natural environment. The NEPA was followed 3 years later by the Endangered Species Act (ESA) which has been one of the most powerful legal and regulatory tools used by the EPA and environmental groups in challenging environmentally destructive public activities and projects, including certain dam and irrigation projects.

Budgetary Pressure: The effect of this shift in public values can also be seen in the third major force driving change funding for water projects. The annual level of USBR's budget authorization between 1974 and 1996, in constant 1996 dollars, is shown in Figure 1. As seen in the figure, after peaking in the mid-1970s, the annual budget has declined to a level of just $750 million (1996 dollars). The sharp spike in 1977 probably relates to expenses incurred in the aftermath of the Teton Dam failure in 1976, a tragic but important event in the history of the Bureau's reorientation.

---

[7] Some projects had a more singular effect than others. The Grand Coulee Dam on the Columbia, for example irrevocably blocked salmon access to the entire upper half of the river's length. The 10 downstream dams, on the other hand, allow fish passage, but each contributes to attrition in their numbers on both upstream and downstream journeys.

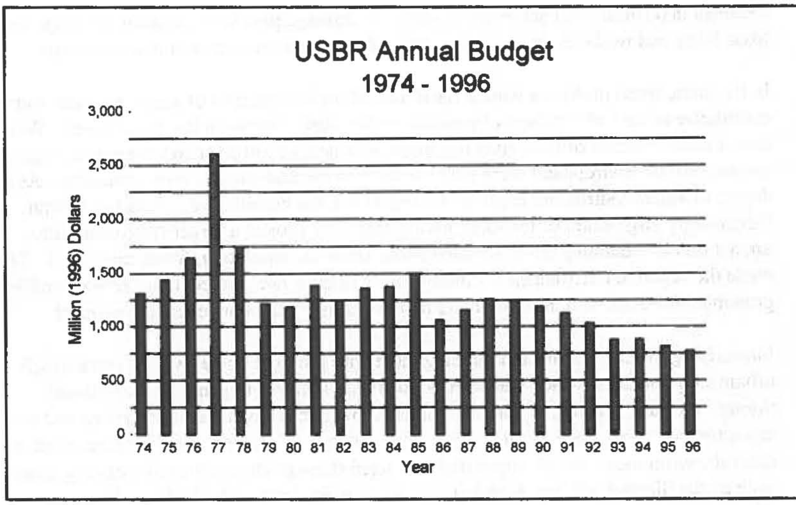


Figure 1. Annual Budget of the United States Bureau of Reclamation

Chronology of Change

If the forces which led to the Bureau's restructuring build up gradually over several decades, the changes themselves took place in a relatively short seven-year period.

Preliminary Events: There were a number of false starts in the remaking of U S water resource development policy, beginning with President Eisenhower's "no new starts" policy in the 1950s. This was an largely unsuccessful attempt to halt initiation of new projects, while completing projects already underway. In 1956, midway through his presidency (and in an election year) Eisenhower signed the $1.6 billion Colorado River Storage Project Act which authorized the building of the Glen Canyon and other dams in the upper Colorado River basin (Reisner, 1993), effectively ending "no new starts."

Around 1960 the Sierra Club, the nation's premiere conservation organization, announced, in an ad in the *New York Times*, its opposition to the building of Glen Canyon dam, which would inundate 200 miles of the Colorado River canyon above the Grand Canyon. This is reckoned by Dan Beard, later a Bureau Commissioner, as marking the beginning of the modern activist environmental movement in the United States. Nevertheless, the Glen Canyon Dam was completed and began filling in 1963[8].

[8]It was from Glen Canyon Dam that the controversial and largely successful flushing releases were made in 1996 in an experiment to restore sandbars and other natural features in the Grand Canyon to improve plant and animal habitat and enhance recreational rafting. Public interest in this experiment can be gaged by the fact that the Bureau received 35,000 comments on plans for the release.

In 1968, the Colorado River Basin Project Act was passed, authorizing the massive Central Arizona Project and several smaller projects. This was the last major authorization for Bureau construction work, though funding under this authorization continued for many years.

Then in 1969, the NEPA was passed, establishing the Environmental Protection Agency and creating an institutionalized public environmental conscience. The EPA has served as a regulatory and policymaking focus for environmental issues since that time, although its mandate and latitude for action has changed somewhat from time to time in response to White House and congressional politics. The NEPA also had the effect of opening up the project planning and decision making processes, making them more transparent and accessible to a wide range of viewpoints and to public and press scrutiny.

In 1976, the Teton Dam in Idaho breached due to foundation piping. The first major failure of a Bureau dam also shook the Bureau's organizational foundations. The failure led to the commissioning of an external review of the failure and an internal review of the management processes which had failed to detect and correct the design and construction problems. As a result, design and construction processes were thoroughly revised and changes made in organizational structure. In addition to shaking the Bureau's self-confidence, the Teton Dam failure brought negative nation-wide attention to Western dams and the Bureau of Reclamation.

The following year President Jimmy Carter took office. President Carter's most lasting claim to water resource fame was the renowned "hit list" of Western water projects. Just prior to his taking office, Carter's transition team assembled a list of water projects which they felt had so many negative environmental impacts they should receive no further funding. Himself an engineer, Carter brought with him to the Presidency an engineer's habit of logical analysis and a set of strong and resolutely-held principles. As governor of Georgia, he had personally reviewed the environmental impact statement for a proposed project on the South Carolina/Georgia border — the Richard Russell Dam — and found it seriously flawed. Partially as a result of this experience, and from a more general concern over "pork barrel" public works projects, when presented with the hit list, to the surprise of many, Carter endorsed it.

The release of the hit list created a flurry of controversy, in Congress and in the press, and the list did much to raise public awareness of the questionable economics and the significant negative environmental impacts associated with at least some of the water projects being proposed by the Bureau and the Corps of Engineers. However, as with Eisenhower before him, Carter could not make the new policy stick in the face of continuing congressional support for water resource projects and strong support from the involved localities. However, it was a powerful "shot across the bow", in the words of a former Bureau Commissioner. Around 1984, the Reagan Administration imposed new beneficiary cost sharing requirements, which had a chilling effect on new investments.

Assessment 87: On 6 March 1987, in the waning years of the Reagan Administration, the Secretary of the Interior commissioned a report, termed *Assessment 87 ... A New Direction for the Bureau of Reclamation*, which was to assess the future role of the Bureau in managing the water resources of the arid West. The assessment was headed by a career Bureau manager and carried out by a team of career Bureau employees.

Assessment 87 concluded that "the era of constructing large federally financed water projects is drawing to a close" and that "the Bureau's mission must change from one based on federally supported construction to one based on effective and environmentally sensitive resource management." This was a powerful conclusion and set the stage for the reorganizations which were to take place in 1988 and 1994. At this time, each of the Bureau's six regions had the capacity to plan and design a major water resource project — a capacity that was no longer needed.

Implementation Plan: In 1988, an *Implementation Plan* was prepared by a team of senior Bureau managers at the request of the Secretary, "evaluating" the assessment report and laying out a program for making the changes outlined in it. The *Plan* report made a number of recommendation, the most dramatic of which was that the headquarters office of the Bureau, including the Commissioner's office, should be moved from Washington, DC to Denver and that a Deputy Commissioner be appointed to manage the day-to-day operations of the Bureau. A small liaison office would be left behind in Washington to manage congressional relations and other representational functions. A "Policy-Technical Service Center" would be established in Denver by consolidating and reorganizing several existing offices, and the 6 regional offices consolidated into 5. Staff reduction targets were set of 10% in 1989, 25% by 1992, and 50% by 1998.

The *Plan* identified as the Bureau's mission, "the pursuit of comprehensive solutions to water resource management problems." It also highlighted four management principles, which are readily recognizable in the Bureau's management philosophy today. These principles were:

- Service orientation — the concept of providing services to clients, as opposed to regulation and control
- Delegation — technical and support functions would be consolidated when there were clear and significant cost advantages in doing so; otherwise responsibility and accountability would remain at the lowest practicable level
- Flexibility — being fluid and responsive through resource reallocation, ongoing training, deregulation, and sunset provisions[9] for internal regulations.
- Employee recognition — recognition of the centrality of its employees to the Bureau's work and performance-based employee evaluation and rewards

In 1989, an *Implementation Plan Update* was prepared which described and assessed results of the implementation during the previous year and suggested additional necessary actions. The *Update* indicated that many of the structural changes which had been proposed had been or were being carried out. A major modification of the proposed reorganization, however, resulted in the Commissioner, one of the three Assistant

[9] A sunset provision is one which specifies that a rule or regulation will be canceled as of a certain date unless explicit steps are taken to retain it.

Commissioners, and a number of administrative offices and staff remaining in Washington. Nevertheless, the *Update* asserted that the reorganization had succeeded in simplifying and clarifying the chain of command and there was a net reduction in Washington staff levels.

The *Update* indicates that extensive training programs were being mounted, in accord with the principles expressed in the *Implementation Plan*, though most of the training opportunities appear to have been directed at senior managers. The *Update* also provides evidence that the work team concept, which is such an important part of the current personnel management strategy in the Reclamation Service Center (RSC), was introduced at this time.

Mission of the Bureau of Reclamation

To manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Box 1. Mission Statement from the 1992 *Strategic Plan*

Strategic Plan: In 1990 under a new Commissioner, the Bureau began work on a *Strategic Plan* for the organization. In 1991, the Bureau also conducted an all-employee survey to determine views on organizational reform from across the agency. In June of 1992, the *Strategic Plan* was issued, articulating a new mission for the Bureau which was much simpler and more action-oriented than the one contained in the 1988 *Implementation Plan* (Box 1).

The *Strategic Plan* also identified five program areas containing 25 separate program elements. The structure of the *Plan* is somewhat muddled, in that it mixed ends and means. It also abandoned the difficult but necessary attempt to establish and adjust priorities among program elements which had been begun under *Assessment 87*. Because developing the *Strategic Plan* was not a broadly-based effort, it was not as widely referred to in discussions with Bureau staff and served more to codify and structure ideas and program elements than to set new directions. During this period, increased attention was given to human resources development and to developing partnerships with other groups and organizations. These two themes have continued to characterize Bureau actions.

The Blueprint for Reform: In early 1993, Daniel Beard took charge of the Bureau as an appointee of the incoming Clinton Administration. In May 1993, he formed an 8-person team, drawn from all levels of the Bureau, to make recommendations concerning the changes needed to "complete the transition from a water resources development agency to a water resources management agency." The team issued its report in August and the Commissioner issued a *Blueprint for Reform* and an accompanying decision memorandum on November 1st, laying out the changes to be implemented.

Blueprint for Reform
1. Mission statement of 1992 reaffirmed
2. No new federal irrigation water supply projects to be built
3. Reimbursable work outside the Interior Department to be eliminated
4. Technical standards to be guidelines, not requirements
5. All existing standards, manuals, and handbooks to be deactivated (sunset) at the end of FY 1995 unless specifically retained or reissued
6. *Project offices* changed into *area offices*
7. Management functions shifted from Denver back to Washington DC

Box 2. Key Features of the 1993 *Blueprint for Reform*

The *Blueprint* made some of the hard choices that had been largely avoided previously. Some key features are presented in Box 2. The new policy stated unequivocally that the federal government will not initiate new federally-owned irrigation projects. In other words, no new dams. Although the trend had been in that direction, this made the decision clear and firm and a matter of policy. It also curtailed contract work for outside agencies, such as the EPA. An element of the reorganization strategy developed after 1987 had been to actively pursue contract work for other federal agencies in order to utilize, and hence preserve, the Bureau's technical capacity. There was considerable policy emphasis on reducing review and oversight within the Bureau, and there was a wholesale scrapping of 90 years of accumulated design standards, processes, and procedures. Only those standards, manuals, and handbooks which were explicitly retained, or revised and reissued, were to remain in force after the end of September 1995[10].

The new policy consolidated project offices into 27 area offices, responsible for all construction and resource management activities within their boundaries. The area offices now provided complete coverage of the respective regions rather than only the areas of selected projects. The role of the area offices was thus enhanced, while that of the five regional offices was diminished somewhat. Finally the reform formally shifted the functional headquarters of the Bureau back to Washington DC and restricted the role of the Denver office to technical and administrative support.

As a part of the restructuring process, senior Bureau staff visited private corporations known for their innovative management approaches. These included the Saturn Automobile Company, and Hermann Miller, a manufacturer and seller of office furniture. Ideas picked up were used in developing new management practices for the Bureau.

Several important features characterized Commissioner Beard's approach to the restructuring.

- He used his political connections from previous work on Congressman Miller's staff and in the Interior Department to develop Congressional support, and support from the Secretary of the Interior, for the intended changes

[10] According to one senior Bureau manager, there is some pressure, presumably from Bureau area and regional offices, to replace some of the technical guidance which was eliminated.

- He built on the thinking and analysis which had already been done
- He transferred or fired several key career executives who carried over from previous administrations to ease the way for his proposed reforms
- He involved Bureau staff from both upper and lower staff grades in assessing the current and future operating environment for the Bureau and in developing strategies for change
- He authorized establishment of extensive support systems for individuals affected by the changes

Throughout the process the strongest motive for active involvement by career Bureau staff was the argument that if the Bureau does not take the lead in restructuring itself, others, e.g. Congress, will do it.

Innovations in American Government Award: In 1995, the Bureau of Reclamation received the prestigious *Innovations in American Government Award* sponsored by the Ford Foundation and administered by the John F. Kennedy School of Government at Harvard University. In that year, there were 1,450 applications for this award, submitted by various government units across the United States. Applications for the *Innovations* awards are evaluated on the basis of their creativity and effectiveness in responding to important problems, transferability to other jurisdictions, and value to the organization's clients. A panel of researchers and practitioners selected by the JFK School of Government reviewed the applications, conducted site visits, heard presentations, and questioned finalists. Finally, 10 programs were selected to receive the awards of $100,000 each. The Bureau was one of these recipients.

Accomplishments listed by the Bureau in submitting the application included:

- Reduced layers of management
- Established a network of area offices
- Delegated decision-making authority to the lowest practical level
- Reduced the work force by more than 1,500 employees between 1993 and 1994 and maintained a spirit of enthusiasm and commitment
- Eliminated many requirements, including nearly 6,500 pages of internal regulations
- Provided training and support to employees, including courses on managing change, open planning meetings, and intensive outplacement assistance

Political Economy of the Reforms

Political Control: The period over which major changes in U. S. water policy took place were marked by alternating political control in both administrative and legislative branches of government. Major reorganizations of the Bureau occurred under both Republican (1987-88) and Democratic (1993-94) Administrations, and the last major decline in funding (1989 -1996) occurred under both Democratic and Republican control of Congress. The conclusion must be that the changes resulted from the influence of major forces, larger than party politics, which affected the physical, economic, demographic, and ecological environment of Western water resources and the Bureau. As discussed earlier, these forces included the already extensive exploitation of Western water, urbanization, rising environmental consciousness, cumulative damage to natural ecosystems, and large public budget deficits.

Change Agents: Nevertheless, particular individuals played important roles as agents of change at particular times in the process. In the case of the two recent rounds of reforms of the Bureau, neither key change agent was an engineer, and both were Bureau "outsiders." It was Jim Ziglar, as Assistant Secretary for Water and Science in the Interior Department[11] who commissioned *Assessment 87* and oversaw the implementation of the reforms proposed in it. Ziglar was a Wall Street broker before becoming Assistant Secretary in the Reagan administration. He was very concerned about operations and maintenance of Bureau facilities and created budget proposals that doubled the level of funding for O&M during his tenure. His approach was direct and polarizing, often bypassing the Commissioner to work directly with senior Bureau staff. Hence the 1987 assessment was carried out with only limited involvement of the Commissioner's Office. The effectiveness of the reforms carried out was also limited by a failure to secure full congressional backing. It seems likely that the interventions of Ziglar speeded consideration of some of the hard choices which had to be made regarding the Bureau's future and were instrumental in generating the first set of reforms of the Bureau in 1988. However, it also left hard feelings and was incomplete.

The central figure in the second round of reforms was the incoming Commissioner, Dan Beard. Beard was a geographer by training, giving him a broad perspective on natural resource issues. More importantly Beard had held various jobs in Washington under Democratic Administrations, many of which related directly to water and the Bureau of Reclamation. He had been involved with the Carter White House Transition Team, which originally generated the "hit list" of Western water projects; had served as Deputy Assistant Secretary for Land and Water Resources (Interior Department) in the Carter Administration, and had been Staff Director for the Water and Power Resources Subcommittee for Congressman Miller (D-California) in the early 1980s[12]. Miller's support was instrumental in securing the Commissioner's appointment for Beard.

Beard took over leadership of the Bureau in 1993 and, aided by his familiarity with the issues, moved very quickly to implement the Bureau's second major round of reforms. He felt that the one weapon which he had that the "traditionalists" dispersed throughout the Bureau did not was the ability to move quickly. His job was made somewhat easier by the fact that the nature of many of the required changes was clear by this time, and what was needed was the will and political acumen to implement them. Once again, it was an "outsider" who directed the program of radical change. Declaring himself satisfied with the changes he had set in motion, Beard resigned his position after little more than two

---

[11] This is the position directly above the USBR Commissioner in the Interior Department structure.

[12] Miller, as head of the congressional sub-committee which had oversight responsibilities for the Bureau's program, had been a key player in many of the political dramas which marked the decline in water project funding and the restructuring of the Bureau and its move to Denver. Unfortunately, a discussion of this interesting history is beyond the scope of this paper.

years on the job to assume a leadership position with the Audubon Society, a major U S conservation organization.

Although many of the Commissioners of the Bureau have been civil engineers, especially in the early years, many have also come from other fields. Of the six commissioners since 1977 one was a pharmacist (Broadbent), another an accountant (Duvall), and a third a geographer (Beard), while the other three were engineers. Interestingly, two of the most aggressive of the commissioners during the boom years of irrigation construction were also non-engineers; Floyd Dominy was an economist by training, and Mike Straus, commissioner under Roosevelt and Truman, was a newspaperman. It seems clear that changing, or promoting, a water resource development program requires skills which are not necessarily taught in engineering college.

Locus of Power: During the first 60 years of its existence, roles and authority relationships within the Bureau had fallen into a well-established pattern. Policy was made in Washington and administrative control was exerted from there. The Denver Center exerted tight technical control and supervision over design processes, designs, and construction. Project Offices interacted with Denver through the Regional Offices who supervised them. Prior to *Assessment 87*, there had already been some shifts in authority as a result of regional pressure, and some regions had wrested increased autonomy in technical matters from Denver. However, the general pattern remained as described above.

The changes called for in *Assessment 87* and the *Implementation Plan* called for a sweeping consolidation of technical control, policy-making responsibility, and administrative control in Denver. Congress thwarted some aspects of this plan, requiring that the Commissioner and certain policy and administrative functions be retained in Washington, but still the result was a significant net shift of power and administrative authority from Washington to Denver. Planning formerly done in Regional Offices was also consolidated in Denver.

In 1994, the *Blueprint* reforms reversed this flow, consolidating policy and administrative control again in Washington[13]. At the same time, the Commissioner simplified and streamlined planning and design processes, eliminated many of the review and control steps which had constituted the Denver Center's technical power base. The result was a clear shift of overall power and control back to Washington and the Commissioner's Office. At the same time, the Commissioner reached out to the newly reconstituted Area Offices to make them the focal points of future programs, reducing somewhat, in the process, the authority of the Regional Offices. The result has been to make Washington DC the unequivocal headquarters of the Bureau and the Area Offices the primary action nodes for management and program implementation. The Denver Regional Service Center has become what its new title implies — a relatively self-contained entity purveying technical and administrative services to the Area Offices and other clients. The RSC receives only about 7% of its budget from appropriated funds, the rest coming from

[13] According to one senior Bureau manager, this reversal was actually begun under Commissioner Underwood, and was brought to completion under Beard.

reimbursements for services. The figure for the Technical Service Center is even lower, 3-4%.

Downsizing

Reducing Bureau staff levels was a major challenge of the reform process. Two primary mechanisms were used to accomplish this. The first, Reduction In Force (RIF), was employed in both 1988-89 and 1994-95. This is the standard and rather rigid civil service mechanism applied government-wide to separate redundant employees. It uses a formal set of rules, taking into account seniority, former military service, performance ratings, and so on, to determine which employees to separate. Individuals separated under a RIF receive severance pay and limited assistance for retraining and locating another job. Of the 600 or so people from the RSC who left the Bureau during the 1994-95 restructuring, 456 were subject to RIF procedures. About 200 of these voluntarily retired or took other jobs. The remainder were given termination notices. In the end, only 26 of these were actually discharged through the RIF process — the remainder having accepted buyouts (described below), been placed in another position, or left voluntarily in the meantime.

The second mechanism employed was a more flexible cash incentive program which paid employees cash bonuses to leave government service or to retire earlier than they would otherwise be required to. This was a government-wide program begun early in the Clinton Administration with the objective of reducing the Federal workforce by 200,000 people. The core of the program is the payment of up to $25,000 to individuals who voluntarily choose to leave government service. Accepting the "buyout" prohibits the individual from being reemployed by the US Government for five years. Agencies were required to permanently eliminate one position for every two employees who accepted cash buyouts. When buy-outs were done early in the fiscal year, the savings in salary and benefit costs paid for the buy-out within the same year. Although the size of the bonuses was not extraordinary, the practice was influential in inducing people to leave government service. Government agencies were permitted considerable flexibility in applying the provisions of the cash incentive program. While many agencies restricted access to the program to senior employees, the Bureau opened the program to employees at all levels. This was important in increasing the acceptance rate and in demonstrating an even-handed approach to all employees.

IMPACTS OF THE REFORMS

We must make fundamental shifts in what Reclamation does and in the way Reclamation operates — Dan Beard, USBR Commissioner, 1993-95

Budget

As seen in Figure 1 and discussed earlier, budget authorizations to the Bureau declined steadily after 1989, falling, in real terms, to less than half of the high of the late 1970s. The composition of the budget changed also. The share of O&M doubled since 1982, while the budget for investigating new projects fell in both absolute and relative terms and is now less than one-third of its 1982 budget share. The general administrative expense share, which increased somewhat during the most recent reforms, presumably to cover the costs of downsizing and restructuring, has now fallen to just below its long term average level of 5.6%.

The most dramatic change, however, is found in the share of construction expenditure. The construction share fell from nearly three-quarters of the total budget in the mid-1980s, to just over half today. In the past, the construction budget were 3 or 4 times the size of the O&M budget. Now they are about equal in size. Note that the construction budget figure includes rehabilitation and upgrading of existing facilities and construction to mitigate environmental damage as well as new facilities construction. Thus, while the share of construction may fall further, it will probably remain an significant component of the Bureau budget for the foreseeable future. By comparison, the *Water Infrastructure Fund* of the EPA, over the last 15 years or so, has averaged about 4 times the size of the USBR construction budget.

Personnel Levels

Bureau staff levels have fallen along with the total budget and the declining share of construction activity. In the 1950s, the USBR employed 10 to 20 thousand people. Not all of these were permanent staff, and the level fluctuated as projects were completed and new projects begun.

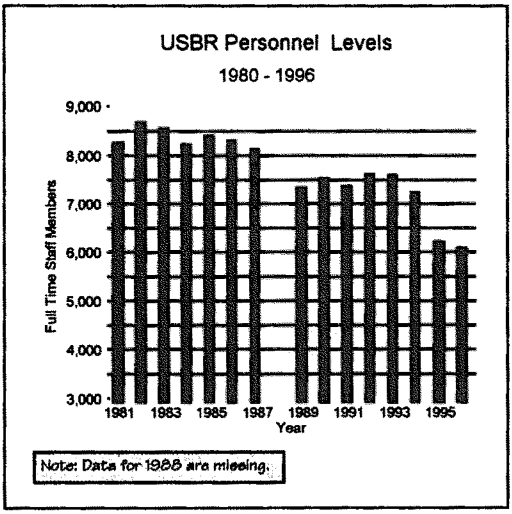


Figure 2. Total Number of Full Time Equivalent Staff Members Employed by the USBR

By the early 1980s the number of Bureau staff was between 8 and 9 thousand persons, and since that time it has declined to around 6 thousand. The two reorganizations can be seen clearly in Figure 2, with sharp declines occurring in 1988-89 and 1995. The decreases amounted to 9.7% of total staff in 1988-89 and 14.2% in 1995. Although the Bureau does not maintain readily accessible figures on changes in staff composition over time, one long-time official estimated that 25 years ago 80% of the staff were engineers, compared with perhaps 60% today.

Traditionally, about three-quarters of Bureau staff have been assigned to Project (now Area) Offices, with the remainder being posted in Denver and Washington, DC. Effects of the recent reforms can be seen in the distribution shown in Table 2. The share of people working in the field declined with the 1988 reforms, as the Denver office grew in relative size. The effect of the shifting Bureau headquarters to Denver in 1988 is also clearly seen, as the Washington DC share decreased from 3.4% in 1987 to 1.4% in 1989 while Denver expanded from 22.1% to 25.9% of the total. The changes of 1995-96 are equally clear, as the Washington Office was restored to its peak size of 3.4%, while Denver shrank by 1.7 percentage points. Interestingly, though one aim of the *Blueprint* reforms was to place more staff in the field, little impact is evident on the relative size of the regional and area office staff.

Table 2. Share of Personnel Working in Different Types of USBR Offices

	1981	1982	1983	1984	1985	1986	1987	1988
Regional & Area Offices	77.5%	78.1%	77.9%	78.4%	78.8%	76.0%	74.5%	
Denver Office	19.7%	18.9%	19.1%	18.2%	18.0%	20.6%	22.1%	
Washington Office	2.8%	3.0%	2.9%	3.4%	3.1%	3.3%	3.4%	

	1989	1990	1991	1992	1993	1994	1995	1996	81-96
Regional & Area Offices	72.7%	73.3%	72.0%	72.5%	73.0%	72.2%	73.8%	72.7%	74.9%
Denver Office	25.9%	25.4%	26.5%	26.1%	25.6%	26.5%	23.4%	23.9%	22.7%
Washington Office	1.4%	1.3%	1.4%	1.4%	1.4%	1.4%	2.9%	3.4%	2.4%

Benefits and Costs

Benefits of the restructuring of the Bureau were less monetary than political and institutional. To be sure, the Bureau's budget has declined, but this is a trend which predates the reforms and would have taken place in any event as support for new water resource projects dried up. What the reforms did accomplished was to help restore the credibility of the Bureau of Reclamation and help insure its survival and long-term relevance. The risk to its continued existence would have been much greater had it attempted to stone-wall and frustrate all attempts at reform. In the process, it has gained an opportunity to address today's problems and to be of renewed service to the nation. Finally, and perhaps most importantly, active involvement in the reform process has revitalized the Bureau, giving it a vigor and eagerness to face the future which it lacked in the recent past.

The financial costs, in comparison, were relatively minor. While the general administration line in the budget increased slightly during the time of the 1994-95 restructuring, the increases were relatively small and were more than compensated for by the reduction in staff costs resulting from the retrenchments. The costs in human terms was potentially more significant, but given the considerable attention which was devoted to helping employees adjust to the new situation, these costs too were relatively small. The most significant cost incurred was probably the partial dismantling of the Bureau's preeminent capacity to design, build, and repair large dams. However since this capacity was created largely in the process of building large dams in preceding decades, a reduction in capacity was, to a large extent, an inevitable consequence of the cessation of large dam construction.

In sum, the financial costs of implementing the reforms were relatively minor and the benefits, in terms of insuring the future viability and relevance of a strong institution with a proud history, and preserving a significant portion of its acknowledged technical capacity are considerable. The nation also gains by having this capacity redirected toward solving difficult second-generation problems affecting a vital natural resource.

THE BUREAU OF RECLAMATION TODAY

Reclamation is now evolving into the premier water management agency in the world — Eluid Martinez, USBR Commissioner, 1995-

Organizational Structure

The 1994-95 restructuring created an organizational structure with three distinct parts. The headquarters of the Bureau is located in Washington, DC, the center of national political power. From there, one axis runs through the Director of Operations to the Regions and the Area Offices. This is the action arm of the Bureau, designing and building projects, carrying out O&M, managing water and associated resources.

The other axis runs through the Director of the Reclamation Service Center in Denver to the support service groups in Denver. Of these, the most important for Bureau programs is the Technical Service Center (TSC), which provides technical analysis and support to Area and Regional Offices and other clients. The TSC has a current staff of about 725, out of a total Denver complement of about 1,450.

The TSC has probably undergone the most radical transformation of all the units in the Bureau. It was formed by combining the divisions of Resource Management, and Engineering and Research, which together had a staff complement of about 1,100 people. During the reorganization, all of the remaining 800 positions in the two predecessor divisions were declared vacant and then restaffed. The units making up the TSC had previously provided design support and oversight and control for the Regions and Project Offices. The TSC is no longer involved in oversight and monitoring, but serves as an in-house consultant to Area Offices and to outside organizations. In addition to the elimination of oversight responsibilities, the reorganization reduced the number of levels in the decision-making hierarchy, increasing the span of control. Levels of control within the TSC have been reduced from 3 or 4 previously, to 2. The corresponding span of control has increased from 1:4-5 up to 1:15-18.

Mode of Operation

The primary program activities of the Bureau today are carried out by the 27 Area Offices under their 5 respective Regional Offices. Both Area and Regional Offices have a considerable degree of operational autonomy today, and are accountable to the Director of Operations in Washington. They operate and maintain the large water control and power generation facilities which are the responsibility of the Bureau and design and construct projects, including mitigation projects — those designed to remedy problems caused by past water control projects or other human activities — and rural water supply projects.

The other main "action arm" of the Bureau, the TSC, has a mandate which covers laboratory and other special study activities and specialized technical design work. The capacity of the TSC is nearly fully utilized at present.

One innovative aspect of the TSCs operational mode is the way it organizes its work. Although individuals are assigned to work groups of 15 to 18 people each, tasks are carried out by *ad hoc* task groups made up of people with needed skills drawn from anywhere in the TSC. The appointed task manager is responsible for assembling the task group and managing its activity. When the task is completed, individual members return to their basic working groups. Accountability is to both the task manager and the work group manager using a matrix model. This provides a great deal of flexibility and responsiveness in responding to clients needs. The major potential negative aspect of such a model, confused lines of accountability, is said not to have been a serious problem to date.

The TSC today operates as a provider and seller of services, rather than a regulator and overseer. It has no programmatic responsibilities and is self-financing, in accordance with the *Blueprint* principle of "being fiscally responsible and using sound business practices." Virtually all of the work undertaken by the TSC is done under contract with another Bureau unit or with an outside organization such as the EPA. Currently about 40% of TSC work is done for the Bureau while the other 60% is done for outside clients.

The TSC is an evolving institution in a changing environment. There are contradictions in its mandate and mode of operation which remain to be sorted out, and its appropriate role and the set of core skills required to implement it require additional definition. It has retained a set of core competencies, however, and has developed a flexible and responsive operational mode which should serve it well during future periods of change.

The entire Bureau has now entered into an era where change is the rule rather than the exceptional event. One of the principal tasks of managers in the Bureau today is to stay abreast of the changing operating environment and to continually make the necessary adjustments in organizational structure and size, functions, partnerships, and modes of operation. In the words of the current RSC director, "the initial part [of the restructuring] was relatively easy (though I didn't think so at the time). Sustaining and continually adjusting is tough."

LESSONS LEARNED

Past development of Western water resources in the United States and changes in the socio-political environment made major changes on the part of the Bureau of Reclamation virtually inevitable. The process which the Bureau went through in making these changes provides useful lessons for others facing similar pressures. These lessons include the following.

- After initial resistance, the Bureau embraced the task of developing a new role and structure for itself and participated actively and creatively in the process. A similar organization which failed to do this, the U S Bureau of Mines, was abolished in 1995.

- Key parts of the change process were very participatory, particularly the second round in the 1990s. This resulted in a pragmatic and functional set of recommendations for changes to be made. It also had the effect of revitalizing the agency.

- Change agents were critical, on several occasions, in initiating key elements of the change process. These change agents came from outside the Bureau of Reclamation, were not engineers, and brought with them an external perspective and experience in business or politics.

- The reorganization of the Bureau did not take place overnight. It was a process which extended over a period of 10 years and was marked by several false starts, repeated studies, and changes in emphasis. However, there was a general accumulation of experience as each new effort was at least partially informed by previous studies and decisions. The general thrust of the various efforts was in a common direction.

- Although the restructuring did result in a smaller organization, there was far more to the restructuring than simply "downsizing." Layers of authority were eliminated, rules and regulations streamlined, working relationships made more flexible, and authority delegated downward.

- Widespread sharing of advance information about planned changes, cash incentives for early retirement or resignation, training for coping with change, and relocation assistance eased the transition process for most employees.

These lessons do not provide prescriptions for change in other organizations and other social and institutional environments. They were, however, effective in this particular context in restructuring and revitalizing an aging agency who's core activity was no longer relevant to the nation. These lessons must be evaluated and considered carefully by those who would apply them elsewhere in the light of particular political, social, and economic conditions.

REFERENCES

ASCE News. 1977. 22(4):1.

Beard, Daniel P. 1993. Blueprint for reform memorandum. USBR, Washington, DC. Photocopy.

Beard, Daniel P. 1996. Creating a vision of rives for the 21st century. Paper presented in Nagaragawa, Japan, 14 September.

Bureau of Reclamation. 1987. Assessment "87 ... A new direction for the Bureau of Reclamation. USBR, Washington, DC.

Bureau of Reclamation. 1988. Implementation Plan .... A new direction for the Bureau of Reclamation. USBR, Washington, DC.

Bureau of Reclamation. 1988. Implementation Plan: Update '89. USBR, Washington, DC.

Bureau of Reclamation. 1992. *Reclamation's strategic plan*. USBR, Washington, DC.

Bureau of Reclamation. n. d. 1992 summary statistics: water, land, and related data. USBR, Denver, CO.

Bureau of Reclamation. 1993. Report of the Commissioner's program and organization review team. USBR, Washington, DC.

Bureau of Reclamation. 1980. 1980 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1983. 1983 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1984. 1984 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1985. 1985 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1986. 1986 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1987. 1987 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1988. 1988 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1989. 1989 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1991. 1991 annual report. USBR, Washington, DC.

Bureau of Reclamation. 1991. The Reclamation Reform Act of 1982, annual report to congress. USBR, Washington, DC.

Bureau of Reclamation. 1992. Bureau of Reclamation accomplishments. USBR, Washington, DC.

Bureau of Reclamation. 1995. Annual Report for fiscal year 95. USBR, Washington, DC.

Bureau of Reclamation. 1995. Semi-finalist application to the Innovations in American Government 1995 Awards Program. USBR, Washington, DC.

Bureau of Reclamation. 1996. Technical Service Center directory of services. USBR, Denver, CO.

Bureau of Reclamation. 1997. Website.

Ford Foundation. 1996. Innovations in American government: an awards program of the Ford Foundation administered by the John F. Kennedy School of Government at Harvard University. Ford Foundation, New York.

Guy, Sam. 1995. Restructuring of the USBR. Paper for the World Bank Water Resources Seminar, 12-13 December. World Bank, Washington, DC. Photocopy.

Microsoft. 1994. Encarta.

Reisner, M. and S. Bates. 1990. *Overtapped oasis: reform of revolution for western water*. Island Press, Washington, DC.

Reisner. M. 1993. *Cadillac Desert*. Revised edition. Penguin Books, New York.

Svendsen, Mark, and Douglas Vermillion. 1994. *Irrigation management transfer in the Columbia River Basin, USA: lessons and international implications*. IIMI Research Paper. Colombo, Sri Lanka: International Irrigation Management Institute.

Technical Service Center, USBR. 1996. TSC Memorandum No 0, The Technical Service Center. USBR, Denver, CO.

Technical Service Center, USBR. 1996. TSC Memorandum No 1, Acceptance and accomplishment of work within the Technical Service Center. USBR, Denver, CO.

Technical Service Center, USBR. 1996. TSC Memorandum No 2, Accountability and quality assurance for products through signatory authority and peer review. USBR, Denver, CO.

Technical Service Center, USBR. 1996. TSC Memorandum No 3, Development and use of service agreements. USBR, Denver, CO.

Technical Service Center, USBR. 1996. TSC Memorandum No 4, Occupational safety and health within the TSC. USBR, Denver, CO.

Technical Service Center, USBR. 1996. Services Outline. USBR, Denver, CO.

PERSONS MET

Washington

Daniel P. Beard, Commissioner, USBR, 1993-1995
Molly Bucky, Program Coordination and Budget Division, USBR
C. Dale Duvall, Commissioner, USBR, 1987-1989
Sam Guy, Head, International Programs Division, USBR, 1984-1995
Richard Ives, Head International Programs Division, USBR
Bill Klostermeyer, Assistant Commissioner for Administration, USBR, 1981-1989
Bob Wolf, Chief, Program Coordination and Budget Division, USBR

Denver

Philip H. Burgi, Manager, W R Research Laboratory, Reclamation Service Center
Felix Cook, Sr., Director, Technical Service Center, Reclamation Service Center
Blaine Dwyer, President, ECI, The Water Resources Division of Frederic R. Harris, Inc
Kathy Gordon, Director, Management Services, Reclamation Service Center
Charles Hutton, Structural Engineer, ECI, The Water Resources Division of Frederic R. Harris, Inc
John C. Lease, Leadership Team, Environmental Resources, Reclamation Service Center
Terry P. Lynott, Director of Program Analysis, 1988-1994
Jim Malila, Director, Reclamation Service Center
Leanna Principe, International Affairs Team, Reclamation Service Center
Margaret Sibley, Director, Human Resources, Reclamation Service Center

PERSONS CONTACTED BY PHONE

Ernie Balenzuela, Training Officer, Reclamation Service Center
Jim Cole, Director, Yakima Area Office, 1995-1997
Jerry Ganiere, Human Resources Office, Reclamation Service Center
Cynthia Hancock, Human Resources Office, Reclamation Service Center
John Harb, Manager, Irrigation Drainage Program, Reclamation Service Center
Gayle Shanahan, Program Coordination and Budget Division, USBR
Rod Vissia, Regional Director (Boise), 1973-1980.
Ed Walsh, Annual Planning Budget Division, US Environmental Protection Agency
Darrel Weber, Director, Reclamation Service Center, 1983-1993

THE AFFECT OF URBANIZATION ON THE COST

OF OPERATING AN IRRIGATION DISTRICT OR CANAL COMPANY

John Wilkins-Wells[1] Thayne Coulter[2]

ABSTRACT

This is a report of data collected on irrigation districts and mutual ditch and irrigation companies (what we broadly term *irrigation enterprises*) in the intermountain region, and comprising the states of Colorado, Idaho, New Mexico, Utah and Wyoming. The data are from a three year study funded by the U.S. Bureau of Reclamation and entitled Irrigation Enterprise Management Practice Study (IEMPS).[1] The study was located at Colorado State University. A variety of data were collected on thirty-six irrigation districts and canal companies, including operation/maintenance and administrative costs from 1945 to 1995. The combined effective irrigated acreage served by the sample is 1,478,720 acres (1995), or a little over one-tenth of all intermountain irrigated lands served by gravity canal systems. Costs reported are, in effect, the water costs borne by farmers for surface gravity supplies provided by local irrigation enterprises. What follows is (1) a brief overview of average water costs, and (2) a more in-depth analysis of the affect of urbanization on the cost of operating these two forms of enterprises. The conclusion is that urbanization in the region appears to be having a dramatic affect on operating costs, and therefore the price paid by farmers for water supplied through these enterprise.

BACKGROUND

The urbanization of prime irrigated lands is clearly one of the major issues facing the West today.[2] The intermountain region, an area that has served for many years as a summer recreational haven for those wishing to escape the heat and noise of urban living, is now experiencing rapid and permanent urban settlement. This is occurring in many areas traditionally dominated by irrigated agriculture and its associated economy. Most of the desirable locations for new urban growth have been developed through the years, in great part, by irrigated agriculture. These

---

[1]Senior Research Scientist, Special Assistant Professor, Department of Sociology, Colorado State University, Fort Collins, Colorado, 80523.

[2] Sociologist, Resource Management, Bureau of Reclamation, Denver Federal Center, P.O. Box 25007-D8580, Denver, Colorado, 80225-0007.

select localities have a good water supply to support urban growth and ample vegetation in an otherwise very arid environment.

At the same time, many communities, particularly those founded on agriculture, mining and tourism, frequently appear unprepared to deal politically or economically with this new urban growth. Many traditional agricultural communities have seemingly resigned themselves to the inevitable change. Others are attempting to gain some control of urbanization through county or municipal ordinances, covenants and zoning that are designed to protect prime irrigated lands.[3]

However, it is not only the number of people, but the changing values and priorities associated with this demographic shift that are of concern to farmers. Differences and conflicts between rural and urban values and priorities are observed in local newspapers daily. Gone are the days when urban residents could casually claim that their parents or grandparents were farmers, and therefore felt a strong kinship with agriculture. Present urban residents are often two or three generations removed from the farm, and their knowledge of farming needs is therefore minimal.

Costs Associated With Urbanization

As urbanization gradually creeps into the service areas of local irrigation districts and canal companies, challenges are made to informal agricultural water exchange agreements and reservoir storage release scheduling that have been practiced over many years. Traditional water exchanges and diversions which have often been practiced as a means of stretching scarce water supplies to overcome peaks and valleys in river flows, and to improve cooperation between enterprises competing over water, are now often viewed by urban dwellers and municipal planners as obstructions to the aesthetic value of rivers meandering through urban corridors. Challenges are frequently made that the traditional practices of irrigation enterprises destroy wetlands, riparian habitat and natural river meanders. One frequently hears that a "new legal order" must be instituted.[4] Even when agriculturalists point out that many of these local wetlands and other habitats were largely created by irrigated agriculture, it is often of no avail.

Urban dwellers tend to view waterways and vegetation created by irrigation enterprises as a haven for light recreation; walking, birdwatching, biking, running, and other activities. All of these activities increase the demands on irrigation enterprise management, and their legal liabilities in numerous ways. Irrigation enterprise rights-of-way are routinely challenged by urban dwellers, developers and planning agencies. The petition for right-of-way crossings to facilitate developer needs, or the request (often in the form of a demand) made by municipalities to bury canals and laterals are major cost items for these enterprises today. Lawsuits associated with accidents along irrigation enterprise rights-of-way may not necessarily lead to unfavorable settlements for these enterprises, but such lawsuits almost inevitably result in substantial costs to retain attorneys to protect enterprise assets.

Irrigation Enterprise Operating Costs

Reporting average costs per irrigated acre, or cost per acre foot of water delivered to the farm headgate, must be treated with extreme caution. There was no strong correlation in the IEMPS data between the size (acreage) of an irrigation enterprise service area and the cost per irrigated acre borne by farmers in financing the enterprise. The cost of operating an enterprise in the intermountain region varies greatly because of differences in physical location, canal engineering and administrative requirements. Therefore, the cost of water for irrigated agriculture varies considerably from one enterprise to the next. Water cost can vary by $20 per acre or more between irrigation enterprises immediately adjacent to each other.

In the IEMPS sample of thirty-six enterprises, and for the year of 1995, operation/maintenance and administrative costs (and therefore the price paid by farmers for water supplied through these nonprofit enterprises), ranged from $5.00 per irrigated acre to $78.00 per irrigated acre, with an average of about $20.68 per irrigated acre. The $20.68 per irrigated acre cost can be broken down as follows: $7.75 per irrigated acre for total O & M costs, $11.87 per irrigated acre for administrative costs, .88¢ for debt payment on the irrigation enterprise and .17¢ for special project costs.

Prorating an average cost of $20.68 per irrigated acre for a sample representing 1,479,000 irrigated acres, and an effective water supply of approximately two acre feet per acre for the region, would give an average cost of $10.34 per acre foot for water supplied by enterprises in the intermountain region. This would mean that for an average size farm of 280 acres, the annual water bill would be about $5,790 dollars. It is believed that these are generally conservative estimates for irrigation water costs in the region.

The cost of irrigation water in the intermountain region may be understated by researchers working in the field. This is primarily due to the fact that (1) farm labor costs associated exclusively with water application, (2) farm capitalization costs associated exclusively with irrigation, and (3) pumping of supplemental water from private irrigation wells are often ignored in the calculation of farm water cost.[5] These additional costs may double the cost per unit (acre foot) of water applied to the land. Only the cost of gravity surface delivery by irrigation enterprises is generally used. This is all that is reported in the present study when the value of $20.68 per irrigated acre is indicated for the sample.

The decade of the 1970s appears to be one in which irrigation enterprise operation/maintenance and administrative costs begin to show their most rapid increase. Some of this is undoubtedly due to the general inflation in the economy at this time. However, the trend continues unabated thereafter. Another jump in cost occurs in the 1980s and continues into the present. Much of this later jump in costs appears due to urbanization in the intermountain region, plus the overall affects of the general increase in environmental regulations, including the administrative burden associated with environmental requirements. Discussions with irrigation enterprise managers and board members clearly point to urbanization as a major factor in explaining the increase in costs of approximately 300 percent between 1975 and 1995.

It will come as no surprise that the relationship between the cost of operating an irrigation enterprise, and the degree--and perhaps even the pattern--of urbanization being experienced by the county that the irrigation enterprise is located in, is an important one. This relationship is a major focus of the IEMPS research effort. The problem has been to develop methods that will allow quantitative data to be used to assess and monitor this relationship over time. These will be discussed momentarily.

A major contributor to administrative costs appears to be employee salaries. Some of this increase is obviously due to incrementally higher pay rates for higher skilled managers, office secretaries and additional field staff over the years. The cost of employee health and retirement benefits is rapidly growing in importance too. IEMPS found that employee salaries were generally sixty percent (60%) higher for enterprises located in urbanizing counties compared to enterprises located in more rural counties.

IEMPS cannot at this time state with any degree of precision what percentage of irrigation district or canal company operating costs are directly the result of accommodating the specific needs of urbanization. Examples of these needs include dealing with housing subdivision development within the irrigation enterprise service area, dealing with a rapidly rising rural non-farm population of hobby farmers and rural-to-urban commuters, litigation over protection of irrigation enterprise rights-of-way, increased costs of liability insurance for irrigation enterprises and other urbanization-generated costs. These costs are often disguised in the annual expense statements of these enterprises. Attempts were made by IEMPS to extract these urbanization-based costs in various ways, since they are rarely reported as separate line items in enterprise expenditures.

As mentioned earlier, most of the thirty-six irrigation enterprises in the sample are located in separate counties. Each of these counties in turn show unique demographic characteristics, based on an analysis of U.S. Census data going back to World War II. The census data have been useful in developing a typology or classification of county demographic trends specifically designed for IEMPS research needs. We will now turn to a discussion of these demographic types and what they tell us about the affect of urbanization on irrigation enterprise costs

COUNTY DEMOGRAPHIC TYPES

The analysis of demographic data on fifty-six important irrigation counties in the intermountain region, of which thirty-six have an irrigation enterprise that IEMPS collected data on, appears to show three principal demographic types for our interest.[6] They are: 1) the urbanized agricultural county; 2) the quasi-urban agricultural county, and; 3) the typical rural agricultural county. These three demographic types are more or less defined by the particular configuration of three principal population groups in these counties over time. These population groups are: 1) the urban population, 2) the rural non-farm population, and; 3) the rural farm population. Table 1 shows the county affiliation of thirty-six irrigation enterprises relative to these three demographic types. We will now examine these three types.

Type #1 - Urbanized Agricultural County

There are several counties in the intermountain region that still possess valuable irrigated lands, despite having a sizable urban population. These counties might be considered as urbanized agriculture counties, rather than urban metropolitan counties. Such counties appear to have been in the process of becoming slowly urbanized since shortly after World War II. Their urban population usually comprises 66% or more of the total county population today. However, it will come as no surprise that they are usually close to a major metropolitan area in the region, such as Denver, Boise or Salt Lake City.

Larimer County, Colorado and Davis County, Utah seem to represent examples of this demographic *Type #1* agricultural county, where irrigated agriculture is still relatively important, but where enormous growth pressures are being exerted daily on irrigated agriculture. The demographic trends for Larimer and Davis counties, along with the population numbers for the three principal population groups (urban, rural non-farm and rural farm) are shown in Figures 1 and 2.

Irrigated acreage in Larimer County has decreased by about 18% since 1949, while irrigated acreage in Davis County has decreased by about 27% over the same period. Despite these decreases, the acreage remaining in irrigation is still important in both counties, and is of course served by important canal companies in these counties too.

In summary, *Type #1* counties tend to show a trend toward urbanization beginning in the 1950s, continuing steadily into the 1990s. Being adjacent to a major

Table 1 - Classifying Thirty-Six Irrigation Enterprises by Three County Demographic Types	
Location of Irrigation Enterprise = Type #1 - Urbanized Agricultural County	
1. Progressive Irrigation District, Bonneville Co., Idaho 2. Cub River Irrigation Company, Cache Co., Utah 3. Davis and Weber Counties Canal Company, Davis Co., Utah 4. Elephant Butte Irrigation District, Dona Ana Co., New Mexico 5. North Poudre Irrigation Company, Larimer Co., Colorado 6. Grand Valley Irrigation Company, Mesa Co., Colorado 7. Orchard Mesa Irrigation District, Mesa Co., Colorado 8. Bessemer Ditch and Irrigating Company, Pueblo Co., Colorado 9. Strawberry Highline Canal Company, Utah Co., Utah	**N = 9**
Location of Irrigation Enterprise = Type #2 - Quasi-Urban Agricultural County	
1. Bear River Irrigation Company, Box Elder Co., Utah 2. Dry Gulch Irrigation Company, Duchesne Co., Utah 3. Riverton Valley Irrigation District, Fremont Co., Wyoming 4. Goshen Irrigation District, Goshen Co., Wyoming 5. Pine River Irrigation District, La Plata Co., Colorado 6. Uncompahgre Water Users Association, Montrose Co., Colo. 7. Bijou Irrigation District, Weld Co., Colorado 8. Catlin Canal Company, Otero Co., Colorado 9. Shoshone Irrigation District, Park Co., Wyoming 10. Heart Mountain Irrigation District, Park Co., Wyoming 11. Wheatland Irrigation District, Platte Co., Wyoming 12. San Luis Valley Irrigation District, Rio Grande Co., Colorado 13. Gunnison Irrigation Company, Sanpete Co., Utah 14. Twin Falls Canal Company, Twin Falls Co., Idaho 15. Burley Irrigation District, Cassia Co., Idaho 16. Upper Hanover Irrigation District, Waskakie Co., Wyoming 17. Henrylyn Irrigation District, Weld Co., Colorado	**N = 17**
Location of Irrigation Enterprise = Type #3 - Rural Agricultural County	
1. Trinchera Irrigation Company, Costilla Co., Colorado 2. Cottonwood Creek Canal Company, Emery Co., Utah 3. Huntington-Cleveland Irrigation Company, Emery Co., Utah 4. Egin Bench Canal Company, Fremont Co., Idaho 5. North Side Canal Company, Jerome/Gooding Co., Idaho 6. Butte and Market Lake Canal Company, Jefferson Co., Idaho 7. Abraham Irrigation Company, Millard Co., Utah 8. Julesburg Irrigation District, Sedgwick Co., Colorado 9. Salmon River Canal Company, Twin Falls Co., Idaho 10. North Sterling Irrigation District, Morgan Co., Colorado	**N = 10**

Figure 1 - Larimer County, Colorado

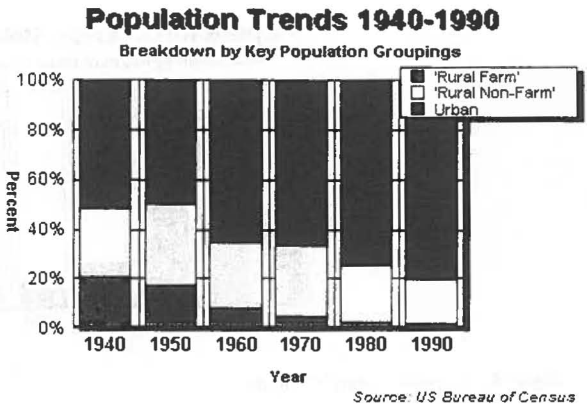


Figure 2 - Davis County, Utah

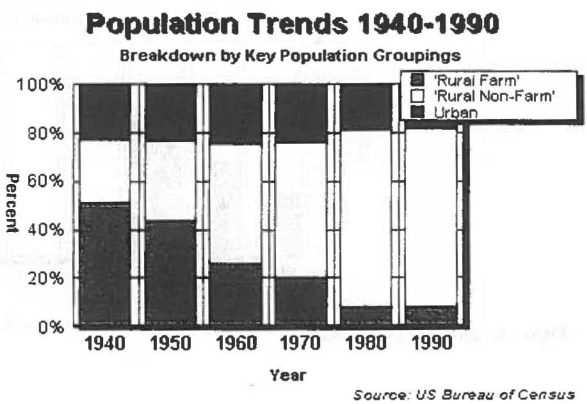


Figure 3 - Delta County, Colorado

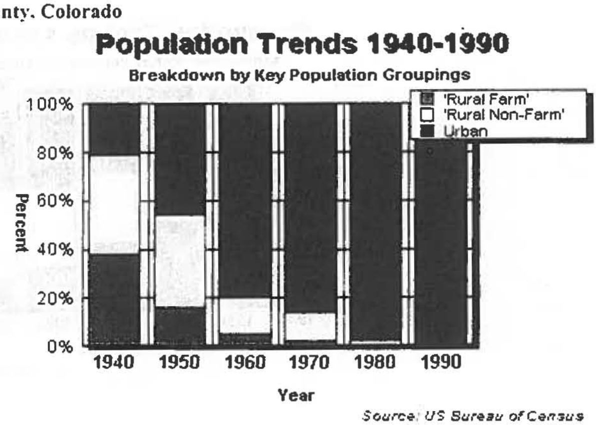


Figure 4 - Goshen County, Wyoming

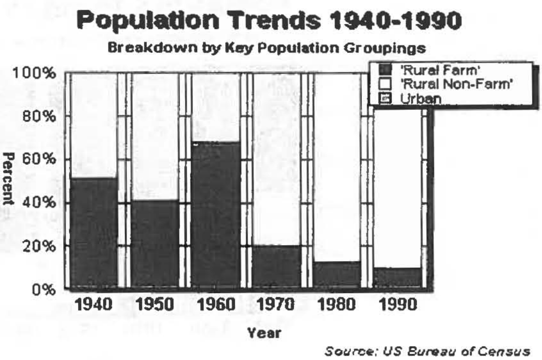


Figure 5 - Sedgwick County, Colorado

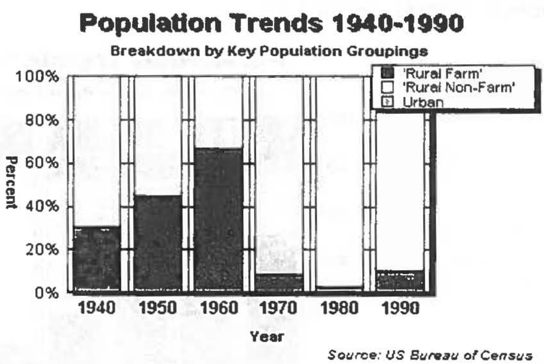
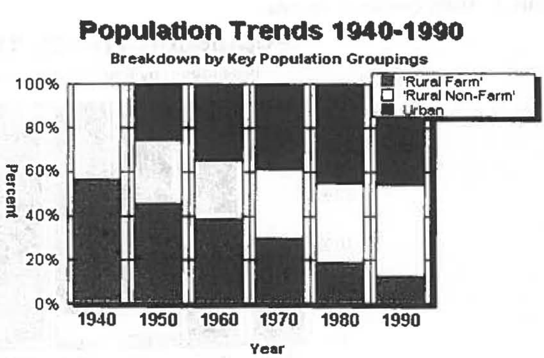


Figure 6 - Emery County, Utah



metropolitan area pretty much defines their growth pattern. Larimer County is an example of an agricultural county that was nevertheless considerably urbanized prior to World War II; being 52% urban in 1940. It is close to the Denver metropolitan area. A regional land grant college was present early on, and several large corporations began moving into the county in the 1960s. On the other hand, Davis County had an urban population of only 21% in 1940. It is located just north of Salt Lake City. Some of the counties of this type, such as Davis County, were *Type #2* counties (our next type described below) shortly after World War II. However, Davis County moved rapidly into the *Type #1* category as Salt Lake City and Ogden grew and people began moving into suburbs developing in neighboring Davis County.

Type #2 - Quasi-Urban Agricultural County

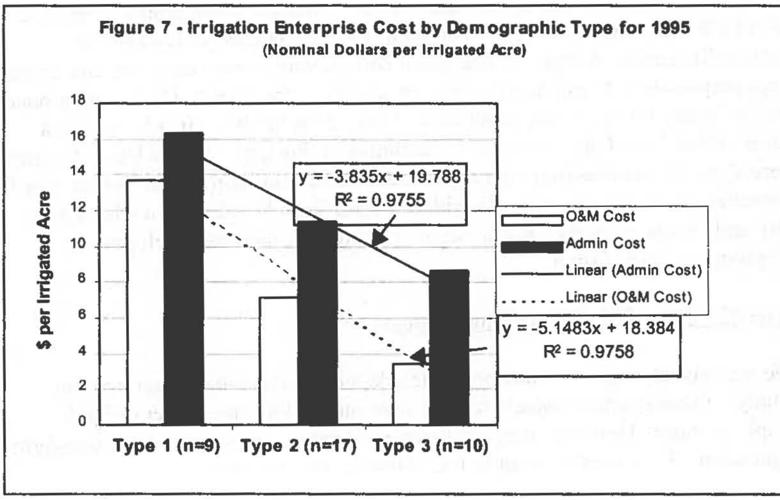
The next county type we encounter in the data is the quasi-urban agricultural county. These counties usually have at least one major trade center of 5000 people or more. However, they are unique in having a substantial rural non-farm population. They tend to occur in the following two situations:

> 1) Several small trade centers combine to make up a mixture of both urban and rural non-farm population. However, the rural non-farm population often makes up 40% to 60% of the total county population. This is generally not the result of the presence of a large nearby metropolitan area, but rather the result of several small trade centers. Delta County, Colorado (Figure 3) and Goshen County, Wyoming (Figure 4) might be examples;
>
> 2) The county has a substantial rural non-farm population spilling into it from an urban metropolitan county or an urbanized agricultural county and its economy (*Type #1* above). Cassia County, Idaho and Sevier County, Utah might be examples (demographic charts not shown).

Regardless of which of the two *Type #2* situations one is discussing, this quasi-urban agricultural county shows an unusually large rural non-farm population relative to the two other population groupings. This rural non-farm population is not engaged in farming, but rather employed in the local agri-business sector or in the neighboring metropolitan area economy. Long-distance commuting by car to work may be noticeable on local state or interstate highways in these counties.

Type #3 - Typical Rural Agricultural County

Finally, there is the typical rural county where the urban population is very small. However, the rural farm population may be only about seven to ten percent of the



total population as well. In fact, today, there are almost no important irrigation counties in the intermountain region that have a rural farm population greater than fifteen percent. Irrigated agriculture tends to produce viable local trade centers supporting a sizable rural non-farm population. Consequently, in these typical rural agricultural counties, the rural non-farm population predominates as well. It is just that the rural farm population remains significant at the same time. Examples of this *Type #3* demographic pattern might be Sedgwick County, Colorado (Figure 5) or Emery County, Utah (Figure 6).

In summary, we would anticipate that the cost of operating an irrigation enterprise in this third demographic type would be the least costly environment. On the other hand, the urbanized agricultural county (*Type #1*) might be expected to show the most costly environment for irrigation enterprises. Finally, irrigation enterprise costs in *Type #2* counties might be expected to fall somewhere in between irrigation enterprise costs in *Type #1* and *Type #3* demographic environments. We can examine this relationship statistically.

Again, Table 1 shows the names of thirty-six irrigation enterprises for which information on operation/maintenance and administrative costs were available for 1995. Figure 7 shows the average overall operation/maintenance cost, and then the average administrative cost alone, for the three county types. It is clear that the *Type #1* demographic pattern shows higher costs per irrigated acre for irrigation enterprises. We will now turn to some supporting evidence for our thesis that urbanization results in increased costs for irrigation enterprises, and therefore the price of water paid by farmers.

THE BEALES CODE

Dr. Calvin Beales is a U.S.D.A. sociologist who developed a numerical classification scheme for more carefully identifying and understanding the demographic character of counties.[7] The Beales Code is a useful and widely accepted typology for classifying counties on a rural-urban continuum, and is based on a variety of economic, social and political factors. The numerical continuum runs from "0" signifying most urban county to "9" signifying most rural county. We will use this classification to further explore and substantiate the effect of urbanization on the cost of operating an irrigation enterprise.

Using IEMPS data from two reporting years (1975 and 1995), Table 2 and 3 show comparisons of the cost per irrigated acre of operating an irrigation enterprise in two rural-urban subgroupings, using the earliest (1974) and the most recent (1993) Beales classification. These data tend to show that the cost per irrigated acre is lower when these enterprises are located in more rural (6 to 9) counties, than in more urban (0 to 5) counties.

Table 2 - Irrigation Enterprise Costs (**Combined O & M and Administrative Costs**) Per Irrigated Acre By Beales Code (1974, 1993)

BEALES COUNTY CODE CLASSIFICATION	Average IE Cost Per Irrigated Acre in 1975	Average IE Cost Per Irrigated Acre in 1995
0 to 5 (A Predominantly Urban County)	$6.51	$22.82
6 to 9 (A Predominantly Rural County)	$5.91	$19.35

Based on 16 irrigation enterprises having complete cost data for 1975 and 1995.

Note also the relatively dramatic increase in operational costs in the short span of twenty years. Irrigation enterprise costs per irrigated acre are similar for the two subgroupings in 1975. However, they are considerably different twenty years later. This may be explained by the fact that many of the urban counties in 1975 were really our *Type #2* counties (quasi-urban agricultural counties) discussed in the previous section; whereas in 1995 these same counties have moved into the *Type #1* county demographic pattern and are more clearly urban in nature. Overall, Table 2 and 3 tend to further substantiate our thesis about the affect of urbanization on irrigation enterprise costs over the years.

When the three county demographic types discussed earlier are taken together with the Beales Code analysis, the data seem to give strong support to the thesis that urbanization increases the cost of operating an irrigation enterprise. Rarely can the costs associated with urbanization be passed on by farmers to the very juggernaut that is imposing them. Growers absorb the cost. It comes out of their farm income.

CONCLUSION

Due to the relatively low returns to primary agricultural production in the region--despite the importance of regional commodity production, and feed production for the cattle/calve, dairy, hog and sheep industry--farmers tend to be very vulnerable to rapidly increasing water costs. A water budget of two acre feet per acre on a 280 acre farm, with an average water cost of $10 per acre foot, would result in an annual water bill of about $5,600. A 30 percent increase in the cost of operating an irrigation enterprise delivering water to the same farm--and primarily as a result of increased operation and administrative costs associated with urbanizing influences--would increase the water bill for this same farm by $1,700. This scenario is not unrealistic considering the information shown here on trends in irrigation enterprise costs associated with urbanization.

Table 3 - Irrigation Enterprise Costs (**Administrative Costs Only**) Per Irrigated Acre By Beales Code (1974, 1993)

BEALES COUNTY CODE CLASSIFICATION	Average IE Cost Per Irrigated Acre in 1975	Average IE Cost Per Irrigated Acre in 1995
0 to 5 (A Predominantly Urban County)	$3.44	$14.83
6 to 9 (A Predominantly Rural County)	$2.01	$12.08

Based on 16 irrigation enterprises having complete cost data for 1975 and 1995.

These factors clearly reflect one type of crisis for irrigated agriculture in the region. One alternative for many irrigated farms is to make greater use of groundwater where available--when the cost of groundwater is less than the surface supplies provided by the local irrigation enterprise. However, this strategy may potentially deplete local groundwater resources. Another strategy is simply to sell the farm and benefit from rapidly increasing farm real estate values. There may be no other alternative for a farm located a considerable distance from the river, and relying upon a canal system to obtain water, when confronted with a sizable increase in its irrigation enterprise water bill due to urbanizing influences. These relationships are often poorly understood, frequently perverse and extremely dangerous to the preservation of food producing areas in the intermountain region.

Urbanizing influences appear to lead to increasing costs of operating an irrigation enterprise. These costs are passed on by the nonprofit enterprise to individual farmers, who in turn are absorbing the costs of urban expansion onto irrigated lands that were developed in part through the U.S. Bureau of Reclamation revolving fund and private capital. The problem is that these and other market factors affecting water rates for irrigated agriculture and reallocation of river basin supplies are generally oblivious to the prioritization of land use. The way of life represented by agriculture and its important value system aside, Class I and Class II farm land is placed in harm's way due to the frequent unwillingness or inability of county and local government to deal effectively and timely with urban sprawl and its associated rural non-farm population growth.

If farmers could pass on at least an equitable share of the costs of the urbanization of agricultural lands to food consumers, the economic impact on agriculture might be considerably lessened. However, these costs are difficult for agriculture to pass on to the consumer. One may argue that water costs could be alleviated through improved water use and conservation. However, on-farm water conservation is primarily paid for by farmers through their individual investment in new and more efficient irrigation technologies, and/or increased labor costs to improve water application on the farm. Concurrently, technological improvements for the irrigation enterprise itself cannot be passed on to the general public. These costs associated with the urbanization of irrigated lands are now absorbed almost entirely by farmers. There is little discussion today about the public's need to share in these costs, costs directly associated with how urban dwellers utilize land.

In recent years, irrigation enterprises have been forced to develop special water rates for subdivisions and other fractional water users to achieve equity in the relationship between water used and the water service provided. These water rates are frequently set higher for fractional water users than for bulk users represented by actual income-producing farms. This is one important way to defray the cost of urbanizing influences, but often does not get at the problem of preserving the geographical integrity and physical assets of irrigation enterprise service areas. There are long-term investments that all farms in an irrigation enterprise service area have made in developing the canal infrastructure over many years. This past investment cannot be easily recuperated through special water rates for non-agricultural users; thereby ensuring equity in benefits received from the irrigation facilities relative to the individual investments made over many years.

IEMPS has shown that thirty-six irrigation enterprises, for which data was available in the single year of 1995, reported $214,202,003 in fixed assets (the book value of primary canal systems only), or approximately $157 per acre of irrigated land. The book value of irrigation infrastructure in the 5 state intermountain region, and again for primary canal systems only, is conservatively estimated at $1.4 billion based on IEMPS data. Dams, local water storage facilities, and major pumps and diversion structures are not included in this book value.

Each converted acre of irrigated land to urbanization potentially represents an income loss from future assessments needed to maintain the operating condition of these valuable canal assets. This is a troubling trend. If irrigated lands and the associated canal infrastructure are needed to meet future food production needs, which they inevitably will, the cost of moving onto Class III and Class IV lands because of the loss of Class I and Class II lands--and the associated cost of reinvesting in new irrigation system infrastructure to serve these new lands--will result in ever increasing costs of producing crops. If past trends continue, farmers may be expected to absorb these costs as well.

The urbanization of agricultural counties is generally not supported by a proper and detailed assessment of the long-term costs and consequences of losing these agricultural assets. Despite the constant warning about the loss of agricultural land, it is the loss of "prime irrigated lands" to urban development that poses the most concern for food production in the future. However it is not just the land, but also the valuable irrigation water delivery infrastructure that has been developed over the years--frequently with federal support--and at a great cost to agricultural incomes in the past. Finally, the trends noted here potentially represent the loss of much needed farm income, viable farms and experienced farmers.

REFERENCES

1. J. Wilkins-Wells, R.L. Anderson, G.A. Pavelis and M..S Anwer, *Irrigation Enterprise Management Practice Study*, U.S. Bureau of Reclamation, Project # EI.D4.4, January (1999).

2. U.S.D.A., *Agricultural Resources and Environmental Indicators, 1996-1997.* An Economic Research Service Report, Agricultural Handbook No. 712, July (1997).

3. J. Freedgood, *Saving American Farmland: What Works*, USDA, Natural Resources Conservation Service (1997).

4. C.F. Wilkinson, *Aldo Leopold and Western Water Law: Thinking perpendicular to the Prior Appropriation Doctrine*, Land and Water Law Review, 24:1 (1989).

5. G. A. Pavelis, *Natural Resource Capital Formation in American Agriculture: Irrigation, Drainage, and Conservation, 1885-1980.* USDA, Economic Research Service, Natural Resource Economics Division (1985).

6. Part of this analysis has been informed by N. Brooks, D. Reimund and R.N. Peterson, *Effects of Population Growth and County Type on Farm Structure, 1970-1980.* USDA, Economic Research Division, Agriculture and Rural Economy Division.

7. P.J. Cook and T.F. Hardy, *Updating the ERS County Typology.* USDA, Economic Research Service, Agriculture and Rural Economy Division, December (1993).

AN ECONOMIC PERSPECTIVE ON THE ENVIRONMENTAL COSTS AND BENEFITS OF IRRIGATED AGRICULTURE

Dennis Wichelns[1]

ABSTRACT

A conceptual framework that depicts some of the private and public costs and benefits of irrigated agriculture is presented for use in identifying situations in which public policies might be implemented to generate a socially optimal use of resources in a competitive equilibrium. The framework is useful in describing the potential social gains or losses due to policies that motivate farmers to internalize the external costs or benefits of their activities. The model is demonstrated using the example of water quality issues pertaining to irrigation and drainage in California's San Joaquin Valley. The potential social costs of public policies designed to reduce the volume of subsurface drain water and selenium loads discharged into the San Joaquin River are examined using the conceptual framework.

INTRODUCTION

In many areas of the world, irrigated agriculture generates both private and public costs and benefits. Farmers have an economic incentive to maximize private net benefits, which are the returns they receive in excess of production costs. In competitive markets, private net benefits will be the same as public net benefits, provided that all costs and benefits are internalized in farm-level decisions. When some costs or benefits are external to farm-level decisions, the social optimum will not be achieved in a competitive equilibrium. In those cases, public policies may be required to motivate farmers to consider pertinent external costs or benefits.

Public policies must be chosen carefully to motivate the desired changes in farm-level decisions, without causing undesirable distortions in resource use or in the set of crops produced. Accurate information regarding external costs and benefits will assist public officials in identifying situations where public policies are required to generate a socially optimal use of resources, and in selecting appropriate policy instruments and parameter values. Useful information includes a conceptual framework that describes the nature of pertinent external costs and benefits, and empirical estimates of those costs and benefits.

[1] Assoc. Professor, Univ. of Rhode Island, Kingston, Rhode Island

The goal of this paper is to present a conceptual framework for evaluating the external costs and benefits of irrigated agriculture, for use in examining public policies that may be implemented to generate a socially optimal use of resources in a competitive equilibrium. The framework is useful in describing the potential social gains or losses due to policies that motivate farmers to internalize the external costs or benefits of their activities. The model is demonstrated using the example of water quality issues pertaining to irrigation and drainage in California's San Joaquin Valley. The potential social costs of public policies designed to reduce the volume of subsurface drain water and selenium loads discharged into the San Joaquin River are examined using the conceptual framework.

CONCEPTUAL FRAMEWORK

In general, the private (farm-level) costs and benefits of irrigated agriculture are described by the prices paid and received by farmers in input and output markets. Market demand curves for farm products generate derived demand curves for farm inputs that describe the incremental values generated by employing additional units of inputs. For example, the farm-level incremental values of irrigation water are determined by the market value of the incremental output generated by delivering additional water. Farmers determine the profit-maximizing amount of water, per unit of land area, by comparing their expectation of incremental benefits with incremental costs. The farm-level incremental cost of irrigation water is constant when farmers face a flat-rate pricing structure.

It is easiest to describe the incremental costs and benefits of one variable input, such as irrigation water, while assuming that the use of other inputs is constant. A graph depicting the typical conceptual relationship between the incremental costs and benefits of irrigation water is presented in Figure 1. The farm-level profit-maximizing choice is denoted as W_F, where incremental cost is equal to the incremental benefit.

The farm-level value of a fixed input, such as land, is determined by summing the discounted present value of net returns that can be generated by keeping the input in production during an appropriate time horizon. For example, farmers expecting to earn \$600 ha^{-1} in net returns from cotton production, in perpetuity, should be willing to pay any price up to \$10,000 ha^{-1} for land if their discount rate is 6 percent. The incremental value of land may decline with farm size for an individual farmer if other resources, such as capital or management, are in limited supply. Variation in land quality will cause the aggregate incremental value curve to be downward sloping, as farmers employ the best land first, before extending production to lower quality land.

A graph depicting the typical conceptual relationship between the incremental costs and benefits of land in production is presented in Figure 2. The farm-level profit-maximizing choice is denoted as L_F, where incremental cost is equal to the incremental benefit. The incremental cost curve is shown as upward-sloping in this example to denote the situation in which land of a given quality is limited in supply. In such cases, farmers desiring additional land will face an upward-sloping supply curve. The incremental benefit curve is downward sloping to reflect diminishing returns to additional land, caused by declining land quality or by capital and management constraints.

In competitive markets in which producers and consumers have full information regarding prices and quantities, farm-level choices regarding fixed and variable inputs will be socially optimal, if all pertinent costs and benefits are included in the curves depicted in Figures 1 and 2. Farm-level decisions will not be socially optimal when input use or resource allocation decisions generate external costs or benefits that increase or reduce measures of social welfare, while not affecting farm-level net returns. Classic examples of agricultural externalities include the use of fertilizer and pesticides that degrade water quality in streams or aquifers receiving surface runoff from farm fields.

In arid regions, the volume of irrigation water delivered to farm fields must exceed crop water requirements to maintain salt balance. The excess water contributes to surface runoff and deep percolation that enter regional drains or shallow water tables. Agricultural drainage water containing salts, boron, or other elements may degrade water quality in receiving streams and aquifers. When this occurs, farm-level decisions regarding water use per unit of land area may not be socially optimal. Figure 3 depicts a situation in which the social incremental costs of irrigation lie above the private incremental costs. The vertical distance between the two curves, at any point, denotes the external incremental cost at that volume of water per hectare. When irrigation generates external costs, the socially optimal water volume, W_S, will be less than the farm-level profit-maximizing volume, W_F. In those cases, public policies that increase the farm-level cost of water may be appropriate to motivate farmers to reduce water use from W_F to W_S.

The external effects of land in agricultural production are often considered to be positive, rather than negative (Hodge, 1991). For example, farmland that is managed appropriately can provide wildlife habitat benefits that might not be available if the land is left idle or is developed for housing or industry. Similarly, farmland can provide scenic vistas of pastoral scenes that are enjoyed by residents of local towns and others traveling along nearby roads and highways. Farmland also generates watershed protection benefits and may be useful in protecting productive soil resources for future generations. These benefits are considered external to agricultural production decisions because they accrue to society, at large, rather than to individual farmers. Empirical estimates of amenity values from farmland support the conceptual notion of external benefits (Bergstrom et al., 1985; Kline and Wichelns, 1996; Ready et al., 1997).

Figure 4 depicts a situation in which the social incremental benefits of farmland lie above the private incremental benefits. The two curves are separated by a vertical distance that describes the external incremental benefits. When this occurs, the socially optimal area of land in agriculture, L_S, will be greater than the farm-level profit-maximizing area, L_F. Public policies that increase the farm-level benefits from farming or reduce the costs of keeping land in production may be appropriate to encourage farmers to cultivate additional land, increasing the area in production from L_F to L_S.

Public policies that encourage the retention of agricultural land include special tax assessment programs that enable farmers to pay taxes according to farm values, rather than values that reflect development potential. Farmers are required to keep their land in agriculture for a specified number of years, in exchange for the reduced tax assessment (Aiken, 1989). Many states and some local governments also purchase the development rights to farmland, so that farmers may benefit financially from development values, while keeping their land in agriculture. These programs are often funded by bond issues that are approved in statewide referenda, reflecting public support for agricultural land preservation programs (Kline and Wichelns, 1994). Such support is also strong in Europe, where public appreciation of agriculture's role in generating and maintaining rural landscapes has led to the use of management agreements and covenants with farmers for the purpose of preserving rural landscapes (Whittaker, et al., 1991).

The conceptual framework described in Figures 3 and 4 depicts a situation in which some of the variable inputs used in irrigated agriculture may generate negative externalities, while the land in production may generate positive external benefits. The social net benefits of agricultural production can likely be enhanced by designing public policies that address these issues, provided that the policies recognize the sources of external costs and benefits. For example, policies intended to reduce the external costs of selected variable inputs will be more efficient if they are designed to address the use of those variable inputs, directly, with minimal impact on the use of complementary inputs that may generate external benefits. The potential social gains and losses of policy alternatives are demonstrated by using this conceptual framework to examine irrigation and drainage issues in California's San Joaquin Valley.

AN IRRIGATION AND DRAINAGE EXAMPLE

Much of the subsurface drain water collected beneath farmland on the west side of California's San Joaquin Valley contains selenium, boron, and other elements that occur naturally in local soils and are leached from the profile during irrigation and drainage activities (Letey et al., 1986; Deverel and Gallanthine, 1988; Gilliom, 1991). The U.S. Environmental Protection Agency has established a national water quality criterion for selenium of 5 parts per billion (ppb), when measured as a 5-day moving average concentration. Selenium concentrations in farm-level drainage systems vary geographically within the region, ranging from less than 10 ppb to 4,000 ppb (Deverel et al., 1984). Drain water concentrations at individual drainage systems are relatively consistent, over time, and are correlated with drain water salinity (Deverel, et al., 1989).

The California Regional Water Quality Control Board for the Central Valley Region has adopted a Basin Plan Amendment designed to achieve the national selenium concentration standard, over time. That amendment includes a set of monthly and annual selenium load targets that are expected to generate acceptable selenium concentrations in the near term, while farm-level and regional efforts are implemented, over time, to achieve the national water quality standard.

The Regional Water Quality Control Board has also established a 2-ppb selenium water quality objective for sloughs and ditches in a wetland habitat located between an agricultural production area and the San Joaquin River. As a result, it became necessary to remove agricultural drainage water from wetland channels, as selenium concentrations in drainage water are often in the range of 20 to 100 ppb (Deverel et al., 1984; Presser and Barnes, 1985). Seven irrigation and drainage districts in the region have formed a regional drainage authority to construct and operate drainage facilities, and to coordinate efforts to achieve the selenium load targets. The group has constructed a new channel that carries drainage water from all seven districts around the wetland area to an existing portion of the San Luis Drain, which carries the water to a tributary of the San Joaquin River. This program, which is known locally as the Grassland Bypass Project, began operating in September of 1996.

The monthly and annual selenium load targets are substantially lower than estimates of historical discharges. As a result, farmers and districts have had to implement aggressive programs to motivate farm-level improvements in irrigation practices that will reduce surface runoff and deep percolation, and to increase the blending and re-use of drainage water with fresh water supplies. Innovations in district policies have included tiered water pricing, low-interest loans for purchasing new irrigation systems, and restrictions regarding surface runoff discharged into drainage ditches. Farmers have responded by replacing traditional surface irrigation methods with improved furrow methods, gated pipe, and sprinkler systems. They have also increased the labor and management components of irrigation activities.

District efforts to motivate farm-level improvements in water management are consistent with the incremental cost and benefit framework shown in Figure 3. Charging higher prices for irrigation water raises the incremental cost curve, motivating farmers to reduce water deliveries from the original W_F, in the direction of W_S. Policies that restrict farm-level surface runoff or drain water volume will also raise the incremental cost curve, as farmers must recycle their drainage water or improve water management to reduce the volume of drainage water generated. These activities raise the implicit cost of water deliveries, even if the explicit price of irrigation water remains the same.

The average volume of water delivered per hectare in the drainage problem area has declined as a result of farm-level and district efforts to reduce drain water volume and selenium loads (Wichelns and Cone, 1992a and 1992b; Wichelns, et al., 1996). However, despite these efforts and the observed reductions in water deliveries, drain water volumes and selenium loads have not been reduced proportionally. It appears that exogenous forces including rainfall and subsurface flows of shallow groundwater into the drainage problem area may contribute significantly to the volume of drain water collected in local drainage systems. As a result, selenium load targets have been exceeded in many months since the Grassland Bypass Project was started. However, negative impacts of selenium concentrations in excess of water quality objectives have not yet been observed.

THE ROLE OF UNCERTAINTY IN POLICY CHOICES

Efforts to reduce water deliveries from the farm-level optimum, W_F, to the social optimum, W_S (Figure 3), have increased the costs of farm-level and district water management. These costs may be justified, from an aggregate efficiency perspective, if the social optimum is, indeed, W_S. However, if scientific information regarding the potential impacts of selenium on aquatic wildlife in the Grassland Area and the San Joaquin River is not yet complete, public officials may not know with certainty the true effects of selenium in those environments. As a result, the selenium load targets may not be truly optimal, and the costs involved in reducing drain water volume and selenium loads to achieve those targets may represent a social loss.

Conceptually, when the potential environmental effects of a constituent are uncertain, the expected incremental costs of an activity that generates that constituent or discharges it into the environment will diverge from the true incremental costs. For example, if the potential effects of selenium are over-estimated, the true social incremental costs of water deliveries will be less than the expected social incremental costs (Figure 5). Regulations or voluntary efforts that reduce water deliveries from W_F to W_S^e will generate social costs in the form of unnecessary expenditures on water management and reductions in output values. The size of the social loss is a function of the relative slopes of the incremental cost and benefit curves. The net social loss in Figure 5 is shown as the shaded area below the incremental benefit curve and above the true social incremental cost curve.

Social losses can also be imposed by policies directed toward inputs not directly responsible for generating an externality. For example, there is much discussion in the San Joaquin Valley regarding land retirement as a policy alternative for reducing drain water volume and selenium loads. Several state and federal agencies have allocated either staff time or program funds in recent years to encourage farmers to retire farmland. The premise motivating this effort is that a smaller volume of subsurface drain water will be generated in the region if a smaller land area is farmed. Unfortunately, the physical relationships that generate drain water volume are not yet understood sufficiently to predict with accuracy the potential reductions in drain water volume and selenium loads that might be accomplished by land retirement.

A land retirement program may generate social losses if it is not successful in reducing drain water volume and selenium loads, or if the environmental effects of selenium are less costly than expected. As shown in Figure 6, the socially optimal area in production, L_S, exceeds the area chosen by farmers, L_F, in cases where the public derives external benefits from farmland. A land retirement program that reduces the land in production to a land area such as L_R will generate a net social loss described by the shaded area in Figure 6. That area represents the potential net social benefits that could be gained by expanding the land area in production from L_R to L_S. The direct costs of implementing a land retirement program, such as the funds required to purchase land and to administer the program, would increase the net social loss.

SUMMARY

Public policies may be appropriate for motivating farmers to modify their profit-maximizing decisions regarding the use of fixed or variable inputs when those decisions generate external costs or benefits. Classic examples of agricultural externalities include the use of fertilizer and pesticides that can reduce water quality in streams and aquifers. The generation of subsurface drain water may also involve an externality if chemicals or naturally occurring constituents in the drain water degrade wildlife habitat in receiving areas. Positive externalities from agriculture include the provision of wildlife habitat, scenic views, and watershed protection in areas that might be developed for other purposes.

The conceptual framework presented in this paper can be used by public officials to analyze the potential social gains and losses of policies designed to achieve socially optimal levels of agricultural activities. Appropriate policy alternatives and parameter values can be determined by comparing incremental cost and benefit information from both the farm-level and social perspectives. This task is particularly important when information regarding the incremental costs and benefits of agricultural activities is uncertain or incomplete, as some policy alternatives will be more effective than others in maximizing the likelihood that social goals will be achieved in those situations. Viewed from another perspective, the social losses that may result from policy implementation when incremental costs and benefits are uncertain will vary among policy alternatives.

The usefulness of this conceptual framework has been demonstrated by examining policy alternatives for reducing drain water volume and selenium loads on the west side of California's San Joaquin Valley. Policies designed to reduce water use by modifying the price of irrigation water may be appropriate in situations where excessive deep percolation generates incremental external costs in excess of incremental benefits. However, policies to reduce the area of land in production may generate social losses in the form of reduced output and reductions in the environmental amenities provided by agricultural land. The likelihood that land retirement will generate social losses is enhanced by the lack of information regarding the effects of such a program on drain water volume and selenium loads, and uncertainty regarding the impact of selenium on receiving waters in the region.

ACKNOWLEDGMENTS

This paper is Rhode Island Agricultural Experiment Station Publication 3689. Support from the Water Conservation Office in the California Department of Water Resources is appreciated.

Figure 1. Incremental costs and benefits of irrigation water

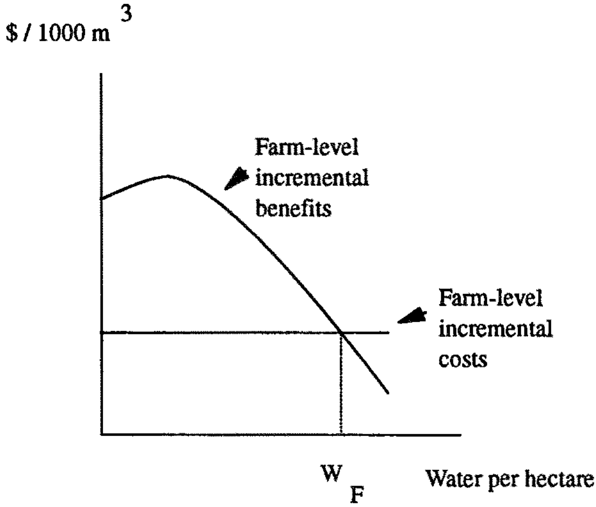


Figure 2. Incremental costs and benefits of agricultural land

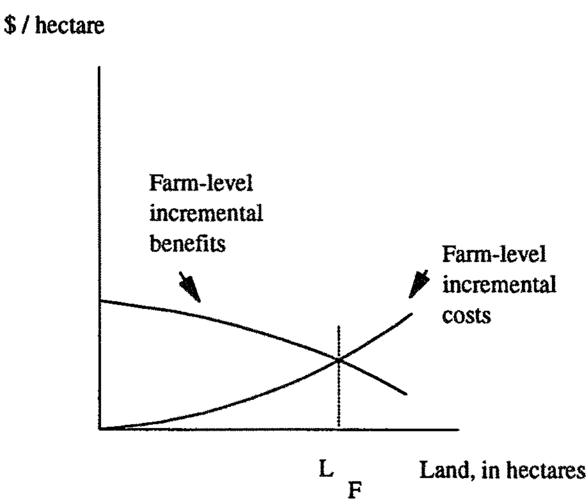


Figure 3. Farm-level and social costs and benefits of irrigation

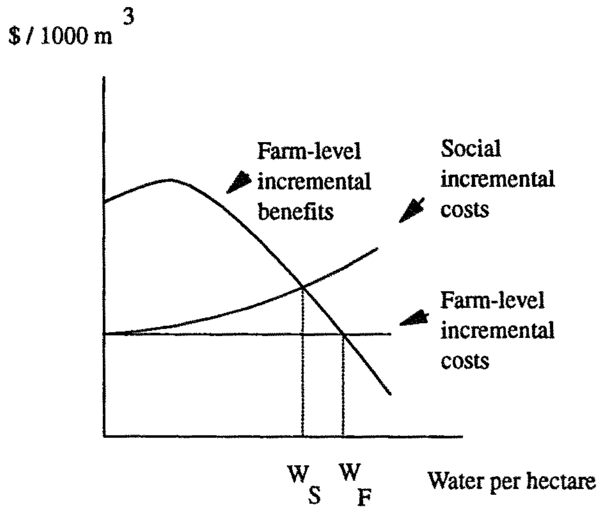


Figure 4. Farm-level and social costs and benefits of agricultural land

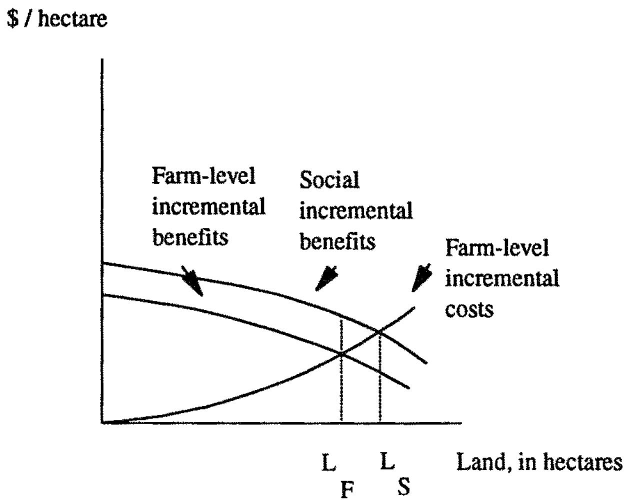


Figure 5. Expected and true social incremental costs

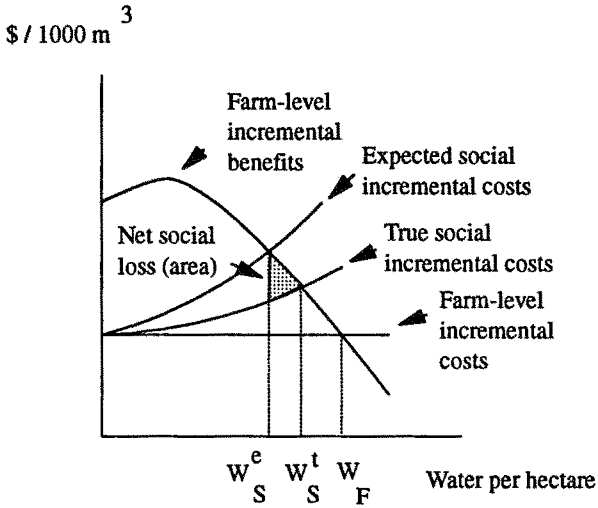
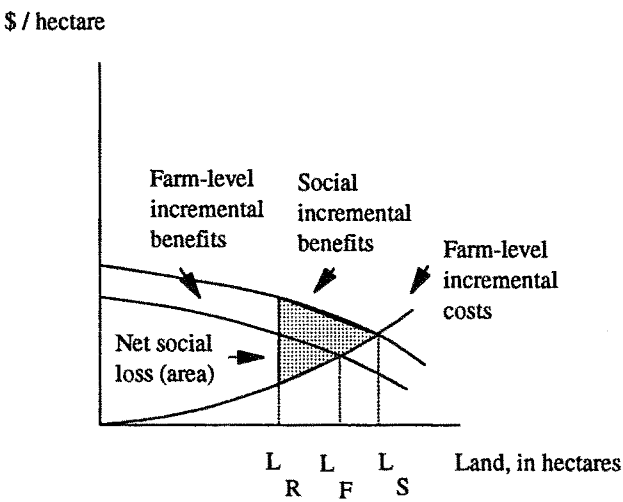


Figure 6. The potential net social cost of a land retirement program



REFERENCES

Aiken, J.D., 1989. State farmland preferential assessment statutes. Research Bulletin 310, Department of Agricultural Economics, University of Nebraska.

Bergstrom, J.C., B.L. Dillman, and J.R. Stoll, 1985. Public environmental amenity benefits of private land: the case of prime agricultural land. Southern Journal of Agricultural Economics 17:139-149.

Deverel, S.J., R.J. Gilliom, R. Fujii, J.A. Izbicki, and J.C. Fields, 1984. Areal distribution of selenium and other inorganic constituents in shallow ground water of the San Luis Drain Service Area, San Joaquin Valley, California - a preliminary study. U.S. Geological Survey Water-Resources Investigations Report 84-4319.

Deverel, S.J. and S.K. Gallanthine, 1988. Relation of salinity and selenium in shallow groundwater to hydrologic and geochemical processes, Western San Joaquin Valley, California. Open-File Report 88-336, U.S. Geological Survey, Sacramento, California, 23 pp.

Deverel, S.J., J.L. Fio, and R.J. Gilliom, 1989. Selenium in tile drain water. In: Gilliom, R.J., et al., 1989. Preliminary assessment of sources, distribution, and mobility of selenium in the San Joaquin Valley, California. U.S. Geological Survey Water-Resources Investigations Report 88-4186.

Gilliom, R.J., 1991. Overview of sources, distribution, and mobility of selenium in the San Joaquin Valley, California. In: Dinar, A. and D. Zilberman (eds.), The Economics and Management of Water and Drainage in Agriculture. Kluwer Academic Publishers, Boston.

Hodge, I., 1991. The provision of public goods in the countryside: How should it be arranged? In: Hanley, N. (ed) Farming and the Countryside: An Economic Analysis of External Costs and Benefits. C.A.B. International, U.K.

Kline, J. and D. Wichelns, 1994. Using referendum data to characterize public support for purchasing development rights to farmland. Land Economics 70(2):223-233.

Kline, J. and D. Wichelns, 1996. Public preferences regarding the goals of farmland preservation programs. Land Economics 72(4):538-549.

Letey, J., C. Roberts, M. Penberth, and C. Vasek, 1986. An agricultural dilemma: drainage water and toxics disposal in the San Joaquin Valley. Special Publication 3319, Division of Agriculture and Natural Resources, University of California.

Presser, T.S. and I. Barnes, 1985. Dissolved constituents including selenium in waters in the vicinity of Kesterson National Wildlife Refuge and the West Grassland, Fresno and Merced Counties, California. U.S. Geological Survey Water-Resources Investigations Report 85-4220.

Ready, R.C., M.C. Berger, and G.C. Blomquist, 1997. Measuring amenity benefits from farmland: hedonic pricing vs. contingent valuation. Growth and Change 28:438-458.

Whittaker, J.M., P. O'Sullivan, and J. McInerny, 1991. An economic analysis of management agreements. In: Hanley, N. (ed) Farming and the Countryside: An Economic Analysis of External Costs and Benefits. C.A.B. International, U.K.

Wichelns, D. and D. Cone, 1992a. Farm-level and district efforts to improve water management during drought. Irrigation and Drainage Systems, 6:189-199.

Wichelns, D. and D. Cone, 1992b. Tiered pricing motivates Californians to conserve water. Journal of Soil and Water Conservation, 47(2):139-144.

Wichelns, D., L. Houston, and D. Cone, 1996. Economic incentives reduce irrigation deliveries and drain water volume. Irrigation and Drainage Systems, 10:131-141.

IMPROVING WATER PROPERTIES TO INCREASE INFILTRATION CHARACTERISTICS

Richard W. O. Soppe[1] Stuart W. Styles[2]

ABSTRACT

Water properties, such as the viscosity and surface tension, can be affected by temperature and surfactants to increase infiltration rates into soils. Specifically, they will change the hydraulic conductivity of the soil. A simple soap solution and the new material PAM (inexpensive polymer chemical) were evaluated as surfactants. Laboratory experiments and field tests on a site in Davis, California were done to quantify the effects of changing the water properties. Additional effects, like the improved soil structure during infiltration and less soil particles in tailwater (reduced erosion due to runoff) were observed and are described in this paper. The conclusions of this study are translated into suggestions for improved on-farm water use in furrows, sprinklers, and drip irrigation.

INTRODUCTION

Infiltration is an important factor in irrigated agriculture in California. It is an intriguing subject: Infiltration depends on the porous media characteristics and on the properties of the fluids that saturates the media. Soil structure, furrow spacing, compaction, surface sealing, tillage and water quality are some of the factors that modify the infiltration rate of soils. Many studies had been developed to determine the influence of some of these parameter on the infiltration rate. Most of the literature refers to soil characteristics and very few

[1] Graduate PhD Student, UC Davis, Hydrologic Science, Davis, CA 95616.

[2] Project Manager, Cal Poly Irrigation Training and Research Center, San Luis Obispo, CA 93407

to the properties of the fluid used to irrigate. Grismer (1986) analyzed the effect of the pores size distribution of the soil over infiltration. Grismer (1994) also studied the effect of air compression and counterflow on infiltration into soils.

Fluid properties are not always studied in relation to infiltration, other than empirically (salt concentration effect on infiltration, adding surfactants). The studies on fluid properties focus on viscosity changes due to temperature effect (Dane and Hopmans, Duke, 1992) and the effect of surfactants on surface tension and density. Viscosity and density are both directly related to the hydraulic conductivity. The surface tension only has an effect on the infiltration as a result of air water interfaces.

This project intends to measure the impact of surfactants by evaluating the change in surface tension, density, and viscosity. In order to evaluate this effect, an optimum concentration of surfactants should be determined. This concentration should maximize the effect of adding a surfactant without reaching a point of diminishing return. We also present the effect of changing the temperature of irrigation water.

EQUATIONS (THEORY)

The following aspects make infiltration an intriguing subject:

- Infiltration rates vary during an irrigation.
- Many design strategies proposed for surface irrigation require knowledge of the precise mathematical constants in advance and infiltration equations.
- Each soil has different infiltration characteristics.
- Infiltration can vary with subsequent irrigations of the same field.
- Laboratory determinations of mathematical constants for the infiltration rates are not the same as unadjusted field results.

- Infiltration rates have traditionally been very difficult to evaluate in the field.

Field Measurement Equations. Several formulas have been developed to describe the advance and infiltration rates as a function of time. The most common form of the depth infiltrated equation is:

$$D = Kt^n \tag{1}$$

where,

D = the depth infiltrated (usually in. or cm.)
K = a constant
n = a constant
Both K and n are soil dependent
T = the opportunity time in minutes

If the constants (K and n) can be determined for a soil and irrigation configuration for a particular event, one can calculate the depth infiltrated at any point if the opportunity time at that point is known. By differentiating the cumulative intake, the equation for an instantaneous intake rate can be determined.

The basic form of the infiltration equation is:

$$I = nC\ T^{n-1} \tag{2}$$

where,

I = Instantaneous intake rate at a point
nC = constants
T = Opportunity time at the point

The constants "nC" in the equation must be determined for every irrigation. On the same soil with the same moisture content, the nC values for furrows will be different than for border strips. There is no reliable and transferable table of nC values available for different soils under furrow, border strip, etc. Figure 1 shows the general relationship of the intake rate for different soil types.

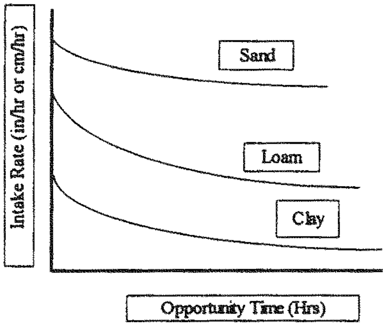


Fig. 1. Hypothetical Relation of Intake Rate to Time for Three Soils. Assumes the Same Percentage of Flooded Soil Surface Area for All Three Soils.

Laboratory Measurement Equations

The basic equation for flow through porous media is given by Darcy (19xx) as:

$$q = -K_{sat} * \frac{\partial H}{\partial L} \tag{3}$$

where,

q = flux
K_{sat} = saturated hydraulic conductivity
H = hydrostatic and elevation potential
L = length over which H occurs

Darcy showed this equation to be true for saturated flow, but it has also been shown that the relation is valid for unsaturated flow, when the unsaturated hydraulic conductvity is used.

The hydraulic conductivity is affected by fluid properties as well as the properties of the porous medium. Poiseuille (19xx) created the following relation:

$$K_{sat} = \frac{k * \rho * g}{\eta} \tag{4}$$

where,

k = instrinsic hydraulic permeability
ρ = density of the fluid

g = gravimetric constant
η = kinematic viscosity of the fluid

Using the properties of water, it can be seen that the hydraulic conductivity is also temperature dependent, since the kinematic viscosity of water changes significantly with temperature. Jaynes (1990) combined the temperature dependent kinematic viscosity with the Darcy equation for unsaturated flow, which resulted in:

$$q(T) = -\frac{\eta_r}{\eta_T} * K_r(h) * \frac{\partial H}{\partial L} \tag{5}$$

where,

η_r = kinematic visc. of the fluid at 21 degrees C
η_T = kinematic visc. of the fluid at temperature T
$K_r(h)$ = unsaturated hydraulic conductivity

Warm water has a lower viscosity than cold water. Equation 5 shows that the ratio of the kinematic viscosity will increase with temperature, resulting in a larger flux through a porous medium. Water with different concentrations of PAM have higher viscosities than water without PAM added. Equation 4 shows that the hydraulic conductivity for fluids with higher viscosities will be lower, hence resulting from equation 3 in a reduced flow through porous media. A surfactant like soap added to water will not change the viscosity significantly, nor the density of water. The only fluid property that changes is the surface tension. The surface tension of a fluid only affects the entry pressure, as shown in Equation 6:

$$h = \frac{2\sigma \cos\alpha}{g} \tag{6}$$

where,

h = air entry pressure
σ = surface tension of a fluid
α = angle of contact between fluid and solid, ($\cos\alpha$ is normally assumed to be 0 in small capillary tubes)
g = gravimetric constant

In the soil water retention curve, the air entry pressure is the pressure when the soil will actually release water when the absolute hydraulic pressure is larger then air entry pressure (see Fig. 2).

Lowering the air entry pressure on a saturated soil with a small negative hydraulic head by adding a surfactant would result in the release of water, and the curve in Fig. 2 would shift down. For infiltration in a dry soil with a high soil matric potential this would not make much difference. However, on a molecular level, water with a low surface tension would be able to access smaller pores, hence wetting the soil more thoroughly and increasing the actual soil moisture content (θ), resulting in a hydraulic conductivity that more closely represents the saturated hydraulic conductivity. A higher hydraulic conductivity will result in a higher flux through the soil.

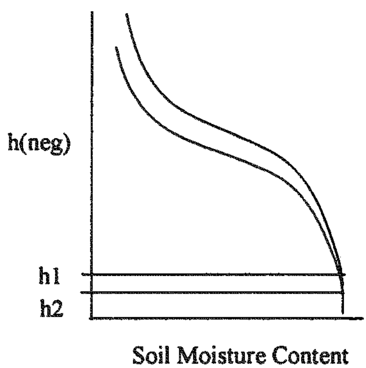


Fig. 2. Theoretical Soil Moisture Retention Curve (Upper Curve) and Lowered Air Entry Pressure (Lower Curve)

LABORATORY TESTING RESULTS

Constant Head Flow Hydraulic Conductivity

Several columns of 10 cm length by 5 cm of diameter were careful packed and filled with the experimental soil (fine sand) in such a way that the density of the soil is similar over the whole length of the experimental column. These columns of soils were setup to measure the hydraulic conductivity for a constant head. From this system the difference in distance between the bottom where water flows out and the top where water is open to the atmosphere is measure in order to determine the total head over the soil column.

Constant head flow experiments were performed for each one of the fluids chosen previously. For each different fluid we used a newly created soil column, so that residual PAM or soap did not influence the other measurements. The soil column was rinsed with each fluid several times to ensure a saturated flow. The volume of fluid over time was measured to calculate the saturated hydraulic conductivity. Four fluids were used: di-water at room temperature, PAM with a concentration of 10 mg/l, PAM with a concentration 1000 mg/l and a soap solution with a concentration of 4 ml/l.

Saturated Hydraulic Conductivity

Results from this experiment showed that the saturated hydraulic conductivity for soapy water was slightly lower than for distilled water. The low concentration PAM showed even a slightly lower saturated hydraulic conductivity. However, the variability of the measurements was high, and differences might not be significant. It is no surprise that there is not a large difference between distilled water and soapy water, since the surface tension is the main difference between the two types of water, and the surface tension does not have any effect on the flow of water through saturated soil. The high concentration PAM, of which the results are not shown, formed a gel-like layer on top of the soil column and did not allow for any water to infiltrate. This might explain why the low concentration PAM shows a slightly lower saturated hydraulic conductivity than the other two fluids. Since PAM keeps the structure of a soil in the field, it is not expected to have a large effect on the infiltration in a sifted soil without much structure.

Water	Average K_{sat} [cm/hr]
Distilled Water	5.46E-05
Soapy Water	3.81E-05
10 mg/l PAM	1.94E-05

Unsaturated Hydraulic Conductivity

A constant head device was connected to one side of a horizontal soil column according to the method described by Bruce and Klute (19xx). A positive head equal to half the diameter of the soil column was applied. The soil column was packed to a constant density using sifted soil (Yolo Sandy Loam). Two different fluids were allowed to infiltrate in the unsaturated soil column for three hours. The soil column was then divided into slices of 1 cm wide, and of each slice, the water content was determined. Using an empirical equation [add equation?] a regression line was created. Using this equation, the diffusivity was obtained according to the method described by Bruce and Klute(19xx). Results for a trial using tap water and a trial using soapy water are shown in Fig. 3.

The results indicate that, although the surface tension does not occur in the Darcy equation, that this is a parameter that affects the infiltration of water in a soil. With a reduced surface tension in the soapy water, advance of the water front is faster, but wetting is not as thorough as the tap water trial showed. The faster advance of the soapy water is explained by a lower surface tension (lower adhesive forces between the water molecules) which allows water to move more easy through the soil/air medium.

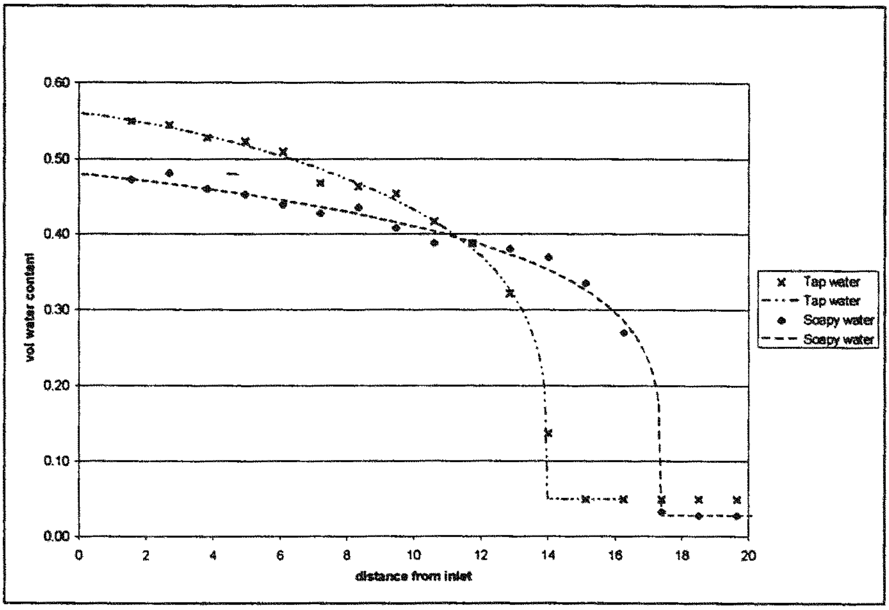


Fig. 3. Results for the Tap Water and Soapy Water Unsaturated Flow

FIELD TESTING RESULTS

The objectives of setting up an evaluation of measuring intake rates was to become familiar with the process of infiltration in the field and the basic concepts of multiphase flow. It was performed to find different infiltration characteristics for the five setups previously selected.

The infiltration was determined in the field using a double ring infiltrometer. This is a widely used method of determining an intake equation. The installation and measurement procedure is well documented in NRCS literature. From the infiltrometer ring experiment we obtained the infiltration rates. Figure 4 shows the results of the infiltrometers by plotting the cumulative intake rate versus time. The results indicate that the warmed water and the surfactant had the highest cumulative intake.

The last plot of the infiltration data (fig. 5), is the intake rate versus time. The results showed that the PAM (1000 mg/l concentration) and the soap (surfactant) had high initial intake rates.

The ring infiltrometer show that water with soap and hot water have a higher rate of infiltration than just the well water. Reservoir water and water with PAM at 1000 mg/l and at 10 mg/l present a very similar plots of depth of infiltration rate versus time.

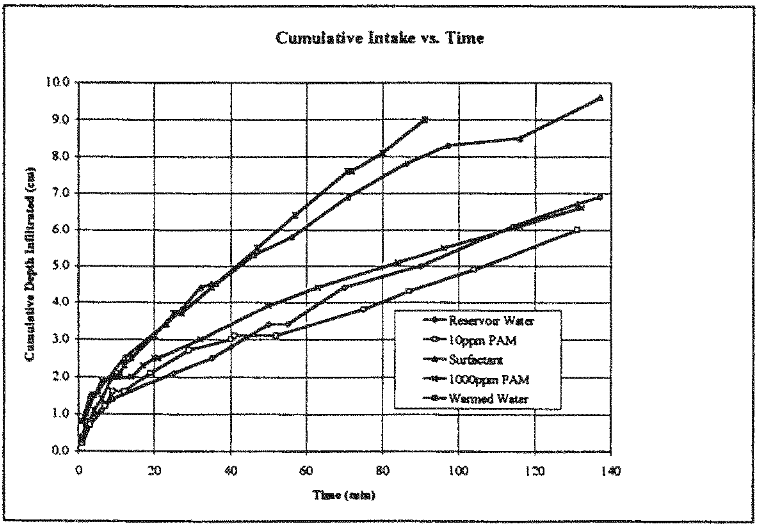


Fig. 4. Plot of the Cumulative Depth of Infiltration Versus Time

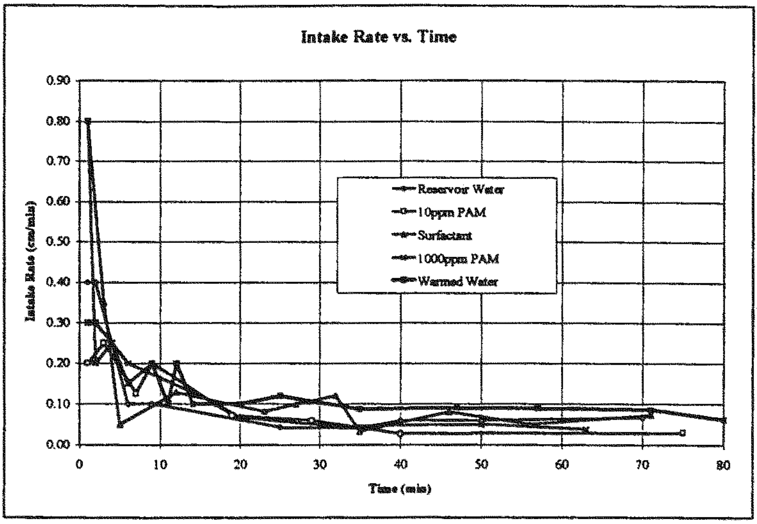


Fig. 5. Plot of the Intake Rate Versus Time

FARMER EXPERIENCES WITH ADDITIVES

Chemical additives for agriculture have been sold to growers with a variety of claims. The use of the dreaded "snake oil" label is readily applied if the product fails to perform as promised. There are several products on the market that seemed to have survived the initial "snake oil" label and farmers are slowly adopting practices that incorporate the chemicals into regular irrigation practices.

Gypsum. For infiltration modification, gypsum additives have been utilize for a number of years. The gypsum provides a rich source of available calcium which is beneficial for soil structure. Gypsum also can be used for water with low salt concentrations. Low salt waters tend to have poor infiltration characteristics due to sealing of the soil surface.

Growers have added gypsum to increase the calcium concentration of the water with commercially available equipment since the late 1980's. There are numerous applications in California where the addition of the gypsum has been shown to be beneficial (Burt 1994).

Surfactants. The use of surfactants have not proven to be as successful in California. The basic idea of how they work is generally misunderstood. Surfactants are only effective during the initial wetting phase of the infiltration. Growers who have used surfactants have seen only limited benefits from the chemical and generally do not endorse the use of surfactants.

Polyacrylamides (PAM). PAM is relatively new to the California market. PAM is being advertised for the settling properties of the material and not entirely the infiltration properties. In general, PAM has two distinct properties:

1) holding soil structure by coating the soil (improves infiltration characteristics)
2) dropping sediments out of suspension

Growers on the west side of the San Joaquin Valley of California have been adopting the use of PAM due to the second property. Tailwater that leaves the field is typically high in sediment load from erosion along the furrow irrigated fields. PAM appears to be quite effective in reducing the sediment load. Typical recommended rates by the manufactures are up to 10 ppm PAM in the irrigation water. Most growers have reduced this value to about 1-2 ppm PAM and only at the beginning of the irrigation event. This is readily done by placing a teaspoon of dry PAM at the head of each furrow at the beginning of the irrigation set.

Table 1 includes the results of a field study completed in 1997 that evaluated the use of PAM. The data support the effectiveness of PAM but also illustrate that water management can be a major part of addressing infiltration and erosion problems.

PAM is not effective in low salt water. In fact, it seems to increase the ability for the water to hold the particles in suspension. Adding gypsum

Table 1. Field Evaluation of PAM

Reference: Irrigation Water Conservation and Sediment Reduction Study. West Stanislaus RCD. 1997.

	Single Event (control)						Single Event (treated)				
Grower	Field Size, ac	Crop	Applied ft	Runoff ft	% Runoff %	TSS	Treatment	Applied ft	Runoff ft	% Runoff %	TSS
D	17	Walnuts	1.10	0.50	45.5	351	dry PAM	1.1	0.33	30.0	274
E	25	Beans	0.16	0.06	37.5	1,212	liquid PAM	0.37	0.1	27.0	27
F	26	Beans	0.34	0.20	58.8	904	dry PAM	0.36	0.17	47.2	393
H	28	Beans	0.29	0.19	65.5	5,610	dry PAM	0.31	0.21	67.7	4,250
I	28	Beans	0.28	0.17	60.7	5,790	dry PAM	0.26	0.05	19.2	1,148
M	50	Asparagus	0.26	0.05	19.2	3,591	dry PAM	0.37	0.08	21.6	2,604
N	25	Spinach	0.26	0.18	69.2	1,124	dry PAM	0.26	0.11	42.3	790
O	34	Broccoli	0.42	0.15	35.7	1,018	dry PAM	0.3	0.08	26.7	732
O'	34	Broccoli	0.42	0.15	35.7	1,018	dry PAM (fish feeder)	0.36	0.09	25.0	62
P	66	Tomatoes	0.52	0.01	1.9	2,250	dry PAM	0.56	0.01	1.8	182
Q	66	Tomatoes	0.64	0.06	9.4	2,140	dry PAM	0.68	0.05	7.4	186
R	34	Broccoli	0.40	0.15	37.5	1,456	dry PAM (fish feeder)	0.38	0.11	28.9	47
Average						**2,205**					**891**

NOTES:

D Small siphon pipes were clogging due to the PAM

E Large reduction in TSS due to liquid PAM.

F Used 1ounce per furrow in small measuring cup. By far the easiest application method.

H Large reduction in TSS due to dry PAM. But the TSS is high. Problem with high furrow inflow rate.

I Same field as H except the furrow flow rate was reduced on the dry PAM furrows.

M Flow rates in the furrow were too great to make the PAM effective. Management needs to reduce the furrow flow rates.

N Flow rates again were the primary problem.

O Applied 1 ounce of dry PAM into head of furrows.

O' Same as O, except dry PAM added to head ditch through fish feeder (1ppm). Very effective.

P Large decrease in the TSS. Very low runoff amount.

Q Large decrease in the TSS. Very low runoff amount.

R Used 2 pounds of Superfloc duri [illegible] first 6 hours of irrigation. Large decrease in TSS.

General Conclusions from Study:

- The most significant problem seems to be associated with the furrow flow rates.
  High flow rates cause erosion and high TSS.
  Using PAM requires higher flow rates because there is more infiltration and the water will not 'get out'.
  High tailwater amounts are associated with high TSS.
  -->>> Need to modify management (reduce tailwater) along with adding PAM to reduce TSS.

- Dry PAM works best if applied in the head ditch with a fish feeder at low rates (1ppm)
- Best way to apply PAM is to inspect tailwater and add an ounce (teaspoon?) to furrows that have high erosion.
- Large siphon tubes (>1 1/4") do not clog with PAM added to head ditch. Small tubes (3/4") have a trendancy to clog.

- There is general confusion on whether the PAM changes infiltration or drops out sediment.
  PAM drops out sediment during the season reducing TSS of tailwater. Not effective with too high flow rates.
  PAM improves infiltration by maintaining structure. Increases infiltration early in the season.
  Keeps furrows from melting later in the season.

dramatically changes the chemistry and improve the capability for the water to drop sediments.

MEASURING INFILTRATION IN THE FIELD

The use of a soil probe to determine the depth of penetration during and after an irrigation is a useful irrigation management tool. There is one probe that is simple to use and make that is increasing in popularity among farmers and field researchers. The tool is a "tile probe" that was historically developed to find tile lines in a field.

Once the soil has reached field capacity, it was found a steel rod with a rounded tip could easily be "pushed" into the soil. This idea was then adapted and promoted by Mr. John Merriam (Professor Emeritus in BioResource and Agricultural Engineering at California Polytechnic State University in San Luis Obispo, California, USA) to be used for irrigation management (Merriam 19xx).

There are several ways to use the probe for irrigation management. It can be used to determine when to shut off an irrigation. It can be used to determine the uniformity of irrigations. An area of increasing use is the use of the probe to evaluate the adequacy of water applied during pre-irrigation.

DISCUSSION AND CONCLUSIONS

Kinematic viscosity and surface tension appear to be two major fluid properties that affect flow through porous media. Infiltration experiments on dry soil created a two-phase flow (air/water), resulting in different infiltration rates for water with and water without surfactants. Saturated column experiments resulted in a one phase flow through porous media. Unsaturated hydraulic conductivity measurements provided a method in the laboratory to study the two-phase flow in a non-structured soil.

The saturated hydraulic conductivity experiment showed that there was no effect of the surface tension under saturated conditions. The surface tension will only be important in a medium where interfaces between two phases occurs (such as between air and water). This was shown clearly in the unsaturated flow experiment.

The infiltration experiments showed a fast infiltration rate for warm water and soapy water. Water with PAM and water at room temperature showed lower infiltration rates. The high infiltration rate for warm water is a result of a lower kinematic viscosity, resulting in a higher hydraulic conductivity. The higher infiltration rate for water with soap cannot be explained with the Pouseuille equation, nor has it been described in equations in the reviewed literature. However, it is believed that the higher rate is a result of a lower surface tension. Not only do the infiltration rates support this idea, but a visual experiment of putting two drops of water with and without soap on a dry soil and a plastic surface showed a difference in behavior between the two fluids. The drop with soap infiltrated faster in the soil and spread out more, while the water without soap formed a curved shape that remained on top of the soil for a longer time.

Possible explanations for this could be that a reduced surface tension allows for a flatter film of water on the soil particles that is interconnected, instead of a situation as in Fig. 2.3 in Corey (1994). When the water is interconnected, it will create a path of less resistance for water to travel through, thus increasing the unsaturated hydraulic conductivity.

Another possible explanation is that the reduced surface tension will allow smaller soil pores to be filled, resulting in an unsaturated hydraulic conductivity closer to the saturated hydraulic conductivity. The column study did not result in a significant difference of the flux rate, which suggests that the surface tension only makes a difference in a two phase flow, when there is a surface interface.

During the infiltration experiment, a high initial water intake was observed during the first few minutes for the high concentration PAM and the water with soap setup. The high initial intake of the PAM can be explained by the immediate stabilization of the soil aggregates, resulting in a large initial intake in the macro pores of the soil. However, after the macro pores are filled, the infiltration rate is slower than that of regular water.

The large initial intake of water with soap supports the explanation above. During infiltration, a larger hydraulic conductivity occurs due to a more continuous path of water in the pores. When the infiltration reaches a steady state (saturated flow), the lower surface tension does not make a difference in the flux rate any more and the infiltration rate becomes similar to the one of regular water.

Overall, the following was concluded from this study:

- Temperature has a high effect on the viscosity of the water resulting in higher intake rates. However, in laboratory and field measurements, the temperature is often not measured.

- Surfactants affecting the surface tension of a fluid will increase the initial intake of the water. Its affect will decrease with increasing volumetric water content due to less air/water interfaces in the soil.

- PAM does not affect any of the water properties we evaluated. It is very effective for erosion control and might be effective in increasing infiltration characteristics of a highly structured soil.

- Any attempt to increase infiltration should be evaluated based on the irrigation efficiency and distribution uniformity effects. Increasing the infiltration rates can be detrimental in some cases causing decreases in the irrigation efficiency and distribution uniformity.

REFERENCES

ASME, 1965
Determination of the Viscosity of Liquids, Instruments and Apparatus. ASME part 17.

Burt, CM., and SW Styles, 1994
Drip and Microirrigation for Trees, Vines, and Row Crops (with special sections on buried drip). Irrigation Training and Research Center. Cal Poly. San Luis Obispo-CA. p44

Corey, AT.,1994
Mechanics of Immiscible fluids in porous media, Water Resources publications, Forth Collins, Colorado, USA

Darcy, 19xx

Duke HR., 1992
Water Temperature Fluctuations and Effect on Irrigation Infiltration. *Transactions of the ASAE,* Jan-Feb, v35 n1:193-199.
Grismer, ME., 1986
Pore-Size distribution and infiltration. In: *Soil Science,* April 1986, Vol 141, no 4

Grismer, ME, MN Orang, V Clausnitzer and K Kinney 1994.
Effects of air compression and counterflow on infiltration into soils. In: *Journal of Irrigation and Drainage Engineering,* Vol 120, no 4, July-Aug 1994.

Jaynes DB., 1990
Temperature Variations Effect on Field-Measured Infiltration. *Soil Science Society of America Journal,* Mar-Apr, v54 n2:305-312.

Merriam J.L., 1997

Poiseuille,

IMPROVED SOFTWARE FOR DESIGN OF LONG-THROATED FLUMES

Tony L. Wahl[1] Albert J. Clemmens[2]

ABSTRACT

Long-throated flumes have received increased emphasis in recent years as a practical, low-cost, flexible means of measuring open-channel flows in new and existing canal systems (Reclamation, 1997). MS-DOS based software to aid in the design of long-throated flumes has been available for several years from the International Institute for Land Reclamation and Improvement, and was documented by Clemmens et al. (1993). To facilitate the future use of long-throated flume technology, the Bureau of Reclamation and the Agricultural Research Service are presently rewriting this software to operate in a Windows-based computing environment. The updated software will include significant user-interface improvements and new features, and will overcome an incompatibility between Windows 95 and the copy-protection system used on the current software. Some of the new features will include an improved design optimization scheme, integrated printing of full-scale wall gauges, improved graphics and units system support, a simplified system for saving, retrieving, and sharing flume designs with other users, and an online help system. At this time most of the new features are in place and the software is in a beta-testing phase. The completed software will be released at the end of the beta-test program along with updated printed documentation.

INTRODUCTION

The term *long-throated flume* describes a broad class of critical-flow flumes and broad-crested weir devices used to measure flow in open channels. These devices are adaptable to a variety of measurement applications in both natural and man-made channels, and both new and existing canal systems. Bos et al., (1984) describe the theory for determining discharge through these flumes, which has been well-developed over the course of the late 19th and 20th centuries. Procedures for determining head loss and developing flume designs for specific applications have been developed in the last two decades, and with the advent of relatively low-cost personal computers, these have become accessible through interactive computer programs (Clemmens et al., 1993).

---

[1] Hydraulic Engineer, U.S. Bureau of Reclamation, Water Resources Research Laboratory, PO Box 25007 (D-8560), Denver, CO 80225-0007. (twahl@do.usbr.gov)

[2] Research Hydraulic Engineer, U.S. Water Conservation Laboratory, USDA-ARS, 4331 East Broadway Rd., Phoenix, AZ 85040. (bclemmens@uswcl.ars.ag.gov)

Figure 1 shows the components of a typical long-throated flume. Primary features are an approach channel and water level measurement device, a converging transition, the control (throat) section in which critical flow occurs, and an optional diverging transition to the tailwater channel.

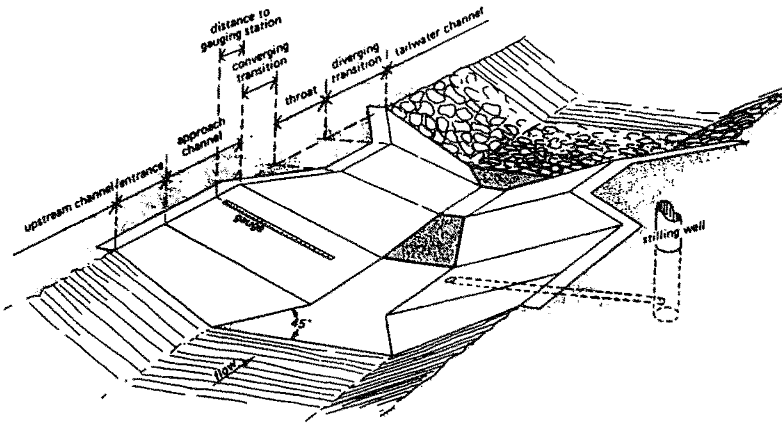


Figure 1. – Components of a Long-Throated Flume (Bos et al., 1984).

In recent years long-throated flumes have become the measurement device of choice for many applications (Reclamation, 1997), superseding Parshall flumes and other well-known standard measurement devices. To briefly review, significant advantages of long-throated flumes include:

- Assuming critical flow occurs in the throat, a rating table specifying the flow rate as a function of the upstream head can be determined with an error of less than 2% in the computed discharge. Rating tables can be computed for any combination of a prismatic control section and an arbitrarily shaped approach channel.
- The throat must be horizontal in the direction parallel to the flow, but can be any shape in the direction perpendicular to the flow, allowing the complete range of discharges to be measured with good precision.
- The required head loss across the flume is minimal to ensure a unique relationship between the upstream sill-referenced head and the discharge.
- Long-throated flumes can be operated with partial submergence (i.e., downstream water level above the sill elevation), and the submergence limit

and associated head loss requirement can be estimated for any structure placed in an arbitrary channel.

- With a properly constructed gradual converging transition, the flumes have virtually no problem passing floating debris.
- Long-throated flumes can be designed to pass sediment transported by channels having subcritical flow. Sedimentation only becomes a problem when sediment loads become excessive, or if the flume causes a significant reduction in flow velocity in the approach channel.
- If the throat is horizontal in the direction parallel to the flow, an accurate rating table can be computed using as-built dimensions. The throat section may also be modified as necessary to accommodate changing site conditions, and a new rating can be computed using the modified dimensions.
- Long-throated flumes are typically the most economical weirs or flumes for measuring open-channel flows, especially in larger sizes. For example, the construction of a large Parshall flume requires considerable care and construction expense to properly construct the sloped floor of the throat and diverging sections and maintain design dimensions within required tolerances. In contrast, long-throated flumes require only an arbitrary gradual transition into the throat section, and can be calibrated using as-built dimensions.
- Because long-throated flumes can be designed for installation into any arbitrary channel, they are very adaptable to installation in existing canals.

The design calculations needed to properly size and locate flumes are iterative; as a result, several generations of computer codes to assist in the design of long-throated flumes have been developed in recent years, primarily by the Agricultural Research Service (ARS) Water Conservation Laboratory in Phoenix, Arizona, USA. Most of these programs operated in a batch mode and were written in FORTRAN. In the early 1990's the International Institute for Land Reclamation and Improvement (ILRI) and ARS contracted the development of an interactive computer program for long-throated flume design. This program was written in the Clipper language and operated in an MS-DOS computing environment. The program included: on-screen graphic displays of flume geometry; a design optimization routine; features to assist in calibration of existing flumes; and printed output of rating tables, rating equations, wall gauge data and discharge-head curves. The program is distributed with the 1993 publication by Clemmens et al., titled FLUME: Design and Calibration of Long-Throated Measuring Flumes. Through the remainder of this paper the Clipper-based program will be referred to as FLUME 3.0.

FLUME 3.0 suffers from several shortcomings that have become more critical with the release and widespread adoption of the Windows 95 operating system. These include:

- A copy-protection system that is incompatible with Windows 95
- The need for a large amount of free DOS memory, which is most critical on DOS and Windows 3.1 systems connected to networks
- A text-based menu system and user interface that does not use a mouse, and in which keystroke sequences required to navigate the menus are tedious and non-intuitive
- Bugs in discharge curve graphics and flume geometry displays
- Lack of support for modern printers and poor management of printed output to modern page-type printers
- Problems in design optimization routines for some complex channel and throat section shapes
- A method of storing flume designs that makes it tedious to transfer individual flume designs among users

Several of these problems could be corrected by making relatively minor modifications to the Clipper source code, but others would require major revisions or simply are not possible within the framework of the existing program. Clipper is also no longer a widely used programming language, so making revisions to the Clipper-based program is not a good long-term strategy for maintaining and improving the software.

For these reasons, the Bureau of Reclamation (Reclamation) and ARS began in mid-1997 to work cooperatively on the development of an updated version of the flume design software. The new program is titled ***WinFlume*** and is being programmed for the Windows 95 and Windows NT environment using the Visual Basic language. The program will also be compatible with Windows 3.x systems, although this is not the target platform for the software. The new program will make use of the same hydraulic theory used in the previous FORTRAN- and Clipper-based programs, but will have an improved user interface, a new design optimization/analysis routine, and several additional features not contained in any of the previous programs. Programming work is being performed in-house by the Bureau of Reclamation to facilitate future maintenance and improvement of the software.

THE WINFLUME APPLICATION

The WinFlume program is being developed as a native Windows 95 and Windows NT application, using the Visual Basic programming language. The application will be compiled into a 32-bit executable for Windows 95/NT systems and a 16-bit executable for Windows 3.x systems. Graphical output from the program (e.g., head-discharge curves, plots of rating equations, comparisons to measured flows) is provided using an integrated third-party graphics library that is distributed with the application.

Operation of the program is centered around an editable graphic display of the flume dimensions (Fig. 2), auxiliary data screens used to specify non-geometric properties of the flume and approach and tailwater channels, and several screens devoted to providing output, analysis, and review of flume designs. Flume bottom-profile dimensions are edited from the screen shown in Fig. 2, while cross-section dimensions are edited in a separate screen that shows additional cross-section detail (Fig. 3). Internally, the program uses an object-oriented programming approach in which the complete definition of the flume and associated channel and hydraulic properties are contained within a single structured data type. This simplifies storage and retrieval of flume designs and internal manipulation of multiple "virtual" flumes in the design optimization and analysis routines.

The WinFlume program shares essentially all of the basic features of the FLUME 3.0 program described earlier. These include:

- Ability to design flumes for specific head loss objectives, while verifying that the design meets criteria for maximum approach flow Froude number, adequate freeboard, maximum allowable tailwater, and desired discharge measurement precision

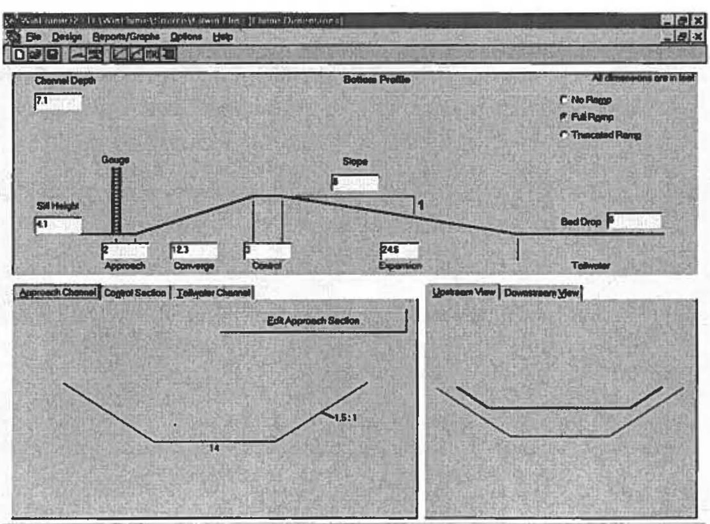


Figure 2. —WinFlume Display of Flume and Canal Geometry.

- Support for both fixed- and movable-crest flumes
- Support for seven possible approach and tailwater channel cross-section shapes, and 14 possible control-section shapes including circular, parabolic, and trapezoidal sections, and complex sections containing trapezoids within simpler sections
- Ability to produce rating tables of discharge versus upstream head for direct field use and for use in constructing customized staff gauges graduated in head or discharge units for field installation
- A curve-fitting module to develop power-law rating equations for use in automated data collection systems

New Features

The WinFlume program contains a number of new and improved features. The most dramatic improvements are in the user interface, which takes full advantage of the Windows environment and includes such features as a standard Windows-style top menu bar, toolbars with tool tip help tags, tabbed dialog boxes to organize program features, grids to display rating table data, standard Windows common dialogs for loading and saving files, and support for exporting data and graphics to the system clipboard. All program output can be directed to a file, to the system clipboard, or to any printing device installed in the user's Windows system.

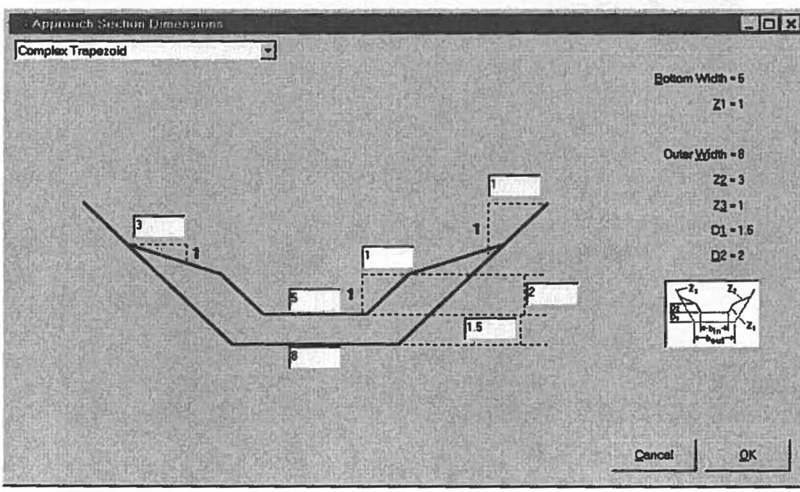
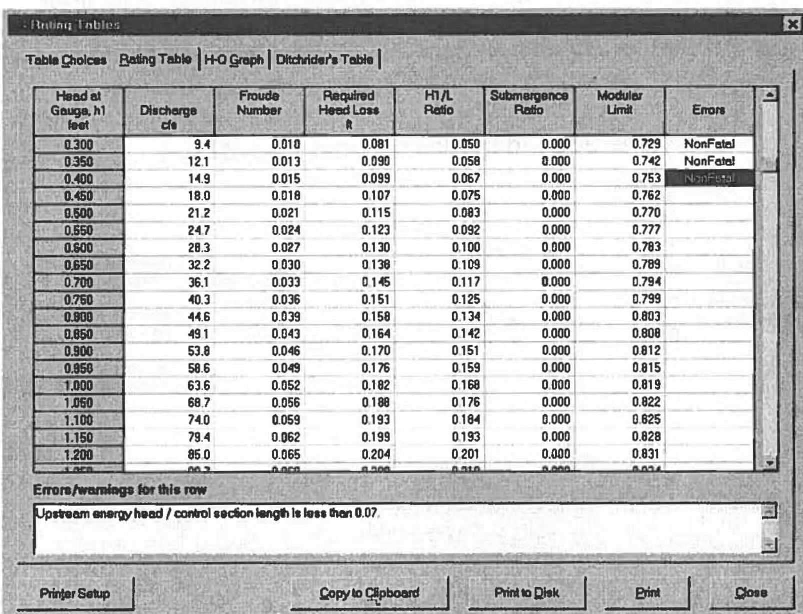


Figure 3. — Editing a Complex Trapezoid Cross-Section.

A key component of the improved interface will be an integrated online help system that will provide the user with both flume-design guidance and assistance with the details of program usage. The online help system will be constructed in combination with the preparation of printed documentation for the new software.

<u>New File Format</u>: WinFlume now saves each flume design into an individual data file with the default extension of .FLM. This is in contrast to FLUME 3.0, which stored all flume designs collectively in a single dBase file. The use of individual data files makes transfer of flume designs among different users more convenient. For backward compatibility, WinFlume is able to load designs from the FLUME 3.0 dBase file format and then save them to new individual files in the WinFlume format. WinFlume and FLUME 3.0 both work internally and save all data in metric units, but WinFlume saves the user's units-system preferences for program input and output with each flume design, rather than as a program-wide preference as was done by FLUME 3.0. This simplifies working on multiple flumes in a variety of units systems. Data are also stored with better precision than was possible in the dBase file format, reducing problems with round-off errors that occasionally occurred in FLUME 3.0.



Head at Gauge, h1 feet	Discharge cfs	Froude Number	Required Head Loss ft	H1/L Ratio	Submergence Ratio	Modular Limit	Errors
0.300	9.4	0.010	0.081	0.050	0.000	0.729	NonFatal
0.350	12.1	0.013	0.090	0.058	0.000	0.742	NonFatal
0.400	14.9	0.015	0.099	0.067	0.000	0.753	NonFatal
0.450	18.0	0.018	0.107	0.075	0.000	0.762	
0.500	21.2	0.021	0.115	0.083	0.000	0.770	
0.550	24.7	0.024	0.123	0.092	0.000	0.777	
0.600	28.3	0.027	0.130	0.100	0.000	0.783	
0.650	32.2	0.030	0.138	0.109	0.000	0.789	
0.700	36.1	0.033	0.145	0.117	0.000	0.794	
0.750	40.3	0.036	0.151	0.125	0.000	0.799	
0.800	44.6	0.039	0.158	0.134	0.000	0.803	
0.850	49.1	0.043	0.164	0.142	0.000	0.808	
0.900	53.8	0.046	0.170	0.151	0.000	0.812	
0.950	58.6	0.049	0.176	0.159	0.000	0.815	
1.000	63.6	0.052	0.182	0.168	0.000	0.819	
1.050	68.7	0.056	0.188	0.176	0.000	0.822	
1.100	74.0	0.059	0.193	0.184	0.000	0.825	
1.150	79.4	0.062	0.199	0.193	0.000	0.828	
1.200	85.0	0.065	0.204	0.201	0.000	0.831	

Figure 4. — Flume Rating Table Displayed in a Scrollable Grid.

Rating Table Improvements: Several improvements have been made to the flume rating tables (Fig. 4). Tables can still be generated for either fixed-head or fixed-discharge increments, and a new Smart Range feature has been added that assists the user to find a reasonable range over which to generate the rating table. Tables can now include the estimated actual tailwater levels and submergence ratios for each flow rate shown in the table, whereas FLUME 3.0 only reported actual tailwater levels at the minimum and maximum design discharge.

The interactive display of rating table data has been improved in WinFlume by using a scrollable grid control. This presents the rating table data in a spreadsheet format that can be browsed by the user (Fig. 4). For each line of the rating table (corresponding to a single flow), any errors or warnings associated with the hydraulic calculations for that line are displayed in a text box below the table. This is in contrast to FLUME 3.0, which displayed only a summary of all errors or warnings for the entire table. The ability to associate individual error messages with specific lines in the rating table helps the user understand the cause of the error; they can then determine the modifications to the design or design criteria that may be needed to correct the problem, or whether the error can be ignored.

Rating table data can also be plotted, and the graphs then printed, copied to the clipboard, or saved to a Windows Metafile format. Graphs can display up to 3 parameters at a time, chosen from the 14 possible parameters that can be included in a rating table. Figure 5 shows a typical graph produced by WinFlume.

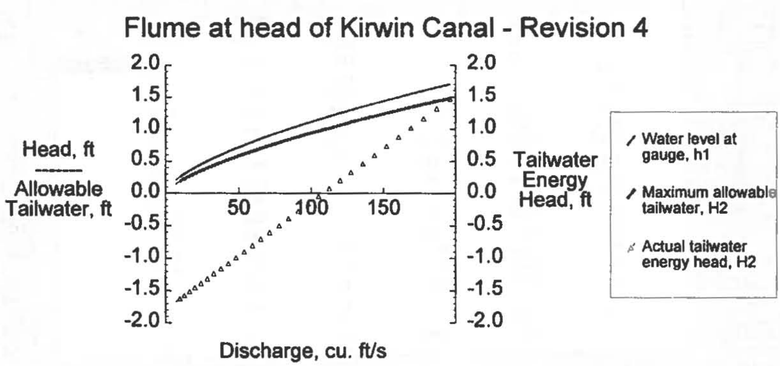


Figure 5. – Rating Table Graph Produced by WinFlume. Heads Are Shown Relative to the Flume Crest.

Ditchrider's Rating Table: A new type of rating table is included in WinFlume, specifically designed for field use. This rating table shows only the discharge as a function of upstream head, and is organized to fit a large amount of data onto one or two sheets of paper. Each row of the rating table displays the discharge for 10 increments of head, as shown in Table 1. This table can display the flow as a function of either vertical head or the slope distance for users who wish to install standard staff gauges on the sloped bank of the upstream channel.

Table 1. – Typical Ditchrider's Rating Table.

User: Tony L. Wahl WinFlume32 - Version 0.32
D:\WinFlume\Source\Kirwin.Flm - Revision 2
Flume at head of Kirwin Canal
Ditchrider's Rating Table, Printed: 4/1/98

Head, h1 ft	Discharges in cu. ft/s 0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.2	--	--	--	--	--	9.7	10.3	10.9	11.6	12.2
0.3	12.9	13.5	14.2	14.9	15.6	16.4	17.1	17.8	18.6	19.4
0.4	20.1	20.9	21.7	22.5	23.4	24.2	25.0	25.9	26.7	27.6
0.5	28.5	29.4	30.3	31.2	32.1	33.0	34.0	34.9	35.9	36.9
0.6	37.8	38.8	39.8	40.8	41.8	42.9	43.9	44.9	46.0	47.0
0.7	48.1	49.2	50.3	51.4	52.5	53.6	--	--	--	--

Rating Equation Curve-Fitting: The curve-fitting module in WinFlume is substantially the same as that used in FLUME 3.0. The user specifies a range and increment of heads for which rating table data are generated, and the program then uses a non-linear curve-fitting algorithm to determine a single equation fitting the data of the form:

$$Q = K_1\left(h_1 + K_2\right)^U$$

in which Q is the discharge, h_1 is the upstream sill-referenced head, and K_1, K_2, and U are all fitted parameters. The coefficient of determination is also reported to the user. A new feature of WinFlume is the option to force the value of K_2 to zero. This option was included to accommodate dataloggers that only support an equation of the form:

$$Q = K_1 h_1^U$$

Plotting Full-Scale Wall Gauges: A major new feature of WinFlume is the integrated printout of full-size flume wall gauges. Both FLUME 3.0 and WinFlume generate rating table data needed to construct a full-size wall gauge graduated in either head or discharge units, that can be installed vertically in the upstream pool or on the sloped bank of the approach channel. WinFlume also provides an on-screen preview (Fig. 6) of the wall gauge and can then print it at full-scale to a Windows printer driver. Controls are provided that permit adjustment of label sizes, labeling intervals, and number format. With a roll-feed plotter, large wall gauges can be printed on a continuous sheet of paper. With smaller page printers, WinFlume will split the gauge into several sections and print each on a separate sheet of paper with the match marks needed to reconstruct the continuous gauge. These full-size paper gauges can then be taken to a fabricator for construction of a durable baked enamel gauge.

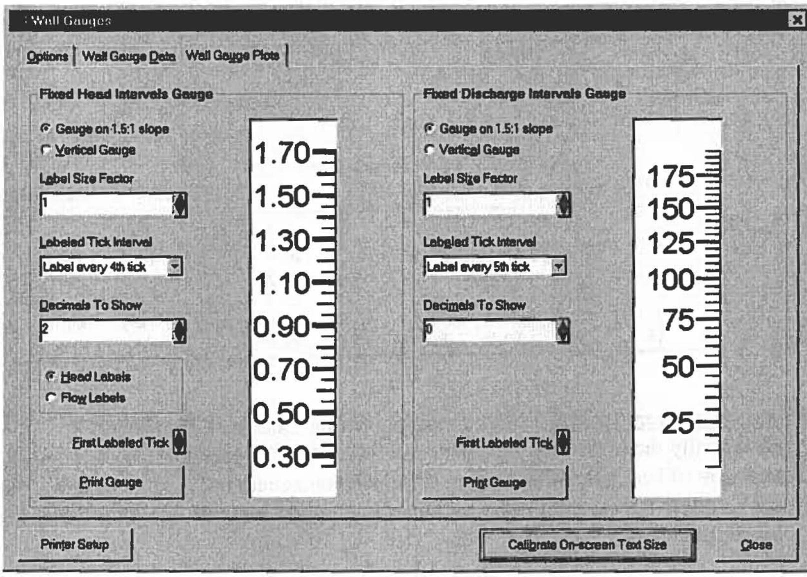


Figure 6. – On-Screen Preview of Flume Wall Gauges.

New Design Routines: The design routines in WinFlume are still under development at this time, but it is expected that WinFlume will use a different approach to flume design than that used by FLUME 3.0. This approach is made possible by the tremendous advances in computing speed since the development of FLUME 3.0.

FLUME 3.0 used an iterative scheme to adjust the throat section geometry until a suitable design was found that satisfied the user-specified criteria for discharge range and measurement precision, while still meeting necessary hydraulic criteria for acceptable flume performance. Six design criteria were evaluated in the design optimization routine:

- User-specified freeboard required at maximum flow
- Minimum head necessary to satisfy user-specified flow measurement precision requirement at maximum flow, based on precision of upstream water level measurement device

- Minimum head needed to satisfy measurement precision requirement at minimum flow
- Maximum allowable tailwater level at maximum flow
- Maximum allowable tailwater level at minimum flow
- Froude number at maximum discharge < 0.5

To begin the computations the user specified a starting throat section shape and sill height, a method for adjusting the contraction, and a design objective. Four methods of contraction change and four possible design objectives were available, shown in Table 2.

Table 2. – User-Specified Design Methods and Objectives

Methods of Contraction Change	Design Objectives
• Raise sill height • Raise entire throat section • Raise inner section of throat (used with complex cross-sections) • Adjust lateral contraction by changing bottom width of throat section or changing diameter or focal distance of circular or parabolic sections	• Minimum head loss • Maximum head loss • Intermediate head loss • Match head loss to drop in canal invert at site

The FLUME 3.0 design algorithm attempted to minimize the number of computations by determining at a given iteration both the direction and amount of contraction change needed to resolve any unsatisfied design criteria and meet the design objective. For straightforward cross-section shapes the technique was effective, but for complex throat sections having compound shapes or circular components there were some situations in which it was difficult to determine the proper direction of contraction change needed to satisfy the design criteria. In these cases FLUME 3.0 would sometimes fail to find an acceptable design, even though one did exist.

To address these problems, WinFlume will use a more direct, brute-force method for finding an acceptable design. The user will still specify one of the four methods of contraction change listed above, and the same 6 design criteria will be evaluated. The design algorithm will begin by bracketing the range of possible designs using a subroutine that can determine the contraction required to produce a given upstream water level at maximum discharge. The maximum possible throat-section contraction will be that needed to produce a maximum upstream water level equal to the channel depth. The minimum allowable contraction will be that which produces either an upstream Froude number of 0.5 at maximum discharge, or an upstream water level that is at least as high as the downstream tailwater at maximum discharge. Once the range of contractions is determined, WinFlume will evaluate possible designs between the lower and upper contraction limits at an interval specified by the user. This interval might be chosen based on a minimum convenient dimensional increment for construction. The results of the design evaluations will be presented to the user, who may choose to accept one of the designs or discard the results of the analysis. This method will not directly produce a single design meeting a specific head loss objective. Instead, designs that have minimum head loss, maximum head loss, intermediate head loss, and head loss matching the bed drop at the site will all be listed in the output, along with the other acceptable designs. The user will be able to evaluate the different head loss characteristics of the possible designs and then choose the design that best meets their needs.

It is possible that if the contraction increment specified by the user is too large, or if design criteria are too limiting, no acceptable design will be found. If this occurs WinFlume will try to determine if an acceptable design might be found by searching with a smaller increment of contraction change. WinFlume searches for two adjacent designs for which all unsatisfied criteria in each of the designs are satisfied in the adjacent design. If such a situation can be found, then an acceptable design may exist between those two designs, and the analysis can be repeated using a smaller increment of contraction change within that range. If no region of possible acceptable designs is found, then the results of the analysis would be presented to the user, perhaps with suggestions of ways to relax the design criteria or change the initial design so that an acceptable design can be found.

Online Help: The WinFlume program will feature an integrated online help system that will provide the user with both flume-design guidance and assistance with the details of program usage. The online help system will be constructed in combination with the preparation of printed documentation for the new software.

In addition, the release version of WinFlume will include a step-by-step flume wizard that leads the user through the process of defining all critical properties of a flume design. This will help ensure that the user does not omit crucial details, such as specification of tailwater levels and discharge ranges.

Possible Additional Features

Several additional features are being considered for future incorporation into the WinFlume program. These include:

- Support for determining proper setting of commercially available pre-fabricated flumes. These flumes can be analyzed with the program as it currently exists, but the user must manually enter the dimensions of each flume. Working with these flumes could be simplified by including a database of dimensions for available flumes.

- A more sophisticated backwater analysis module for determining tailwater levels. Presently the user can specify tailwater levels at minimum and maximum flow using either field data or the results of an independent backwater analysis, or by using the Manning equation and assuming flow is at normal depth in the downstream channel.

HOW TO OBTAIN WINFLUME

At this time the WinFlume program is in a beta-test state. The majority of the new features are in place. A demonstration version of the program is available on the World Wide Web at http://ogee.do.usbr.gov/twahl/. Because the program has not yet been thoroughly tested and verified, this demonstration version does not support printed output. Users are still urged to use FLUME 3.0 for production design work. If you are interested in beta-testing a fully functional version of the program, you may do so by contacting the authors of this paper.

Once the beta-test program is complete, the software will be made available to the public, in combination with updated printed documentation.

ACKNOWLEDGMENTS

This work is funded by the U.S. Bureau of Reclamation's Water Conservation Field Services Program.

REFERENCES

Bos, M.G., J.A. Replogle, and A.J. Clemmens, 1984, Flow Measuring Flumes for Open Channel Systems, John Wiley & Sons, New York, NY.

Clemmens, A.J., M.G. Bos, and J.A. Replogle., 1993, FLUME: Design and Calibration of Long-Throated Measuring Flumes, ILRI Publication 54, International Institute for Land Reclamation and Improvement, The Netherlands.

U.S. Bureau of Reclamation, 1997, Water Measurement Manual, 3rd ed., U.S. Government Printing Office, Washington, DC 20402.

THE GREAT CALIFORNIA FLOOD OF 1997 AND ITS AFTERMATH

By

Maurice Roos[a]
Chief Hydrologist
California Department of Water Resources
Post Office Box 219000
Sacramento, California 95821-9000
(916) 574-2625; Fax (916) 574-2767
E-Mail mroos@water.ca.gov

ABSTRACT

The New Years floods of 1997 in northern California were notable in their intensity, volume of flood water, and areal extent from the Oregon border to the Southern Sierra. Many new records were set. It was a classic orographic event with warm moist winds from the southwest blowing over the Sierra Nevada and dumping amazing amounts of rain. Antecedent conditions were conducive to high runoff. Overall the major flood control systems performed well, greatly reducing peak flood flows on the Sacramento and San Joaquin River systems. But there were two serious levee breaks in the Sacramento Valley and the sheer volume of flood waters overwhelmed dikes along the lower San Joaquin River.

In the wake of the flood, revised estimates of the risk of flooding are being assembled and some fundamental questions are being raised as how to best provide reliable flood protection for an increasingly urbanized region. The Governor's Flood Emergency Action Team was appointed soon after the flood and published a final report in May of 1997 which had over 50 recommendations which are now the basis of the State's flood policy. Some of the options, such as levee setbacks, would affect agricultural lands. A joint federal-state four year comprehensive study of the Central Valley flood management system is now underway.

---

[a] Chief Hydrologist, California Department of Water Resources, P.O. Box 219000, Sacramento, California 95821-9000. Presented at U.S. Committee on Irrigation and Drainage Conference, June 1998, in Phoenix, AZ.

INTRODUCTION

First, some background information on floods. In looking at a relief map of California, three ingredients of flooding are apparent. There is an adjacent ocean which is a source of moisture. There are large flat areas of low ground, especially in the great Central Valley. Then there are high mountain ranges which can generate large volumes of flood water to spill down on the lowlands.

The flood threat for northern and central California is seasonal. On average, half of our precipitation comes in three months (December, January, and February) and three-fourths during the 5-month period from November through March. See Figure 1. This does have one advantage in that rain flood space is only needed during the rainy season, allowing use of reservoir flood space for water storage in the spring. The sample on Figure 2 is the Corps of Engineers flood control diagram for Folsom Reservoir. Wetness parameters adjust the space requirement for antecedent precipitation on the watershed.

THE FLOOD

The New Year's flood of 1997 was probably the largest in the 90-year Northern California record, which begins in 1906. It was notable in the intensity, volume of flood water, and the areal extent from the Oregon border down to the southern end of the Sierra. Many new flood records were set.

This was a classic orographic event with warm moist winds from the southwest blowing over the Sierra Nevada and dumping amazing amounts of rain at the middle and high elevations, especially over a 3-day period centered on New Year's Day. The sheer volume of runoff exceeded the flood control capacity of Don Pedro Dam on the Tuolumne River near Modesto and Millerton Reservoir on the upper San Joaquin River near Fresno with large spills of excess water. Most of the other large dams in northern California were full or nearly full at the end of the storms.

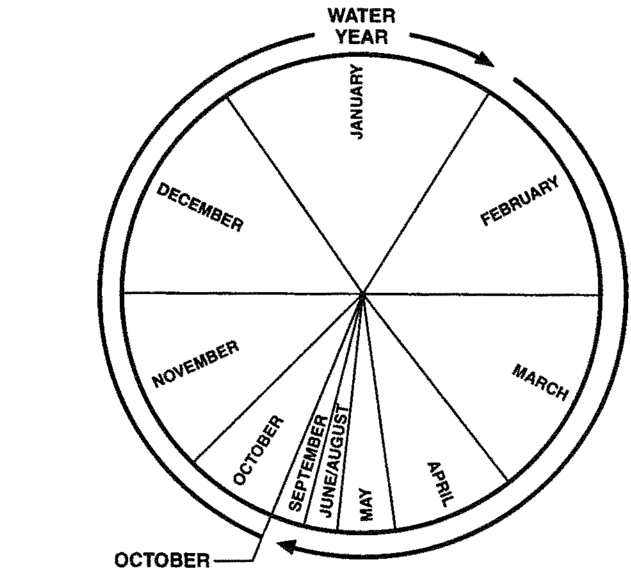


Figure 1 - MonthlyPrecipitation Distribution In The Sierra Nevada

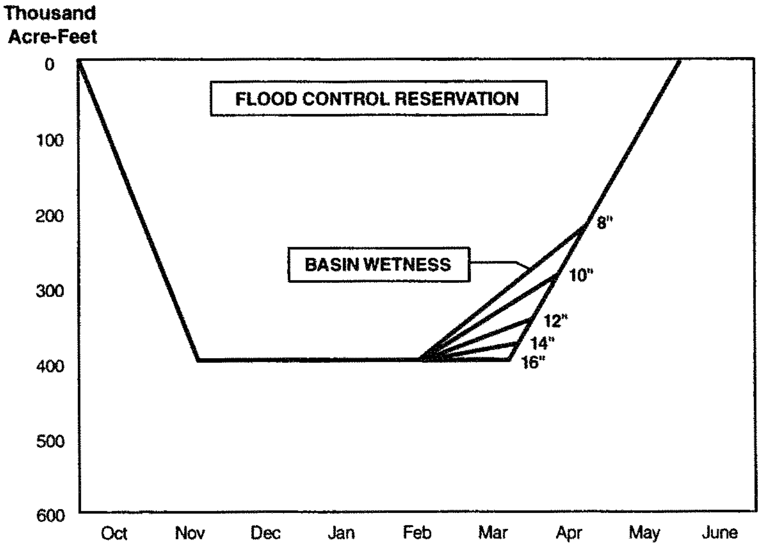


Figure 2 - Folsom Flood Control Diagram

Amounts of rain at lower elevations were not unusual. For example, downtown Sacramento in the middle of the Central Valley had 3.7 inches during the week from December 26 through January 2. But Blue Canyon, at the one-mile elevation between Sacramento and Reno, had over 30 inches, an orographic ratio of over 8, far more than the usual 3 to 4 for most storms. The entire northern Sierra was observing 20 inches, some 40 percent of average annual precipitation.

Antecedent conditions favored high runoff. Storms during the first half of December had saturated the ground. The big storm was preceded by a cold snowstorm which produced heavy snows to low elevations a few days before Christmas. The lower elevation snow melted during the New Year's storm (for example, at the mile high Blue Canyon station where over 30 inches of rain melted the 5 inches of water content in the snowpack there) but the middle and high elevation snowpack remained with the rain percolating through the pack. Not much loss was observed on the snow sensors over 6,000 feet in elevation in the northern Sierra, in spite of rain levels up to 9,000 feet at times. Figure 3 shows changes in snow water content and accumulated precipitation at the Alpha sensor near Echo summit.

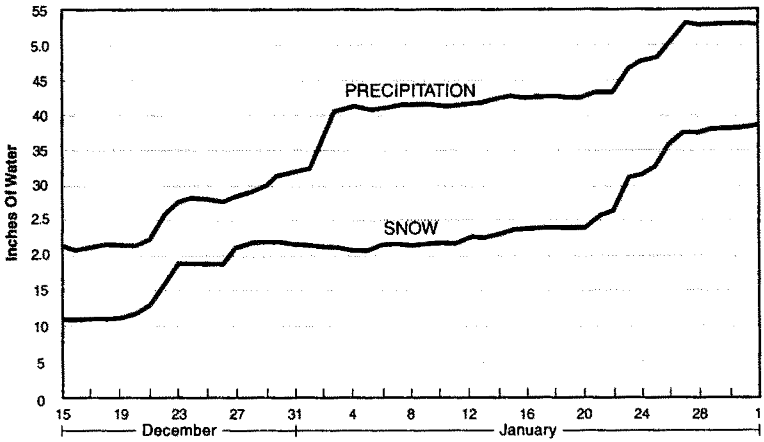


Figure 3 - Accumulated Precipitation and Snow Water Content, Alpha Snow Sensor, at 7600 Feet in the American River Basin

This contrasts with the popular impression that the melting snow caused the floods. Snowmelt, mostly from lower elevations added to the runoff, perhaps 15 percent. But the bulk of runoff was simply too much rain.

The total amount of precipitation at Blue Canyon for the two month December through January period was a record 75 inches, about 43 inches during December and 32 inches in January. The annual total at this station averages about 63 inches. The December amount was second wettest, after that of 1955, also a flood year.

Floods were produced on the Coast Range as well during the New Year's storms, but not to record levels. The Russian, Napa, and Pajaro Rivers did not rise as high as the severe floods of 1995. Further north, the Eel, Klamath and Smith Rivers rose higher than 1995, but did not set records. (See Figure 4 map.)

THE RIVERS

The overall Sacramento River region flood control system performed well, greatly reducing the peak flows on the Sacramento River system. Even so, total flow at the latitude of Sacramento in the river and in Yolo bypass approached 600,000 cubic feet per second which is nearly half that of the combined Missouri and Mississippi River flow of 1.3 million cfs at St. Louis in the great flood of 1993. There were two serious levee breaks in the Sacramento Valley, one on the Feather River south of Marysville, the other on Sutter Bypass west of Yuba City. (See Figure 5, a map of the Flood Control Project). Major flooding occurred along the uncontrolled Cosumnes River southeast of Sacramento and on the Tuolumne River near Modesto and the San Joaquin River near Fresno. The volume of flood waters overwhelmed dikes along the lower San Joaquin River from near the mouth of the Tuolumne River to the Delta near Manteca with massive flooding. This was one for the record books. See second map, Figure 6, of the San Joaquin River Flood Control Project.

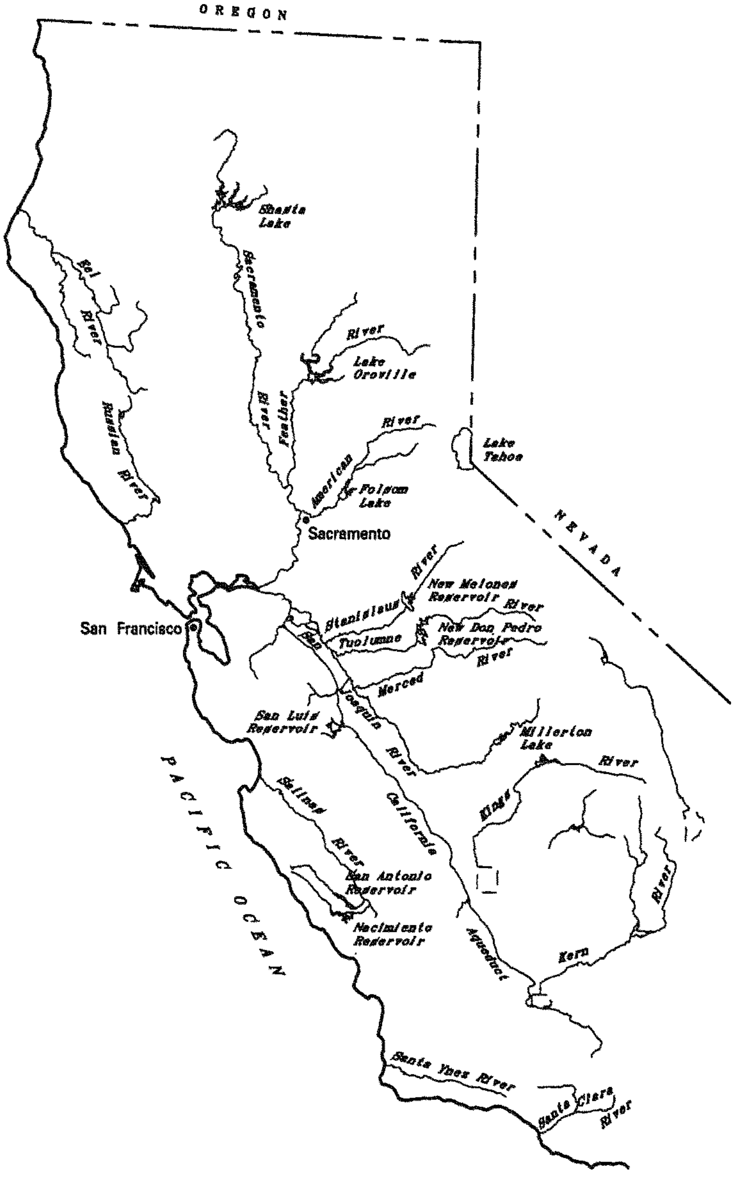


Figure 4. Location Map

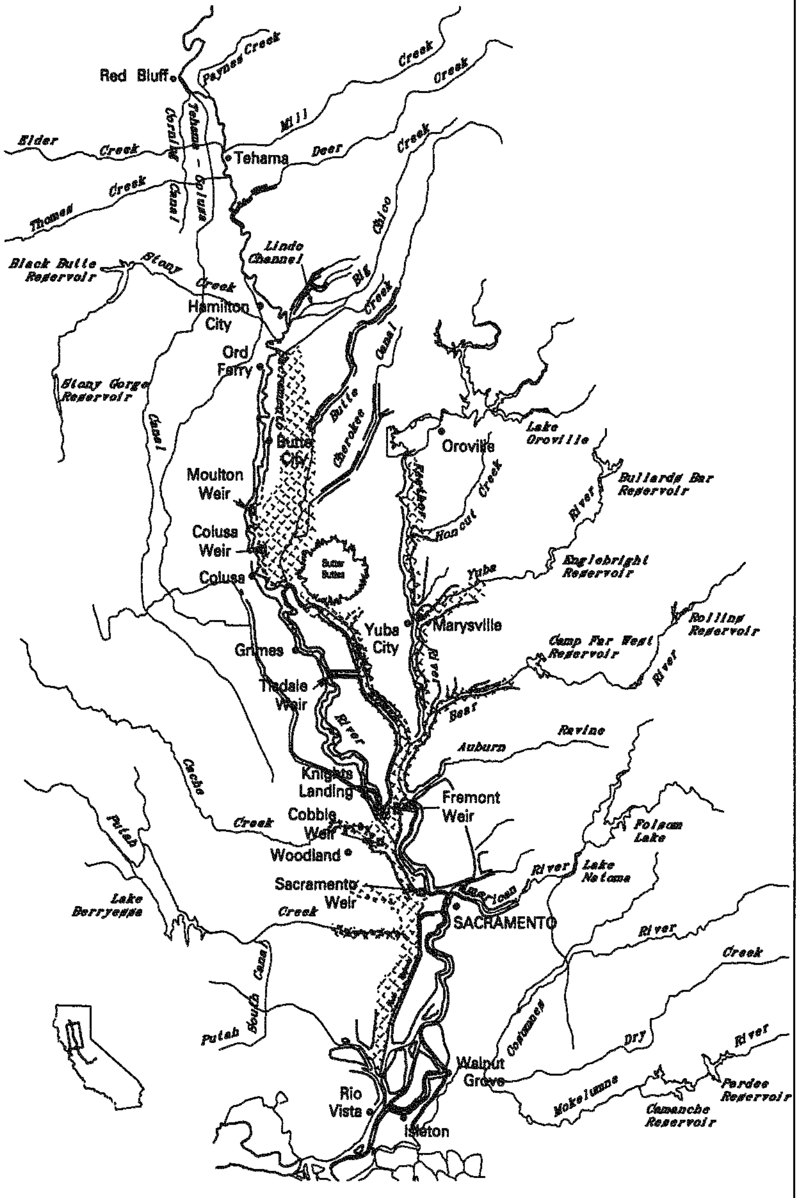


Figure 5. Sacramento River Flood Control Project

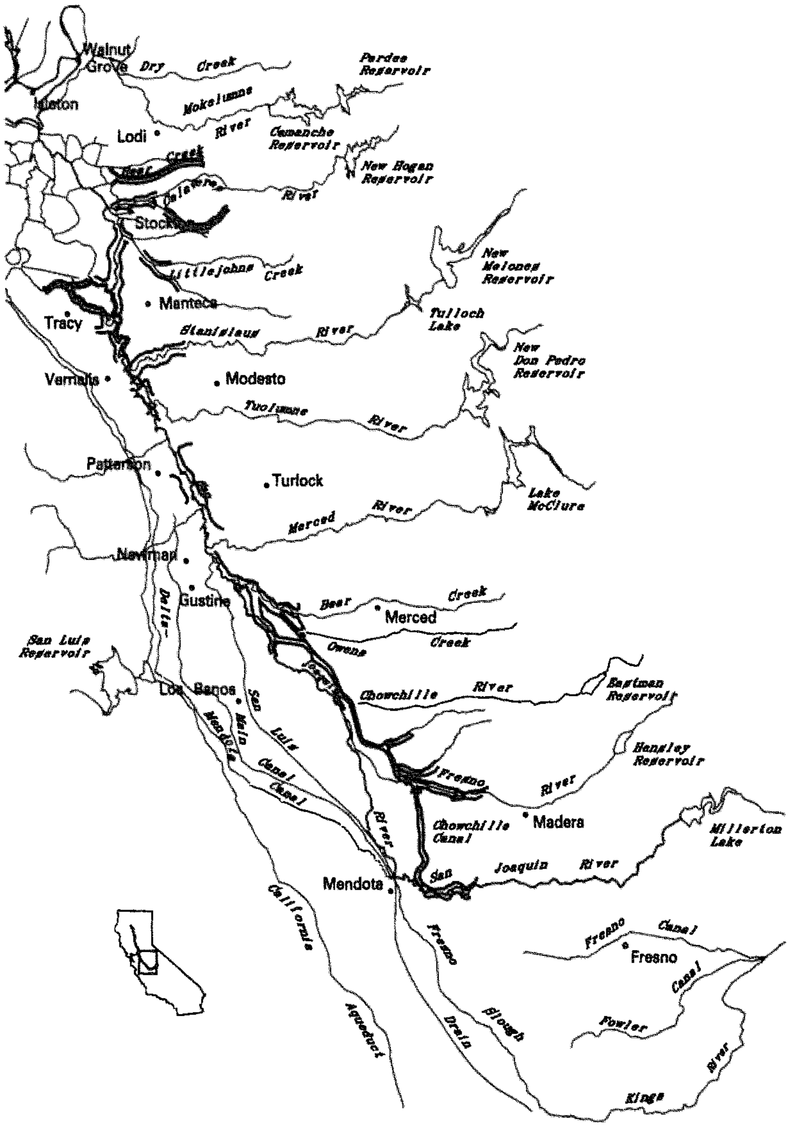


Figure 6. San Joaquin River and Tributaries Flood Control Project

Rainfall was relatively light after January 3, allowing the flood control system to drain and restoring reservoir flood control space. After the twentieth of January, another siege of heavy rain occurred. This was not as heavy as the year-end storms (about two-thirds as much and perhaps half as much at Shasta Dam) and, although warmer than normal, had snow levels about 2,000 feet lower which helped hold more water on the mountains as snow. But even so, runoffs were large with higher peaks on a few streams.

Sacramento River region reservoir flood control storage had been pretty well restored before the second storm and it was handled quite easily, even restraining normal flood releases to avoid overtopping the partly completed levee break repairs on Sutter Bypass and along the Feather River south of Marysville. This time low elevation stations also caught heavy rain with some local creek flooding.

In the San Joaquin River region there had not been enough time to restore full flood control space in the reservoirs. The channel capacity of the rivers is much more constricted than in the Sacramento Valley, limiting downstream releases. Amounts were quite heavy with over 11 inches estimated in the Stanislaus and San Joaquin (above Friant) basins during this second 7-day rain period. At one point, on January 24, in view of the weather forecasts, it appeared that a number of the foothill reservoirs would fill and spill and the Office of Emergency Service and others were alerted. Happily, the next two days of rain were less than forecast and releases were controlled to channel capacity downstream. Daily conference calls were held during the storm period with all operators in the region including the U.S Army Corps of Engineers, hosted by our Division of Flood Management.

OROVILLE OPERATIONS

Oroville Reservoir is a 3.5 million acre feet State Water Project reservoir on the Feather River of Northern California. It has 0.75 MAF of flood space, designed to control the standard project flood, a roughly 1/200 event.

Estimated peak inflow at Oroville was just over 300,000 cfs. The estimated 3-day unimpaired volume seems to have been about 1.4 MAF, about 92 percent of SPF. The peak is strange; looking almost truncated for about 12 hours (see chart on Figure 7.)

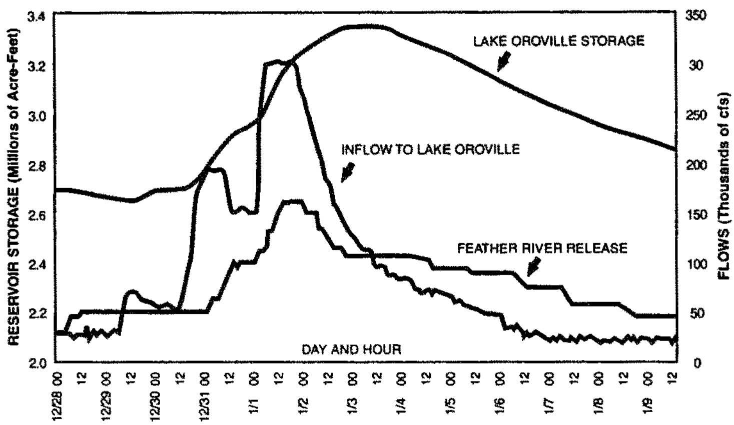


Figure 7 - Lake Oroville Operations, December 1996 and January 1997

In volume, this flood exceeded the previous record of the 1986 flood by quite a margin, perhaps 25 percent. At one point on January 1, we thought the inflow would be so much that the Lake would fill and spill perhaps 250,000 cfs worth. People were evacuated from the City of Oroville downstream. Fortunately the rain eased a little sooner than expected and the dam contained the runoff.

COMPARISON WITH OTHER FLOODS

Let's spend a little time comparing this flood to other floods this century, using 3 Sacramento basin rivers. First, look at the one-day estimated peak annual flows of the American River near Sacramento. Flood control on this stream has been the subject of controversy after the 1986 flood. Auburn Dam has been proposed to provide badly needed flood protection for the downstream urban area, but the proposal has been bogged down in environmental and cost controversy.

The existing major flood control dam, Folsom Dam, was designed just prior to 1950 and built in 1955, presumably to contain the standard project flood, with the record on hand. But look what has happened since. (See chart, Figure 8). It seems there is a trend for bigger and bigger floods. Now, the original design probably doesn't provide over a 1 in 70-year protection. Because of high levees the consequences of a failure would be disastrous.

Where there are large flood control reservoirs, a better measure of comparison is the 3-day volume - which more nearly corresponds to the need for flood space. The 3-day chart on the American River shows 1997 as essentially the same as 1986. Comparatively on the American River, the 1997 flood was a somewhat smaller flood than in river basins north and south of the American. As a side note, big floods on major Sierra rivers do not seem to happen in the big El Niño years; note that the peak flood flow in 1998 was near average.

The Feather River Chart shows the comparison for Oroville Dam. As noted before, this flood was perhaps 25 percent bigger than 1986, which itself was the biggest to that time, although not too much more than a 1907 flood. Also, you can see that the increasing trend with time is not as obvious as on the American River, unless one looks at the last 1997 event.

The 3rd chart is for the upper Sacramento River at Shasta Dam. Again, 1997 really stands out. Lastly, let's look at the Tuolumne River at Dan Pedro Dam which has 100 years of record. Again 1997 stands way out with only water year 1956 (December 1955) being comparable.

The last chart, Figure 13, shows 3-day flood flows on the 3 Sacramento basin rivers in one chart expressed as a ratio of the median flood. The historical medians are 44,300 cfs for the Sacramento River at Shasta Dam, 35,300 cfs for the Feather River at Oroville, and 22,400 cfs for the American River at Folsom. I've shown the 11 largest floods this century. Notice that the range (the ratio) becomes larger as one

Figure 8. Annual Maximum 1-Day Flood Flow, American River, in 1,000 cfs

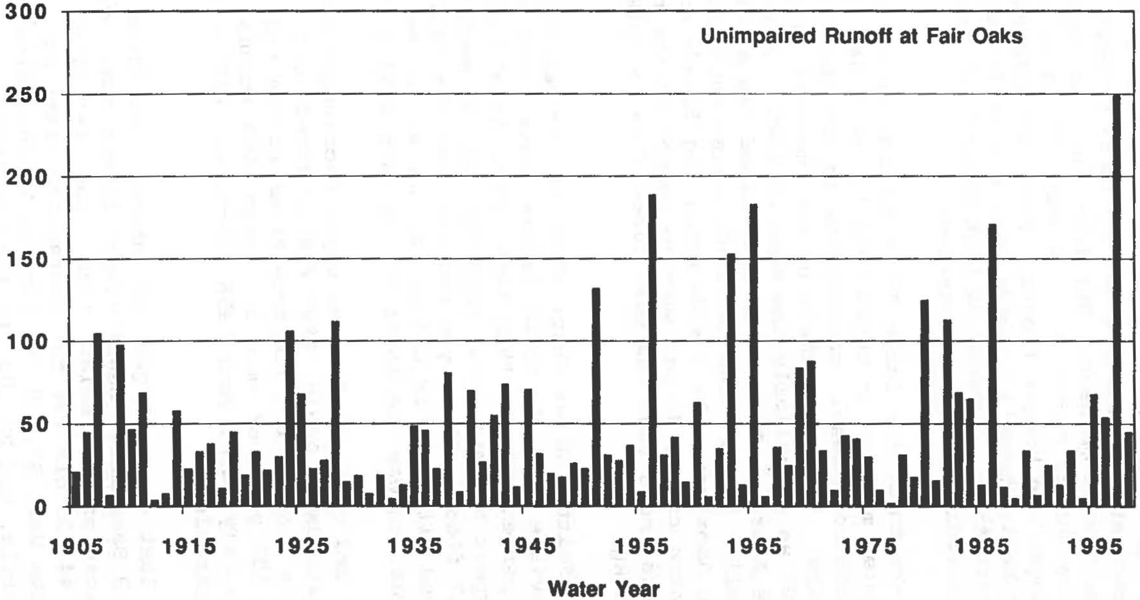


Figure 9. Annual Maximum 3-Day Flood Flow, American River, in 1,000 cfs

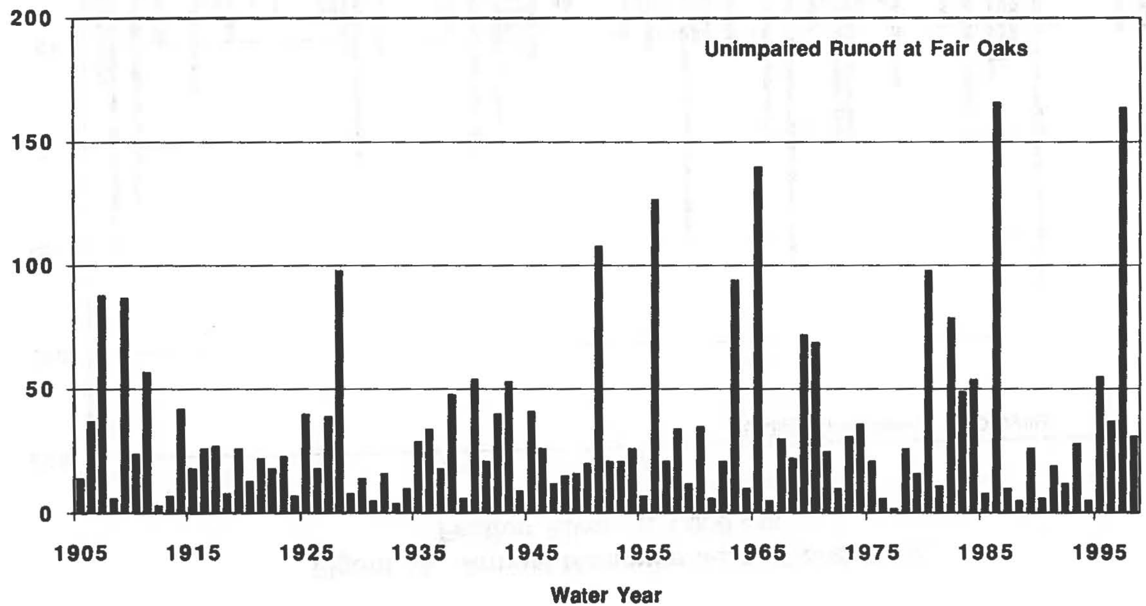


Figure 10. Annual Maximum 3-Day Flood Flow, Feather River, in 1,000 cfs

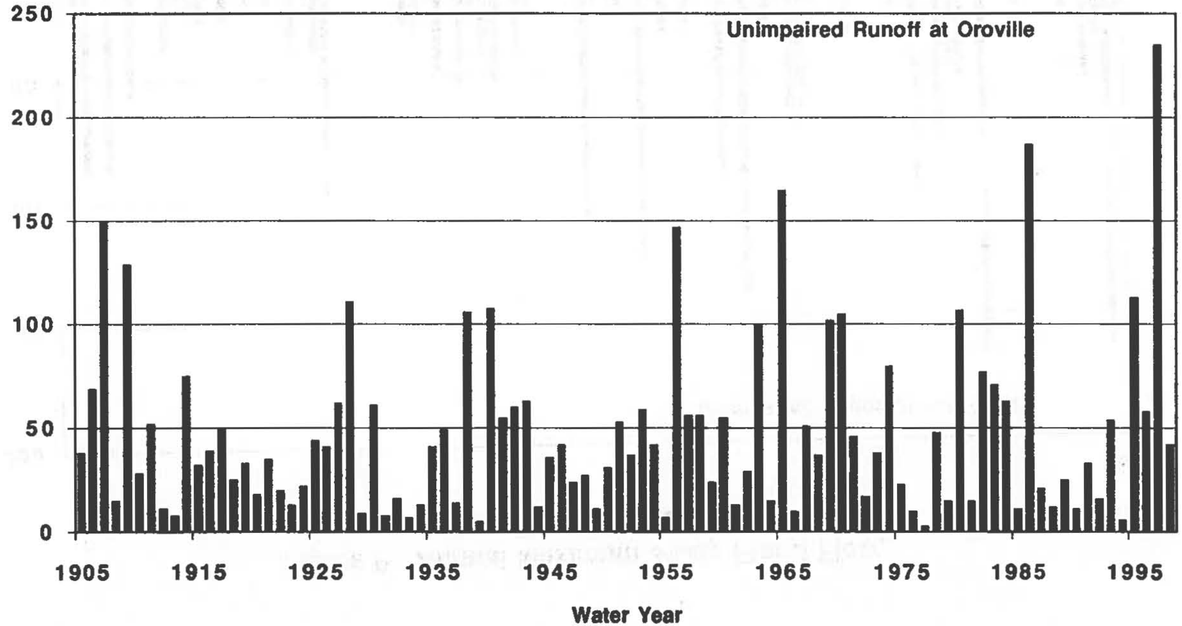


Figure 11. Annual Maximum 3-Day Flood Flow, Upper Sacramento River, in 1,000 cfs

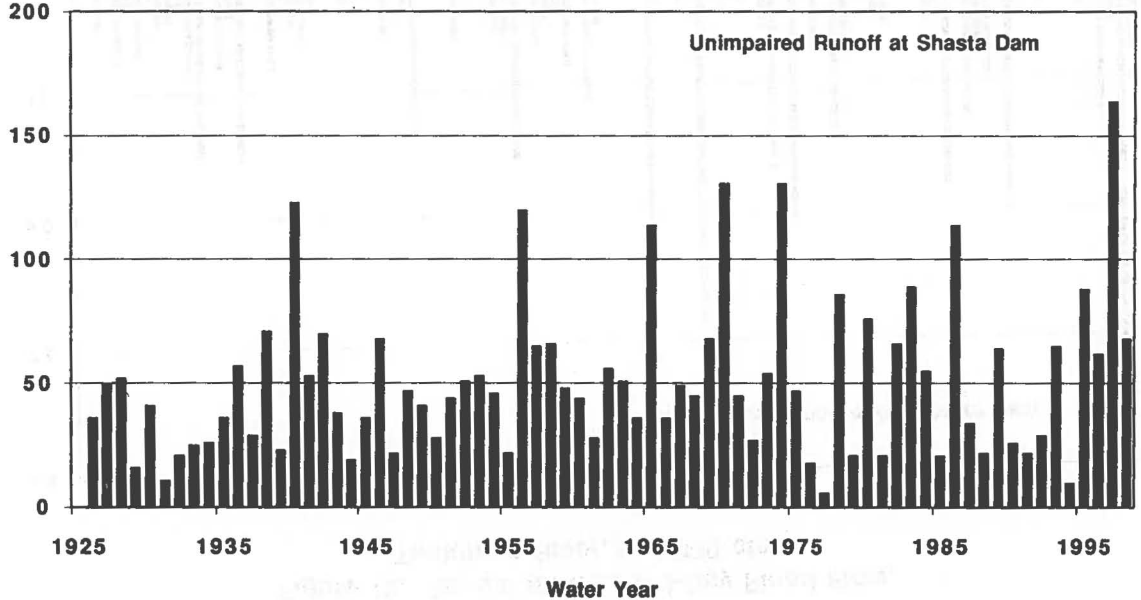


Figure 12. Annual Maximum 3-Day Flood Flow, Tuolumne River, in 1,000 cfs

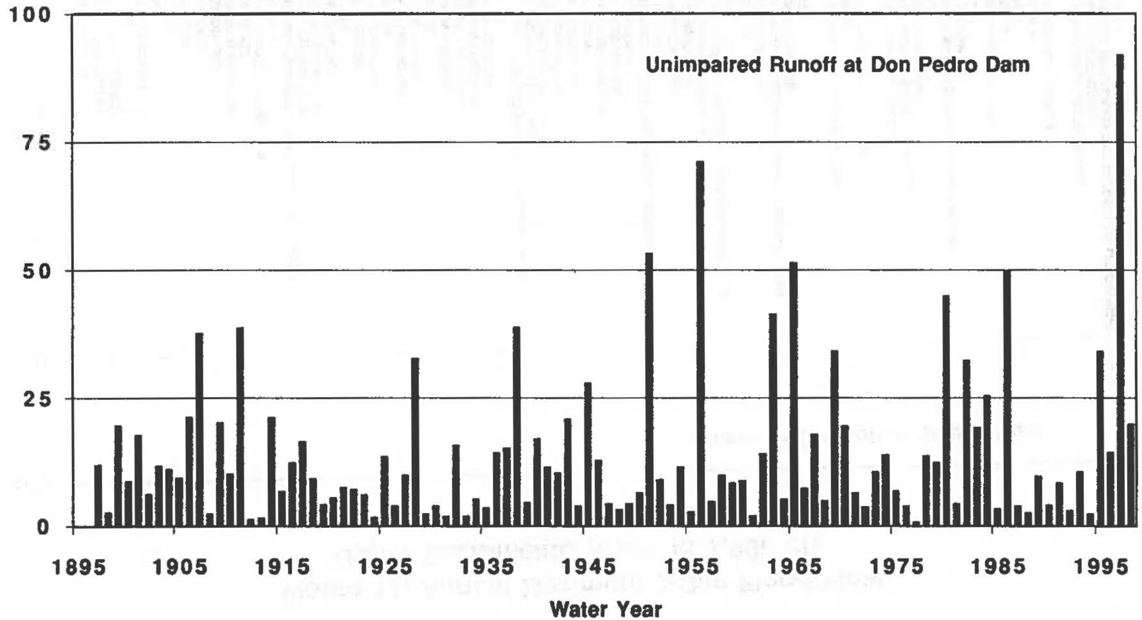
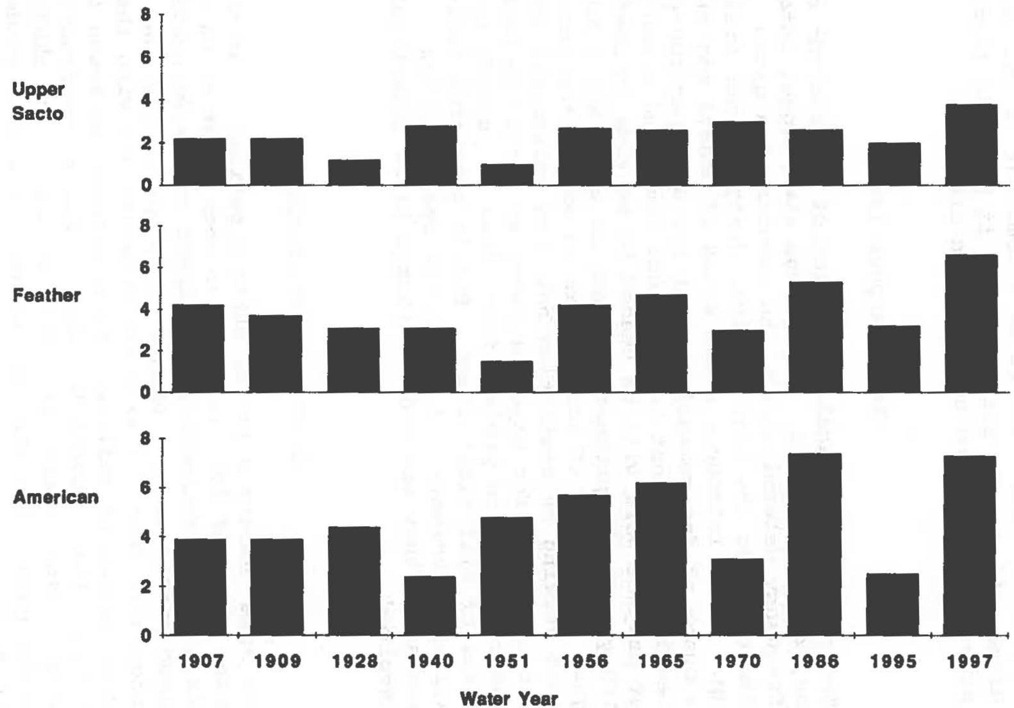


Figure 13. Comparison of Flood Sizes (to Median)
Three-Day Volume Divided by Median



goes south to the American River. Also, the apparent increasing size trend is more dominant in the American River. Why? I'm not sure. Is it possible there's some influence from upwind urban areas?

FLOOD FREQUENCIES

The recent flood again set a lot of new records on major Sierra rivers. When these are plugged into a frequency determination, the amounts at a given frequency or the risk at given design levels will go up. We'll introduce a new round of charts and probably a bunch of determinations that the existing 100-year levels are not that anymore, but less, and a new round of project work will be needed to provide revised 100-year flood protection, some in areas which have just done a lot of work. This is one of the problems with working on statistics based on relatively short record. Maybe for major projects we should go back to the old standard project flood idea or justify to some level of historical storm. People are being fooled by all these numbers and risks, not realizing how tentative they are and the rather large uncertainty involved.

CONCLUSION AND AFTERMATH

I'm sure there's a lot of other questions. But the great flood of 1997 is going to keep a lot of us busy for a while, evaluating its impact on how we provide flood protection for people and property and what the acceptable level of risk is--commensurate with the consequences of failure. Looking back, it seems that we have often thought too small. Are we overreacting to an unusual event this time, or are things changing to the point when the past is not a reliable guide to the future?

The final report of the Governor's Flood Emergency Action Team (FEAT), published in May of 1997, has over 50 long term action recommendations which are now the basis for the State's flood policy. Hopefully, as these are implemented, they will help Californians to work together to reduce flood damage.

In the aftermath of the 1997 flood we see much activity to improve flood protection and to restrict development in flood plains, much more so than after the very large 1986 event. I believe the primary reason was the 1997-98 El Niño and the press hype which went with it. It was possible for the public to dismiss the 1986 flood (which didn't cover as much area) as an unusual event which probably wouldn't happen again for a long time. Suddenly, with the El Niño threat, people felt vulnerable.

It was evident in 1997 that the existing Sacramento/San Joaquin River Flood Control System had some shortcomings. The Sacramento River flood project design was essentially a composite of the 1907 and 1909 floods laid out in a 1910 report. We now expect over 4 million more people in the Central Valley by year 2020, pushing its population over 10 million. An increasing urban area desires and needs a higher level of flood protection than predominately farm areas.

The El Niño wet winter threat did accelerate levee restoration to repair the damage from the 1997 flood so the levees would be as good as they were before. Certain ongoing reconstruction work also was undertaken and should be largely finished this year so that we can have more confidence the floodways will carry their design flow.

For the longer run, a 4-year comprehensive study of flood control in the Sacramento and San Joaquin River basins has started sponsored by the U.S. Army Corps of Engineers and The Reclamation Board of the State of California. The new study will take a systematic look at the entire region, not piece by piece. The goal will be to develop and implement plans for long range management of the river systems. It will include consideration of structural and nonstructural (nontraditional) flood damage reduction measures, as well as various ecosystem restoration measures. An important element is development of hydrologic/hydraulic models of the two river systems.

The FEAT report recommended and the State Legislature has provided funds for an increase in the State floodplain mapping program, to conduct computer modeling studies and prepare floodplain mapping of newly defined floodplains and rural areas with potential for urbanization.

Another element in the FEAT report was for the Governor to appoint a Floodplain Management Task Force to examine some thorny issues related to State and local floodplain management. Some of the items of study include:

- roles and responsibility of the State Reclamation Board, including its designated floodway program
- remedial action when levee maintenance is deficient
- requiring future urban developments to exceed national Flood Insurance program elevation requirements
- Mandatory flood insurance for structures with less than 200-year protection.
- evaluate land use policies; require notice on titles to make buyers aware of flood risk

The task force has been formed, and has started work on its report. They appointed a rather large advisory team, mostly from the private sector, to help.

Some measures in the FEAT report can be implemented right away as funds become available. The State and national legislatures have appropriated funds to make a significant start this year and we are optimistic that that support will continue.

There has also been more emphasis on giving rivers more room since the floods. This is often more economical in the long run. But we also have to recognize where people are; the existing cities and other developed areas have no alternative but need to rely heavily on structural measures for flood protection.

For the very long term, it will be important to maintain a general awareness that no project or program can ensure complete protection from flood damage. As the 21st century approaches, it may be appropriate to review the terminology one more time (having already changed from flood <u>control</u> to flood <u>management</u>) to flood <u>risk</u> management. This will make clear to people that there will always be some risk from any natural hazard, and that we all can (and should) do our part to recognize and minimize that risk by prudent individual and governmental choices.

WATER RESOURCES MANAGEMENT IN ARIZONA

Rita P. Pearson[1]

The famous satirist Mark Twain once said that "water, taken in moderation, cannot hurt anybody." Twain also noted that a lack of water could lead to severe consequences.

Former President Teddy Roosevelt recognized that with these consequences came a long-term need to preserve our precious natural resources. He said that the nation behaves well if it treats its natural resources as assets turned over to the next generation increased in value, and not impaired.

Arizona's founding fathers recognized that this preservation of our natural resources would guide the quality of life for future generations in Arizona to enjoy. They also recognized the opportunity this arid region offers and that planning and foresight must be initiated to avoid the depletion of these precious supplies.

INNOVATIVE PROGRAMS

Arizona has gone to great lengths to ensure a secure, reliable future water supply. Whether successfully negotiating with our sister states for an equitable Colorado River entitlement, to initiating the plan to create the Central Arizona Project to transport Colorado River water to the most populated areas in the state, Arizona has taken control of its destiny.

With the creation of the Groundwater Management Code in 1980, Arizona created a Code that would protect against severe groundwater depletion. The Department of Water Resources followed up with the Assured Water Supply Program, taking the requirements of the Code one step further by placing more restrictions on groundwater use by future growth.

In 1995, the Arizona Water Protection Fund was created to preserve and enhance riparian areas across the state. In 1996, the Arizona Water Banking Authority was created to ensure Arizona's full use of its 2.8 million acre-feet Colorado River water entitlement.

UNIQUE CHALLENGES

Through the years, Arizona has become one of the fastest growing states in the nation, and we continue to grow. Our current population is 4.6 million. Projections indicate that the population will reach almost 8 million by the year 2025. It will reach more than 11 million by the year 2050.

---

[1]Director, Arizona Department of Water Resources, Phoenix, Arizona

GROUNDWATER MANAGEMENT CODE

With this rapid growth and development occurring in areas across Arizona, our leaders recognized that we must begin planning to avoid a severe depletion of our groundwater supplies. This led to the creation of the Groundwater Management Code in 1980.

The Code created four Active Management Areas, or AMAs, in areas where groundwater depletion was most severe. These areas include Phoenix, Tucson, Pinal and Prescott. A fifth AMA was added in 1994 in Santa Cruz County. Currently more than 80 percent of Arizona's population lives in these AMAs, making up 70 percent of the state's water use.

The Arizona Department of Water Resources was mandated to create a series of five management plans to reach the ultimate goal of "safe-yield" by 2025. Safe-yield is achieved when no more groundwater is being withdrawn from the ground than is annually replaced. The Management Plans focus on conservation requirements, Gross Per Capita Demand (GPCD) requirements for cities and allotments for golf courses, turf facilities and agriculture.

The Department is working on the Third Management Plan, or TMP, which will go into effect in the year 2000. I would like to focus on a few hurdles the Department has encountered while developing the TMP.

Our main concern while developing the TMP is whether or not the right tools are in place to achieve safe-yield.

Our main concern while developing the TMP is whether or not the right tools are in place to achieve safe-yield. Following our current path, the answer is no. Conservation alone will not allow us to reach safe-yield.

Another concern is that our overall demand is increasing. The Phoenix AMA's demand, for example, is currently about 2.2 million acre-feet. By 2025, it's expected to increase to more than 2.6 MAF. Per capita demands are also no longer decreasing and could, in fact, be increasing. In Phoenix, the GPCD is 239 gallons per day, compared to Tucson at 178. The GPCD for Las Vegas is 333 gallons per day; compare that with Las Angeles, which uses approximately 300 gallons per day.

Agricultural pumping is also a main concern because many of the agricultural pumpers in Arizona do not have incentives or mandates to switch from using groundwater. The Groundwater Code prohibits new agricultural acreage going into production and places conservation requirements on water used for irrigation purposes. The problem, however, is that most existing farms have long-standing rights to pump groundwater that have not been abrogated. To date, the most effective technique to reduce groundwater pumping is to subsidize the costs of Central Arizona Project (CAP) water to farmers.

Golf courses are a highly visible use of water in Arizona and if not managed appropriately, can send the wrong message when attempting to create a culture of conservation. In the Phoenix AMA along, more than 25 golf courses have been added since 1992. Currently, there are more than 140 golf courses in the Phoenix AMA. Golf courses are typically dependent upon the use of groundwater. In fact, 60 percent of the state's golf courses are supplied with groundwater. The average water use for an 18-hole golf course is 580 acre-feet per year, enough water to meet the needs of nearly 3,000 people. One incentive being examined in the TMP is to switch to effluent and untreated surface water to water the golf courses, rather than further depleting our groundwater supplies.

The Assured Water Supply Program is another program designed to curtail groundwater use and encourage the use of renewable water supplies. These rules require all new subdivisions in the Phoenix and Tucson AMAs to secure renewable supplies or replenish their groundwater use. This is the Department's most effective tool in limiting additional groundwater mining.

These regulations were created because the state recognized that pumping too much groundwater could result in serious consequences. One impact is land subsidence. Drops in the land surface by as much as 18 feet have been discovered at Luke Air Force Base, just west of Phoenix. We also have future impact concerns in areas just east of the Phoenix area. Poor water quality, increased energy costs to pump, inadequate supplies in drought periods, and the elimination of some riparian habitats are also major concerns.

ARIZONA WATER BANK

While our groundwater supplies are diminishing more rapidly than they are being replaced, the use of surface water and specifically Colorado River water has become a more significant course of water in Arizona. A substantial amount of Arizona's water supplies derive from the Colorado River via the CAP. CAP service areas include Maricopa, Pinal and Pima counties. The CAP is designed to deliver 1.5 million acre-feet of water, but as we know, CAP deliveries are subject to shortage. By 2050, estimates indicate that shortages to CAP deliveries will occur almost 50 percent of the time.

As Arizona's dependency on the Colorado River increases, so does the dependence of the other Lower Basin states on the Colorado. Population growth is continuing in California. California has been using almost 1 MAF more than its 4.4 entitlement for years. Las Vegas is currently the fastest growing city in the country. Nevada is at almost full use of its 300,000 acre-feet annual entitlement.

These circumstances let to the creation of the Arizona Water Banking Authority by the Arizona State Legislature in 1996. The Water Bank purchases a portion of the state's unused Colorado River water, brings it into central and southern Arizona through the CAP, and either stores it in underground aquifers or substitutes the more expensive CAP water for groundwater pumping. The Water Bank has strengthened the long-term reliability of Colorado River supplies delivered via the CAP and ensured Arizona's use of its full 2.8 MAF entitlement of Colorado River water. The Water Bank serves as a drought plan. It has been a huge success. By the end of 1997, more than 325,000 acre-feet of Colorado River water had been stored. The Bank intends to store at least this much in 1998. The Bureau of Reclamation's most recent projections indicate that 2.67 MAF of our 2.8 MAF Colorado River water entitlement was used in 1997.

INTERSTATE RULES

The Arizona Water Bank is also involved in interstate discussions. At the end of 1997, Secretary of the Interior Bruce Babbitt released proposed rules that would govern interstate water transfers of Colorado River water within the Lower Basin. The proposed rules are permissive, and with an interstate storage agreement, could allow for the storage of Colorado River offstream in one Lower Basin state and the recovery of water in another.

More than 40 entities have commented to the Bureau of Reclamation on the proposed rules. The Bureau is expected to release final rules in early July. The final rule will then go to Congress for a mandatory waiting period of 60 days. If Congress does not act on it, the rule becomes final.

REDUCING CALIFORNIA'S DEMAND

The Department has also been involved in discussions with the other five Basin states to get California to reduce its dependence on the Colorado River. California unveiled a plan last August to reduce its dependence on Colorado River water, but the plan does not go far enough. And unfortunately, it has not yet been agreed to by the six California agencies with Colorado River contracts.

I would like to first commend the California water agencies for their effort in working towards a solution, but more progress needs to be made before the Basin States sign off on the plan. One issue is the implementation of core transfers from agricultural to urban uses within California. A major concern is the reliance on "surplus" declarations on the system and Lake Mead water banking (also known as top water banking). The discussions on interim and long-term surplus and shortage operating criteria will only begin when California makes a solid commitment to reduce its dependence on the Colorado River.

More work still needs to be completed on the California 4.4 Plan. It's vital that the six agencies work out a plan that will reduce their dependence on the Colorado River and that is also supported by the six Basin States that share the Colorado River supplies with California.

CAWCD SETTLEMENT

Both the Arizona Water Bank and the Department have been involved in conversations with the Central Arizona Water Conservation District (CAWCD), major water users in the CAP service area and federal entities on the proposed CAWCD Settlement with the Bureau of Reclamation over the states' share of the construction costs of the CAP. The CAP was completed in 1993. A proposed settlement was introduced in February that would provide that CAWCD's share of the construction costs was $1.7 billion, rather than $2.3 billion. The proposal would also divert additional CAP water to Indian users, meaning almost one-half of all CAP deliveries would be under tribal control. Also, more than 65,000 acre-feet of high priority water was proposed to go to the Indian Tribes that would have otherwise gone to municipalities and other water providers within the CAP service area. We are convinced that this M&I priority water will be needed to meet future growth in our communities.

The Department has major concerns with this initial proposal. Secretary Bruce Babbitt and Arizona Senator Jon Kyl stepped into the negotiation process to assist in the development of a proposal that can be agreed upon by all involved parties. They asked the Department of Water Resources to facilitate additional settlement discussions. The plan must be acceptable to the water users of the State of Arizona, including municipal, agricultural and Indian water interests.

ENDANGERED SPECIES ACT ISSUES

Arizona was closely watching the appeal before the Ninth Circuit filed by the Southwest Center for Biological Diversity, an environmental group. The original suit, filed last year by Southwest Center, claimed that the storage of water above elevation 1178 feet in Lake Mead by the Bureau of Reclamation was hindering the habitat of the Southwestern Willow Flycatcher. Federal district court Judge Earl Carrol ruled in favor of the defendants in the action brought by the Southwest center, but Southwest Center appealed Judge Carroll's decision to the Ninth Circuit Court of Appeals. Fortunately, upon appeal, the Ninth Circuit affirmed Judge Carroll's decision, ruling that the Southwest Center had not properly notified the defendants of their intention to file a lawsuit.

If Southwest Center were successful in their litigation, more than 3 million acre-feet of our water could have been released from Lake Mead, impeding water deliveries throughout the Lower Basin.

Although this is a major victory for the water users in the State of Arizona, we must recognize that issues relating to the enforcement of the ESA are here to stay.

ARIZONA STRATEGIES FOR THE FUTURE

Arizona has been blessed with a sizeable amount of water, both surface and groundwater, but our problems are in the delivery of the water.

Good water management in Arizona is not just an issue in the largely populated areas, such as Phoenix and Tucson. Arizona will face some concerns with rural community growth and the impacts on groundwater supplies in areas that are not within AMAs. Many of these areas do not have imported supplies, such as CAP water.

Arizona is also confronting some major challenges in the management of our future water supplies. We are attempting to meet the demands of both man and nature with programs such as the Water Protection Fund, conservation and augmentation grants and recharge projects, which include riparian restoration and preservation.

There still remains an overriding need for Arizona's leaders to continue to plan. We must continue with the commitments we have made to the people of Arizona with the Groundwater Management Code and the Arizona Water Bank. We must also continue with interstate discussions. We must continue with our commitment to preserve our finite resources.

And, as Teddy Roosevelt suggested, we must turn over our natural resources increased in value for our next generation of users to enjoy.

Participant Listing

Saeed Akbar
Hokkaido University
Kita Ku Kita 14, Nishi 4
Sky Heights N14, Room No. 216
Sapporo Hokkaido 001, Japan
Telephone: 81-747-7860
E-mail: memon@agecon.agr.hokudai.ac.jp

Janet Albers
Campbell Scientific, Inc.
815 West 1800 North
Logan, UT 84321
Telephone: 435-753-2342
Fax: 435-750-9540
E-mail: info@campbellsci.com

Richard G. Allen
University of Idaho
Research and Extension Center
3793 North 3600 East
Kimberly, ID 83341
Telephone:208-423-6601
Fax: 208-423-6559
E-mail: rallen@kimberly.uidaho.edu

Michael C. Archer
Murray, Burns and Kienlen
1616 29th Street, Suite 300
Sacramento, CA 95816
Telephone: 916-456-4400
Fax: 916-456-0253
E-mail: mcarcher@jps.net

Brian Betcher
Maricopa-Stanfield Irrigation & Drainage District
41630 West Louis Johnson
Maricopa, AZ 85204
Telephone: 602-253-8905
Fax: 520-424-3281
E-mail: brian@msidd.com

Thad Bettner
Fresno Irrigation District
2907 South Maple
Fresno, CA 93725
Telephone: 209-233-7161, x19
Fax: 209-233-8227

Harold Blume
Natural Resources Conservation Service, USDA
225 West Marco Polo Road
Phoenix, AZ 85027
Telephone: 602-379-4934 x135
Fax: 602-379-4935
E-mail: hblume@az.nrcs.usda.gov

David Bradshaw
Imperial Irrigation District
333 East Barioni Boulevard
Imperial, CA 92251
Telephone: 760-339-9083
Fax: 760-355-1268
E-mail: smousel@quixotic

Gary Brady
Stantech Consulting Inc.
7776 Pointe Parkway West, Suite 290
Phoenix, AZ 85044
Telephone: 602-438-2200
Fax: 602-431-9562
E-mail: gbrady@stantech.com

L. Albert Brower
Bureau of Reclamation
P.O. Box 25007, D-8550
Denver, CO 80225
Telephone: 303-445-2507
Fax: 303-445-6351
E-mail: abrower@do.usbr.gov

Joseph I. Burns
Consulting Civil Engineer
1730 Cathay Way
Sacramento, CA 95864
Telephone: 916-481-4092
Fax: 916-481-4095
E-mail: jiburns@aol.com

Peter Canessa
Consultant
P.O. Box 14307
San Luis Obispo, CA 93406
Telephone: 805-544-1725
Fax: 805-544-1725
E-mail: pwcanessa@aol.com

Charles Caruso
Office of the New Mexico State Engineer
10432 Manzanillo, NE
Albuquerque, NM 87111
Telephone: 505-275-8826
Fax: 505-827-7874
E-mail: agrimex@rt66.com

Robert W. Charley
CH2M Hill, Inc.
P.O. Box 28440
Tempe, AZ 85285
Telephone: 602-847-8588

Jonathan Chee
San Carlos Apache Tribe
P.O. Box 0
San Carlos, AZ 85542
Telephone: 520-475-2320
Fax: 520-475-2484
E-mail: jchee@juno.com

Albert J. Clemmens
Agricultural Research Service, USDA
4331 East Broadway Road
Phoenix, AZ 85040
Telephone: 602-379-4356
Fax: 602-379-4355
E-mail: bclemmens@uswcl.ars.ag.gov

Gene Cockran
Fresno Valves & Casting, Inc.
P.O. Box 40
Selma, CA 93662
Telephone: 209-834-2511
Fax: 209-834-4821

Chad Comes
Bureau of Reclamation
304 East Broadway Avenue
Bismarck, ND 58501
Telephone: 701-250-4242
Fax: 701-250-4326

David Cone
Broadview Water District
P.O. Box 95
Firebaugh, CA 93622
Telephone: 209-659-2004
Fax: 209-659-3526
E-mail: bwd@inreach.com

Grant G. Davids
Davids Engineering, Inc.
1105 Kennedy Place, Suite 5
Davis, CA 95616
Telephone: 530-757-6107
Fax: 530-757-6118
E-mail: grant@de-water.com

Wayne Deason
Bureau of Reclamation
P.O. Box 25007
Denver, CO 80225
Telephone: 303-445-2781
Fax: 303-445-6464
E-mail: wdeason@do.usbr.gov

Richard Dent
Bureau of Reclamation
2222 West Dunlap
Phoenix, AZ 85304
Telephone: 602-216-3809
Fax: 602-216-4003

Johannes J. DeVries
University of California, Davis
324 Encina Avenue
Davis, CA 95616
Telephone: 530-756-8372
Fax: 530-757-1035
E-mail: jjdevries@ucdavis.edu

Franklin E. Dimick
Dames & Moore Group
7552 Wickham Drive
Citrus Heights, CA 95610
Telephone: 916-386-4200
Fax: 916-387-0802
E-mail: hysfd@dames.com

Arnold K. Dimmitt
Metropolitan Water District of Southern California
107 South 5th Street, Suite 200
El Centro, CA 92243
Telephone: 760-352-4242
Fax: 760-352-0814
E-mail: adimmitt@mwd.dst.ca.us

Anisa Joy Divine
Imperial Irrigation District
1531 Ross Avenue, #229
El Centro, CA 92243
Telephone: 760-339-9036
Fax: 760-339-9009

Sandjar Djalalov
Texas A & M University
Department of Agricultural Economy
College Station, TX 77843
Telephone: 409-845-891
Fax: 409-845-6378

Guy Fipps
Texas Agricultural Extension Service
207B Agricultural Engineering
College Station, TX 77843
Telephone: 409-845-3977
Fax: 409-847-3932
E-mail: g-fipps@tamu.edu

Ivan W. Flint
Weber Basin Water Conservation District
2837 East Highway 193
Layton, UT 84040
Telephone: 801-771-1677
Fax: 801-544-0103

Jay W. Franson
Franson-Noble & Associates, Inc.
P.O. Box 606
American Fork, UT 84003
Telephone: 801-756-0309
Fax: 801-756-0481
E-mail: franoble@xmission.com

Wilton Fryer
Turlock Irrigation District
333 East Canal Drive
Turlock, CA 95380
Telephone: 209-883-8316
Fax: 209-632-3864
E-mail: wbfryer@tid.org

Rick L. Gold
Bureau of Reclamation
1886 South Wasatch Drive
Salt Lake City, UT 84108
Telephone: 801-524-3602
Fax: 801-524-3855
E-mail: rgold@uc.usbr.gov

Robert S. Gooch
Salt River Project
P.O. Box 52025
Phoenix, AZ 85072
Telephone: 602-236-5227
Fax: 602-236-2737
E-mail: rsgooch@srp.gov

Javier Grajeda
Bureau of Reclamation
700 East San Antonio, Room B-318
El Paso, TX 79901
Telephone: 505-248-5357
Fax: 505-248-5356

Stephen R. Grattan
University of California, Davis
Department of Land, Air and Water Resources
One Shields Avenue
Davis, CA 95616
Telephone: 530-752-4618
Fax: 530-752-5262
E-mail: srgrattan@ucdavis.edu

J. C. Guitjens
University of Nevada, Reno
1000 Valley Road
Reno, NV 89512
Telephone: 702-784-4071
Fax: 702-784-4583
E-mail: guitjens@unr.edu

Dean Hagstrom
Bureau of Reclamation
2222 West Dunlap
Phoenix, AZ 85069
Telephone: 602-216-3823
Fax: 602-216-4000

Steve Hansen
Bureau of Reclamation
505 Marquette Street, NW, Suite 1313
Albuquerque, NM 87102
Telephone: 505-248-5349
Fax: 505-248-5356
E-mail: shansen@uc.usbr.gov

George H. Hargreaves
Utah State University
1660 East 1220 North
Logan, UT 84341
Fax: 801-797-1248

Pierce R. Harrison Jr.
BIA-Wapato Irrigation Project
P.O. Box 220
Wapato, WA 98951
Telephone: 509-877-3155
Fax: 509-877-3478
E-mail: pharrisn@wolfenet.com

Curt Hartzell
Bureau of Reclamation
P.O. Box 25007, D-8510
Denver, CO 80225
Telephone: 303-445-2482
Fax: 303-445-6351
E-mail: chartzell@do.usbr.gov

Jerry Jacoby
Bureau of Reclamation
P.O. Box 1749
Yakima, WA 98907
Telephone: 509-575-5848
Fax: 509-454-5650
E-mail: jjacoby@pn.usbr.gov

Marvin E. Jensen
Consultant
1207 Springwood Drive
Fort Collins, CO 80525
Telephone: 970-226-5402
Fax: 970-226-5402
E-mail: mjensen@aol.com

Sam H. Johnson III
Consortium for International Development
8460 East Fernhill Drive
Tucson, AZ 85750
Telephone: 520-721-9543
Fax: 520-885-8093
E-mail: shj3@aol.com

J. Ronald Johnston
U.S. Department of the Interior
302 East 1860 South
Provo, UT 84606
Telephone: 801-379-1103
Fax: 801-379-1209
E-mail: rjohnston@uc.usbr.gov

Stephen M. Jones
Bureau of Reclamation
P.O. Box 61470
Boulder City, NV 89006
Telephone: 702-293-8186
Fax: 702-293-8042
E-mail: sjones@lc.usbr.gov

David N. Kennedy
State of California,
Department of Water Resources
P.O. Box 942836
Sacramento, CA 94236
Telephone: 916-653-7007
Fax: 916-653-6985
E-mail: ddeanda@water.ca.gov

John H. Korinetz
MSO Technologies, Inc.
571 North Poplar Street, Suite C
Orange, CA 92868
Telephone: 714-939-8668
Fax: 714-939-6310
E-mail: msotechoc@earthlink.net

Michael D. Kube
Bureau of Reclamation
P.O. Box 1607
Grand Island, NE 68803
Telephone: 308-389-4622, x217
Fax: 308-389-4780

Darcey Lapetina
U.S. Committee on Irrigation and Drainage
1616 Seventeenth Street, Suite 483
Denver, CO 80202
Telephone: 303-628-5581
Fax: 303-628-5431
E-mail: stephens@uscid.org

Ed Latimer
CH2M Hill
1620 West Fountainhead Parkway
Tempe, AZ 85282
Telephone: 602-966-8188
Fax: 602-966-9450
E-mail: elatimer@ch2m.com

Tom Levy
Coachella Valley Water District
P.O. Box 1058
Coachella, CA 92236
Telephone: 760-398-2651
Fax: 760-398-3711

Joe Lima
Modesto Irrigation District
P.O. Box 4060
Modesto, CA 95352
Telephone: 209-526-7562
Fax: 209-526-7352
E-mail: joel@mid.org

Mark Limbaugh
Water District No. 65, State of Idaho
102 North Main Street
Payette, ID 83661
Telephone: 208-642-4465
Fax: 208-642-1042
E-mail: wdist65@primenet.com

John Linder
Fresno Valves & Casting, Inc.
P.O. Box 40
Selma, CA 93662
Telephone: 209-834-2511
Fax: 209-834-4821

Shane Lindstrom
Gila River Indian Community
P.O. Box E
Sacaton, AZ 85247
Telephone: 520-562-3301
Fax: 520-562-4008

Gordon E. Little
Metropolitan Water District of Southern California
P.O. Box 54153
Los Angeles, CA 90054
Telephone: 213-217-7924
Fax: 213-217-7778

Jim A. Lively
Bureau of Reclamation
P.O. Box 640
Carson City, NV 89702
Telephone: 702-882-3436
Fax: 702-882-7592

Joseph M. Lord, Jr.
JMLord, Incorporated
267 North Fulton
Fresno, CA 93701
Telephone: 209-268-9755
Fax: 209-486-6504
E-mail: jmlord@jmlordinc.com

Godswill Makombe
Colorado State University
Department of Agricultural
and Resource Economics
Fort Collins, CO 80523
Telephone: 970-491-8087
Fax: 970-491-2067
E-mail: gmakombe@lamar.colostate.edu

Cameron McCormick
Kansas Geological Survey
1930 Constant Avenue
Lawrence, KS 66044
Telephone: 785-864-3965
Fax: 785-864-5317
E-mail: mccormic@eagle.ukans.edu

Hector Mendez
Imperial Irrigation District
P.O. Box 937
Imperial, CA 92251
Telephone: 760-339-9265
Fax: 760-339-9262

John L. Merriam
California Polytechnic State University
235 Chaplin Lane
San Luis Obispo, CA 93405
Fax: 805-756-2626

Thomas E. Mitchell
Bureau of Reclamation
2133 South Yukon Way
Lakewood, CO 80227
Telephone: 303-236-8550, x424
Fax: 303-236-3929
E-mail: tmitchell@do.usbr.gov

David J. Molden
International Water Management Institute
P.O. Box 2075
Colombo
Sri Lanka
Telephone: 94-1-867-404
Fax: 94-1-866-854
E-mail: d.molden@cgnet.com

Bakhtiyor Mukhammadiev
Texas A & M University
315 Scoates Hall
College Station, TX 77840
Telephone: 409-485-4076
E-mail: bmukhammadiev@agen.tamu.edu

Reed R. Murray
Department of Interior
960 West 680 South
Orem, UT 84858
Telephone: 801-379-1237
Fax: 801-379-1209
E-mail: rmurray@uc.usbr.gov

James Narvaez
Elephant Butte Irrigation District
P.O. Box 1509
Las Cruces, NM 88004
Telephone: 505-526-6671
Fax: 505-523-9666

Azimjon S. Nazarov
Texas A & M University
315 Scoates Hall
College Station, TX 77840
Telephone: 409-485-4076
Fax: 409-845-3932
E-mail: anazarov@agen.tamu.edu

David E. Nelson
Bureau of Reclamation
8448 Monad
Billings, MT 59106
Telephone: 406-247-7630
Fax: 406-247-7898
E-mail: dnelson@gp.usbr.gov

Mark Niblack
Bureau of Reclamation
P.O. Box D
Yuma, AZ 85366
Telephone: 520-343-8155
Fax: 520-343-8320

James D. Oster
University of California, Riverside
Soil and Environmental Sciences
Riverside, CA 92521
Telephone: 909-787-5100
Fax: 909-787-5522
E-mail: oster@mail.ucr.edu

Mahinda Panapitiya
Davids Engineering, Inc.
1105 Kennedy Place, Suite 5
Davis, CA 95616
Telephone: 530-757-6107
Fax: 530-757-6118

Rita P. Pearson
Arizona Department of Water Resources
500 North Third Street
Phoenix, AZ 85004
Telephone: 602-417-2410
Fax: 602-417-2415
E-mail: rppearson@adwr.state.az.us

Craig Pope
Texas A & M University
207 Scoates Hall
College Station, TX 77843
Telephone: 409-845-3607
Fax: 409-847-8828
E-mail: cpope@tamu.edu

Daydra Presley
Remote Control Technology, Inc.
18342 Redmond Way, Building B
Redmond, WA 98052
Telephone: 425-885-6362
Fax: 425-883-4667
E-mail: sales@remotecontroltech.com

William A. Price
Consultant
P.O. Box 1698
Silverthorne, CO 80498
Telephone: 970-468-8222
Fax: 970-468-8341
E-mail: wmaprice@aol.com

John Redlinger
Bureau of Reclamation
P.O. Box 61470
Boulder City, NV 89006
Telephone: 702-293-8592
Fax: 702-293-8042
E-mail: jredlinger@lc.usbr.gov

John A. Replogle
Agricultural Research Service, USDA
4331 East Broadway Road
Phoenix, AZ 85040
Telephone: 602-379-4356, x246
Fax: 602-379-4355
E-mail: jreplogle@uswcl.ars.ag.gov

W. Martin Roche
Consultant
4045 Dale Road, Unit D
Modesto, CA 95356
Telephone: 209-545-4183
Fax: 209-545-4183
E-mail: wmroche@aol.com

Manuel C. Rodriguez
Bureau of Reclamation
2222 West Dunlap
Phoenix, AZ 85304
Telephone: 602-216-3811
Fax: 602-216-4003

Steven G. Rolph
Bureau of Reclamation
P.O. Box 1749 (UCA-1104)
Yakima, WA 98907
Telephone: 509-575-5848
Fax: 509-454-5611
E-mail: srolph@pn.usbr.gov

Maurice Roos
State of California, Department of Water Resources
3310 El Camino Avenue, Room 200
Sacramento, CA 95821
Telephone: 916-574-2625
Fax: 916-574-2767
E-mail: mroos@water.ca.gov

Ralph R. Ross
Consultant
352 N Street, SW
Washington, DC 20024
Telephone: 202-485-9708
Fax: 202-485-9720

Ron Samuelian
Fresno Irrigation District
2907 South Maple
Fresno, CA 93725
Telephone: 209-233-7161
Fax: 209-233-8227

Jerry Schaack
Garrison Diversion Conservancy District
348 Joal Drive
Carrington, ND 58421
Telephone: 701-652-3194
Fax: 701-652-3195
E-mail: gdcd@daktel.com

Sam Schaefer
Bureau of Reclamation
1080 South Van Gordon Court
Lakewood, CO 80228
Telephone: 303-445-2515
Fax: 303-445-6351
E-mail: sschaefer@do.usbr.gov

Larry J. Schluntz
Bureau of Reclamation
7101 West Yale Avenue, #802
Denver, CO 80227
Telephone: 303-445-2901
Fax: 303-445-6470
E-mail: lschluntz@do.usbr.gov

Bruce Shank
Irrigation Association
P.O. Box 875
Acton, CA 93510
Telephone: 800-455-4320
E-mail: irricom1@earthlink.net

Yoshiyuki Shinogi
International Water Management Institute
127 Sunil Mawatha,
Battaramulla,
Sri Lanka
Telephone: 94-1-867-404
Fax: 94-1-866-854
E-mail: yshinogi@cgnet.com

Larry D. Simpson
Resource Management Services, Inc.
P.O. Box 268
Loveland, CO 80539
Telephone: 970-613-1933
Fax: 970-613-1943
E-mail: lsimpson@worldbank.org

Richard Soppe
Agricultural Research Service, USDA
2021 South Peach Avenue
Fresno, CA 93727
Telephone: 530-756-2678
Fax: 530-756-2678
E-mail: rsoppe@asrr.arsusda.gov

Tom Spofford
Natural Resources Conservation Service, USDA
101 SW Main Street, Suite 1600
Portland, OR 97204
Telephone: 503-414-3075
Fax: 503-414-3101
E-mail: tspofford@wcc.nrcs.usda.gov

Larry D. Stephens
U.S. Committee on Irrigation and Drainage
1616 Seventeenth Street, Suite 483
Denver, CO 80202
Telephone: 303-628-5430
Fax: 303-628-5431
E-mail: stephens@uscid.org

Stuart Styles
California Polytechnic State University
233 Stagecoach Road
Arroyo Grande, CA 93420
Telephone: 805-756-2429
Fax: 805-756-2433
E-mail: sstyles@calpoly.edu

Steve Sullivan
Maccaferri Gabions, Inc.
8433 North Black Canyon Highway, #100
Phoenix, AZ 85021
Telephone: 602-246-9071
Fax: 602-245-9075
E-mail: magawc@aol.com

Joseph B. Summers
Summers Engineering, Inc.
887 North Irwin Street
P.O. Box 1122
Hanford, CA 93232
Telephone: 209-582-9237
Fax: 209-582-7632
E-mail: jsummers@cnetech.com

Peter P.C. Sun
World Bank
1818 H Street, NW
Washington, DC 20443
Telephone: 301-469-7507
Fax: 301-365-7872
E-mail: psun@worldbank.org

Mark Svendsen
Consultant
24144 Price Creek Road
Philomath, OR 97370
Telephone: 541-929-4855
Fax: 541-929-4824
E-mail: msvendsen@compuserve.com

Bryan Thoreson
Davids Engineering, Inc.
1105 Kennedy Place, Suite 5
Davis, CA 95616
Telephone: 916-757-6112
Fax: 916-757-6118
E-mail: bryant@de-water.com

Gary Velder
Bureau of Reclamation
P.O. Box 226
Newell, SD 57760
Telephone: 605-456-2695
Fax: 605-456-2153

Carlos Villalon
Imperial Irrigation District
P.O. Box 937
Imperial, CA 92251
E-mail: czvillalon@iid.com

Tony L. Wahl
Bureau of Reclamation
P.O. Box 25007, D-8560
Denver, CO 80225
Telephone: 303-445-2155
Fax: 303-445-6324
E-mail: twahl@do.usbr.gov

Ivan A. Walter
W. W. Wheeler & Associates
6030 South Cherrywood Circle
Littleton, CO 80121
Telephone: 303-761-4130
Fax: 303-761-2802
E-mail: iacjwalter@worldnet.att.net

Jack B. Wergin
Bureau of Reclamation
RR1, 1706 West 3rd
McCook, NE 69001
Telephone: 308-345-4400
Fax: 308-345-6470

Dennis Wichelns
University of Rhode Island
Department of Resource Economics
Kingston, RI 02881
Telephone: 401-874-4565
Fax: 401-782-4766

John Wilkins-Wells
Colorado State University
Clark Building, #B258
Fort Collins, CO 80523
Telephone: 970-491-5635
Fax: 970-491-2191
E-mail: jwilkinswells@vines.colostate.edu

David S. Wilson, Jr.
Central Arizona Water Conservation District
23636 North 7th Street
Phoenix, AZ 85024
Telephone: 602-869-2333
Fax: 602-869-2332

James M. Wolf
Development Alternatives, Inc.
4811 Chippendale, #208/5
Sacramento, CA 95841
Telephone: 916-344-5345
Fax: 916-344-1611
E-mail: jim_wolf@dai.com

Ruth M. Wright
Attorney
1440 High Street
Boulder, CO 80304
Telephone: 303-443-8607
Fax: 303-473-9109
E-mail: rmwright@sni.net

Joel Zander
Bureau of Reclamation
2800 Cottage Way
Sacramento, CA 95825
Telephone: 916-978-5270
Fax: 916-978-5290
E-mail: jzander@mp.usbr.gov