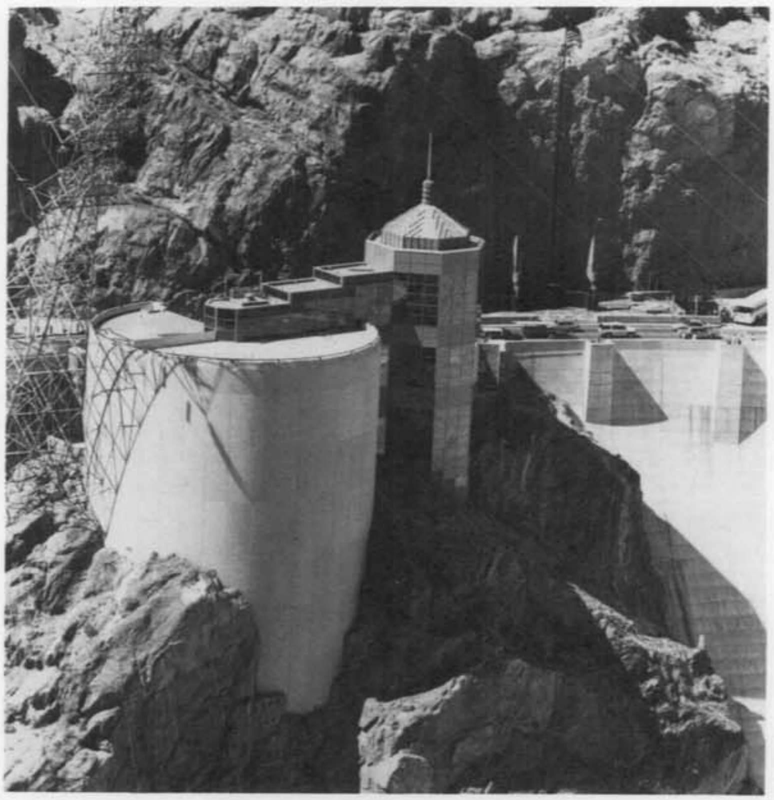
WATER OPERATION AND MAINTENANCE

BULLETIN NO. 174

December 1995



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UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

The Water Operation and Maintenance Bulletin is published quarterly for the benefit of water supply system operators. Its principal purpose is to serve as a medium to exchange information for use by Reclamation personnel and water user groups for operating and maintaining project facilities.

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Cover photograph: View of the Visitor Center and a partial view of Hoover Dam.

UNITED STATES DEPARTMENT OF THE INTERIOR

Bureau of Reclamation

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WATER OPERATION AND MAINTENANCE BULLETIN

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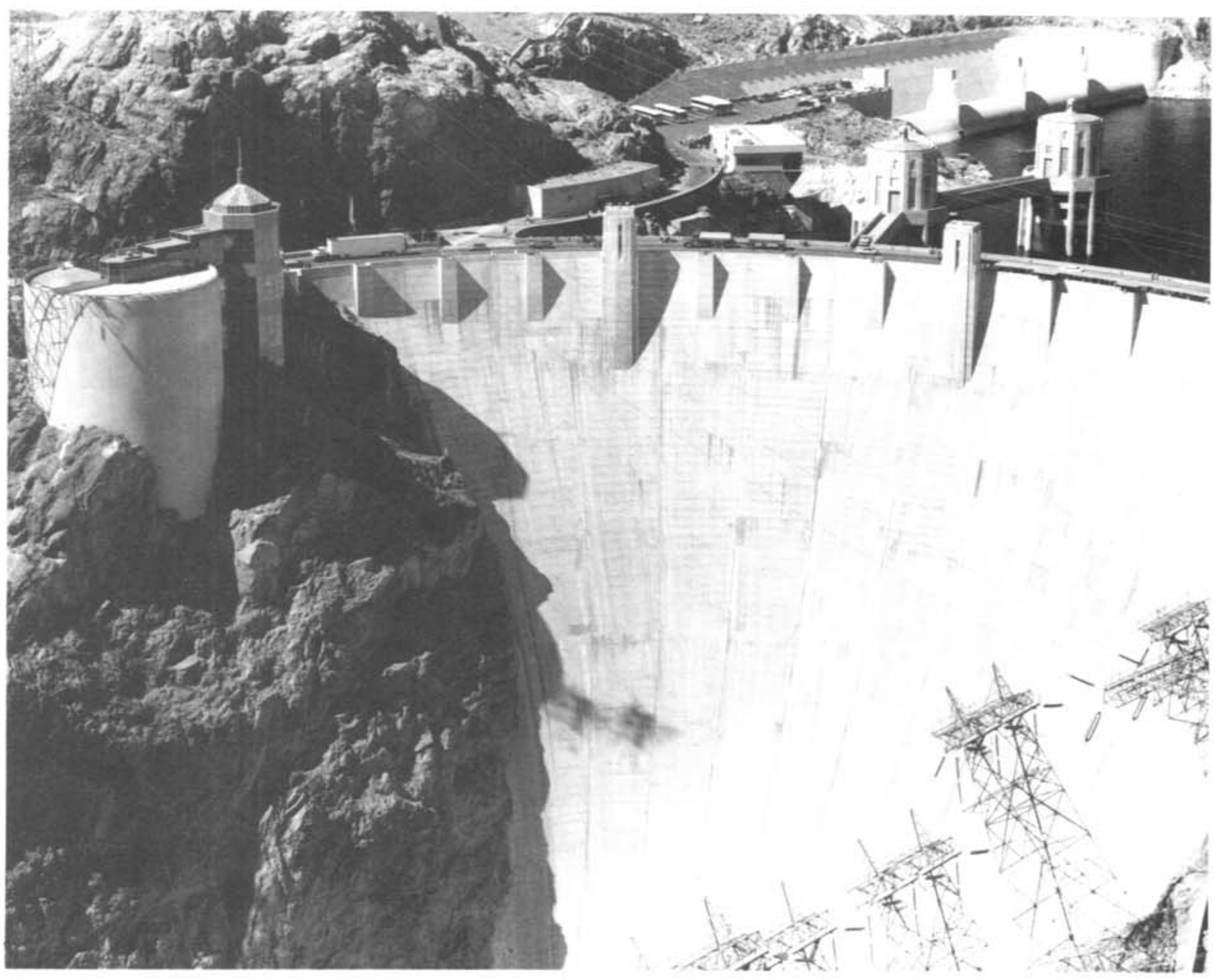
HOOVER DAM VISITOR FACILITIES ATTRACT RECORD CROWDS

by Colleen Dwyer

In the first 3 months of operation, the new Hoover Dam visitor facilities have hosted record numbers of people. "In the first month after the center opened (June 21 to July 21), over 103,000 people toured the dam—nearly double the May 1995 amount," said Dan Jensen, Hoover Dam Visitor Services Manager. "In July, over 106,000 tickets were sold, setting the record for the most tours ever conducted in a single month."

This increased visitation in the first 9 months of 1995 to nearly 700,000 almost equals the entire 12-month total in 1994.

The Visitor Center is now operating with temporary exhibits and one of three theater bays in operation. In addition, paintings of Bureau of Reclamation (Reclamation) projects rendered by renowned artists are now displayed in the Exhibit Hall where they will remain until the permanent exhibits are installed next year. The 67 paintings are part of a collection commissioned by Reclamation in the late 1960's which records the role of Reclamation and water in the West through the imaginations of some of America's finest artists.



Partial view of Hoover Dam with the Visitor Center shown in the upper left of this photo.

The reduced interaction between vehicles and pedestrians has also enabled more traffic to move across the dam safely. Over 407,000 vehicles crossed the dam in July—a record number which surpassed the July 1994 total by nearly 131,000.

A long-planned complete interpretive program featuring new permanent exhibits, audiovisual programs, and other features is now being developed and should be installed by fall 1996. Over the next year, Reclamation also anticipates that boat access to the dam and Visitor Center will be available through an agreement with Lake Mead Cruises; a 5-mile trail from the National Park Service's Alan Bible Visitor Center to the dam will also be completed, offering additional access for hikers or mountain bikers; and the States of Arizona, California, and Nevada will install a copper and bronze exhibit on the Observation Deck level.

With the current rate of visitation, the new facilities are expected to permit over 1.25 million visitors to tour Hoover Dam annually; the maximum number the dam accommodated previously was about 790,000 people. Nearly 31 million people have toured the site since it was first opened to the public in 1937.

LABORATORY AND FIELD EVALUATION OF ACOUSTIC VELOCITY METERS AT HOOVER, DAVIS, AND PARKER DAMS

by Tracy Vermeyen[1]

INTRODUCTION

The material for this article is part of a study requested by the Bureau of Reclamation's (Reclamation) Lower Colorado Regional Office. The Project Manager was Albert Marquez. The purpose of the study was to improve the flow measurement at the major dams along the lower Colorado River—namely Hoover, Davis, and Parker Dams. This study is only one of many being conducted in support of the Lower Colorado River Accounting System (LCRAS) program. LCRAS is a water management computer program which will allow Reclamation to better utilize water resources in the Lower Colorado River basin.

In an effort to improve the accuracy of flow measurement at Hoover, Davis, and Parker Dams, a two-stage study was initiated. The first stage was to evaluate the existing flow measurement system, which consists of Accusonic acoustic velocity meters (AVM's) with four or eight acoustic paths mounted on the turbine penstocks. Field surveys were conducted to determine if all 27 AVM installations conformed to ANSI/ASME standards and ASME's Performance Test Code for hydraulic turbines. The second stage was to determine if the AVM installations were performing to manufacturer's specified accuracies of 0.5 percent of the true discharge. A published error analysis by the AVM manufacturer (Lowell and Hirschfeld, 1979) did not adequately address the error related to the integration of an asymmetrical velocity distribution. To verify the flowmeter's integration techniques when applied to an asymmetrical velocity distribution, physical models were used to determine the penstock velocity distributions at the AVM measurement sections. Model and field study results were used to establish overall uncertainty bounds on discharge measurements and to develop modifications which will improve discharge measurement accuracy.

MODEL STUDY CONCLUSIONS

This article will not go into the details of the model studies but will include the model study results. If further information is needed, the author can be contacted at (303) 236-2000, extension 451. A report for this study will be available by the end of fiscal year 1995.

Davis Penstock No. 5 Model

The model study identified an asymmetrical velocity distribution for Davis Penstock No. 5 for all discharges tested. A combined bend just upstream from the AVM measurement cross section creates a secondary current which results in a reduced velocity along the inside of the bend (figure 1). Data analysis showed that for this asymmetrical velocity distribution, velocities measured along the four acoustic paths were considerably different depending on acoustic path orientation. Discharge

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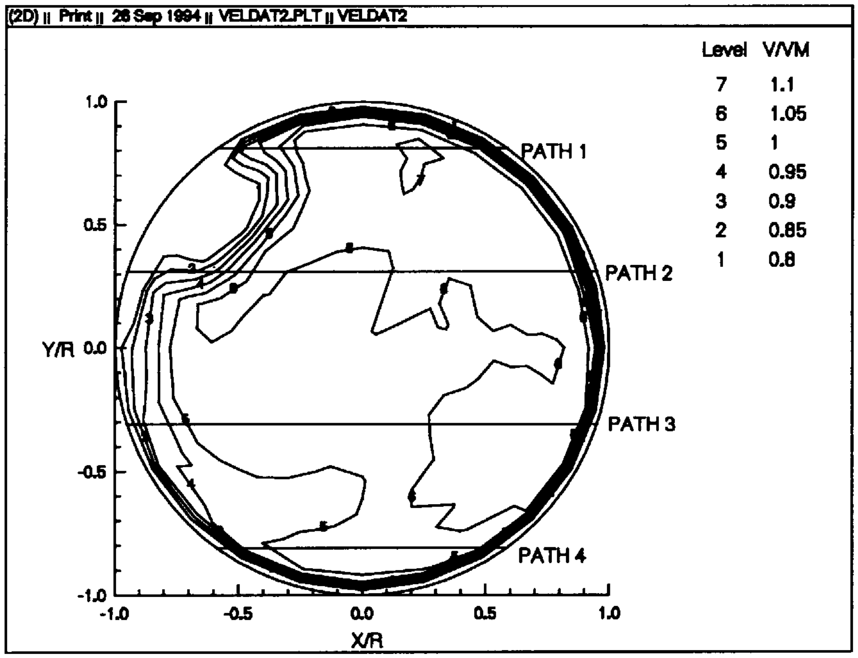

Figure 1.—Davis Penstock No. 5 nondimensional velocity distribution (looking downstream) for a prototype discharge of 5,373 cubic feet per second and reservoir elevation 570 feet. The AVM computed flow was biased -0.31 percent from the actual flow.

measurement errors as large as 2 percent were measured. An analysis to determine the optimum path orientation (with respect to rotation around the pipe axis) showed the existing condition, horizontal acoustic paths, is optimum (figure 2). For a horizontal path orientation, integration errors in Accusonic's discharge measurement calculations for model tests 2 through 4 were found to be -0.31, -0.44, and -0.75 percent, respectively.

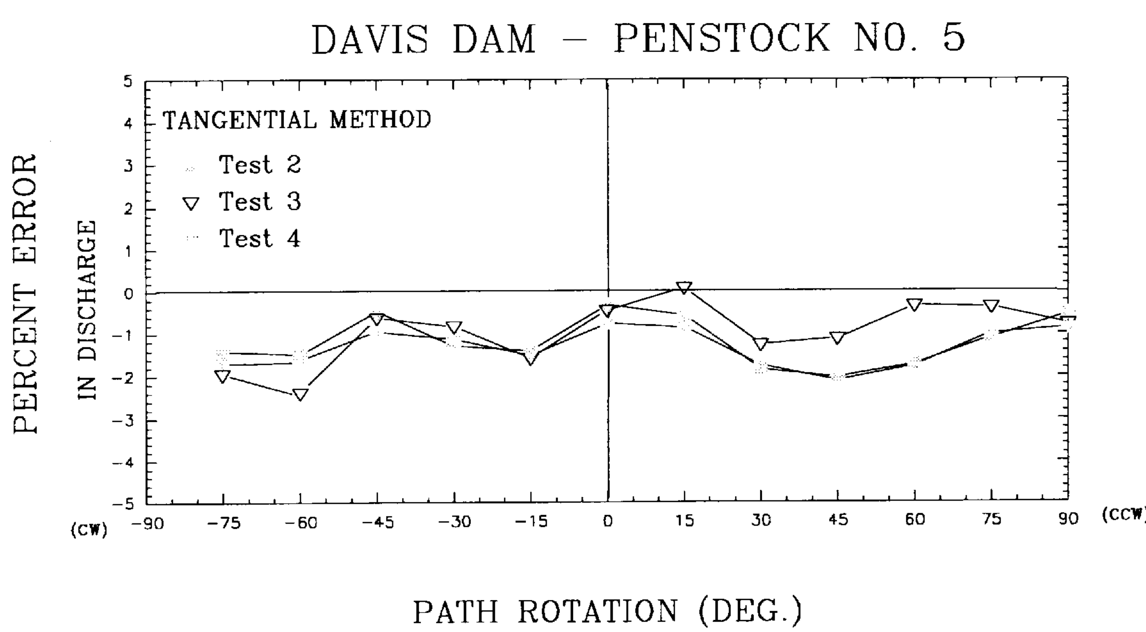

Figure 2.—Percent error in discharge measurement as a function of path rotation. This plot indicates the best transducer configuration is the existing horizontal acoustic paths.

Parker Penstock No. 1 Model

A nearly symmetrical velocity distribution was identified for Parker penstock No. 1 for all discharges tested (figure 3). A combined bend upstream from the AVM measurement cross section creates a slightly skewed velocity distribution and lower velocities near the pipe invert. Data analysis showed that for this particular velocity distribution, velocities measured along the four acoustic paths are very similar, and path velocities are essentially independent of path orientation.

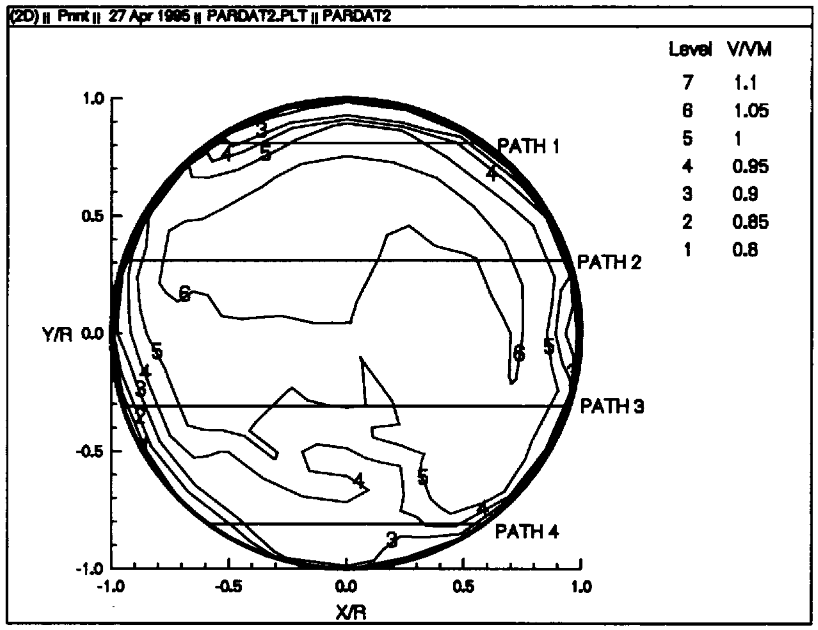

Figure 3.—Parker penstock No. 1 nondimensional velocity distribution (looking downstream) for a prototype discharge of 4,736 cubic feet per second and reservoir elevation 449 feet. The AVM computed flow was biased -0.18 percent from the actual flow.

FIELD STUDY CONCLUSIONS

All 18 AVM installations at Hoover Dam conform to ANSI/ASME standards. However, all AVM installations at Davis and Parker Dams are nonstandard because they do not conform to the required length of straight pipe upstream and downstream from the AVM measurement section. Furthermore, AVM installations at Davis and Parker Dams do not meet the requirement in ASME's PTC 18, which states: "The intersection of crossed acoustic planes shall be in the same plane as the upstream bend to minimize the effects of the cross flow components on the accuracy of the measurement."

Field data analysis identified cross flow errors at Davis penstock No. 5 and Parker penstock No. 1 and were measured to be 0.5 and 1.8 percent, respectively. These errors are compensated for by using crossed acoustic planes. Therefore, all penstocks with single plane AVM's are likely to have some degree of cross flow error.

Crossed plane AVM's are recommended on all penstocks at Davis and Parker Dams, except for Parker penstock No. 4. Analysis of path velocity data from Parker penstock No. 3 indicates a minimal cross flow error. Therefore, because Parker penstock No. 4 has better flow conditions than penstock No. 3, it is reasonable to conclude that crossed plane AVM's are not necessary for accurate discharge measurements for penstock Nos. 3 and 4.

BASIC AVM OPERATION[2]

Operation and theory of acoustic velocity meters are thoroughly described in ANSI/ASME standard MFC-5M-1985. The following section will provide a brief overview of transit-time acoustic velocity meters.

Transit-Time Acoustic Velocity Meters

Transit-time acoustic velocity meters are based on the principle that the transit time of an acoustic signal along a known path is altered by the fluid velocity. An acoustic signal sent upstream travels slower than a signal traveling downstream. By accurately measuring the transit times of signals sent in both directions along a diagonal path, the average axial velocity can be calculated. Then, using the known path length and path angle, with respect to the direction of flow, the average axial velocity can be computed (figure 4).

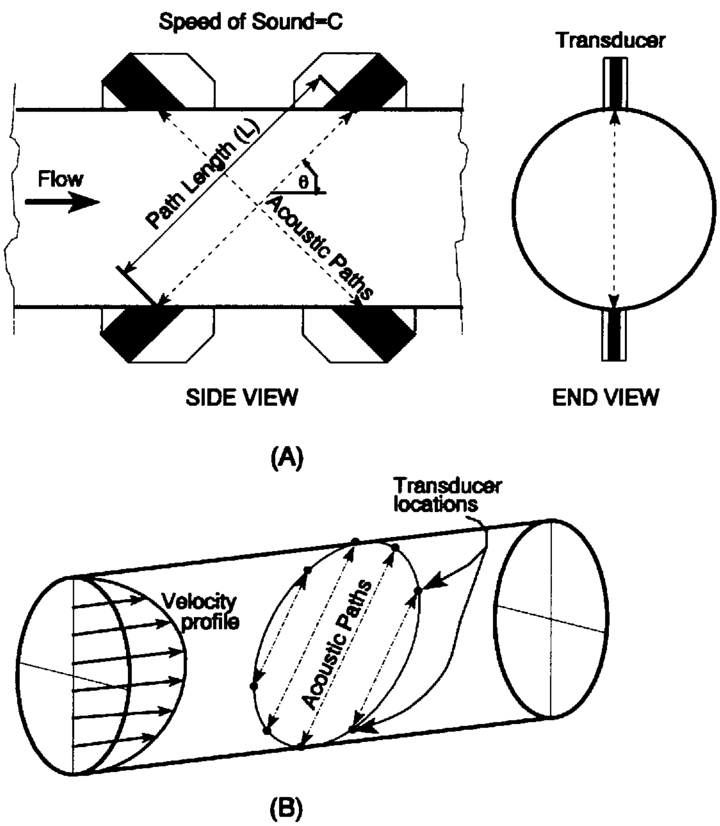

Figure 4.—Transit-time acoustic flowmeters: (a) crossed, diametral path configuration and (b) single plane chordal path configuration.

AVM FIELD EVALUATIONS

To determine the accuracy of flow measurement at Hoover, Davis, and Parker Dams, field surveys were conducted in September 1992 to document and review AVM equipment, AVM system parameters, as-built drawings, and perceived system performance. Each of the 27 AVM sites and installations was evaluated using ANSI/ASME Standard MFC-5M-1985, *Measurement of Liquid Flow in Closed Conduits Using Transit-Time Ultrasonic Flowmeters.* Likewise, ASME's Performance Test Code for Hydraulic Turbines (PTC 18-1992) was used in evaluations because it is the standard procedure for performing turbine performance tests and is in some instances more stringent than the ANSI/ASME standard.

[2] Information on AVM theory is available from Mr. Vermeyen.

Standard and Nonstandard Installations

Surveys at Hoover, Davis, and Parker Dams resulted in a large amount of site-specific data and personal opinions as to how the AVM systems were performing. Survey information is summarized as follows below.

Hoover Dam

Eighteen AVM's at Hoover Dam were installed from 1989 to 1991. A review of AVM equipment, system parameters, and as-built drawings at Hoover Dam revealed that all AVM installations satisfied ANSI/ASME standards and were configured properly. On average, there are 30 pipe diameters upstream and 4 pipe diameters downstream from the AVM measurement sections. The only exception to ASME's PTC 18 is that not all penstocks are equipped with crossed acoustic planes. Single acoustic planes were used on every other penstock because cross flow information from an adjacent penstock could be measured and applied to the penstock with one acoustic plane. This practice reduced equipment and installation costs.

Davis Dam

Five AVM's were installed in 1989. A review of AVM equipment, system parameters, and as-built drawings revealed that all five AVM installations were nonstandard because of inadequate lengths of straight pipe upstream and downstream from the meter section. ASME's PTC 18 recommends a minimum of 10 pipe diameters upstream and 3 pipe diameters downstream from the measurement section. The amount of straight pipe upstream from the meter section ranged from 1/2 to 1-1/2 diameters for the five 22-foot-diameter penstocks (figure 5). Better locations were not available because of short penstock lengths. All AVM's were installed just upstream from the turbine scroll cases to maximize the length of straight pipe upstream. Because of short penstock lengths and bends upstream, cross flows (flows with nonaxial velocity components) were anticipated. Crossed plane AVM's are typically used in difficult installations to compensate for cross flow errors. The shortest of the five penstocks, penstock No. 5, was equipped with a crossed plane AVM system. It should be noted that ASME's PTC 18 requires installation of two four-path measurement planes, and that the intersection of the two planes shall be in the plane of the upstream bend. The crossed plane AVM installation on penstock No. 5 at Davis Dam does not meet these criteria.

Parker Dam

Four AVM's were installed at Parker Dam in 1989. A review of AVM equipment, system parameters, and as-built drawings at Parker Dam revealed that all four AVM installations were nonstandard because of inadequate lengths of straight pipe upstream and downstream from the meter section. The length of straight pipe upstream from the meter section ranged from 1/2 to 6 pipe diameters for the four 22-foot-diameter penstocks (figure 6). These lengths could not be increased because of the short penstocks. As at Davis Dam, all AVM's were installed just upstream from the turbine scroll cases to maximize the length of straight pipe upstream from the meter section. Two of the four penstocks (Nos. 1 and 3) were equipped with crossed plane AVM systems, including the shortest penstock. The crossed plane AVM installations at Parker do not meet ASME's PTC 18-1992 requirement on acoustic path orientation with respect to the upstream bend.

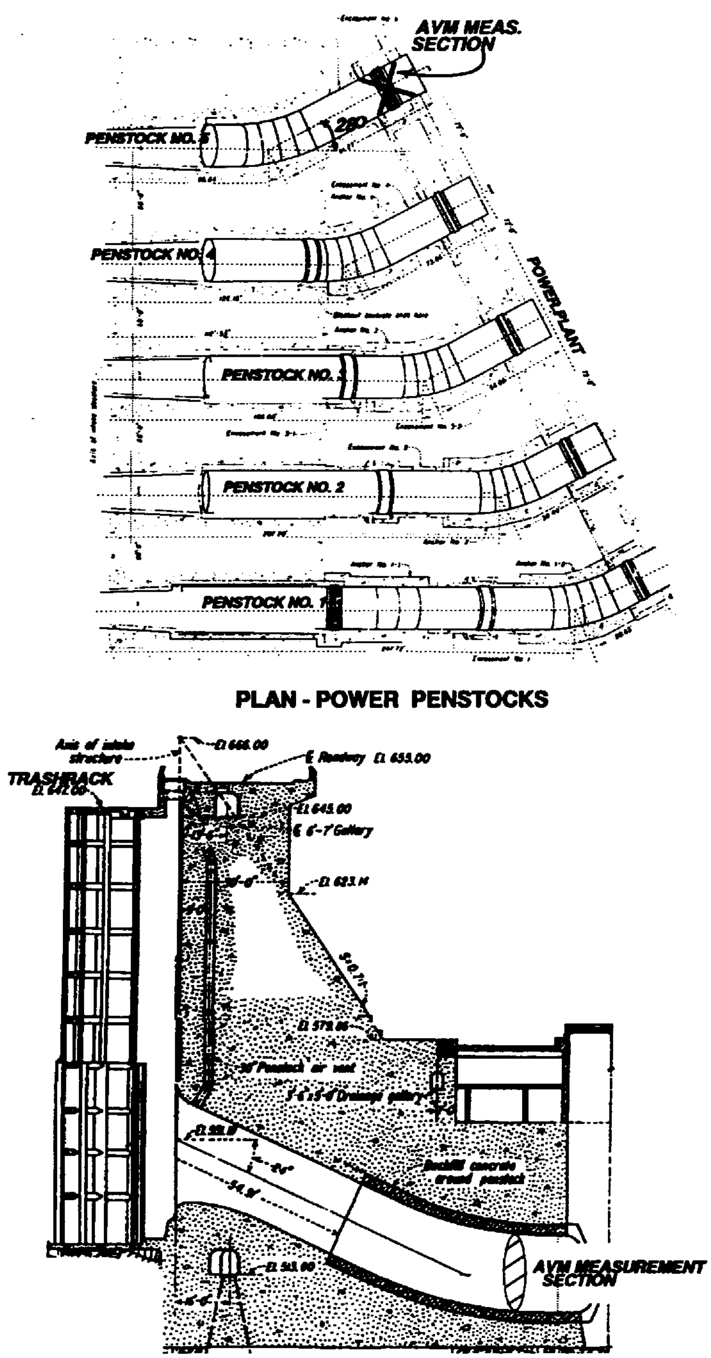

Figure 5.—Plan and section of penstock No. 5 at Davis Dam. The AVM measurement section is shown at the end of the penstock. The combined bend has a 24° vertical and a 28° horizontal angle.

General Findings

AVM system operators felt their systems were operating satisfactorily. However, interviews indicated a disparity in knowledge levels among AVM system operators. Degrees of expertise in system testing and troubleshooting varied depending on the AVM maintenance history. To alleviate this problem, it is recommended that a training course be given to all AVM system operators. It was also apparent that an experienced electronics technician is necessary to effectively operate and maintain an AVM system. We also recommended developing a data base to log maintenance and repair information, as well as keeping records of system parameters and error logs.

AVM Data Analysis

Individual path velocities and discharge values were collected for the crossed plane AVM's at Davis and Parker Dams to determine the errors associated with cross flows and asymmetrical velocity distributions. Data were collected using a laptop computer to log the real-time flow meter data. Figure 7a contains a typical sample (120 measurements taken over 2 minutes) of path velocity data collected from Davis penstock No. 5 for a 65-percent gate opening. This penstock is equipped with a crossed plane AVM, so a total of eight path velocities and two discharges (one discharge measurement for each crossed plane) was recorded. Paths 1 and 4 are the upper and lowermost acoustic paths. Paths 2 and 3 are located in between paths 1 and 4.

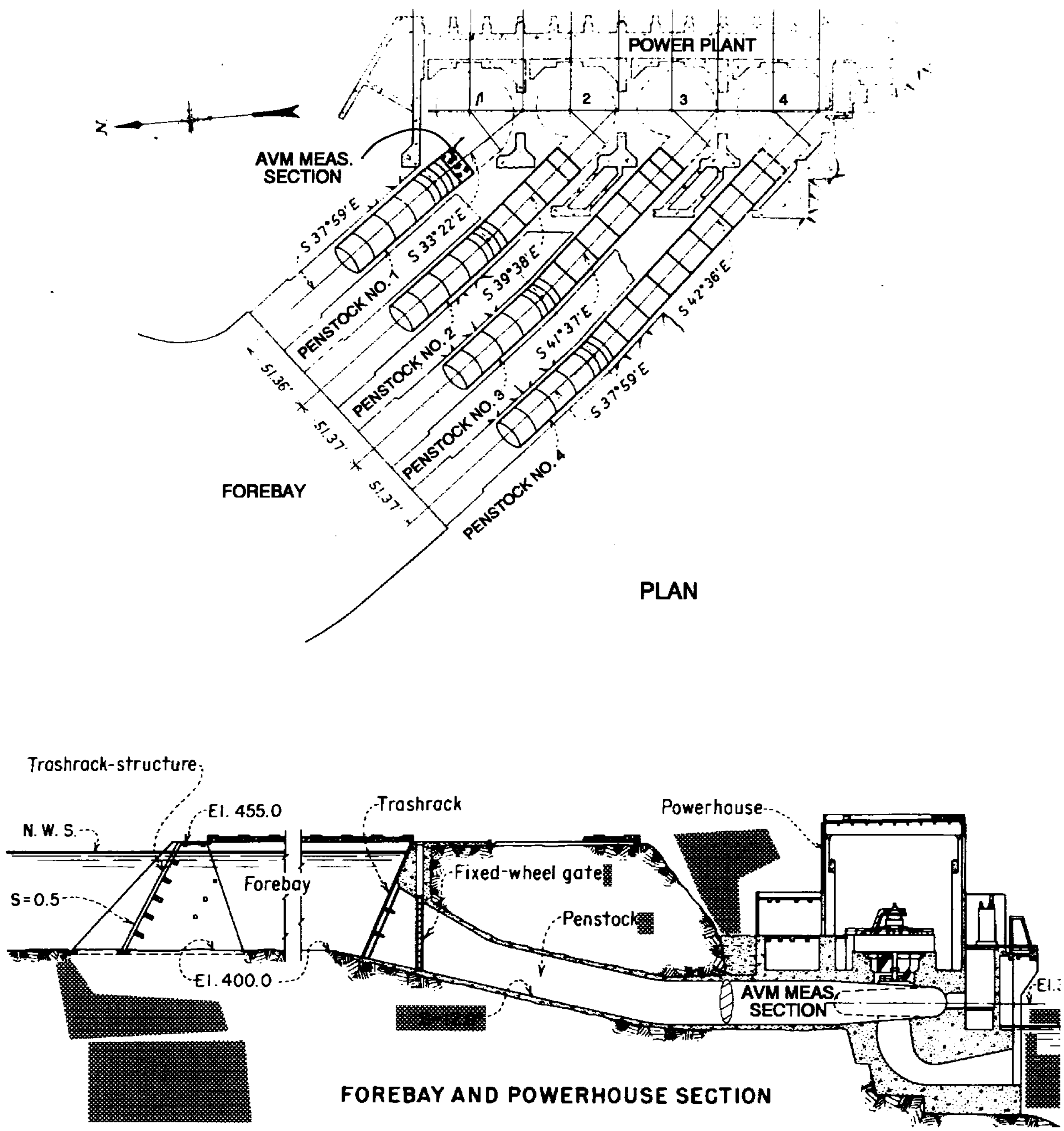

Figure 6.—Plan and section of penstock and powerhouse at Parker Dam. For penstock No. 1, the bend is located immediately upstream from the AVM transducer section and has a 12.9° vertical and a 4.6° horizontal angle.

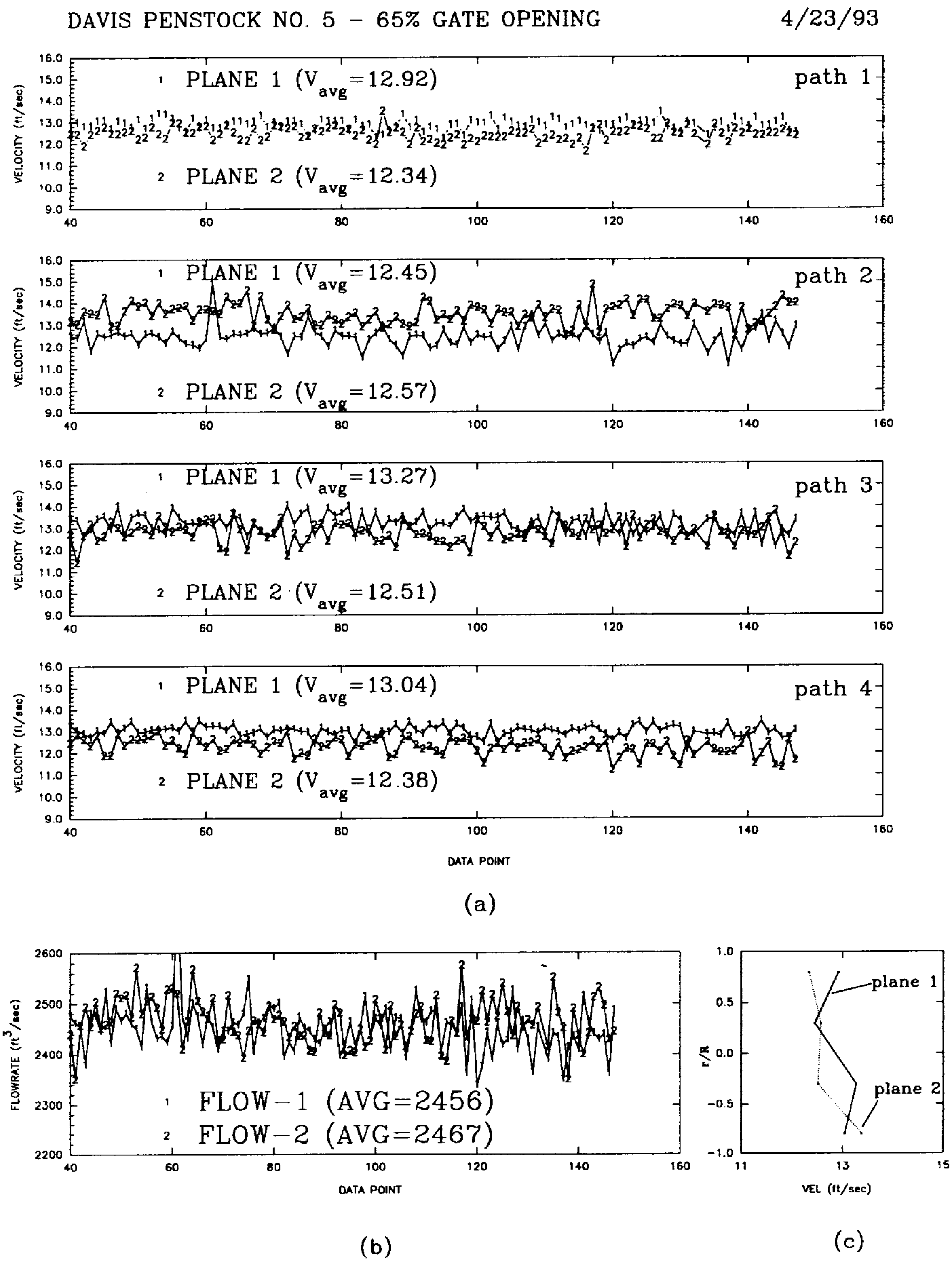

Figure 7.—(a) Individual path velocity data for crossed acousitic planes on penstock No. 5. (b) Instantaneous flow rates for acoustic planes 1 and 2. (c) Average velocity profiles for acoustic planes 1 and 2.

DAVIS DAM ACOUSTIC VELOCITY METER EVALUATION

Field tests were conducted to collect real-time data from the Accusonic Model 7410 AVM's at Davis Dam. Path velocities and discharge data were collected for a wide range of wicket gate openings. Of the five penstocks at Davis Dam, only No. 5 has a crossed plane AVM. Penstock No. 5 was chosen for the crossed plane installation because it is the shortest penstock. However, it also has the longest section of straight pipe upstream from the measurement section (see figure 5). As a result, penstock Nos. 1 through 4 may have velocity distributions which are worse than No. 5.

AVM data were analyzed to identify problems with the data collection, AVM setup, cross flow, and skewed velocity distributions. A summary of the data analysis for penstock No. 5 is as follows below.

Penstock No. 5

This penstock has 1-1/2 diameters of straight pipe upstream from the measurement section. The combined bend has a vertical curve of 24° and a horizontal curve of 28°. Analysis of AVM data collected for many gate openings (11, 20, 30, 40, 50, 60, and 64 percent) at a reservoir elevation of 630.8 feet resulted in the following observations:

- Velocity data indicate close agreement between the two acoustic planes (see figure 7a), but the average velocity profiles are skewed and have different shapes (see figure 7c), which indicates poor flow conditions caused by the short penstock and combined bend upstream.

- For all wicket gate openings tested, the two crossed acoustic planes produce discharge measurements which are offset by an average of 1.06 percent (see table 1 and figure 7b), which is a systematic error in the AVM discharge measurement. This offset is caused by a cross flow velocity component through the AVM measurement section. As a result, this penstock requires cross plane measurements.

- The average of Flow-1 and Flow-2 is a discharge corrected for cross flow error. For the wide range of wicket gate openings tested, Flow-2 values were on average 1.06 percent higher than Flow-1 values. Therefore, if one plane of transducers should fail, a correction should be made as follows: increase Flow-1 value by 0.53 percent or decrease Flow-2 value 0.53 percent.

- Given the similar geometry of penstock Nos. 1 through 4 to penstock No. 5, cross flows are probably also occurring in those penstocks.

PARKER DAM ACOUSTIC VELOCITY METER EVALUATION

Field tests were conducted to collect real-time data from the Accusonic Model 7410 AVM's at Parker Dam. Path velocities and discharge data were collected for a wide range of wicket gate openings (table 2). Of the four penstocks at Parker Dam, penstock Nos. 1 and 3 have crossed plane AVM's. Penstock No. 1 was chosen for the crossed plane installation because it is the shortest

Table 1.—Comparison of cross plan discharge measurements for Davis penstock No. 5 over a range of wicket gate openings

Wicket gate opening (percent)	AVM Flow-1 (cubic feet/second)	AVM Flow-2 (cubic feet/second)	Percent difference
11	427.1	433.8	-1.53
20	726.5	733.1	-0.89
30	1140.2	1159.3	-1.64
40	1418.1	1439.7	-1.50
50	1909.9	1928.4	-0.95
60	2319.9	2322.0	-0.09
64	2467.6	2487.5	-0.80

Note: Flow-1 and Flow-2 are internally divided by 2 so that they correct for cross flow errors when summed. The percent difference between Flow-1 and Flow-2 is an indication of cross flow error.

Table 2.—Comparison of cross plan discharge measurements for Parker penstock No. 1 over a range of wicket gate openings

Wicket gate opening (percent)	AVM Flow-1 (cubic feet/second)	AVM Flow-2 (cubic feet/second)	Percent difference
15	846.9	814.8	3.79
25	1458.8	1405.6	3.64
35	2033.5	1954.5	3.88
50	3179.0	3058.0	3.80
60	3974.0	3814.0	4.00
70	4690.0	4510.0	3.84
80	5135.0	4940.0	3.79

Note: Flow-1 and Flow-2 are internally averaged to correct for cross flow errors in Parker penstock No. 1. The percent difference between Flow-1 and Flow-2 is an indication of cross flow error.

penstock, with no diameters of straight pipe located upstream from the AVM measurement section. Penstock No. 3 was also selected for a crossed plane system to provide high accuracy during turbine performance tests.

Several AVM data sets were analyzed to identify problems with the data collection, AVM setup, cross flow, and skewed velocity distributions. A summary of the data analysis is as follows below.

Penstock No. 1

This penstock has less than 1 pipe diameter of straight pipe located upstream from the AVM measurement section. The combined bend is a vertical curve of 13° and a horizontal curve of 4.6°. Analysis of these AVM data collected for several wicket gate openings (15, 25, 35, 50, 60, 70, and 80 percent) at a reservoir elevation of 446.6 feet resulted in the following observations:

- The velocity profile is skewed with path 2 velocities greater than path 3 (figure 8a). The degree of skewness escalates with increasing gate opening. Velocity profile skewness is caused by the combined bend located directly upstream from the AVM measurement section.

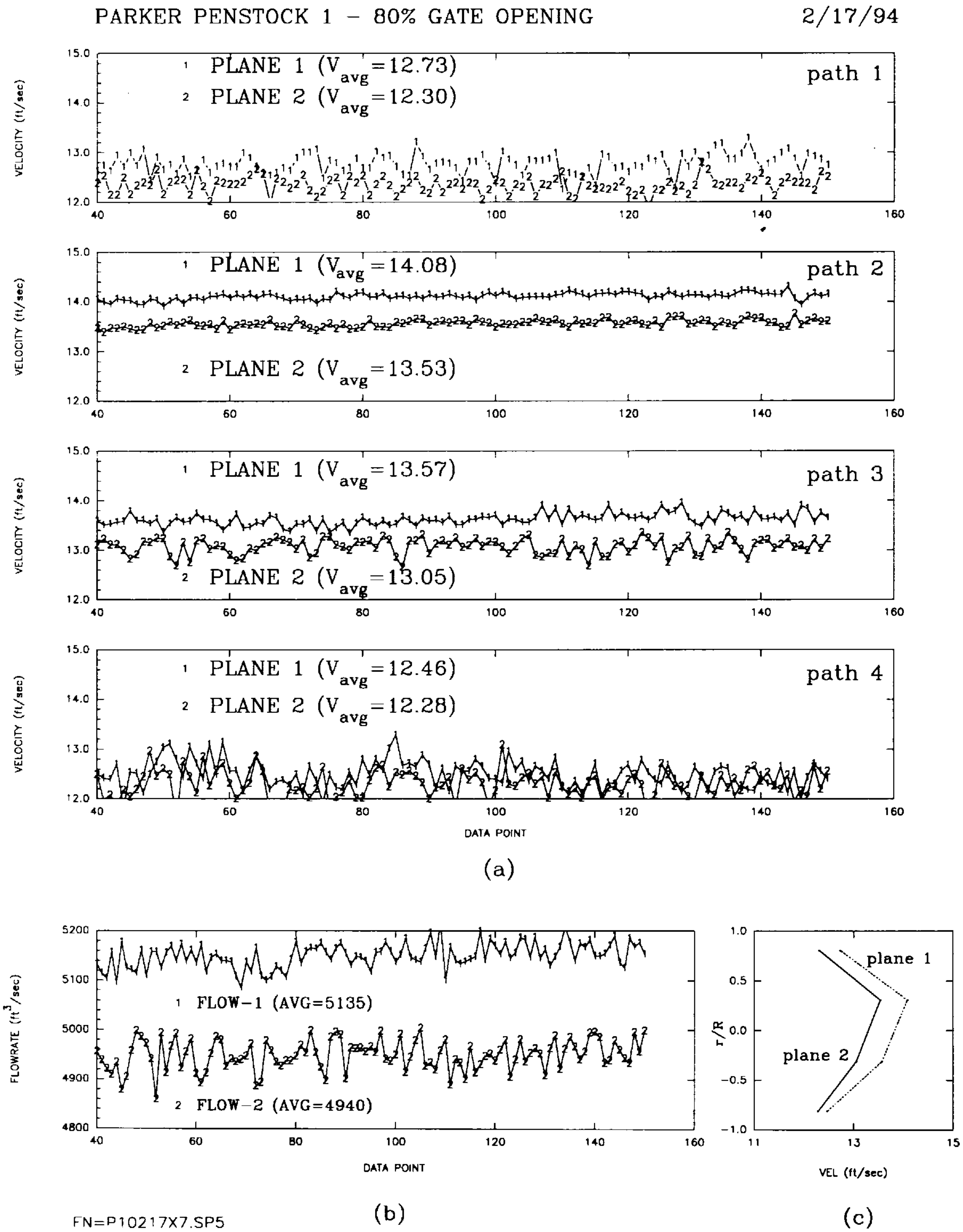

Figure 8.—(a) Instantaneous path velocities for Parker penstock No. 1. Data were collected for crossing acoustic planes. (b) Instantaneous flow rates for plane 1 and 2. (c) Average velocity profiles for planes 1 and 2.

- The crossed acoustic planes produce discharge measurements which are consistently offset by an average of 3.8 percent (see table 2 and figure 8b), which indicates the cross flow component in the AVM measurement section is significant for the range of wicket gate openings tested.

- This penstock requires cross plane measurements for accurate flow measurement. The average of Flow-1 and Flow-2 is the discharge value corrected for cross flow. If one plane of transducers should fail, a correction should be made as follows: reduce Flow-1 value by 1.9 percent if plane 2 fails, or increase the Flow-2 value 1.9 percent if plane 1 fails. This correction will give a good estimate of the true discharge.

- Field observations of a vortex near the intake for penstock No. 1 could also be contributing to the development of cross flows. Vortices were not observed in the model.

RECOMMENDATIONS FROM FIELD EVALUATIONS

Field Surveys

Some interesting equipment problems were identified during the surveys. At Hoover and Davis Dams, when acoustic transducers were removed for cleaning or when the penstock was dewatered, a large number of transducers failed. Subsequently, transducer failures have been prevented by keeping transducers submerged in water during maintenance operations. Another common concern was the accuracy of field surveys of path angles and lengths and cross-sectional areas of the penstocks. These parameters are very difficult to measure accurately and must be determined to a high degree of accuracy. Therefore, operators should be comfortable with the survey accuracy prior to going on-line with an AVM system. Survey information should be recorded for future reference because it is necessary information for setting up the AVM system parameters.

Review of the system parameter lists identified several errors in the system parameters. Errors in path angle and diameters resulted in relatively large systematic errors. Once identified, these errors are easily corrected provided as-built information is available. Another installation had two cables crossed, which resulted in a negative path velocity. Of course, this error leads to a very large error in discharge measurement.

RECOMMENDED INSTALLATION AND SET-UP PROCEDURES

Installation of an AVM requires a layout survey, installation of transducers and their mounts, and an as-built survey of acoustic path geometry and average cross-sectional area. Accurate installation and AVM setup is critical to assure accurate discharge measurements. ASME's Performance Test Code 18-1992 and ANSI/ASME Standard MFC-5M-1985 are good resources for AVM installation, operation, and maintenance guidelines. A list of field procedures which should be considered are:

- When installing the acoustic transducers, the orientation of the acoustic paths depends on the geometry of any upstream bends, valves, or bifurcations. Even though horizontal paths may be easier to install, they may not produce the highest level of accuracy.

- Acoustic signals received at each transducer should be examined, using an oscilloscope, to look for excessive noise and sufficient signal strength (amplitude) to assure proper signal detection. Signal strength should be examined periodically to check for transducer fouling.

- To check for timing bias errors in travel time measurements, the upstream and downstream cables should be reversed, and a repeat set of measurements should be taken. If the two time-averaged discharge values are different, a bias error associated with a timing offset exists. This bias may be caused by unequal cable lengths or delays in the electronic circuitry. PTC 18-1992 recommends measuring the ultrasonic pulse transit times independently and comparing the transit times measured by the AVM.

- If crossed acoustic planes are installed, comparisons of the two velocity profiles and two discharge measurements should reveal the presence of cross flow (nonaxial) velocity components (see figures 8a and 8b). Cross flow can be caused by an upstream change in flow direction or by vortices which form near the penstock intake.

- The velocity profile established by the four acoustic path velocities should be studied to determine profile distortion. Likewise, large fluctuations in instantaneous measurements can indicate nonuniform flows through the AVM measurement section.

- Travel time along each acoustic path is calculated by the flowmeter. The AVM estimates the speed of sound for each measurement, which should correspond to the theoretical speed of sound for the water temperature moving through the penstock. A difference between these two values could be caused by a temperature gradient in the water moving along the acoustic path. This problem can occur if the reservoir's thermocline develops at the same elevation as the penstock intake structure. A difference in speed of sound could also indicate a survey error in path length or path angle.

- Tests should be performed to determine if any long period fluctuations exist, which may affect the average discharge measurement during short period performance evaluations. This type of fluctuation could be generated by a vortex which periodically develops near the intake.

ACKNOWLEDGEMENTS

The study was performed in cooperation with Reclamation's Lower Colorado Regional Office. The study was managed by Albert Marquez from the regional office; he also collected the AVM field data. Jim Davis at Davis Dam, George Kraft at Parker Dam, and Chuck Wiley at Hoover Dam provided as-built information on the AVM installations. Brent Mefford, Technical Specialist, provided technical support throughout this study. Jim Nystrom, Alden Research Laboratory, performed the peer review.

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STEPPED OVERLAYS PROVEN FOR USE IN PROTECTING OVERTOPPED EMBANKMENT DAMS

by Kathleen H. Frizell[3]

INTRODUCTION

The Bureau of Reclamation (Reclamation), in conjunction with Electric Power Research Institute and Colorado State University (CSU), has completed a 4-year research study on concrete step overlay protection for embankment dams. The program included laboratory flume studies of various step shapes covering 2:1 (H:V) to 4:1 slopes and verification of the designs in a large-scale facility with an individual block shape, representing the stepped overlay. These combined test programs showed that a properly designed stepped overlay is inherently stable because of the combined effect of the impact of the flow on the step surfaces and the ability of the stepped overlay to relieve the uplift pressure.

A brief summary of the large-scale facility used for testing the tapered block shape on a 2:1 slope will be presented. Test results from the large-scale facility show that the block system developed from the laboratory data (Frizell, 1992) is stable, allowing the results to be applied to an actual embankment dam with confidence. Although laboratory testing has been completed on 4:1 slopes, some analysis still remains before results can be reported and extended to other typical embankment slopes. Basic design guidelines developed from the laboratory and large-scale testing will be presented. The design guidelines will be demonstrated in the design of an overlapping, tapered block system for a small embankment dam.

Large-Scale Test Facility and Block Description

The outdoor overtopping facility, located at CSU in Fort Collins, Colorado, was sized to be similar in height to a typical embankment dam in need of rehabilitation. The facility (figure 9) consists of a concrete headbox, chute, tailbox, and sump with a pump. The concrete chute is on a 2:1 slope and has a height of 15 meters (50 feet). The maximum width of 3 meters (10 feet) was reduced to a width of 1.5 meters (5 feet) to increase the unit discharge capacity to 3 cubic meters/second/meter (32 cubic feet/second/foot) for the tapered block testing. Water is supplied through a 0.9-meter (3-foot) pipe from Horsetooth Reservoir. A portion of the flow can be recirculated by pumping back from the tailbox to increase the total discharge through the facility.

Based on prior laboratory studies, the overlapping, tapered, concrete blocks (figure 10) were designed and constructed for the large-scale tests. The blocks are 0.4 meter (1.23 feet) long and 0.06 meter (0.21 foot) high with a maximum thickness of 0.11 meter (0.375 foot). Drains, which aspirate water from the filter layer, are formed in the overlapped portion of the block. The blocks were placed over 0.15 meter (0.5 foot) of free-draining, angular, gravel filter material. The filter material and thickness were designed according to Reclamation design guidelines. The gravel filter was placed on the

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Figure 9.—Large-scale, 15-meter- (50-foot-) high dam overtopping test facility, q = 3 cubic meters/second/meter (32 cubic feet/second/foot).

concrete floor with 102-millimeter (4-inch) angle iron (with a gap above the floor to allow free discharge underneath) placed every 1.8 meters (6 feet) up the slope to prevent sliding of the gravel. A wooden strip was installed along each wall to easily screen the gravel filter and to prevent failure along the wall contact during operation. A combination of 0.6-meter- and 0.3-meter- (2-foot- and 1-foot-) wide blocks were placed on the "embankment" shingle-fashion from the slope toe leaving no continuous seams in the flow direction.

At the crest of the structure, a small concrete cap was placed to transition from the flat approach to the first row of blocks. At the toe of the concrete slope is a fixed concrete end block to support the blocks up the slope. About every 25th row of blocks was anchored to the floor to prevent gradual migration of filter material, which could result in bowing or settling of the block overlay. Where the blocks will be under the tailwater at the toe of the slope, the blocks are pinned together longitudinally through the overlapping area parallel to the slope.

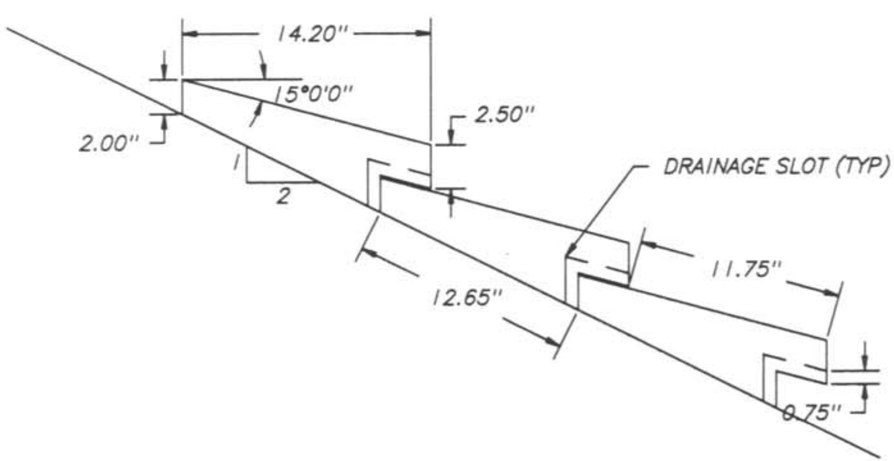

Figure 10.—Dimensions of the tapered blocks used in the large-scale facility.

Flow Description

During initial startup of the flume, under a very low discharge, the fines and dirt were flushed from the filter material. Flushing lasted a very short time and was observed by the coloring of the water. After shutting off the water, slight settling of the blocks was apparent; however, no sliding or noticeable trend to settling occurred. Throughout the testing, no further noticeable settling of the blocks occurred. The maximum settlement was about 25 millimeters (1 inch). The blocks were exposed to two winters of freezing conditions with no measurable movement or damage.

The many discharges tested in the flume produced varied flow conditions over the blocks. Very small flows were almost entirely broken up by the block shape, leaving no noticeable thickness of solid water and a tumbling, highly-aerated, flow condition. As the discharge increased, skimming flow occurred. For the largest flow rate, the flow did not become fully aerated until traveling about half the distance down the slope. Uniform flow was attained for all flow rates tested.

Design Criteria

Tests of the block system were completed in the large-scale, outdoor facility in the fall of 1993. The block system remained stable and performed excellently, even after some blocks were intentionally cracked and partially removed.

Model/prototype comparisons between the laboratory and the outdoor facility are complete. These results were used to develop general design guidelines that provide the necessary information to design a stepped overlay for embankment dams with downstream slopes of approximately 2:1.

The design guidelines include:

- The discharge coefficient for a typical embankment dam crest shape to be used for flood routings
- The block shape and vent port area required for use in determining block dimensions for various embankment slopes of about 2:1
- The total force per foot of width of stepped overlay or the expected stability based upon the hydraulic forces. This value will allow the designer to vary the block weight to provide a stable overlay for a given dam height, unit discharge, and predicted seepage
- The velocity at the dam toe for any step height and total dam height to be used in designing toe protection or energy dissipators
- The friction factor attributable to step roughness down the slope to be used with the bulked flow depth for designing wall heights

Data for all step shapes (15°, 10°, and horizontal) and embankment slopes (2:1 and 4:1) tested are currently being reanalyzed to include the results from the large-scale facility. These results will allow development of entirely generalized design criteria for embankment slopes from 2:1 to 4:1 and extrapolation up to larger unit discharges.

Discharge Coefficient

The discharge coefficient for an overtopping embankment dam is a function of the upstream slope of the dam, the top width, and the abutment geometry (for short crest lengths), and varies with the overtopping head. An average coefficient of about 2.9 may be used for most flood routing applications to determine the depth of overtopping that will pass the desired probable maximum flood (PMF).

Block Shape

The most stable block shape on a 2:1 slope is the 15° tapered or sloping block (Frizell, 1992). Included in this recommendation is that the percent of the vertical block face area occupied by the vents be 2.8 percent (Baker, 1991). This block shape was tested in the large-scale facility for unit discharges up to 3 cubic meters/second/meter (32 cubic feet/second/foot). (Greater top slopes may produce instabilities by providing an overly large low pressure zone or an undersized solid vertical block surface.) Any block design is based upon keeping the difference between the top slope and the embankment slope constant for a given embankment dam slope. Therefore, when designing a block with a top slope that provides effective aspiration, the difference between slopes is 11.56° (embankment slope = 26.56° (2:1) minus top slope = 15°). Obviously, this criterion is only applicable to embankment slopes greater than this difference.

In addition, the ratio of the step height to the step tread length exposed to the flow should remain between four and six. If the step height is chosen to match that of our testing, 64 millimeters (2.5 inches), then the tread length should optimally be chosen to match as well. This selection would produce slightly different horizontal tread lengths for dams of different slopes based upon the chosen top slope of the block. This horizontal tread length is then used to determine the length of the block surface along the embankment slope.

The block thickness is determined from the stability analysis. A minimal thickness of 51 millimeters (2 inches) at the upstream end of the block is required to maintain the integrity of the concrete and allow proper forming of the block.

Stability

The question of stability of the protective system is the most critical for an embankment dam. Any failure or instability in the system could cause a catastrophic failure of the entire dam during an overtopping event. Laboratory data show that the ability of the blocks to relieve the uplift pressure, combined with the impact of the water on the block surface, make the blocks inherently stable. The 15° sloping block was used for the large-scale tests (see figure 10). The 0.30-meter- (1-foot-) wide block used in the large-scale flume had a dry weight of 17.4 kilograms (38.4 pounds).

Pressure data were gathered to compute the magnitude of the forces acting on the block surfaces and in the underlying filter. For discharges producing skimming flow, impact pressures increased to a maximum about 44 steps down the slope then decreased because of aeration effects. The filter pressures were assumed to vary linearly between the measurement locations. The filter pressures showed a gradual increase over about the top 40 steps, indicating a buildup of flow in the filter near the top of the slope. At about 45 steps down the slope, the filter pressures quickly decreased as aspiration increased to an average of about 0.03 meter (0.1 foot) at the toe of the slope for all flow rates.

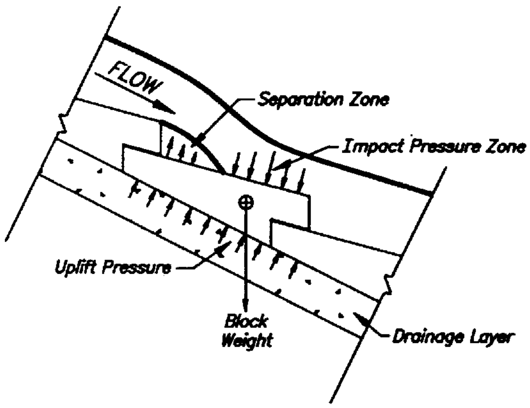

Figure 11.—Forces acting on the tapered concrete block.

The stability of the block system has been analyzed as a function of the total forces acting on individual blocks down the slope. Block weight and pressure yield a net downward or positive force normal to the slope. The uplift pressure in the filter material underneath the block and the low pressure zone created by the block offset act in an upward (negative) direction, tending to lift the blocks from the embankment surface (figure 11). Aspiration ports in the vertical face of the block limit the uplift forces by venting the filter layer to the low pressure separation zone. The gradation of the filter must be designed to prevent the filter material from being transported through the aspiration ports. In the analysis, a net positive force indicates a stable block.

The block geometry optimizes the hydraulic forces to produce downward impact pressure and aspiration of subgrade pressures. The stability of the block system is quantified in the graph showing the net force per foot of block width versus the vertical distance below the crest (figure 12). This graph shows the sum of the forces normal to the slope, including integration of the pressure profile on the step tread and the measured filter layer pressure (uplift). The sum of the forces is positive for this block shape and filter, indicating a stable overlay for a 2:1 slope for the range of critical depth (D_C) to step height (H_S) ratios tested. The submerged block weight of about 6 kilograms (13 pounds) has been added to the forces in the curve formed by the dashed line for $D_C/H_S = 10.36$ to show the additional stability added by a block of minimal thickness. Not included in the analysis is the additional stability provided by the block overlap, which creates an interlocking affect. These results indicate no decrease in block system stability with unit discharge.

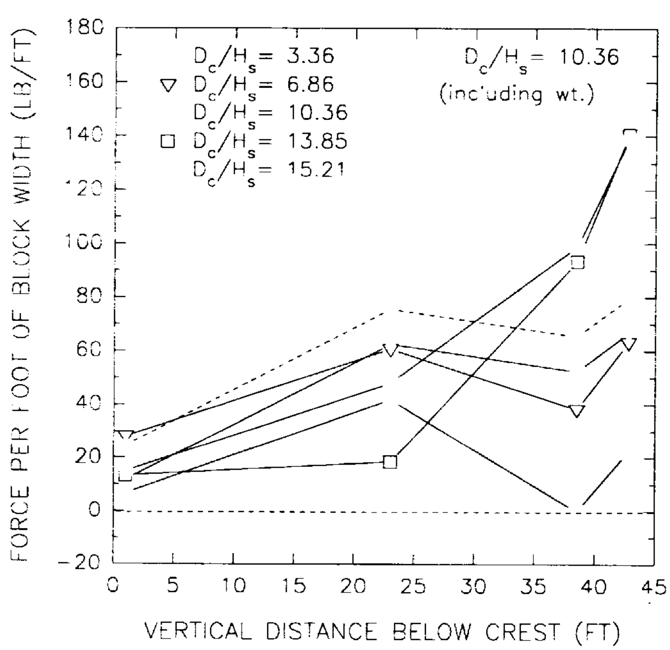

Figure 12.—Net force acting on the block versus distance below the crest.

The Construction Industry Research and Information Association has developed criteria for determining the optimum percent of port area to vertical surface area of the step face. Port sizes for providing aspiration of filter pressures should be 2.8 percent of the surface area of the step. Proper sizing of the port area will limit the uplift pressure developed in the filter layer. The length of blocks used across the width of the dam will also influence the amount of flow entering the filter. Using longer blocks across the dam width will reduce the jointing, thus the infiltration of flow to the filter layer. If excessive seepage is expected, then the block weight could easily be increased accordingly.

Energy Dissipation or Toe Velocity

Of secondary benefit is the amount of energy dissipated by flow over the steps formed by the block surface. In general, a stepped surface reduces the energy of the flow at the dam toe compared to a smooth surface. Figure 13 allows the designer to vary the step height (within reason), for a dam of a given height and known unit discharge, to directly determine the desired velocity at the toe of the dam as a function of the energy remaining. The graph has been developed for unit discharges ranging from 0.3 to 3 cubic meters/second/meter (3.3 to 32 cubic feet/second/foot) using both laboratory and large-scale results. The laboratory data were adjusted for the effects of aeration by using the average air concentrations measured in the large-scale facility. The air content reduces the friction, and, therefore, increases the velocity over that predicted by the laboratory data alone.

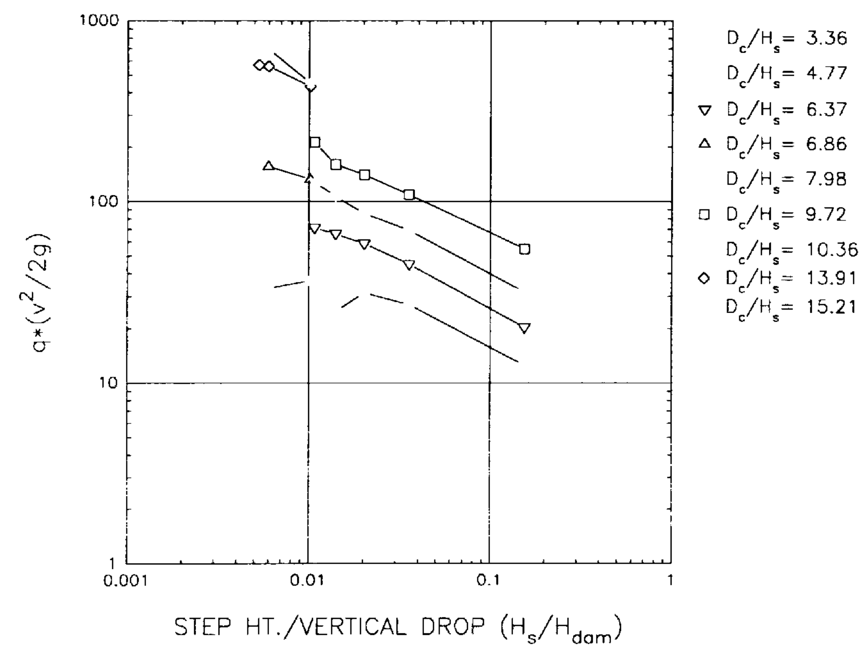

Figure 13.—Graph used to determine the velocity remaining in the flow after traveling down a stepped spillway with 15° tapered over-lapping blocks on a 2:1 slope.

From the graph, the larger the step height for a given dam height, the less the energy remaining in the flow at the toe of the dam. Conversely, as the ratio of the step height to dam height decreases, the energy in the flow increases. The energy remaining in the flow is also a function of the critical flow depth to step height ratio. This graph includes a range of critical depths to step height ratios of 3.36 to 15.21. A designer should attempt to remain within this range if possible. At some point for all flow rates, uniform flow is attained and the energy per foot of width remains constant. This equilibrium is shown by the indication of the leveling off of the curves as the step height to dam height ratio decreases. When uniform flow is reached, then the velocity and depth will remain constant regardless of the dam height. (We are currently working to provide extrapolation of these curves to higher flow rates and to dams of other slopes.)

If the tailwater elevation and velocities indicate that a hydraulic jump will occur over the blocks, then the blocks should be pinned to restrict rotation caused by the dynamic pressure fluctuations of the jump (see figure 16). Loosely pinned blocks were successfully tested under a hydraulic jump at the CSU facility.

Roughness and Bulked Depth

Darcy-Weisbach friction factors are computed based upon velocity profiles corrected for air concentration. The friction factor, f, varied down the slope as the flow developed, eventually becoming constant at 0.11 (Manning's $n = 0.03$) for uniform flow. Using this value in a standard step method calculation will determine the flow depths down the chute. An average air concentration of 34 percent is reached for the fully developed flow condition; therefore, the wall heights should be raised by 34 percent above the calculated flow depths to contain the flow. An additional safety factor may be added if deemed necessary.

Design Example

This design example will demonstrate the use of the design information by presenting a feasibility level design of a block system on a small embankment dam. The design example will include the crest treatment, block dimensions and stability on the slope, and required toe treatment based upon energy remaining at the toe of the dam.

A small embankment dam owned by the Department of the Interior is scheduled for rehabilitation because of its inability to pass the PMF. The embankment dam is about 8 meters (27 feet) high with a 2.5:1 downstream slope. An outlet works tunnel is located through the dam on the right side and an emergency, grass-lined spillway is located on the left abutment. The lake formed by the dam is a highly used recreational site with hiking trails crossing the dam and following the left abutment. The top of the existing dam is at elevation 2498. The 12-meter- (40-foot-) wide grass-lined spillway with crest elevation 2495.6 is currently designed to pass less than 14 cubic meters/second (500 cubic feet/second) before the dam is overtopped. Overtopping is predicted to breach the dam, resulting in a discharge of 651 cubic meters/second (23,000 cubic feet/second) for 20 minutes, which is unacceptable. To prevent overtopping of the dam, another spillway constructed with the tapered, overlapping blocks has been designed. The design must still allow pedestrian passage across the top of the dam and must have minimal impact to the existing aesthetics. As a result, the block system will be covered with topsoil.

The reservoir water surface elevation is restricted to elevation 2502 by reservoir rim development. The top of the dam will be raised by 1.5 meters (5 feet) with a rock and masonry wall tied to the existing impervious core wall to prevent overtopping of the existing dam during the PMF. Pedestrian access will be upstream from the dam raise, over the top of the covered blocks. After using several spillway crest elevations and widths to route the PMF with maximum water surface of elevation 2502, the configuration shown on figure 14 was determined. (The design includes use of the existing grass-lined spillway. This option may not be the case in the final design.)

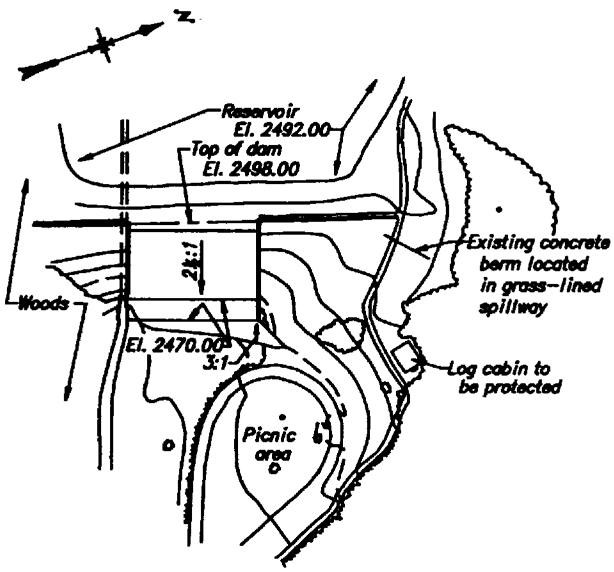

Figure 14.—General plan of the block spillway layout on the 8-meter- (27-foot-) high embankment dam with 2.5:1 downstream slope.

Flood Routing and Crest Detail

The block spillway will have a maximum spillway discharge of 118 cubic meters/second (4,183 cubic feet/second) or 3 cubic meters/second/meter (32.2 cubic feet/second/foot). Using a crest coefficient of 2.9 in the flood routing determined that a 40-meter- (130-foot-) long block spillway crest at elevation 2497, 0.3 meter (1 foot) below the top of the existing dam, is needed. The grass-lined spillway will pass a maximum of 680 cubic meters/second (2,230 cubic feet/second) and will begin discharging when the reservoir is 0.5 meter (1.5 feet) below the block spillway crest. The top of the dam upstream from the block spillway crest will also be covered with blocks to prevent undermining the slope protection by flows over the top of the dam. To prevent seepage through the dam, a cutoff trench will be placed along the axis of the dam that extends from the top of the existing core wall to the surface where the spillway blocks will be placed.

Block Design

The tapered blocks will be designed for effective aspiration or stability on the 2.5:1 (21.8°) downstream slope of the dam.

Top slope of block: 21.8°-11.56° = 10.24°

This top slope on the block produces the following geometry for the remainder of the block surfaces on the 2.5:1 slope:

Tread length exposed to the flow	297 millimeters (11.71 inches) (same as 15° step on 2:1 slope)
Horizontal length	318 millimeters (12.52 inches)
Block length along the embankment slope	293 millimeters (11.52)/cos(21.8°) = 315 millimeters (12.41 inches)

Choose the step height to be between the ratio of tread length to step height of 4 to 6.

Step height, H_s = 64 millimeters (2.5 inches)

Stability

The block thickness is then determined by finding the block weight required for stability based upon the predicted uplift or seepage pressure. Inspection of the dam indicates that excessive seepage will not be a concern. As a result, a free-draining gravel filter underneath the block system will be adequate, and stability may be predicted from figure 12. A dam height, H_{dam}, of 8 meters (27 feet) and $D_c/H_s = 15.21$ gives a total positive force of about 89 kilograms/meter (60 pounds/foot) acting to hold the block on the surface. Therefore, a minimal block thickness at the upstream end of the block of 51 millimeters (2 inches) and appropriate aspiration port area will produce a stable block (figure 15).

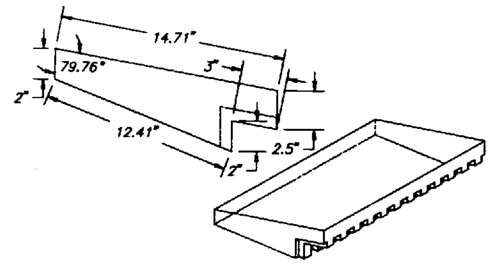

Figure 15.—Dimensions of the blocks designed for overtopping protection on a 2.5:1 slope.

Figure 16 shows the toe block required for stability at the toe of the slope. The toe block is used as a base for the first row of overlapping blocks and continues completely across the spillway width at the toe of the dam. Blocks located beneath the tailwater should be cast with two holes per block to receive loose-fitting pins. This detail (figure 16) will produce stability of the rows of block exposed to the fluctuating pressures of the jump.

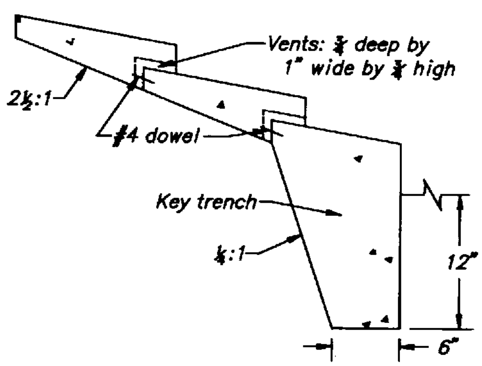

Figure 16.—Toe block and example of block pattern on 2.5:1 slope.

Toe Velocity

The velocity at the toe of the dam may now be determined for the 64-millimeter- (2.5-inch-) high block step height.

$H_s/H_{dam} = 2.5/12/27 = 0.0077$
$D_c = (32.2^2/32.2)^{1/3} = 0.97$ meter (3.18 feet)
$D_c/H_s = 3.18/0.2083 = 15.26$

Using figure 13, $q * V^2/2g = 580$. Solving for the velocity gives 10.4 meters/second (34 feet/second) at the toe of the dam (assuming a 2:1 slope). Laboratory data have shown slightly less velocity for flatter downstream slopes; therefore, the toe velocity will be increased by 5 percent to 10.9 meters/second (35.7 feet/second).

Energy Dissipator

Using a velocity of 10.9 meters/second (35.7 feet/second) and Reclamation's Monograph No. 25, the stilling basin length is determined. The depth of flow at the toe of the dam is determined using the continuity equation.

Depth at toe, *d*: 32.2/35.7 = 0.3 meter (0.9 foot)
Froude number, $V/(gd)^{1/2}$: 6.63

Using these values in Reclamation's Monograph No. 25, figure 16 produces a required stilling basin length of 15 meters (50 feet) (Peterka, 1978). This result is shown on figure 14 and will most likely be formed with grouted riprap.

Spillway Wall Height

The wall height at the crest must be equal to the overtopping depth, or in this case, simply the reservoir elevation, giving a wall height of 1.5 meters (5 feet). The flow depth down the chute is determined by using the friction factor and velocities down the slope in a standard step calculation. This calculation produces a required wall height that varies down the slope to a minimum of 0.3 meter (1 foot) at the toe. (As expected, this value compares well with the depth computed using continuity.) Adding the bulking caused by 34-percent air concentration gives the necessary freeboard to contain the highly aerated flow over the spillway blocks and a wall height of 0.41 meter (1.34 feet) at the dam toe, measured from the tip of the steps. This wall could easily be obtained by anchoring highway jersey barriers in the embankment and butting the blocks up to them.

Cost

The dam owners have labor available to work on the dam rehabilitation. This availability makes the block design an attractive alternative. A general cost estimate obtained for manufacturing and installation of the block system on a typical embankment dam including crest and toe treatments, preparation of the embankment slope, and filter placement with precast spillway walls, varied between $131 to $233 per square meter ($110 to $195 per square yard). The predicted cost for initial production of the forms for the blocks varied greatly and was probably greatly overestimated. Reclamation and CSU have both formed blocks with great success and minimal cost. For construction of the block overlay only (not the dam raise, etc.) and assuming a coverage of 1,256 square meters (1,050 square yards), total cost would be $115,500 to $204,750.

Conclusion

The tapered block system has been tested well beyond the limits of other concrete revetment systems. The design criteria presented define their application for a wide range of overtopping. The block system is particularly applicable for dams in remote or environmentally sensitive locations where use of a batch plant or large machinery is limited. The cost of the system is not well known at this time but will be competitive once the forms have been constructed and the ease of placement is discovered.

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Baker, R., August 1991, "Performance of Wedge-Shaped Blocks in High Velocity Flow," CIRIA Research Project 407, Stage 2, University of Salford, England.

Peterka, A.J., 1978, "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, Bureau of Reclamation, Denver, Colorado.

CANAL SEALANTS FOR USE ON "GREEN" CONCRETE

by Jay Swihart[4]

BACKGROUND

Flexible sealants are used to seal joints in concrete-lined canals to minimize water loss through seepage (see photo below). Sealant manufacturers typically require that the concrete be fully cured (28 days minimum) prior to application of the joint sealant. Otherwise, moisture vapor escaping from the "green" concrete can interfere with the bond between the concrete and the sealant. For concrete less than 28 days old, sealant manufacturers may recommend either a 4-percent maximum concrete moisture content, as determined with a Delmhorst moisture meter, or the absence of any moisture vapor drive as indicated by a "mat test" per ASTM D 4263 "Moisture in Concrete by the Plastic Sheet Method." The mat test consists of taping an 18-inch square of clear plastic to the concrete. The plastic is left in place overnight and then checked the next day for the presence of moisture beneath the mat.

These recommendations are generally quite conservative, and Bureau of Reclamation (Reclamation) specifications require only a 7-day concrete cure prior to placement of the joint sealant. This practice has worked quite well, especially in dry, warm climates (such as Arizona) where the concrete cures

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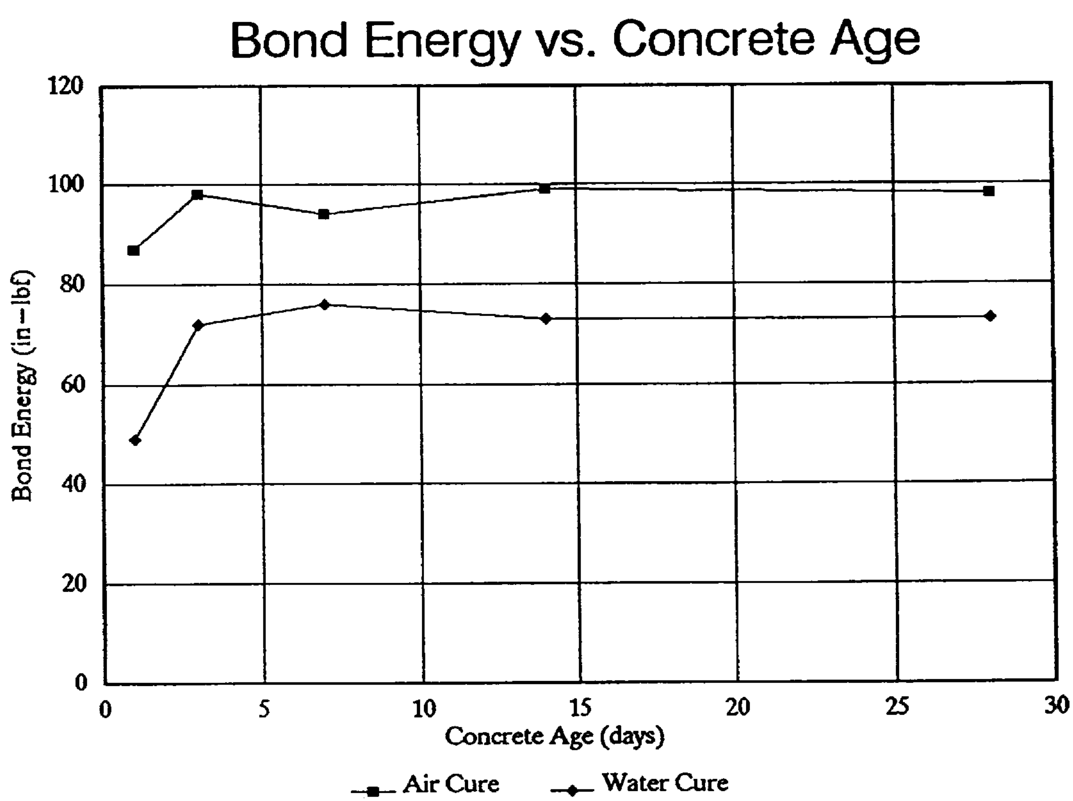
quickly and the moisture vapor drive is minimal after 7 days. However, short downtimes for construction, maintenance, and rehabilitation work do not always allow for even a 7-day concrete cure. In these situations, Reclamation has specified special two-part polysulfide sealants from Koch Materials that can be applied to "green" concrete (1 to 3 days old). Unfortunately, Koch is no longer in business, which has caused delays on recent projects such as Towaoc Powerplant Forebay and Columbia Cold Springs Canal. A suitable replacement for the Koch sealants is needed.

LABORATORY TESTS

Using Water Technology and Environmental Research funds, the Denver Technical Service Center performed laboratory testing on seven promising canal sealants. The sealants were applied to "green" concrete ranging in age from 1 to 28 days. To simulate field conditions, the sealant specimens were then cured for 35 days in two different environments. The first set of specimens was cured for 35 days in air at 70 °F and 50-percent relative humidity. The second set was cured for 7 days in air, followed by a 28-day water-immersion cure. The water cure more closely simulates field conditions where the concrete structure and sealant are returned to service as soon as possible. After curing, triplicate specimens were tested for modulus, ultimate strength, elongation, and bond energy (area under the stress-strain curve). Of the seven sealants tested, three sealants demonstrated good bond to 1-day-old concrete and full bond to 3- to 7-day-old concrete. Typical results are shown in the graph below.



Primers

Three of the sealants were tested both with and without primer. The specific primers were recommended and provided by the sealant manufacturers. Theoretically, primers can improve the bond of sealants to "green" or wet concrete, especially epoxy primers, which are quite impermeable and insensitive to moisture. However, in these and other tests, the primers proved to be of little or no value. One of the sealants performed equally well both with and without primer; the two other sealants actually performed better without their epoxy-based primer. Finally, the time window for applying the sealants over the primers is critical and quite short (typically 1 to 8 hours), which complicates the construction process and has caused problems in the past. Therefore, use of primers is not recommended.

CONCLUSIONS

1. Because field cure conditions vary considerably, the most reliable and durable results will be obtained by waiting to apply joint sealant until the concrete has cured for 28 days. This concrete cure time will avoid any sealant bond problems caused by moisture vapor drive out of the concrete. The sealant should meet the requirements of Reclamation Standard M-41 "Standard Specification for Elastomeric Canal Joint Sealant," dated August 1, 1983.

2. If time constraints will not allow for the full 28-day concrete cure, Reclamation routinely achieves excellent sealant bond to 7-day-old concrete.

3. For sealant application to "green" concrete less than 7 days old, the following sealants have demonstrated good bond to 1- to 3-day-old "green" concrete in the laboratory. The concrete must be cured sufficiently to allow necessary foot and equipment traffic without damage to the concrete, and the joints must be clean and dry with no standing water.

Sealant	Manufacturer	Phone number
Sonolastic 2-part polysulfide	Sonneborn—Chem Rex	1-800-433-9517
Sikaflex 2c	Sika Corporation	1-800-933-7452
Sikaflex 1a	Sika Corporation	1-800-933-7452

4. This study and list of sealants for use on "green" concrete is by no means exhaustive. Interested parties are invited to submit additional sealants for consideration.

5. Proper surface preparation of the concrete (prior to application of the sealant) is critical. Joints must be clean, dry, and free of laitance, scale, dust, dirt, oil, and other foreign materials. Sandblasting is the preferred surface preparation technique and is mandatory to remove curing compound. Other surface preparation techniques which may be acceptable include saw-cutting, grinding, and wire-brushing. After cleaning, the joints should be blown out with compressed air to remove all residue.

6. The sealant should be allowed to cure for 7 days prior to water immersion. Shorter sealant cure times may be considered (especially for two-part sealants) if recommended by the sealant manufacturer.

7. Primers did not improve the performance of any of the sealants tested in this study or in previous studies. Therefore, use of primers is not recommended.

8. If further information is needed, the author can be contacted at (303) 236-3730, extension 436.

ROLLER COMPACTED CONCRETE OVERTOPPING PROTECTION IN THE USA[5]

by Kenneth D. Hansen, P.E., Portland Cement Association

Many older embankment dams in the U.S.A. have been determined to be hydraulically deficient. They are unable to either store or safely pass floods of at least one-half the probable maximum flood (PMF). The overtopping of unprotected embankments may result in serious erosion and ultimate failure depending on the height of overtopping during flow, together with the slope and properties of the embankment's downstream surface.

Since the development and initial acceptance of the concept of safe overtopping in the mid-1980s, roller-compacted concrete (RCC) overlays have become the most widely accepted overtopping protection method used to increase spillway capacity of older earth dams in the U.S.A. By the end of 1994, RCC had been used to increase the flood handling capacity of older earth dams in the U.S.A. By the end of 1994, RCC had been used to increase the flood handling capacity of 41 dams in 18 States in the U.S.A. The widespread acceptance of the RCC overtopping protection concept is further evidenced by the fact that 28 different consulting engineering firms or government agencies designed the 41 projects. Simply defined, roller-compacted concrete is a no-slump concrete that is consolidated or compacted by a roller, usually a vibratory roller. A true concrete is thus produced by methods usually associated with earth dam construction.

The wide and relatively rapid acceptance of RCC for overtopping protection can be attributed to its low cost, relatively simple design concept and construction method, as well as proven performance. It is generally the lowest cost alternative for increasing spillway capacity for older existing embankment dams. An RCC overlay on the downstream slope of an embankment can usually be accomplished without lowering the reservoir.

The basic design concept for RCC overtopping protection is to provide an overlay on the downstream slope which has sufficient weight and durability to resist displacement and erosion during infrequent flood flows. Most designs for RCC overlays have been intuitive. They have generally been based on placing 300-millimeter compacted thickness horizontal RCC lifts in a stair-stepped fashion adjacent to the existing downstream slope. A minimum 2.4-meter lane width is required for proper hauling, spreading, and compacting the no-slump RCC mixture.

Thus, a 3H:1V embankment slope results in a minimum thickness of concrete measured perpendicular to the slope of about 600 millimeters. For more detailed information on design see McLean and Hansen (1993).

A review of structural and hydraulic information on the 41 RCC overtopping protection projects in the U.S.A. indicates the following with respect to dam height, volume and cost of RCC, maximum depth, and duration of overflow.

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[5] Reprinted from *Canadian Dam Safety Association Newsletter*, April 1995, pp. 7-9.

Dam Height: Of the 41 projects, 32 are 15 meters or less in height. Of the remaining projects, 8 are in the 15.5-to 30-meter height range. The highest dam to receive RCC overtopping protection to date is 33.5 meters high.

Volume and Cost of RCC: Most projects have used between 2300 and 5000 cubic meters of material. In-place costs for medium volume projects have ranged from $65 to $100 U.S. per cubic meter.

Depth of Overflow: Design depths of overflow have ranged from less than 0.6 meter to more than 6 meters, usually to accommodate a 100-percent PMF event. Eleven projects were designed to accommodate a maximum depth of overflow of 3 meters or greater.

Duration of Overflow: For half of the first 30 projects built, the calculated maximum duration of overtopping was determined to be 12 hours or less. The overtopping time ranged from 4 hours to 11 days.

Most of the dams that have received RCC overlays are small dams designed to accommodate a maximum height of overtopping of 2 meters or less for 12 hours or less.

All of the completed RCC overtopping protection projects have been exposed to the weather, but only a few of them have experienced overtopping flows. The reason that only a few of the projects have been hydraulically tested is that they were designed to accommodate infrequent flood events, generally exceeding the 1-in-100 year event.

An inspection of Spring Creek and Goose Pasture Dams, located in the mountains of Colorado, revealed that increased durability could be obtained by a combination of increased cement content and compaction of the outer edge of each lift. After exposure to 6 winters, Spring Creek Dam, at an elevation of 2900 meters, showed some freeze-thaw deterioration at the edges of the RCC. The poor durability was attributable to a low cement content of only 133 kilograms per cubic meter (the lowest of all the projects in the U.S.A.) and the lack of compaction, which resulted in low density and low durability at the outer edge of the step. The 1-year-old Goose Pasture Dam showed improved durability with a cement content of 193 kilograms per cubic meter and compaction of the outer edge of each 300-millimeter-thick lift at a 0.6H:1V slope by a hand-held "wacker."

Most of the RCC mixtures have been designed to rely on high density and high strength to provide adequate durability in cold climates. However, on three projects (Rosebud, Ponca, and He Dog) in South Dakota, air-entrainment was incorporated in a wetter RCC mixture to improve freeze-thaw resistance.

Some recent overtopping protection projects have incorporated formed steps of RCC on the downstream slope. The steps are relatively easy to construct because of the horizontal nature of the RCC construction process. The steps are intended to take energy out of the overtopping flow step by step and thus minimize the need for energy dissipation at the downstream toe of the embankment. Formed 600-meter steps of RCC were used for Ashton Dam in Idaho as shown in figure 17. The RCC for this hydroelectric dam was designed to accommodate a maximum overtopping of 3.6 meters and also served as a zone to improve the structural stability of the steep 1.5H:1.0V downstream embankment slope.

Figure 17.—Ashton Dam in Idaho.

Three of the RCC overlays have experienced significant overtopping flows. They include Ocoee Dam No. 2 in Tennessee, the North Fork of the Toutle River debris retaining dam in Washington, and Brownwood Country Club in Texas. Ocoee, a 9.1-meter-high RCC buttress timber crib dam completed in 1980, is intentionally overtopped at least 82 times a year to accommodate river rafting.

For the second project, the steel reinforced RCC spillway was subjected to continuous flows containing abrasive materials for 11 months. The unprotected embankment on either side of the RCC section failed because of overtopping after another eruption of the Mt. St. Helen's volcano caused flows to exceed the capacity of the 2.4-meter-deep by 91-meter-wide RCC spillway.

The RCC for the 5.8-meter-high embankment in Texas has experienced at least 6 overtoppings up to 300 millimeters in depth because of floods since its completion in 1982. The uncompacted RCC edges for the Brownwood Country Club Dam are shown in figure 18 following the flows over the RCC protected embankment. Therefore, experience to date has demonstrated satisfactory hydraulic performance and durability of RCC overlays during overtopping flows.

Figure 18.—Brownwood Country Club Dam.

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