PAP-594

ASSESSMENT OF THE WAPATO IRRIGATION PROJECT

By

R. A. Dodge

T. Vermeyen

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INTRODUCTION

Wapato Irrigation Project (WIP) is operated by the Bureau of Indian Affairs to provide irrigation water service to 150,000 acres of land in the Yakima Indian Reservation, Washington. Reclamation is working with WIP staff and the Yakima Indian Nation water resources planning staff as part of the Yakima River Basin Water Enhancement Project in addressing water conservation in terms of both structural and non-structural opportunities in the WIP service area. "Saved water" could be used for expanding irrigation acreage, for fish and wildlife, cultural uses, and other purposes on the reservation. A viable and accurate water measurement program is essential if maximizing the conservation objective is to be achieved.

The Hydraulics Branch was asked by the PN Region to assist the WIP with an assessment of their water measurement program. Tracy Vermeyen and Russell Dodge from the Hydraulics Branch, Research and Laboratory Services Division visited WIP from August 27 to 30, 1990. A tour of the Wapato-Satus Unit (the Ahtanum Unit and the Toppenish-Simcoe Unit were not reviewed) and discussions of its operation were conducted and are the basis of observations and suggestions contained in this report.

GENERAL OBSERVATIONS DURING INSPECTION

The present measurement system consists primarily of rated channel cross sections at 35 locations along the unlined canals, laterals and drains. Nine of 35 measurement stations are equipped with Leupold-Stevens type A-35 stage recorders. The balance of the stations are a combination of staff gauges or staff gauges in stilling wells and bridges for current metering the flow. In general, the stage recorders are only installed on natural channels used in the conveyance system.

On-farm delivery systems consist of single gate turnout boxes, of which, only a few are equipped with measuring devices to determine delivery rates. A typical farm delivery box is shown in Photo No 1.

The majority of the water supply for the Wapato-Satus Unit is obtained by diversion of Yakima River flows at the Wapato Diversion Dam into the Main Canal. There are four drop structures along the Main Canal. Flow through Drop-1 is monitored using a Marsh-McBirney Model 280 doppler flow meter. Drop-2 and Drop-3 contain hydroelectric generators, while Drop-4 is strictly an energy dissipation structure. The flow measurement systems for the turbines were not reviewed, but should be evaluated when Bureau of Reclamation personnel review the turbine-generator facility later this year.

Just downstream of each drop there is a major lateral turnout to convey water to lands lying east between the Main Canal and Wanity Slough which runs in a north and south direction. Five major drains also extend from the Main Canal in the same general vicinity to capture return flows from lands irrigated from the lateral system. For example, Drain-2 captures return flows from lands provided water by Lateral-1. Each lateral has an upstream measurement station that is equipped with a staff gauge, stilling well, and a current metering bridge, as shown in Photos No 2 and No 3. Discharge measurements are determined (using a Price AA current meter) after major flow changes in the canal system are implemented.

Three laterals (Track, Spencer, and Lateral-4 Extension) divert water from Wanity Slough, to irrigate lands lying to the east and extending to the Yakima River. Water supply for these lands is a combination of return flow and "feed" water from the Yakima River conveyed through the Main Canal, laterals, and drains to Wanity Slough.

Lateral-3 was cited by project personnel as the most troublesome to manage. They reported that small changes of delivery in the upstream part of the lateral restrict delivery at downstream turnouts.

Spring runoff and irrigation return flows originating from the laterals are conveyed through canals, drains, and natural channels to the Satus Basin. When necessary, during the drier part of the irrigation season, water is directly crossed over or fed to the drains to supplement supply available to the Satus-Unit.

RECOMMENDATIONS

General

It was concluded that the WIP measurement program could be improved through installation of standard measurement devices. Accurate measurement of lateral flows will improve management of the water delivery system. Individual user water consumption must also be measured to improve water delivery efficiency. A proposed plan for improving of the current program of water measurement is as follows:

Canals and Upstream End of Laterals

It is recommended that Ramp flumes with stilling wells, water level recorders, and electronic data loggers (contained in protective shelters) be installed in the upper ends of the canals and laterals. We recommend talking to Onni Perala and Steve Fanciullo from Yakima Projects Office (YPO) for assistance in instrumentation system selection and stilling well fabrication and installation. Additional information concerning capabilities and constraints of various electronic data loggers provided by the Yakima Projects Office is in Attachment No. 1. Track Lateral has sufficient drop (Photo No 4) to install either a rectangular or Cipolletti weir without submergence problems. Weirs should be installed and maintained as specified in Reclamation's Water Measurement Manual.

Drains and Laterals at Crossover and Feed Locations

Install and maintain permanent gauging stations on main drains, Wanity Slough and Toppenish Creek to replace those already in use. The installations should include stilling wells, water level recorders and data loggers, contained in protective shelters. Rating tables should be established for each installation and periodically re-rated to determine calibration shifts. A continuous discharge record will help in monitoring the supply of water available to the

Satus Basin. Accurate measurements of drainage flow are important for managing feed water delivery to supply the Satus-Unit, crop management and control of drainage water quality.

Pumping Plants

Improve flow measurement and discharge control systems at pumping plants so system demand can be achieved without over-pumping.

Individual Users

To maximize water conservation all individual water use needs to be measured. As new users are added their turnouts should include a suitable measuring device. As existing turnout structures are replaced, flow measurement devices should be included. Individual user measurement improvements should be implemented in stages working from the Satus Unit towards Wapato Diversion Dam. Starting with the Satus Unit will lower pumping costs and minimize operational problems encountered with initiating an overall water conservation program, as this unit is largely dependent on drainage flow for supply. An individual measurement program in the Satus-Unit could be used as a pilot study by the operators and as demonstration or part of a well planned water user training program prior to phasing in subsequent stages of the irrigation measurement upgrade program. Most turnouts observed could be easily modified to accommodate propeller meters to measure and totalize flow. Either fixed or portable propeller meters could be utilized for turnouts that flow full.

SOME REQUIREMENTS OF SUGGESTED MEASURING DEVICES

The material in this section is meant to be a basic short form discussion of the devices suggested above. Detailed information for these devices can be found in the Attachments Nos. 1 to 3. Material provided by the Yakima Project Office, Attachment No. 1, contains information concerning gauging stations in terms of stilling wells, recorders, data loggers and protective shelters. Besides detailed information concerning weirs and propeller meters, report REC-OCE-70-38 (Attachment No. 2) also contains information on other devices. The entire report was attached because it contains good advice on operating and maintaining water measurement devices.

Approach Channel

Good approach flow conditions are required to attain potential flow measurement accuracy of water measurement devices. To achieve this, sufficient length of straight approach conduit and channel is generally specified in terms of the required number of diameters, measuring heads, average approach channel widths, or control widths (throat width for flumes and crest length for weirs). The approach to a measuring device should be symmetrical, straight and free of bends and projections from the sides or bottom of the channel. For open channel devices, when the control width is equal to or greater than 1/2 the average approach width, then 10 channel widths of straight approach channel is required. If the control width is less than 1/2, then 20 control widths of straight approach is required. Downstream of highly turbulent or skewed flows such as a hydraulic jump, a minimum of 30 measuring heads of straight approach is recommended. Where these requirements can not be met, baffles or slats can be used to improve flow conditions and thereby compensate for poor approach conditions. A minimum of 10 measuring heads of straight approach are required between baffles and the measuring station.

Ramp Flumes

Ramp flumes provide an accuracy comparable to Parshall flumes (a potential accuracy of 3 to 5 percent) and are generally less costly to install in existing canals. Ramp flumes cause relatively small head loss and can be used to measure flows under high submergence (small water surface drop over the flume). Another advantage is that ramp flumes can be computer calibrated or recalibrated based on as-built flume dimensions. The main construction requirements are that the crest is of proper length and is level. The main calibration requirement is that all the dimensions, especially the crest width and side slopes of the crest and canal sections, be carefully measured after construction. The cost for large ramp flumes are about 45 to 60 percent of that for Parshall flumes. Cited costs for smaller flumes have been one-tenth to one-third of those for equivalent Parshall flumes. More information concerning ramp flumes is presented in Attachment No. 3.

Weirs

Weirs have a potential accuracy of about 1 percent. To attain good discharge measurements the weir geometry, installation and approach conditions must closely follow published guidelines as presented in Reclamation's Water Measurement Manual or other published standards. The velocity of approach must be less then 1/2 ft/s to use standard calibrations. The weir crest should be 2 measuring heads or more above the approach channel bottom and the side contraction should be at least 2-1/2 measuring heads from the sides of the approach channel. The measuring station should be 4 to 6 measuring heads upstream from the weir blade. To maintain the 2 measuring head crest height criteria may require periodic channel maintenance and cleaning out sediment deposits, especially between the measuring station and the weir blade. Flow over a weir can be totalized by adding a stilling well and a data logger to the measuring station.

Propeller Meters

Propeller meters have a potential accuracy of about 3 to 5 percent. The discharge accuracy is highly dependent on installation of the meter in a turnout that corresponds to the manufacturers specifications. The inside diameter of the pipe should be the same as specified for the meter being installed. If flow straightening vanes are recommended by the manufacturer, they should have the same geometry and relative location with respect to the propeller and turnout geometry as specified. In the field, provision may be needed to assure full submergence of propellers. The flow velocity should be greater than 1 ft/s. The propeller diameter should be from 0.5 to 0.8 of the pipe diameter. Many propeller meters measure total volume of water delivered and instantaneous flow rate for setting turnout deliveries.

Propeller meters require a maintenance routine where bearings are replaced based on time of operation. Maintenance frequency varies with water quality, and especially sediment load. Newer propeller meters generally have sealed bearings or ceramic bearings to minimize sediment wear problems. The Mechanical Branch of the Denver Office of Reclamation can be consulted regarding operation and maintenance experience of recent installations. Attachment No 2, references detailed information on propeller meter installations.

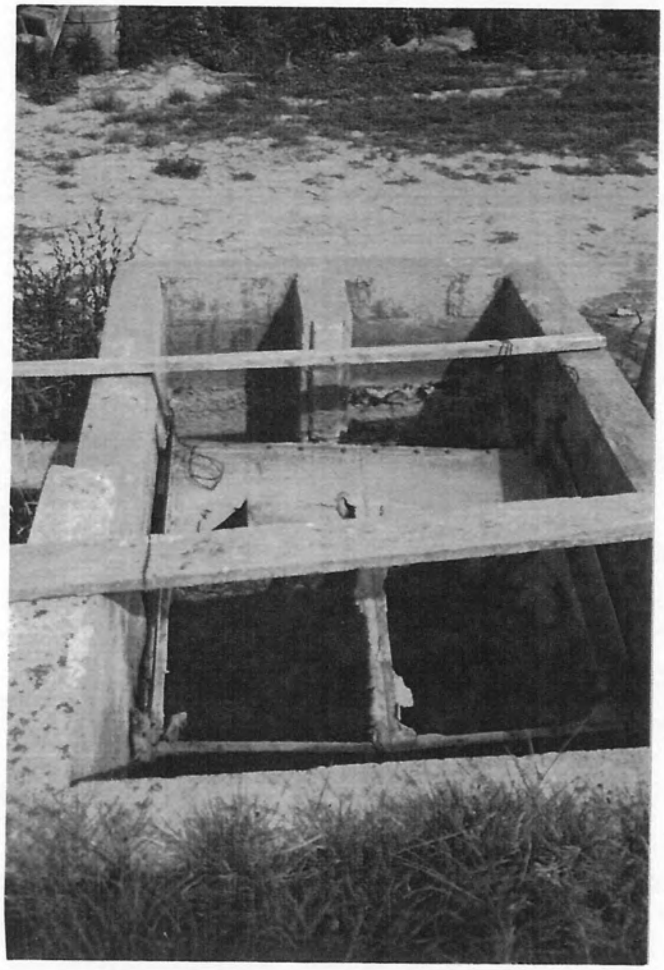


Photo No 1- A typical farm delivery box.

Photo No 2- General view of a bridge current meter rating section in Track Lateral

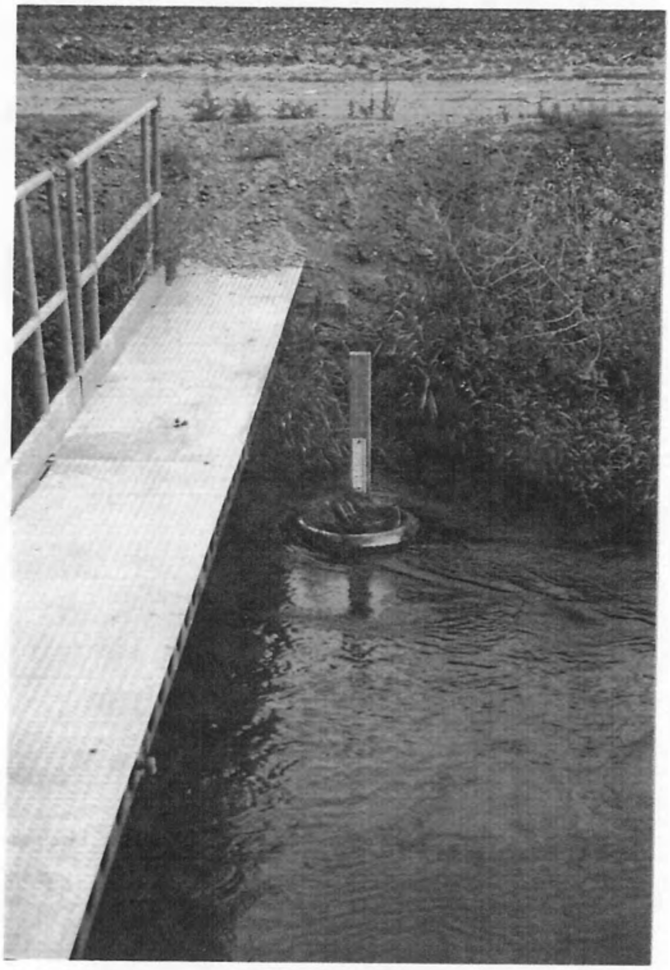


Photo No 3- A closeup view of a rating bridge and staff gage in a stilling well.

Photo No 4- Rated drop in Track Lateral that could be replaced with a weir.

Attachment No 1

INFORMATION CONCERNING DATA LOGGERS AND STREAM GAUGING

OFFICE MEMORANDUM
UNITED STATES GOVERNMENT, BUREAU OF RECLAMATION
YAKIMA PROJECT OFFICE

1700/PRJ-23.00 October 2, 1990

Blue Envelope

Memorandum

To: Russ Dodge and Tracy Vermeyen, E&R Center
Attention: D-3751

From: Onni J. Perala, Yakima, Wa

Subject: Streamgaging - Wapato Irrigation Project (Streamgaging)

This is a comparison of four configurations of equipment that might be used at sites on the Wapato Project. Costs can be cut to use equipment already on hand. However as we look at this proposal we need also consider expansion, how many sites may be instrumented and telemetered initially. Another key consideration is whether only instantaneous operating data is required or long term historical data as well. The technology is available to provide just about any informational need, however budget allotments and data processing time requirements will place constraints on how much can be accomplished.

Onni

cc: L. Vinsonhaler

Campbell Scientific

A

CR-10 data logger	940
ACRO encoder	250
power cable	(inc)
battery w/enc	247
solar charger	157
A35	1575
clock	369
pulley	58
tape (w/hooks & indicator)	94
counterweight + 10" float	78
stilling well (installed)	1500
hutch (installed)	750
	6018

Acro

B

ACRO data logger DL86	550
ACRO encoder ECOO	250
DMOO Data Module	200
power cable	(inc)
battery	80
solar charger	157
A35	1575
clock	369
pulley	58
tape (w/hooks & indicator)	94
counterweight + 10" float	78
stilling well (installed)	1500
hutch (installed)	750
	5661

Leupold - Stevens

C*

A/F datalogger	745
A/F encoder	280
Data Card	220
power cable	22
battery	80
solar charger	157
A-35	1575
clock	369
pulley	58
tape (w/hooks & indicator)	94
counterweight & 10" float	78
stilling well (installed)	1500
hutch (installed)	750
	5928

Use Leupold Stevens Gear
w/o strip chart
ie - leave out the A-35

D*

A/F datalogger	745
A/F encoder	280
Data Card	220
power cable	22
battery	80
solar charger	157
———	
———	
pulley	58
tape (w/hooks & indicator)	94
counterweight & 10" float	78
stilling well (installed)	1500
hutch (installed)	500
	3734

A, B, & C, interface with an A-35 strip chart recorder

* prices for loggers, encoders, and peripherals do not include discounts. Discounts of 10, 16, 20 percent can be realized based on volume. This equipment is on GSA contract.

Option A:

provides readout at site
signal to telemetry - to transmit stage to master station
provide strip chart for visual display of stage change

Options B & C:

provides all of the above plus
-module which can be interfaced to a PC to get all gage heights per day - for period recorded. This gives the capability to enter the data automatically into a data base ie without keyboard entry.

Option D:

Provides all options of A,B,C <u>except</u> the strip chart recorder - all records and workup can be done from data on module which can be downloaded to a PC. The strip chart recorder, while giving visual display of stage change is labor intensive for record workup - ties up personnel time, uses charts, paper, and manual posting and keyboard entry.

Datalogger Support Products for IBM® PC's and Compatibles

Campbell Scientific, Inc. (CSI) supports communication between IBM® PC's (and compatibles) and CSI dataloggers with PC208 Software, PC201 Card/Software and the PC203 Power-up Control Box. These products facilitate the following datalogger operations:

- Data retrieval
- Remote monitoring of measurements and/or status
- Datalogger program development and editing
- Telecommunications over direct cable, radio frequency, multidrop or switched telephone links
- Data handling and report generation
- Remote programming

These products are designed for IBM PC's and compatible computers. IBM PS/2 computers run PC208 Datalogger Support Software, but do not accept the PC201 Clock-Serial I/O Tape Read Card. (Please refer to Minimum Requirements section - last page).

PC208 Datalogger Support Software

The PC208 Software Package consists of six separate programs; each supports various aspects of an integrated data acquisition system. PC208 programs assist the user in performing the following functions:

- EDLOG - Developing and editing datalogger programs
- SPLIT - Analyzing data and generating reports
- SMCOM and SMREAD - Retrieving data from Storage Modules
- TERM - Remotely downloading programs and monitoring datalogger measurements at the PC via telecommunication links
- TELCOM - Retreiving data over telecommunication links

EDLOG - Datalogger Program Editor

EDLOG is used to develop and document programs for CSI's CR10, 21X and CR7 dataloggers. Full editing features allow the user to insert, delete, move, copy and mark program instructions. In-line documentation is provided and additional comments may be inserted. A "help" function assists the user in the selection of program instructions and parameter options. Alphanumeric labels can be assigned to Input Locations to aid in program readability, program debugging and assessment of incoming data.

SPLIT - Data Split, Merge and Report Generation

SPLIT is a general purpose data reduction program that operates on data produced by any of CSI's dataloggers. SPLIT can select data from one or more files, process and/or combine the data, and generate a titled report with labeled data columns. Limits can be specified for each column of data; out-of-range elements are flagged. Report files are configured to be compatible with popular spreadsheet programs; the presence of a DOS command line allows SPLIT to run in a DOS batch file mode.

SMCOM and SMREAD - Storage Module Data Retrieval

SMCOM establishes communication with the SM192/716 Storage Modules for data retrieval and storage on disk. It also enables two-way transfer of datalogger programs between PC and Storage Module for transfer to/from the datalogger site. SMCOM prompts the user for the required setup information such as COM port and root collection file name. The user is then presented with a menu of options for data collection, program storage and keyboard entry of Storage Module commands. Data download to the PC can be largely automated through batch file operation.

SMREAD retrieves and stores data from the SM64 Storage Module.

Both SMREAD and SMCOM require either the SC532 Peripheral Interface or the PC201 Card to interface Storage Modules to the PC.

PC208 Programs continued on next page. . .

PC208 Datalogger Support Software
(continued)

TERM and TELCOM provide datalogger-PC communication over a variety of links. The modems and other interface devices required by the various communication links are listed in the table below. Datalogger type, interface option and baud rate are specified by the user in a Station File which is accessed by TERM and TELCOM during auto-dialing.

TERM - Terminal Emulator

TERM is a terminal emulator program that contains functions specialized to datalogger-PC communication. TERM contains ten user-selectable modes; each mode controls a different aspect of datalogger-PC interaction. TERM provides a mechanism for remote keyboard entry, datalogger program upload/download, and datalogger monitoring. The presence of a DOS command line allows TERM to run in a DOS batch file mode. A noteworthy TERM option is Monitor mode:

Monitor Mode - allows instantaneous appraisal of conditions at the datalogger site. Monitor mode interrogates the datalogger and automatically updates the screen with values from any 33 of the first 254 Input Locations. Incoming data from each Input Location can be labeled. Monitor mode also displays (and allows user control of) the status of datalogger control ports and program flags; in short, it allows a user to both monitor and control a remotely located experiment or production process.

TELCOM - Telecommunications Controller

TELCOM automates data retrieval from CSI dataloggers over the various communication links. Data is collected in blocks; error checking assures data integrity. Prompts help the user create a Station File for each datalogger station. This file specifies the communication link(s), destination disk file, data format (ASCII or binary), and interrogation interval (e.g. every 24 hours at 2:00 a.m.). Incoming data are appended to the destination file. DOS commands or other programs can be executed under TELCOM control.

The user can (1) initiate a call manually by running TELCOM and specifying the Station File name, or (2) execute TELCOM in the unattended mode on a predefined schedule. In unattended operation, TELCOM controls the computer, calls stations sequentally, and retries failed or interrupted calls.

TELCOM can also answer incoming calls while in the unattended mode. If called by another computer or terminal, TELCOM executes a user-written batch file to establish communication with the calling device. If called by a datalogger, data is downloaded.

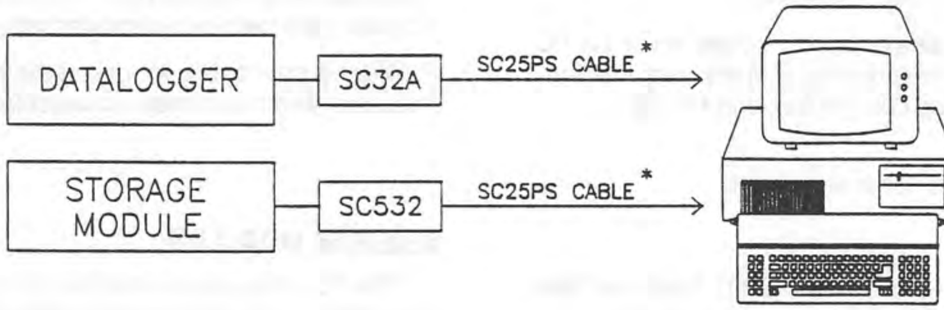
Telecommunications Interface Devices

Link*	Calling End (PC)	Remote End (datalogger)
Switched Telephone	Hayes Smartmodem or compatible	DC112 Telephone Modem
Radio Frequency	RF232 RF Base Station	RF95 RF Modem
Coax Cable (up to 3 miles)	MD9 Multidrop Interface and SC532 Peripheral Interface or PC201 Card	MD9 Multidrop Interface
4-wire unconditioned telephone line (two twisted pairs)	SRM-6A RAD® Short Haul Modem	SRM-6A RAD® Short Haul Modem with SC932 9-pin to RS232 DCE Interface
Direct Cable (short indoor distances)§		SC32A Optically Isolated RS232 Interface

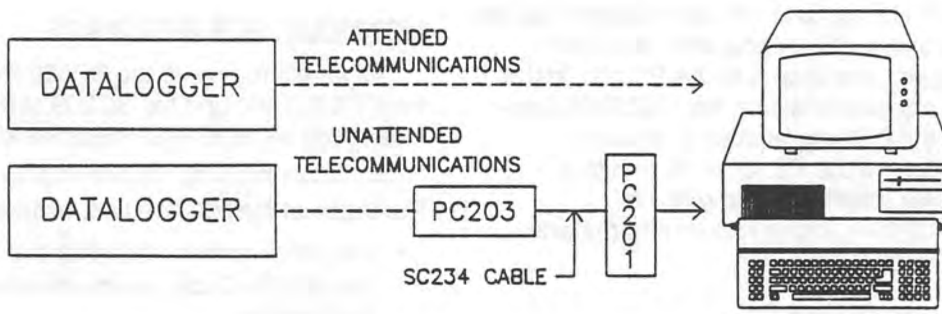
* Consult factory for combinations of communication interfaces.

§ Less than 100 ft., distance varies with the PC and its RS232 voltage levels.

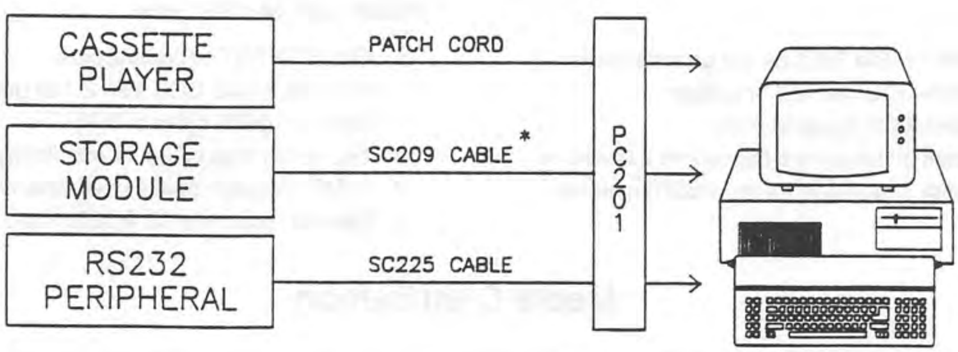
Data Retrieval and Telecommunications Options



Direct communication with datalogger and Storage Module via PC208 Software.



Attended and unattended Telecommunications via PC208 Software.
Appropriate Telecommunications Interface Devices and additional cabling required (see facing page).



Data retrieval from Cassette Tape and Storage Module, or interface to RS232 Peripheral via PC201 Card and Software.

*Some cables must be purchased separately

PC201 Clock - Serial I/O Tape Read Card and PC201 Software

- Retrieves data from cassette tape
- Automates the power-up and shutdown of the PC for unattended telecommunications (requires the PC203 Power-up Control Box and PC208's TELCOM)
- Provides an additional serial port.
- Retrieves data from SM192/716 or SM64 Storage Modules (alternative method described below)
- Provides a connection for use of the MD9 Multidrop Interface (alternative method described below)

TAPE

The primary function of the PC201 Card and Software is to retrieve data from cassette tape. The TAPE program prompts the user for the information required to read the cassette then stores the data in a specified disk file. TAPEWR is used to write a file to cassette tape, e.g., a datalogger program. Both the current (FORMAT II) and original (FORMAT I) CSI tape formats are accommodated.

AUTOMATED DATA RETRIEVAL

The PC201/203 configuration is used to power up the PC at designated times, collect and store data from specified dataloggers, and then turn the PC off. In the event of a temporary power failure, the PC201/203 delays powering up the PC long enough to ensure proper reinitialization. If the PC locks up due to a static glitch or power transient while running TELCOM☆, the PC201/203 shuts down and restarts the PC to restore normal operation.

ADDITIONAL SERIAL PORT

The PC201 Card contains an Asynchronous Communications Adapter which can be configured as COM 1,2,3 or 4. The Card also contains a battery-powered clock that sets the PC clock on power-up.

STORAGE MODULES

The PC201 Card and the SC209 Cable (ordered separately) are used to retrieve data from the SM192/716 and SM64 Storage Modules and also to download/upload datalogger programs to/from the SM192 and SM716. Storage Module functions can be accomplished without the PC201 Card via the SC532 Peripheral Interface and a standard 25-pin ribbon cable connected directly to a PC serial port. In either case, SMCOM or SMREAD provides software support.

MD9 MULTIDROP INTERFACE

As an alternative to the SC532 Peripheral Interface, the PC201 Card and the SC209 cable (ordered separately) can be used to connect the MD9 to the PC.

Purchase of the PC201 Card includes:

- one patch cord for cassette playback
- one SC225 Cable, used with standard 25 pin peripherals
- two AAA batteries
- an Operator's Manual
- one PC201 software disk

☆ **TELCOM is not included in PC201 Software.**

Minimum Requirements

CSI supports operation of its PC products ONLY where 100% IBM compatibility has been verified.

PC208 Software

- IBM PC/XT/AT or IBM PS/2 Series or compatibles
- PC-DOS or MS-DOS Ver. 2.1 or greater
- Recommended 640K bytes of RAM
- Two floppy disk drives or one floppy and a hard disk
- 652K bytes disk space used when PC208 installed

PC201 Card and Software

- IBM PC/XT/AT or compatibles
- PC-DOS or MS-DOS Ver. 2.1 or greater
- Minimum 256K bytes of RAM
- Two floppy disk drives or one floppy and a hard disk
- 115K bytes disk space used when PC201 installed
- One full length slot for PC201 Card

Media Distribution

PC208 (3 disks) and PC201 (1 disk) Software are distributed on 5-1/4", 360Kb disks. CSI also provides PC208 and PC201 Software on 3-1/2", 720Kb disks if requested at time of order.

CAMPBELL SCIENTIFIC, INC.

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FAX 01144509674928

Printed Apr

U.S. PRICE LIST

Effective April 3, 1989

Model		Description	Price
		CR10 MEASUREMENT AND CONTROL MODULE AND ACCESSORIES	
CR10	*	Measurement & Control Module w/ CR10WP Wiring Panel [Customer must specify PROM OS10-0, OS10-1, or OS10-2 when ordering]	$ 990.00
CR10KD	*	CR10 Keyboard & Display	250.00
EM10-64	*	64K RAM (29,900 Data Values)	100.00
10TCRT		Thermocouple Reference-Thermistor	25.00
AVW1		1 Channel Vibrating Wire Sensor Interface	150.00
AVW4		4 Channel Vibrating Wire Sensor Interface	320.00
SDC99		9 Pin Synchronous Communication Interface	135.00
020/ALK		Enclosure w/ Alkaline Power Supply	215.00
020/LA		Enclosure w/ Sealed Rechargeable Battery & Charger	275.00
021/ALK		Enclosure w/ Alkaline Power Supply, Mounts, & Radiation Shield	285.00
021/LA		Enclosure w/ Sealed Rechargeable Battery, Charger, Mounts, & Radiation Shield	350.00
020/EXT		Enclosure for use w/ External Power Supply	185.00
021/EXT		Enclosure w/ Mounts & Radiation Shield for use w/ External Power Supply	260.00
10ALK/C		C Cell Battery Pack for CR10	60.00
		21X MICROLOGGER AND ACCESSORIES	
21X	*	MICROLOGGER w/ 40K RAM (19,296 Data Values) [Customer must specify PROM OSX-0, OSX-1, or OSX-2 when ordering]	$ 1800.00
21XL	*	MICROLOGGER w/ Sealed Rechargeable Battery & Charger	1900.00
SP21X		Spare Parts Kit	250.00
021		21X Enclosure w/ Radiation Shield & Mounts	190.00
020		21X Enclosure Only	110.00
022		21X Large Enclosure w/ Radiation Shield & Mounts	270.00
022RF		Same as 022 w/ Mounts for RF Telemetry Peripherals	300.00
028		21X Large Enclosure Only	185.00
		MEASUREMENT AND CONTROL PERIPHERALS	
SDM-A04		4 Channel Continuous Analog Output Module	$ 350.00
SDM-CD16		16 Channel Control Port Module	370.00
SDM-INT8		8 Channel Interval Timer	375.00
SDM-SW8A		8 Channel Pulse Counter	210.00
A21REL-12		4 Channel Relay Driver (12VDC Coil)	130.00
A6REL-12		6 Channel Relay Driver w/ Manual Override (12VDC Coil)	280.00
AM416		16 Channel 4 Wire Input Multiplexer w/o Enclosure	475.00
AM-ENC		Multiplexer Enclosure	135.00
AM-ENCT		Thermally Insulated Multiplexer Enclosure	250.00
QD1		Incremental Encoder Interface	98.00
		CR7 MEASUREMENT & CONTROL SYSTEM AND ACCESSORIES	
700X	*	Control Module w/ 40K RAM (Enclosure Required)	$ 1700.00
706X	*	24K Memory Card	175.00
720	*	I/O Module (Enclosure or Mount Required)	1900.00
720XL	*	Large I/O Module (Special Mounting Required)	2270.00
723	*	Analog Input Card	400.00
723-T	*	Analog Input Card w/ RTD	525.00
724	*	Pulse Counter Card	325.00
725	*	Excitation Card	400.00
726	*	50 Volt Full Scale Analog Input Card	425.00

* **2 Year Warranty**

CAMPBELL SCIENTIFIC, INC.
P.O. Box 551 • Logan, Utah 84321 • (801) 753-2342 • TLX 453058 • FAX 801-752-3268

Printed June 1989

Model		Description	Price
		CR7 MEASUREMENT & CONTROL SYSTEM AND ACCESSORIES CONT.	
SC94		4 Wire Current Loop Interconnect for use w/ Remote I/O Modules (Lead Cost $0.17/ft.)	$ 260.00
A3497		TC Psychrometer Cooling Current Interface	60.00
S3497X		Psychrometer Software for 700X	75.00
ENC-7L		Aluminum Enclosure for Laboratory use	360.00
ENC-7F		Environmentally Sealed Fiberglass Case w/ 2 Ports Fitted w/ 3/4 inch Dia. Conduit Bushings (**)	560.00
		(**) Connector Option:	
A3536		Two Complete 19-pin Connectors w/ Sealing Glands	125.00
		TRIPODS AND INSTRUMENT MOUNTS	
CM6		Instrument Tripod & Grounding Kit (6 ft.)	$ 245.00
CM10		Instrument Tripod & Grounding Kit (10 ft.)	245.00
019		Crossarm Sensor Mount	50.00
015		Pyranometer Mounting Arm	70.00
025		Pyranometer Crossarm Stand	30.00
41004-5		12 Plate Gill Radiation Shield	150.00
41301-5		6 Plate Gill Radiation Shield	85.00
		DATA STORAGE AND RETRIEVAL ☆	
Communications Software & Hardware			
PC201		Clock-SIO Tape Read Card & Software for IBM-PC®	$ 500.00
PC203		Power-up Control Box	255.00
PC208		Datalogger Support Software	200.00
Direct Line/Display			
SC32A		Optically Isolated RS232 Interface	$ 130.00
DSP4		Heads Up Display	1850.00
Multidrop			
MD9		Coax Multidrop Interface	$ 235.00
MD9 CT		Coax Terminator (pair)	15.00
Radio			
RF95		RF Modem (Radio Telemetry)	$ 300.00
P50 VHF	†	5 Watt VHF Transceiver (Requires Antenna & Cables)	350.00
P50 UHF	†	5 Watt UHF Transceiver (Requires Antenna & Cables)	400.00
RF232		RF Base Station	550.00
Short Haul			
SRM-6A		RAD® Short Haul Modem (2 Required)	$ 100.00
SC932		9 Pin to RS232 DCE Interface	130.00
5563		4 Wire Surge Protector	15.00
Storage Module			
SM192	*	Solid State Storage Module (96,000 Data Values)	$ 620.00
SM716	*	Solid State Storage Module (358,000 Data Values)	1530.00
SC90		9 Pin Serial Line Monitor	35.00
SC532		9 Pin Peripheral to RS232 Interface	175.00
Tape			
C20		Cassette Interface	$ 960.00
RC35		Cassette Recorder	110.00
RC35P		Playback Recorder (Includes Patchcord & AC Charger)	125.00
SC92A		Cassette Write Only Interface	70.00
SC93A		Cassette Read/Write Interface	105.00
Telephone			
DC112		Telephone Modem	$ 300.00
2372-01		Joslyn Telephone Surge Protector	10.00

(Call or write for Satellite Transmission Pricing)

☆ Standard Datalogger - Peripheral Connection cables are included

† Consult CSI for assistance on antennas and cables.

*** 2 YEAR WARRANTY**

Model	Description	Lead (*)	Price
	SENSORS		
Temperature			
107	Temperature Probe (-35° to 50°C)	(.17*)	$ 38.00
107B	Temperature Probe (-35° to 50°C; Suitable for Burial)	(.25*)	40.00
108	Temperature Probe (-5° to 95°C)	(.17*)	42.00
108B	Temperature Probe (-5° to 95°C; Suitable for Burial)	(.25*)	44.00
TCR-6	Thermocouple Reference Junction		380.00
105T	Type T Thermocouple Probe (Suitable for Burial) w/ 5 point calibration from -78° to 100°C	(.30*)	20.00
A3537	Type T Thermocouple Wire (Suitable for Burial) Plus $10 Handling Charge per Length		0.32/ft.
TCAV	Averaging Soil Thermocouple Probe (2X2) [20 ft. Lead]	(.50*)	90.00
FWTC-3	Type E Fine Wire Thermocouple (0.003 inch dia.)		60.00
FWTC-1	Type E Fine Wire Thermocouple (0.001 inch dia.)		60.00
FWTC-05	Type E Fine Wire Thermocouple (0.0005 inch dia.)		65.00
FWTC/ENC	Carrying Case (holds 4 probes)		20.00
FWTC-C	Connector Cable for Fine Wire Thermocouple [20 ft. Lead]	(.50*)	20.00
Temperature and RH			
207	Temperature & RH Probe, 10-95% (Requires 41004-5 Radiation Shield or Equivalent)	(.20*)	$ 200.00
PCRC-11	RH Replacement Chip for 207 Probe		65.00
XN217	Hygrometrix[1] Temperature & RH Probe (Requires 41301-5 Radiation Shield or Equivalent)	(.20*)	400.00
Precipitation			
TE525	Texas Electronics[2] Tipping Bucket Rain Gage [25 ft. Lead]		$ 190.00
Wind Speed and/or Direction			
014A	Met One[3] Wind Speed Sensor	(.17*)	$ 275.00
024A	Met One[3] Wind Direction Sensor	(.20*)	395.00
05103	RM Young[4] Wind Monitor	(.50*)	595.00
03101-5	RM Young[4] Wind Sentry Anemometer	(.17*)	135.00
03001-5	RM Young[4] Wind Sentry Wind Set	(.37*)	300.00
CA27	Sonic Anemometer w/ Case & w/o Thermocouple [25 ft. Lead]	(.30*)	2100.00
27S	Sonic Transducer Spare		75.00
127	Fine Wire Thermocouple for CA27 (0.0005 inch dia.)		95.00
	Repair Charge for 127 Thermocouple		25.00
Solar Radiation			
LI200S	LI-COR®[5] Silicon Pyranometer	(.17*)	$ 190.00
LI2003S	LI-COR®[5] Pyranometer Base & Leveling Fixture (for use w/LI200S & LI190SB)		35.00
LI190SB	LI-COR®[5] Quantum Sensor	(.17*)	350.00
Q-5	REBS[6] Net Radiometer (Fritschen) [20 ft. Lead]	(.17*)	490.00
Barometric Pressure			
SBP270	Setra[7] Barometric Pressure Sensor [3 ft. Lead]	(.50*)	$ 1350.00
Soil Moisture			
227	Soil Moisture Block [20 ft. Lead]	($10 plus .17*)	$ 40.00
223	Soil Moisture Block w/o Blocking Capacitors (for use w/ AM416) [20 ft. Lead]	($10 plus .17*)	7.00
Miscellaneous			
237	Wetness Sensing Grid [25 ft. Lead]	(.17*)	$ 50.00
ACL1	Opto-Isolated AC Line Monitor		50.00
KH20	Krypton Hygrometer w/ Case [25 ft. Lead] (Source & Detector Tubes warranted for 90 Days)	(.32*)	3900.00
	KH20 Calibration		200.00
HFT-1	REBS[6] Soil Heat Flux Plate [20 ft. Lead]	(.17*)	155.00
SGB19	Dynagage™[8] Stem Flow Gage 3/4 inch 0.5 Watt [5 ft. Lead]		310.00
SGA10	Dynagage™[8] Stem Flow Gage 3/8 inch 0.1 Watt [5 ft. Lead]		265.00

(*) Price per foot of lead wire in excess of standard lengths. Ten ft. leads provided unless noted in brackets. Longer leads stocked in 25, 50, & 100 ft. lengths except for 227.

[1] Hygrometrix, Inc., Oakland, CA; [2] Texas Electronics, Inc., Dallas, TX; [3] Met One, Inc., Grants Pass, OR; [4] RM Young Company, Traverse City, MI; [5] LI-COR is a registered trademark of LI-COR, Inc., Lincoln, NE; [6] REBS is Radiation Energy Balance Systems, Seattle, WA; [7] Setra Systems, Inc., Acton, MA; [8] Dynagage is a Trademark of Dynamax, Inc., Houston, TX

Model	Description	Price
	SOLAR PANELS, POWER SUPPLIES, AND TRANSIENT PROTECTION	
MSX5	5 Watt Solar Panel & Mounts	$ 125.00
MSX10	10 Watt Solar Panel & Mounts	165.00
MSX10R	10 Watt Solar Panel with Mounts & Regulator	230.00
PS34	12 Volt Charging Regulator	130.00
PS35	12 & 5 Volt Charging Regulator	155.00
BP10	10 Amp Hour Battery Pack	110.00
036	20-Conductor J-Box w/ Transient Protection	210.00
038	40-Conductor J-Box w/ Transient Protection	325.00
	DATA CABLES	
SC12	Two Peripheral Connector Cable for Dataloggers	$ 20.00
SC25PS	RS232 Ribbon Cable w/ Pins to Pins/Socket	25.00
SC209	PC201 - Storage Module Connector Cable	25.00
SC225	34 Rectangular Socket to 25 Pin "D"	40.00
	SUPPLIES	
AV-D60	Cassette Tape (30 min. per side/case of 10)	$ 25.00
DSC 50/2	Desiccant (50 ea./.7 oz pack)	20.00
H3-10	Spark Gap Kit (set of 10)	25.00
	Datalogger Manual for Replacement or Evaluation	25.00
	Datalogger Service/Repair Documentation	25.00
	(Some documentation requires a signed Non-Disclosure Agreement prior to release)	

PRICES: Prices are subject to change without notice.

WARRANTY POLICY: CSI warrants products manufactured by CSI to be free from defects in materials and workmanship under normal use and service for twelve (12) months from date of shipment unless specified otherwise, subject to the following conditions:

CSI's obligation under this warranty is limited to repairing or replacing (at CSI's option) products which have been returned prepaid to CSI. CSI will return warranted equipment by surface carrier prepaid. This warranty shall not apply to any CSI products which have been subjected to modification, misuse, neglect, accidents of nature, or shipping damage. Batteries are not warranted. Under no circumstances will CSI reimburse the claimant for costs incurred in removing and/or reinstalling equipment. This warranty and CSI's obligation thereunder, is in lieu of all other warranties, expressed or implied, including warranties of suitability and fitness for a particular purpose. CSI is not liable for consequential damages.

FACTORY REPAIR: Products may NOT be returned without prior authorization (RMA) by the factory. Repair charges: $40.00/hour (minimum charge $50.00).

QUANTITY DISCOUNTS: Consult the factory for quantity discounts on orders of more than ten units.

PREPAYMENT REBATE POLICY: Effective on orders of $2000.00 or more, a REBATE of 5% is offered if full payment is received before the order is shipped. Notice must be given of intent to prepay. Freight and insurance will be prepaid and deducted from the rebate amount. The rebate will be made at time of shipment.

MINIMUM ORDER: Orders for less than $50.00 must be accompanied by a check. Please include $10.00 for handling.

PAYMENT TERMS: Net 30 days on approved credit; all other orders accepted with cash in advance or shipped C.O.D. Application for credit requires 1 bank reference, 2 trade references, and a current balance sheet.

SHIPPING POLICY: All shipments are F.O.B. Logan, Utah with freight and insurance prepaid and added to the invoice as a separate item.

15% RESTOCKING FEE: Unused products may be returned only with prior written authorization and must be resaleable as new. Radios and antennas may not be returned.

FACTORY AND ORDERING ADDRESS: CAMPBELL SCIENTIFIC, INC.
P.O. BOX 551
LOGAN, UT 84321
PHONE: (801) 753-2342
TELEX: 453058
CAMPBELL LOGA
FAX: 801-752-3268

CAMPBELL SCIENTIFIC, INC.
P.O. Box 551 • Logan, Utan 84321 • (801) 753-2342 • TLX 453058 • FAX (801) 752-3268

DC112 MODEM

PRELIMINARY SPECIFICATIONS

FEATURES

- Bell 212A and CCITT V.22 Compatible
- Full Duplex at 300 and 1200 Baud
- "AT" Command Set
- RJ-11C Telephone Jack
- Pulse or Tone Dialing
- Direct Connection to and powered by:

  CSI Dataloggers
  CSI PC201 Card in IBM Compatible Computer
  CSI PC203 Power-up Control Box
  CSI SM232A RS232 Interface
- Signal Level Connects/Disconnects 5Vdc External Power
- Operational from -25° C to +50°C
- Size: 4 3/8" x 3" x 1"
- Weight: 12 oz.
- Current Drain: Approximately 2 uA when modem enable is low; 35 mA when modem enable is high but phone is on-hook; 48 mA when off-hook.

DESCRIPTION

The DC112 Modem is a 300/1200 baud Hayes compatible modem. It employs the popular "AT" command set. Its primary use is as a remote site modem connected to a CSI datalogger. The modem is powered and enabled by the battery-powered datalogger. When disabled, the DC112 draws less than 2 uA from the datalogger 5 Vdc output.

CAMPBELL SCIENTIFIC, INC.
P.O. Box 551 • Logan, Utah 84321 • (801) 753-2342 • TLX 453058 • FAX (801) 752-3268

SM192 AND SM716 STORAGE MODULES

Preliminary Specifications

STORAGE CAPACITY

SM192 - 192,000 bytes; 96,000 Final Storage locations
SM716 - 716,000 bytes; 358,000 Final Storage locations

PACKAGING

Sealed, stainless steel canister. Dimensions: 7.8"x 3.5"x 1.5".

POWER REQUIREMENTS

5 VDC supplied by the datalogger on pin 1 of the 9-pin connector.
Typical current drain: when active and processing - 18 ma
active but not processing - 3 ma
standby state ("asleep" but still connected to datalogger) - 250 ua

OPERATIONAL TEMPERATURE -25°TO +60°

MEMORY BACKUP POWER

3.5 VDC lithium thionyl chloride battery. Battery life is temperature - dependent; 6 years @ 25 deg C, 4 years @ -25 deg C, and 2 years @ 50 deg C.

MEMORY CONFIGURATION

User selectable for either ring-style or fill-and-stop memory.

FILE MARK

A File Mark is automatically placed in the data when the Storage Module is first connected to a datalogger or upon command from the CR10 when it compiles a program. The user can place a File Mark in the data in the *9 Mode in the CR10.

BAUD RATE

21X, CR7X, DSP4 Mode - 76,800 or 9,600 baud, detected by the Storage Module.

CR10 Mode - 9,600 baud unless directed otherwise by the CR10.

NOTE: The rate at which the Storage Module can store continuous (no pause between transmissions) data is 5,600 bytes per second.

DATA RETRIEVAL

Retrieval of data from the Storage Module can be accomplished either:

1) automatically with the PC201 card, the SC532 Peripheral Interface or the SM232A Storage Module Interface and the program SMCOM or
2) through the use of telecommunications commands.

Attachment No 2

WATER MEASUREMENT PROCEDURES
IRRIGATION OPERATORS WORKSHOP
REC-70-38

Weirs pages 5 to 12

Propeller meters pages 34 to 41

REC-OCE-70-38

WATER MEASUREMENT PROCEDURES IRRIGATION OPERATORS' WORKSHOP

Edited by
J. C. Schuster

September 1970

Hydraulics Branch
Division of General Research
Engineering and Research Center
Denver, Colorado 80225

UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION
Walter J. Hickel
Secretary

PREFACE

This report contains a condensation of the lecture notes "Water Measurement Procedures" accumulated by A. J. Peterka between 1961 and his retirement in 1968 for the "Irrigation Operators' Workshop" conducted by the Bureau of Reclamation in Denver, In the early years, the notes were made available only to Workshop participants. However, being of general interest and because of numerous requests, the notes were reproduced as a Bureau of Reclamation Report Hyd-577 to make them accessible to a greater number of readers. The following report is a revision of Hyd-577. Most of the section on Parshall flumes was excluded because the information has recently been included in the Water Measurement Manual, Second Edition, published by the Bureau. A section has been added to this report on the use of radial gates for water measurement. Information on "Special Measuring Devices and Techniques" was modified to indicate recent trends in water measurement. Other sections of the report have been revised slightly to include ways and means of improving methods and devices for measuring flow. With the exception of these revisions, the notes are essentially the same as those written originally by A. J. Peterka.

FOREWORD

This report has information that can be used by operators of irrigation systems for making meaningful and accurate measurements of water. Information from the report has been used in previous workshop sessions that began in 1961.

After a discussion of the need for improved measuring devices and techniques, standard and nonstandard devices are defined and their implications in regard to water measurement are discussed.

Water-measuring devices and methods are classified under three categories: (1) the velocity device, (2) the head device, and (3) miscellaneous devices, including chemical and dye dilution methods, total count radioisotope methods, magnetic methods and sonic methods.

Under "Some Basic Hydraulics," the concepts of the discharge equation and velocity head are developed. These two concepts are used to derive the basic equations for both orifice and weir discharge using the simplest methods possible. Several of the general aspects of water-measurement accuracy are also discussed.

The procedures for operating flumes and weirs, probably the most common devices, are used to furnish examples of good and poor water-measurement practices because the effects of good and bad measurement practices are often not visible on some of the more sophisticated measuring devices.

The more complicated devices and techniques such as the submerged orifice meter, Venturi meter, metergate, and constant-head orifice turnout measuring device, and propeller meters are then described.

Hints are given for troubleshooting metering devices suspected of being inaccurate and instructions are given for selecting the proper size and obtaining proper installation of metergates, constant-head orifice meters, and Parshall flumes.

Progress in water measuring techniques, including the chemical dilution and radioisotope methods is reported, and an evaluation of a commercially available open-channel deflection meter is given. Progress in the development of magnetic and acoustic meters is reported.

The section on Parshall flumes brings together under one cover the essentials needed to understand the size selection, vertical placement, discharge determination procedures for free and submerged flows, and the theory of flume operation and performance. Sample problems are used to illustrate proper procedures.

A list of reference material and a chart showing the head required to operate certain measuring devices are presented.

CONTENTS

CONTENTS—Continued

LIST OF FIGURES

LIST OF FIGURES—Continued

NEED FOR IMPROVING WATER-MEASURING DEVICES AND TECHNIQUES

It is impossible at the moment to overstate the need for an improved water-measuring program, not only in your district, project, or region, but nationwide, and even worldwide. The population explosion and the shifting of the population center of the United States toward the West is causing concern in that water needs may eventually limit this movement. According to the latest figures, if we account for births, deaths, immigrants, and emigrants, a new person arrives on the scene every 10 seconds; 8,000 a day; 3,000,000 a year. By the end of the century the population will have doubled. Twice as many people—the same amount of water. Within the next decade, every water supply, not only those in the West but throughout the United States, will be critically examined to determine quantity, use, and waste. Plans will then be formulated for extending the use of the water.

One way to increase the quantity of available water, of course, is to find new water sources. This is not always possible and is usually costly. The other way is to conserve and equitably distribute the water now developed and available. This latter course is usually possible and less expensive. We are interested in the latter course because this is our business and, as all of you know, more extensive use of the available water can be made if all measuring devices are accurate and dependable at all times and the best water-management procedures are followed at all times. It is, therefore, important that every attempt be made to upgrade existing water-measuring devices, not only in ditches, laterals, and canals, but in the supply systems as well. Every cubic foot of water "saved" as a result of improving the measuring devices is more valuable than a similar amount obtained from a new source because the saved water can be produced at considerably less cost.

Accuracy in water measurement is therefore of prime importance in the operation of any water-distribution system. Even though a surplus of water may now exist on your project, it is time to begin accurate accounting of what happens to your water. An experience on a project actively engaged in upgrading water-measurement devices and procedures will serve as a good example. A water user entitled to 1 cfs (cubic feet per second) had been receiving up to 5 cfs because water had been plentiful and no accurate measuring device had been installed at his turnout. When a new meter was installed and 1 cfs was delivered, he complained that his new concrete field-distribution system would not operate properly and much of his acreage would have to go without water until he could change the elevation and slope of his ditches.

This actual occurrence illustrates another point which is important in water measurement. Practically all measuring devices, when in rundown condition or when improperly installed, deliver more water than they indicate they are delivering. The very nature of most measuring devices makes it impossible for a device to deliver less water than it indicates. For this reason, water accounting records may not show a proper division between water used and water lost through seepage or waste. Proper evaluation of losses is necessary to establish the economic advisability of providing canal linings. Canal linings obviously cannot help to recover water lost through poor measuring equipment or procedures.

The purpose of this presentation is to discuss the factors in flow-measurement devices which affect the accuracy of discharge measurement. To accomplish this, it is necessary to understand the basic flow principles involved and to know how each flow factor can influence the flow quantity indicated. By upgrading existing measurement devices or by properly installing and maintaining new devices, considerable quantities of water can be made available for new uses. This "new" water can be produced at considerably less cost than can a similar quantity be developed from a new source.

STANDARD AND NONSTANDARD DEVICES

It has been said that a waterlogged boot which is partially blocking the flow in a ditch can be a measuring device—if it is properly calibrated. Certainly the boot would be a nonstandard device because no discharge tables or curves are available from which to determine the discharge. Many other devices including certain weirs, flumes, etc., are also nonstandard because they have not been installed correctly and, therefore, do not produce standard discharges. Although these commonly used devices may appear to be standard devices, closer inspection often reveals that they are not, and like the boot, must be calibrated to provide accurate measurements.

A truly standard device is one which has been fully described, accurately calibrated, correctly installed, and sufficiently maintained to fulfill the original requirements. Standard discharge tables or curves may then be relied upon to provide accurate water measurements.

Any measuring device, therefore, is nonstandard if it has been installed improperly, is poorly maintained, is operated above or below the prescribed limits, or has poor approach (or getaway) flow conditions. Accurate discharges from nonstandard structures can be obtained only from specially prepared curves or tables based on calibration tests as current meter ratings.

Calibration tests can be quite costly when properly performed. Ratings must be made at fairly close discharge intervals over the complete operating range, and curves and/or tables prepared. It is, therefore, less costly and usually not too difficult to install standard devices and maintain them in good condition. Standard discharge tables may then be used will full confidence.

In maintaining a standard structure it is only necessary to visually check a few specified items or dimensions to be sure that the measuring device has not departed from the standard. In maintaining a nonstandard device it is difficult to determine by visual inspection whether accuracy is being maintained except by recalibration, an expensive procedure.

BASIC PRINCIPLES OF WATER MEASUREMENTS

To upgrade existing water-measuring devices and improve the quality of installation of new devices, it is necessary to understand some of the basic principles which influence the quantity of water passed by a measuring structure. Most devices measure discharge indirectly; i.e., velocity or head is measured directly and prepared tables or an indicator are used to obtain the discharge. Measuring devices may be classified, therefore, in two groups: (1) velocity type and (2) head type. Those using the velocity principle include:

Float and stopwatch	Vane deflection meters
Current meters	Magnetic and acoustic meters
Propeller meters	Salt velocity method
Flow boxes	Color velocity method

When the velocity (V) principle is used, the area of the stream cross section (A) must be measured and the discharge (Q) computed from $Q = AV$.

Devices using the head principle include:

Pitot tubes
Rectangular weirs
V-notch or multiple-notched weirs
Cipoletti weir
Parshall flume, Venturi flume
Metergates
Orifice or Venturi meters
Constant-head turnouts
Clausen weir gage (or stick)

When the head (H) principle is used, the discharge (Q) may be computed from an equation such as the one used for a sharp-crested rectangular weir of length (L),

$$Q = CLH^{3/2}$$

The area of the cross section (A) does not appear directly in the equation but C, a coefficient, does. C can vary over a wide range in a nonstandard installation, but it is well defined for standard installations.

Special methods and devices may also be used and these include dilution methods which utilize chemicals or radioisotopes, acoustic or magnetic meters, tapered-tube and float devices, and many others which are not commonly used. In the dilution method the discharge is determined by calculating the quantity of water necessary to dilute a known quantity of concentrated chemical, dye, or radioisotope solution, injected into a flowing stream, to the strength obtained by sampling the stream after thorough mixing has taken place. Chemical analysis, color comparison, or gamma-ray counting may be used to determine the degree of dilution of the injected sample. In the acoustic meter the variation in velocity of a sound pulse produced by the water velocity is used to determine the average water velocity of the flow. In the magnetic meter the flowing water disturbs the lines of force in a magnetic field to produce a voltage that can be related to the discharge. Pitot tubes and tapered-tube and float devices utilize the velocity head to indicate velocity.

SOME BASIC HYDRAULICS

Derivation of Discharge Equation

The volume of a cube, Figure 1, may be found by multiplying a times b times c

a times b gives the area A
$2 \times 2 = 4$ sq ft

A times c gives the volume
$4 \times 2 = 8$ cu ft

The volume of a cylinder (or pipe) may be similarly found. If the A for the cylinder is 2 sq ft and l is 4 ft then

area, A, times l gives the volume
$2 \times 4 = 8$ cu ft

Instead of using l, as before, substitute the velocity, V, Figure 2, and,

Discharge = Area x Velocity

or

$$Q = AV$$

Since "Area" units are in square feet and "Velocity" units are in feet per second

Q = square feet times feet per second
Q = cubic feet per second

Therefore, the discharge Q in cubic feet per second is equal to the Area of the flow cross section in square feet times the Velocity in feet per second. The relationships between A and V in a pipeline of varying cross section are shown in Figure 3.

The first basic equation is

$$Q = AV \tag{1}$$

Derivation of Velocity Head Concept

It has been found by direct measurement that an object (including water) falling from rest will fall 16.1 feet in the first second. Because of the continuing acceleration caused by gravity (32.2 feet per second per second), the object will fall 64.4 feet in 2 seconds, etc., as shown in Table 1.

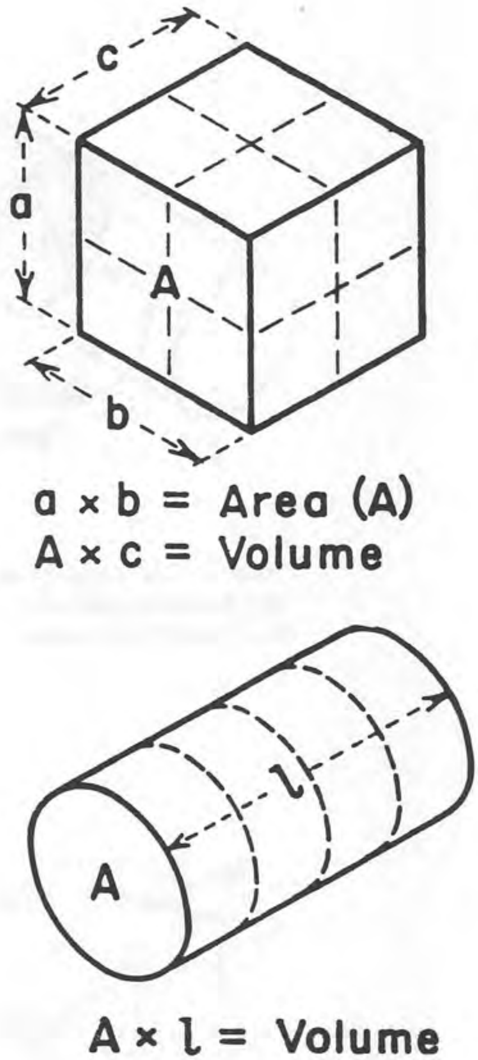


Figure 1. Definition of volume.

Table 1

Time in seconds	Verticle fall feet	Instantaneous velocity
1	16.1	32.2
2	64.4	64.4
3	144.9	96.6
4	257.6	128.8
5	400.0	161.0

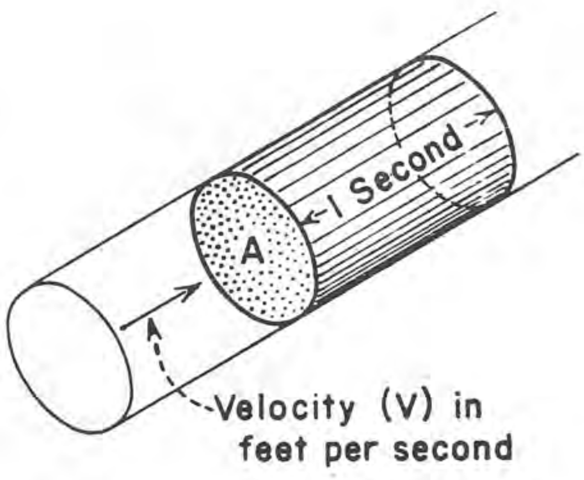


Figure 2. The measurement of discharge is actually a volume measurement and can be calculated in the same way using different terms.

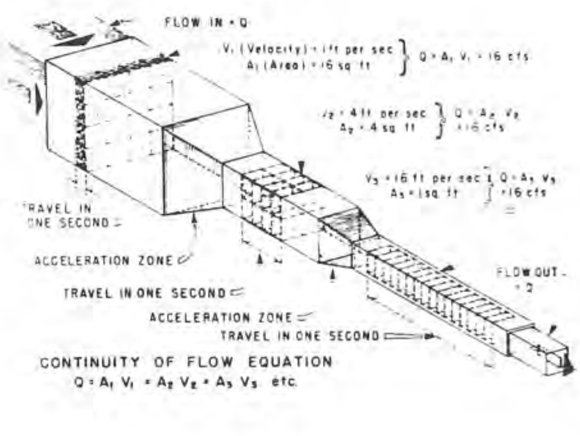


Figure 3. Continuity of flow equation.

At the end of the first second the velocity will be twice the vertical fall distance (because the object started from rest at zero velocity) or 32.2 feet per second, Table 1.

Since the acceleration caused by the steady pull of gravity, g, is constant, the velocity values in Table 1 increase 32.2 for each succeeding second.

An equation to express these facts is

$$V = \sqrt{2gH} = \sqrt{2g}\sqrt{H} \qquad (2)$$

where g is the acceleration due to gravity and H is the height of fall.

Substituting values from Table 1

$$V = 8.02\sqrt{16.1} = 32.2$$

$$V = 8.02\sqrt{64.4} = 64.4, \text{ etc.}$$

The equation is therefore seen to be valid.

After squaring both sides, Equation 2 becomes

$$V^2 = 2gH \text{ or } H = \frac{V^2}{2g} \qquad (3)$$

The latter expression is often used to express velocity head, i.e., the head necessary to produce a particular velocity. It is discussed further under "Velocity of Approach," page 8.

Basic Orifice Relationship

Equations 1 and 2 can be used to develop an equation for the flow through an orifice (a hole in the side or bottom of a container of water, Figure 4).

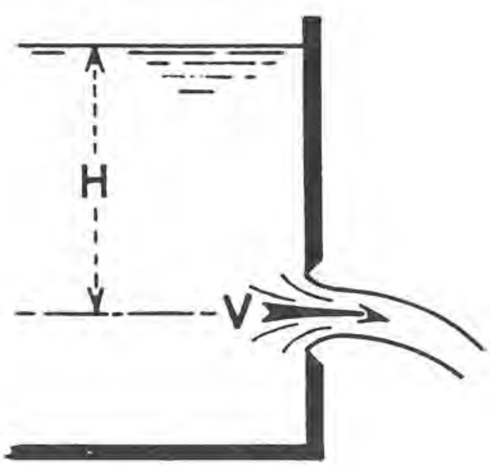


Figure 4. Flow from an orifice.

To find the velocity of flow in the orifice, use Equation 2, $V = \sqrt{2gH}$. Then using the size or area of the hole, the quantity of water being discharged, Q, can be determined using Equation 1, $Q = AV$. This can be accomplished in one step, however, if the two equations are combined.

Inserting the value $\sqrt{2gH}$ for V in Q = AV gives

$$Q = A\sqrt{2gH} \qquad (4)$$

The equation for the theoretical discharge through an orifice. This equation assumes that the water is frictionless and is an ideal fluid. Since water is not an ideal fluid, a correction must be made.

The diameter of the water jet continues to contract after passing through the orifice, and if the orifice edges are sharp, the jet will appear as shown in Figure 5.

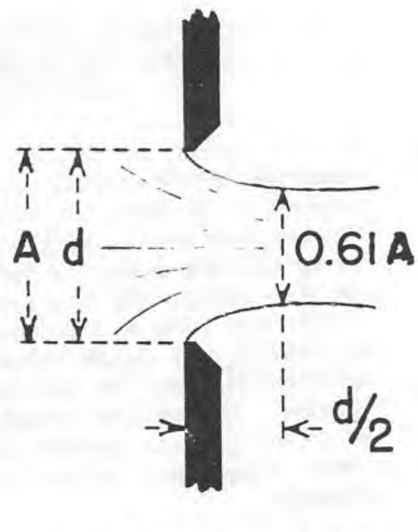


Figure 5. Contraction at an orifice.

To explain, at one-half of the orifice diameter downstream from the sharp edge, d/2, the maximum jet contraction will occur and the cross-sectional area of the jet of water will be only about 6/10 of the area of the orifice, A. The maximum velocity also occurs at this point, and so Equation 4 must contain a coefficient "C" (0.61 for an orifice in a large container) to determine the quantity of water being discharged.

$$Q = CA\sqrt{2gH} \qquad (5)$$

For a coefficient of 0.61, the equation would become

$$\begin{aligned} Q &= 0.61\ A\sqrt{2g}\sqrt{H} \\ &= 0.61\ A\ 8.02\sqrt{H} \\ &= 4.89\ A\sqrt{H} \end{aligned}$$

The value 0.61 should not be used indiscriminately; however, calibration tests establish the proper value for a particular orifice under a given set of conditions.

Basic Weir Relationship

Equations 1 and 2 can also be used to develop an equation for flow over a weir (a sharp-edged blade that measures discharge in terms of overflow depth or head, Figure 6).

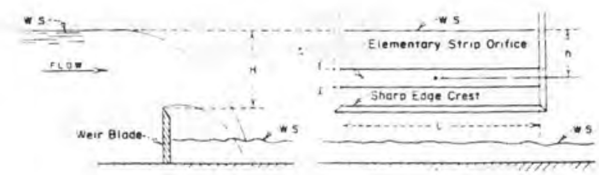


Figure 6. Weir flow.

The weir flow is divided into small elementary horizontal strips and each is considered to be a long, narrow rectangular orifice of length (L), height (t), and a flow producing head (h). The area (a) of each strip is

$$a = tL$$

The velocity (B) of each element is expressed by Equation 2:

$$V = \sqrt{2gh}$$

Since q = VA by Equation 1, the discharge (q) of each element is:

$$q = tL\sqrt{2gh}$$

To obtain the total discharge (Q) over the weir, the sum of all elementary discharges (q) must be taken. For example, the flow is divided into two elementary strip orifices, Figure 7.

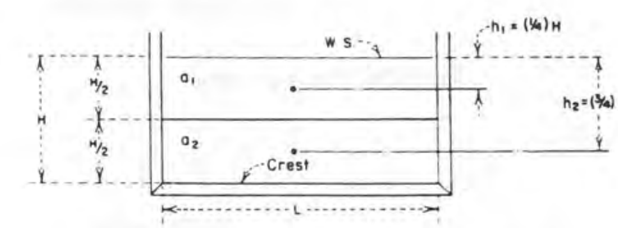


Figure 7. Definition sketch for weir flow.

By Equation 2,

$$V_1 = \sqrt{2gh_1} = \sqrt{2g\left(\frac{1}{4}\right)H}$$

$$V_2 = \sqrt{2gh_2} = \sqrt{2g\left(\frac{3}{4}\right)H}$$

and the areas of the elements are equal and expressed by

$$a_1 = a_2 = L\left(\frac{H}{2}\right)$$

By Equation 1 and addition, the total discharge (Q) over the weir is given by

$$Q = a_1V_1 + a_2V_2$$

$$Q = L\left(\frac{H}{2}\right)\sqrt{2g\left(\frac{1}{4}\right)H} + L\left(\frac{H}{2}\right)\sqrt{2g\left(\frac{3}{4}\right)H}$$

Taking $(\sqrt{2g} \cdot LH\sqrt{H})$ out of each expression on the right-hand side:

$$Q = \left(\frac{1}{2}\sqrt{\frac{1}{4}} + \frac{1}{2}\sqrt{\frac{3}{4}}\right)(\sqrt{2g} \cdot LH\sqrt{H})$$

and

$$\left(\frac{1}{2}\sqrt{\frac{1}{4}} + \frac{1}{2}\sqrt{\frac{3}{4}}\right) = K = 0.683$$

As more and more strips are used in the analysis K approaches the value 0.667. Therefore,

$$Q = \frac{2}{3}\sqrt{2g} \cdot LH\sqrt{H}$$

Introducing a constant (C') to account for contraction and friction losses, a more accurate discharge formula is:

$$Q = \frac{2}{3}\sqrt{2g} \cdot C' \, LH\sqrt{H}$$

By regrouping and letting

$$C = \frac{2}{3}\sqrt{2g} \cdot C'$$

$$Q = CLH\sqrt{H}$$

or

$$Q = CLH^{3/2} \qquad (6)$$

This relationship is the basic weir formula and can be modified to account for weir blade shape and velocity of approach. However, C must be determined by calibration tests carefully conducted. C usually varies from about 3.3 for a broad-crested weir to about 3.8 or more for a sharp-crested weir.

The two examples given indicate that discharge determination methods are a mixture of rationalized thinking and coefficient evaluation. The equations are useful in making calibrations because they reduce the number of calibration points required to make up a discharge table. However, the equations alone will not suffice without sufficient testing to establish the value of C.

GENERAL ASPECTS OF WATER MEASUREMENT ACCURACY

In this portion of the text general aspects of water-measurement accuracy will be considered. The performance of weirs and flumes will be used to illustrate flow and accuracy principles because the average irrigation operator is more familiar with their use. Also, many of the factors which adversely affect accuracy are visible on these devices and are invisible on closed-system devices. Many of the facts and principles established for weirs and flumes also apply to other water-measuring devices.[1 2] These principles will be mentioned and elaborated upon later when the more sophisticated meters and techniques are considered.

Flow Characteristics Reducing Accuracy of Measurement

In inspecting a water-measuring station to judge or evaluate its probable accuracy, it should be determined whether the device is a head- or velocity-measuring station. This is the key to the order and importance of other observations. In either case, the first observation should concern the visible flow conditions just upstream from the measuring device.

Approach Flow

Extremely large errors in discharge indication can occur because of poor flow conditions in the area just upstream from the measuring device. In general, the approaching flow should be the same as tranquil flow in long straight canals (without obstructions) of the same size. Any deviation from a normal horizontal or vertical flow distribution, or the presence of water-surface boils, eddies, or local fast currents, is reason to suspect the accuracy of the measuring device. Errors of 20 percent are not uncommon and may be as large as 50 percent or more, if the approach flow

[1 2] Numbers refer to References at end of report.

conditions are very poor. Sand, gravel, or sediment bars submerged in the approach channel, weeds or riprap obstructions along the banks or in the flow area can cause unsymmetrical approach flow. Other causes may be too little distance downstream from a drop, check, turnout or other source of high velocity or concentrated flow, a bend or angle in the channel just upstream from the measuring device, a too rapid expansion in the flow section, or an eddy tending to concentrate the flow cross section.

Figure 8 shows an example of a poor approach to a weir. The high-velocity, turbulent stream is approaching the weir at a considerable angle. Head measurement is difficult because of the high-velocity approach flow and the waves on the surface. This weir will not discharge a "standard" quantity of water consistent with the measured head.

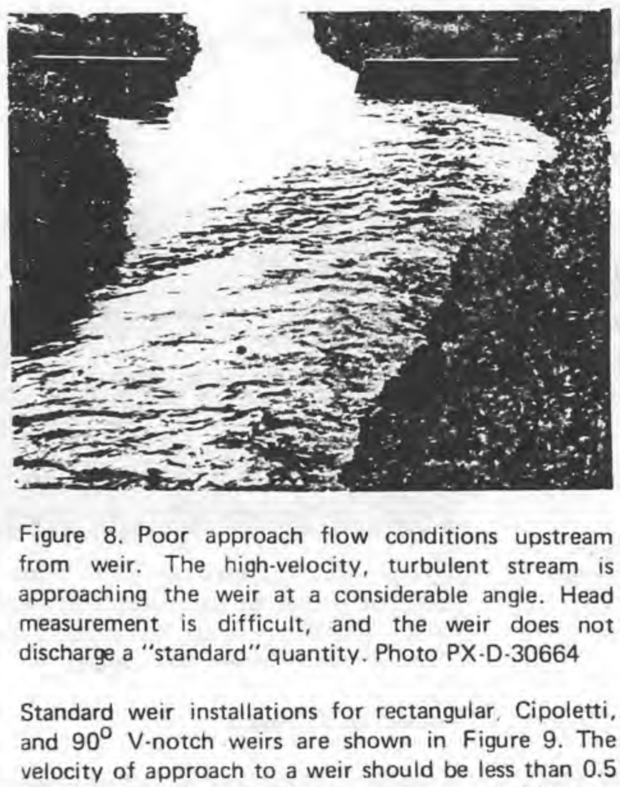


Figure 8. Poor approach flow conditions upstream from weir. The high-velocity, turbulent stream is approaching the weir at a considerable angle. Head measurement is difficult, and the weir does not discharge a "standard" quantity. Photo PX-D-30664

Standard weir installations for rectangular, Cipoletti, and 90° V-notch weirs are shown in Figure 9. The velocity of approach to a weir should be less than 0.5 foot per second. This value is obtained by dividing the maximum discharge by the product of channel width B and depth G measured at a point 4H to 6H upstream from the blade.

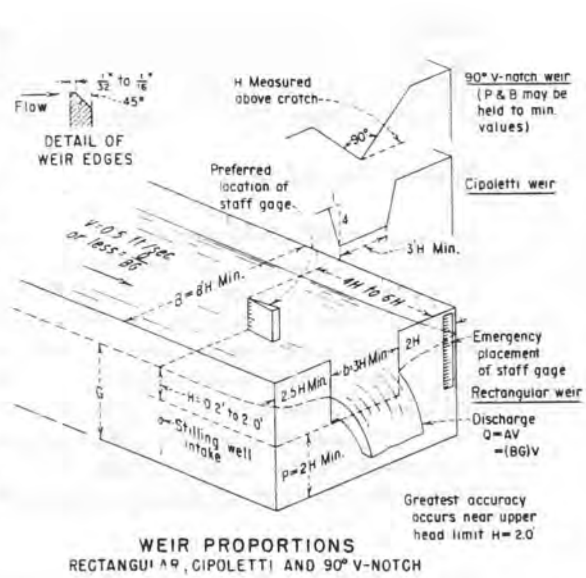


Figure 9. Weir proportions.

Turbulence

Turbulence is the result of relatively small volumes of water spinning in a random pattern within the flow mass as it moves downstream. It may be recognized as water-surface boils or three-dimensional eddies which appear and disappear in a haphazard way. Because of this local motion within the general motion of the flow mass, any particle of water may at any given instant be moving forward, sideways, vertically, or even backward. In effect, then, the water is passing a given point with a start and stop motion rather than with a uniform velocity which is ideal. It may be said that turbulent water does not flow as a train of railroad cars on a level track, but rather as a train of cars coupled with elastic bands, traveling over a series of rises, dips, and horizontal curves. Thus, fewer or more cars may pass a given point over identical short periods of time, depending on the observation point chosen. Turbulent water flows in the same manner, Figure 8.

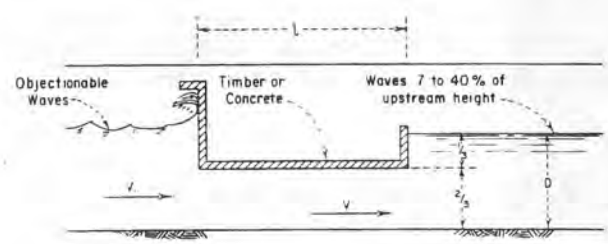
Excessive turbulence will adversely affect the accuracy of any measuring device, but is particularly objectionable when using current meters or propeller meters of any kind. Turbulence can be objectionable even without the "white water," caused by air entrainment, which is often associated with it. Turbulence is usually caused by a stilling basin or other energy dissipator immediately upstream, by a sudden drop in water surface or by obstructions in the flow area such as operating or nonoperating turnouts having projections or indentations from the net area. Shallow flow passing over a rough or steep bottom can also be the cause. Weeds or riprap slumped into the flow area or along the banks, or sediment deposits upstream from the measuring device also can cause excessive turbulence.

Measuring errors of up to 10 percent or more can be caused by excessive turbulence and it is absolutely necessary that all visible signs of turbulence be eliminated upstream from a measuring device.

Rough Water Surface

A rough water surface, other than wind-generated waves, can usually be eliminated by reducing turbulence or improving the distribution of the approach flow. A rough water surface can cause errors in discharge measurements when it is necessary to (1) read a staff gage to determine head, or (2) determine the cross-sectional area of the flow. A stilling well will help to reduce errors in head measurement, but every attempt should be made to reduce the water-surface disturbances as much as possible before relying on the well.

Errors of 10 to 20 percent are not uncommon if a choppy water surface makes it impossible to determine the head accurately. It is sometimes necessary to resort to specially constructed wave-damping devices to obtain a smooth water surface. Figure 10 shows a schematic of an underpass type of wave suppressor successfully used in both large and small channels.



LENGTH L	PERCENT WAVE REDUCTION
1D TO 1.5D	60 TO 75
2D TO 2.5D	80 TO 88
3.5D TO 4.0D	90 TO 93

Figure 10. Underpass wave suppressor.

The channel may be either rectangular or trapezoidal in cross section. Waves may be reduced as much as 93 percent by constructing the suppressor four times as long as the flow is deep. A slight backwater effect is produced by the suppressor for the most effective vertical placement. The suppressor may be supported on piers, can be constructed of wood or concrete, and need not be watertight. The design of several other types of suppressors, along with sample problems, is covered in Engineering Monograph No. 25, available through the Engineering and Research Center, Denver, Colorado. Figures 11 and 12 (before and after) show the effectiveness of an underpass wave suppressor at a Parshall flume measuring station.

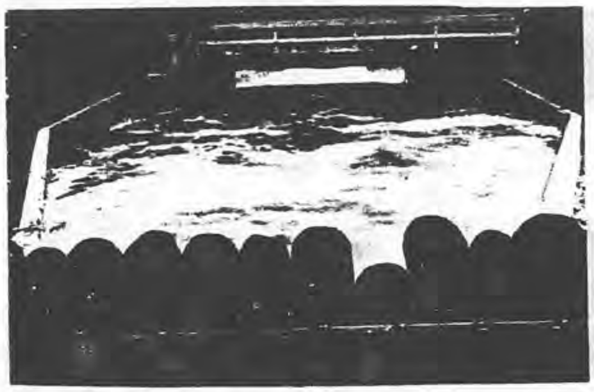


Figure 11. Turbulence and waves in a Parshall flume produced by an outlet works stilling basin made accurate discharge determination impossible. Log raft in foreground, used in futile attempt to quiet the flow, is inoperative. Photo P245-D-30666

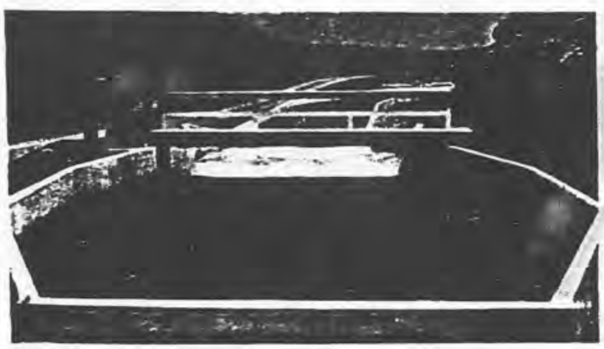
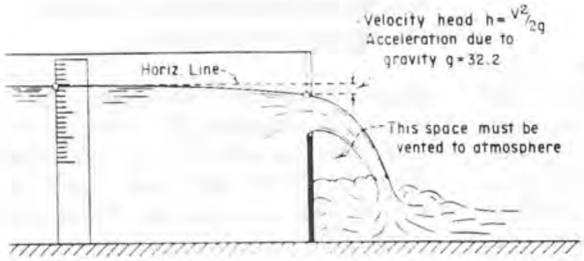


Figure 12. Underpass-type wave suppressor significantly reduces turbulence and waves in Parshall flume, making accurate discharge determination a routine matter. Photo P245-D-30663

Velocity of Approach

It can be observed that as flow approaches a weir, the water surface becomes lower on a gradually increasing curve, Figure 13.

At the weir blade, the water surface is considerably lower than, say, 5 feet upstream. The difference in elevation between the two circled points on the surface of the approach flow is called the velocity head and represents the potential required to produce the increase in velocity between the points. The



SECTION ON LONGITUDINAL CENTERLINE

Figure 13. Weir approach flow.

relationship between head (h) and velocity (V) is expressed as

$$h = \frac{V^2}{2g}$$

g is the acceleration due to gravity, 32.2 (feet per second per second).

A drop in water surface of 0.1 foot is not uncommon just upstream from a weir and (from the equation above) represents an increase in velocity of 0.8 foot per second. If the head on the weir is measured too close to the weir, the head measurement can be 0.1 foot too small. For a weir 6 feet long and discharging 7 cfs, the corresponding error in discharge would be about 35 percent, based on an indicated or reported discharge of 5.1 second-feet.

Standard weir tables are based on the measured head on the weir (velocity had negligible) and do not compensate for excessive velocity head. Any increase in velocity above standard conditions, therefore, will result in measuring less than the true head on the weir and more water will be delivered than is measured.

Causes of excessive velocity head include (1) too shallow a pool upstream from the weir, (2) deposits in the upstream pool, Figure 14, and (3) poor lateral velocity distribution upstream from the weir, Figure 8.

Poor Flow Patterns

It is often found that the poor flow distribution which exists upstream from a measuring device cannot be resolved on the basis of any one of the above-discussed causes. The best solution, then, is to assume that several or more basic causes have together caused the difficulty. Starting with the easy factors, work through the list, improving each probable cause of poor flow patterns until the desired flow conditions are obtained.

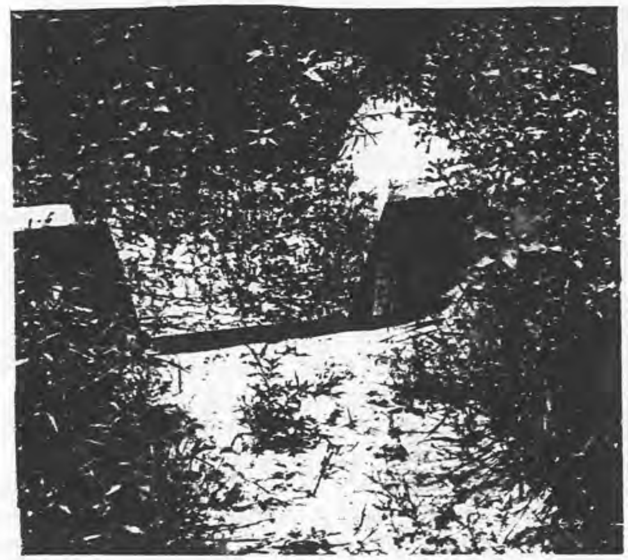


Figure 14. Sediment deposits have reduced the depth of the weir pool sufficiently to increase the velocity of approach to well above the desirable level. The head gage should not be located close to the weir blade. The weeds should be removed and the "edge" of the weir should be sharp. Discharges over this weir will be larger than indicated in "standard" tables. Photo P20-D-21558

Operating or nonoperating turnouts located just upstream from a measuring device may cause poor approach conditions as may bridge piers, channel curves, or a skewed measuring section. Relocating the measuring device may be the only remedy in these cases.

Submerged weeds or debris can cause excessive turbulence or local high-velocity currents. Eddies adjacent to the shoreline can cause the flow approaching a weir to contract into a narrow band. Sediment bars deposited from inflow or from sloughing banks can also produce undesirable flow conditions. More drastic remedial measures include deepening the approach area, widening the approach channel to make it symmetrical, or introducing baffles or other devices to spread the incoming flow over the entire width of the approach. Surface waves are usually very difficult to reduce or eliminate by ordinary procedures. These may require special treatment, as discussed under "Rough Water Surface."

Exit Flow Conditions

Exit flow conditions can cause as much flow-measurement error as some of the approach flow problems. However, in practice, these conditions are seldom encountered. In general, it is sufficient to be sure that backwater does not or tend to submerge a device designed for free flow. Occasionally, a Parshall flume is set too low and backwater submerges the throat excessively at high discharges. Extremely large errors in discharge measurement can be introduced in this manner. The only remedy is to raise the flume, unless some local obstruction downstream can be removed to reduce the backwater. Weirs should discharge freely rather than submerged, although a slight submergence (the backwater may rise above the crest up to 10 percent of the head) reduces the discharge a negligible amount (less than 1 percent). Whenever a weir operates at near submergence, the operation should be checked. Submergence may not affect the discharge as much as the possible lack of nappe ventilation as a result of the rising backwater.

The underside of weir nappes should be ventilated sufficiently to provide near atmospheric pressure beneath the nappe, between the undernappe surface and the downstream face of the weir, Figure 13.

If the nappe clings to the downstream side of the weir (does not spring clear) the weir may discharge 25 percent more water than the head reading indicates. An easy test for sufficient ventilation is to part the nappe downstream from the blade for a moment with the hand or a shovel, to allow a full supply of air to enter beneath the nappe. After removing the hand or shovel, the nappe should not gradually become depressed (over a period of several or more minutes) toward the weir blade. If the upper nappe profile remains the same as it was while fully ventilated, the weir has sufficient ventilation.

When the head on a straight weir is about an inch or less, the weir may not give reliable discharge values unless the weir has been calibrated under exactly similar flow conditions. On V-notch weirs, reliable results may not be obtainable for heads of 2 inches or even 3 inches.

Gates calibrated for free discharge at partial openings should not be submerged nor should eddies interfere with the jet of water issuing from the gate. Gaging stations should be kept free of deposited sediment bars or other obstructions to prevent backflow or eddies from interfering with the uniform flow conditions which should exist in the cross section being measured.

Equipment Characteristics Reducing Accuracy of Measurement

Measuring devices themselves may be at fault in producing measurement errors rather than the flow conditions discussed in the previous section. The faults may be divided into two types—those caused by normal wear and tear, and those resulting from poor installation.

Weathered and Worn Equipment

An unwelcome but fairly common sight on older irrigation systems are weir blades which were once smooth and sharp, in a sad state of disrepair. Edges are dull and dented. The blade is pitted with large rust tubercles—weir plates are discontinuous with the bulkheads and are not vertical. Weir blades have sagged and are no longer level. Staff gages are worn and difficult to read. Stilling well intakes are buried in sediment or partly blocked by weeds or debris. Parshall flumes are frost heaved and out of level. Meter gates are partly clogged with sand or debris and the gate leaves are cracked and warped.

These and other forms of deterioration are often the causes of serious errors in discharge measurements. This type of deficiency is difficult to detect because normal wear and tear may occur for years before it is apparent to a person who sees the equipment frequently. On the other hand, it is readily apparent to an observer viewing the installation for the first time.

It is imperative, therefore, that the person responsible for the measuring devices inspect them with a critical eye. His attitude should be—I am looking for trouble—not, I will excuse the little things because they are no worse today than they were yesterday.

Measuring devices which are rundown are no longer a standard measuring device, and indicated discharges may be considerably in error. To be certain of the true discharge, they should be rehabiliated and/or calibrated.

Repairing or refurbishing a rundown measuring device is sometimes a difficult or impossible task. Fixing the little things as they occur will prevent, in many cases, replacing the entire device at great cost at some later date. Regular and preventive maintenance will extend the useful life of measuring devices.

Poor Workmanship

Contrasting with the measurement devices which were once accurate and dependable and have deteriorated, are those which, because of poor workmanship, were never a standard device. These include devices which are installed out of level or out of plumb, those which are skewed or out of alinement, those which have leaking bulkheads with flow passing beneath or around them, and those which have been set too low or too high for the existing flow conditions. Inaccurate weir blade lengths or Parshall flume throat widths, insufficient or nonexistent weir nappe ventilation, or incorrect zero setting of the head or staff gage can also be the cause of measuring errors.

A transverse slope on a weir blade can result in errors, particularly if the gage zero is referenced to either end. The error can be minimized by determining the discharge based on the head at each end and using the average discharge. Errors in setting the gage zero are the same as misreading the head by the same amount. At low heads a relatively small zero setting error can result in errors up to 50 percent of the discharge or more. A head determination error of only 0.01 foot can cause a discharge error of from 5 percent on a 90° V-notch weir, to over 8 percent on a 48-inch Cipoletti weir (for a head of 0.20 foot). The same head error on 6- and 12-inch Parshall flumes can result in 12 and 6 percent errors, respectively, for low heads.

Weir blades which are not plumb or are skewed will show flow-measurement inaccuracies of measurable magnitude if the weir is out of line by more than a few degrees. Rusted or pitted weir blades or those having projecting bolts or offsets on the upstream side can cause errors of 2 percent or more depending on the severity of the roughness. Any form of roughness will cause the weir to discharge more water than indicated. Rounding of the sharp edge of a weir or reversing the face of the blade also tends to increase the discharge. On older wood crests a well-rounded edge can cause 15 to 25 percent or more increase in discharge, Figure 15.

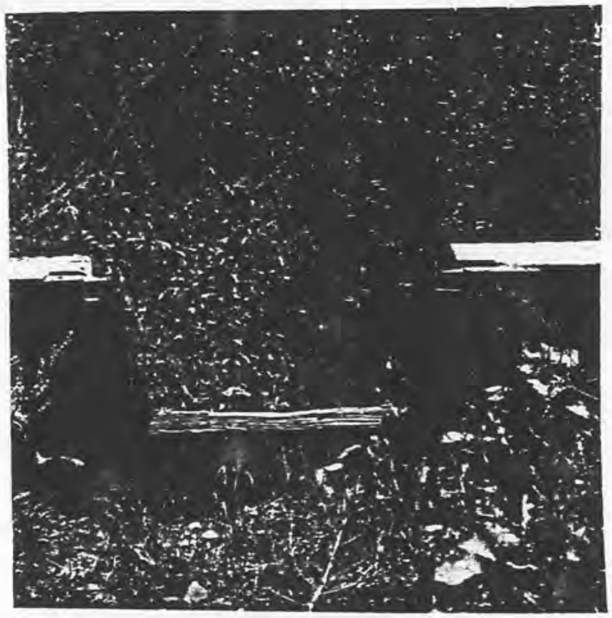


Figure 15. The well-rounded edge on this once sharp-crested weir will increase the discharge well above "standard." The weeds are also undesirable as is the weir gage which projects into the flow area. Photo P20-D-21557

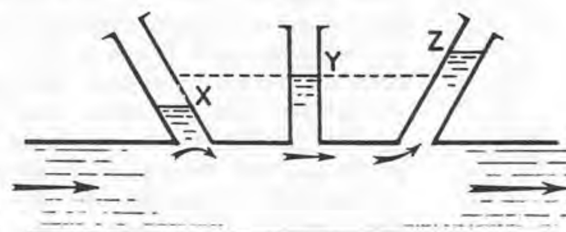


Figure 16. Note: Piezometer openings above are shown larger than they should be constructed in practice. Always use the smallest diameter opening consistent with the possibility of clogging by foreign material.

Pressure readings are needed to determine discharges through certain types of meters. Piezometers, or pressure taps as they are sometimes called, must be regarded with suspicion when considering accuracy of flow measurements.

Piezometers must be installed with care and with a knowledge of how they perform, otherwise the pressure values they indicate can be in error. For example as shown in Figure 16, the three piezometers will indicate different pressure readings (water levels) because of the manner in which flow passes the piezometer opening. Unless the piezometer is vertical as in Y, the water elevation will be drawn down as in X, or increased as in Z. Rough edges or surfaces in the vicinity of the piezometer can also result in erroneous indications in that they deflect the water into or away from the piezometer opening. The higher the pipe velocity, the greater the error will be.

The effect of a few deficiencies often found in measuring devices has been given to illustrate the degree of error to be expected in making ordinary measurements under ordinary conditions. Other effects have not or cannot be stated as percent error without an exact definition of the degree of fault or deterioration. The examples given should be sufficient, however, to emphasize the importance of careful and exact installation practices, as well as regular and prompt repair or rehabilitation of the devices after they have been installed.

Measuring Techniques Reducing Accuracy of Measurement

It is possible to obtain inaccurate discharge measurements from regularly maintained equipment properly installed in an ideal location, if poor measuring techniques are used by the operator. Measurement of head is very important and some of the techniques now in use are not compatible with the relationships between head and discharge known to exist.

The frequency of head measurement is also important and may be the cause of inaccurate water measurement. These and other related miscellaneous techniques are discussed in the next paragraphs.

Faulty Head Measurement

Measurement of the head on a weir seems to be a simple matter but can be difficult under all but ideal conditions. The head is the height of water above the blade edge (or crotch of a V-notch) measured at a point where the velocity head (or velocity of approach) is a negligible value, Figure 9. In practice this means a point located four to six times the head upstream from the center of the weir blade. If the head is measured farther upstream, the head necessary to produce flow in the approach channel (water surface slope) may be inadvertently included to give a larger head measurement. If the head is measured closer to the weir blade, some drawdown (caused by increased velocity near the weir) may occur and less than the true head may be measured. If the head is measured at the side of the approach channel, more or less than the true head may be measured depending on the geometry of the approach pool, Figures 9 and 13.

The practice of placing staff gages on weir bulkheads or on bankside structures should be investigated in each case to be sure that a true head reading can be obtained. Placing a rule or a Clausen-Pierce gage on the weir blade also gives an erroneous reading. The taking of head measurements when debris or sediment has a visible effect on the flow pattern can also result in faulty head determination, Figure 17. Measuring head, when the measuring device has obviously been damaged or altered, is also to be avoided.

Figure 18 shows a weir performing properly for the discharge shown. At larger discharges the unsymmetrical approach pool may produce undesirable conditions.

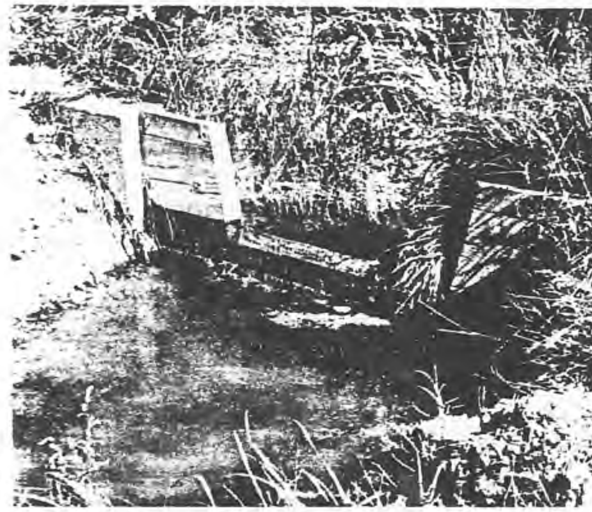


Figure 17. Weeds protruding through the opening and sediment in the approach pool will result in inaccurate discharge determinations. Photo P-D-30665

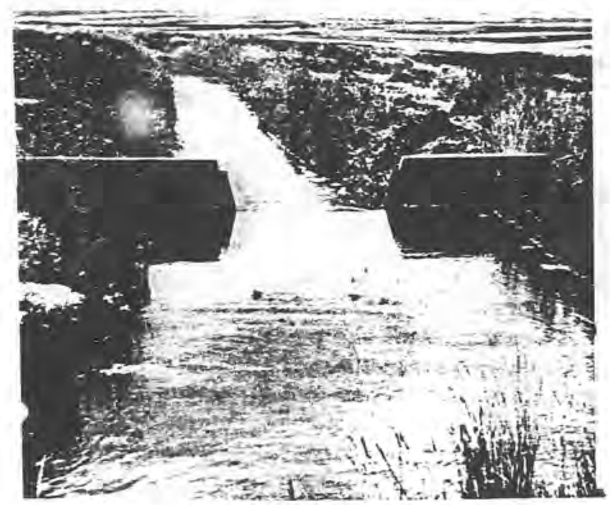


Figure 18. Cipoletti weir operating with good flow conditions in the approach pool. Flow is well distributed across wide pool and shows no evidence of excessive turbulence. Accurate or "standard" discharges can be expected under these conditions. Photo POAX-D-18350

The principles described above also apply to head measurements on Parshall flumes, metergates or any other device dependent on a head measurement for discharge determination.

Improper gage location, or an error in head measurement in a Parshall flume can result in very large discharge errors. Throat width measurements (and weir lengths) can also produce errors although these errors are usually small because of the relative ease of making accurate length measurements. (Operators should measure lengths in the field and not rely on values stated or shown on drawings.) Readings obtained from stilling wells, whether they are visual or recorded, should be questioned unless the operator is certain that the well intake is not partially or fully clogged. Data from an overactive stilling well can also be misleading, particularly if long-period surges are occurring in the head pool. In fact, all head determinations should be checked to be sure that the instantaneous reading is not part of a long-period surge. Sufficient readings, say 10, should be taken at regular time intervals, say 15 seconds, and averaged to obtain the average head. More readings may be required if it is apparent the pool is continuing to rise or fall. If this is too time consuming, the cause of the instability should be removed.

Readings from gages or staffs which may have slipped or heaved should be avoided. Periodic rough checks can sometimes be made with a carpenter's level or square from a reference point on another structure. A stillwater level at weir crest height is a valuable check on the staff gage zero.

In short, it is desirable that each operator understand the measurement he is trying to make, and then critically examine each operation to be sure he is measuring what he intends to measure. He should try to find fault with every step in making a head measurement and try to improve his technique wherever possible.

Infrequent Measurement

When a head or velocity measurement is made to determine discharge, it can be concluded that the measured discharge occurred only at the moment of the measurement. It cannot be concluded that the discharge was the same even 5 minutes later or 5 minutes earlier. Therefore, water deliveries can be accurate only if enough measurements are made to establish the fact that the discharge did or did not vary over the period of time that water was being delivered.

In many systems, measurements are made only once a day, or only when some mechanical change in supply or delivery has been made. Problems introduced by falling head, rising backwater, gate creep or hunting are often ignored when computing a water delivery. The problem is not a simple one, at times, and there are many factors to consider in determining the number of readings to be made per day or other unit of time. If the discharge in the supply system is increasing or decreasing, it will be necessary to take more than a single reading. If the rate of rise is uniform, the average of two readings, morning and night, would be better than one. If the rate of change is erratic, frequent readings may be necessary. If a great many readings are known to be necessary, a recording device may be justifiable.

Sometimes when the discharge in the supply system remains constant, the water level or velocity reading change because of a change in control, or because checks have been placed in operation. Temporary changes in discharge in the main supply system may occur, for example, because water, in effect, is being placed in storage as a result of the rising water level. Conversely, the discharge may temporarily increase in parts of the system, if the operating level is being lowered. The changing water level may make it necessary to take more frequent head readings.

Here again, the operator should try to visualize the effect of any change in discharge in the supply system, upstream or downstream from a measuring device, and attempt to get more than enough readings to accurately compute the quantity of water delivered.

Use of Wrong Measuring Device

Every water-measuring device has limitations of one kind or another and it is impossible to choose one device that can be used in all locations under all possible conditions. It is to be expected, therefore, that for a given set of conditions there may be several devices which would be suitable, but none could be considered entirely satisfactory. If flow conditions change or are changed by modified operations, an original device, which was marginal in suitability, may be found to be totally inadequate. It is possible, too, that the wrong device was selected in the first place. Whatever the reason, there are instances where accurate measurements are being attempted using a device which cannot, even with the greatest care, give the desired results. The operator should call attention to such a situation and attempt to have remedial measures taken.

For example, a weir cannot be expected to be accurate if the head is appreciably less than 0.2 foot, or greater than about one-third of the weir blade length. Large measurement errors can be expected (departure from standard), if these limits are exceeded appreciably. If a weir is submerged appreciably by backwater, large errors may be introduced depending on other factors.

In view of uncertainties which cannot be explained satisfactorily, submerged weirs should be avoided wherever possible. Parshall flumes should not be operated at more than the critical degree of submergence (80 percent); in fact, they should not be submerged at all, unless provisions have been made in the flume for a downstream head measuring well, and the method of computing submerged discharges from the published tables is thoroughly understood. This is explained in detail in the section on "Parshall Flumes."

Propeller meter devices should not be permanently installed where weeds, moving debris, or sediment are apt to foul the meter or grind the bearings. Submerged devices, such as metergates, should not be used where a moving bedload can partly block the openings.

In short, it is necessary to analyze the flow conditions to be encountered at a particular site, and only then, select the measuring device that can best cope with the unusual condition to be encountered. The user should expect an inaccuracy not to exceed about plus or minus 2 percent for a properly selected and maintained standard measuring device.

ELABORATIONS ON THE BASIC WATER-MEASURING DEVICES AND TECHNIQUES

Orifices

Submerged orifice

For a free-flow orifice, Figure 5, the discharge equation was shown to be approximately equal to $Q = 4.89 A\sqrt{H}$.

If the head H on the orifice was 4 feet and the area of the orifice was 2 square feet, the discharge would be

$$Q = 4.89 \times 2 \times \sqrt{4} = 19.6 \text{ cfs}$$

If an orifice is discharging into a ditch, as shown in Figure 19, there may be some backwater to prevent free-flow conditions.

Since the head H is pushing water through the orifice and the head h is attempting to hold it back, h must be subtracted from H before using a head value in the equation

$$Q = CA\sqrt{2g(H - h)}$$

In the preceding problem, if h was 1 foot, the discharge would be

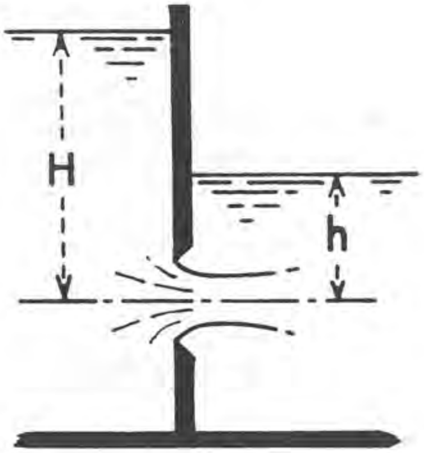


Figure 19. Flow from an orifice.

$$Q = 4.89 \times 2 \times \sqrt{3} = 17.1$$

or a reduction of 2.5 cfs from the 19.6 cfs computed without submergence.

Orifice in pipeline

If an orifice is placed in a pipeline as shown in Figure 20, there is almost certain to be a backwater effect, and it will be necessary to measure both the upstream and downstream heads. Since there is no free water surface in a full pipe, piezometers or pressure taps must be used to obtain the necessary data.

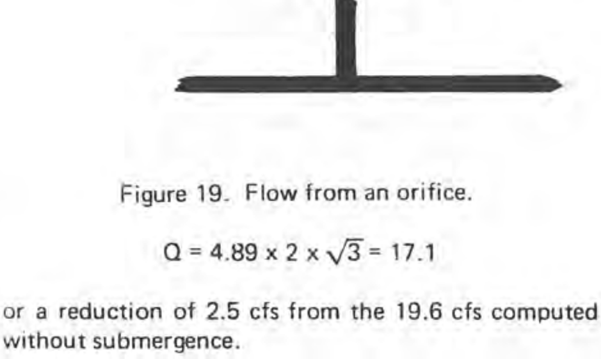


Figure 20. Orifice in a pipeline.

Piezometers are no more than small-diameter standpipes in which water rises sufficiently to balance the pressure inside the pipe. The accuracy aspects of piezometers have been discussed, Figure 16.

If piezometers were placed in a pipe as shown in Figure 20, the differential pressure H − h (between B and C) would be the head-producing flow to be used in the orifice discharge equation. The head at A would be lower than at B because some of the total head would have been used in producing the velocity in the pipe. At B the velocity of flow would be nearly zero, and so the true head would be indicated by the piezometer. The pressure at C would be very low because of the high velocity. At D some head recovery would occur because of the reduction in velocity caused by the spreading of the orifice jet.

At E normal pipe flow has been reestablished and the loss A – E represents the head lost because of the disturbances in the flow caused by the orifice. Energy in the flow was converted to heat as a result of turbulence in the flow and extra friction losses at the orifice plate and pipe boundaries.

Orifice discharges may be calculated with reasonable accuracy if all the factors affecting the flow are evaluated and the coefficient "C" is adjusted accordingly. For example, the graph in Figure 21 shows the variation in C to be expected for various combinations of pipe size and orifice diameter.

The orifice coefficient is seen to be 0.61 (in the solid line curve) when the orifice is 0.2 of the pipe diameter or less and increases to 1.0 when the orifice is 0.9 of the pipe diameter. It would therefore appear that large orifices would be preferable to small. This is not necessarily so, however, because large orifices give such a small differential that the error in reading the head is a large part of the differential. Also, the head tends to fluctuate severely so that at times it may appear that there is a reverse differential.

Thus, orifice installations should provide sufficient head (and/or differential) to make head reading errors negligible in terms of the differential head. In fact, it has been shown that the head on a freely discharging orifice should be at least twice the diameter of the orifice. For lower heads, the coefficient falls off rapidly and may be as low as 0.2.

Rounding of the sharp edge of a circular orifice may be the cause for considerable error in determining discharges. A 1-inch-diameter circular orifice rounded to a radius of 0.01 inch will discharge 3 percent more water than a sharp edge. This is because the contraction is not as great with a rounded edge as with a sharp edge. (Note that this is a very slight degree of rounding.)

In general, the percent increase in C (or discharge) due to rounding, equals three times the percent that the radius of rounding is of the diameter of the orifice.

The dotted-line curve shows coefficients (for $H - h = 3$ feet) obtained from a careful volumetric calibration of five orifices 1-1/2, 2-3/8, 3-7/8, 6, and 8-1/2 inches used in a 12-inch pipeline as a laboratory metering system. The departure of the coefficient from the generally accepted solid-line curve is considerable.

The broken-line curve shows coefficients for five orifices 1-1/4, 1-3/4, 2-3/8, 3-3/8, and 4-3/8 inches used in an 8-inch pipeline having a 8- to 5-1/2-inch reducer placed upstream from the orifice. The 8-inch pipe size was used to compute the ratio plotted as the abscissa in the sketch. Here, again, is a departure from the generally accepted coefficient curve and if a coefficient had been assumed from the solid-line curve, serious discharge measuring errors of perhaps as much as 15 percent could have occurred.

Because of the many factors which affect orifice discharges, it is usually desirable to calibrate an installation by volumetric measurements, current meter, Pitot tube, or other primary means. This may not be possible, and it may then be necessary to improvise a calibration. Another objection to the use of orifice meters is that the head loss caused by the meter may be excessive. Losses may run as high as one or more velocity heads. One velocity head is equal to the head required to produce the velocity in the pipe upstream from the orifice determined from Equation 3.

Orifice meters are not generally available from commercial supply houses, and it is not ordinarily possible to buy a meter complete with piezometers and head or differential head gages.

When a submerged orifice is used in an open ditch, the area of the orifice should be no more than about one-sixth the ditch cross-sectional flow area to minimize velocity of approach effects. This is roughly equivalent to using an orifice-to-pipe-size ratio, Figure 21, of about 0.25; coefficient 0.62. A high velocity of approach means that some of the head (which is to be measured) has been converted to velocity and cannot be measured directly. To account for this head, the velocity must be determined, converted to head, and added to the measured head.

The height of the rectangular orifice should be considerably less than the width to minimize the effect of variable head on the orifice coefficient. The submerged orifice equation (6) may be used along with a coefficient of 0.61-0.62.

If the velocity of approach is excessive (head has been converted to velocity and cannot be read on the staff gage), the velocity head (use the average velocity in the ditch upstream from the orifice and convert to H by $V^2/2g = H$) must be added to the measured head.

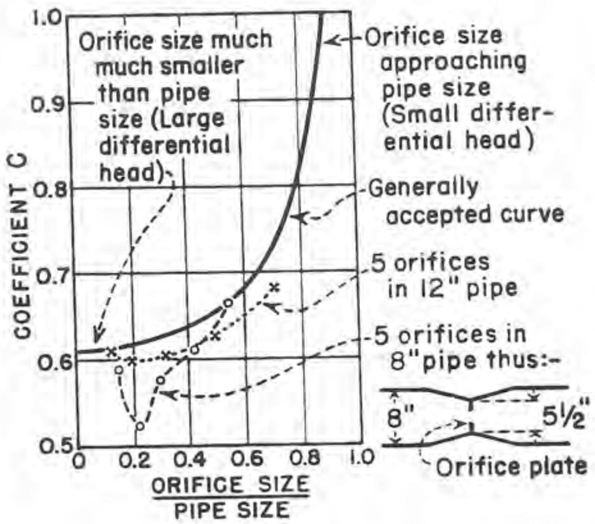


Figure 21. Orifice coefficients.

If the orifice is suppressed (hindered by floor, walls, or other) from a normal approach flow pattern, use the equation

$$Q = 0.61\ (1 + 0.15r)\ A\sqrt{2gH} \qquad (7)$$

where r is the ratio, length of suppressed portion of perimeter of orifice divided by total perimeter.

Discharges for standard rectangular orifices are given in Table 29 and correct coefficients for suppressed orifices in Table 30 of the Water Measurement Manual. Other information on submerged orifices is given in Chapter IV of the Manual.

Venturi Meters and Flow Tubes

The Venturi meter is basically a streamlined orifice meter and was devised to reduce the head loss produced by the orifice meter. The meter consists primarily of a constriction in a pipe with a curved approach to the constriction and a gradual expansion to the pipe diameter as shown in Figure 22.

A typical Venturi meter is shown in the sketch for a constriction of one-half the pipe meter (d/D = 0.5). The piezometric heads are shown as H and h and the differential used to determine meter discharges as H – h. The head loss is shown at the downstream piezometer as being about 0.1 velocity head, considerably less than for an orifice of the same size. Line A (the hydraulic grade line) shows the elevation of the water surface which would be indicated if a large

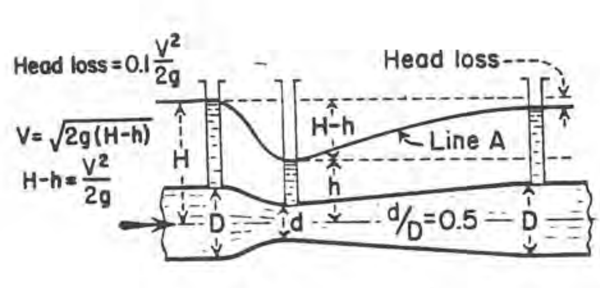


Figure 22. Venturi meter.

number of piezometers were installed in the meter to indicate the variation in pressure from point to point.

Although tables are usually used, the curve in Figure 23 shows a typical rating curve for a commercial 8-inch Venturi meter (approximately 4.5-inch throat and 8-inch pipe).

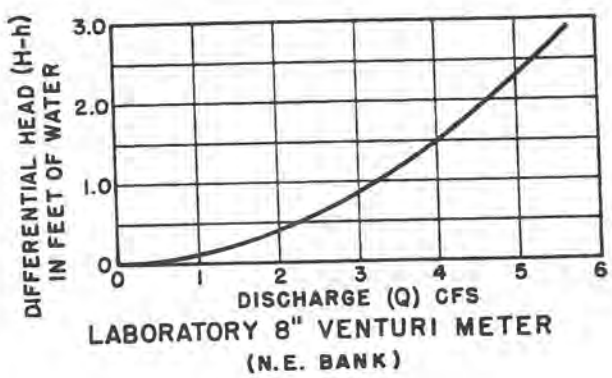


Figure 23. Venturi meter rating curve.

Of particular interest and concern is the shape of the lower portion of the rating curve. A differential head of only 0.1 produces a discharge of 1 cfs. It would be difficult to measure 0.1 foot accurately because of the usual fluctuations; and, consequently, it would be difficult to say whether the discharge was 0.7, 1.0, or 1.2 cfs. The meter should not be used, therefore, for discharges less than 2.5 cfs where the differential head is about half a foot. For discharges of 3 to 5 cfs, the 8-inch meter could be expected to be extremely accurate. If a discharge of 1 cfs must be measured accurately, a smaller Venturi meter should be used. Because of the nature of the meter, the differential varies as the square of the rate of flow. This means that when the meter is discharging 50 percent of capacity, the differential is 25 percent of maximum; 10 percent of capacity shows only 1 percent of the maximum differential; 5 percent of capacity shows only one-fourth percent of maximum differential. An orifice meter has the same characteristic rating curve, and the above statements apply to orifice meters as well.

Venturi meters are available commercially in a range of sizes and can be purchased with an accompanying set of discharge tables or curves. Venturi meters must always be calibrated because it is impossible to calculate discharges accurately. Calibrated meters are usually accurate to within 2 percent.

Venturi meters are usually machined castings and are relatively expensive, although cheaper cast concrete has been successfully used in some cases. Some success has been achieved in constructing meters from standard pipe fittings which can be screwed or bolted together. Two standard pipe reducers with a standard gate valve between them makes a satisfactory measuring device which has been found to be accurate to plus or minus 1.7 percent. Some of the early Work on this subject is contained in a Master of Science Thesis, 1942, "Hydraulic Characteristics of Simplified Venturi Meters," by R. A. Elder, Oregon State University, Corvallis, Oregon.

Venturi meters are usually of two basic types—the standard long form or the short form. The Dall tube is a commercial version of the short form tube, but is claimed by its manufacturers to have a low head loss. The long-form tube usually has less head loss than the short form because more head is recovered in the long tapered expanding section downstream from the throat than in the more abrupt short section. The short form usually costs less and requires less space, however. Flow tubes constructed of fiberglass and epoxy are available for installation between flanges in a pipeline. These require little space because the flow tube is placed, essentially, inside the pipe.

Head loss must be considered when selecting a meter because pump sizing may be affected and pumping costs may be a part of the daily operating cost. The head loss is governed chiefly by the length of the tube and the ratio of throat to inlet diameters, the loss being greater for short tubes and small throats. Attempts to reduce the head loss by increasing the throat diameter will result in smaller differential pressures for a given discharge. Too large a throat, therefore, may result in measurement inaccuracies.

Comparative head losses for several types of meters are given in Column 2 of the table below, for a throat-to-inlet diameter ratio of 0.5.

The yearly cost of electricity for pumping 1 cfs against these heads is shown in Column 3 (75 percent efficiency, power cost $0.02/kwh).

Meter type	Head loss, feet	Yearly pumping cost, dollars
Flow tube	0.6	13
Long-form Venturi	1.0	23
Short-form Venturi	1.2	29
Orifice	6.3	140

Concrete meters, Figure 24, have been constructed and used by the Fresno Irrigation District, Fresno, California. The Fresno meter consists of a length of standard concrete pipe into which has been formed a circular throat section to give a reduction in area so that the principle of the Venturi meter is applicable for the measurement of flow.

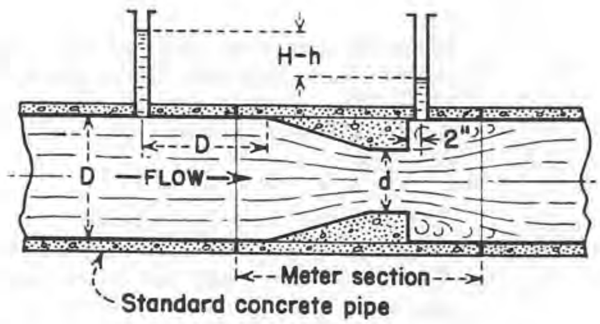


Figure 24. Fresno irrigation flowmeter.

Meters are available in 8-, 10-, 12-, 14-, 16-, 18-, 20-, and 24-inch sizes (D). Accurate laboratory calibrations have been made for the 8-, 10-, 12-, and 18-inch sizes. Head loss versus discharge curves are also available for these sizes. The losses range from 0.2 foot for the 8-inch meter discharging 1 cfs, 0.4 foot for a 10-inch meter discharging 2 cfs, 0.7 foot for a 12-inch meter discharging 4 cfs, to 0.9 foot for an 18-inch meter discharging 10 cfs.[21]

VENTURI FLUMES

Trapezoidal

Venturi flumes are of two types, the "free-flow" type where a simple head reading is required to determine discharge and the "submerged" type which requires two head readings to account for the backwater depth effect and determine the discharge. The latter type is sometimes called a "critical depth flume" and/or a "standing wave flume." The former is sometimes called the "true Venturi" type. The Parshall flume is an example of a Venturi flume that is often used in either category or both.

When head must be conserved, the flat-bottomed Venturi flumes are more desirable than flumes having vertical configurations in the floor. If the canal is trapezoidal, the flat-bottomed Venturi flume can also be made trapezoidal for convenience of construction of placement. But, like all measuring devices, they should be either calibrated before use or be constructed exactly the same as an existing flume (standard device) that has been calibrated.

From studies made on Venturi flumes it has been found that for any given flume, each value of the discharge (Q) has a unique and corresponding head (H). The results of these studies indicate that the relation between discharge and head may be expressed in the general form:

$$Q = K H^n$$

where the coefficient (K) and the exponent (n) are predominantly dependent upon the geometry of the flume. When the values of (K) and (n) are determined from actual measurements, the device is said to be calibrated.

The Bureau of Reclamation has studied the flat-bottomed trapezoidal Venturi flume shown in Figure 25. This particular flume was studied for discharges ranging from 0.5 to 5 cfs; the discharge equation was found to be

$$Q = 3.48 H^{2.24}$$

Studies of other flumes of both larger and smaller sizes will be continued and will be directed toward standardizing the flumes in terms of geometry and in providing rating tables for general use.

Small flat-bottomed trapezoidal Venturi flumes were studied by A. R. Robinson and A. R. Chamberlain, "Trapezoidal Flumes for Open-Channel Flow Measurement," ASAE, Volume 3, No. 2, 1960. Their study presents the calibration test results on seven flumes with side slopes (θ) ranging from 30^o to 60^o, throat bottom width varying from 0 to 4 inches, and contraction angles (ϕ) varying between 8^o to 22^o. The discharge range covered by these flumes is from 0.02 to 2.0 cfs. If flumes of this type are to be built and used without field calibration, the dimensions and limitations discussed in the article should be carefully followed.

Flat-bottomed Venturi flumes can be made of concrete, metal, or wood. However, the use of wood should be avoided wherever possible because the effect of swelling and warpage can be severe. Regardless of the material used for construction, the flumes should be sufficiently rigid to prevent bowing caused by earth pressure from backfilling.

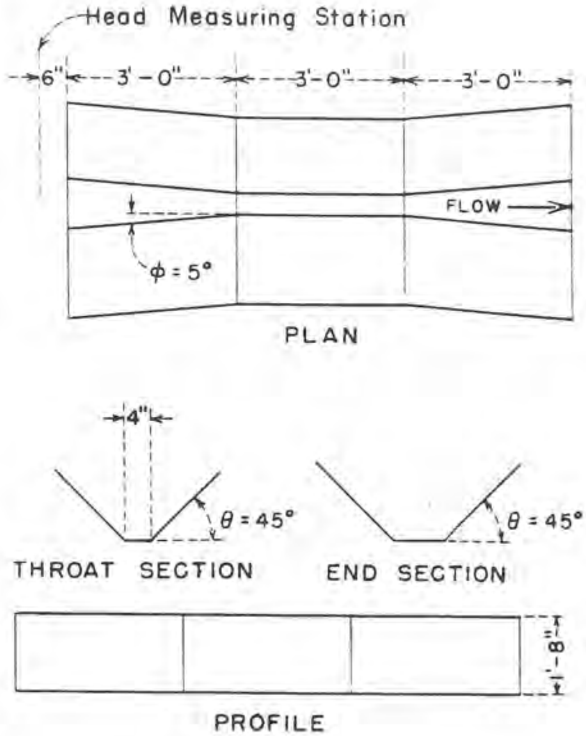


Figure 25. Flat-bottomed Venturi flume.

For best results the flat-bottomed flume should be set flush with the bottom of the incoming canal. If possible, the cross sections of the canal and the start of the converging portion of flume should match. If matching is not possible, transitions to the flume can be made of concrete, metal, wood, or gravel large enough to resist movement with the flow.

The head-measuring station should be located just upstream from the start of the convergence in the flume, Figure 25. If a stilling well and hook gage are used, the pressure tap or piezometer should be placed about 2 inches above the bottom of the flume to prevent sediment and other debris from plugging the lines to the stilling well. Staff gages may also be used to measure the head. To indicate the necessary accuracy for a head determination (in terms of discharge error), the following table may be used. This table is for the flume size shown in Figure 25.

Error in head reading feet	Discharge error range percent
0.005	1 to 2
0.010	2 to 6
0.020	4 to 10

In an ordinary installation the velocity of approach to a Venturi flume would not have an effect on accuracy. Excessive flow velocity at the flume entrance can cause errors of up to 4 percent, however, and some care in installation and maintenance is required if the Venturi flume is to be considered an accurate measuring device.

Reasonable entrance velocities will result in no measurable discharge errors. If the water surface just upstream from the flume is smooth (shows no surface boils, waves, or high-velocity current concentrations), it may be concluded that the Venturi flume accuracy is not being affected by the approach flow.

Venturi flumes are calibrated in a channel having a horizontal bottom. Field installations should approximate this condition if accuracy is important.

Parshall Flumes

The Parshall flume is an open-channel-type measuring device containing a specially shaped constricted-throat section, and was developed for use in a stream, canal, ditch, or other open flow way, to measure the rate of flow of water. Standard flumes are available over a wide range of sizes and each will measure discharges accurately within the limits of general irrigation requirements; usually within plus or minus 2 percent accuracy for free flows and plus or minus 5 percent for submerged flows.

Each flume is capable of measuring a wide range of discharges:

(a) Without excessive loss of head (less head loss than an orifice or weir).

(b) Without excessive amounts of sediment depositing in the structure.

(c) Without backwater or submergence effects nullifying the accuracy of measurement.

(d) Without a deep and wide upstream pool to reduce the velocity of approach.

The constricted throat of the flume, a modified form of Venturi profile, produces a differential head, between the upstream and downstream water surfaces, that can be related to discharge, Figure 26. The raised crest and the other configureations of the flume bottom give the Parshall flume the ability to withstand a relatively high degree of submergence without reducing the rate of flow through the flume. In other words, the tailwater (H_b) may rise to at least 50 percent of the upstream depth (H_a), shown in Figure 26 without causing a reduction in discharge. (The 50 percent figure assumes that both H_a and H_b are measured from the crest elevation to the water surface as in Figure 29.)

The converging upstream portion of the flume accelerates the entering flow and almost eliminates the depositing of sediment in critical parts of the flume. Sediment deposits tend to reduce the accuracy of flow measurements. High velocity of approach, which often is a detrimental factor in the operation of weirs, has little, if any, effect on the rate of discharge of the flume. The approaching flow should, however, be well distributed across the channel and should be relatively free of turbulence, eddies, and waves for accuracy of measurement.

Discharge through a Parshall flume can occur for two conditions of flow. The first, called free flow, occurs when there is insufficient backwater depth to reduce the discharge rate; water surface line FF in Section A-A of Figure 26 would not reduce the discharge.

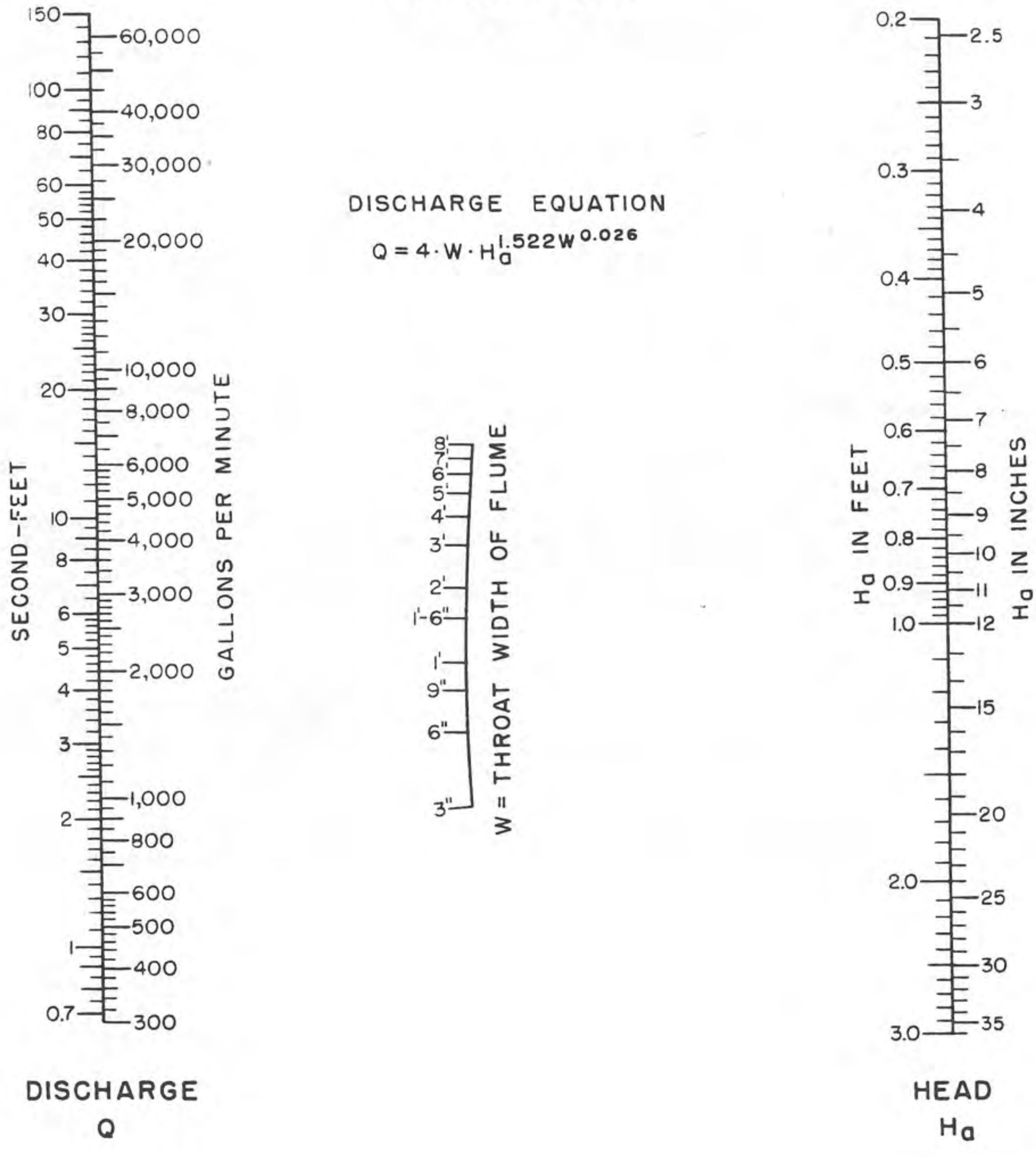
The second condition, called submerged flow, occurs when the water surface downstream from the flume is sufficiently above the elevation of the crest of the flume to reduce the discharge. Water surface line S in Figure 26 would reduce the free-flow discharge rate.

For free flow only the head, H_a, at the upstream gage location is needed to determine the discharge from the standard tables. The free-flow range includes some of the range which might at first be considered the submerged flow range, because Parshall flumes can tolerate 50 to 80 percent submergence before the free-flow rate is reduced to a measurable degree. For submerged flows (when submergence is greater than 50 to 80 percent, depending on flume size), both upstream and downstream heads, H_a and H_b, are needed to determine the discharge.

A distinct advantage of the Parshall flume is its ability to function as a single-head device over a wide operating range with minimum loss of head. This loss is only about one-fourth of that needed to operate a weir having the same crest length. Another advantage is that the velocity of approach is automatically controlled if the correct size of flume is chosen and the flume is used as it should be as an "inline" structure. Parshall flumes are widely used in irrigation systems because there is no easy way for the unscrupulous to alter the flume dimensions or change the flow channel to obtain an unfair proportion of the water. Also, it is easy for one water user to check his own water delivery against those of neighboring users.

The main disadvantages of a Parshall flume are: (1) it may not be an accurate measuring device when used in

NOMOGRAPH FOR FREE FLOW DISCHARGE
THROUGH 3-INCH TO 8-FOOT
PARSHALL FLUMES



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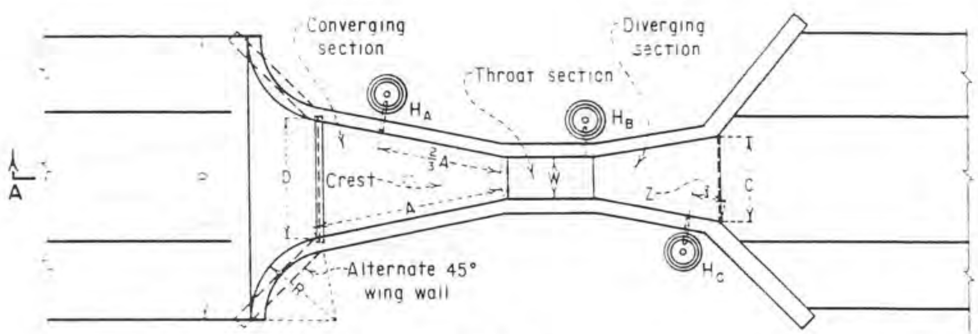

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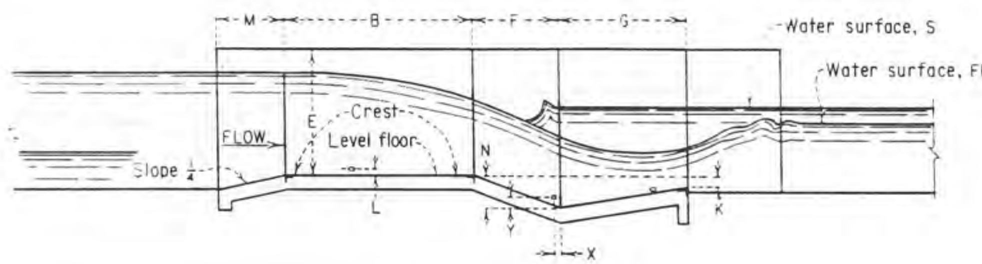


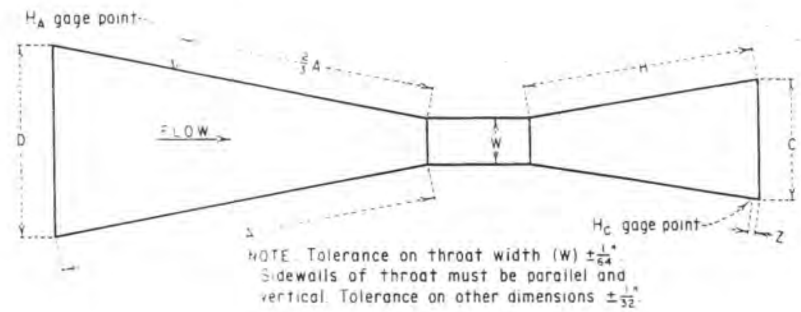
SECTION A-A

Plan and elevation of a concrete Parshall measuring flume. W, size of flume, in inches or feet; A, length of side wall of converging section; $\frac{2}{3}$A, distance back from end of crest to gage point; B, axial length of converging section; C, width of downstream end of flume; D, width of upstream end of flume; E, depth of flume; F, length of throat; G, length of diverging section; K, difference in elevation between lower end of flume and crest; M, length of approach floor; N, depth of depression in throat below crest; P, width between ends of curved wing walls; R, radius of curved wing wall; X, horizontal distance to H_B gage point from low point in throat; Y, vertical distance to H_B gage point from low point in throat. Water surface FF is free flow profile, S is submerged flow profile; Z, applies to 1 to 3-inch flumes, L, $1\frac{1}{2}$ inches, flume sizes 1-0 to 8-0 feet.

DIMENSIONS AND CAPACITIES OF THE PARSHALL MEASURING FLUME, FOR THROAT WIDTHS, W, FROM 3 INCHES TO 8 FEET

W	A	$\frac{2}{3}$A	B	C	D	E	F	G	K	N	R	M	P	X	Y	FREE-FLOW CAPACITY	
																MINIMUM	MAXIMUM
Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	In.	In.	Ft. in.	Ft. in.	Ft. in.	In.	In.	Sec.-ft.	Sec.-ft.
0 3	1 $6\frac{3}{8}$	1 $\frac{1}{4}$	1 6	0 7	0 $10\frac{3}{16}$	2 0	0 6	1 0	1	$2\frac{1}{4}$	1 4	1 0	2 $6\frac{1}{4}$	1	$1\frac{1}{2}$	0.03	1.9
0 6	2 $\frac{7}{16}$	1 $4\frac{5}{16}$	2 0	1 $3\frac{1}{2}$	1 $3\frac{5}{8}$	2 0	1 0	2 0	3	$4\frac{1}{2}$	1 4	1 0	2 $11\frac{1}{2}$	2	3	.05	3.9
0 9	2 $10\frac{5}{8}$	1 $11\frac{1}{8}$	2 10	1 3	1 $10\frac{5}{8}$	2 6	1 0	1 6	3	$4\frac{1}{2}$	1 4	1 0	3 $6\frac{1}{2}$	2	3	.09	8.9
1 0	4 6	3 0	4 $4\frac{7}{8}$	2 0	2 $9\frac{1}{4}$	3 0	2 0	3 0	3	9	1 8	1 3	4 $10\frac{3}{4}$	2	3	.11	16.1
1 6	4 9	3 2	4 $7\frac{7}{8}$	2 6	3 $4\frac{3}{8}$	3 0	2 0	3 0	3	9	1 8	1 3	5 6	2	3	.15	24.6
2 0	5 0	3 4	4 $10\frac{7}{8}$	3 0	3 $11\frac{1}{2}$	3 0	2 0	3 0	3	9	1 8	1 3	6 1	2	3	.42	33.1
3 0	5 6	3 8	5 $4\frac{3}{4}$	4 0	5 $1\frac{7}{8}$	3 0	2 0	3 0	3	9	1 8	1 3	7 $3\frac{1}{2}$	2	3	.61	50.4
4 0	6 0	4 0	5 $10\frac{5}{8}$	5 0	6 $4\frac{1}{4}$	3 0	2 0	3 0	3	9	2 0	1 6	8 $10\frac{3}{4}$	2	3	1.3	67.9
5 0	6 6	4 4	6 $4\frac{1}{2}$	6 0	7 $6\frac{5}{8}$	3 0	2 0	3 0	3	9	2 0	1 6	10 $1\frac{1}{4}$	2	3	1.6	85.6
6 0	7 0	4 8	6 $10\frac{3}{8}$	7 0	8 9	3 0	2 0	3 0	3	9	2 0	1 6	11 $3\frac{1}{2}$	2	3	2.6	103.5
7 0	7 6	5 0	7 $4\frac{1}{4}$	8 0	9 $11\frac{3}{8}$	3 0	2 0	3 0	3	9	2 0	1 6	12 6	2	3	3.0	121.4
8 0	8 0	5 4	7 $10\frac{1}{8}$	9 0	11 $1\frac{3}{4}$	3 0	2 0	3 0	3	9	2 0	1 6	13 $8\frac{1}{4}$	2	3	3.5	139.5

Figure 26. Parshall flumes of small size.



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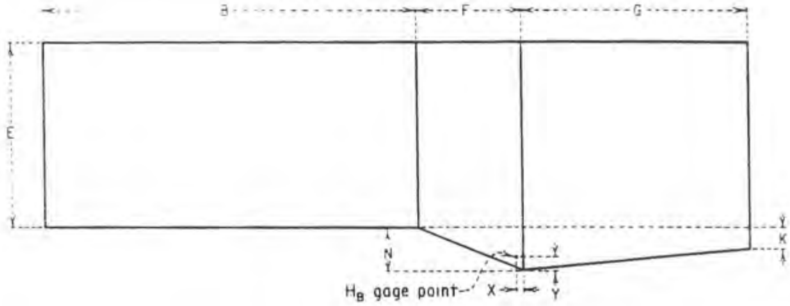


ELEVATION

DIMENSIONS OF 1, 2 AND 3-INCH STANDARD PARSHALL MEASURING FLUME, INCHES

W	A	$\frac{2}{3}$A	B	C	D	E	F	G	H	K	N	X	Y	Z
1	$14\frac{9}{32}$	$9\frac{17}{32}$	14	$3\frac{21}{32}$	$6\frac{19}{32}$	6–9	3	8	$8\frac{1}{8}$	$\frac{3}{4}$	$1\frac{1}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{1}{8}$
2	$16\frac{5}{16}$	$10\frac{7}{8}$	16	$5\frac{5}{16}$	$8\frac{13}{32}$	6–10	$4\frac{1}{2}$	10	$10\frac{1}{8}$	$\frac{7}{8}$	$1\frac{11}{16}$	$\frac{5}{8}$	1	$\frac{1}{4}$
3	$18\frac{3}{8}$	$12\frac{1}{4}$	18	7	$10\frac{3}{16}$	12–18	6	12	$12\frac{5}{32}$	1	$2\frac{1}{4}$	1	$1\frac{1}{2}$	$\frac{1}{2}$

DIMENSIONS FOR PARSHALL MEASURING FLUMES OF LARGE SIZE

SIZE (THROAT WIDTH)	FREE-FLOW CAPACITY		AXIAL LENGTH			WIDTH		WALL DEPTH CONVERGING SECTION	VERTICAL DISTANCE BELOW CREST		H_A GAGE DISTANCE (NOT AXIAL)*
	MAX.	MIN.	CONVERGING	THROAT	DIVERGING	UPSTREAM END	DOWNSTREAM END		DIP AT THROAT	LOWER END FLUME	
Feet	Sec. ft.	Sec. ft.	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Inches	Feet
10	200	6	14	3	6	15' 7.25"	12' 0"	4	1' 1.5"	6	6'0"
12	350	8	16	3	8	18' 4.75"	14' 8"	5	1' 1.5"	6	6' 8"
15	600	8	25	4	10	25' 0"	18' 4"	6	1'6"	9	7' 8"
20	1000	10	25	6	12	30' 0"	24' 0"	7	2' 3"	12	9' 4"
25	1200	15	25	6	13	35' 0"	29' 4"	7	2' 3"	12	11' 0"
30	1500	15	26	6	14	40' 4.75"	34' 8"	7	2' 3"	12	12' 8"
40	2000	20	27	6	16	50' 9.5"	45' 4"	7	2' 3"	12	16' 0"
50	3000	25	27	6	20	60' 9.5"	56' 8"	7	2' 3"	12	19' 4"

NOTE: For all large sizes the H_B gage is located 12 inches upstream from, and 9 inches above, the floor at the downstream edge of throat.

* H_A gage distance is measured along flume wall, upstream from the crest line.

Figure 27. Parshall flumes of "very small" and large size.

Figure 28. Four-foot Parshall flume discharging 62 cfs under free-flow conditions. Downstream scour protection is required with this height of fall. Photo PX-D-55348

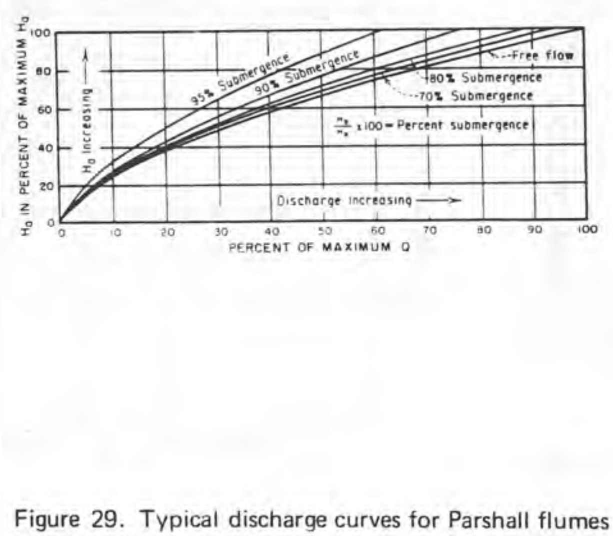


Figure 29. Typical discharge curves for Parshall flumes with free flow and with submerged conditions.

a close-coupled combination structure consisting of turnout, control, and measuring device, and (2) it is (usually) more expensive than a weir or submerged orifice.

Parshall flume sizes are stated in terms of the throat width, W, and are available from the 1-inch size for discharges as small as 0.01 cfs* to the 50-foot size for discharges up to 3,000 cfs, Figures 26 and 27.

Parshall flumes may be constructed of wood, concrete, galvanized sheet metal, or other materials. Large flumes may be constructed piece by piece on the site, whereas small flumes may be purchased as prefabricated structures to be installed in one piece, Construction methods and materials are discussed in References 1, 30, 31, and 32 listed at the end of this report. Some flumes are available as lightweight shells which are made rigid and immobile by placing concrete against the outside of the walls and beneath the bottom. The larger sizes are used in rivers and large streams; the smaller ones for measuring farm deliveries or for row requirements in the farmer's field.

Free-flow Operation

In free flow the quantity of water a flume of width W will pass is uniquely related to the depth of water at the gaging point, H_a, in the converging section, Figure 28. Free-flow conditions in the flume are similar to those that occur at a weir or spillway crest. Water passing over the crest is not impeded or otherwise affected by downstream conditions.

The lower curve, labeled free flow in Figure 29 shows a typical relationship between discharge, Q, and head H_a, for a Parshall flume discharging in the free-flow range.

Submerged Flow Operation

In most installations when the discharge is increased above a certain critical value, the resistance to flow (produced by the downstream reaches of the channel) is sufficient to reduce the velocity, increase the flow depth, and cause a backwater effect (sometimes called "flooding") at the Parshall flume. The effect is similar in some respects to that explained for the submerged orifice where (H – h) was the flow-producing head.

It might be expected, therefore, that the discharge would begin to be reduced as soon as the tailwater level exceeded the elevation of the flume crest. This is not the case, however. Calibration tests show that the discharge is not reduced until the submergence, in percent exceeds the following values:

*0.01 cfs is approximately 5 gpm.

$\frac{h}{H}$ = 0.50 = 50 percent for flumes 1 to 9 inches wide

$\frac{h}{H}$ = 0.70 = 70 percent for flumes 12 inches to 8 feet wide

$\frac{h}{H}$ = 0.80 = 80 percent for flumes 8 to 30 feet wide

In the terminology used with Parshall flumes H is known as H_a and h is known as H_b.

A typical submergence curve which illustrates the effect of submergence in reducing the discharge through an open channel control structure such as a weir or small Parshall flume is shown in Figure 30.

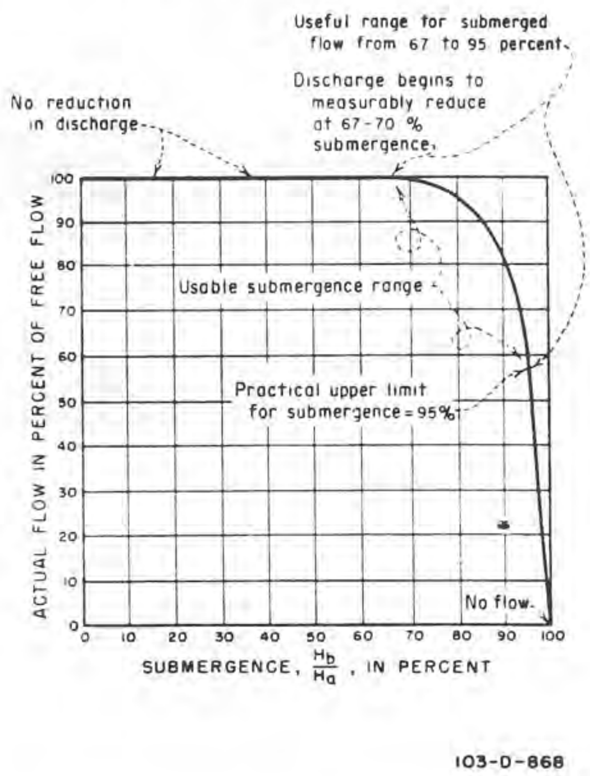


Figure 30. Discharge reduction caused by submergence.

The submergence curve shown is schematic, but characteristic, and indicates that the downstream tailwater surface can rise to 67-70 percent of the height of the headwater above the crest before the reduction in discharge becomes significant. Then with each additional percent increase in submergence, the discharge is reduced at a more rapid rate, until, when H_b equals H_a there is no flow. This ultimate condition is purely theoretical, however, and would occur in nature only when the flow was stopped.

The 95 percent mark on the curve also represents a point of practical interest. In this range the Parshall flume ceases to be an accurate device. In other words, a small differential between H_a and H_b is difficult to measure and any slight error in differential determination results in a large error in discharge.

The submergence curve of Figure 30 helps to explain the relative positions of the typical discharge curves shown in Figure 29 and indicates why the 70 percent submergence curve lies so close to the free-flow curve. Also, Figure 29 shows that 60 percent of the maximum discharge occurs for 95 percent submergence; therefore, the remaining 40 percent of the maximum flow must be reduced to zero in the next 5 percent of submergence. Prohibitively accurate measurements of H_a and H_b would be required to make accurate discharge measurements in this range.

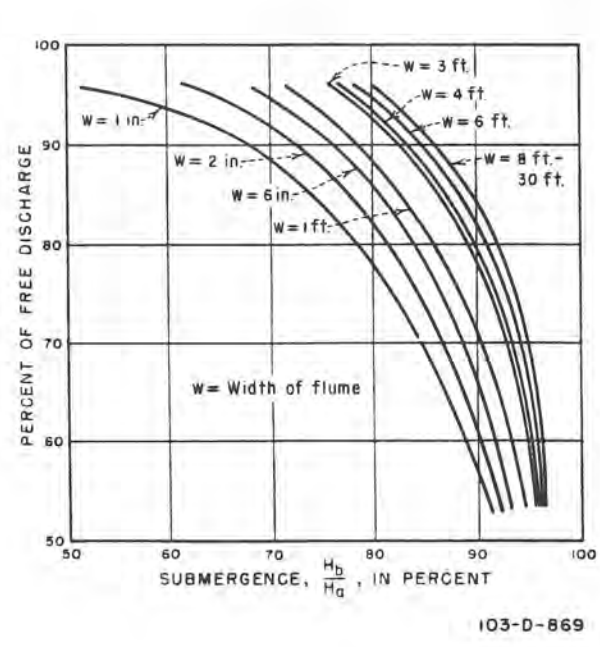


Figure 31. Approximate discharge after submergence.

Approximation of Discharge Rate—Submerged Flow

It is difficult to visualize or estimate the change in discharge rate caused by changing the degree of submergence in various sizes of flumes. The curves in Figure 31 can be used to estimate the effect of increasing or decreasing the submergence, or increasing or decreasing the flume size without the need for knowing or considering the upstream head, H_a. For example, Figure 31 shows that at 60 percent submergence, only the 1-inch flume would be affected; the discharge would be reduced to about 93 percent of the free discharge rate. At 70 percent submergence, only the 1-foot and smaller flumes would be affected; the 1-inch flume would discharge 89 percent of the free discharge rate and the 1-foot flume would discharge 94 percent of the free discharge rate. Also, at a submergence of 80 percent the discharge from all flumes will be affected to some degree. The 8- to 30-foot sizes will discharge 96 percent of the free discharge while the 1-inch flume will discharge only about 79 percent of the free discharge. The nomograph for 3-inch to 8-foot flumes, page 20, can be used to obtain most free-discharge values.

It may be seen, therefore, that submergence effects on discharge rates occur at the lower submergence ratios (percentages) on the small flumes, and that the greatest percentage reduction in discharge also occurs on the small flumes.

Although the curves in Figure 31 are approximate, because they do not include the small changes in discharge which also occur as a function of the magnitude of the discharge, the values are reasonably accurate and are useful for most estimating purposes. The graph may also be useful in making rough calculations for determining the size of flume required, and the best vertical placement in the channel. The curves represent observed data (obtained during calibration runs and checks) with a maximum deviation of plus or minus 7 percent. In most of the usually used regions of the curves, the overall accuracy is somewhat better. Methods for determining exact submerged discharge values are discussed later.

Approach Flow

In the original writings on Parshall flumes, only brief mention is made of the importance of good or bad approach flow conditions. A few statements are made such as "* * * the accuracy is not affected by silt or slow water" * * * "nor by changing velocity of the stream * * *," "the velocity of approach is automatically controlled * * *" "the angles of convergence and divergence are such as to eliminate the effect of switching of the current in the diverging section * * *." Over the years, these and similar phrases in the original writings have been loosely interpreted to mean that the Parshall flume is able to overcome poor flow conditions in the approach to the flume and still maintain a good degree of accuracy. Nothing could be further from the truth. It should be remembered that these flumes were intended for use as "inline" structures in a stream or canal where reasonably smooth flow, uniformly distributed across the width and depth of the cross section was the normal condition. It was not expected that the flumes would be placed in turnouts where the main flow lines are at right angles to the flume, or below control gates where turbulence and flow concentrations are in evidence.

The converging section of the flume is designed to accelerate the flow and smooth out small differences in velocity and flow distribution so that the flow passes through the flume throat in a "standard" smooth pattern. Each lineal inch of throat width is expected to pass the same quantity of water as every other inch. Only under these conditions can the flume pass a standard discharge.

It is obvious, then, that any upstream flow condition which results in changing the standard pattern at the flume throat will cause a departure from the standard discharge indicated in the tables. Considering the length of the approach section and the geometry of the flume, it is understandable that the relatively short converging section can exert only a limited influence on redistributing flow concentrations, or in correcting excessively high velocities. Therefore, if accuracy is of importance, the approach conditions to the flume should be obviously good.

Experience has shown that the flume should not be placed at right angles to the flowing stream—such as in a canal turnout—unless the flow is straightened and redistributed to form a uniform velocity and flow pattern before entering the flume. Surges in the flow, which sometimes persist through the flume, should be eliminated as should surface waves of any appreciable size, Figure 12. The water should enter the converging section reasonably well distributed across the entrance width and the flow streamlines should be essentially parallel to the flume centerline. Also, the flow at the flume entrance should be free of "white" water and turbulence in the form of visible surface boils. Only then can the flume be considered as being capable of fulfilling all of the claims made for it.

Experience has also shown that it is better to provide "standard" conditions of approach and getaway than to try to estimate the effect of nonstandard conditions on accuracy. Nonstandard flow conditions are impossible to describe and evaluate in terms of measurement accuracy. Poor approach flow conditions should therefore be eliminated by deepening, widening, or straightening the flow channel; or by resetting or rearranging the measuring station. These procedures are preferred over attempting to correct the indicated discharges.

In locations where inequalities in the approach flow have resulted in flow measurement difficulties and no upstream wingwalls have been included in the original construction, the curved wingwalls shown in Figure 1 should be considered for installation. If it appears that a more gradual acceleration of the approaching flow would improve the flow patterns in the flume, the wingwalls should be constructed (perhaps on a temporary trial basis at first). Curved wingwalls are preferred over straight 45^{o} walls, although any arrangement of walls, channel banks, or other, that improves the uniformity and smoothness of the approaching flow is acceptable.

METERGATES

A metergate is basically a modified submerged orifice arranged so that the orifice is adjustable in area, Figure 32.

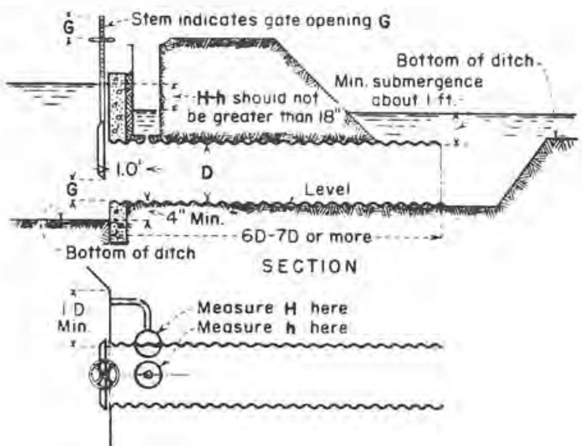


Figure 32. Metergate.

Although it should be possible to compute the discharge, this is rarely done because there are usually too many departures from standard definable conditions for which correction coefficients are not known. Metergates are usually purchased from a commercial supplier who supplies a discharge table. Ordinarily, the tables give a good accounting of the flow, but in some instances, errors of 18 percent or more have been found.

If a discharge error is suspected, the installation should be thoroughly checked to be sure that it complies with the essential conditions shown in the above sketch, particularly that there is no blockage of flow and that the outlet is sufficiently submerged to make the pipe flow full. The many factors affecting metergate performance and accuracy are described in detail in the following paragraphs. These suggestions apply particularly when the gate is operated at large openings (50 percent or more) and/or with small upstream submergence (1D or less).

Sources of Discharge Indication Error

Type of gate

The discharge table being used should be checked to be sure that it applies to the metergate in question. Tables for round-bottom gates will not work with square-bottom gates, or vice versa, except perhaps at the wide-open position. Be sure that the table being used is for the brand of gate, model number, or other identifying symbols.

Stilling well blockage

If there is no blockage of flow at the gate or in the pipe, make sure that the stilling wells are open. A bucket of water poured into the well should readily drain out or, if the gate is in operation, the water level in the well should rapidly return to the head indicated before the water was added. As a matter of general maintenance, it would be a good idea to flush the wells occasionally, push a probe through the piping, and flush again. Any difference in readings before and after cleaning might indicate the need for further flushing and cleaning. Staff gages or scales should also be checked to be sure they have been installed at the proper zero position and that they have not become displaced vertically.

Gate and gate opening indicator

Be certain that the gate opening indicator, whether it is the rising stem on the gate or some other device, has not become displaced to give a false gate-opening indication. Check the installation of the gate on the end of the pipe. The gate must seal when closed. Too much clearance may allow an excess of water to flow between the gate or frame and the end of the pipe, changing the flow pattern and indicated head in the downstream stilling well.

Approach area

Weeds, trash, or sediment in the approach to the gate can change the pattern of flow sufficiently at the gate leaf to produce sizable discharge errors. The flow along the sidewalls (wingwalls) has more effect on discharge than the flow along the bottom. Be sure that flow can follow the sidewalls without interference. Large amounts of sediment deposited in the area just upstream from the gate can upset the normal flow patterns as can waterlogged trash, rocks, or other submerged material. The approach area should be cleaned and reshaped, if necessary, until no flow lines or velocity concentrations are visible on the water surface.

Submergence

The water level at the gate should be at least one pipe diameter (preferably two) above the crown of the pipe during operation (flow measurement). As previously shown for the orifice, considerable error results when the head is less than one diameter above the top of the pipe. The pipe outlet must also be sufficiently submerged to make the pipe run full. Usually, if the pipe length is standard, at least six or seven diameters (discussed later), the submergence need be only about 6 inches above the crown of the pipe. Unless the pipe runs full at the outlet, the downstream head measuring stilling well may not contain enough water to indicate the true differential pressure across the metergate, and serious discharge measuring errors can occur.

Small differential head

Large errors in discharge determination can be introduced if the differential head (difference in water surface elevation between the two stilling wells) is small. For example, in reading the two water surface elevations in the stilling wells, an error of 0.01 foot could be made in each reading, giving a differential of 0.10 foot instead of 0.08 foot. The difference in indicated discharges would be about 0.12 cfs for a discharge of 1.10 through an 18-inch metergate open 5 inches, an error of about 11 percent.

If the gate opening was reduced to 2 inches and the upstream pool could be allowed to rise to pass the same discharge, the differential head would be 0.40 foot and the same head reading error of 0.02 foot would indicate a change of only 0.03 cfs. The error in discharge determination would be reduced from about 11 percent to less than 3 percent.

If the pool level cannot be elevated as described and it is necessary to operate continually with small differential heads, it would be well to consider installation of a smaller gate. This would allow operation in the upper ranges of capacity where the differentail head is larger. If a smaller gate cannot pass the required maximum flow, it might be necessary to use two small gates in place of one large one.

Aside from head-reading errors, it is desirable to operate with larger differentials because (1) the flow is more stable and the water surface in the stilling wells does not surge as badly and (2) the higher velocity through the meter prevents a reduction in orifice coefficient (as discussed for the orifice meter).

Other methods of achieving a larger differential might include reducing the backwater level, if excessive, or reducing the pipe length, if it is considerably longer than six or seven diameters, to reduce the backwater effect; change the location of the downstream stilling well and recalibrate the meter (discussed later).

Location of stilling well intakes

Because the discharge is directly related to the difference in water levels in the two stilling wells, it is essential that the stilling well intakes (pressure taps or piezometers) be located exactly as they were when the meter was calibrated.

The upstream intake should be located in the headwall several inches (at least) from the gate frame, several inches (at least) from any change in headwall alinement in plan (see Figure 32), and at an elevation such that the intake will be covered at minimum operating level. The opening should be flush with the surface of the headwall and the piping arranged so that a cleaning probe may be pushed through for cleaning purposes. The pipe should slope continuously downward from well to headwall to prevent air locks in the system. If air is suspected in the piping, it may be flushed by pouring water into the well at a rapid rate to force the air out through the intake end, taking care not to entrain air in the pouring process.

The downstream piezometer (pressure tap) should be located on the centerline of the top of the pipe, exactly 1 foot downstream from the downstream face of the gate. The intake pipe must be flush with the inside surface of the pipe (grind off any projections beyond corrugated or smooth surface) and absolutely vertical (the effect of tilted piezometers is illustrated in Figure 16).

As shown in Figure 33, the rate of change in pressure is very rapid in the region of the downstream pressure tap and any displacement of the tap from the location used during calibration will result in large discharge determination errors.

A better location for the downstream piezometer would have been D/3, measured from the downstream face of the gate. The pressure gradeline here is lower and flatter. Minor variations in piezometer locations would not result in major measuring errors. However, if the piezometer is moved to this point (to increase differential head), the meter must be recalibrated because the published tables will not apply.

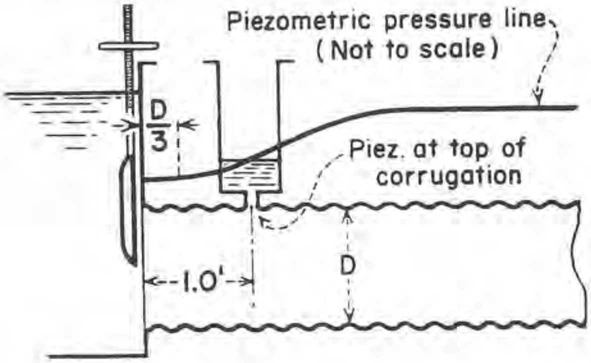


Figure 33. Location of stilling well intake.

Laboratory tests have been conducted on metergates to determine the coefficient of discharge C_d for a pressure tap located at D/3, as discussed. This curve shown in Figure 34 is valid for all sizes of metergates under certain standard conditions. These include:

1. Approach channel floor sloping upward, 2:1, toward gate with downstream end of floor 0.17D below pipe entrance invert.

2. Flaring entrance walls, 8:1, starting D/4 distance from edges of gate frame.

3. Zero gate opening set when bottom of leaf is at invert of entrance.

4. Upstream submergence is greater than D.

5. Downstream end of pipe is submerged to make pipe flow full.

It should be noted that the coefficient C_d is a different coefficient that the C used in the orifice equation. C_d is used with A which in this case is the area of the pipe and not the gate opening. Discharges may be computed from this equation with an accuracy of plus or minus 2-1/2 percent. The degree of downstream submergence does not affect the accuracy of the meter if water rises sufficiently in the downstream well to obtain an accurate reading and the pipe runs full at the outlet.

Metergate Installation

Metergates have been found to be set too low, too high, or the wrong size of gate was employed. To aid in the proper selection of gate size and the elevation at

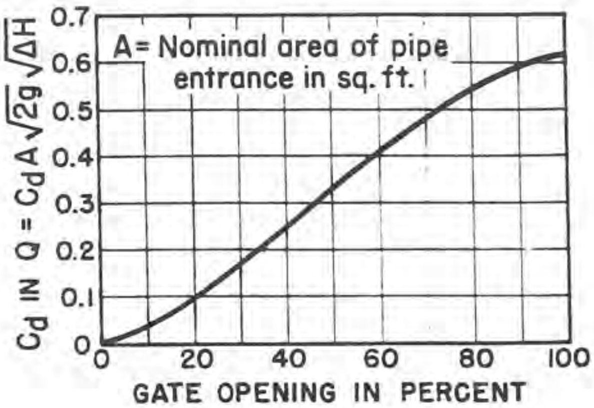


Figure 34. Metergate discharge curve.

which the gate should be placed, the following suggestions are given in the drawing in Figure 35. The metergate entrance structure should be as described in the preceding discussion.

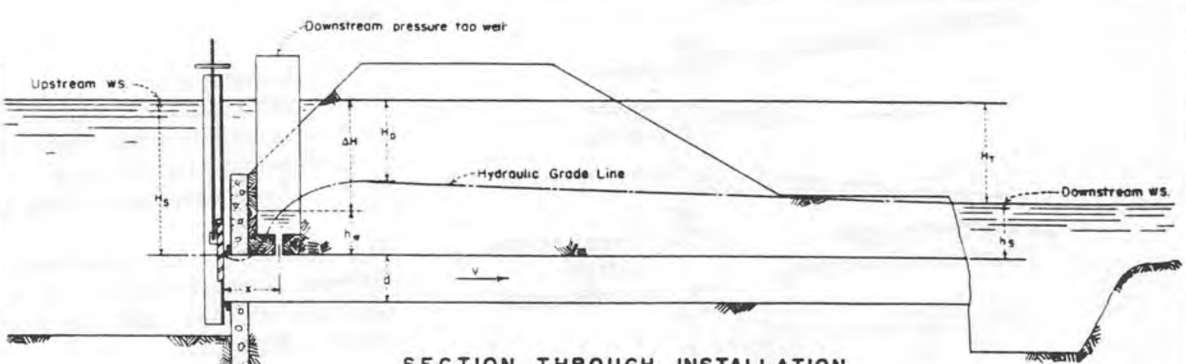
An analysis of other factors that influence metergate performance and accuracy, in cases where the installation is not standard, is given in Hydraulic Laboratory Report No. Hyd-471, dated March 15, 1961, "Flow Characteristics and Limitations of Screw Lift Vertical Metergates." This report covers various entrance problems effects of submergence, velocity and gate design, and gives rating curves for 18-, 24-, and 30-inch gates for both confined and unconfined approaches.

Constant-head Orifice Turnout (CHO)

The constant-head orifice turnout, Figure 36, is essentially a submerged orifice-meter type of measuring device. The upstream or orifice gate controls the discharge while the downstream or turnout gate controls the submergence on the upstream gate.

As a means of standardizing the device, it was arbitrarily decided to always submerge the orifice gate sufficiently to produce a 0.2-foot difference in water surface elevation (differential head) across the upstream or orifice gate.

The constant-head orifice is usually operated as follows: The orifice gate opening for the desired discharge is obtained from the discharge table and set. The turnout gate is adjusted until the differential head across orifice gate is at the required constant head of 0.2 foot. The discharge will then be at the desired value. Two standard sizes of constant-head orifice meters have been calibrated and the discharge values



SECTION THROUGH INSTALLATION

DETERMINATION OF METERGATE INSTALLATION

GIVEN

1. Upstream water surface El. 100.0.
2. Downstream water surface El. 99.0.
3. Turnout discharge, Q, = 8 cfs.
4. Depth of water in downstream measuring well, h_w, should be 6 inches above crown of pipe.
5. Length of metergate pipe, 50 feet.
6. Submergence of metergate inlet, H_s, should be equal to or greater than d above the crown of the pipe.

FIND

1. SIZE OF METERGATE (One of two methods may be used)
   a. Where downstream scour may be a problem.
      Select exit velocity that will not cause objectionable scour, say 4 feet per sec.
      From $A = \frac{Q}{V} = \frac{8}{4} = 2.00$, $d = 19\frac{1}{8}$ inches.
      Requires 20-inch metergate.
   b. Where scour downstream is not a problem.
      Assume metergate to be operated at openings up to 75 percent. (The influences of entrance design, upstream submergence and downstream pressure tap location are minor for these openings.)
      For 75 percent gate opening coefficient of discharge, $C_d \approx 0.5$, and maximum $\Delta H \approx 1.85\ H_D$
      $$\Delta H \approx 1.85\,(1.0) \approx 1.85 \text{ ft.}$$
      From $Q = C_d A \sqrt{2g\Delta H}$
      Area of pipe, $A = \frac{8}{0.5\,(8.02)\,(1.36)} = 1.47$ sq. ft.
      $d = 16\frac{3}{8}$ inches.
      Requires 18-inch metergate, d = 18 inches.
   c. Check capacity of gate using 18 inch metergate.
      $H_D = H_T - H_f$ (H_f is friction loss from pressure recovery point to pipe exit).
      $H_f = f\frac{L}{d}\frac{v^2}{2g}$
      Where f is coefficient of friction, L is length of pipe, d is pipe diameter and v is velocity in pipe.
      From $v = \frac{Q}{A} = \frac{8}{1.767} = 4.53$
      Assume f for concrete or steel pipe as 0.025.
      $H_f = \frac{0.025\,(26.3)\,20.5}{64.4} = 0.21$ feet.
      $H_D = 1.0 - 0.21 = 0.79$ feet.
      In order to have a measurable water surface in the downstream well for all gate openings and downstream tap positons the installation should be designed for maximum ΔH.
      $$\frac{\Delta H}{H_D}\ (\text{maximum}) \approx 1.85$$
      $$\Delta H \approx 1.85\,H_D \approx 1.85(0.79) \approx 1.46$$
      Using this adjusted value of ΔH, turnout capacity at 75 percent gate opening, $Q \approx 0.5(1.767)(8.02)(1.21) \approx 8.57$ cfs.
      18-inch metergate is adequate.
2. ELEVATION AT WHICH METERGATE SHOULD BE PLACED.
   a. To meet upstream submergence requirement, H_s, of 1.0d, crown of pipe entrance should be set at El. = 100.0 − d = 98.5.
   d. To meet requirement of water surface 6 inches above crown of pipe in downstream well, elevation of crown of entrance would be set at El. 100.0 $- \Delta H - h_w = 100.0 - 1.46 - 0.50 = 98.04$, say El. 98.0.
      Depth requirement for measurable water surface in downstream well is governing factor and gate should be set with crown of entrance not higher than El. 98.0.
3. MAXIMUM CAPACITY OF METERGATE (Full open)
   C_d for full gate opening with downstream pressure tap at $x = 2d/3$ (12 inches from entrance on 18-inch gate) is about 0.75
   $\frac{\Delta H}{H_d}$ for $x = 2d/3 \approx 1.1$
   $\Delta H = 1.1\,(0.79) \approx 0.87$
   From $Q = CA\sqrt{2g\,\Delta H}$
   $$\approx 0.75\,(1.767)\,(8.02)\,0.93$$
   $$\approx 9.9 \text{ cfs.}$$

SCREW LIFT VERTICAL METERGATE

INSTALLATION CRITERIA AND EXAMPLE

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Figure 35. Metergate installation criteria.

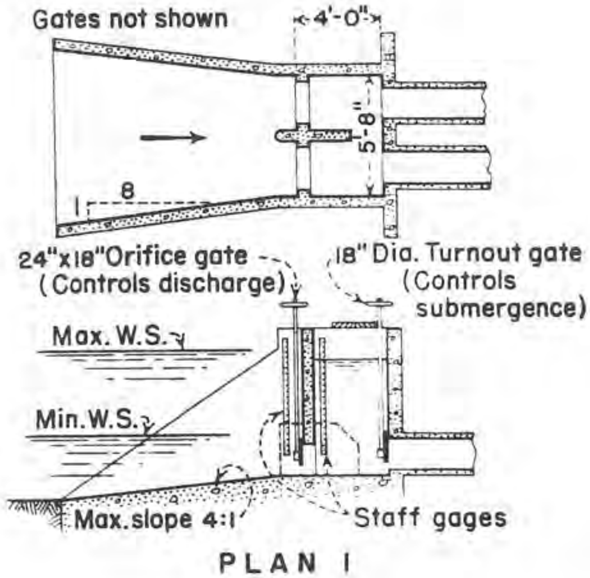

Figure 36. Constant head orifice turnout.

are given in Tables 32 and 33 of the Water Measurement Manual.

The 10-second-foot-capacity turnout is designed to operate with the canal water surface from 21 inches to 6 feet above the orifice gate seat. Minimum operating depth is 18 inches. This turnout uses a rectangular 24- by 18-inch screw lift vertical gate for the orifice gate and an 18-inch-diameter screw lift vertical gate for a turnout gate; two sets of gates are used side by side in the turnout structure which employs 18-inch-diameter pipe.

The 20-second-foot-capacity turnout is designed to operate with the canal water surface from 27 inches to 6 feet above the orifice gate seat. Minimum practical operating depth is about 24 inches. This turnout uses a rectangular 30- by 24-inch screw lift vertical gate for the orifice gate, a 24-inch-diameter screw lift vertical gate for the turnout gate; two sets of gates are used side by side and discharge into 24-inch-diameter precast concrete pipe.

Discharge Characteristics

The discharge through a constant-head orifice turnout may be computed from the orifice equation

$$Q = CA\sqrt{2gH}$$

where

Q = discharge in cfs
H = differential head on orifice gate (0.2)
A = area of orifice gate opening in square feet
C = coefficient of discharge
g = acceleration due to gravity (32 ft/sec/sec)

The coefficient "C" determined in 98 tests on 6 different designs of turnout, for a complete range of gate openings and canal water surface elevations is shown in Figure 37.

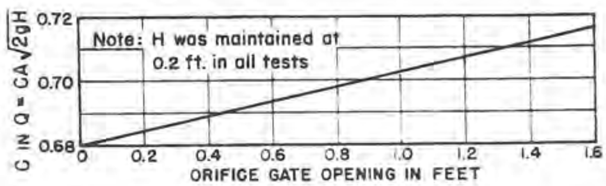

Figure 37. Orifice gate discharge coefficients.

The discharge tables (referred to above) were prepared from this curve. Single- and double-barrel tests gave the same discharge coefficients.

When only one of the two orifice gates is open, it is desirable to open the turnout gate directly downstream from the opened orifice gate. The head should be read on the sidewall of the pool next to the open gates. An incorrect head reading will be obtained if the gages on the sidewall opposite the open gate is used. If both turnout gates are opened with only one orifice gate open, an incorrect head reading will be obtained on all gages.

More consistent results will be obtained if the downstream gage is relocated adjacent to the orifice gate instead of adjacent to the turnout gate. Any arrangement of open and closed gates that produces a tilted water surface between the orifice and turnout gates should be avoided because of the difficulty in determining the head by any means.

Discharge Determination Errors

Since the principle of operation of the constant-head orifice turnout is to maintain a constant differential of 0.2 foot across the orifice gate, it is extremely important that this differential be determined accurately in the field if accurate discharge determinations are to be expected. The equation for discharge may be written

$$Q = CA\sqrt{2g}\sqrt{H}$$

where H is 0.2 foot.

If an error of 0.01 is made in reading each gage, H could be as small as 0.18 foot or as large as 0.22 foot. The error in discharge would be proportional to the square root of the head or

	Difference	
$\sqrt{0.18} = 0.4243$		
	0.0229	0.0224
$\sqrt{0.20} = 0.4472$		average
	0.0218	
$\sqrt{0.22} = 0.4690$		

and

$$\frac{0.0224}{0.4472} \times 100 = \pm 5 \text{ percent}$$

For an error of 0.02 foot in reading each gage the discharge determination error would be plus or minus 10 percent.

It is, therefore, apparent that accurate discharge measurements can be obtained only if great care is used in determining the differential head. Some operators have complained that it is next to impossible to read a staff gage accurately when looking downward at a steep angle into a dark hole at a choppy water surface which may also be surging. Since there is a good bit of truth in this statement, two suggestions are given to help obtain a better differential head reading.

The staff gages could be mounted in stilling wells made from a suitable length of commercially available transparent plastic pipe. This would prevent the choppy water surface from interfering with making an accurate reading. The wells should be quickly removable for cleaning. If surges are causing the water level in the well to rise and fall, a wood bottom having a 3/8-inch hole drilled in it could be fastened in the well. This would allow the well to average the surges and provides a more dependable head determination.

A second method to be temporarily mounted on the transverse concrete wall between the staff gages of stilling the flow can be constructed as shown in Figure 38. A troughlike basin is included in the compartment between the orifice and control gates.

The differential head for normal operation may be increased if difficulty still exists in setting, reading, or maintaining the 0.2-foot standard differential. Discharges may be calculated using the coefficient for the orifice gate opening actually used and the differential head actually measured. Turbulent flow

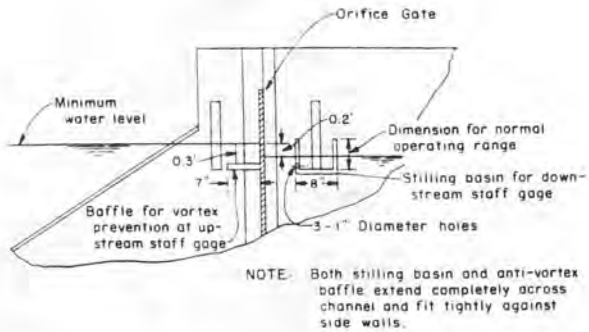

Figure 38. Orifice gate installation.

conditions or a reduced submergence at the turnout gate will not affect the discharge if it is possible to obtain a true downstream head. To be certain of the accuracy of these higher differential head discharges, it would be desirable to check several gate settings using a current meter or other calibration method to measure the discharge. This displacement, if any, of the coefficient curve from the values given for the 0.2-foot differential could be determined from calculations, and a new coefficient curve drawn parallel to the one shown. Only a few accurate check points would be required because the curve shape would necessarily be the same as for the 0.2-foot curve.

Errors in discharge measurement might be caused by factors other than the head measurement as discussed for metergates and orifices.

Effect of Entrance Structure Geometry

The preceding sketch of the constant-head orifice turnout indicates 8:1 flaring walls in plan on the entrance or approach structure and a 4:1 sloping floor. The floor slopes downward away from the orifice gate, Plan 1, Figure 36. Other common installations are shown in Figure 39; each has 8:1 flaring walls in plan.

Selection of the type of entrance for a new installation will usually be limited by the relative elevations of the canal bottom and the pipe invert. If there is any choice in the matter, however, it should be noted that Plans 1 and 2 provide the best operating structures. Flow conditions with these entrances are steady and smooth and the differential head is not difficult to read. When the entrance is partially constricted as in Plans 3 and 4, by an adversely sloping floor, the flow pulsates and the surface is rough. Surges and boils upstream from the turnout gates tilt the water surface and make the head difficult to read.

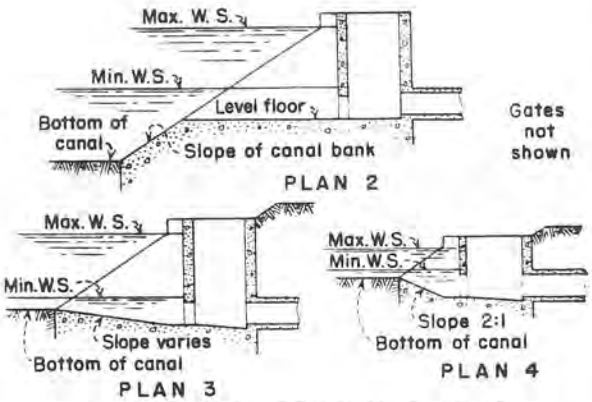

Figure 39. Typical orifice turnout entrances.

It has been noticed that some structures in the field are now of the Plan 4 type, even though they were Plan 2 type when installed. Sediment and debris have collected near the entrance to the turnout and maintenance crews have cleaned for only a short distance upstream, producing the sharp downward slope to the orifice gate. More extensive pool cleaning would improve the ease of obtaining head readings and might improve the accuracy of the measurement.

Current Meter Gagings

The current meter uses the velocity principle to obtain discharge. Velocity is measured in a small area at one time and therefore, enough readings must be taken to insure obtaining accurate values for the average velocity and for the area of the flow section.

In selecting a site for a gaging station or a location for a meter or any other propeller device, it is important that smooth uniform flow exists upstream (to some degree downstream) from the location at all times. The approach to the site should be straight for several hundred feet or more and, to the eye, the surface velocity should be the same across the entire width of the section. The cross section of the site should be typical of the sections upstream and downstream and should be in stable material. Locations where banks or bottom can erode or where sediment is known to deposit, should not be used. A site where meters can be operated from the upstream side of a bridge is desirable because a cableway or other crossing need not be constructed. The depth at minimum flow should be sufficient to use a current meter in its usual velocity range. If this is not possible, choose one site for low flows and another one for high flows.

The sensitivity of the station, in general, should b suitable—small errors in stage reading should not result in large measurement errors. Conversely, a significant change in discharge should be accompanied by a significant change in stage.

Sites affected by variable depth backwater should be avoided as should those having seasonal growths of aquatic weeds, or those having a confluence with a sizable tributary downstream.

The selected site should be close to a benchmark for easy zeroing of the staff and other gages. The station should be suitable for the installation of a water stage recorder and an intake to the stilling gage well. These should be closely grouped because it is imperative that the recorder and staff gage indicate the same water level.

Reference or staff gages should be firmly anchored and in an inclined or vertical position. Vertical gages should be vertical in all respects. Inclined gages should be graduated on the site by precise leveling after the gage is installed. Clear, accurate markings (0.01 foot) should extend to above and below the anticipated levels. Stilling wells should be vertical and have sufficient depth and height to allow the float to travel the entire range of water levels. The float diameter should be large enough to overcome friction and actuate the recorder as necessary. The intake piping to the well should be large enough to allow the water in the well to rise and fall with the river or canal stage without undue delay. All pipe joints should be watertight. Elbows should be made up of plugged tees to allow rodding if clogged.

Recorder chart records should be legible to read plus or minus 0.02 foot or better. If waves are causing the recorder to blur, use a restriction (perhaps a partially open gate valve) in the stilling well piping. A steel tape (electric indicator) should be installed in the well to set and check the recorder.

Recorders should be of such a design and type that a change in the chart record can be produced only by a change in water level. The recorder should be sensitive to changes of 0.02 foot or less. Clocks should be reliable and keep good time.

The paper chart scale chosen for recording should permit readings to be made which are within 1 percent of the depth of the water (above zero flow level) or within 0.02 foot, whichever is greater.

Price-type current meters should not ordinarily be used where velocities are less than 0.5 foot per second. The upper range should not exceed the calibrated range of the meter. Meters should not be operated in shallow water when the horizontal axis of the meter is closer to the surface than 1-1/2 times the vertical dimension of the rotor. Similar bottom clearance should be provided, measured from the top of any obstruction such as a rock or ledge. Meters should be rerated after about 100 hours of use or at least checked against some known standard. They should be rerated immediately if dropped, bumped, or used extensively in sediment-laden water.

In making a gaging measurement, choose a time when the stage will remain constant throughout the measurement. Discharge corrections for a changing stage are never completely satisfactory, even when all of the factors are known.

Current meter measurements are usually made on "verticals: i.e., vertical lines on the cross-section," chosen so that they provide an adequate sample of the velocity distribution in the cross section. These verticals should be chosen so that (1) the error in computing the area of the segment between two verticals does not exceed 3 percent if the portion of the bed profile between the verticals is treated as a straight line, and (2) the difference between the mean velocities on adjacent verticals does not exceed 20 percent by reference to the lower two (except close to the banks). In general, this means that the intervals between verticals should not be greater than 1/15 of the cross-section width (when the bottom is smooth), or 1/20 of the width when the bottom is irregular. Verticals need not be closer than 1 foot in any case; the number may be reduced when working in small lined channels having a regular geometric profile.

Provisions should be made to operate the meter from a cable or rod suspended in such a way that the performance of the meter is not affected by disturbances in the flow caused by the observer or the suspension equipment. The meter should be held in a given position, after allowing operation to become stabilized for 40 seconds or more. Total operation of the meter at each vertical should be not less than two consecutive periods of at least 40 seconds. If significant differences are apparent, more readings should be taken. The mean of all the readings at that point should be used for the velocity, unless there is an ovbious reason for eliminating one or more readings. The meter should be removed from the water between readings to be sure that its rotation is not being impeded by debris or any other cause.

Errors will arise if the meter:

(1) Is used to measure velocities less than 0.5 foot per second or beyond the calibrated range.

(2) Is not held steady in the same location during the timing sequence, or if the meter is held in an unsteady flow area such as an eddy.

(3) Is used when there is a significant water surface disturbance by wind.

(4) Is used in flow which is not parallel to the axis of a propeller-type meter or is oblique to the plane of the cup-type meter.

If only one or two velocity measuring points on each vertical are obtained, an arithmetic solution to obtain the discharge is appropriate. If only one velocity on a vertical has been determined, the mean velocity is (1) the value observed at 0.6 depth used without modification; (2) the value observed at 0.5 depth multiplied by 0.96. If two velocity points, such as the 0.2 and 0.8 depth have been determined, the average of the two points should be used.

To compute the discharge, the cross section should be regarded as being made up of a number of segments, each bounded by the two adjacent verticals. If V_1 is the mean velocity at the first vertical and V_2 the mean velocity at the second vertical and, if D_1 and D_2 are the depths measured at the respective vertical, and if b is the horizontal distance between these verticals, the discharge of the segment is:

$$Q = \frac{(V_1 + V_2)}{2} \times \frac{(D_1 + D_2)}{2} \times b$$

This calculation is repeated for each full segment. Segments adjacent to the banks may be handled by assuming zero depth and velocity at the water's edge. The total discharge is obtained by adding together the discharges from all the segments.

Careful plotting of a stage-discharge relationship curve for each gaging station will help to evaluate the accuracy of each gaging measurement as it is made, and will help to establish confidence in the station. After the station is put into operation, the cross section should be checked periodically and maintained in its original condition. Sediment bars should be removed from the bottom and corrections to the net section made, if erosion occurs on the banks or bottom. If the water surface is raised or lowered by checking, careful time records should be kept to determine when the staff gage or water stage records are an indication of the discharge.

Weirs

Since weirs were frequently used as examples in "General Aspects of Water Measurement Accuracy," they will not be elaborated upon in this portion of the text. They have been specifically referred to under headings of: Approach Flow, Turbulence, Velocity of Approach, Exit Flow Conditions, Weathered and Worn Equipment, Poor Workmanship, Faulty Head Measurement, and Use of Wrong Measuring Device.

Propeller Meters

Propeller meters have been in use since about 1913 and are of many kinds, shapes, and sizes. They are used submerged near the ends of pipes or conduits or "inline" in pressurized pipe systems. Many meter designs and modifications for special conditions and purposes are available from manufacturers and it is therefore impractical to try to discuss all makes and models. All propeller meters have certain common features, faults, and advantages, however, which can be analyzed to provide a better understanding of meter operation. A thorough comprehension of meter principles, their inherent limitations and advantages, and the operating experiences of many users may be beneficial when purchasing, installing, operating, and maintaining meters for a field installation. The information presented herein has been gathered from project and water district personnel, letters, reports, inspections, complaints, and from laboratory and field tests conducted specifically to evaluate propeller meter performance. Much information, some good and some doubtful has been sifted to emphasize basic material and eliminate incorrect or conflicting statements. An attempt has been made to eliminate material which applies only, or particularly to, one make of meter or to one specific installation. The material presented applies to all meters, in general, and to all installations, except as noted.

Propeller meters utilize a multibladed propeller (two to six blades) made of metal, plastic or rubber, rotating in a vertical plane and geared to a totalizer in such a manner that a numerical counter can totalize the flow in cubic feet (perhaps to within 10 cubic feet), acre-feet, or any other desired volumetric units, and/or an indicator to show the instantaneous discharge, in cubic feet per second, acre-feet per day, or any other desired units. The propeller, designed and calibrated for operation in a pipe or conduit, should always be fully submerged, that is, the pipe or conduit must be flowing full. The propeller diameter is always a fraction of the pipe diameter; usually varies between 0.5 to 0.8 of the diameter of the pipe. Compound flowmeters have more than one propeller and are more complicated in design. Meters are available for a range of pipe sizes from 2 to 72 inches in diameter.

The measurement range of the meter is usually about 1 to 10; that is, the maximum discharge the meter can indicate or totalize is about 10 times the minimum that the meter can handle. The meter is ordinarily designed for use in water flowing at from 0.5 to 17 feet per second although inaccurate registration may occur for the lower velocities in the 0.5 to 1.5 ft/sec range.

The meter size is usually stated as the diameter of the pipe in which it is to be used. The propeller size may vary for a given size of meter. For example, a 24-inch meter might have a 12-inch-diameter propeller for use in a 24-inch-diameter pipe. Thus, the principle involved in measuring discharges is not a displacement principle as in certain municipal water meters or indicating devices used on gasoline service station pumps, but rather a simple counting of the revolutions of the propeller as the flow passes the propeller and causes it to rotate. Anything that changes the frictional resistance of the propeller, the number of revolutions in a given time, or the relationship between pipe and propeller areas, therefore, affects the registration or accuracy of the meter. The many factors affecting propeller rotation are discussed in the following pages.

Flow Patterns

The accuracy of a propeller meter (in new condition) is primarily dependent upon the similarity of the flow patterns in the vicinity of the propeller during calibration and during regular use. Factors which change the flow pattern approaching or leaving the propeller will change the accuracy or registration of the meter.

Spiral Flow

A poor entrance to a turnout pipe, elbows, fittings, unsymmetrical approach flows and many other factors can produce spiral flow in a pipe. The propeller meter, because it has a hub at the center of the pipe and a revolving propeller, is therefore very sensitive to water flowing in a spiral pattern. Significant errors in registration can result when the meter is used in spiral flow. Depending on the direction of rotation of the flow with respect to the pitch and direction of rotation of the propeller, the meter will over or under register. Flow straightening vanes inserted in the pipe upstream from the meter will help to eliminate errors resulting from this cause. The meter manufacturer usually has specific instructions regarding the size and placement of vanes and these should be followed when installing a propeller meter. It is usually suggested that vanes be several pipe diameters in length and that they be located in a straight, horizontal piece of pipe just upstream from the propeller. The horizontal pipe should be seven diameters or more in length. Vanes for clean waterflow are usually made in the shape of a plus sign to divide the pipe into equal quarters. Laboratory experiments have shown that vanes of this type (used where no spiral flow exists) may reduce registration by 1 to 2 percent compared to readings made with the vanes removed. This is because the area taken up by the vanes tends to reduce the velocity near the center of the propeller. Some manufacturers have suggested the use of vanes which do not meet in the middle but divide the pipe into thirds as shown in Figure 40. A variation of these two configurations, that leaves the central part of the pipe open and provides four vanes, is shown in Figure 41, Photo PX-D-59226.

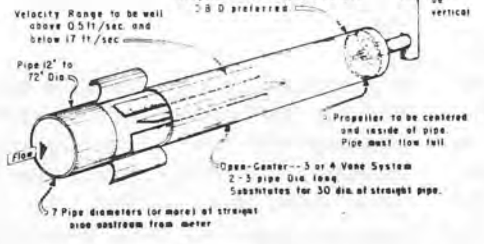

Figure 40. Desirable features for propeller meters.

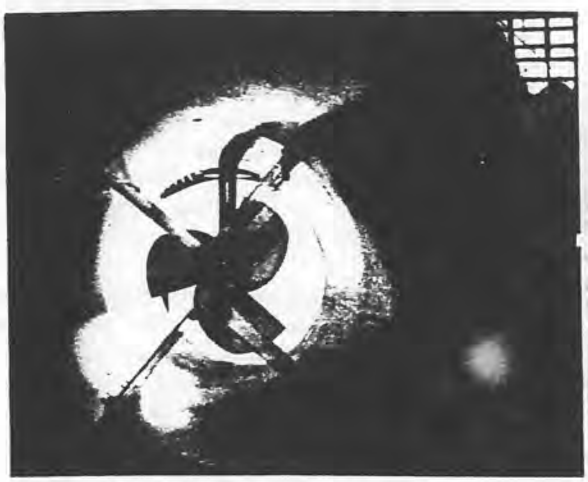

Figure 41. Variation of standard propeller meter installation.

Because of the open flow area in the center of the pipe there is less disturbance to the flow pattern in the center of the pipe. One or more diameters of clear space between the downstream end of the vanes and the propeller helps to nullify any adverse effects caused by either type of vane.

If straightening vanes are not used, a long length of straight, horizontal pipe (30 or more diameters long) may be required to reduce registration errors. Venting the pipe to the atmosphere just downstream from the control gate, if this is possible, may help to reduce spiral flow.

Velocity Profiles

In any pipe—even a very short one—the friction between the inside flow surface of the pipe and the water is greater than the internal friction of the water. This results in the water in the center area of the pipe having a higher velocity than the water near the boundary.

In a short pipe the velocity profile would be similar to Case A, Figure 42; in a longer pipe the profile would look as shown in Case B. In the latter fully developed velocity profile the difference between the center and edge velocities is quite large but is stable and does not increase further. It is obvious therefore that the propeller, which receives its impetus from the central area of the pipe, will receive different total forces for Cases A and B above, and that a greater number of revolutions will occur in the long pipe, Case B. On the other hand, less force, and fewer revolutions (under registration), will occur for Case A. Laboratory tests have shown that long turnouts or pipes (30 or more diameters long) have fully developed velocity profiles and give 3-4 percent greater registration than short pipes, 6-10 diameters long. Rough or corrugated pipes tend to produce the Case B velocity profile in shorter lengths than smooth-wall pipes. No exact data are available to define every situation, however.

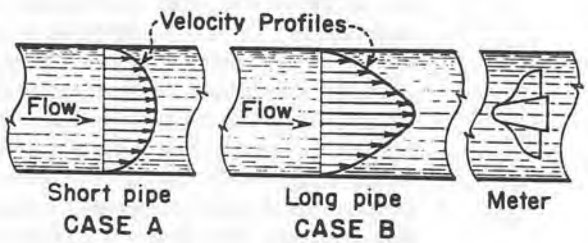

Figure 42. Velocity profiles.

Control gates, such as the slide gate often used at a turnout entrance, may affect the flow pattern and/or the velocity profile in a short pipe. For example, tests have shown that for a 24-inch pipe turnout 23 feet long from gate to meter, in the 5-cfs range, a full gate opening resulted in an indication of 96 percent of the true discharge. When the same discharge was put through a 6-inch gate opening, the meter indicated 100 percent of the actual discharge. The 4 percent difference in discharge was the maximum effect noted in these tests.

It is apparent, therefore, that changes in velocity distribution in the pipe cross section, that make the distribution significantly different from that used during the meter calibration, will cause a change in meter registration. Changes may be either plus or minus with respect to the original discharge calibration. Inspection and analysis of the flow conditions upstream from the meter may prove beneficial in troubleshooting a field installation suspected of giving incorrect meter registrations. Checking the flow distribution in a cross section just upstream from the meter with a Pitot tube would conclusively establish whether a poor velocity pattern was present. Meters are never calibrated with poor velocity patterns in the pipe.

Propeller and Pipe Size Relationships

Meters should always be used in pipes of the proper or recommended diameter. The meter manufacturer can supply this information. However, a discussion of the relative effects of propeller diameter will be helpful in understanding the trends in over and under registration where a meter is used in a pipe larger or smaller than the recommended size. Propeller diameters vary between 0.5 and 0.8 of the pipe diameter. It is therefore not always possible to determine the proper pipe size by measuring the propeller diameter.

In visualizing the effects of a larger or smaller pipe on the accuracy of a given propeller meter, Cases A and B of Figure 42 will be helpful. Depending on the type of velocity profile in the proposed larger or smaller pipe, the propeller will tend to intercept higher velocities (overregister) or lower velocities (underregister) and will be in error depending on the change in the average velocities intercepted. No numerical values can be given because different meter manufacturers have different propeller designs. For example, propeller tips are affected differently by varying velocity values; also, different clearance requirements between the propeller and the pipe wall produce different register values. Items that are important for one propeller shape may not affect another. Tests have shown, however, that the larger the pipe diameter with respect to the propeller diameter, the more a change in velocity profile will affect the meter registration. Thus, meters having propellers nearly the diameter of the pipe they are to be used in should provide most accurate results. Conversely, propellers that are half or less than the pipe diameter will give the least accurate results.

It has been established in laboratory tests that changes in registration for two pipe sizes will be minor if, for both pipe sizes, the propeller diameter is 75 percent or more of the pipe diameter. In all cases, registration errors will be less in rough-wall pipe because the rough wall helps to establish quickly a fully developed velocity profile.

Even when it is established that the differences in velocity profiles intercepted by the propeller will be negligible, the meter must be corrected (change in gear ratio or other means) to account for the change in pipe diameter.

Propeller Motion

Since the meter head, in effect, counts the number of revolutions of the propeller to indicate the discharge, any factor that influences the rate of propeller turning can affect the meter registration. Practically all propeller effects are negative; that is, they reduce the number of propeller revolutions which would otherwise occur and result in underregistration. More water is therefore delivered than is indicated or paid for.

Propellers are usually designed to turn on one or more bearings. The bearings are contained in a hub and are protected from direct contact with objects in the flow. However, water often can and does enter the bearing. Some hubs trap sediment, silt, or other foreign particles, and after these work into the bearing a definite added resistance to turning of the propeller becomes apparent. Some propellers are therefore designed for flow-through cleaning action so that particles do not permanently lodge in the bearings.

Silt has been found to be particularly damaging to bearings. It is present in many flows and as a result many otherwise satisfactory meters have been rendered unfit for service. Figure 43 shows the wear in the worm gear teeth of an inline meter laboratory tested for 2,200 hours in fairly clear water.

In another laboratory-controlled test a flow containing 5,000 parts per million of silt and very fine sand (0.005-0.15 mm) was used to test a medium-size open-flow propeller meter. In less than 2 hours of operating time silt particles had collected in the bearing and produced propeller binding. After 5 acre-feet of water had passed the meter, 22 grams of silt (dry weight) were found in the propeller hub and bearing housing (a 50-cent piece weighs about 12 grams). A check of the propeller showed that it turned intermittently and slower than when new, in water flowing at 0.5 ft/sec and less.

Figure 43. Flowmeter gear teeth worn on one face—2,200 hours of operation. Photo P368-D-21251

After 45 acre-feet of water had passed the meter, the unit was disassembled; bearing wear was easily visible. Holes were drilled through the bearing assembly. Less silt was then found to accumulate. In effect, when the hub could be drained, less bearing damage was evident, but bearing wear was not eliminated. Sand traps may be necessary in field installations to reduce the amount of sediment (bedload) reaching the meter.

Care should be taken in lubricating meter bearings. Use of the wrong lubricant (perhaps none should be used) can increase the resistance to propeller motion, particularly in cold water. It should also be established that the lubricant is reaching the desired bearing or other surfaces after it is injected. For some meters, the manufacturers did not recommend lubrication of the bearings.

Although propellers are designed to pass (to some degree) weeds, moss, and other debris, there is a limit to the amount of foreign material that can be tolerated in the flow. Even moderate amounts of floating moss and/or weeds can foul a propeller unless it is protected by screens. Heavy objects can break the propeller. With larger amounts, or certain kinds, of foreign material in the water even screens may not solve the problem.

Meter Screens, Sand Traps

Screens to protect meter propellers are usually designed for a particular type of turnout on a particular canal, and to handle a particular type and size of debris; however, screens have certain common features which seem to be universally desirable and which help to prevent head losses across the meter and improve the quality of water measurement.

Screens usually consist of a metal frame, covered with wire mesh, which fit into a slot at the upstream end of the turnout. Double screens (set a foot or more apart in small turnouts) are usually desirable so that protection is provided while one screen is being removed and cleaned. The wire mesh usually varies from 1- by 1-inch (No. 9 wire) to 1/4-inch galvanized hardware cloth. Openings of one-half inch seem to be most successful and popular. Another common size is No. 5 by 5, or 5 by 4 mesh, and No. 19-20-gage wire.

Small screens may be set on a slope, but larger screens should be set in the vertical position so that a winch (sometimes portable) can be used to raise and lower the screen for cleaning. Cleaning may be done by broom, wire brush, or water jet. Reverse flow through the screens may also be used, but provisions must then be made for wasting the cleaning water. In large turnouts from canals, traveling screens may be used to remove debris and reduce the trash problems for several meters in the turnout.

Screen area should be a minimum of 8-10 times the area of the flow cross section; in many installations the screen area is 15-20 times the flow area and this has not been found excessive. Where sizable head losses cannot be tolerated, the screen area should be large, the cleaning frequent, or both.

Sand traps, to catch the bedload (sand and gravel that moves along the bottom), should be arranged so that the trapped material can be flushed along the main canal—not into the turnout. Settling basins to trap the larger particles of suspended sediment (suspended in the flowing water) may be helpful at a meter installation. To remove suspended sediment, the velocity of the approaching flow must be reduced to allow sediment to settle out. To accomplish this, fairly large and relatively costly settling basins are required. The advice of an expert should be obtained before considering a facility of this type.

Head Losses

The head loss across a propeller meter is usually considered to be negligible, although there is evidence that losses for open-flow meters may run as high as two velocity heads. This is equivalent to 0.6 foot of lost head in a 24-inch-diameter pipe carrying 8 cfs. The losses for certain inline compound and other type meters may be as high as 6 to 8 feet of head. In general, however, losses are low but measurements of loss for all meters have not been made.

In many cases turnout losses including losses through the pipe entrance, screens, sand trap, pipe, etc., are large enough to make the losses at the meter seem negligible. Some allowance for meter losses should be made during turnout design, however, and the meter manufacturer can usually supply the necessary information.

Meter Accuracy

The accuracy of most propeller meters, stated in broad terms, is within plus or minus 2 to 5 percent of the actual flow. Greater accuracy is sometimes claimed for certain meters and this may at times be justified. On the other hand, it is sometimes difficult to repeat calibration tests under controlled conditions in a laboratory within plus or minus 2 percent. A change in lubricating practice or lubricant, along with a change in water temperature can cause errors of this magnitude. A change in line pressure (the head on the turnout entrance) can cause errors of from 1 to 2 percent.

Effect of Meter Setting

The setting of the meter in the turnout may be responsible for sizable errors if the meter is not carefully positioned. A meter (24-inch-diameter pipe, 12-inch-diameter propeller, 8-cfs discharge) set with the hub center 1 inch off the center of the pipe showed an error of 1.2 percent. When the meter was rotated 11.5° in a horizontal plane (1/4 inch measured on the surface of the 2-1/2-inch-diameter vertical meter shaft housing), the error was 4 percent; for 23°, the error was 16 percent (underregistration). Setting the meter (shaft housing) in a nonvertical position would introduce the same degree of error.

Effect of Initial Counter Setting

Meter manufacturers recognize that meters tend to underregister after they have been in use for a time and some meters are set to read 101.5 percent of the actual flow as their initial registration. This is done in anticipation that the meter will read correctly (100 percent of the actual flow) during the middle portion of the meter's life. Meters which are readjusted to record a particular flow (the lower end of the scale) with greater accuracy may cause registration errors of up to 10 to 15 percent at greater flows (the high end of the scale).

Effect of Rapidly Varying Discharge

Meters are most accurate when a near constant discharge is to be measured. Considerable error can be introduced by varying the flow rate greatly or quickly. Registration accuracies may vary from 97 to 102 percent of true measurement as a result of continually varying the flow over a significant range during the measurement period. The greatest error always occurs for low flows (at the lower end of the meter capacity scale). Propeller meters are always most dependable and accurate when used in uniformly flowing clean water and a closed system.

Effect of Turnout Design

The exact position of the meter in the turnout and the arrangement of the turnout are responsible for sizable differences in meter registration. Since the relative location of the meter with respect to the entering flow can vary, the intercepted velocity profile can vary and different meter registrations can occur. Figure 44 shows the range of accuracies that exist for three different types of turnouts.

The geometry of the outlet box downstream from the flowmeter may also affect meter accuracy. If the outlet is sufficiently constricted to cause turbulence, boils, and/or white water, the meter registration may be affected.

Curve 1 of Figure 45 shows a meter calibration where the pipe discharges into a large open box that has no backwater or other effect on the meter except to keep it submerged. Curve 2 shows the calibration for the same meter using the outlet structure shown in the sketch of Figure 46. This outlet structure (shown to scale) is believed to be the smallest that can be built without significantly affecting the meter calibration. The vertical step is as close to the meter as is desirable. Larger outlet structures—those providing more clearance between the meter and the vertical step—would probably have less effect on the registration. More rapidly diverging walls (in plan) should be avoided since they tend to produce eddies over the meter and/or surging flow through the turnout. This has been observed as a continuously swinging indicator hand which follows the changing discharge through the meter. The surging may often be heard as well as seen. As previously discussed, large registration errors can occur when rapidly or continually changing discharges are being measured.

Meter Costs, Maintenance

Propeller meter and maintenance costs are difficult to state in terms of dollars, but some relative figures may

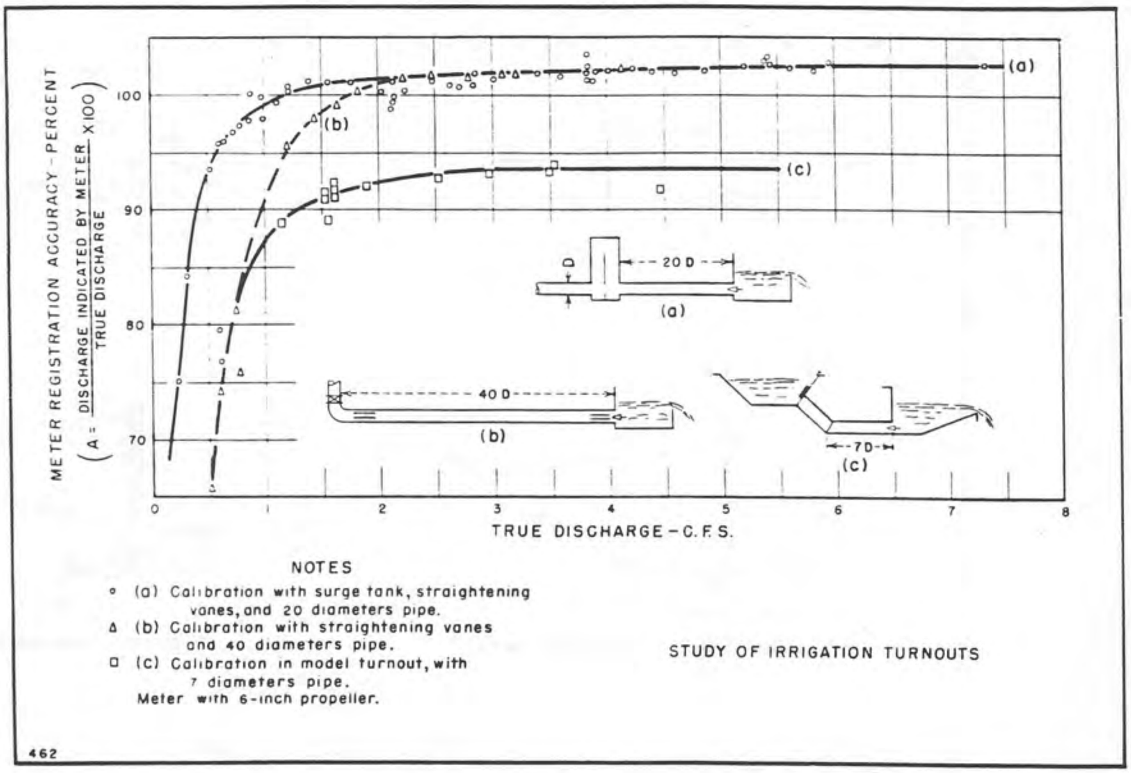

Figure 44. Effect of meter setting on registration accuracy.

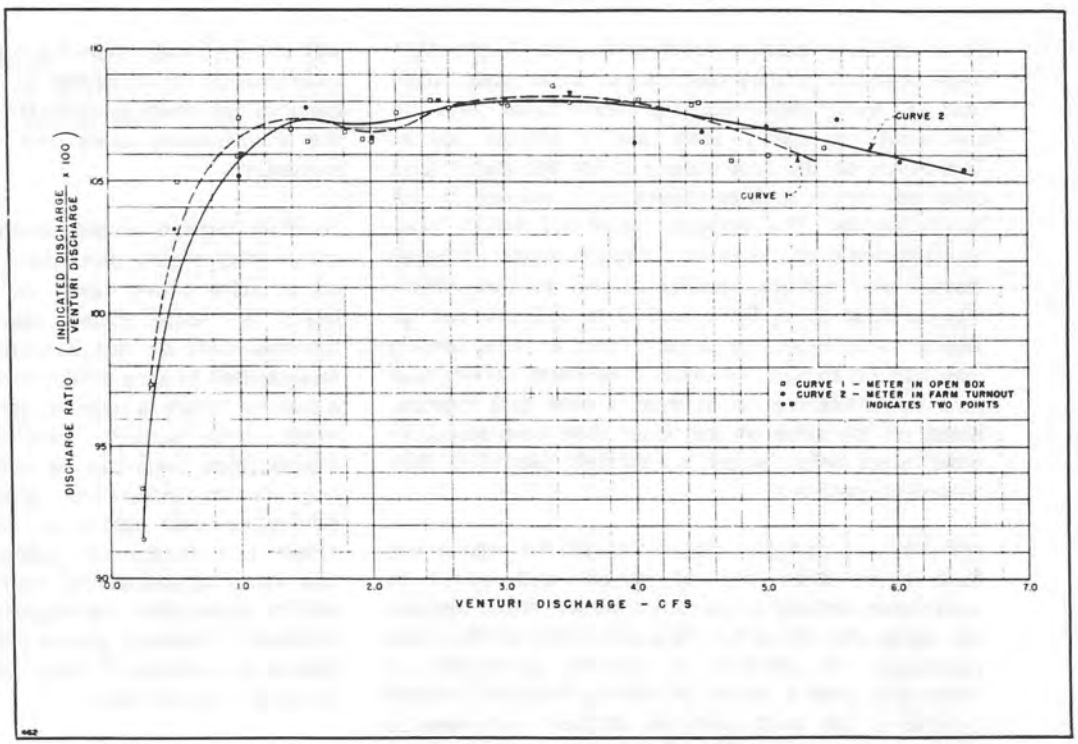

Figure 45. Calibration curves—Open flowmeter turnouts.

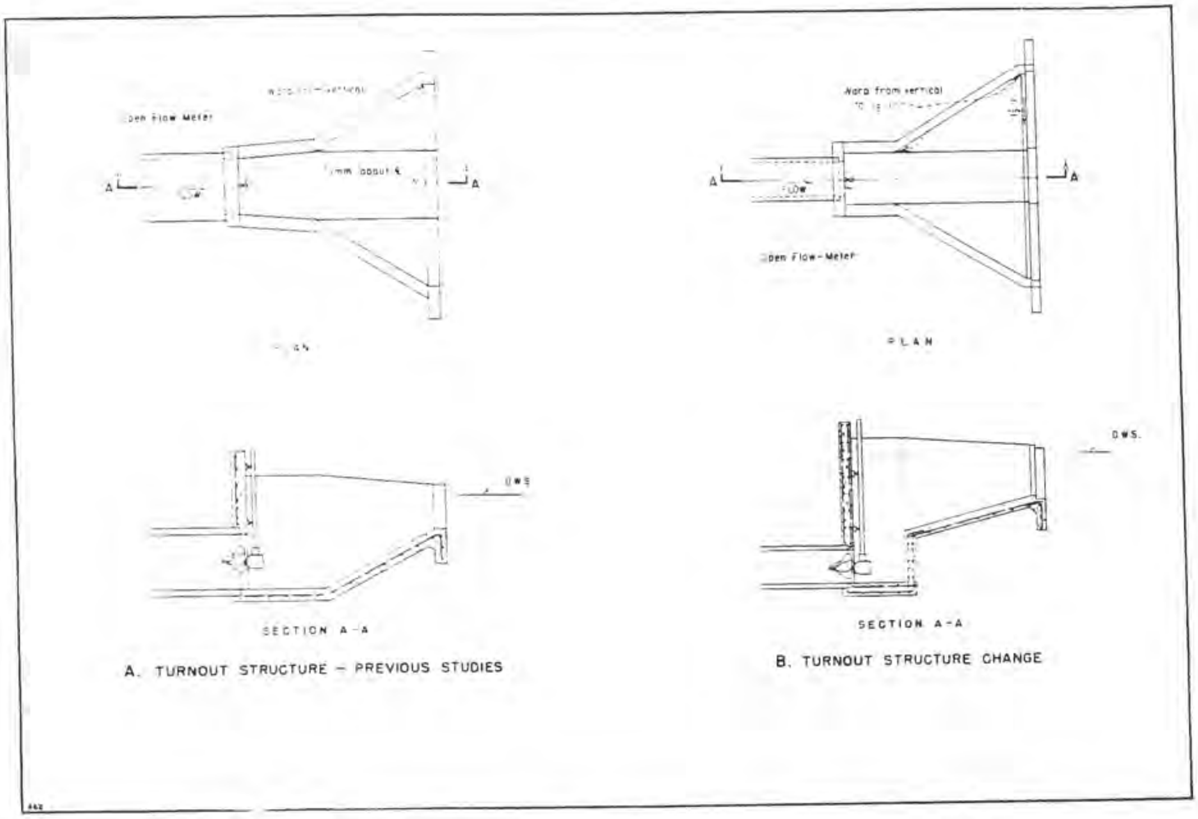

Figure 46. Open flowmeter turnout structures.

be of value in making rough estimates. A propeller meter installation may cost two or three times more than a weir, depending on labor rates, and be somewhat less costly than, say, a Venturi meter. Two-thirds of the cost usually is for the meter (and other equipment such as screens, etc.) and one-third is for installation. The propeller meter will require more maintenance than a weir or a Venturi meter. Propeller meters may require continuous maintenance which may amount to as little as $10 to $25 per year or several times more. In some meters a single bearing may cost up to $75. To offset these costs, meters have paid for themselves in as short a time as 2 months, based on the value of the water they have saved. In other areas where water is relatively plentiful, they have never paid out.

Hundreds of propeller meters bought for regular use have been taken out of service and stored in warehouses because of various troubles, either because the meter did not serve the purpose for which it was purchased, or because it became unreliable or inoperable after a period of service. Propeller bearing trouble is the most common problem and may be difficult to overcome except by means of a well-planned maintenance program. In districts where maintenance is accepted as inevitable, and where bearings and spare parts are stocked for immediate use, the maintenance costs and problems seem to be minimum.

In other districts where personnel are unfamiliar with meter mechanisms, and where spare parts are ordered on a one-at-a-time basis, the maintenance costs are high. In some cases, users expect to replace bearings—they do not consider the need for bearing replacement to be a defect in the meter; in other areas a bearing failure is cause for permanently removing the meter from service. Experience has shown that maintenance costs can be reduced by establishing a regular maintenance program which includes lubrication and repair of meters; screen cleaning, repair, and replacement (about every 2 years); sand trap cleaning; and general maintenance of the turnout and its approaches. In a regular program many low-cost preventive measures can be made routine and thereby reduce the number of higher cost curative measures to be faced at a later time.

Choice of Meter Size

Many meters have been retired from service without ever having accomplished their original purpose, simply because a larger than necessary meter was purchased and the meter was not able to record the usual smaller daily flows. In attempting to use an existing turnout pipe (pipe sizes may not have any relationship to the discharge to be measured) a large meter was purchased to fit the existing pipe. The meter could then handle the maximum possible flow through the turnout, but was too large to handle the small flows that were the usual daily requirement. In some districts it has been necessary to state the minimum flow that can be delivered and measured; the user is then expected to arrange his water use so that smaller discharges are not necessary. Care should be taken not only to match the meter to the pipe size but to match the meter to the proper discharge range. It may be necessary to reduce the turnout pipe size as a result, but the savings in purchasing a smaller meter might help to offset this cost. If possible, the meter size should be selected so that usual operation will occur in the midrange of the meter.

The velocity in the pipe should be above 1.5 feet per second for best performance. If sediment is present in the water, the velocity should be even higher to minimize the added friction effect produced by worn bearings. A meter that operates continually in the lowest range (or highest) will not be as accurate as one that operates in midrange.

SPECIAL MEASURING DEVICES AND TECHNIQUES

Vane Deflection Meter

A portable vane flowmeter is on the market and, according to the manufacturer's claims, the meter is accurate and useful. The meter has been evaluated from comprehensive tests made under simulated field conditions in the Hydraulic Laboratory and the claims of the manufacturer were found to be quite truthful. The meter is indeed an accurate and useful device.

The portable deflector vane rests in permanent brackets mounted in a 6-foot-long ditch liner, either rectangular or trapezoidal in cross section, set in an earth ditch. Therefore, one meter head will service any number of ditches of the same general flow capacity having liners and brackets permanently installed. About 30 sizes of meter and ditch liner are available. Each meter handles a wide range of flows in a given size of ditch and automatically compensates for different combinations of velocity and depth. There is negligible loss of head caused by the ditch liner or meter. Installation is simple and the cost is reasonable, especially if several or more ditches can be served with one meter. Instantaneous discharges may be read and if an available special recorder is purchased, the total delivery may be recorded.

Since the meter works on the deflection principle, wind effects on the exposed portion of the meter can cause serious measurement errors unless precautions are taken. A wind break made from a piece of plywood was found to be effective in minimizing wind-caused errors.

Under ideal conditions the meter was found to be accurate to 1.6 percent, and to about 3 percent under less favorable conditions. Wind produced errors of up to 100 percent, but simple precautions eliminated practically all of this error.

The meter is durable, well constructed, and should retain its original factory calibration indefinitely. Interchangeable calibrated scales are available from the manufacturer to indicate cubic feet per second, gallons per minute, miners inches, acre-feet per day, etc.

Dilution Method

In making the usual water measurement, it is necessary to measure head, velocity, cross-sectional area, depth, meter revolutions or some factor(s) that may be difficult to measure, because the measurement must be made in, on, over, or beneath the flowing water. One method of determining discharge that circumvents the need for making difficult mechanical measurements is the dilution method. In this method a substance in concentrated form is introduced into the flowing water and allowed to thoroughly mix. At a downstream station a sample is taken, and from the degree of dilution of the concentrate, the discharge is computed. Since only the quantity of water necessary to accomplish the dilution is involved, there is no need to measure velocity, depth, head, cross-sectional area, or any of the other hydraulic factors usually considered in a discharge measurement.

Figure 47 illustrates schematically the use of the dilution method. A relatively small quantity of chemical or dye, called a tracer, is dissolved in a small quantity of water and placed in a bottle so that the tracer solution can be discharged into the flowing water at a known rate. A canal is illustrated, but a pipe or pressure penstock could also have been shown. In either case, the discharge to be measured is referred to as Q.

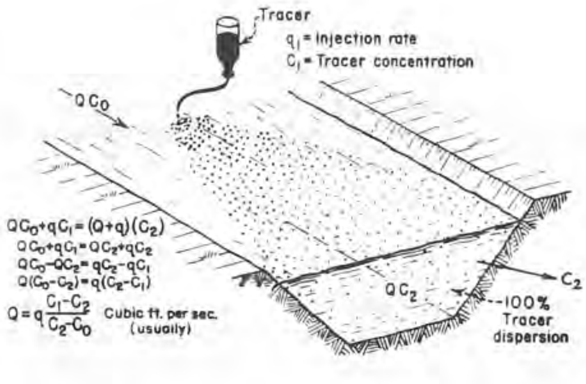

Figure 47. Dilution method of determining discharge. Photo PX-D-43865

The concentration (weight of tracer/weight of water) of tracer in the bottle is a ratio C_1. The rate of injection (cubic feet per second) is q_1. To account for any tracer which might already be in the upstream flow, the original tracer concentration will be called C_o. At the downstream station shown in the figure, it will be assumed that thorough transverse mixing of the tracer with the flow has taken place. Since no flow has entered the canal during the mixing of the tracer and since no loss of flow has occurred, including seepage losses, the upstream Q is the same as the downstream Q plus the quantity of added tracer. The concentration of tracer at the downstream station is C_2.

The above conditions may be stated in a simple relationship—the canal discharge Q multiplied by the concentration of upstream tracer C_o, plus the injection rate q_1 multiplied by C_1, the tracer concentration in the bottle, are equal to the quantity Q plus q_1 multiplied by the concentration C_2 at the downstream station, or

$$QC_o + q_1C_1 = (Q + q_1)\, C_2$$

or

$$QC_o + q_1C_1 = QC_2 + q_1C_2$$

Rearranging terms

$$QC_o - QC_2 = q_1C_2 - q_1C_1$$

Simplifying

$$Q(C_o - C_2) = q_1(C_2 - C_1)$$

or

$$Q = q_1 \frac{(C_2 - C_1)}{(C_o - C_2)} \qquad (8)$$

In effect, Equation (8) states that the discharge in the canal (pipe, conduit or other) may be obtained by multiplying the injection rate of the tracer by a ratio obtained from three concentration values. There is no need to know or measure canal velocity, depth, cross section or any of the usual hydraulic elements. A thorough understanding and realization of these facts is the key to understanding the dilution method of measuring discharges.

Many different substances have been used in dilution tests. Chemicals, including ordinary salt, have been used and the dilution has been evaluated by the change in the ability of the flowing water to conduct an electric current. Or chemicals such as sodium dichromate have been utilized and the quantity in a sample determined by means of chemical analysis or flame photometer determination. Sodium dichromate has been recommended for use in many waters because it is stable and chromium ions do not ordinarily appear in natural waters. Dyes have been used to color the water and colorimeters and comparators have established the dilutions by comparing the samples with standard concentrated or dilute solutions.

Fluoresein, Rhodamine B, or Pontacil Pink dyes have been used. The latter two are very stable, do not occur in nature, and can be simply and accurately detected by means of a fluorometer in very low concentrations (several parts per billion). New dyes are being developed to fill the needs of dilution testing. Rhodamine WT (water tracer) is one of the newer dyes found to give better response in a flurometer.

Radioisotopes are used in open-channel and closed-conduit dilution measurements and the degree of dilution is determined by counting the gamma ray emissions from the diluted isotope solution (the downstream flow) using Geiger counters or scintillation counters. Since the counters can at best account for only the emissions in the sphere of influence surrounding the counter device, and since the understanding of the radioisotope method involves complete comprehension of the laws of probability and of mathematical and physical derivations beyond the scope of this paper, no attempt will be made here to explain directly the theory behind the radioisotope method of making discharge measurements. However, considerable comprehension of the method may be had by understanding the principles involved in making a dilution discharge determination, Figure 48.

Radioisotopes may be used in the dilution or in the "pulse" or "total-count" method. A known amount of Radioisotope A (corresponding to C_1v_1 where v_1 is the volume of injected tracer) is introduced into the flow in a relatively short time. At the measuring station

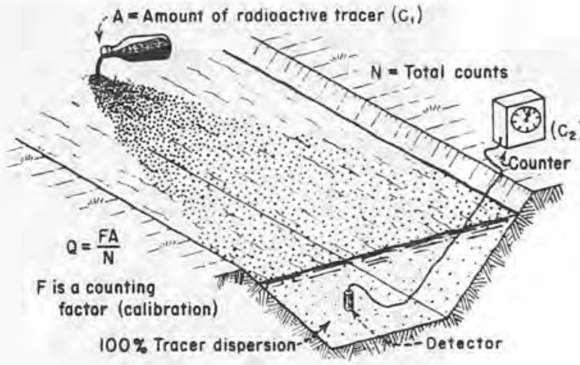

Figure 48. Radioisotopes total count method of measuring discharge. Photo PX-D-43066

downstream where the radioisotope is thoroughly mixed, the concentration of the radioisotope tracer is determined from the gamma ray emissions detected and counted by Geiger-Muller or scintillation detectors. Where C_2 was a constant of concentration in the chemical dilution, Equation (8), the concentration of radioactivity in the pulse is a variable with respect to time. Thus, C_2 in the latter case must be measured with respect to time. In mathematical terms this may be written

$$Q = \frac{C_1 v_1}{\int C_2 dt} = \frac{A}{\int C_2 dt} \tag{9}$$

In preparing for a discharge determination test, the Geiger or other counters must be calibrated, taking into account the exact conditions under which the counters will be used. The counters are submerged in a large container filled with a mixture of water and radioisotope of known concentration. The container must be large enough that any further increase in volume would not change the counting rate. Determination of the counting rate in this manner simulates the action of the counter in a canal where the container is, in effect, infinite in size.

Since the total number of gamma rays (counts) N, counted during the passage of a pulse of tracer is

$$N = F \int C_2 dt \tag{10}$$

where F is the calibration or correction coefficient for a specific counting system in a specific location substitution of (10) into (9) gives

$$Q = \frac{FA}{N}$$

in which

Q = volume per unit time (cubic feet per second)

F = counts per unit of radioactivity per unit volume per unit of time (counts per second) (millicuries per cubic feet)

A = total units of radioactivity to be introduced for each discharge measurement (millicuries)

N = total counts (number corrected for natural radiation background)

The greatest deterrent to obtaining consistently accurate measurements has been the inability to obtain thorough transverse mixing. Work on this phase of the problem is continuing, both as a research subject in the laboratory and as a test problem in the field. Tests on devices to produce or introduce turbulence into canal or pipe flows have not been made, mainly because every attempt is being make to keep the measuring procedure and equipment as simple as possible. However, it may be necessary to provide turbulence-inducing injectors which utilize an external source of energy to start the eddy action and promote transverse mixing. If this can be done with simple and reliable equipment, the problem of adequate mixing will be greatly simplified.

The accuracy of the results achieved in using isotopes to measure canal discharges in over 100 tests has, in general, been encouraging. If thorough transverse mixing of the isotope with the flow can be assured, measurement accuracy to within plus or minus 1 percent can be achieved. Comparisons of isotope discharge determinations with "operational" discharges, current meter measurements, or other means, have been used to evaluate the isotope tests. It is believed that the isotope method shows real promise of becoming a routine method having accuracy limits which are comparable to other devices and methods. Figures 49, 50, and 51 show the detection and counting equipment in operation on medium-sized canals. The radioisotope discharge method has been expanded and injection equipment developed to introduce radioisotopes into pipelines or penstocks flowing under high heads. Pipe capacities or turbine or pump efficiencies can be determined by the radioisotope discharge method.

Acoustic Flowmeter

Commercial development of acoustic flowmeters has progressed rapidly since the practical nature of the method was realized about 1950. Systems for small pipes (up to 10 inches) have been available, but in 1964 a system was installed and experimentally evaluated in a 24-foot-diameter steel penstock at Oahe Dam on the Missouri River with satisfactory results. An

Figure 49. Turbulent water aids dispersion and mixing of the radioisotopes and water. Photo P212-D-36460NA

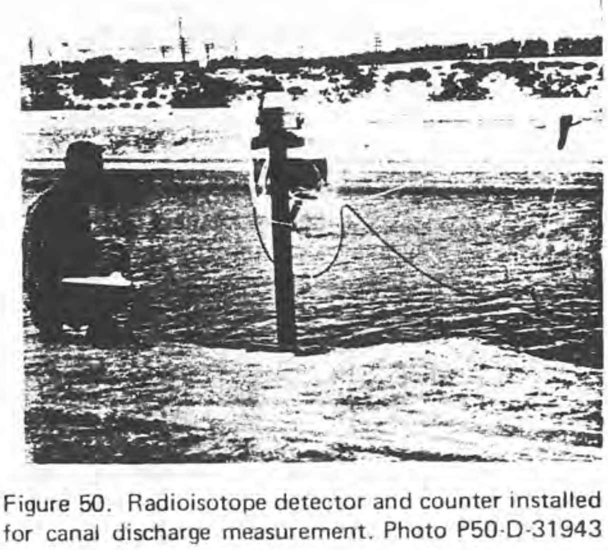

Figure 50. Radioisotope detector and counter installed for canal discharge measurement. Photo P50-D-31943

open-channel acoustic flowmeter has been installed in the Delta-Mendota Canal of the Central Valley Project by Government agencies. Research and development of the system shows promise that the principle utilized in the pipe systems may be satisfactorily applied to most open-channel discharge measurements. Comprehensive studies of both open-channel and closed-conduit types of acoustic meters are continuing with the expectation that the method can be applied to measuring large

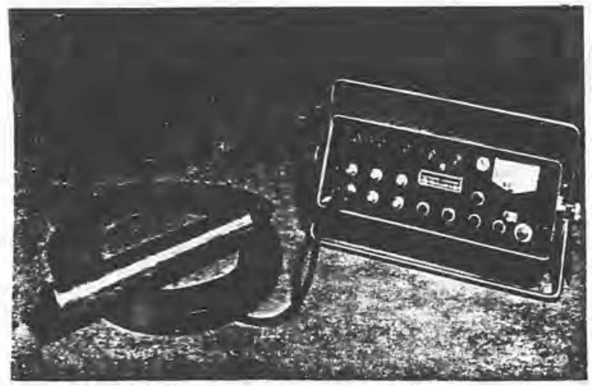

Figure 51. Radioisotope detection and counting system used in Bureau of Reclamation tests. Photo PX-D-59217

discharges in both open-channel and closed-conduit systems.

Acoustic flowmeter systems are expected to offer, eventually, a relatively inexpensive, rapid, and reliable method of measuring discharges in large pressure conduits and in large canals. The system uses the principle that the difference in time of arrival of two acoustic (pressure) pulses traveling in opposite directions through the water can be related to the water velocity. Figure 52 shows a schematic arrangement of an acoustic meter installation in a pipeline. In one direction the waterflow velocity increases the speed of the acoustic pulse ($C + V_W$) while in the other direction the flow velocity delays the arrival of the pulse ($C - V_W$). Acoustic transducers are used to transmit and receive the acoustic pressure energy along oblique upstream and downstream paths in the channel or conduit. The accuracy of the system depends on positioning the transducers to obtain a true average velocity in the pipe or canal. Several pairs of transducers may therefore be required to obtain the true average velocity if the velocity profile in the conduit is not symmetrical about the centerline of the conduit. In an open channel the transducers must be raised or lowered as the flow depth changes to compensate for a changing velocity profile.

Acoustic systems are being developed that can operate over a range of conduit diameters or open-channel path lengths. The power supply, transducers, and velocity readout components can be made compact for ease of installation in the structure. Power requirements have been reduced so that a 110-volt supply source can be used. There are indications that velocities can be measured with 1 percent accuracy. Consequently, discharges may be determined with considerable accuracy depending on the velocity profile at the measuring station.

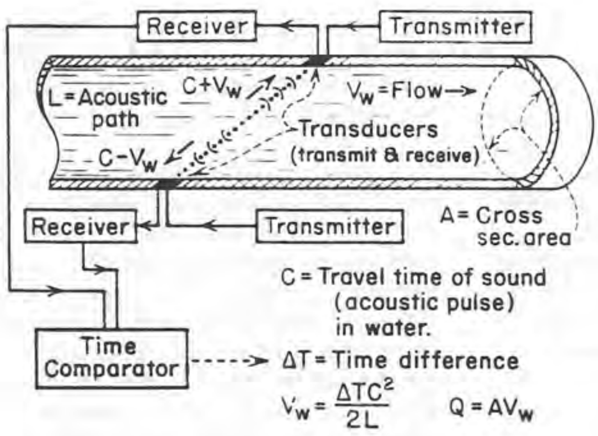

Figure 52. Acoustic flowmeter installation.

Laser Flowmeter

Laser (light amplification by stimulated emission radiation) beams have been used for studying the turbulent characteristics of flowing liquids and for determining the velocity of fluid flow. As in the acoustic meter, the Doppler principle is used in comparing the shift in frequencies. The flowing water scatters part of a beam of light (laser) directed through it. By comparing the frequencies of the scattered and unscattered rays, collected in receiving lenses on the opposite side of the stream, the velocity of the water (hence the discharge) can be calculated. In laboratory experiments the instrument has measured fluid flows as slow as a fraction of an inch per second and as fast as 5,000 feet per second. The device is a valuable research tool but should be considered a possible future water-measuring device.

Magnetic Flowmeter

Magnetic flowmeters are used commercially in chemical and process industries to measure and proportion liquids used in manufactured products. Recently with the increased cost of irrigation, municipal and industrial water, magnetic flowmeters are being considered for use as turnout measuring devices.

The operating principle of the meter is based on Faraday's law of induction, in that the voltage induced across any conductor moving at right angles through a magnetic field is proportional to the speed of the conductor. The principle is the same as used in direct- and alternating-current generators.

In the flowmeter, water flows through a nonmagnetic tube which is installed as a portion of the pipeline, and surrounded by a magnetic field, produced by an electromagnet. The flowing water induces an alternating-current voltage between two electrodes located in the tube walls. The voltage produced is proportional to the speed of the water in the tube and is used to indicate and record the rate or volume of flow. These meters are capable of measuring velocities of up to 30 feet per second (fps) through the tube.

Magnetic flowmeters are supplied in sizes ranging from an inch or less to several feet (diameter of the nonmagnetic flow tube). Development of the meter is continuing and considerations are being given to installing electromagnetic coils and electrodes on the inside surfaces of existing concrete pipes to form large flowmeters for use in pumping plants or distribution systems.

Accuracy of the smaller flowmeters has been shown by calibration to be relatively high. For velocities less than 1-fps accuracy is plus or minus 3-4 percent; 1-3 fps, plus or minus 2 percent; 3-30 fps, plus or minus 1 percent. Calculations indicate that the accuracy of large meters should also be high and attempts are being made to establish the accuracy of meters having diameters of 10 to 15 feet. The discharge measurement range of any particular meter can be extended by suitable switching in the electronic controls, to as much as 30 to 1, a definite advantage in most installations. Other types of meters usually have a 10 to 1 range. A minimum velocity of about 0.5 fps through the flow tube is desirable for producing a sufficiently high voltage for discharge measurement. The meter will operate below the 0.5-fps range but with decreased accuracy.

Contrary to most meters little effect on the accuracy of the flowmeter is caused by elbows, gates, or valves located within a few pipe diameters upstream from the entrance to the meter flow tube, or near the outlet of the tube. The head loss in the system caused by the meter is negligible; is no more than the loss produced by the same length of pipe of similar roughness.

Radial Gates

Radial gates are increasing in importance as movable devices for controlling and measuring water. The gates are incorporated in schemes of controlling canal flow by automation. The depth of flow is measured upstream of the gate. The measurement, converted to an electrical signal, can be used to control either the downstream or upstream gate of the canal reach.

For water measurement or control, knowledge is needed of the head-discharge relationship and physical characteristics of the gate structure. General information on gate capacity is available from texts, and reports are available on specific installations.[35, 36, 37]

Information may be needed on both free- and submerged-flow conditions of gate operation, Figure 53. In canals, the radial gate normally operates with the discharged water submerged by a checked depth from downstream. Planning for discharge measurement or control requires a matching head-discharge relationship. When specific information cannot be obtained, in-place calibrations of the gate may be performed by current meter or other comparisons.

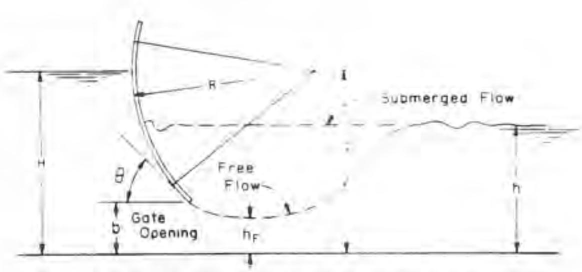

Figure 53. Radial gate parameters.

A general form of the discharge equation:

$$Q = C_d\, b\, B\sqrt{2g\, H - h_s}$$

in which

C_d = Coefficient of discharge

B = Gate width

h_s = 0 in free flow

may be used in establishing a head-discharge relationship. Careful application of the equation and measuring of the heads should provide a reliable way of using a radial gate for discharge measurement.

Measuring, Indicating, and Flow-Controlling Meters

Rising labor rates and the increasing value of water has accelerated the trend toward automated distribution systems and has helped make pipe delivery systems competitive with open ditches in some areas. Pipe systems often operated under high head and need a pressure-reducing device, control valve, and flowmeter to make them practical. Automatic and/or remote control, along with indicating and recording devices on the flowmeter are also desirable. Devices that eliminate, reduce, or combine these essential functions will reduce the cost of a turnout and its maintenance, and will find ready acceptance if they are dependable.

Two devices, utilizing the tapered-tube variable area principle, have been investigated in the laboratory and one has undergone short-duration field tests in poor-quality water (dissolved salts and silt). The meters indicated a greater than usual head loss and some technical difficulties that limited their expected performance. Both had a so-called "float" (actually a weight) which moves in the tapered tube to provide more flow area when the discharge is increased. The position of the float in the tube is used to indicate the quantity of water flowing. In one meter a rod attached to the float was used to actuate a punch which punched holes in paper tape to record the flow. The tape could be put through decoding and billing machines to simplify customer-paying procedures.

In the other meter a pilot device controlled a large main valve which maintained a constant discharge (limited to 1 to 3 cfs) for a head range of from 7 to 175 feet, while indicating and totalizing the discharge. Pilot control could be manual or by remote wire or wireless signal. These valves, and others expected to be designed and promoted, are expensive by ordinary standards, but are certain to find a market in thriving agricultural areas. Although they are new and relatively untested under adverse field conditions or for long periods of time, they are being installed in some areas. More and more of these devices are certain to be developed as automation continues. There is no technical reason why these devices cannot be made to perform satisfactorily.

CONCLUSIONS

The accurate measurement of water is both an art and a science and requires the full understanding of the irrigation operator. To achieve success, the operator must understand the workings of his water-measuring equipment and must apply his knowledge in a practical way to the degree necessary to achieve the desired accuracy of measurement. He must be alert to ever-changing flow conditions and make the necessary alternations in equipment or procedures. He must exercise proper judgment in all matters and this is best accomplished when as many facts as possible regarding the behavior of water are known to him.

It is difficult, if not impossible, to establish definite rules which apply generally to water-measurement procedures and equipment. Similarly, one measurement device cannot be recommended over any other device until all variables at the particular installation site are considered and properly weighed. It is therefore necessary for each operator to learn as much as possible about the device he is using and to evaluate the effect of each variable (at the particular site) on the measurement he is making.

Each operator must learn to look objectively at his equipment and procedures. He must be able to "see" that his equipment is rundown and in need of maintenance or that his measurement procedures are not compatible with what he is trying to measure. He should become familiar with various types of measuring equipment, learn the advantages and disadvantages of each, and decide whether the existing equipment is the best for the job at hand. He should try to find fault with his equipment and every step he uses to make a discharge measurement, and try to improve wherever possible. This means that he must understand the basic measurement he is trying to make and then modify, if necessary, his methods of getting it. He should try to understand why he is doing the things he does and develop confidence in his knowledge. He should read available literature as much as possible to get background information on water measurement. He will thereby not only obtain more meaningful information, but will also have the satisfaction of knowing his job is well done.

REFERENCES

The following publications are suggested as an aid in acquiring background in water-measurement practices. The items have been selected to provide practical help or background information, or both, and should be of value to both new and experienced personnel. Copies may be obtained from a public library or from the sources listed. The textbook chapters referred to are not difficult to read and will supply background material in gage-reading problems and examples, orifice theory, gate and meter problems, and information on head losses and other flow phenomena.

The Bureau reports contain practical information pertaining to discharge measurements through metergates and constant-head turnout orifices which can be applied to field problems directly. In fact, a complete analysis of the flow through screw-lift vertical metergates is contained in these reports and may cover specific problems encountered in your area. The report on the constant-head orifice turnout contains calibration data and operating instructions.

The textbooks are available in most public libraries and the reports should be on file in your Bureau Regional Office or Project Office, or can be obtained from the Technical Services Branch, Bureau of Reclamation, Denver Federal Center, Denver, Colorado 80225.

Items 1, 2, and 8 are valuable as handbooks and would be of permanent value as reference books. The principles discussed will be found helpful in understanding and operating almost every type of measuring device. Item 8 contains a wealth of information on stream-gaging techniques developed by USGS.

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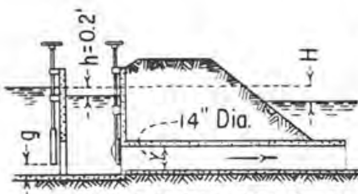
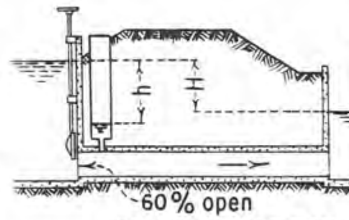


Q	H	g
3	0.32	30
5	0.48	50
10	0.93	99

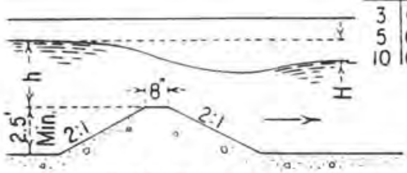
g = Gate open, %

18-BY 24-INCH CONSTANT-HEAD ORIFICE TURNOUT
(DOUBLE BARREL)



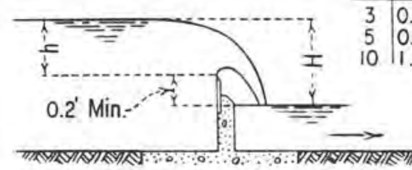
Q	H	h
3	0.30	0.31
5	0.81	0.82
10	3.39	3.46

15-INCH METER GATE



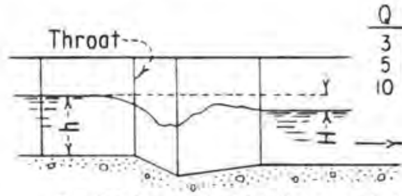
Q	H	h
3	0.30	0.38
5	0.42	0.53
10	0.66	0.82

TRAPEZOIDAL WEIR
(CREST LENGTH 4.0 FEET)



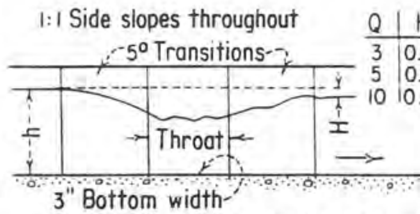
Q	H	h
3	0.58	0.38
5	0.73	0.53
10	1.05	0.85

CONTRACTED RECTANGULAR WEIR
(CREST LENGTH 4.0 FEET)



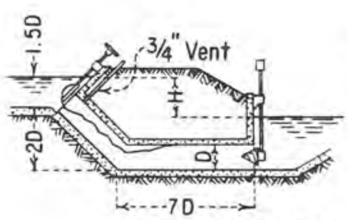
Q	H	h
3	0.20	0.53
5	0.25	0.74
10	0.36	1.15

2-FOOT PARSHALL FLUME



Q	H	h
3	0.11	0.94
5	0.14	1.17
10	0.19	1.60

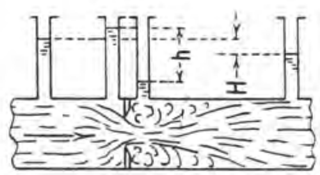
TRAPEZOIDAL VENTURI FLUME



Q	H	D
3	1.64	12
5	1.77	14
10	2.19	18

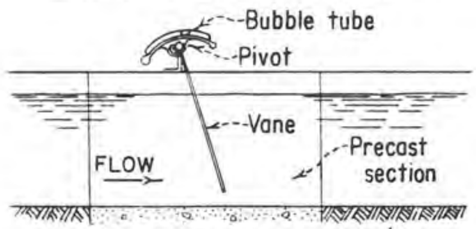
D = Pipe dia. in inches

OPEN FLOW METER*



Q	H	h
3	6.8	9.4
5	18.7	26.0
10	74.2	103.0

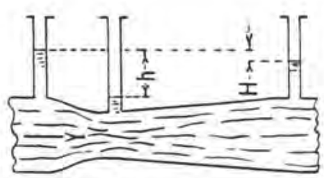
6-INCH ORIFICE IN 12-INCH PIPE



DEFLECTION METER*

FRESNO FLOWMETER

Q	H	h
3	0.38	1.00
5	1.10	2.75
10	4.50	12.50



Q	H	h
3	0.02	1.90
5	0.06	5.28
10	0.25	21.14

12-INCH VENTURI METER

NOTES
Q = Discharge in second-feet.
H = Head required in feet.
h = Measuring head in feet.
* = Substantially no head required to operate.

COMPARISON OF OPERATING AND MEASURING HEADS FOR VARIOUS WATER MEASURING SYSTEMS

103-D-991

ATTACHMENT NO 3

RAMP FLUMES

The Water Conservation Laboratory, Agriculture Research Service, USDA (U.S. Department of Agriculture), developed computer programs for calibrating measuring flumes. The simplest type, the ramp flume, consists of a 3:1 approach ramp up to a horizontal broad sill or crest with a vertical downstream drop back to the canal invert. Ramp flumes (figures 1 and 2 in enclosed report) are simple to form and construct. Also, ramp flumes are easy to install in existing canals.

Ramp flumes have relatively small head losses and are able to tolerate rather high submergence (85 percent of the measuring head for a vertical downstream crest face and 93 percent of the measuring head with an added 6 horizontal to 1 vertical diverging downstream ramp). Therefore, the minimum required head loss for flow measurement is 15 percent of the measuring head on flumes with vertical drops and 7 percent of the measuring head for ramp flumes with 6:1 sloped downstream diverging ramps. Discharge measurement errors increase very rapidly beyond the submergence limit.

In general, ramp flumes are designed for step heights from 40 to 60 percent of the approach canal normal flow depth. Thus the minimum head loss (using 50 percent) that can be allowed and still measure flow is 7.5 percent of the approach flow depth for a ramp flume with a vertical drop and 3.5 percent of the approach flow depth for a ramp flume with 6:1 sloped downstream ramp.

It is difficult to visually determine when a flume is submerged. An estimate of submergence can be made by noting the location of the disturbance wave relative to the downstream end of the crest. To aid operators in making visual determinations of submergence conditions a line should be painted on the channel sides to demark the downstream end of the crest which is hard to see during operation. If a flume commonly operates under high submergence it is recommended that a staff gage be installed downstream of the flume (downstream of the hydraulic jump and where flow has returned back to normal canal flow). Submergence levels can then be calculated from readings of both upstream and downstream water level.

A major advantage of a ramp flume is the ability to numerically calibrate the flume using post-construction dimension measurements. Thus construction errors or flume settlement can be better accounted for.

To insure approximate parallel flow (assumed for numerical calibration purposes) the basic design criteria of approach measuring head (H_1) relative to the length of crest in the direction of flow (L_3) is;

$$(H_1/L_3) < 0.50$$

Also, the head at the measuring station should be greater than one-twentieth of the crest length to assure that undulating flow (caused by frictional control) does not occur on the crest. Thus;

$$(H_1/L_3) > 0.05$$

Accuracy comparisons of field and model calibrations with numerical (computer program) calibrations indicate that numerically-calibrated ramp flumes are as accurate as Parshall flumes (3 to 5 percent, see enclosed report GR-82-14 and Travel Report).

To-date there have been no significant operational problems with sediment at ramp flumes. In new designs, if the normal flow depth required to move the sediment is known, sufficient drop can be included to cause near normal flow both upstream and downstream of the flume. Because flow accelerates, it is generally expected that most sediment traveling in the approaching canal will go over the ramp. However, sediment deposition will also be a function of the approach velocity and the amount of sediment being fed into the canal by wind and other sources. Although predicting sediment deposition is difficult, keeping the approach head close to the normal depth is a good general rule to avoid deposition problems. Designers should note their particular situation and compensate accordingly.

The main construction requirements are that the crest is of proper length in the direction of flow and is level both in the direction of and transverse to the flow. The main calibration requirement is that all the dimensions, especially the crest width and side slopes of the crest and canal sections, be carefully measured after construction. Calibration is especially sensitive to the transverse crest width dimension and side slope. Ramp flumes can be computer calibrated, using the after-construction dimension measurements. Thus, some form slipping and construction errors can be accounted for accurately. Computer calibration allows more tolerance during construction, saving time and cost. Farmers Bulletin Number 2268 by the United States Department of Agriculture (attached) gives some advice concerning construction of ramp flumes.

Approach velocities less than 1 ft/s tend to encourage aquatic pest and insect populations. As a result, the approach velocity should exceed 1 ft/s if at all possible. To prevent wave interference of head measurement the Froude number of the approach flow should be less than 0.5. To provide sufficient measuring head relative to precision of head measurement, the measuring head should be greater than 60 mm (0.2 ft). More information concerning ramp flumes is in the enclosed Report GR-82-14.

Some specification drawings for the Pick-Sloan Missouri Basin Program are attached which show examples of the structural design requirements for ramp flumes built in earth lined canals. These drawings illustrate how ramp flumes can be keyed into the soil.

Rough cost estimates for ramp flumes (proportional to the geometry of the flume studied in Report GR-82-14) are given in the following table. Actual costs will depend on site selection, required downstream erosion protection, and local material and labor costs.

Discharge Ft^3/s	Bottom Ft	Cost $
50	2	2,550
100	3	3,200
300	4	4,400
600	5.5	26,000
1200	7	38,000
2000	9	65,000

For example, a cost estimate of ($97,000) for a large 930-$ft^3/s$ ramp flume at the Klamath Project, Oregon (see the enclosed Travel Report) was higher than above estimates due to special forming requirements on steep side slopes (0.25:1). In this case (retrofit situation) ramp flume costs were still about 45 to 60 percent of that for the Parshall flume. Clemmens and Replogle* cite costs for smaller flumes that are one-tenth to one-third of those for equivalent Parshall flumes. For existing canals, costs associated with canal modifications such as service road relocation or canal transitions are generally much less for a ramp flume.

* Clemmens, A. J., and John A. Replogle, " New Flume Breakthrough for Ditch Irrigation", Irrigation Age, April 1978

GR-82-14

RAMP FLUME MODEL STUDY—PROGRESS SUMMARY

March 1983
Engineering and Research Center

U.S. Department of the Interior
Bureau of Reclamation
Division of Research
Hydraulics Branch

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16. ABSTRACT

A 1:3 hydraulic model was used to study small (irrigation ditch sized) ramp flumes developed by J.A. Replogle and A.J. Clemmens of ARS (Agriculture Research Service), Tempe, Ariz. The accuracy of computer calibration for the model ramp flume was found to be at least as accurate as that for Parshall flumes.

Flumes can be computer calibrated using after-construction dimension measurements. With a vertical drop at the end of the crest, the measuring device has a *submergence depth limit* of 85 percent of measuring depth above the crest. With a ramp slope 6:1 diverging from the end of the crest to the bottom of the canal, the measuring device has a 92-percent submergence depth limit. Laboratory testing results indicate that ramp flume water measuring devices have a strong potential for accurately measuring flow in canal systems.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS-- /hydraulics/ *water measurement/ hydraulic structures/ *flumes/ Parshall flumes/ critical depth/ weirs/ broad-crested weirs/ calibration/ trapezoidal flumes/ submerged flow/ ramp flume

b. IDENTIFIERS--

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GR-82-14

RAMP FLUME MODEL STUDY — PROGRESS SUMMARY

by

R.A. Dodge

Hydraulics Branch
Division of Research
Engineering and Research Center
Denver, Colorado
1982

UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

The research covered by this report was funded under the Bureau of Reclamation PRESS (Program Related Engineering and Scientific Studies) allocation, "USDA Ramp Flume for Discharge Measurement," DR-432. Additional funds supporting this work were furnished by the OCCS (Open and Closed Conduit Systems) committee.

ACKNOWLEDGMENTS

This ramp flume study was conducted in the Hydraulic Research Section, Hydraulics Branch, Division of Research. It was supported by the Bureau's OCCS (Open and Closed Conduit Systems) Committee and PRESS (Project Related Engineering and Scientific Studies) funds. In addition to the author, the model was operated by and data were obtained by Jim Vandeveer, Dave Dollar, and Leo Baca. Jerry Martin did concept design computations [to help designers determine dimensions for a possible ramp flume for the Charles Hansen Feeder Canal (Colorado Big Thompson Project)] and computer studies on the effects of variable changes. John Replogle provided E. James Carlson, who supervised these studies, two computer programs for obtaining calibrations. Jerry Martin modified one of the programs to output appropriate warnings when certain design criteria are violated. Final editing and preparation of the manuscript for printing were done by Richard N. Walters.

CONTENTS

TABLES

CONTENTS—Continued

FIGURES

PURPOSE

Progress achieved in this study is part of a program to gain Bureau experience with ramp flumes:

- to verify accuracy of computer calibrations,
- to verify existing design criteria,
- to develop further criteria, if needed, and
- to determine flume response to some simulated field conditions.

CONCLUSIONS

1. Comparisons of accuracy between model calibrations and computer calibrations for four different length ramp flumes indicate that computer-calibrated small-size ramp flumes are at least as accurate as Parshall flumes. Ramp flumes have a potential accuracy within 2 to 3 percent.

2. Computer programs were modified to output *limit warnings* where:
   - submergence limit,
   - Froude number criteria, and
   - crest length criteria

   have been violated, so that inexperienced users will reevaluate and provide adequate design data.

3. The main construction requirements are crests of proper length and level both in the direction of and transverse to the flow. The main calibration requirement is that all the dimensions — especially the crest width of the ramp and canal section — be measured carefully after construction. Ramp flumes can be calibrated by computer, using the after-construction dimension measurements. Thus, form movement and other construction errors can be accounted for accurately. Calibration by computer allows more tolerance during construction — saving time and cost.

4. Model data indicate that the total measuring head should be less than half the crest length so that a potential accuracy of 2 to 3 percent is obtained.

5. Total head at the measuring station should be greater than one-twentieth of the crest length to assure no undulation of flow on the crest caused by frictional control. To provide sufficient measuring head relative to precision of head measurement, the measuring head should be greater than about 60 mm (0.2 ft).

6. The Froude number of the approaching canal flow should be less than 0.5 to prevent standing waves from interfering with measurements.

7. A research program should begin using the Bureau's laser-Doppler anemometer to determine velocity distribution coefficients in terms of Reynolds number and flow section shape to improve mathematical hydraulic modeling.

8. From the model having the vertical drop, at the downstream end of the crest, data indicate a *submergence limit* of about 85 percent. That agreed with the claimed submergence limit for small ramp flumes at which the actual discharge deviates -1 percent from the free flow head-discharge relationship. Therefore, the required minimum head loss was 15 percent. For the model ramp flume with 6:1 downstream diverging ramp slope, the *submergence limit* was about 92 percent.

9. Pressure measurements indicated that ramp flumes are relatively insensitive to measuring station location. A measuring station location 305 mm (12 in) upstream from the toe of the ramp is the minimum that should be allowed for 3:1 ramp flumes in small trapezoidal canals.

10. Cost estimates for a 26.3m^3/s (930-ft^3/s) ramp flume were from 45 to 60 percent of that for an equivalent capacity Parshall flume in a retrofit situation. Investigators have cited costs for the small ramp flumes of one-tenth to one-third the cost of equivalent capacity Parshall flumes. This cost effect probably was due to more

common foundation requirements for both Parshall and ramp flumes at the large capacity sizes.

11. Because of good accuracy potential and possible cost savings, a large prototype ramp flume should be built and calibrated in the field and compared with computer calibration. During field calibration, the *submergence limit* should be determined to ascertain if there is a scale effect — similar to Parshall flumes — causing increased submergence depth limits for larger flumes.

12. Laboratory tests need to be made to determine the capability of the ramp flume to pass sediment without allowing deposits to affect or interfere with flow measurements.

INTRODUCTION

From the Water Conservation Laboratory, Agriculture Research Service, USDA (U.S. Department of Agriculture), J.A. Replogle and A.J. Clemmens developed computer programs for calibrating trapezoidal measuring flumes. Their programs account for boudary layer development and accuracies of 2 percent are claimed. The simplest type of flume consists of a ramp slope 3:1 approach up to a horizontal broad sill or crest having a vertical downstream drop back to the canal invert (see figs. 1 and 2).

Various articles by Replogle and Clemmens (see bibliography) indicate that ramp flumes are easy to install in existing canals to meet present and future water-measuring requirements for operation and conservation needs. Small (farm ditch size) ramp flumes are reported to cost one-tenth to one-third of Parshall flumes [1].[1] They have relatively small head losses and are able to tolerate higher submergences. *Submergence limits* of 85 percent for a vertical downstream crest face [2] have been

[1] Numbers in brackets refer to the Bibliography.

cited. The limits approach 95 percent for *long-throated* flumes. A similar increase in submergence limit was expected to occur for ramp flumes with 6:1 slopes beginning from the downstream edge of the crest. The authors cite another advantage; ramp flumes can be calibrated by computer using after-construction dimension measurements. Thus, form slipping and construction errors can be accounted for accurately which allow more construction tolerance.

Ramp flumes are reported to have no more significant problems with sediment [5] than other flume devices. For a new design and if the normal flow depth required to move the sediment is known, sufficient drop can be included to cause near normal flow both upstream and downstream of the flume. Because the flow accelerates, it is expected that most sediment carried into the approach canal (fig. 1) will go over the ramp [5].

An article by Clemmens and Replogle [1] was reproduced in [2]Bulletin No. 107. It describes some experience by the Arizona Agriculture Research Center with ramp flumes.

The Bureau's Upper Missouri Region is using ramp flumes as checks and is planning to install more. The Chief, Design Branch, Billings, Montana, contacted the Hydraulics Branch (E&R Center) and inquired about the flumes and data verifying the claimed accuracy for using ramp flumes as measuring devices. Lack of experience was the main reason for this study and investigation.

The Hydraulics Branch was provided with two computer programs by the investigators:

— BASIC program for calibrating simple trapezoidal flumes [3]

— FORTRAN program capable of calibrating complex trapezoidal flumes [6] with multiple side slopes in the approach and throat section.

[2] Water Operation and Maintenance Bulletin No. 107, Bureau of Reclamation, p. 1-8., March 1979.

BASIC PRINCIPLES

Weir Regimes and Crest Length Criteria

The ramp flume is actually a broad-crested weir having a 3:1 slope approaching the crest. Bos [7] summarizes flow regimes in terms of H_1/L_3 for a rectangular weir profile, where H_1 is total head (fig. 2) relative to crest elevation at the measuring station and L_3 is crest length. Insight into design criteria and performance limits of ramp flumes requires understanding these regimes:

When

$$H_1/L_3 < 0.08 \tag{1}$$

friction of the crest controls and undulations can occur on the crest.

When

$$0.08 \leqq H_1/L_3 \leqq 0.33 \tag{2}$$

parallel flow exists on the downstream third of the crest and the coefficient of discharge is constant over this range of H_1/L_3. Only when a weir operates between these limits, it is operating in a true broad-crested manner.

When

$$0.33 < H_1/L_3 < \text{from about 1.5 to 1.8} \tag{3}$$

parallel flow does not occur over the crest. Flow curvature causes increase in the coefficient of discharge, and control is near the leading edge of the crest over a separation cavity.

When

$$H_1/L_3 > \text{about 1.5} \tag{4}$$

flow becomes unstable and, depending on corner sharpness, can spring free. At H_1/L_3 of 3 or greater, the flow acts like sharp weir flow and is stable.

These inequality relations define flow regimes. Replogle chose a criterion similar to relationship (2) to assure sufficient parallel flow so that the Bernoulli equation can be used without curvature correction in the computer programs. Replogle's [8] recommended design criterion in terms of total head H_1 at the measuring station was:

$$0.05 \leqq H_1/L_3 \leqq 0.50 \tag{5}$$

To prevent wave interference, Replogle further specified an upper limiting Froude number $V/(gD_1)^{1/2}$ of 0.5 for the approach flow.

where

g = gravitational constant (acceleration)

D_1 = hydraulic mean depth or A_1/T_1

A_1 = area of approach flow section

T_1 = top width of approach flow section

Computer Programs

Basically, the ramp flume is a critical-depth measuring device. In the computer programs, Replogle uses the relation for discharge Q at critical depth (occurring somewhere in the downstream 1/3 to 1/4 of the crest, fig. 2) for any shape channel expressed as:

$$Q = \left(\frac{gA_3^3}{\alpha_3 T_3}\right)^{1/2} \tag{6}$$

where

Q = discharge

A_3 = flow area for the entire critical depth or control location which varies with discharge

g = gravitational constant (acceleration)

α_3 = velocity distribution coefficient or kinetic energy correction factor $\Sigma(V_3^3 \Delta A_3 / \overline{V}_3^3 A_3)$ at the control section

T_3 = top width at the control section

V_3 = velocity for an incremental flow area (ΔA_3)

$\overline{V}_3$ = average velocity for the entire control section

Replogle [6] used the energy relation, for the reach between the measuring station and critical depth location, with friction loss H_f included and expressed as (fig. 2):

$$h_3 = h_1 + \alpha_1 (Q^2/2gA_1^2) - A_3/2T_3 - H_f \tag{7}$$

where

h_1 = measuring station head relative to crest

h_3 = control station head relative to crest

Computer routines were developed ([3] and [6]) to determine the *velocity distribution coefficient* α_3 for wide flow and friction loss during boundary layer development. The computer programs assume that α_1 is 1.04 and that design criteria relations in (5) make flow sufficiently parallel so that curvature effect is insignificant.

THE MODEL

Laboratory Flume and Measuring Techniques

Upper Missouri Region personnel use ramp flumes for checking flows up to 1.42 m^3/s (50 ft^3/s). The Bureau's Hydraulic Laboratory cannot supply this capacity so a scale model was considered. A 1:3 scale model was selected as the smallest that could be useful in checking accuracy claims of 3 percent.

Figure 1 shows the laboratory test arrangement with the 1:3 scale model ramp flume installed. The approach had about a 1:1¼ side slope, a length of about 4.9 m with a top width of 1.04 m and a depth of about 0.34 m (16- 3.4- 1.11-ft respectively). A headbox having a rock stilling and distribution baffle, and a bellmouth entrance to the canal section, was provided to smooth the approach flow. A downstream flap-type tailgate was installed to vary the submergence. The ramp and crest were poured in concrete as shown on figure 1.

Flow through the ramp flume was measured with volumetrically calibrated venturi meters. Venturi meters are an integral part of the permanent Hydraulic Laboratory facility and accurately measures discharge to within ±1 percent. The meters have a potential accuracy of ±0.5 percent.

Measuring head h_1 and submergence head h_2 were transmitted to hook gage wells by plastic tubing for more accurate measuring. The repeatability of reading water surface elevations with hook gages in the wells was ±0.3 mm (±0.012 in). Measuring head was measured 305 mm (12 in) upstream from the toe of the 3:1 ramp. Originally, the model submergence measuring station was located 1.42 m (4.67 ft) downstream of the crest and 406 mm (1.33 ft) from the flap gate. This location was considered too close to the downstream control flap gate to study submergences having a 6:1 downstream diverging ramp slope to compare with the vertical drop. Therefore, 2.44 m (8 ft) of downstream channel was added making the model submergence station 1.52 m (5.0 ft) downstream of the crest and 1.83 m (6.0 ft) from the flap gate.

Velocity measurements were made with a pitot static probe mounted on point gage vernier racks that were referenced to the ramp flume crest elevation. Pitot tube pressures were transmitted to a pressure cell connected to a digital voltage display scaled to read pitot differential directly in feet of water.

RESULTS

Test of Location for Measuring Station

Calibrations of all the different crest length ramp flumes were done with measuring stations that were 0.30 m (1 ft) upstream of the toe of the ramp. This provision is intended to keep the measuring station out of accelerating flow and/or curved water surface. Sometimes measuring stations are placed deliberately in the accelerating part of the flow; consequently, making installation of pressure taps or staff gages a critical construction measurement regarding accuracy. In doing this, one presumed advantage is that the measuring station is located within the device itself, providing better control for prefabricated devices. However, putting the measuring station in flow curvature generally makes computer calibrations more difficult. Determination of head loss and submergence limitations for "setting" crest elevations are more difficult.

To investigate the effect of water surface curvature, seven piezometer taps — including one at the measuring station — were spaced 0.15 m (0.5 ft) apart, starting from the toe of the ramp, to 0.91 m (3 ft) upstream. Water surface elevations, for five discharges ranging from 0.28 to 1.42 m^3/s (10 to 50 ft^3/s), were obtained with these taps to compare them with the measuring station values. Discharge error caused by water surface curvature or from using another location than the calibration measuring station was determined. Discharge error for the tap at the ramp toe ranged from -1¾ to -5 and averaged about -3 percent. At 0.15 m (0.5 ft) from the toe, discharge error ranged from +1¼ to -2 and averaged about -0.4 percent. The discharge errors for all the remaining upstream taps other than the measuring station — for discharges greater than 0.28 m^3/s (10 ft^3/s) — were within ±0.7 percent. For some unknown reason, error at 0.28 m^3/s was -2.0 percent for taps 0.46 and 0.90 m (1.5 and 3 ft) upstream of the toe. Variation between piezometer tap geometry probably contributed to these results to some extent. The percent error of discharge from using heads other than the head measured at the measuring station (for all the discharges and piezometer locations) is given in table 1. Based on these results, placing the measuring stations 0.30 m (1 ft) upstream of the toe of the ramp is considered generally adequate for small ramp flumes.

As a further precaution, the range of flow — where the approach length (the measuring station distance plus the ramp length) is greater than five measuring heads — should be minimized. Maintaining this approach length criterion maximizes the flow range which matches the computer assumption that contraction and roughness determines the computer calibration exclusively. However, measurements made when the approach length is greater than five measuring heads are not necessarily wrong but should be infrequent.

Model to Computer Calibration Comparison

Laboratory calibrations for 0.46-, 0.91-, 1.08-, and 1.6-m (1.5-, 3.0-, 3.53-, and 5.25-ft) crest lengths for a crest height of about 0.30 m (1 ft) are plotted with circle symbols on figures 3 through 6, respectively. Curves were fitted by eye through model data. Computer calibrations were made and are triangle symbols on the same plots. Log-log least squares curve fits also were made with the model data of discharge versus measuring head. The values of percent difference between least squares fit computed and model values of discharge were not as small as the author expected. Maximum differences are given in table 2 along with the coefficient A and the exponent n for the equation:

$$Q = Ah_1^n \tag{8}$$

where Q is the discharge and h_1 is the measuring head. Values for these equations are indicated as squares on figures 3 through 6.

Table 3 summarizes comparisons between model and computer calibrations. Column 1 lists the crest lengths. Column 2 gives the percent deviation of model from computer calibration at the head of 0.27 m (0.9 ft). Column 3 gives percent deviation at the maximum discharge at a measuring head h_1 equal to one-half the crest length. Column 4 gives the maximum percent deviation of the model data about the curve (eye) fit of the model data. Columns 5 and 6 list maximum discharge and measuring head determined on the basis of equation 5. Table 3 shows that the computer program generally determines calibrations that were always less in discharge for given mesuring heads than measured flows in the model. All model data in table 3 were within +4 percent (column 3) of the laboratory calibration which is accurate to ±1.0 percent. Therefore, it was concluded that the computer program is potentially accurate to -2 to -3 percent for small ramp flumes.

Although the computer programs produced calibrations of sufficient accuracy, deviations were consistently one sided. For given measuring heads, the computer programs generally predicted lower discharges compared to those measured in the model. This could be due to the combined results of one or more of the approximate equations and assumptions of how velocity distribution coefficient α and friction loss H_f vary with shape and hydraulic parameters. With a laser-Doppler anemometer (available in the laboratory) the Bureau should begin a research program to quantify the velocity distribution coefficient α in terms of Reynolds number and shape for flow sections. Better capability of selecting proper α values would be of considerable help to mathematical modelers in solving hydraulic problems.

Submergence Tests

The *submergence depth limit* percentage (fig. 2) can be defined as the value of $(h_s/h_1) \times 100$ where the actual discharge is 1 percent higher than the discharge computed from the free-flow relationship. This definition was used for this study. Subtracting this value of *submergence depth limit* percentage from 100 is the *minimum* required percent of water depth change required to deliver water without having submergence interfere significantly with the accuracy of flow measurement. Other investigators and the computer program use total head (fig. 2) to determine total head submergence limits $(H_s/H_1) \times 100$. *Total head submergence limits* are generally 1 to 1.5 percent greater than *submergence depth limits* $(h_s/h_1) \times 100$.

To study submergence, the 1.08-m (3.53-ft) crest length ramp flume was used. Three different discharges were set and held constant by laboratory venturi meters and valves. Submergence was varied by the downstream flap gate. The results for 0.67 m^3/s (23.6 ft^3/s) are plotted on figure 7. This plot and the data for the other two discharges indicate that the *submergence depth limit* is at about 85 percent and that discharge measurement is extremely sensitive to error just beyond the submergence depth limit. Visual determination of whether submergence exists is difficult to observe near the limit. It requires actual experience of having seen flow near the limit.

Figure 8a shows flow conditions when the ramp flume is definitely operating in the free-flow mode or with submergence depth less than 85 percent. The wave or roller generally is transverse to the downstream canal. Figure 8b shows the flow conditions when the submergence depth is just at the 85-percent limit. The straight portion of the wave persists in the center of the flow, but at the side slopes the wave forms unstable diagonal disturbance lines oscillating from just downstream of, and to the end of, the sill drop. Figure 8c shows definite submergence with the disturbance lines at the side slope starting over the downstream one-fourth of the sill crest. Figure 9 shows sketches for the same conditions shown on figure 8.

A 6:1 downstream diverging ramp was added to the ramp flume to determine how much the submergence depth limit increased. Measuring heads versus percent submergence at three different constant discharges covering the device range resulted in a submergence limit of 92 percent.

Parshall flume experience indicates that as Parshall flumes get larger, they have increasing submergence depth limits. This may be due to the location of the downstream measuring station or scale effect. This possible scale effect should be checked on large ramp flumes in the field.

Although correction procedures and submerged calibrations frequently are provided for flow measuring devices, generally it is not considered good practice to use flume-type measuring devices under submerged conditions. Any technique that provides for submergence correction increasingly sacrifices accuracy as submergence increases.

Designing a device that is to be submerged throughout all or part of its flow range requires using a calibration related to a measuring head differential. Having a second downstream measuring head station doubles the chance of wrong readings. Submerged discharge ratings are related to small differences of measuring heads. Small imprecisions of water surface elevation measurement cause large errors. As submergence becomes greater, the measuring head differentials become smaller and the differential approaches values that are about the same magnitude as for minor variations of hydraulic form and friction loss. Thus, corrections for submergence can be quite inaccurate.

A device designed for submergence is sensitive to change of downstream flow conditions. Users can temporarily dam the ditch downstream of a measuring device and then remove the obstruction after the irrigation operator has set a flow and gain considerably more than their fair share of water.

Knowledge of *required minimum head loss* is needed to design a ramp flume for a particular site or case. Because of this and the above reasons, consideration of submergence in this study is directed mainly toward determining the ***submergence depth limits*** for ramp flumes rather than attempting to provide submergence correction data.

Water Surface Profiles

Measured water surface elevations versus distances in the direction of flow are plotted on figure 10 for discharges of about 1.4-, 0.57-, 0.28-, and 0.14-m^3/s (50-, 20-, 10-, and 5-ft^3/s). Although water surfaces are curved throughout this range of discharge, the assumption of parallel flow for computing ***measuring heads*** is apparently close enough to produce computer calibration discharges that are within -2 to -3 percent. Figure 10 profile, for discharges less than 0.03 m^3/s (1 ft^3/s), is strictly schematic and shows undulating flow that can occur when friction controls crest flow. The ramp flume will not function as a measuring device in this case.

Velocity Distribution Coefficient

Velocity data were recorded on the downstream third point of the crest (fig. 2) to calculate velocity distribution coefficient α for a discharge of 0.34 m^3/s (12.2 ft^3/s) for 5 vertical profiles:

- for 5 velocity area zones, α was 1.129
- for 11 verticals and 9 zones, α was 1.094
- for 11 verticals and 10 zones, α was 1.065.

The computer program computed 1.013 for that same discharge. The difference between the computer and measured α does not explain the one-sidedness of the difference between model calibration and computer calibration discussed previously because larger values of computer α would increase the difference. Further study of velocity distribution coefficients should be made in an effort to determine the cause or causes for the one-sidedness. Possible causes are the computer routines for the α coefficients, friction, and modeling assumptions.

Demonstrations and Cost Analyses

The ramp flume was demonstrated for water measurement sessions at three Bureau Water Mangement Workshops. About 25 percent of the participants requested copies of any written reports generated by the studies. Denver Office staff requested dimensions for some larger proposed ramp flumes from 5.66 to 26.3 m^3/s (200 to 930 ft^3/s) for design studies. In 1981 a design study was made for the Charles Hansen Feeder Canal comparing costs of a 9.14-m (30-ft) Parshall flume and a ramp flume. The Parshall flume was estimated to cost $100,000, and the ramp flume was estimated to cost between $46,000 to $60,000. The 9.14-m (30-ft) Parshall flume had to be used rather than a 6.10-m (20-ft) flume because of head loss problems. In this case the costs are not as small as cited in [1] (one-tenth to one-third) for small ramp flumes. However, expected savings would diminish with increase in size because flume foundation requirements become more similar, but cost savings are still substantial for large ramp flumes. Thus, a large ramp flume should be built in the field. The flume could be either permanent or temporary and studied during the early stages when the project is operating below design capacity so that more freeboard versatility is available for checking possible scale effect on submergence depth limit.

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Table 1. — *Percent difference of discharge using head from a location other than the measuring station*

Piezometer location distance upstream from ramp toe		Discharge m^3/s (ft^3/s)				
ft	m	1.41 (50)	1.13 (40)	0.85 (30)	0.57 (20)	0.28 (10)
		Percent				
0	0	-3	-3	-0.7	-2.5	-5.0
0.5	0.15	-2	+1.25	-0.7	-0.5	-5.0
1.0*	0.30*	--	--	--	--	--
1.5	0.46	0	+0.5	-0.4	-0.5	-2.0
2.0	0.60	0	0	0	0	0
2.5	0.76	0	0.5	0.7	0.4	0
3.0	0.91	0	0.5	0.7	0.5	-2.0

* Measuring station.

Table 2. — *Coefficients and exponents for equation* and percent comparison between the* least squares fit *and the* measured discharge *for four different length crests of about 0.3-m (1-ft) height*

Crest length		*Coefficient A		*Exponent n	Percent comparison
ft	m	in-lb system	SI system		
1.5	0.46	19.41	4.857	1.834	±3.0
3.0	0.91	18.63	4.362	1.778	±3.5
3.53	1.08	18.83	4.472	1.790	±4.0
5.25	1.60	18.54	4.367	1.783	±2.5

* For discharge $Q = Ah_1^n$, m^3/s (ft^3/s). h_1 = measuring head, m (ft).

Table 3. — *Accuracy estimates determined from calibration curves*

1 Crest length ft	 m	2 Percent deviation of model discharge from computer discharge at 0.27-m (0.9 ft) measuring head	3 Percent deviation of model discharge from computer discharge at max. discharge (head = ½ crest length)	4 Percent maximum deviation of model discharge from model curve fit	5 Maximum discharge[2] ft^3/s	 m^3/s	6 Maximum measuring head[2] ft	 m
1.5	0.46	[1]	--	--	11	0.31	0.75	0.229
3.0	0.91	+2	+2½	±1½	38	1.08	1.50	0.457
3.53	1.08	+3	+4	±1	54	1.53	1.75	0.533
5.25	1.60	+3½	+3	±1½	[3]		2.63	0.802

[1] Head violates design criteria equation (5)
[2] Based on design criteria equation (5)
[3] Not enough model depth to determine

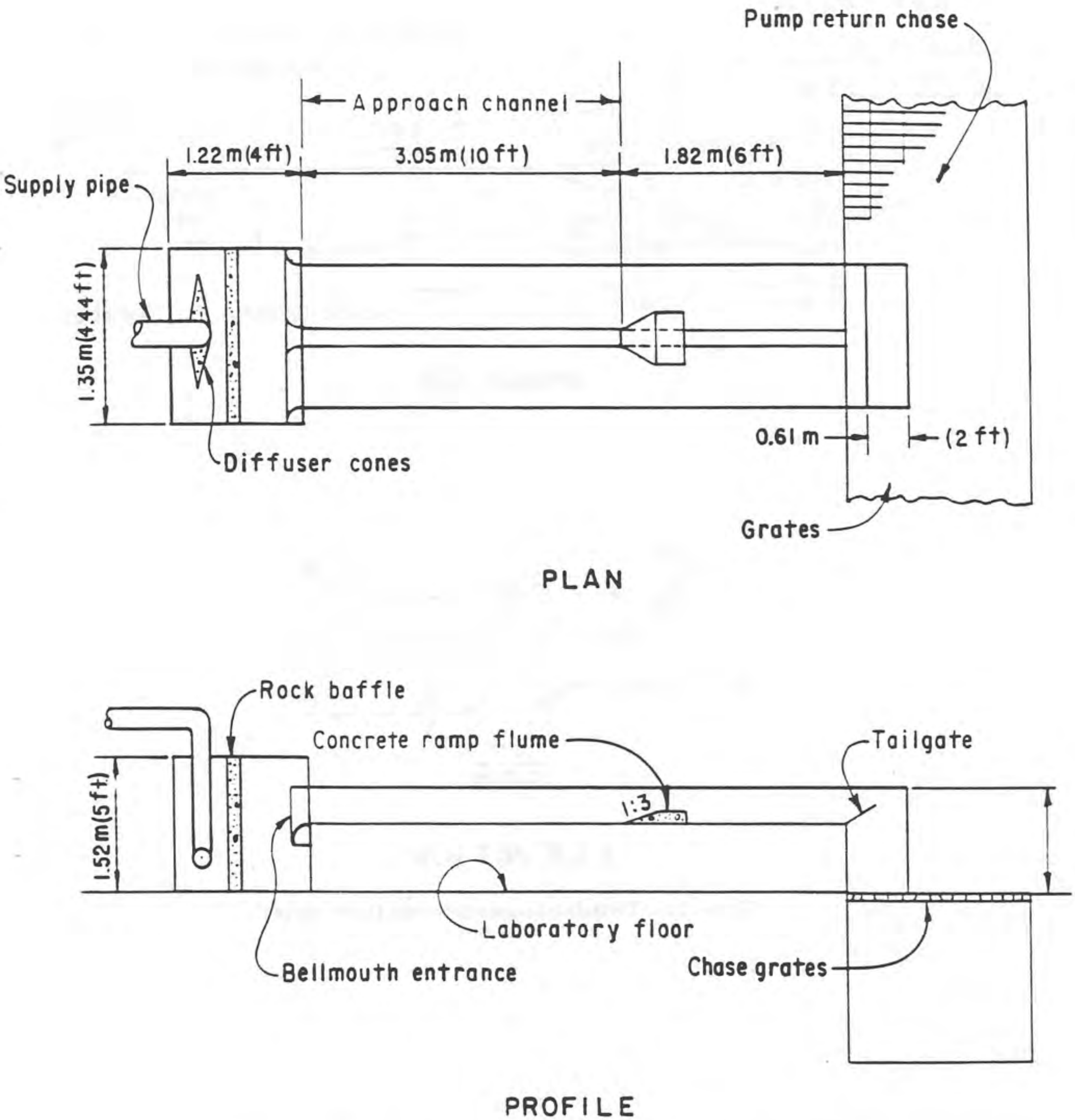
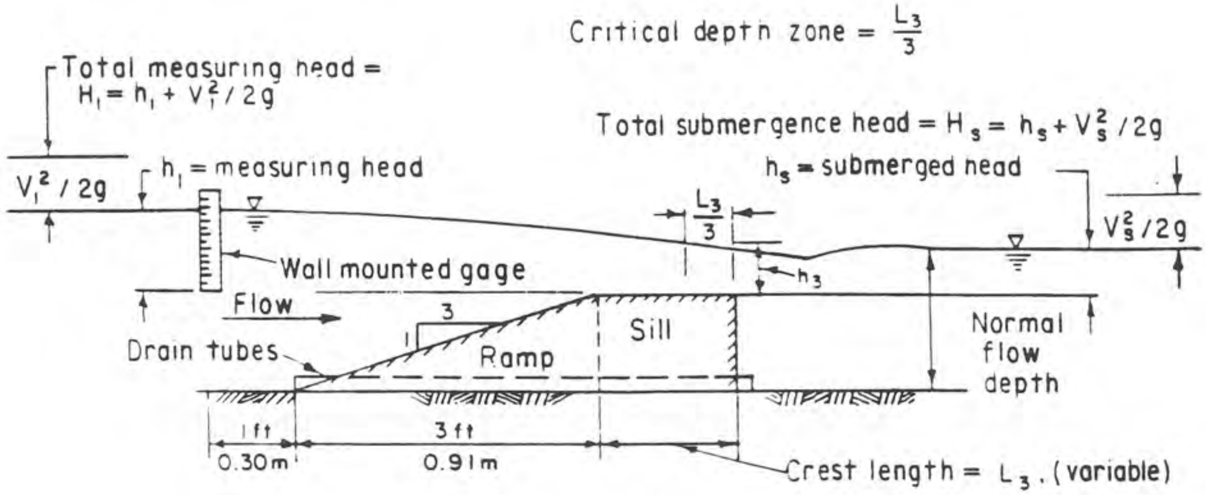
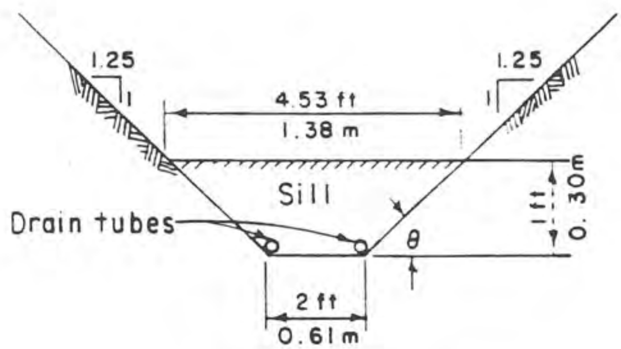


Figure 1. — Laboratory ramp flume — model test arrangement



PROFILE



ELEVATION

Figure 2. — Details of concrete ramp flume tested

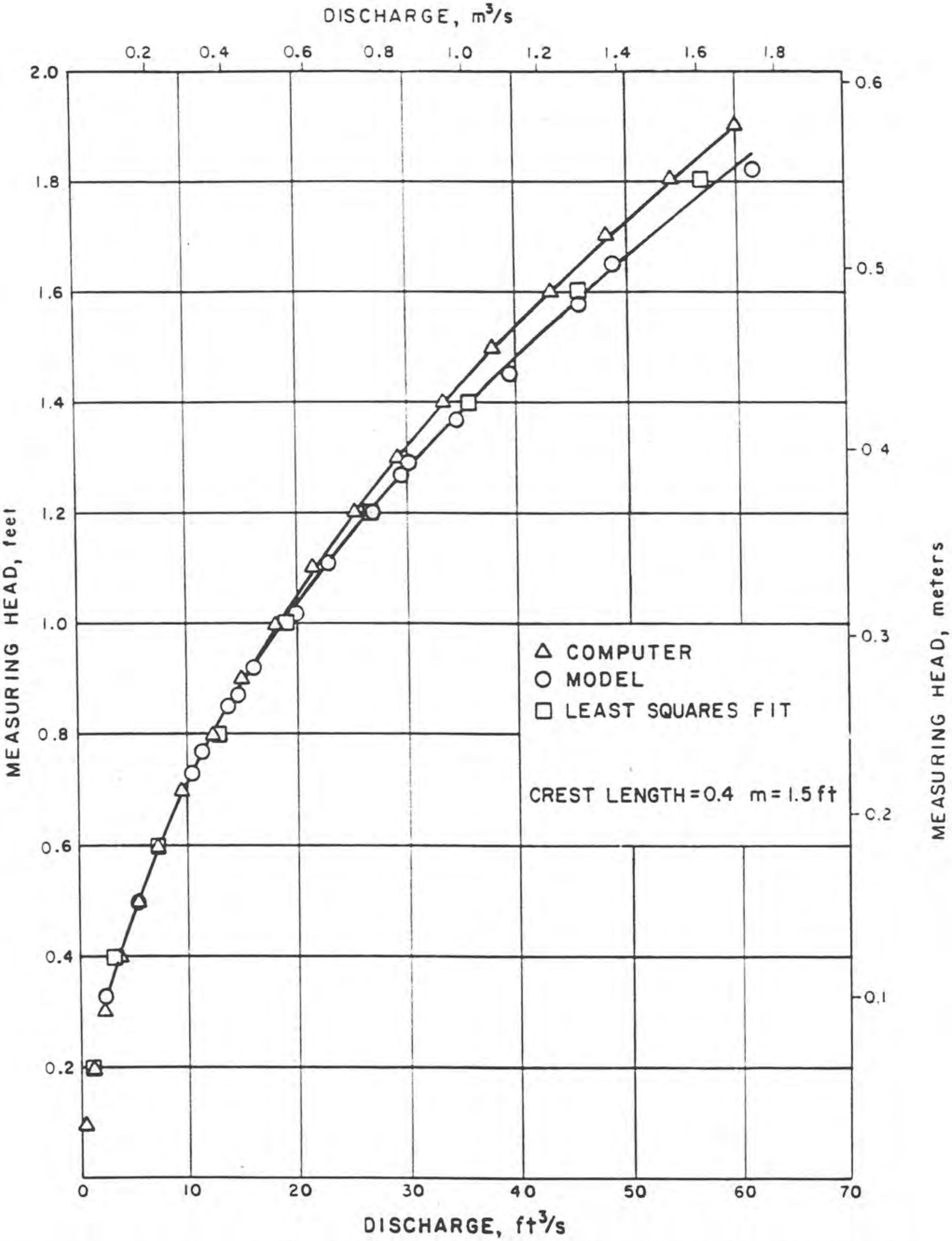


Figure 3. — Ramp flume calibration curve for 0.3-m high crest by 0.46-m crest length (1- by 1.5-ft)

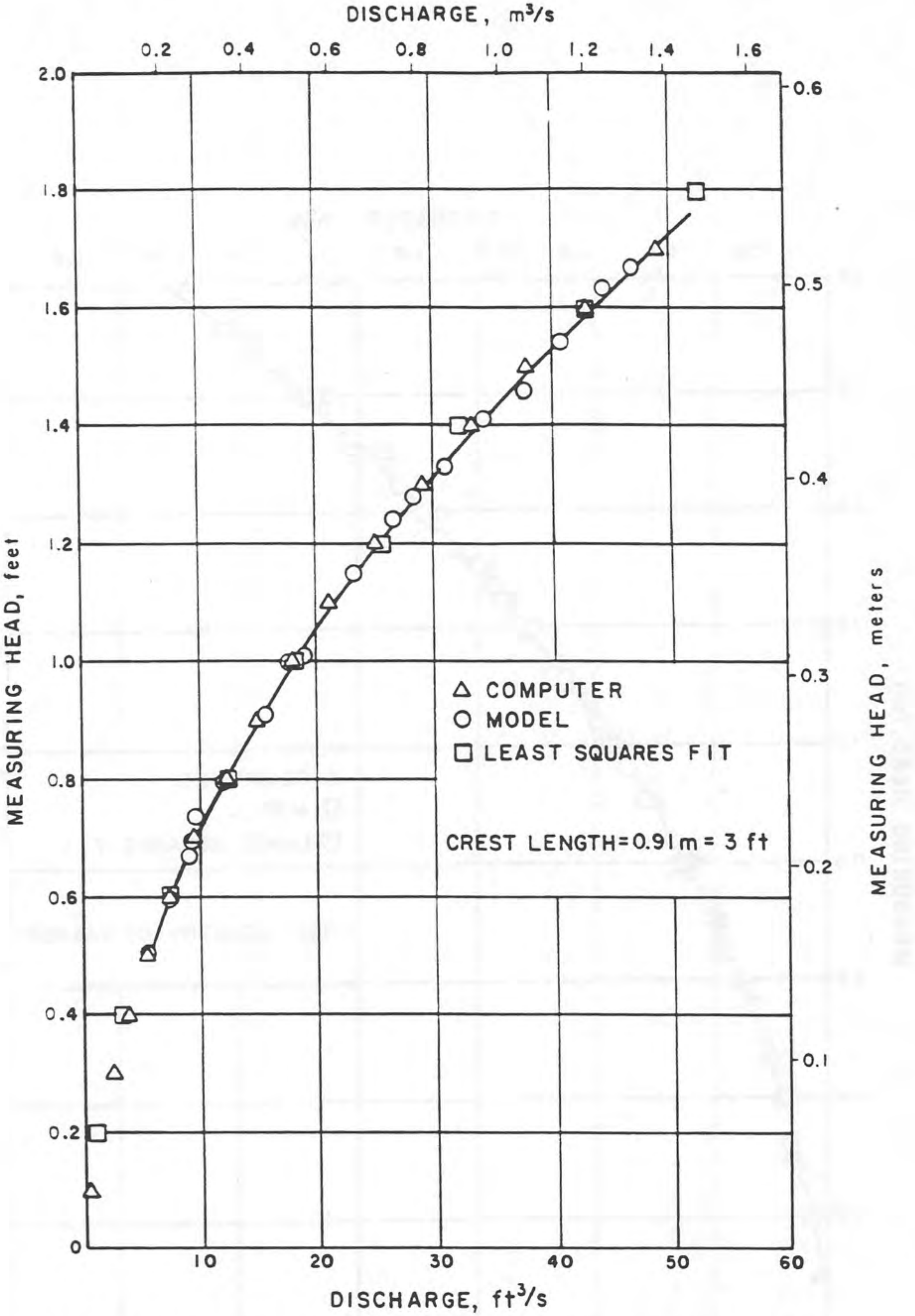


Figure 4. — Ramp flume calibration curve for 0.3-m high crest by 0.91-m crest length (1- by 3.0-ft)

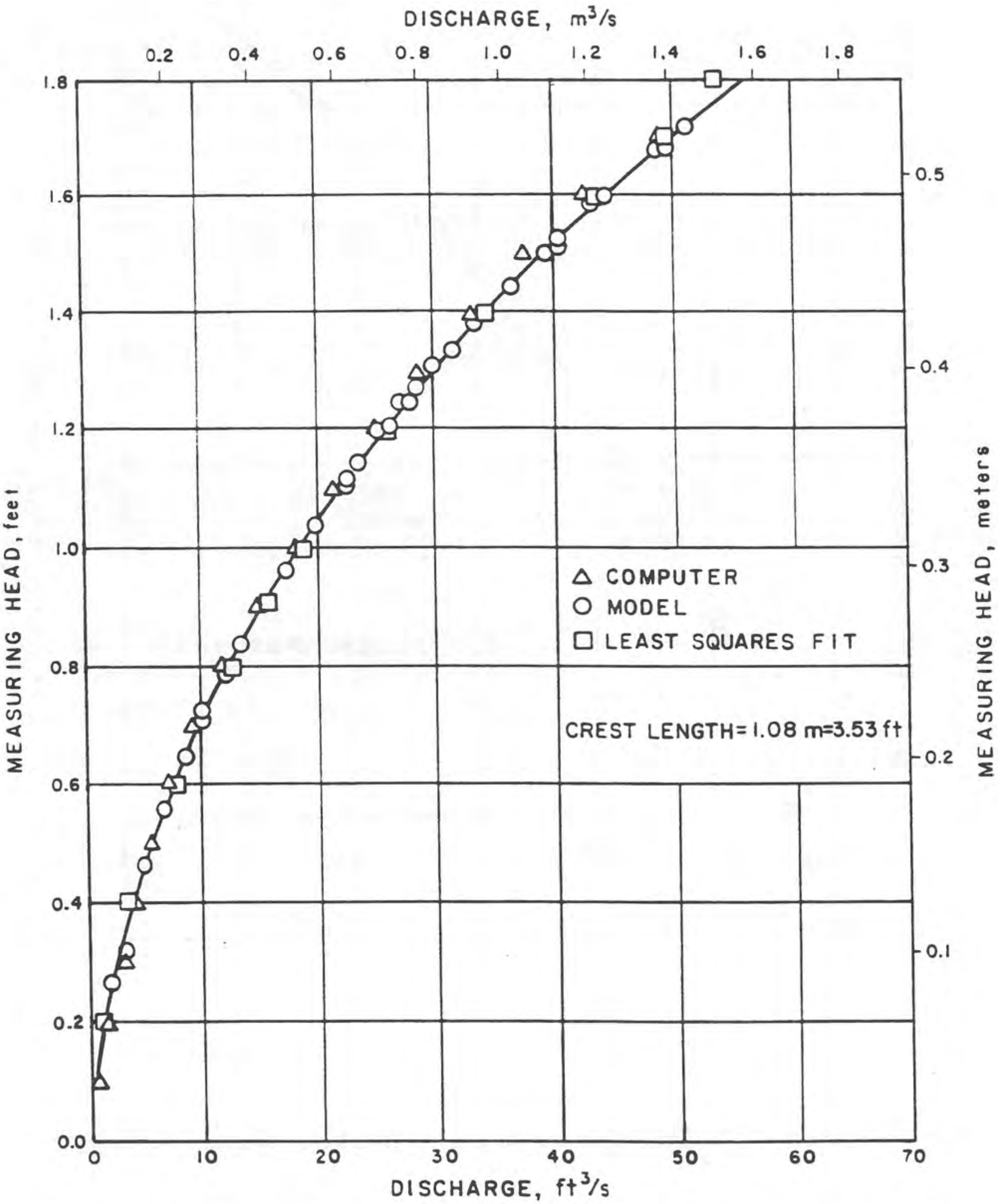


Figure 5. — Ramp flume calibration curve for 0.3-m high crest by 1.08-m crest length (1- by 3.53-ft)

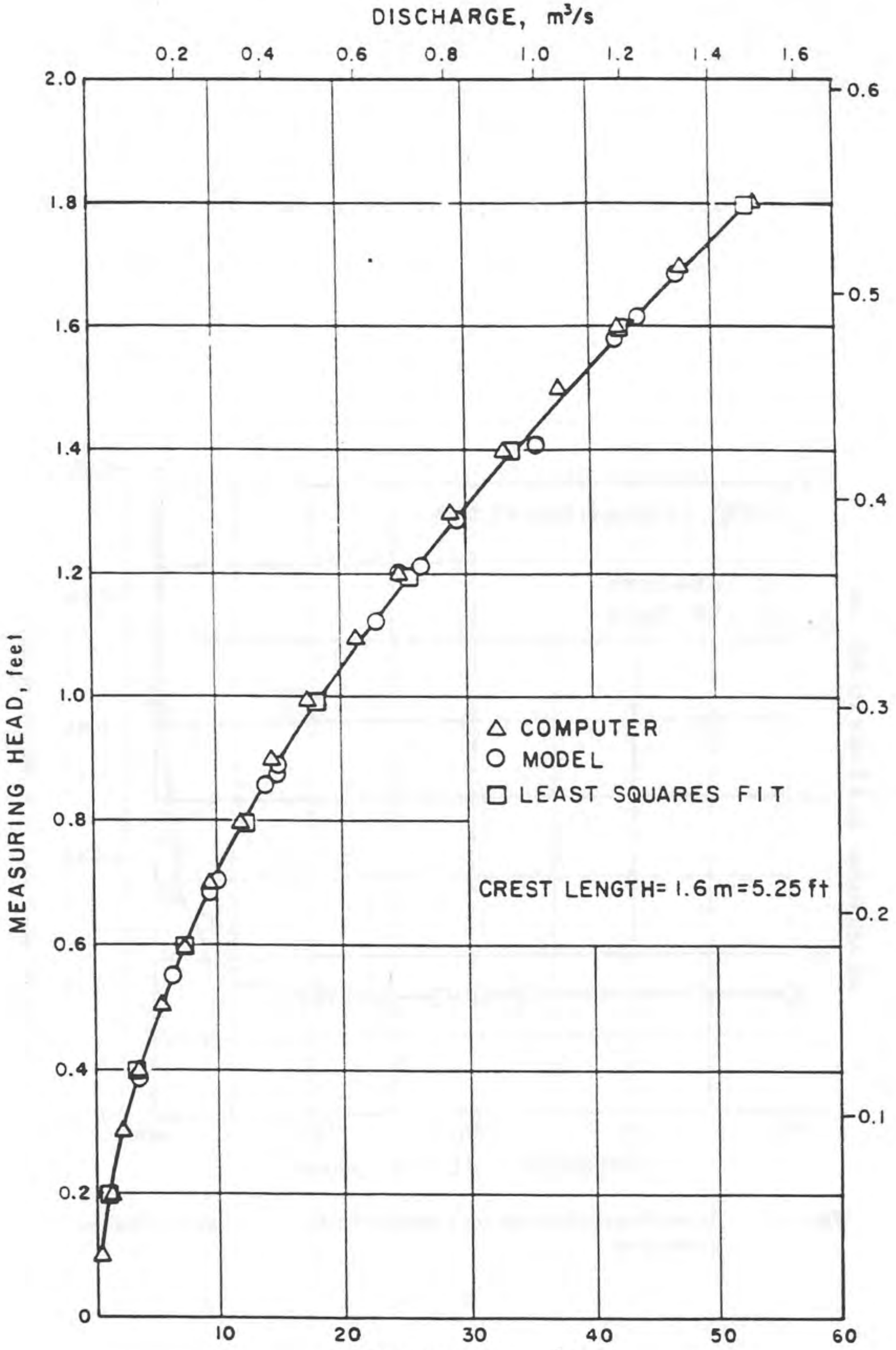


Figure 6. — Ramp flume calibration curve for 0.3-m high crest by 1.6-m crest length (1- by 5.25-ft)

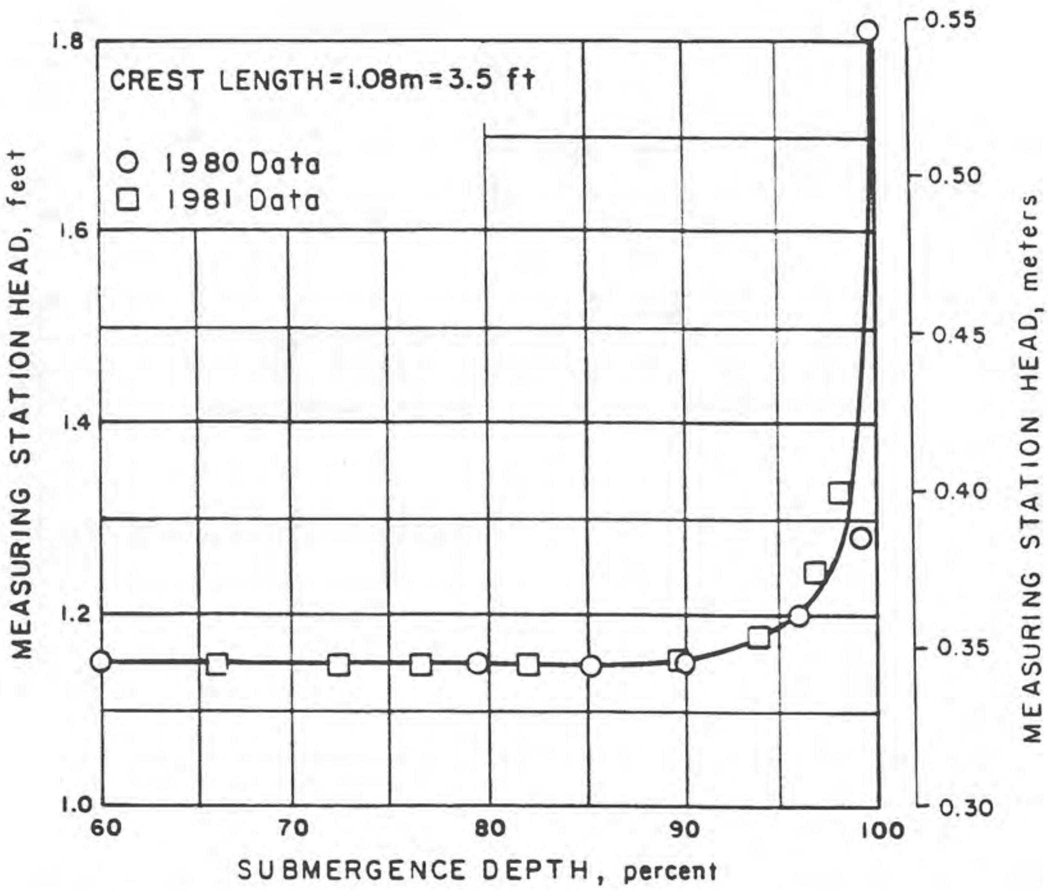


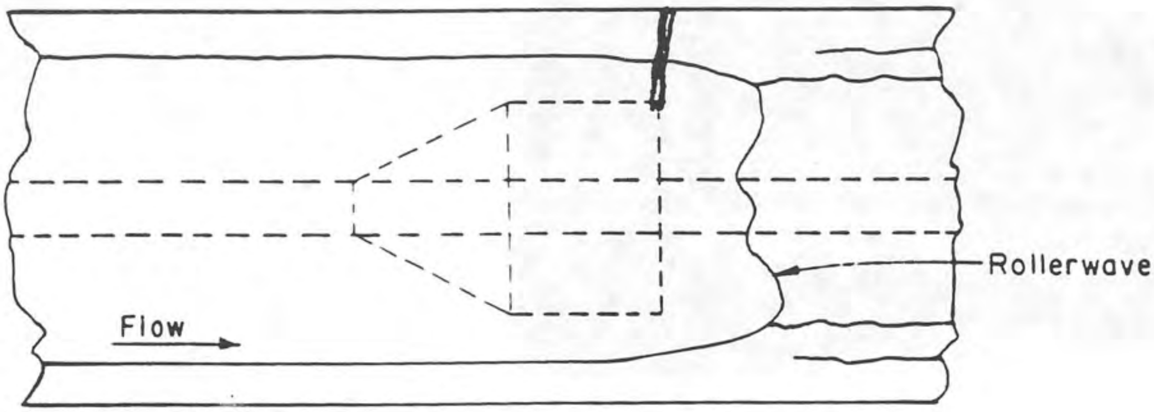
Figure 7. — Ramp flume submergence characteristics for discharge of 0.67 m^3/s (23.6 ft^3/s)

a. Submergence less than 85 percent. Photo P801-D-80115

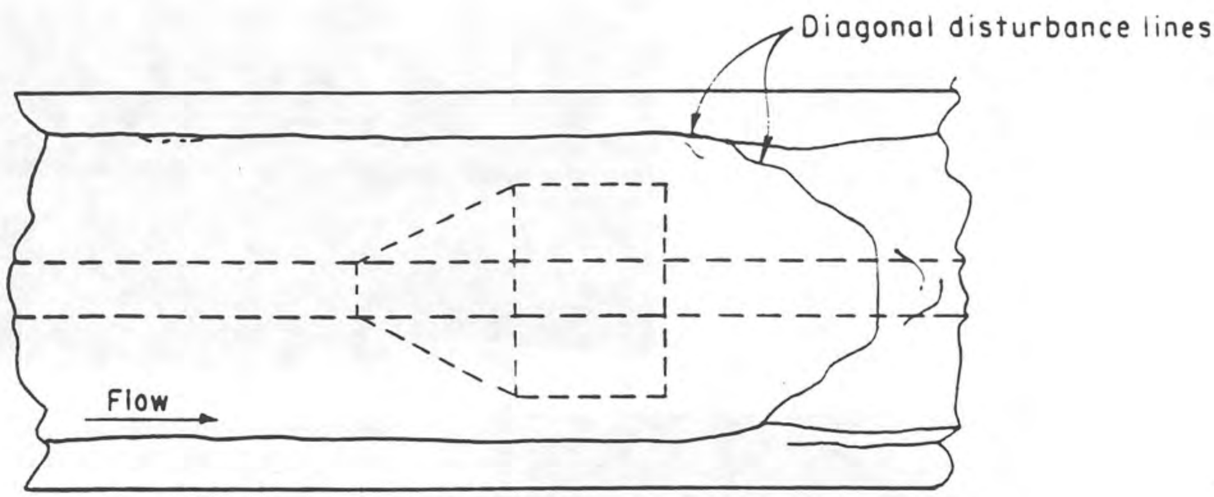
b. Submergence limit 85 percent. P801-D-80116

c. Submergence greater than 85 percent. P801-D-80117

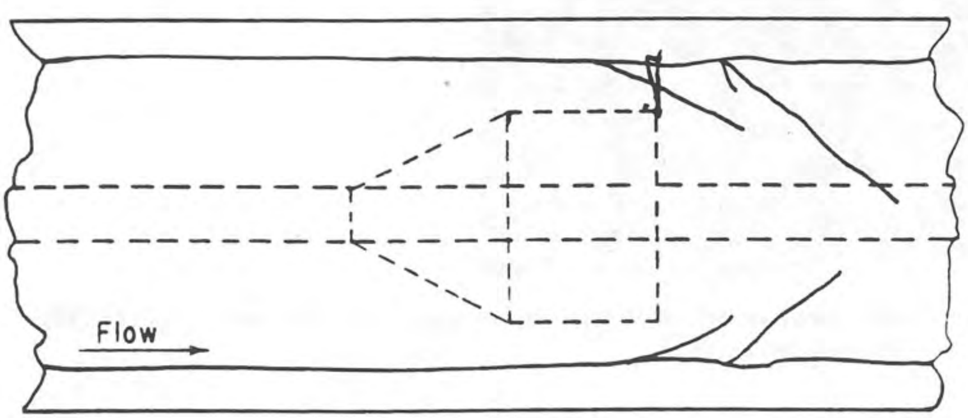
Figure 8. — Waveforms downstream of ramp flume having a 1.08-m crest length (3.53-ft) —flow left to right



(a) Submergence less than 85 percent



(b) Submergence limit 85 percent



(c) Submergence greater than 85 percent

Figure 9. — Waveform sketches downstream of ramp flume

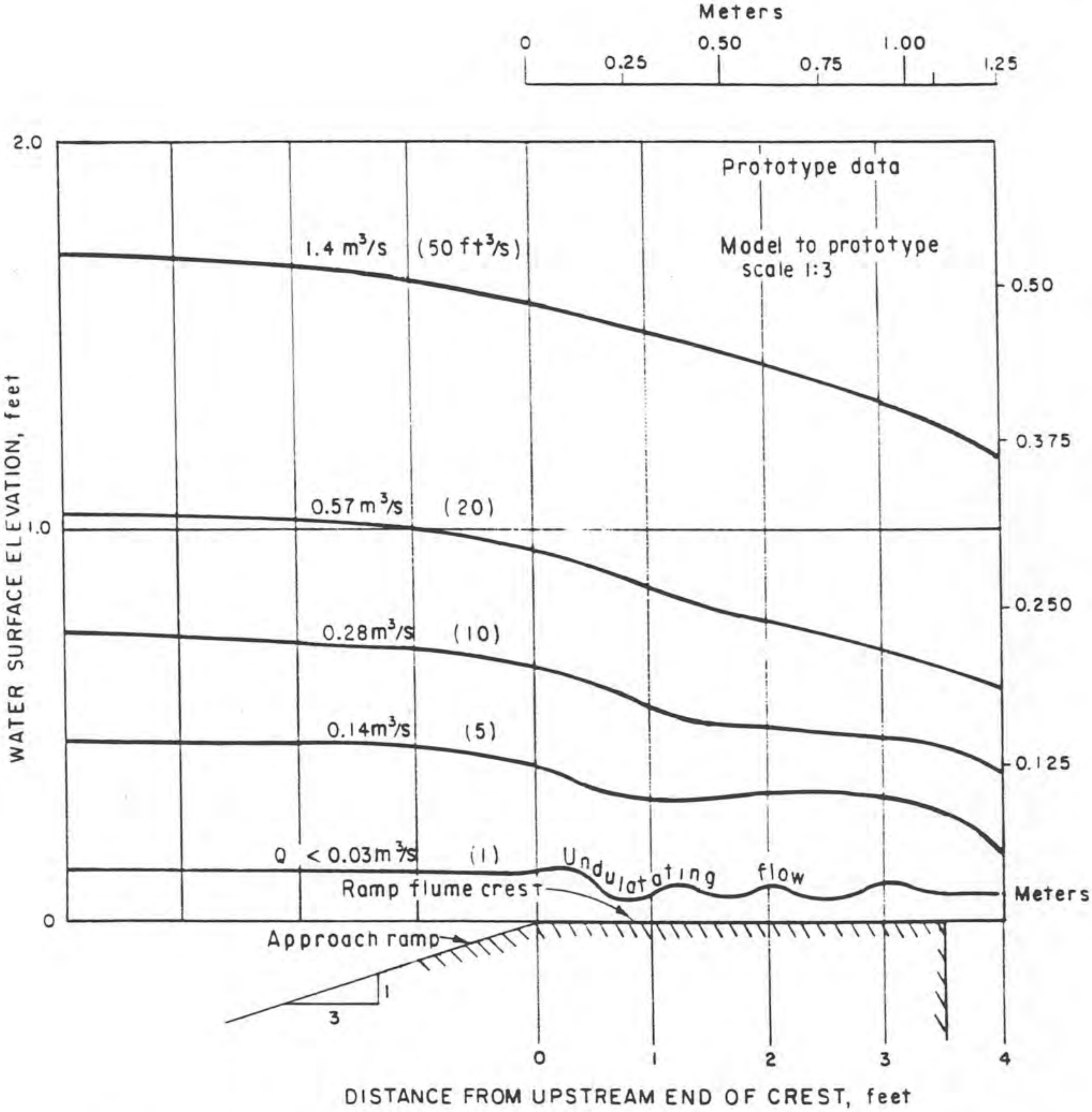


Figure 10. — Water surface profiles for various discharges — crest length 1.08 m (3.53 ft)

GPO 839-693

BUREAU OF RECLAMATION
Engineering and Research Center
Denver, Colorado

TRAVEL REPORT

Code : D-1531, D-1532 Date: December 17, 1984

To : Chief, Division of Research and Laboratory Services

From : R. Dodge, C. Klumpp, J. Fitzwater

Subject: Field Calibration Check of 1,050 ft^3/s Ramp Flume, Klamath Project

1. Travel period: October 14-19, 1984.

2. Places or offices visited: Klamath Project, Oregon.

3. Purpose of trip: To check calibration of 1,050 ft^3/s ramp flume in A-Canal and to obtain data to verify the momentum balance mathematical model, WSPROM.

4. Synopsis of trip:

October 14 to 19, 1984

We left Denver October 14 and because of flight delays and cancellation problems we did not arrive in Klamath Falls until the evening of October 15. The next morning we contacted Mike White of the Project Office who took us to the canal site and explained the project's pretest survey, flume measurement section locations and instrument beam shim leveling technique. From October 16 to 18, the Regional hydrologists obtained current meter ratings for ramp flume nominal discharges of 1,050, 900, 600, and 300 ft^3/s. We obtained water surface elevations at water's edge and across the two current meter bridges during these flume ratings using a surveyors level and rod. The electronic water surface measurement system could not be used because the equipment never arrived in Klamath Falls. On October 19, we returned to Denver.

Participation by Others

The Hydraulics Branch, E&R Center, provided the Mid-Pacific Regional Office, Sacramento, with a design parameter study at two possible ramp flume stations for the A-Canal. The Regional Office completed the final detailed design. Bud Cook and Mike White of the Project Office provided close liaison with KID (Klamath Irrigation District) and obtained as-built dimensions and elevations for computer recalibration and field measurement calibration check of the ramp flume. Timothy Bradley was especially helpful in setting and maintaining discharges during velocity and water surface elevation measurements

Travelers: R. Dodge, C. Klumpp, J. Fitzwater   Date: December 17, 1984

and in moving, positioning and stay bracing the current meter. Gary Rood and Dennis O'Connor from the Regional Office did the current metering in a highly professional manner. Lee Elgin and Dave Ehler helped during the pretesting of the electronic systems for measuring water surface profiles and providing alternate systems in case of failure. Paul Fujitani and Helen Bradley of the Mid-Pacific Regional Office visited the ramp flume site during measurements. The interest shown by the OCCS Committee and their considerable funding for the laboratory studies and field tests is greatly appreciated.

5. Conclusions:

a. Current meter rating and water surface profile data were obtained for 1,050, 900, 600 and 300 ft^3/s nominal discharges.

b. Using the chart recorded measuring heads and the as-built computer recalibration, the ramp flume discharge measurements all agreed with the current meter ratings to within 3.5 percent. The average absolute discharge deviation for the seven discharge ratings was about 1.2 percent.

c. These accuracy results indicate that large ramp flumes can be designed and computer calibrated using the RAMPF computer program to within equivalent Parshall flume accuracy.

d. Ramp flumes are a cost effective alternative for Parshall flumes. The bid cost ($97,000) for the A-Canal ramp flume is about 45 percent of the estimated design cost for a Parshall flume.

R A Dodge
C. C. Klumpp
J. Fitzwater

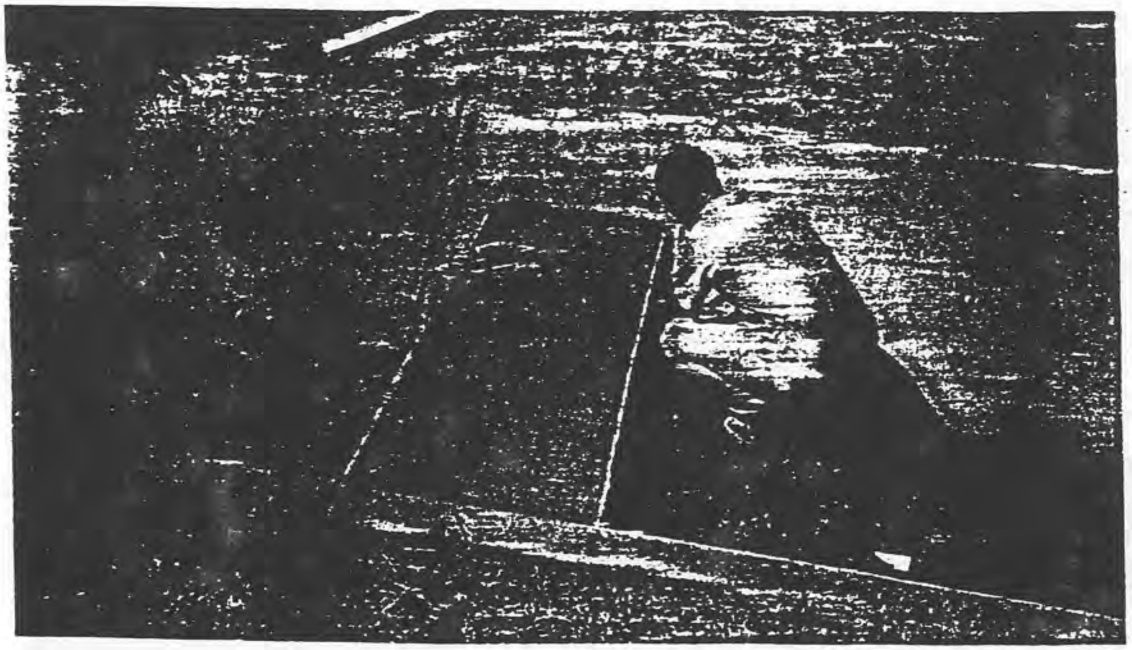
Copy to: Regional Director, Sacramento, California, Attention: MP-213 (Fujitani), MP-200, MP-700
Project Manager, Klamath Falls, Oregon

Blind to: D-272 (Kinney)
D-430 (Yocum)
D-1500
D-1530
D-1531 (file)
D-1532

Noted: DEC 26 1984
H. Walter Anderson
FOR Chief, Division of Research and Laboratory Services

RADodge:flh

Constructing Simple Measuring Flumes For Irrigation Canals



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NUMBER 2268

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Constructing Simple Measuring Flumes For Irrigation Canals

Albert J. Clemmens and John A. Replogle[1]

Introduction

Poor irrigation water management causes many serious water-supply and water-quality problems. Farmers and other water users are being encouraged to develop their own water management plans or improve existing ones. A major step in implementing any plan is measuring the amount of irrigation water applied. Correct application depends on accurate discharge rate information, both for the main irrigation stream entering the field and for runoff leaving the field.

Typical water-measuring devices—sharp- and broad-crested weirs and flumes such as the parshall, cutthroat, and short- and long-throated trapezoidal that are used in open channels or ditches—are cumbersome and expensive. Since most of these measuring devices require laboratory calibration, flumes used in the field are limited to the particular shapes and sizes for which these calibrations are available.

[1]Research hydraulic engineers. Science and Education Administration, Agricultural Research, U.S. Water Conservation Laboratory, Phoenix, Ariz. 85040.

Any deviation in the field-constructed flumes or weirs from the exact dimensions of the laboratory-calibrated devices may cause the depth-discharge relationship (rating curve) to change and often requires the new as-built flume to be duplicated and recalibrated in the laboratory. This strict dimensional requirement makes most flumes difficult and costly to construct. Another disadvantage of weirs and conventional flumes is the amount of head loss (or water depth change) necessary to obtain a valid measurement of water flow rate.

A new style flume that is simply constructed has been developed that eliminates most of the problems connected with other flumes. The rating curves for these flumes are derived from a laboratory-tested mathematical modeling technique which can be used because the flume has a control section (flume throat) that is long enough to cause nearly parallel flow. The resulting flow properties make it possible to design flumes that cause very low head losses, yet produce accurate discharge measurements. The mathematical model can be used to determine changes in discharge rate that

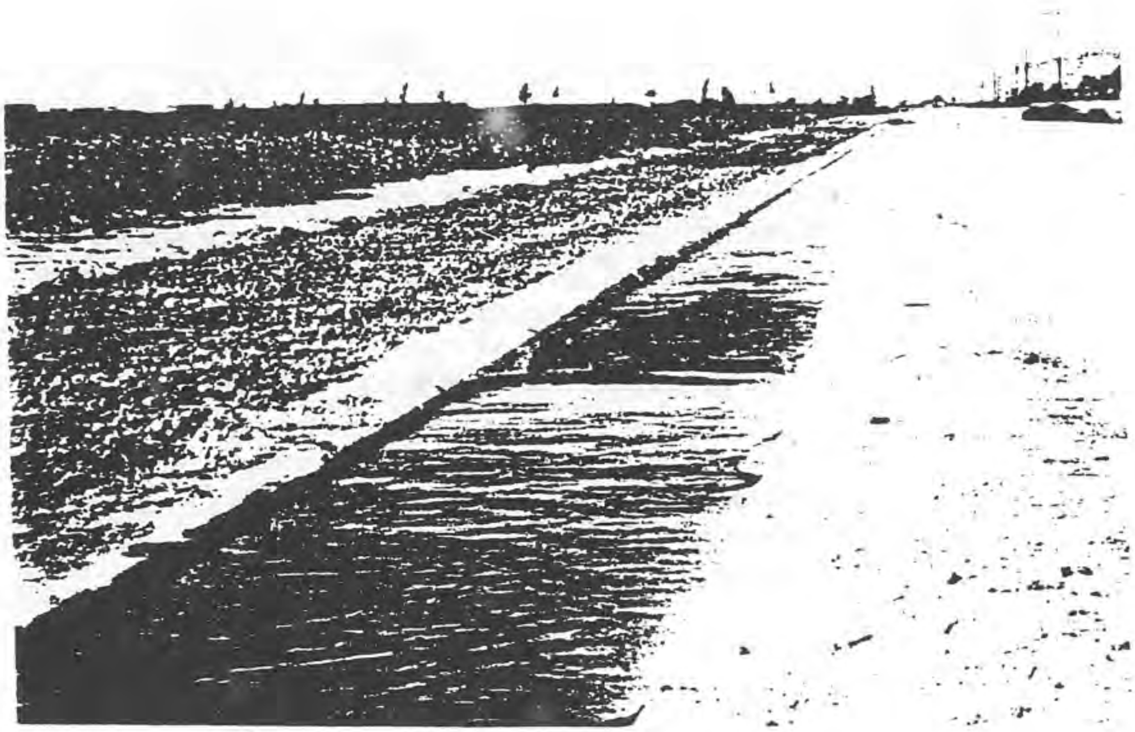

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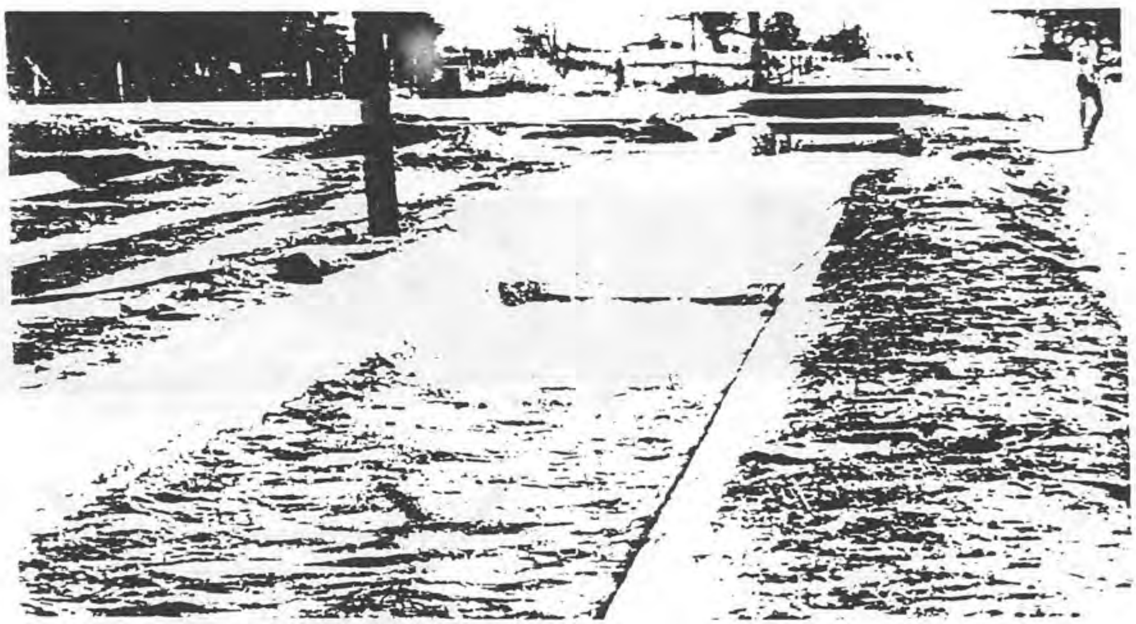
FIGURE 1.—Flume installation at the University of Arizona Cotton Research Center in Phoenix shows the low head loss requirements of the b-c-w style flume. Here, b-c-w flume FA2 is running at 8-9 ft^3/s in a 24-in ditch.

result from changes in the flume dimensions. Information on the changes can then be used to determine allowable tolerances in various flume dimensions.

The new, simplified flumes resemble modified broad-crested weirs (b-c-w) and are actually a style of long-throated flume. The b-c-w flumes are generally much easier to construct and have lower absolute design head losses (head loss at design discharge) than conventional trapezoidal flumes. The b-c-w flumes, as shown in figures 1 and 2, and flume designs presented here for standard slipform ditches can make flow measurement simpler, more convenient, and more accurate than formerly available devices.



PX-6552

FIGURE 2.—Flume installed near Gilbert, Ariz., for the Salt River Project. The flow rate shown (40 to 50 ft³/s) is much less than the design flow rate (75 ft³/s) resulting in more head loss than necessary. The flume is FC2, but with L = 4.0 feet. The ditch is 36 inches deep.

Flume Design Criteria

The profile and cross section of the b-c-w flume are shown in figure 3. The important design dimensions are the sill height, *S*, and the sill length, *L*. The sill represents the floor of the throat section for these flumes. The sides of the throat section are the existing channel walls. The sill length is less important for design and need not be considered in selecting sizes for the new style measuring flume. There are certain conditions, however, under which the sill length limits the design. If it is too short, the flow lines will not be parallel enough in the throat section and the flume ratings may deviate by as much as 5 percent from those presented in this publication (see p.9). When this condition exists, the rating tables are marked to indicate that a longer throat should be used.

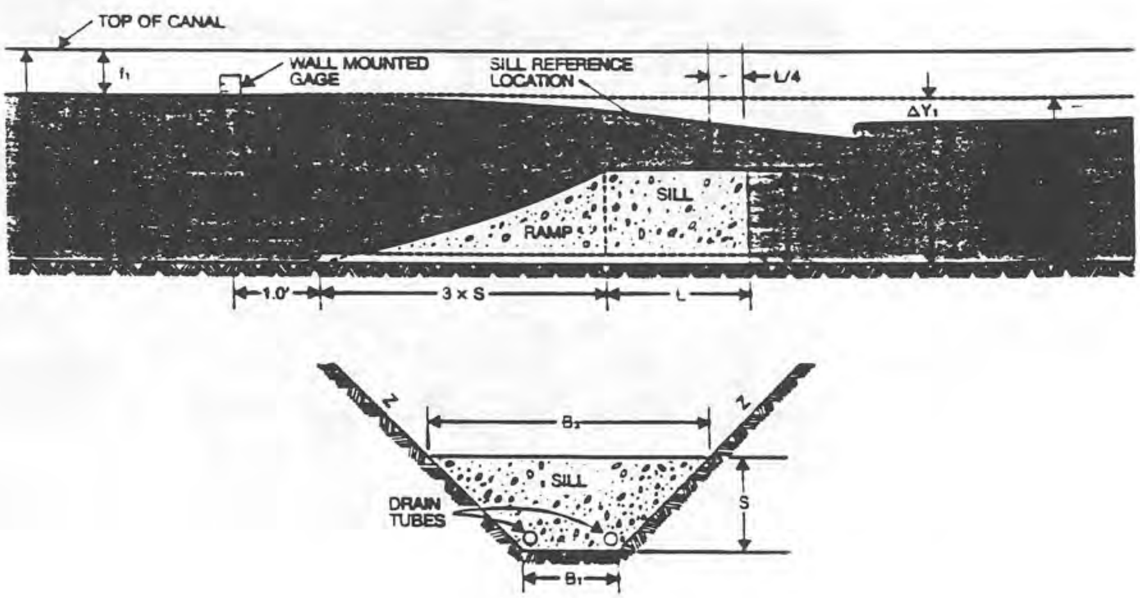


FIGURE 3.—Flume dimensions in profile (top) and cross section (bottom). Abbreviations used in the profile are as follows: d = constructed depth; D_1 = upstream water depth; f_1 = actual freeboard; Y_1 = sill-referenced flow depth; L = sill length; ΔY_1 = actual increase in water depth caused by the flume; S = sill height; d_m = flow depth; 3 × S = three times the sill height (ramp length).. Abbreviations used in the cross section (bottom) are as follows: B_3 = sill width; B_1 = canal bottom width; Z = sideslopes; S = sill height.

The sill height is the most important design dimension for controlling the water levels in the ditch. The sill must be high enough so that it obstructs the usual flow in the ditch, causing the water surface upstream from the flume to rise as shown in figure 3. Field and laboratory experience with this style of flume has shown that flumes will operate satisfactorily at 85 percent submergence. For example, the downstream water depth should be no more than 85 percent of the upstream water depth, both measured from the elevation of the top of the sill. Thus, one criterion for design is that the required minimum depth change caused by the flume at maximum flow should be 15 percent of the sill reference depth, Y_1.

Another criterion of sill height is that it not cause so much of an obstruction in flow that the ditch upstream from the flume is overtopped. The Soil Conservation Service recommends that the freeboard for stable subcritical flow in trapezoidal channels be 20 percent of the energy head at normal depth. (This is approximately 20 percent of the flow depth for canals with very slow-moving flow.) Because of the stable nature of the flow upstream from the b-c-w flumes, the free board requirement there can usually be reduced from 20 percent of D_1 to 20 percent of Y_1, which is the sill reference depth (fig. 3). These two criteria must be met for the full range of expected flow conditions. The conditions for both criteria are most crucial at the maximum expected flow rate, Q_m, for these shapes of flumes. Thus, the design is based on the maximum expected discharge and the associated depth, d_m. Since this water depth is important to the design, some engineering assistance may be required to get accurate maximum depth-discharge relationships.

Three selected sill heights have been chosen to accommodate the usual range of flows for three standard-sized slipformed canals. Design and selection of a flume is a simple, straightforward procedure.

Flume Design Procedure

Table 1 can be used as a design aid for selecting the appropriate flume. The example shown in table 1 is for a 36-inch ditch with a 2-foot bottom and 1:1 sideslopes that is used to carry a maximum of 20 cubic feet per second (ft^3/s). If the ditch flows about 25 inches deep at this discharge, what sill height should be selected? To use the table for this example, begin by filling in the basic information about the canal or ditch where the flume is to be located (lines 1-3). Line 1 is simply the vertical depth of the canal, *d*, line 2 is the width of the canal bottom, B_1, and line 3 is the canal wall sideslope, *Z*, expressed as a ratio of horizontal to vertical distance. Line 4 is the maximum discharge rate. Line 5 is the flow depth. Line 6 is the existing freeboard in the ditch prior to flume construction (line 1 minus line 5).

Once this information is obtained, the selection procedure is straightforward. Table 2 gives flume calibrations for three sizes of flumes (sill heights) for each of three ditch shapes. To begin the selection procedure, find the ditch shape in table 2 (A, B, or C) that matches lines 2 and 3 in table 1. For this example, the ditch shape corresponds to canal shape B. Choose any one of the three flume sizes from that group. For this example, FB1 was tried first. Record the information from table 2 for that flume size in the blanks provided for lines 7 through 12 on table 1. (The repetition of some information in table 1 assures that the appropriate flume group and size are chosen from table 2.)

The remaining section of table 1 is a check on the flow conditions that would exist if the trial flume size were installed. Figure 3 shows a profile of the flume and a typical water surface resulting from flume placement. From the appropriate column of table 2, find the sill-referenced flow depth, Y_1, for the design discharge, Q_m and enter it under line 13 in table 1. For the example, $Y_1=1.143$, for $Q_m=20$. Add lines 12 and 13 to obtain the total ditch flow depth, line 14, $D_1=S+Y_1$. Subtract line 5 from line 14 to get the actual increase in water level caused by the flume, line 15, $\Delta Y_1=D_1-d_m$ (fig. 3).

Next, calculate the rise in water surface that the flume must cause so that it will operate properly, line 16, $\Delta Y_2=0.15\ Y_1$. If the actual increase in water depth, ΔY_1, is less than that required, ΔY_2 ($\Delta Y_1-\Delta Y_2=\Delta Y_3<0$) then the flume cannot be expected to operate properly (line 17), because it will be submerged, as for FB1 of the example.

This limit on the flume submergence assures that the water depth downstream from the flume will not affect the water depth reading upstream from the flume at the gage location. If ΔY_3 is negative, a higher sill must be used, so lines 7-17 are repeated for a higher sill height, as for FB2. For this flume size, ΔY_3 is positive and the design criteria have been met. Where the first selection produces a ΔY_3 that is greater than about 3 inches, a lower and more economical sill should be tried.

Table 1. **Broad-crested weir design and selection**

Canal Data

1. Constructed depth-- d = 3 ft. = 36 in.
2. Bottom width ----- B_1 = 2 ft. = 24 in.
3. Sideslopes ------- Z = 1:1

Flow Data for Canal Without Weir in Place

4. Maximum discharge rate - Q_m = 20 cfs
5. Flow depth for Q_m ----- d_m = 2.08 ft. = 25 in.
6. Freeboard ------------ f_b = 0.92 ft. = 11 in.
(line 1 minus line 5)

Weir Selection Using Tables

			Trial 1	Trial 2	Trial 3	
7.	Trial flume identification:----------------------------		FB1	FB2	FB3	
8.	Bottom width, B_1 (matches line 2)----------------------	B_1 =	2.0	2.0	2.0	ft.
9.	Sill width, B_3 --	B_3 =	4.00	4.50	5.00	ft.
10.	Sideslopes, Z (matches line 3) ----------------------	Z =	1:1	1:1	1:1	-
11.	Critical section (sill) length, L ---------------------	L =	3.00	2.50	2.00	ft.
12.	Trial sill height, S ----------------------------------	S =	1.00	1.25	1.50	ft.
	Submergence - Freeflow Check, Weir in Place					
13.	Sill-referenced flow depth for Q_m, (tables)------------	Y_1 =	1.14	1.09	1.04	ft.
14.	Upstream water depth (line 12 plus line 13)------------	D_1 =	2.14	2.34	2.54	ft.
15.	Actual increase in water depth: (line 14 minus line 5)	ΔY_1 =	0.06	0.26	0.46	ft.
16.	Required increase in water depth: (15% x line 13)-----	ΔY_2 =	0.17	0.16	0.16	ft.
17.	Submergence check: (line 15 minus line 16)------------	ΔY_3 =	-0.11	0.09	0.30	ft.
	(If negative a higher sill must be used, and repeat lines 7-17 with a new trial). (If positive, this flume is okay, but a lower sill might also be used).					
18.	Actual freeboard: (line 1 minus line 14)-------------	f_1 =	-	0.66	0.46	ft.
19.	Required freeboard: (20% x line 13)------------------	f_2 =	-	0.22	0.21	ft.
20.	Freeboard check: (line 18 minus line 19)-------------	f_3 =	-	0.44	0.25	ft.
	(If negative, try a lower sill and repeat lines 7-20) (If positive, this flume is okay, but a higher sill might also be used).					

21. Flume selection: If both checks, (lines 17 and 20), are okay, use that trial.
If one or the other are not met, repeat lines 7-20 for a new trial.
If none of the standard flumes works, see text.

22. Flume selected FB3

Location Example Problem

Comments FB3 was chosen due to uncertainty about the actual water depths. Since ΔY_3 and f_3 are both fairly large, FB3 should work.

How were Q_m and d_m determined? estimated.

The actual freeboard with the flume in place, line 1 minus line 14, is entered in line 18. The required freeboard, 20 percent × line 13, is entered in line 19. If the actual freeboard, f_1, is less than the required freeboard, f_2, $(f_1 - f_2 = f_3 < 0)$ then the freeboard requirement has not been met (line 20).

If the criteria outlined for lines 17 and 29 are both met ($\Delta Y_3 > 0$ and $f_3 > 0$), as in the example for FB2, then the flume trial will work for that situation. If one or the other is not met, a different flume size (sill height) should be tried. For the example, flume FB2 could have been selected; however, if FB3 had been tried first, as shown in trial 3 of table 1, it also would satisfy the criteria of line 19 and could be selected.

For individual situations, the selection may be based on the certainty of the values used for Q_m and d_m. Here, if d_m were actually 27 inches (2.25 ft), ΔY_3 for flume FB2 would be −0.073', and the flume would be submerged. Then flume FB3 would probably be chosen, since there is still plenty of freeboard (line 20, table 1). If two trials work and neither is close to the two limits, then in general, select the lowest sill that satisfies the criteria; in this case FB2 would have been chosen. The reason for choosing the lower sill is that it will cause the least head loss in the canal if it works, and if it doesn't work, the sill can easily be raised by capping it with new concrete. If the largest sill height had been used and the design flow rate had been too low, the canal might overtop and the sill height would be difficult to lower.

In some situations, more than one sill height will work. In other situations none of the flume sizes given here will work. If the highest sill is too low or the lowest sill too high, try another location along the ditch where the flow conditions are different. If a suitable location cannot be found, a new flume size can be computed that will be tailored to the particular site. If a flume sill is too low and the next largest sill is too high, either look for a location where the lower sill can be used or use the larger sill and add additional sidewall height to the ditch upstream from the flume.

There are some limitations on these flumes that are related to the ratio between the throat or sill length, L, and the upstream depth, Y_1, presented in the discussion of parallel flow in the section on Flume Design Criteria (p.4). In table 2, several columns have a horizontal dashed line near the bottom dividing the column (that is, for FA3, between 0.720 and 0.773). If flow depths and corresponding discharges below these lines (higher values) are used, about 6 inches should be added to the throat length, L. This will not have a significant effect on the other discharge ratings as listed above the dashed line.

Table 2. **Flume calibrations—water depth at gage location referenced to weir crest (in feet)**

CANAL SHAPE A $B = 1.0'$ $Z = 1{:}1$				CANAL SHAPE B $B_1 = 2.0'$ $Z = 1{:}1$				CANAL SHAPE C $B_1 = 2.0'$ $Z = 1.25{:}1$			
Flow rate (ft^3/s)	*FA1* B_3=2.50' L=2.00' S=0.75'	*FA2* B_3=3.00' L=1.50' S=1.00'	*FA3* B_3=3.50' L=1.00' S=1.25'	*Flow rate* (ft^3/s)	*FB1* B_3=4.00' L=3.00' S=1.00'	*FB2* B_3=4.50' L=2.50' S=1.25'	*FB3* B_3=5.00' L=2.00' S=1.50'	*Flow rate* (ft^3/s)	*FC1* B_3=5.125' L=3.50' S=1.25'	*FC2* B_3=5.75' L=3.00' S=1.50'	*FC3* B_3=6.375' L=2.50' S=1.75'
0.5	0.160	0.143	0.130	1	0.189	0.175	0.164	2	0.252	0.234	0.217
.6	.179	.161	.146	2	.292	.271	.254	4	.387	.362	.340
.7	.197	.177	.161	3	.375	.350	.328	6	.495	.465	.437
.8	.214	.192	.175	4	.447	.418	.393	8	.589	.554	.523
.9	.230	.207	.189	5	.511	.480	.452	10	.673	.634	.599
1.0	.246	.221	.202	6	.570	.536	.506	12	.749	.708	.670
1.2	.275	.248	.226	7	.625	.589	.556	14	.819	.776	.735
1.4	.302	.273	.249	8	.676	.638	.604	16	.885	.839	.796
1.6	.327	.296	.271	9	.725	.685	.649	18	.947	.899	.854
1.8	.351	.318	.292	10	.771	.729	.691	20	1.006	.956	.909
2.0	.374	.340	.312	11	.814	.771	.732	22	1.061	1.010	.962
2.5	.427	.389	.358	12	.856	.812	.771	24	1.115	1.062	1.012
3.0	.476	.434	.400	13	.896	.851	.809	26	1.166	1.112	1.061
3.5	.520	.477	.440	14	.935	.889	.845	28	1.215	1.159	1.107
4.0	.562	.516	.477	15	.973	.925	.881	30	1.26	1.206	1.153
4.5	.601	.553	.512	16	1.009	.960	.915	32	1.31	1.25	1.196
5.0	.638	.588	.546	17	1.044	.994	.948	34	1.35	1.29	1.24
6.0	.707	.654	.608	18	1.078	1.027	.980	36	1.40	1.34	1.28
7.0	.771	.715	.666	20	1.143	1.091	1.042	38	1.44	1.38	1.32
8.0	.829	.772	.720	22	1.205	1.152	1.101	40	1.48	1.42	1.36
9.0	.885	.825	.773	24	1.26	1.21	1.158	45	1.58	1.51	1.45
10.0	.937	.875	.821	26	1.32	1.27	1.21	50	1.67	1.60	1.54
11.0	.986	.923	.867	28	1.38	1.32	1.26	55	1.75	1.69	1.62
12.0	1.033	.968	.911	30	1.43	1.37	1.31	60	1.84	1.77	1.70
13.0	1.078	1.014	.954	32	1.48	1.42	1.37	65	1.92	1.85	1.78
14.0	1.121	1.056	.994	34	1.53	1.47	1.41	70	1.99	1.92	1.85
15.0	1.162	1.096	1.034	36	1.58	1.52	1.46	75	2.06	1.99	1.92
16.0	1.20	1.135	1.071	38	1.62	1.56	1.50	80	2.13	2.06	1.99
17.0	1.24	1.173	1.108	40	1.67	1.60	1.55	85	2.20	2.13	2.06

Add 6 in. to *L* if depths and discharges below dashed line (higher flow rates) are used.

Flume Construction

Once a satisfactory flume design has been chosen, flume construction is simple and straightforward. The flume has two sections—a ramp and a sill (throat section) (fig. 3). The sill should be level with no large irregularities. The width of the sill, B_3, should be as close to the value given in table 2 as the desired accuracy of measurement (that is, a 1 percent error in B_3 will produce roughly a 1 percent error in discharge). With the wide sills used here, an error of one-half inch in sill width would cause about a 1 percent error. Within limits, the sill length in the flow direction has very little effect on the discharge reading. Errors of 2 or 3 inches in length would cause virtually no change in discharge reading, especially if the error makes the throat longer.

While the sill height is of prime importance to design because it controls the water surface elevation and submergence properties, precise vertical location of the sill is not critical to discharge rating, except when it causes the flume width dimension to deviate significantly. For example, an error of one-half inch in sill height, which can be caused by irregularities in the channel bottom, would change the sill width by 1 inch for a channel with 1:1 sideslopes. This would cause only about a 2 percent error. The flume dimensional tolerances thus allow flumes to be placed in slipform ditches, which vary slightly in shape from the standard. These tolerances should not, however, be an excuse for sloppy or poor construction but can make construction quick and easy.

The upper edge of the ramp should join the sill edge such that it does not rise above the sill to cause unpredictable flow separation. Precision edge matching of the two sections or slight rounding of the joint corner is desirable. When the ramp is hand-troweled into place avoid making large lumps or bumps. The ramp should be fairly uniform across with an approximate 3:1 slope. The ramp may be slightly concave as shown in figure 3. The ramp need not taper to zero thickness but can be ended abruptly when it becomes 2 to 3 inches thick.

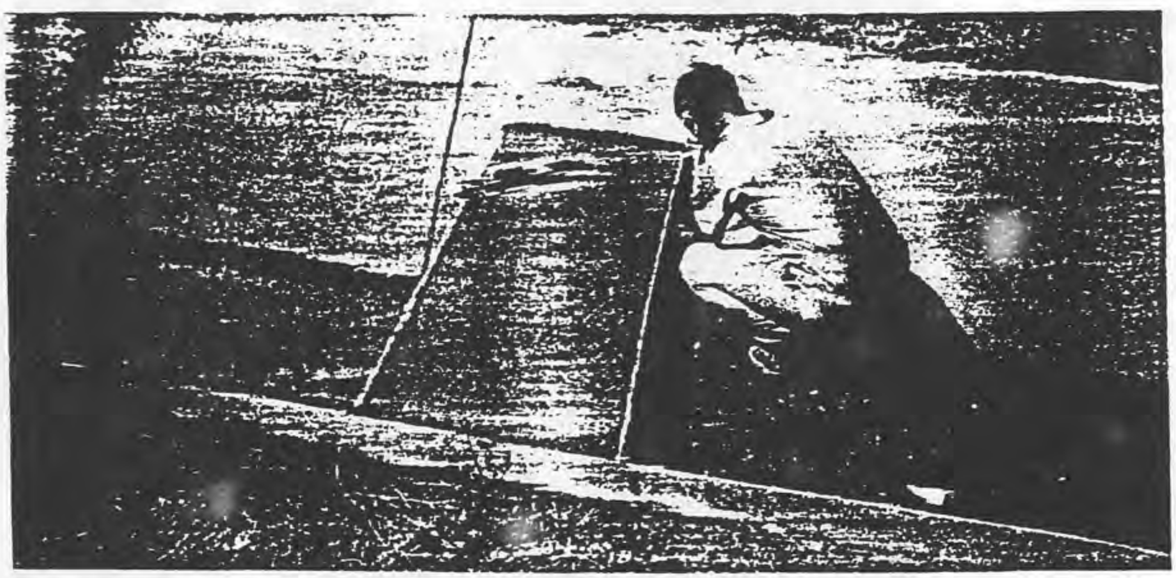
Drain tubes should be installed as shown in figure 3 to allow the ditch to drain for mosquito control. Flow through the drain tubes will usually be negligible compared with flow through the flume. If tubes 2 to 3 inches in diameter or larger are used, the upper ends can be plugged to stop the flow. Sediment, which tends to accumulate at the base of the ramp, may plug the drains and should be cleaned out periodically. Sediment will not significantly accumulate in the ditch behind the flume if it did not normally accumulate in the ditch previously. Special flumes can be designed for sediment-laden ditches where 6 inches or more of sediment is normally deposited between cleanout periods.

Cast-in-place construction is straightforward. Cut out two pieces of plywood, each matching the cross section of the sill as shown in figure 3, with the corners cut out for inserting the drain tubes (cutting out the corners is necessary anyway since few slipform ditches have distinct corners). Cut 2- by

2-inch spacers to length, *L*, for holding the two forms apart. With the spacers between the two plywood forms, wire tie the plywood forms together. This makes the form for pouring the sill. Next, clean out a section of the ditch and place the forms on the bottom of the ditch. Insert the drain tubes and shim the forms so that the tops are level in the cross-ditch direction and with respect to each other. It is easy to see whether the form closely matches the ditch. Small errors are not important. As the form is filled with concrete, remove the 2- by 2-inch spacers. The concrete will hold the plywood apart. Use the plywood forms as screed edges for screeding and troweling the sill to a smooth, level surface.

After the concrete in the sill has taken initial set, the upstream form can be removed and the ramp hand-troweled into place (fig. 4). Both the ramp and sill can be broom-finished.

For single-pour construction, the upstream form can be made of a metal frame constructed from angle iron. This frame provides the upstream screeding edge for finishing the sill and is left permanently in place within the flume structure. Thus, both the sill and the ramp can be constructed at the same time, without waiting or returning to the site.

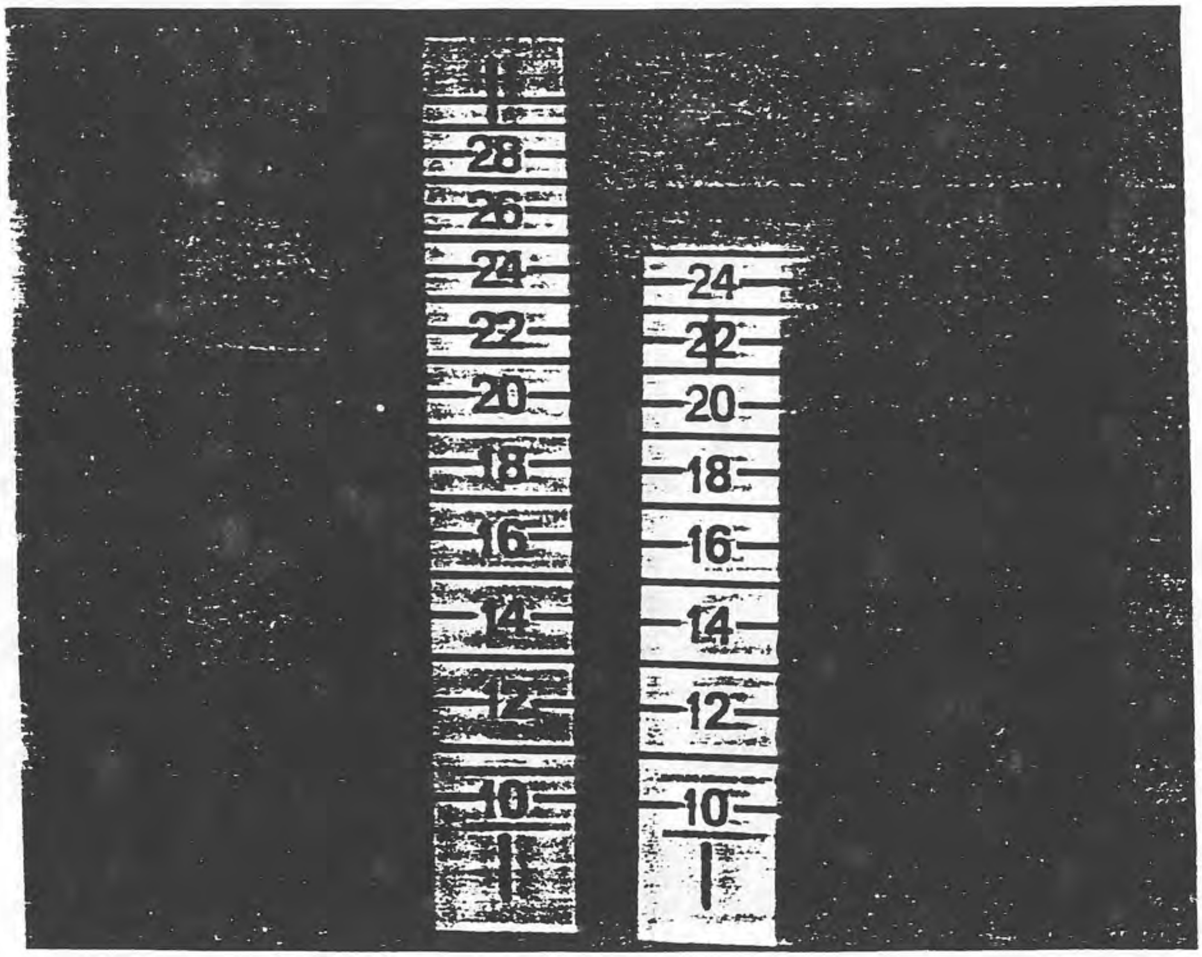


PN–6553

FIGURE 4.—Construction of b-c-w flume FB3 installed in a 30-in ditch near Tacna, Ariz. The sill (throat section) has been poured. The upstream form is being removed so that the ramp can be poured (flow will be from right to left).

Gage Construction and Placement

A convenient type of gage to use is one marked in discharge units and attached to the canal wall. Gages can be constructed of 0.125- by 1.5-inch aluminum bar stock and stamped with a chisel, hammer, and a metal-stamping die set. The gages must be cleaned with a wire brush periodically to remove deposits. Since baked enamel gages are easier to read but must be custom ordered, they are considerably more expensive (fig. 5). The gage is placed on the sidewall and the vertical depths given in table 2 must be converted to sloping distances by multiplying the vertical depths by 1.414,



PN-6554

FIGURE 5.—Commercially available baked enamel, wall-mounted gages marked in ft^3/s. The slotted holes make gage placement easier.

1.414, and 1.601 for ditch shapes A, B, and C, respectively. The discharge values are then marked on the gage at these calculated distances from the gage zero (the zero point need not be marked on the gage).

To obtain accurate discharge ratings, the gage must be carefully surveyed into place with a surveyor's level and rod. The reference elevation for mounting the gage is along the centerline of the canal on top of the sill at about L/4 from the downstream end of the flume (fig. 3). This eliminates errors caused by a nonlevel sill. Often the ditch sideslopes are not exactly as intended, thus a wall-mounted gage could be in error. To eliminate most of the error, the gage is referenced and mounted so that the gage is most accurate at the most commonly used discharge. The greatest errors will then occur at gage readings which are seldom used. Mount the gage so that the mark for the most commonly used discharge is at a vertical distance, Y_1, above the sill reference location, corresponding to the same discharge in table 2.

The gage can be mounted on the wall with lead anchors and screws. A slotted hole can be used to adjust the gage to the proper elevation, or the holes can be drilled in the gage after the anchors are in place. Always check the gage after it has been fastened to make sure that is hasn't slipped. Because of the controlled error in the design and construction processes, which can hold the basic uncertainty of the device to less than ± 2 percent, most of the introduced error will be in the final gage reading. The wall-mounted gage is easily read to within ± 5 percent at most all flows, and for the midrange and higher flows ± 5 percent of the true discharge is easily accomplished. If the water surface is choppy, a skimmer can be placed upstream to reduce the surface waves. Use a stilling well and point gage if greater accuracy is desired.

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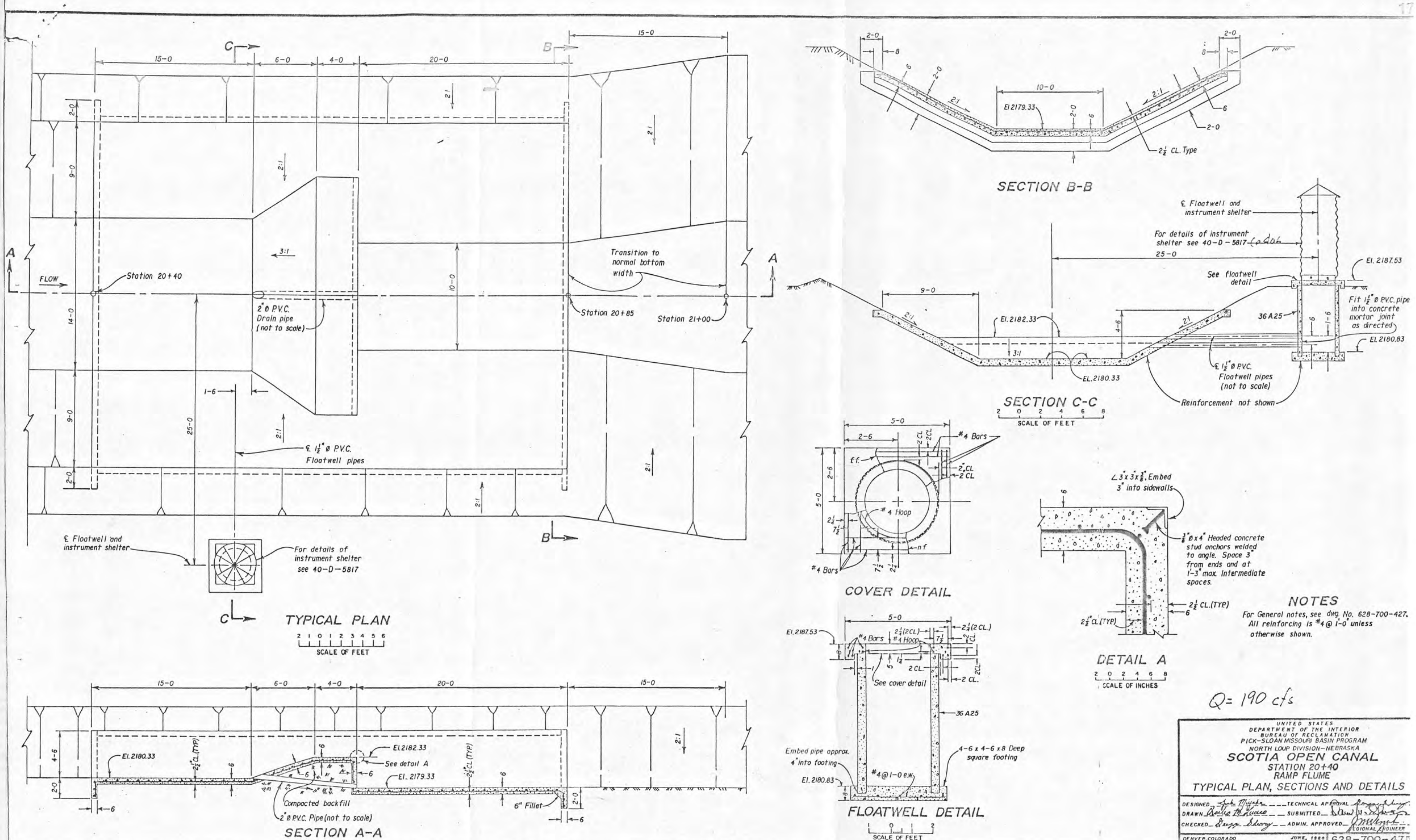
17
TYPICAL PLAN
SCALE OF FEET
FLOW
Station 20+40
Station 20+85
Station 21+00
2" Ø P.V.C. Drain pipe (not to scale)
Transition to normal bottom width
℄ 1½" Ø P.V.C. Floatwell pipes
℄ Floatwell and instrument shelter
For details of instrument shelter see 40-D-5817
SECTION A-A
El. 2180.33
El. 2182.33
El. 2179.33
See detail A
Compacted backfill
2" Ø P.V.C. Pipe (not to scale)
6" Fillet
SECTION B-B
El 2179.33
2½ CL. Type
SECTION C-C
SCALE OF FEET
℄ Floatwell and instrument shelter
For details of instrument shelter see 40-D-5817
El. 2187.53
See floatwell detail
Fit 1½" Ø P.V.C. pipe into concrete mortar joint as directed
El. 2182.33
El. 2180.83
El. 2180.33
℄ 1½" Ø P.V.C. Floatwell pipes (not to scale)
Reinforcement not shown
COVER DETAIL
#4 Bars
#4 Hoop
FLOATWELL DETAIL
SCALE OF FEET
El. 2187.53
See cover detail
36 A25
Embed pipe approx. 4" into footing
El. 2180.83
#4 @ 1-0 e.w.
4-6 x 4-6 x 8 Deep square footing
DETAIL A
SCALE OF INCHES
∠3"x3"x⅜". Embed 3" into sidewalls
⅜" Ø x 4" Headed concrete stud anchors welded to angle. Space 3" from ends and at 1-3" max. Intermediate spaces.
2½" CL.(TYP.)
NOTES
For General notes, see dwg. No. 628-700-427.
All reinforcing is #4 @ 1-0" unless otherwise shown.
Q= 190 cfs
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PICK-SLOAN MISSOURI BASIN PROGRAM
NORTH LOUP DIVISION-NEBRASKA
SCOTIA OPEN CANAL
STATION 20+40
RAMP FLUME
TYPICAL PLAN, SECTIONS AND DETAILS
DESIGNED
DRAWN
CHECKED
TECHNICAL APPROVAL
SUBMITTED
ADMIN. APPROVED
REGIONAL ENGINEER
DENVER, COLORADO
JUNE, 1980
628-700-475

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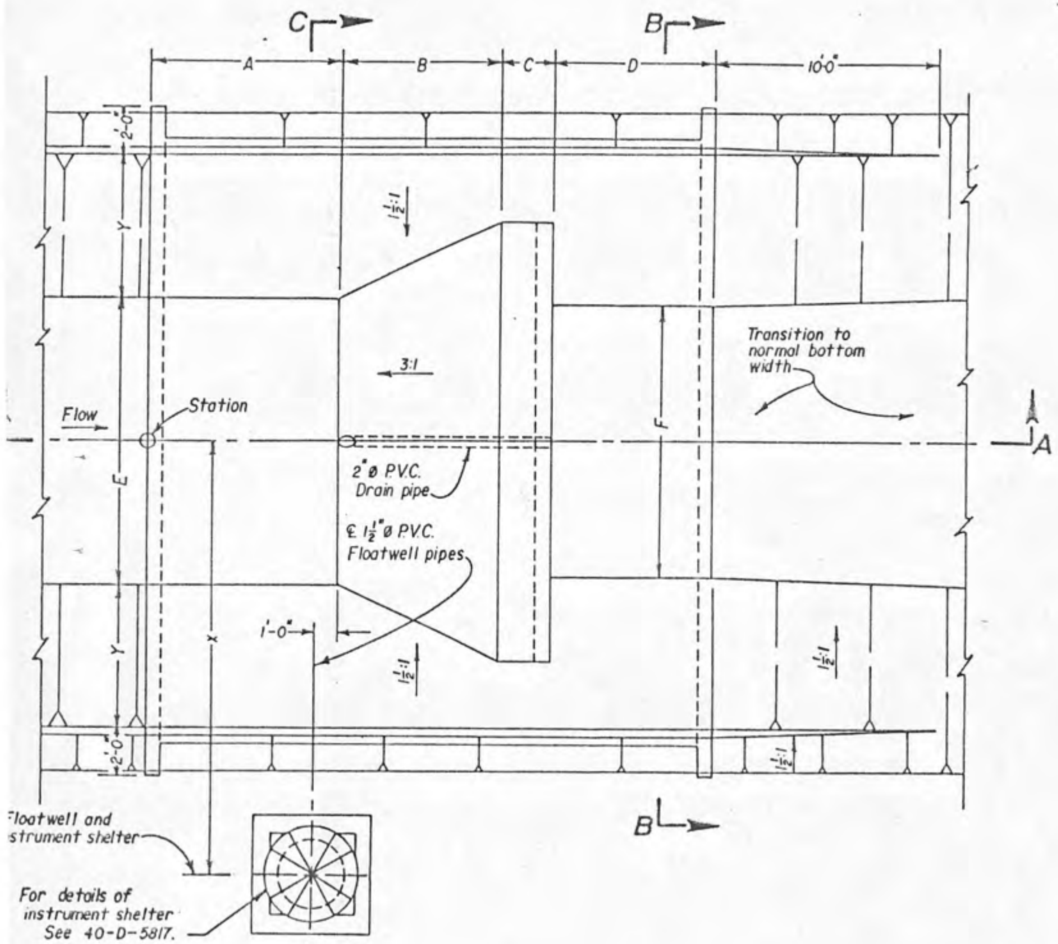
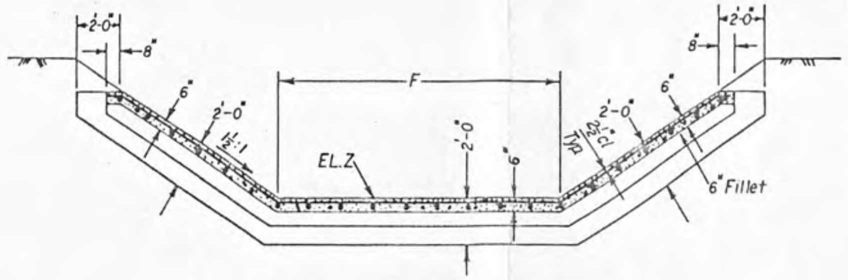
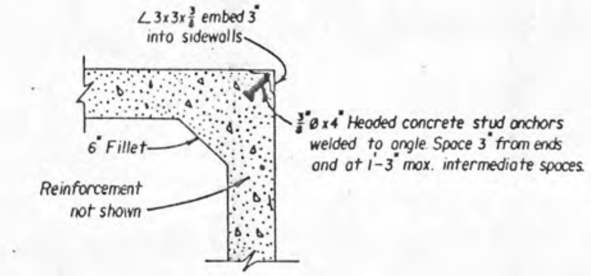


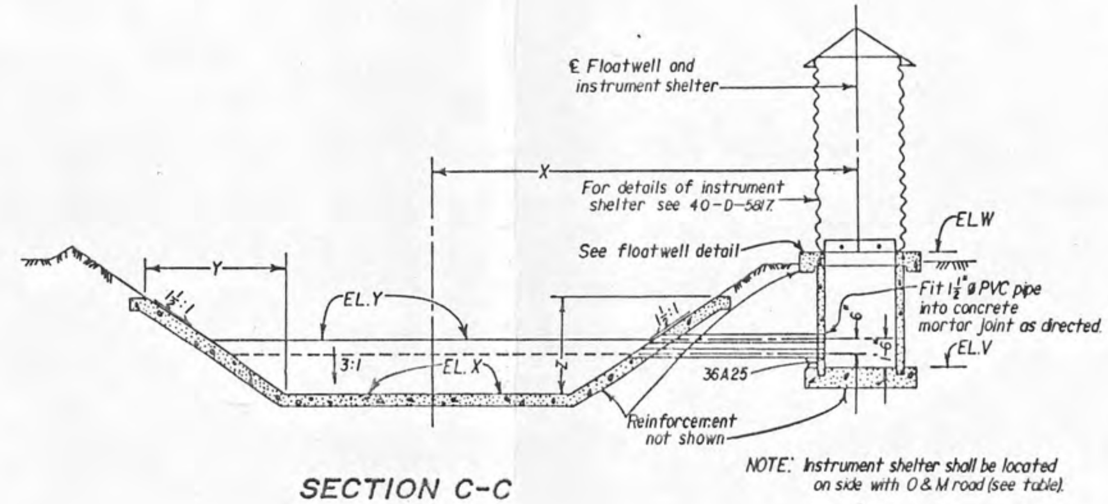
TYPICAL PLAN



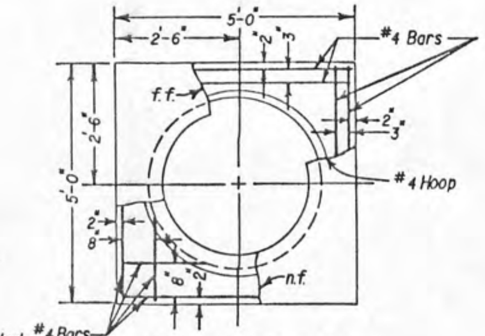
SECTION B-B



DETAIL A



SECTION C-C



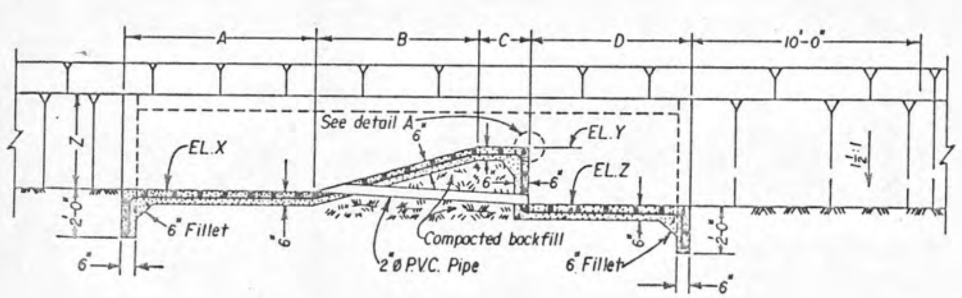
COVER DETAIL

NOTES

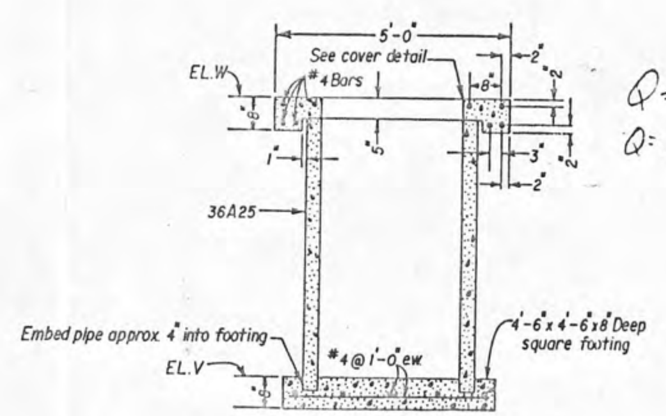
For General notes see dwg. No. 628-700-260.
All reinforcing is #4 @ 1-0" unless otherwise shown.

DIMENSIONS & ELEVATIONS

	LATERAL, STATION	A	B	C	D	E	F	X	Y	Z	EL. V	EL. W	EL. X	EL. Y	EL. Z
Q = 24 cfs	LAT. 6.1 Sta 1+80	5-0	3-0	2-0	10-0	5-0	2-0	13-0 Rt	4-6	3-0	2300.61	2306.31	2301.11	2302.11	2300.11
Q = 60 cfs	LAT. 11.9 Sta 13+60	7-0	3-10⅞	2-6	6-6	10-0	8-6	18-0 Lt	4-6	3-0	2284.24	2291.04	2284.44	2285.74	2283.94



SECTION A-A



FLOATWELL DETAIL

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PICK-SLOAN MISSOUR BASIN PROGRAM
NORTH LOUP DIVISION-NEBRASKA

GERANIUM OPEN LATERALS

LATERALS 6.1, 11.9
RAMP FLUME
TYPICAL PLAN, SECTIONS, AND DETAILS

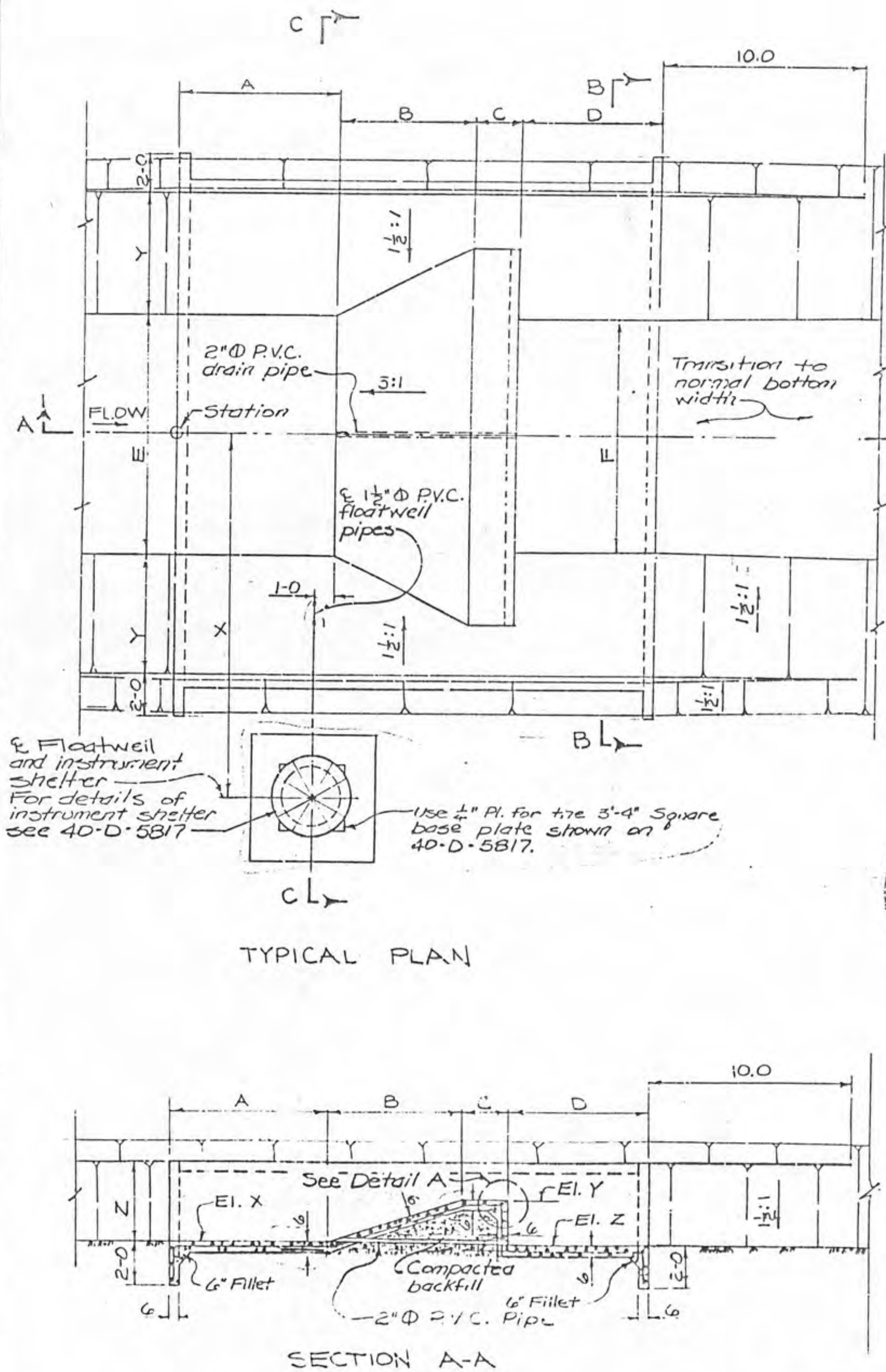
DESIGNED _______ TECHNICAL APPROVAL G. O. Thomas
DRAWN _______ SUBMITTED G. O. Thomas
CHECKED _______ ADMIN. APPROVED _______
REGIONAL ENGINEER

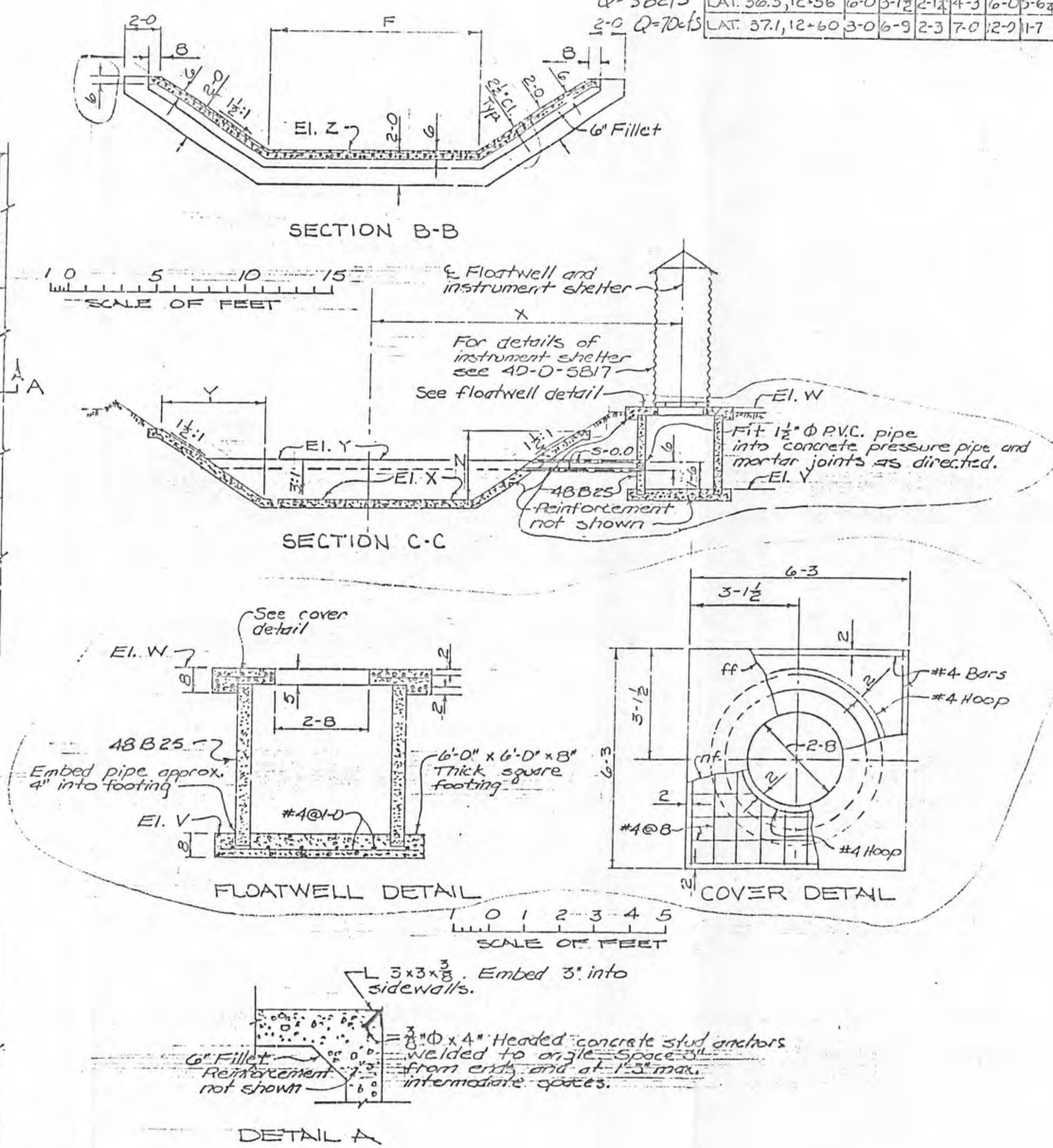
DENVER, COLORADO JULY 31, 1984 628-700-299

FM

TABLE OF DATA

	LATERAL, STATION	A	B	C	D	E	F	X	Y	Z	EL. V	EL. W	EL. X	EL. Y	EL. Z
Q = 24 cfs	LAT. 25.5, 14+50	6-0	5-2	1-9	5-6	5-0	4-7⅝	4-6	4-6	3-0	2185.37	2190.40	2185.15	2186.87	2185.03
Q = 36 cfs	LAT. 36.3, 12+56	6-0	3-7½	2-1¼	4-3	6-0	5-6¼	13-6	4-1½	2-9	2166.54	2171.13	2166.83	2168.64	2166.67
Q = 70 cfs	LAT. 37.1, 12+60	3-0	6-9	2-3	7-0	2-0	1-7	18-6	6-0	4-0	2175.06	2179.91	2174.31	2176.56	2174.17





NOTES

All reinforcing is #4@1-0 unless otherwise shown.
For General Notes see 628-D-

2-17-84 D- RAP	Checked. Minor revisions. Signatures added. Note added.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PICK-SLOAN MISSOURI BASIN PROGRAM
NORTH LOUP DIVISION-NEBRASKA

MIRDAN LATERALS

LATERALS 25.5, 36.3, AND 37.1

RAMP FLUME
TYPICAL PLAN, SECTIONS, AND DETAI

DESIGNED [signature] TECHNICAL APPROVAL [signature]
DRAWN [signature] SUBMITTED [signature]
CHECKED [signature] APPROVED [signature]
CHIEF, WATER CONVEYANCE BRA

DENVER, COLORADO NOVEMBER 1, 1982 628-D-85