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June 1990

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16. ABSTRACT

Seven papers prepared by authors from the Hydraulics Branch, Research and Laboratory Services Division, Bureau of Reclamation, are presented in this report. These papers cover some of the studies conducted by Hydraulics Branch personnel from 1988 to 1990.

The topics covered in these papers include:

- Aeration of submerged jet-flow gates as determined during a model study of the Seminoe Dam, Wyoming, jet-flow gate.
- A comparison of flush- versus chamber-mounted pressure transducers and the differences in time and frequency domain responses.
- A description of emergency closure guard gate tests performed at Tieton Dam near Yakima, Washington, and the method used to analyze downpull forces on the gate.
- A summary of hydraulic model tests conducted to develop safe boating conditions at Union Avenue Dam on the South Platte River in Denver, Colorado.
- A summary of model studies conducted at Colorado State University to investigate the influence of uniformity on riprap stability.
- A description of field tests and analysis of momentum and kinetic energy coefficients as they affect calculation of flow profiles through ramp flumes.
- A description of investigations being conducted to modify an existing embankment dam (A. R. Bowman Dam in Oregon) to allow safe overtopping during large flood events.

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Hydraulics Branch
Research and Laboratory Services Division
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Denver, Colorado

June 1990

UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

Mission: As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural and cultural resources. This includes fostering wise 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 promotes the goals of the Take Pride in America campaign by encouraging stewardship and citizen responsibility for the public lands and promoting citizen participation in their care. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

PREFACE

Authors from the Hydraulics Branch, Research and Laboratory Services Division, prepared seven papers for the American Society of Civil Engineers 1990 National Conference on Hydraulic Engineering, in San Diego, California, July 30 to August 3, 1990.

These papers are reprinted in this report; the titles are as follows:

- "Aeration of Submerged Jet Flow Gate," by K. Warren Frizell.
- "Comparison of Flush and Chamber Mounted Dynamic Pressure Transducer," by R. J. Wittler and K. W. Frizell.
- "Guard Gate Tests at Tieton Dam," by C. A. Pugh, R. J. Wittler, and J. R. Fitzwater.
- "Hydraulic Modeling of Boating Hazards and Sedimentation - Union Avenue Dam, Denver, Colorado," by C. A. Pugh and C. C. Klumpp.
- "The Influence of Uniformity on Riprap Stability," by R. J. Wittler and S. R. Abt.
- "Momentum and Kinetic Energy Coefficient Research - Ramp Flumes," by C. C. Klumpp.
- "Overtopping Protection for Embankment Dams," by K. H. Frizell, P. J. Hensley, D. L. Hinchliff, and C. Henning.

CONTENTS

AERATION OF A SUBMERGED JET-FLOW GATE

K. Warren Frizell[1], A.M. ASCE

Abstract

Submerged operation of a jet-flow gate requires special treatment of the area downstream from the gate to avoid cavitation damage. The Bureau of Reclamation studied operation of a submerged jet-flow gate in a 1:21 scale hydraulic model of the Seminoe Dam outlet works. A 60-inch (1524-mm) jet-flow gate discharges into an 84-inch (2134-mm) diameter discharge tube (1.4D expansion). Due to the inability to provide a larger diameter expansion, aeration of the discharge tube by an air vent downstream from the gate was studied. Four different locations for the air vent were tested. Discharge characteristics, pressures in the discharge tube, and air demands were measured for each of the locations. A design curve relating air demand, gate opening, and a submergence parameter was developed.

Introduction

The jet-flow gate was originally developed by the Bureau of Reclamation in 1945 for use in the intermediate and upper tiers of outlets in Shasta Dam (Lowe, 1946). Since then, jet-flow gates have been installed at numerous sites as regulating gates. Their wide acceptance is largely due to low cost, low maintenance, and a high discharge coefficient. Traditionally, the jet-flow gate has been used for discharging into atmospheric conditions. The use of this gate in submerged conditions exposes the area downstream of the gate to shear layer cavitation. Shear layer cavitation has been shown to have a high damage potential, especially at small gate openings (Oba et.al., 1985). Normally, an expanded conduit section downstream from the gate is used to ensure that the cavitation cloud generated in the shear layer does not collapse on the discharge tube walls. The size of the recommended expansion is 3D, where D is the diameter of the jet-flow gate (Isbester, 1975). This size expansion allows good circulation around the jet, protecting the downstream surfaces from cavitation damage (Isbester, 1974; and Burgi and Fujimoto, 1973). The diameter of the expansion can be reduced somewhat if the length of the discharge tube is kept very short (Mefford, 1987). Alternative measures, such as a stainless steel lining or aeration, can also offer levels of protection to the discharge tube.

In the past, a major design feature of jet-flow gate installations has been a vent system to aerate the jet at partial gate openings. When discharging

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into atmospheric conditions, this allows the jet to spring free from the control surfaces of the gate. However, under submerged conditions, the back pressure is too large to allow complete aeration of the jet. In the early 1950's, it was shown that when air was entrained into the flow in amounts of 5-10 percent (air to water volume ratio), cavitation damage on surfaces adjacent to the flow was greatly reduced (Peterka, 1953). Since then, aeration has been used successfully many times to mitigate cavitation damage in hydraulic structures (Falvey, 1990) and equipment (Arndt, 1981).

The Study

The studies were made using a 1:21 scale hydraulic model (Frizell, 1990). The model included one jet-flow gate with existing piping upstream and downstream of the gate (fig. 1). Model scaling was based on equal Froude numbers in the model and prototype. Tests were conducted to compare four different air vent locations. Measurements of upstream head, discharge, gate opening, air velocity and flow into the vent piping, and piezometric pressures at four locations on the invert of the discharge tube were taken. All tests were run with a submergence of 2.5 ft (0.76 m) above the gate centerline.

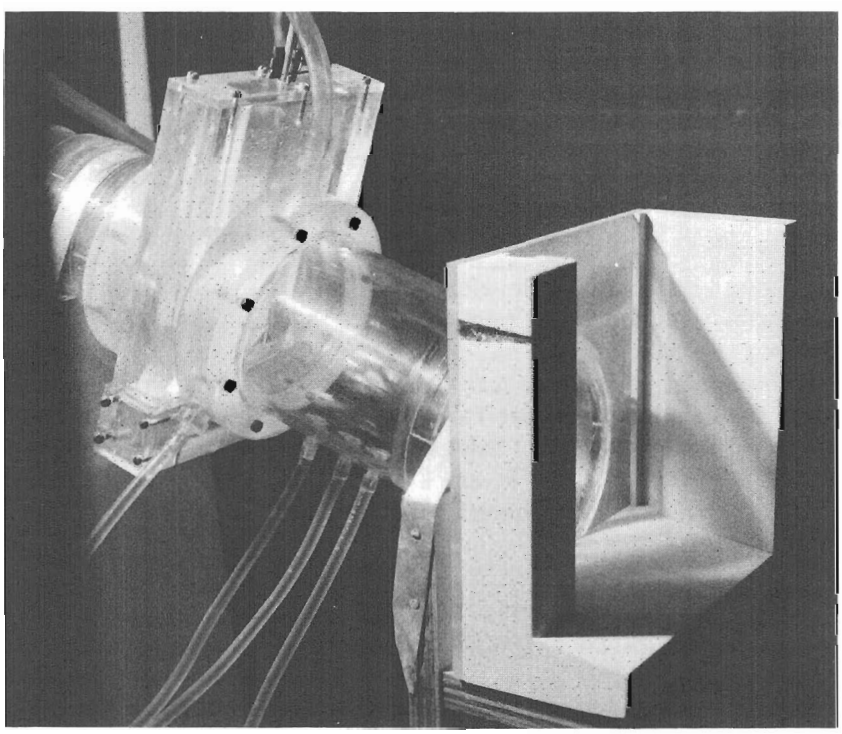

Figure 1: 1:21 scale hydraulic model of the Seminoe 60-inch jet-flow gate.

In addition to the tests mentioned above, a more general design curve relating aeration, gate opening, and a nondimensional submergence parameter was developed. In these tests, gate position, upstream head, and downstream head were varied widely for a single air vent location.

Results and Discussion

The tests specific to the Seminoe gate yielded some interesting results. In particular, the coefficient of discharge varied widely depending on air vent location. The discharge coefficient C_d is defined as:

$$C_d = \frac{Q}{A_{up}(2g(H_{up}-H_{dn}))^{\frac{1}{2}}} \tag{1}$$

where: Q - discharge
A_{up} - area of the upstream pipe
g - gravitational constant
H_{up} - pressure head upstream from the gate
H_{dn} - pressure head downstream from the gate

Without aeration, the discharge coefficient reaches a value of 0.88 when the gate is fully open. This value drops to 0.62 when the discharge tube is aerated with the top or 45° vent configurations and to only 0.41 for the bottom vent configuration (fig. 2).

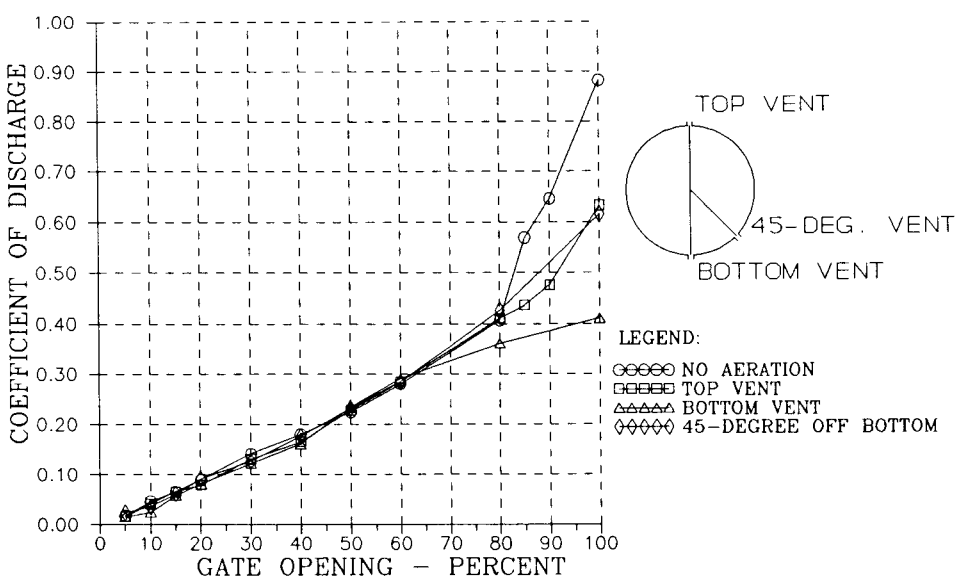

Figure 2: Discharge coefficient for different vent configurations.

Aeration was best accomplished through the vent on the crown of the discharge tube (top vent). Air was pulled into the discharge tube for all gate openings tested (fig. 3). The quantities ranged from 13 to 22 percent (air to water volume ratio). The air was pulled down the gate slots and distributed evenly across the bottom of the orifice seal ring (fig. 4). The other vent locations tested were less effective, pulling in only 2 to 6 percent air to water, and only with gate openings above 20 percent. Due

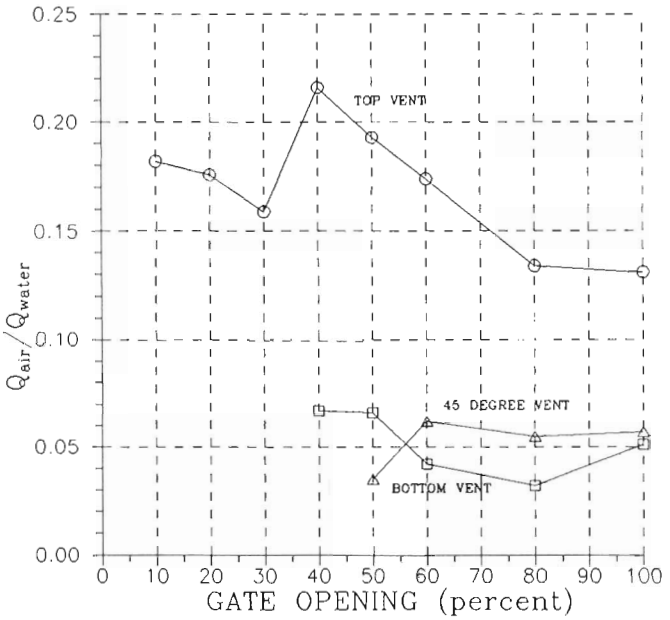

Figure 3: Air demand curves for the three vent configurations tested.

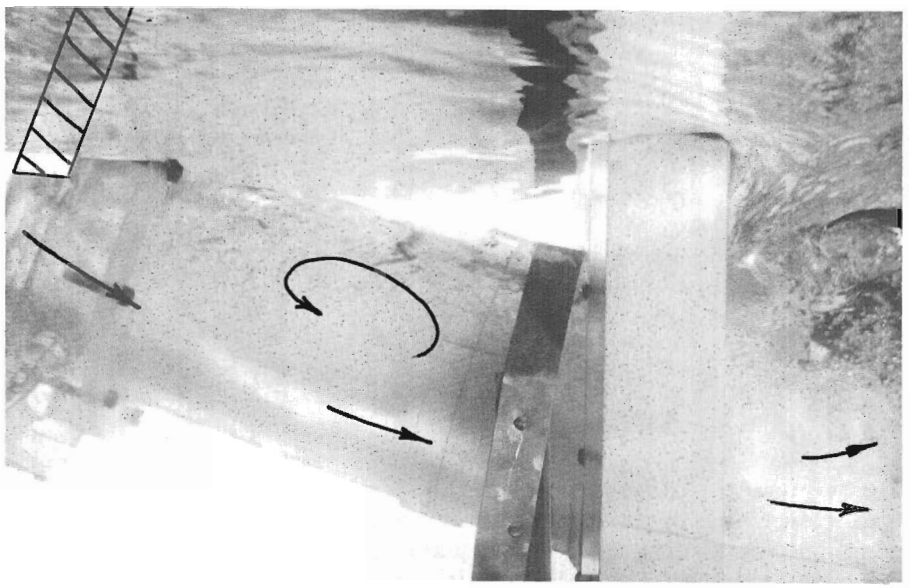

Figure 4: Aeration from top vent, air-water mixture flowing along bottom of discharge tube.

to the limited data on aeration of submerged jet-flow gates, additional tests were run to investigate the effect of the amount of submergence on aeration and to define the inception point for aeration as a function of gate opening. A systematic group of tests were run where the upstream head and level of submergence were varied. Air volume flowrate was measured with an orifice meter. These data were correlated using a nondimensional submergence parameter (fig. 5). The critical point for inception of aeration is where the curves intersect the horizontal axis (β=0).

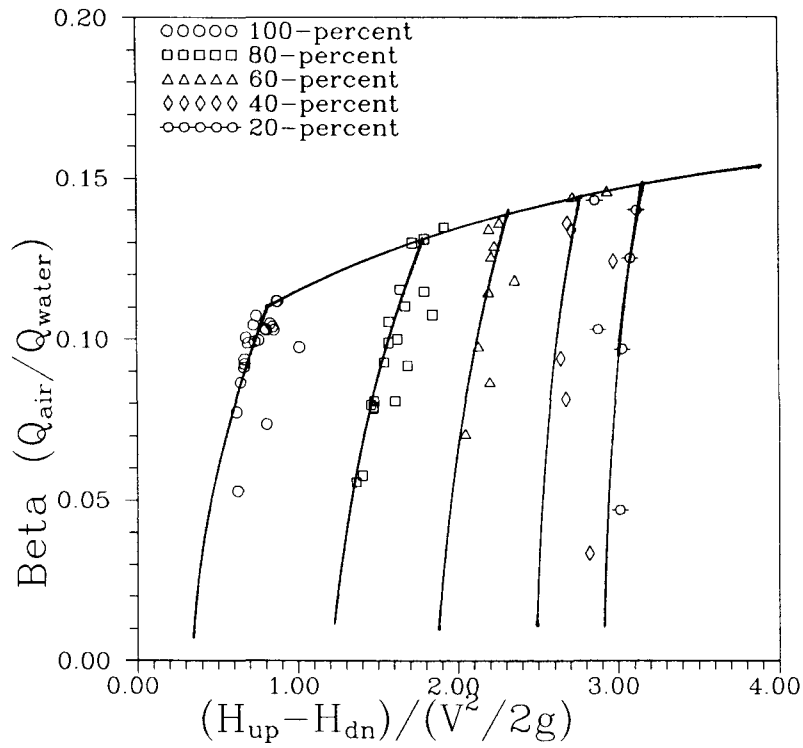

Figure 5: Air demand data from a 1:21 scale hydraulic model. Vent is on top centerline, just downstream from gate.

However, from a closer look at these data, ΔH does not appear to be the sole parameter which influences aeration as the actual submergence head is also quite important.

While the research presented in this paper does not fully address all the factors which influence the scaling of aeration, it does provide additional understanding into the operation of submerged jet-flow gates. In particular, it addresses the effects of aeration on typical design parameters, such as the coefficient of discharge. Because the majority of these data were taken with a single vent configuration (or head loss), care should be exercised when using model data to size prototype structures.

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COMPARISON OF FLUSH AND CHAMBER MOUNTED DYNAMIC PRESSURE TRANSDUCER

By Rodney J. Wittler and K. Warren Frizell[1]

Modeling of hydraulic structures often includes the measurement of dynamic pressure fluctuations. The accuracy of the pressure measurements depends upon the geometry of the tap. Flush mounted and chamber mounted transducers were installed in a 1:21 scale model of the U.S. Bureau of Reclamation Blue Mesa dam and spillway. Differences were observed in the spectral characteristics of the two tap designs. Future prototype tests at Blue Mesa will allow model data to be compared with the prototype response.

INTRODUCTION

Following the 1983 flooding of the Colorado River, the Bureau of Reclamation discovered extensive cavitation damage in the two tunnel spillways of Glen Canyon dam. An air slot was installed to mitigate cavitation damage in each spillway. The potential for similar damage was identified at the Blue Mesa Dam spillway. The Glen Canyon and Blue Mesa spillways were modeled at the Bureau of Reclamation Hydraulics Laboratory in Denver, Colorado. Dynamic pressure measurements made in the Glen Canyon model did not follow Froude scaling when compared with the prototype measurements (Frizell, 1988). The piezometry in the Glen Canyon model consisted of square edged chamber mounted transducers. The prototype installation featured flush mounted transducers.

Two piezometric designs are compared in this paper. The influence of tap design upon the accuracy of the model measurement is the subject of this research.

PIEZOMETRY BACKGROUND

The accuracy of piezometry is influenced by the relative intensity of pressure, temperature, density, and velocity fluctuations. Also, the length to diameter ratio of the chamber, the rigidity of the wall material, the cross flow velocity at the chamber entrance and exit, and fluid interaction with the transducer are factors.

Static piezometry or the design of pressure taps for static pressure measurement is summarized by Bennedict (1984). Allen and Hooper (1931) reported on the relative accuracy of 31 static pressure tap configurations. These included square edged, rounded, beveled, and protruding taps. Allen and Hooper concluded that rounded tap geometry had the least error when

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measuring static pressures. Other researchers verified these findings (Rouse, 1931). The resonant or natural frequency, f_R, of a chambered tap is defined as

$$f_R = \frac{a}{4L} \tag{1}$$

where a is the celerity of sound waves in water and L is the effective chamber length. Den Hartog (1985) showed that amplification of the near-resonant frequency amplitude occurred and varied with the degree of damping. To avoid resonant amplification or attenuation, Den Hartog concluded, the resonant frequency of the chamber must be 20 to 100 times that of the frequencies of interest. Piezometer design is therefore a critical component of measuring dynamic model characteristics. The static pressure tap design recommended by Allen, Hooper, and Rouse was constructed in the Blue Mesa model adjacent to a flush mounted transducer. Identical transducers were installed in both the flush and chambered taps. The dynamic pressure readings from each transducer differed due to the configuration of the tap.

PIEZOMETRY DESIGN

The transducer mounts were made for Kistler Quick Change pressure transducers. The transducer diameter is 0.221 inches, the same diameter as the chamber. The length of the chamber tap is 1.287 inches from the inside wall of the spillway to the diaphragm of the transducer. The corresponding length to diameter ratio is 5.82. This is within Bennedicts (1984) recommended range of

$$1.5 < L/D < 6 \tag{2}$$

where D is the diameter of the chamber. The corresponding resonant or natural frequency of this tap is roughly 8 kHz, depending on the ambient water temperature. The chamber tap has rounded edges with a radius of 0.4 times the diameter of the chamber, or 0.088 inch, as specified by Allen, Hooper and Rouse.

The flush tap is located ten degrees clockwise, looking downstream, off the spillway invert. The chamber tap is located ten degrees counterclockwise off the spillway invert. Figure 1 shows the flush mounted tap, designated T4F, and the chambered tap, designated T4. Figure 2 shows both T4 and T4F located 3 inches upstream from the point of tangency of the spillway elbow. A static pressure tap is located on the invert between the two dynamic taps. Three other chambered taps were installed as shown in the figure. Taps T1, T2, and T3 are identical to tap T4. Tap T1 was installed midway on the ramp of the air slot. This location was chosen for the absence of entrained air. The comparison of spectral characteristics of both air-entrained and non-entrained flow was necessary to separate air effects from piezometry effects.

INSTRUMENTATION

The Kistler Model 211-B low impedance subminiature transducer has a quartz diaphragm with a frequency response from near D.C. to 100 kHZ. A coupler controller conditioned the transducer signal. Spectral analysis was performed on an HP 3562A Dynamic Signal Analyzer.

RESULTS

Observations of pressure time records and spectral quantities were used to evaluate differences between the two tap designs. Time records shown in Figure 3 show an obvious difference in signal characteristics. The chamber tap (top of Figure 3) shows larger fluctuations, but at a

much lower frequency. This is reaffirmed by a plot of the power spectral densities shown in Figure 4. At frequencies less than 3 Hz the chamber tap has more power than the flush tap. However, above this value the curves cross and the flush tap has power approximately 10 dB above the chamber tap throughout the frequency range of interest. Similar trends were found in the comparison between the prototype flush tap and the model chamber tap used in the Glen Canyon study (Frizell, 1988).

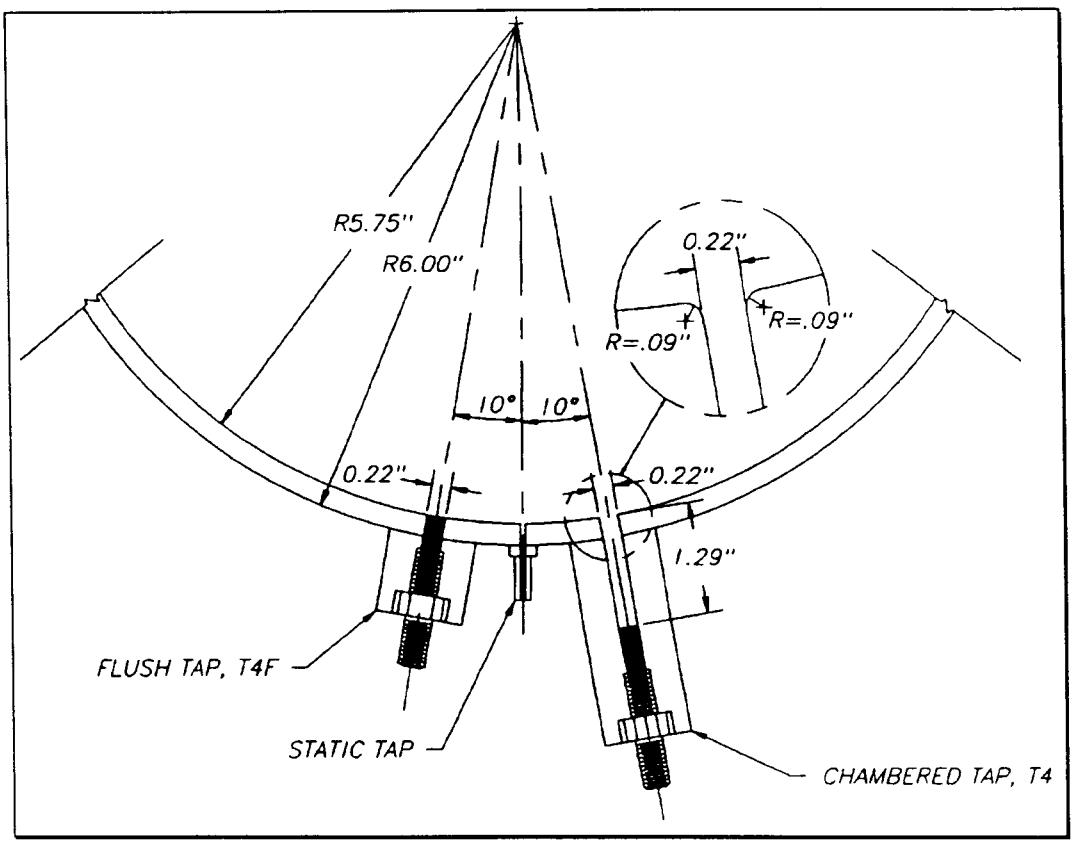

Figure 1. Flush and Chambered Tap Schematic.

The comparison of flush and chamber mounts is complicated by the presence of air in the flow. When air entrainment is present, the chambering factors become less discernible. Identical chamber taps located in areas of unaerated (T1) and aerated (T4) flow show that air bubbles in the flow may cause a shift in power to the frequencies less than 5 Hz. This trend is shown in Figures 5 and 6. However, with increasing discharge (flow depth) the trend does not continue. At the greatest discharges the tap least affected by aeration, tap T1, actually shows greater power at the low frequencies than tap T4.

CONCLUSIONS

It has been observed that chambering attenuates the higher frequency fluctuations and tends to amplify the low frequencies. In addition, the presence of air bubbles in the flow complicates the problem of chambering. In general, it is recommended that flush mounted transducers be used whenever possible. This is especially true when air entrainment is expected. The scaling

of dynamic pressure fluctuations is possible when both model and prototype are flush mounted. However, if either or both are chamber mounted, development of a reasonable correlation is highly unlikely. Plans are currently being explored to take prototype measurements in the Blue Mesa spillway tunnel. These measurements will be made with flush mounted transducers.

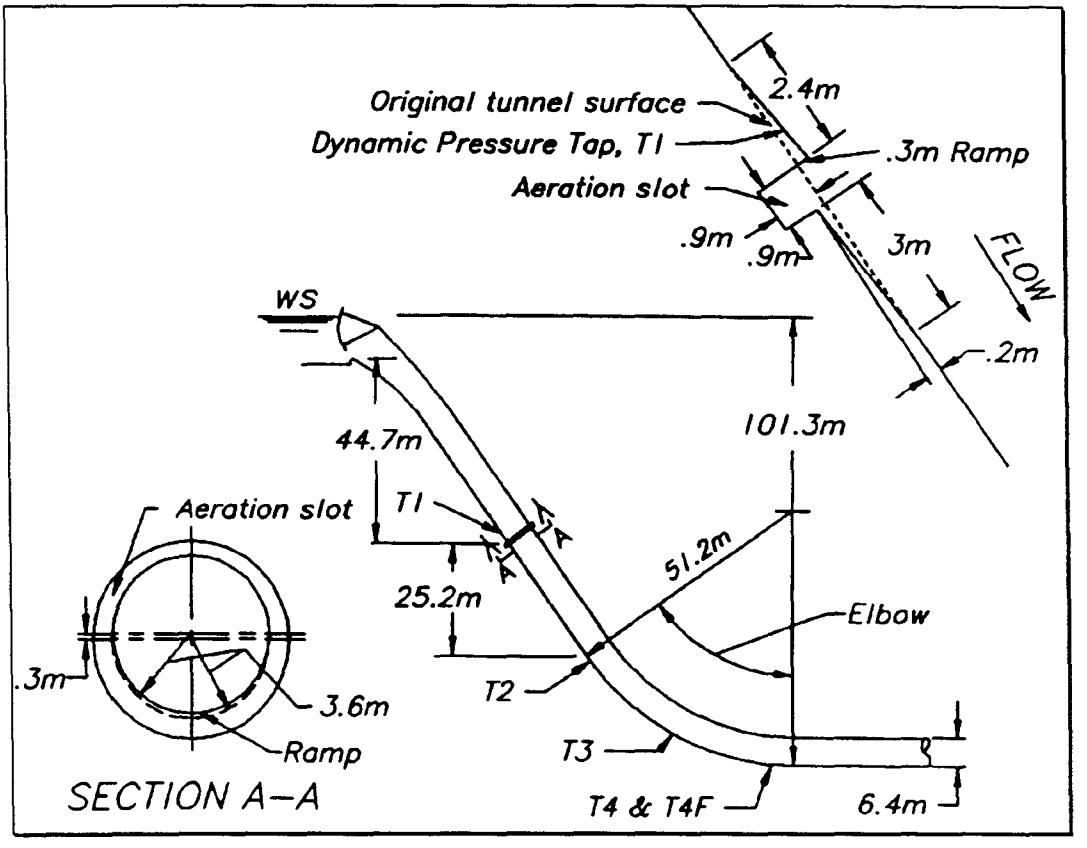

Figure 2. Cross Section of Blue Mesa Spillway. (Prototype Dimensions)

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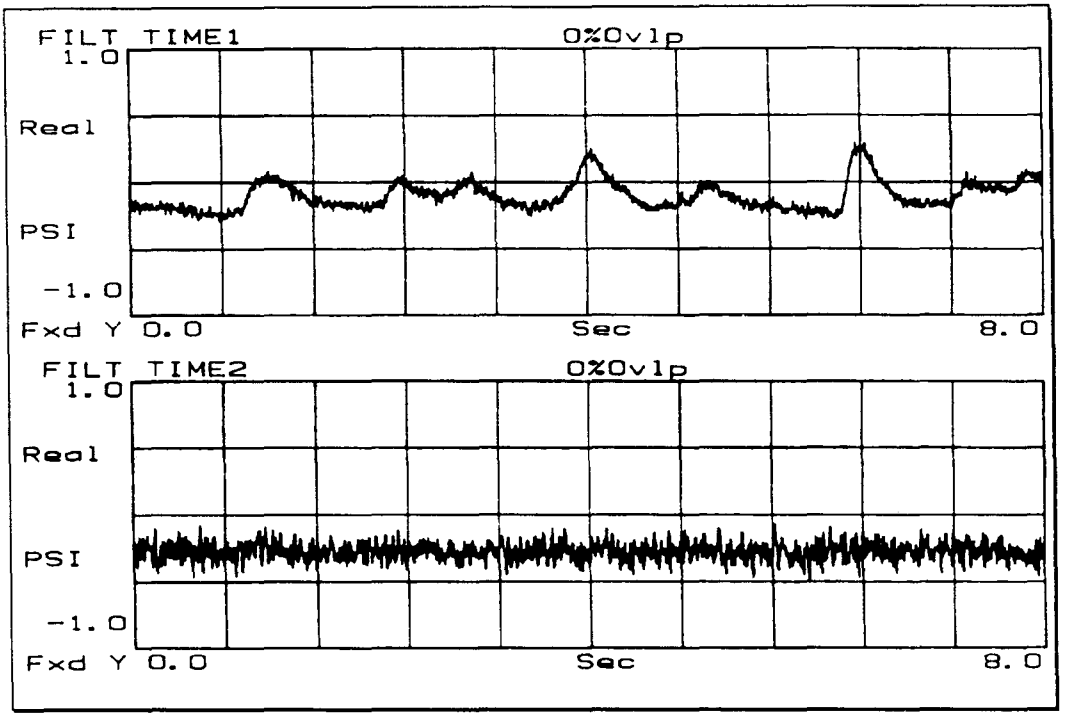

Figure 3. Time Record of Taps T4 (top) and T4F (bottom).

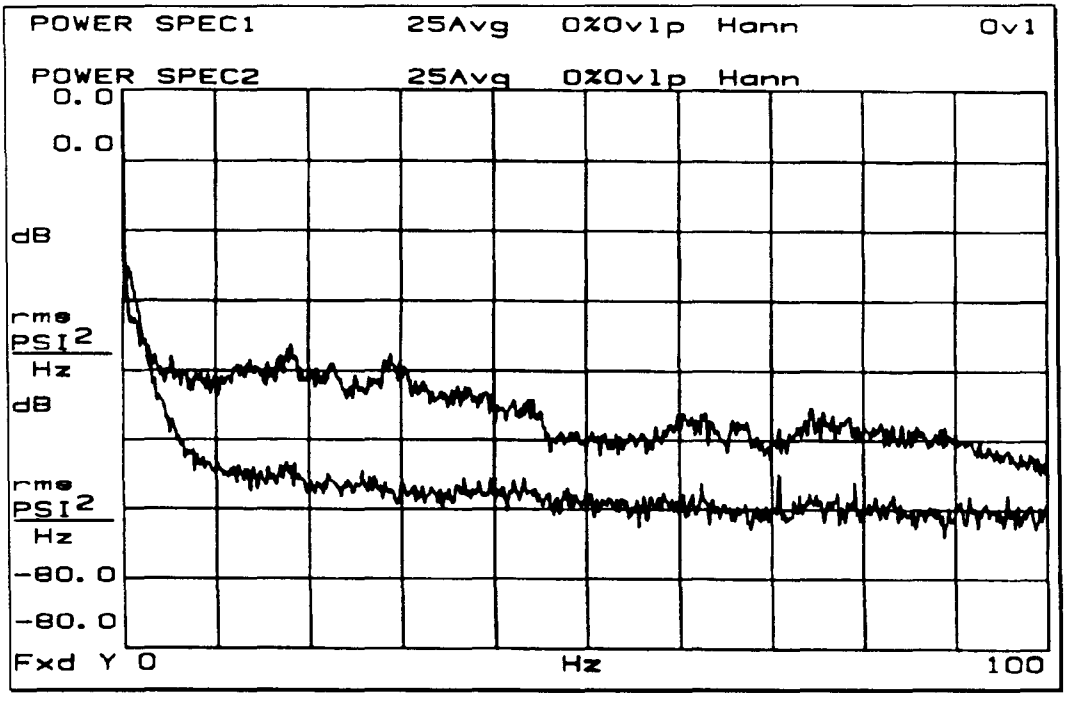

Figure 4. Spectral Power Density for Taps T4F and T4
(T4F upper and T4 lower respectively, $f > 5$ Hz).

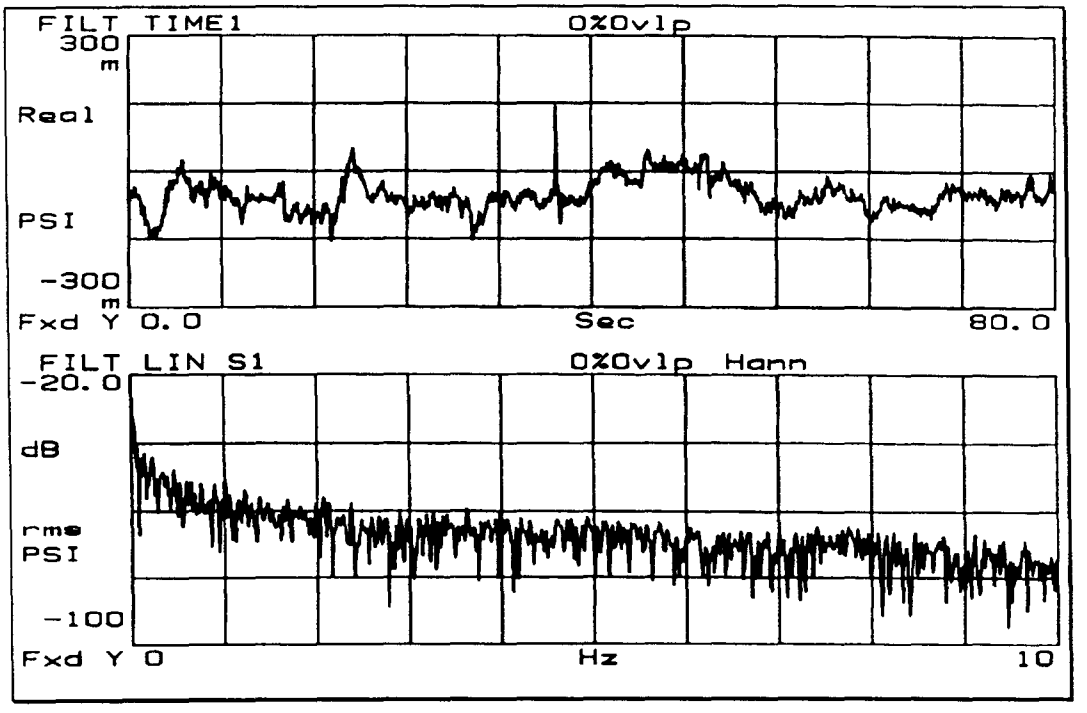

Figure 5. Time Record (Top) and FFT (Bottom) of Tap T1 at 1000 Ft^3/s.

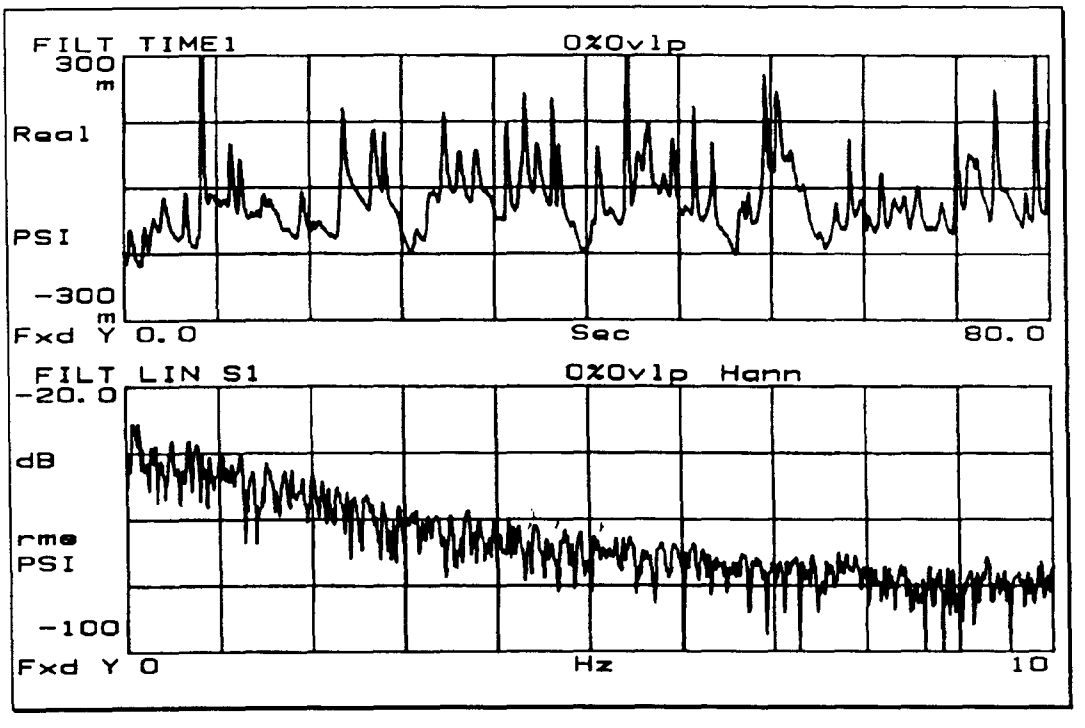

Figure 6. Time Record (Top) and FFT (Bottom) of Tap T4 at 1000 Ft^3/s.

GUARD GATE TESTS AT TIETON DAM

by

Clifford A. Pugh, M., A.S.C.E., Rodney J. Wittler, A.M., A.S.C.E., and Jerry R. Fitzwater[1]

Introduction

The emergency gate hoist system on the guard gates at Tieton Dam, near Yakima, Washington, was tested as part of the United States Bureau of Reclamation Safety Evaluation of Existing Dams (SEED) program. The Tieton tests are the final part of a comprehensive study by Reclamation for developing procedures for testing the adequacy of guard gates in emergency closure. Previous studies of guard gates in the SEED program include a hydraulic model of the Cedar Bluff Dam gate and a field test at Silver Jack Dam. A computer model, developed with data from the Silver Jack model and Cedar Bluff Field tests, simulates pressure and air demand during guard gate closure. The test at Tieton provides information for further calibration of the computer model.

If the service gates are not operable in an emergency, the guard gates must control and stop the flow of water. The two primary questions associated with an emergency closure are:

1.) Is the hydraulic hoist system capable of operating the gate?

2.) Does the air vacuum relief valve supply sufficient air to the downstream side of the gate?

The purpose of this paper is to explain the hydraulics acting on typical flat bottom gates and to provide a method of analysis for determining the forces acting on a gate.

During operation with unbalanced head, a cavitation pocket can form over the entire bottom of a flat bottom gate. The low pressure in this region creates downpull on the gate. The geometry of the gate bottom affects the flow and thus the loading on the gate. Static loads include the differential hydrostatic load, the upthrust and the downthrust. The difference in head applied to the upstream and downstream faces of the gate is the differential load. The differential load on the gate causes a large friction force on the gate guides. The reservoir head acting on the bottom of the gate causes upthrust. The pressure in the gate bonnet causes downthrust.

Test Procedures

Balanced head (conduit downstream of the gate pressurized and no flow) and unbalanced head tests employed identical procedures. There are two parallel conduits with guard gates and service gates at the Tieton installation. Figure 1 profiles the right 5 foot by 6 foot guard gate and gate chamber at Tieton Dam. All tests were performed on the right guard gate. Flow through the left gate and conduit was maintained at 900 ft^3/s for downstream deliveries. Dual hydraulic systems

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allow the guard gates to be operated from the gate chamber or from the top of the dam. Unbalanced head and balanced head tests were performed using each system.

During the tests, the following measurements were recorded:

(1) Gate position, using a string transducer.

(2) Oil pressure on the top and bottom of the hoist cylinder, using 0 to 1,000 lb/in^2 transducers.

(3) Air pressure on the lower side of the 6-inch air vacuum valve, using a 0 to -5 lb/in^2 transducer.

(4) Air velocity at the entrance to the 6 and 8-inch air valves during the test.

(5) The reservoir head. The reservoir head was 180.40 feet on the guard gate invert.

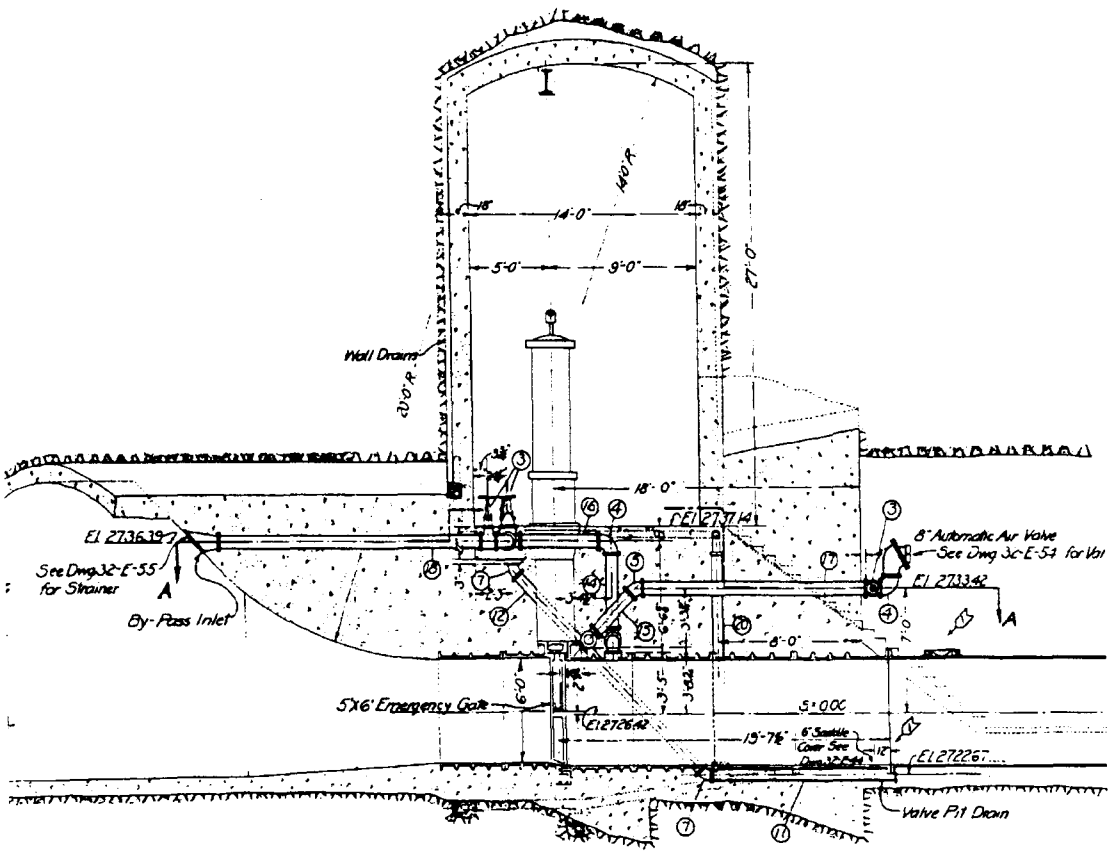

Figure 1. Profile of Tieton Dam Guard Gate Chamber.

The first test, Test no. 1, was a balanced head opening and closure test. During this test the right service gate, a jet-flow gate, remained closed. The conduit remained full and under pressure while the guard gate was exercised from closed to open to closed. The hydraulic system was operated from the gate chamber during this test. Test no. 2 was also a balanced head opening and closure test. The hydraulic system was operated from the top of the dam during this test. The third test, Test no. 3 was an unbalanced test. The guard gate is subjected to the entire reservoir head on the upstream face and subatmospheric pressure on the downstream face during an unbalanced test. Initially, the right guard gate was closed, the bypass conduit filling line valve was closed, and

the right conduit was drained. The jet-flow gate was opened. The hydraulic system was operated from the guard gate chamber. The guard gate was opened roughly 20 percent (1.2 feet) and maintained at that opening for several minutes and then closed. The recording oscillograph was not set at the proper resolution to record the maximum oil pressure encountered during test no. 3. Visual readings from a Bourdon type pressure gage on the hydraulic system were substituted for some of the transducer readings. The procedure for an unbalanced head test operated from the gate chamber was repeated at the proper recorder resolution during test no. 5.

During the second unbalanced head test, test no. 4, the hydraulic system was operated from the top of the dam. The gate opened less than 1 inch during this test. This hydraulic system could not open the gate beyond this point. Two attempts to open the gate failed. Test no. 5, the last unbalanced head test, was completed by operating the hydraulic system in the gate chamber. The guard gate was opened roughly 20 percent (1.2 feet) and maintained at that opening for several minutes and then closed. After the tests, the jet-flow gate was closed and the conduit between the guard gate and jet-flow gate was refilled.

Results

Data from the oscillograph chart were read manually. The readings were converted to engineering units according to the transducer calibrations. The readings were entered into a spreadsheet for analysis. There are three categories of information. The first category is the rate of gate opening and gate closing. The second is the hoist load and gate opening values. The third is the air demand and gate opening values. For Test no. 1 the rate of opening of 10.4 percent per minute (7.5 ft/min) and the rate of closing of 10.8 percent per minute (7.8 ft/min) are roughly equivalent. During opening, the only opposing force is the submerged weight of the gate, roughly 8,000 pounds. During closing the weight of the gate is sufficient to close the gate.

The rate of opening for Test no. 2 is 7.9 percent per minute (5.7 ft/min) and 7.4 percent per minute (5.3 ft/min) for closing. These are roughly equivalent. Opening and closing during test no. 2 took about 13 minutes each compared to 9.5 minutes for test no. 1. This difference is due to the losses in the longer hydraulic oil lines from the top of the dam.

The rate of opening for Test no. 3 is 4.0 percent per minute (2.9 ft/min). The rate of closing is 9.6 percent per minute (6.9 ft/min). The rates of opening and closing are different due to the change in loading. The absolute pressure in the throat of the 6 inch air vacuum relief valve dropped 2.66 lb/in^2 below the atmospheric pressure of 13.66 lb/in^2. The rate of opening for Test no. 5 is 4.4 percent per minute (3.2 ft/min). The rate of closing is 9.4 percent per minute (6.7 ft/min). Figure 2 shows the hoist load versus gate opening for Test no. 5. Figure 3 shows the air pressure versus gate opening for Test no. 5.

A free body analysis of the gate including the dynamic and static forces on the gate under unbalanced head was performed. These forces include the weight of the gate (W), downthrust on the top of the gate (D), upthrust on the bottom of the gate (U), downpull on the bottom of the gate (D_p) and the friction force, (F_f), developed in the gate guides. The following formulas summarize the free body equations. The forces in the vertical direction sum to zero. The positive direction, y, is upwards.

$$\Sigma F_y = 0 \tag{1}$$

Under opening conditions Equation (1) may be written as follows.

$$P_o - W - D - F_f + U - D_p = 0 \tag{2}$$

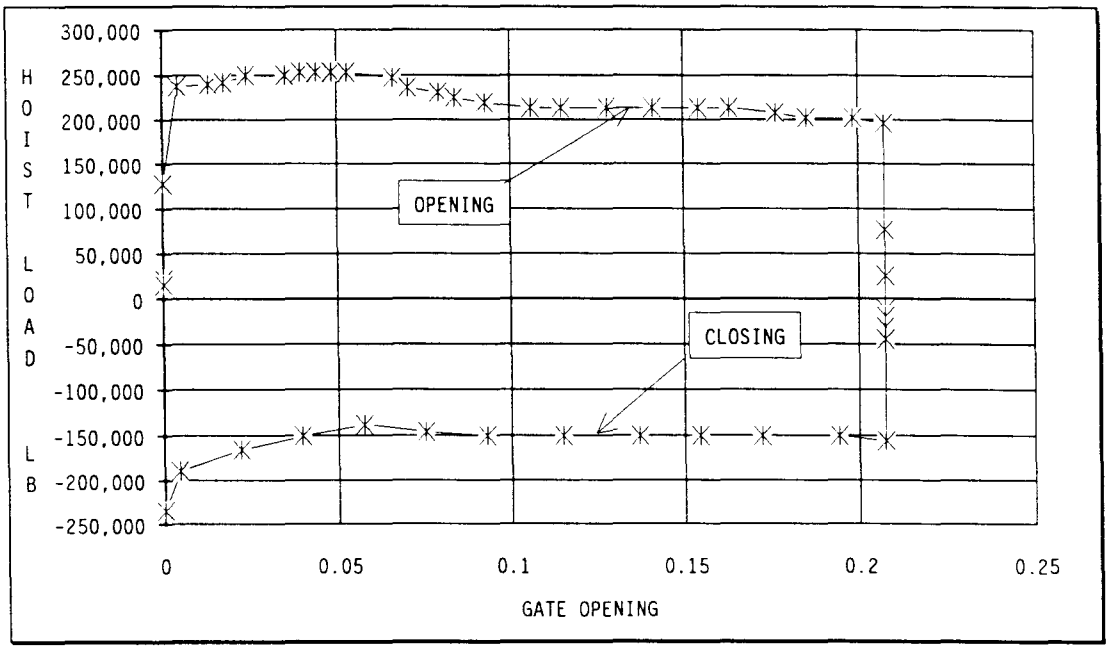

Figure 2. Hoist Load Versus Gate Opening Unbalanced Test no. 5.
Gate Opening is in Decimal Fraction.

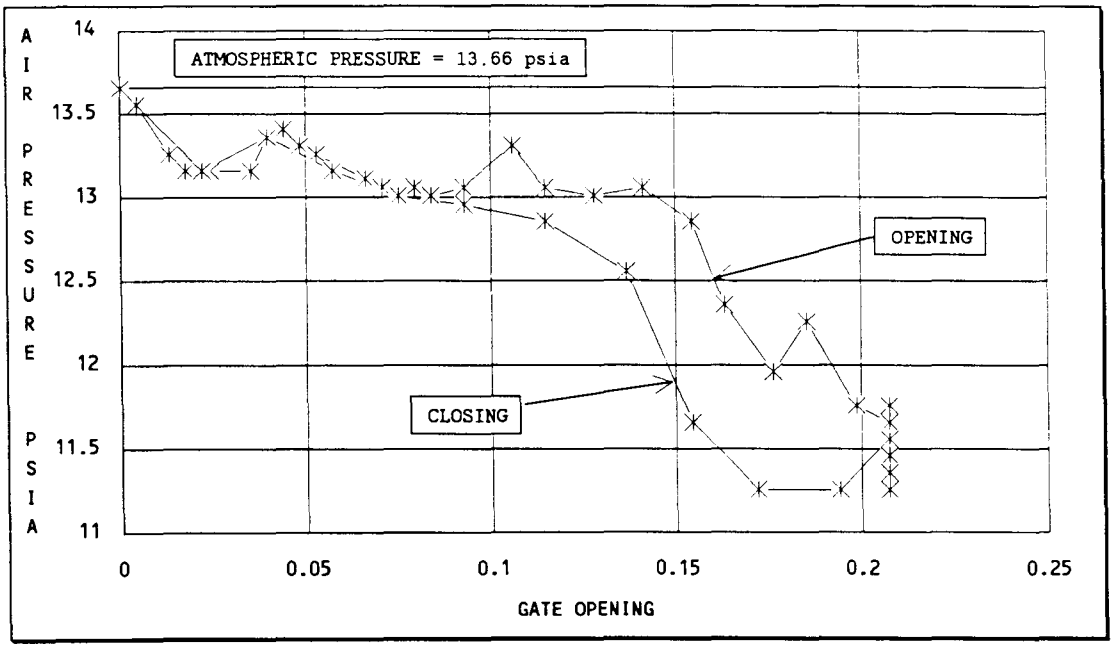

Figure 3. Air Pressure in 6 Inch Air Vacuum Relief Valve.
Unbalanced Test no. 5. Gate Opening is in Decimal Fraction.

In the preceding expression P_o is the hoist load while opening. Under closing conditions Equation (1) may be written as follows.

$$P_c - W - D + F_f + U - D_p = 0 \quad (3)$$

By subtracting Equation (3) from Equation (2), the following expression is derived.

$$P_o - P_c = 2F_f \quad (4)$$

The horizontal hydrostatic load, H, on the gate is a function of the static head acting on the gate, h, the vertical area of the gate, A_g, and the fractional gate opening, G_o.

$$H = 62.4\, h\, A_g\, (1-G_o) \quad (5)$$

The sliding friction force, F_f is given by the following expression, where μ is the sliding coefficient of friction.

$$F_f = \mu H \quad (6)$$

When the opening and closing equations are subtracted, the remaining terms P_o and P_c become a function of the friction force. Therefore, the friction force for any gate position can be determined by subtracting the closing force from the opening force at that gate position and dividing by 2.

$$F_f = (P_o - P_c)/2 \quad (7)$$

The friction coefficient, μ, is computed as the ratio of the friction force to the hydrostatic load on the gate. During Test no. 5 the friction coefficient varied from 0.61 to 0.66. The average value of the coefficient is 0.64. This value is very high considering the sliding coefficient of friction for newer gates with self-lubricating guides is typically 0.35. Because the weight of the gate, friction force, and cylinder force are known, the remaining hydraulic forces can be grouped together and determined. The remaining hydraulic forces are the upthrust, downthrust and downpull. Figure 4 is a plot of these hydraulic forces. The maximum hydraulic force of -48 kips is obtained at a gate opening of roughly 5 percent (0.3 ft).

At a gate opening of 5 percent (0.3 ft) the flow separates off the front edge of the gate and a vapor cloud forms over the bottom of the gate. The maximum downpull due to subatmospheric pressure is roughly -7,000 pounds (-30 ft x 62.4 lb/ft^3 x 3.75 ft^2). The hydrostatic upthrust on the bottom of the gate is eliminated because of the vapor cloud on the bottom of the gate. However, the down-thrust on the top of the gate remains. The downthrust is roughly -41,000 pounds (174 ft x 62.4 lb/ft^3 x 3.75 ft^2). The top and bottom cross-sectional area of the gate is estimated to be 3.75 ft^2. These forces, -7,000 pounds downpull, and -41,000 pounds downthrust, equal the maximum hydraulic force measured on the gate at a 5 percent (0.3 ft) opening. This is shown in figure 4.

Air Demand

Figure 3 shows the pressures measured in the 6-in air vacuum relief valve. This valve is mounted on a manhole cover just downstream from the 8-in automatic air valve (see Figure 1). The 6-in air valve is not shown in the figure. Previous studies have shown that both the 6 and 8 inch air valves are required to supply the necessary volume of air.

The collapse pressure of the conduit was calculated to be -7 lb/in^2 or 6.7 lb/in^2 absolute. A computer program calculation indicated that the air vent pressure would be -3.47 lb/in^2 at 20 percent (1.2 ft) open. This pressure assumes that both vents are operating. The measured air vent pressure at Tieton for a 20 percent (1.2 ft) opening (closing cycle) was -2.46 lb/in^2 [11.2 lb/in^2 absolute]. Therefore the computer program is slightly conservative in this case.

Conclusions

The hydraulic forces, friction forces, and weight of the gate were determined during the test. The maximum hoist load is encountered during the opening cycle at a 5 percent (0.3 ft) gate opening. The sliding coefficient of friction is about 0.64. The 6-in and 8-in automatic air valves provide an adequate supply of air to alleviate low pressure in the conduit between the service gate and guard gate. The computer programs provide a good tool for predicting pressures and air demand in the conduit.

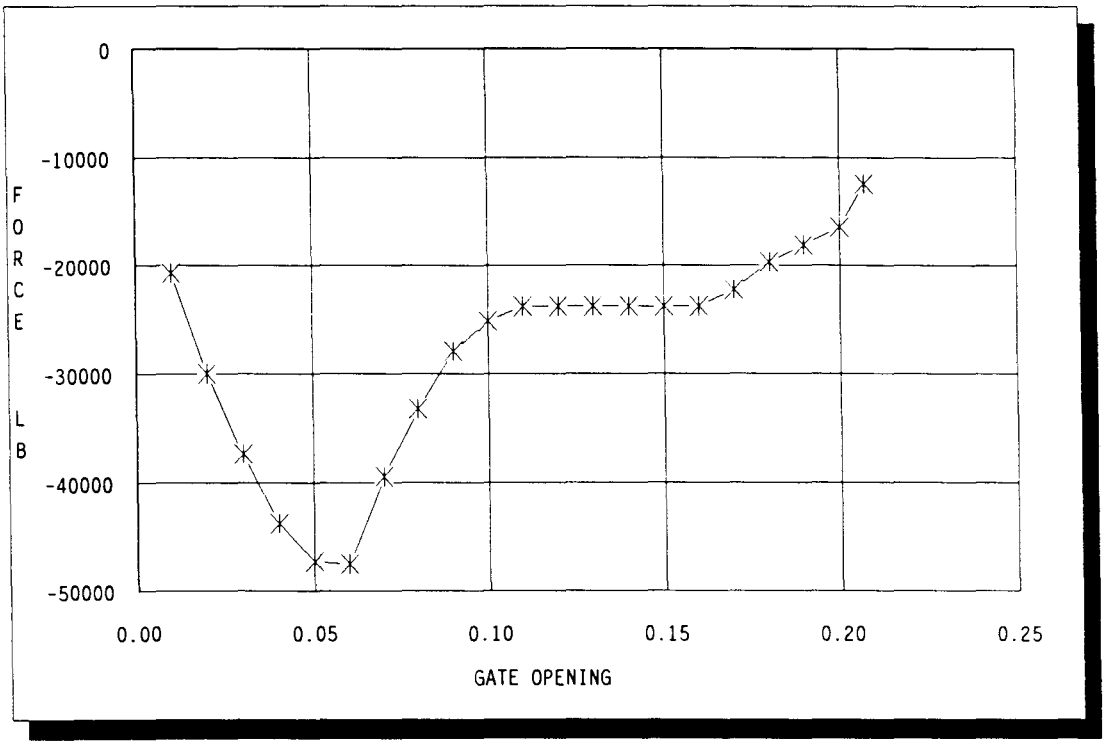

Figure 4. Hydraulic Forces Versus Gate Opening.
Unbalanced Test no. 5. Gate Opening in Decimal Fraction.

The inability of the hoist system to open the gate when operated from atop the dam prompted an evaluation and subsequent modification of the hydraulic operating system.

The measurements at Tieton Dam support theoretical calculations of hoist loads. The maximum hydraulic forces measured were substantiated by assuming that a vapor cloud is covering the bottom of the gate and calculating the resulting forces. Therefore, the maximum hoist loads for full reservoir head and any other head can be calculated. The methodology should also apply to other similar installations.

"Hydraulic Modeling of Boating Hazards and Sedimentation - Union Avenue Dam, Denver Colorado"

by Clifford A. Pugh[1] M.ASCE, and Cassie C. Klumpp[2] A.M.ASCE

Abstract. -

Hydraulic jump type energy dissipators are a hazard to recreational boaters. Many white water boaters have been trapped in "reverse roller" hydraulic jumps. Numerous drownings have resulted. The Colorado Water Conservation Board (CWCB) requested the Bureau of Reclamation's assistance in developing a modification to the Union Avenue Dam on the South Platte River in Denver, to reduce risks to recreational boaters while protecting the major functions of the structure. A model study was conducted at Reclamation's hydraulic laboratory in Denver with direction from the CWCB consultant, Wright Water Engineers (WWE). The model was used to develop refinements to the boatchute configuration to improve boatability and safety. Several recommendations were made to reduce construction costs. A water supply intake structure for the city of Englewood located at the dam was studied with respect to sediment buildup and sluicing. A standard boatchute configuration evolved from the Union Avenue study. Flow over the structure during high discharges was also studied to ensure that the flood capacity of the river channel is not affected by the boatchute modifications.

Introduction -

In 1985, a dam was constructed downstream of Union Avenue in Englewood on the South Platte River. The dam serves as a diversion structure for the city of Englewood water supply and also controls the grade of the South Platte River for flood control. The reinforced concrete dam, is 18.5 feet high and has a 3 to 1 slope on the face of the spillway. A section through the existing dam is shown in figure 1.

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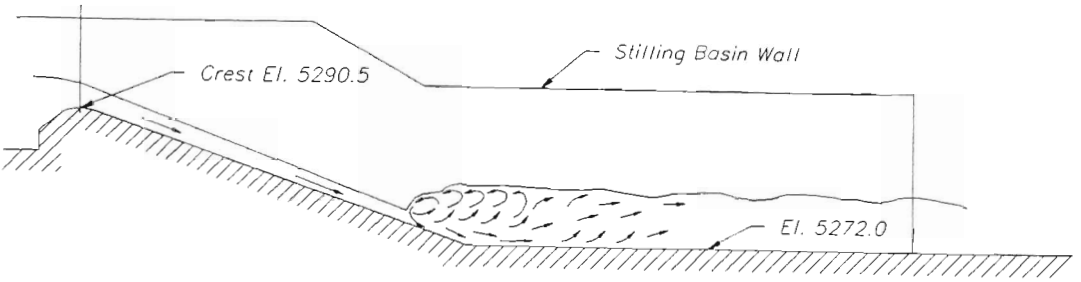

Figure 1. - Section through Union Avenue Dam, showing reverse roller hydraulic jump.

The hydraulic jump type energy dissipator has been used successfully at hundreds of structures throughout the world. However, the reverse roller in the jump can trap a boat or person and is hazardous if a boat should go over the dam. Recreational boating has become more popular on the South Platte in recent years; therefore, boater safety has become a primary concern.

The CWCB is investigating methods to solve the safety problem at the Union Avenue Dam. One solution is to construct a series of boatchutes, each with a drop of 2 to 4 feet to handle the entire drop at the present dam. The CWCB selected WWE to plan the boatchutes at the site. The CWCB also requested assistance on the project from the Bureau of Reclamation. Reclamation agreed to assist in the project, and a physical model of the boatchute was built in the hydraulic laboratory. The testing was divided into three phases: Phase A. - Model tests to achieve safe boating conditions at discharges between 100 and 1,500 ft^3/s. Phase B. - Tests to observe sediment deposition and scour patterns as they affect the Englewood water intake structure and sluiceway and deposition and scour patterns in the first boatchute pool. Phase C. - Flood flow tests up to the 100-year flood to determine the effect of the proposed modifications on the flood stages.

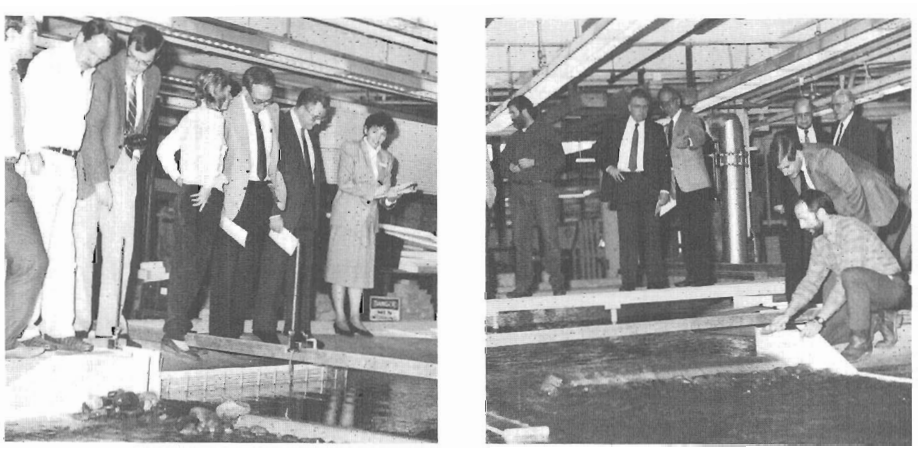

Figure 2 - Model of Union Avenue Dam boatchute modifications.

The Model -

After considering several model scales, the model scale was set at 1 to 18 (undistorted). This scale was chosen primarily to simulate sediment transport and also to permit simulation of boating flows as low as 50 to 100 ft^3/s. The model features included the Union Avenue Dam, City of Englewood intakes, radial gate sluiceway, downstream pool, and the second rockfill dam and boatchute. A large bend in the river upstream from the bridge was included in ensure proper flow conditions at the Englewood intake structure.

Study of Boating Conditions -

Wright Water Engineers provided drawings to construct the original boatchute configuration in the physical model. The first boatchute configuration included a gradually narrowing chute which curved to the left to direct the flow toward the next chute downstream. The model included the first two rockfill embankments and chutes. The third boatchute was simulated by incorporating a weir in the downstream end of the model with the proper shape and elevations to simulate boatchute 3. Tests were conducted for boating flows ranging between 100 and 3,000 ft^3/s. The original configuration caused model rafts and kayaks to impinge on the right side of the boatchute.

During the course of the study numerous changes were tried to improve the boating conditions. The major modifications included: (1) Straightening the chute and superelevating one side to redirect the flow. -- This arrangement caused an irregular wave where the supercritical flow met the tailwater. The boats veered abruptly toward the sluice. Attempts to correct this configuration were unsuccessful. (2) Shortening the chute and eliminating the superelevation. -- The shortening was considered desirable because it reduced embankment volume and lowered costs. The roller wave was still a problem at the end of the chute. (3) A step or ramp in the chute was then tried to deflect the flow across the surface where it reaches the tailwater. -- This concept showed promise, however it was found that a series of two ramps was necessary to eliminate the roller wave throughout the entire flow range. (4) Large boulders (4-5 feet in diameter) were placed on either side of the chute to direct the flow. Through careful placement of the boulders, the flow direction in the downstream pool could be adjusted to direct the boats. However, slight changes in placement of the boulders would cause the boats to veer to one side or the other unpredictably. The boulders were also a safety concern because they could be dangerous to recreational boaters if they should fall out of their boat. (5) A straight chute with two steps or ramps across the entire chute. -- This arrangement worked well. The boulders were not necessary to direct the flow. The only problem with this arrangement was the tendency for boats to turn sideways as they passed over the first ramp. The best solution consisted of extending the low flow notch in the center of the chute through the first ramp. This formed a V shaped undular wave pattern on the water surface downstream of the chute. The notch will also allow boats to clear the first ramp during

low flow. For this configuration flows up to the U.S. Army Corps of Engineers (USACE) 100-year flood of 16,400 ft^3/s were contained within the banks of the river. Figure 3 shows the details of the final configuration.

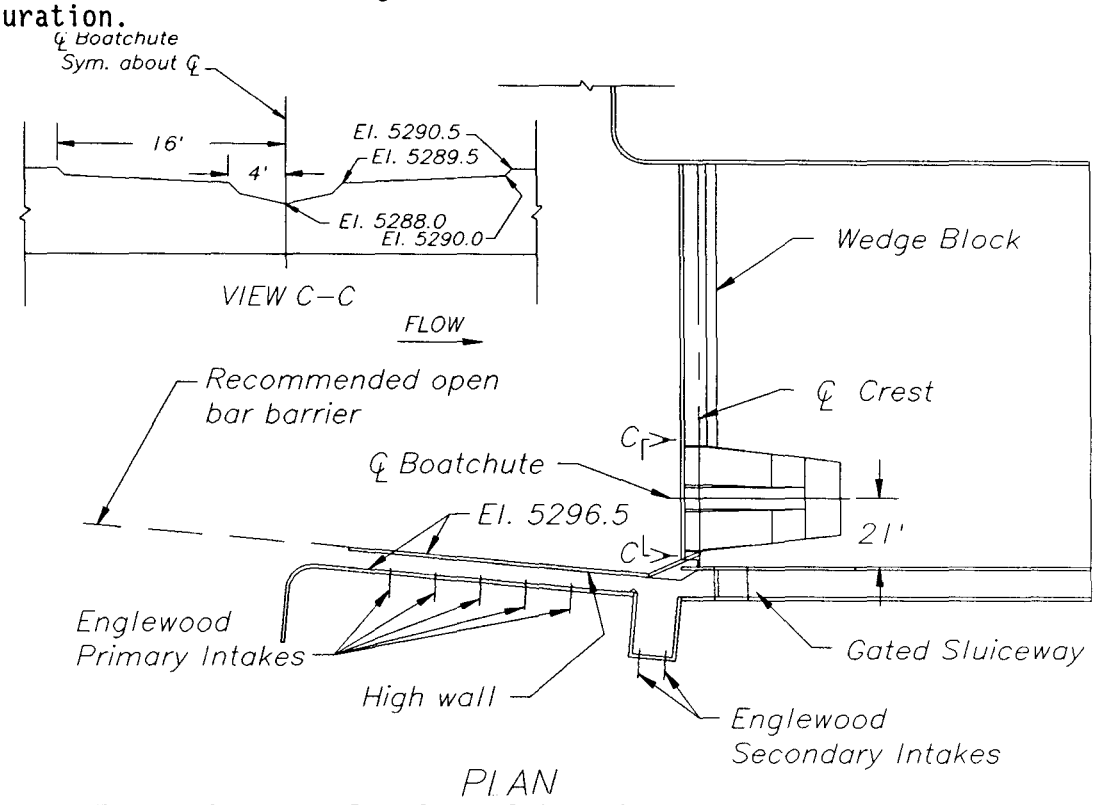

Figure 3 - Details of final boatchute configuration.

Final Boatchute Configuration -

After the final configuration of the boatchute was established, including the low flow notch in the center and the combination of two ramps to disperse the wave, a series of tests was conducted to determine the optimum elevation of the ramps with respect to tailwater elevation. The capacity of the low flow notch in the boatchute is approximately 30 ft^3/s. For flows exceeding 30 ft^3/s (including sluice flows), the second ramp in boatchute 1 will be submerged. The low flow notch extends through the first ramp; therefore, small boats will be able to pass through the notch at low flows. As the flow increases and the pool rises, the combination of the two ramps spreads the wave while maintaining a V pattern in the center.

The final design of the boatchutes was optimized for a river flow of 500 ft^3/s; however, the wave characteristics are acceptable throughout the entire range of boating flows from 50 to 3,000 ft^3/s.

Guidelines were developed during this model study based on tailwater level and total head drop across the chute. Details of the guidelines are given in the model study report.

At flows greater than 500 ft^3/s, the water will start overtopping the main spillway crest. The exact flow will depend on the amount of sluicing and flow into the Englewood intakes. For flows between 500 ft^3/s and 1,500 ft^3/s, boats will tend to get "hung up" on the main crest and downstream wedge, due to lack of water depth. When the water is deep enough for boats to clear the crest, they will pass on through into the tailwater pool area. The configuration with a triangular shaped wedge (Figure 3) mounted on the face of the dam to deflect the flow across the water surface and downstream riprap prevents reverse rollers from forming downstream from the main crest.

Sediment Tests -

A typical bedload particle size distribution curve was obtained from Wright Water Engineers (WWE) for this section of the South Platte River. The model particle size distribution curve was scaled based on techniques outlined in the model study report which account for low Grain Reynolds Numbers in the model.

Estimates of prototype bedload discharge rate were made for a spring flow of 3,000 ft^3/s, which has a return period of 10 years. Sediment discharge was computed with a computer program using several sediment equations, including Schoklitsch, Kalinske, Meyer-Peter and Muller, and Rottmer. Sediment was fed into the model every 15 minutes upstream of the Union Avenue bridge. The existing low (submerged) wall in front of the Englewood intakes was tested first. At a flow of 3,000 ft^3/s, a large deposit formed in the intake area covering the three primary intakes at the upstream end. At a 100-percent sluice gate opening, some of the sediment deposit was reduced. However, the first few intakes were still covered. The sediment test was then continued at a river flow of 3,000 ft^3/s for 3 days with a high wall in place of the low wall along the Englewood intakes (see Figure 3). All of the water entering the intake area was forced to enter at the upstream end. To assess the effect of this change, velocities were measured at various points in the intake area with and without the high wall in place. The sluicing action is greatly enhanced by adding the high wall. For example, the same velocity can be obtained at a 25-percent gate opening with the high wall as with a 100-percent gate opening with the low wall. When the sediment test was conducted at 3,000 ft^3/s and a 30-percent gate opening with the high wall, the primary intake area was almost entirely sluiced out.

After 3 days, the deposition in the pool between the Union Avenue Dam and the first rockfill dam appeared to be stable. Figure 4 shows the sediment deposits in the first pool after the test. A large sand bar formed downstream of the main weir to the left of the boatchute almost to the end of the original stilling basin wall. Another deposit formed downstream of the boatchute in an alluvial fan shape. Downstream from the sluice gate a deposit formed in the submerged sluice area; however, there was no indication of any deposition in the area just downstream of the radial gate. The high velocity under the gate keeps this area clear.

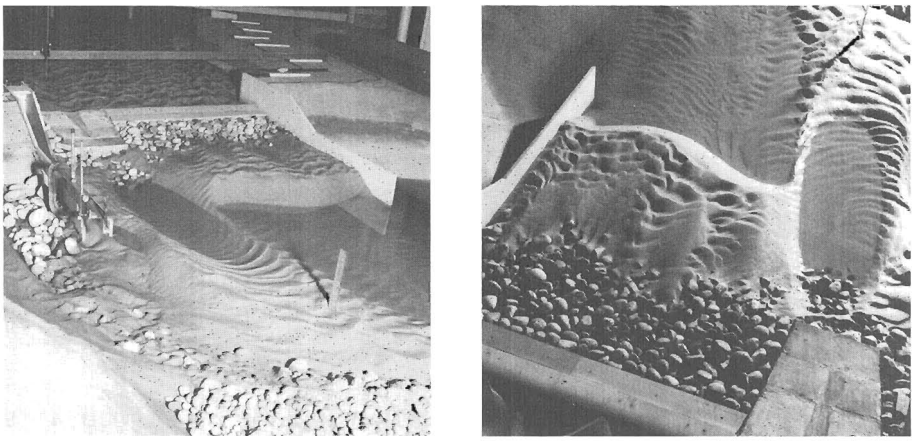

Figure 4 - Sediment deposits in the first pool after sediment test.

Conclusions -

- A final boatchute configuration was developed which eliminates dangerous reverse roller hydraulic jumps. The boatchute employs a double ramp configuration, which provides boatable waves through the boatchutes from 50 ft^3/s to 3000 ft^3/s.

- Guidelines were developed for designing the boatchutes. These guidelines may also be applied to other sites with head drops between 2 and 3.5 feet.

- A solid wall is recommended in front of the primary Englewood intakes to protect boaters from being drawn into the intake and to enhance the present sluicing capacity.

- The volume of rockfill was reduced on the first embankment during the study. Downstream embankments can also be reduced in volume compared to the initial design.

- Sediment deposition tests indicated that deposits in the first pool between boatchutes 1 and 2 will not affect sluicing operations. Flow velocities, directions and waves are acceptable for boating before or after the deposition occurs.

- A long wedge was added along the downstream face of Union Avenue Dam to eliminate a reverse roller that could trap boaters when water is flowing over the main dam crest.

- The water level stays within the banks of the river channel up to the USACE 100-year flood of 16,400 ft^3/s.

Appendix - Reference

Klumpp, C.C., Pugh, C.A., and Fitzwater, J.R. "Union Avenue Boatchute Study," U.S. Bureau of Reclamation, R-89-12, Denver, Co., Sept. 1989.

The Influence of Uniformity on Riprap Stability

by

R.J. Wittler[1], A.M., A.S.C.E., and S.R. Abt[2], M., A.S.C.E.

Abstract

Studies have shown that size uniformity influences the stability of riprap. The ratio of fractional sizes of the riprap mixture, D_{60}/D_{10}, is defined as uniformity. Several riprap mixtures of differing uniformity were tested in a flume study. This study was limited to overtopping flow and angular shaped riprap with the median rock size ranging from 2 to 4 inches. The unit discharge ranged from 1 to 4 cubic feet per second per foot. Stability of the different mixtures is calculated as the ratio of failure discharges. A uniformity of 2.15 was the basis of the stability calculation.

The results show that stability of riprap is proportional to the uniformity of the mixture. Other factors have an influence upon stability. Proper sizing of bedding and filter layers is a factor. The behavior of the mixture as part of an overall structural unit is also a factor. Riprap consists of an interlocked matrix of rocks that forms a structural unit whose stability is greater than its individual members. It was observed that interlocking is a function of uniformity and rock shape. A seventy percent variation in stability occurred over a range of uniformity of 1.56 to 5.33. A graphical method of determining the stability as a function of uniformity is presented.

Introduction

There are many factors that influence the stability of riprap. Some of these are size, or weight, shape, layer thickness, filters, and uniformity of gradation. The influence of gradation uniformity upon riprap stability is the focus of this paper. Three prototype studies conducted at Colorado State University are the basis for the results presented.

Background

Riprap design procedures such as Safety Factors, C.O.E., U.S.B.R., Stephenson, and others are reviewed by Abt et. al. (1988). Each procedure recommends a unique riprap gradation. Gradation is expressed as a uniformity coefficient which is normally a quotient of various size fractions of the riprap mixture. The size fractions are determined by sieve analysis. There are several uniformity coefficients. The coefficient of uniformity, C_u, is defined as the size that sixty percent of

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riprap mixture by weight is finer, D_{60}, divided by the size that ten percent by weight, D_{10}, is finer.

$$C_u = \frac{D_{60}}{D_{10}} \tag{1}$$

The recommended values of C_u for some of the design procedures are listed in Table 1. The designer is left to guess at the response to a gradation other than that specified by the design procedure. None of the design procedures offer a prescription for adjusting rock size based upon uniformity criteria. The question is, how does a gradation with C_u greater or less than specified affect the stability of the riprap layer?

Table 1. Recommended Values of C_u

Design Procedure	S.F.	C.O.E.	U.S.B.R.	F.H.W.A.	C.D.H.	Abt et. al.
C_u	2.50	1.75	2.55	2.70	1.10	2.15

S.F.- Safety Factors; C.O.E.- Corps of Engineers; U.S.B.R.- U.S. Bureau of Reclamation; F.H.W.A.- Federal Highway Administration; C.D.H.- California Dept. of Highways; Abt et. al.- Nuclear Regulatory Commission.

Three studies were conducted between 1985 and 1988 at Colorado State University for the purpose of answering that question. The first two studies are described by Abt et. al. (1987-Phase I, 1988-Phase II). Flumes of eight and twelve feet in width and embankment slopes of one percent to twenty percent were covered with one, two, four, five and six inch median diameter, angular riprap. A tilting flume, three feet wide and thirty feet long, was sloped at five, ten, fifteen and twenty percent and covered with three inch median diameter riprap in the third study. Each of the applicable tests in the three studies maintained a constant median diameter while the gradation was varied from test to test. The minimum value of C_u tested was 1.56 and the maximum value of C_u tested was 5.33.

The unit discharge at failure, q_f, ranged from 1.00 $ft^3/ft/s$ to 4.12 $ft^3/ft/s$. Abt et. al. (1988) correlated the unit failure discharge with the embankment slope, S, and the median diameter, D_{50}, for 18 tests. The result of the correlation is an expression for sizing riprap on embankments.

$$D_{50} = 5.23\, S^{0.43} q_f^{0.56} \tag{2}$$

The coefficient of uniformity varied from 1.72 to 2.30 for these tests. The normal value of C_u was defined as 2.15.

Results

Fifteen tests from the three studies had coefficients of uniformity that deviated significantly from 2.15. These tests are summarized in table 2. Equation 2 may be rearranged to solve for the unit failure discharge given the slope and median diameter

$$q_f^* = (D_{50}/5.23S^{0.43})^{(1.79)} \tag{3}$$

where q_f^* is the normal unit failure discharge of a riprap mixture with $C_u = 2.15$. The unit failure discharge of each of the fifteen mixtures divided by the corresponding normal unit failure discharge is defined as the coefficient of stability, C_s.

$$C_s = \frac{q_f}{q_f^*} \tag{4}$$

For example, a C_s of 1.50 indicates a fifty percent increase in the failure discharge due to a change in uniformity. The coefficient of stability is included in Table 2.

The coefficient of stability, C_s, is shown plotted versus the coefficient of uniformity, C_u, in Figure 1. The maximum increase in stability occurs at the minimum values of uniformity. The stability continues to decrease for values of C_u greater than 3.0. The greatest increase, forty five percent, occurs at a C_u of 1.56. The greatest decrease, thirty four percent, occurs at a C_u of 4.00.

Table 2. Results of Studies.

D_{50}	Slope	C_u	q_f	q_f^*	C_s
inches	Ft/Ft	D_{60}/D_{10}	$Ft^3/Ft/s$	$Ft^3/Ft/s$	q_f/q_f^*
2	0.10	2.14	1.00	1.05	0.95
2	0.10	2.14	1.11	1.05	1.05
4	0.10	2.30	3.79	3.63	1.04
4	0.10	4.00	2.41	3.63	0.66
4	0.10	1.72	4.12	3.63	1.13
3.3	0.15	1.56	2.67	1.84	1.45
3.3	0.10	1.56	3.13	2.51	1.25
3.3	0.20	1.56	1.96	1.47	1.33
3.3	0.10	2.92	2.68	2.57	1.04
3.3	0.20	2.92	1.58	1.51	1.04
3.3	0.15	2.92	1.95	1.89	1.03
3.2	0.05	5.33	3.24	4.15	0.78
3.2	0.10	5.33	2.07	2.44	0.85
3.2	0.15	5.33	1.46	1.79	0.82
3.2	0.20	5.33	1.11	1.43	0.78

The average decrease in stability for values of C_u greater than 3.0 is close to twenty percent. The increase or decrease in stability is less than ten percent for values of C_u between 1.8 and 3.0. Abt et. al. fit a line to the five data points of the tests performed in the Phase I and Phase II studies. This line is shown in Figure 1 as the "1988" line. The additional ten tests performed in 1988 by Wittler are also shown in Figure 1. A new "1990" curve has been fit to all fifteen points.

Influence of Uniformity

Uniformity has a definite influence upon the stability of riprap. Observation of the tests performed in the three studies has identified several stages that the riprap layer passes thru prior

to failure. Failure is defined as rock movement that results in a breech of the riprap layer. The uniformity of the mixture magnifies some of these stages and minimizes others. As the riprap is inundated the voids fill with water and interstitial flow begins. Initial settling of the smallest particles occurs at this stage. As surface flow begins, the entire riprap layer settles and assumes the load imposed by the shear stress. The shallow surface flow breaking over the individual rocks causes fluctuating pressures that vibrates and settles the riprap layer. As the shear stress increases with increasing flow, the riprap layer continues to settle and some surface rocks are swept downstream by the moving water. Most of these rocks relodge a short distance downstream.

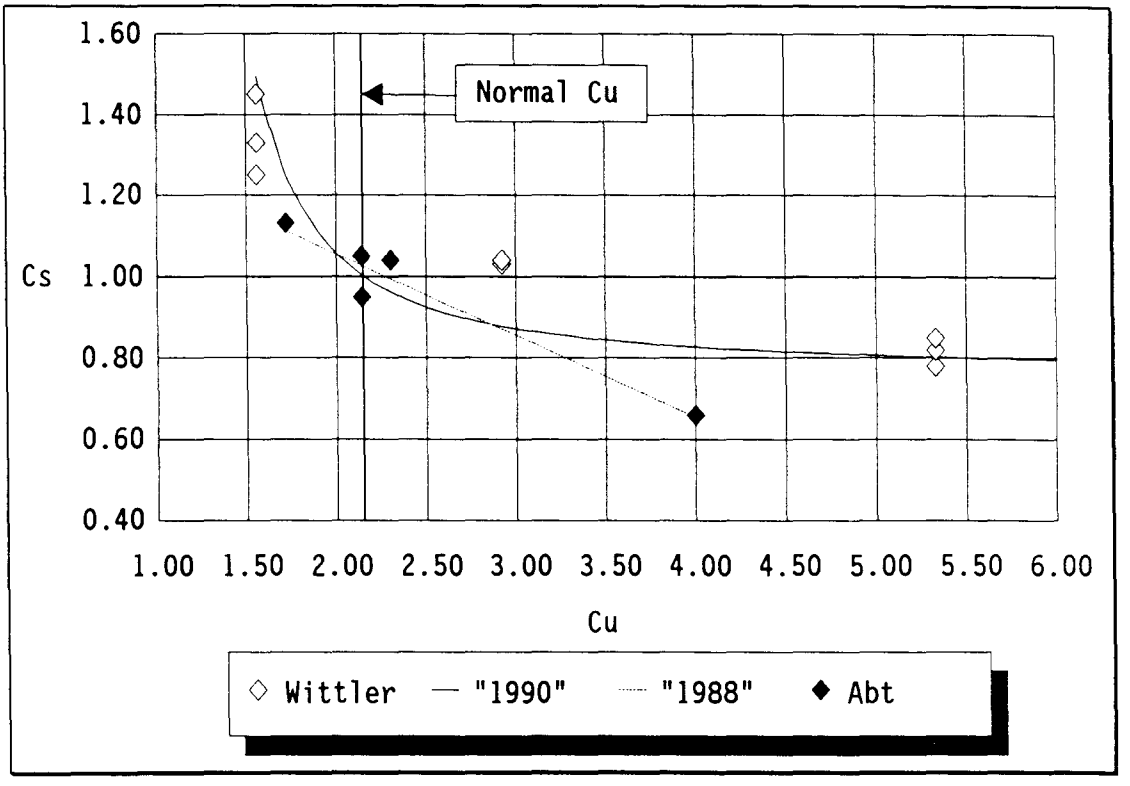

Figure 1. Coefficient of Stability, C_s, versus Coefficient of Uniformity, C_u.

At roughly thirty-three percent of the failure discharge, settlement and large scale movement ceases. Occasionally a single rock is dislodged and carried downstream. At this stage the load carrying capacity of the riprap layer is fully developed. Observations indicate that the riprap layer as a whole has become a structural unit. The shear stress of the flowing fluid is transferred through the layer to the soil layers beneath. The riprap layer is static until the flow reaches approximately seventy five percent of the failure discharge. At this flow the surface rocks tip up into the flow. Abt et. al. describe this stage as the movement stage. There is some localized dislodging of the surface rocks and sometimes channels will form at this stage. Failure is initiated in the majority of cases by movement of several key rocks. Rocks that support a local matrix of rocks are called keystones. When a keystone moves, a number of surface rocks supported by the keystone move and mass movement of the surrounding surface rocks follows. Failure of the entire riprap layer occurs when the flow is concentrated in the spaces the surface rocks have voided and the flow

erodes through the lower layers of rock. The other type of failure is characterized by a large area of the surface rocks becoming mobile and piling up a short distance downstream. As in the case of well graded riprap, flow is concentrated in a weak area and failure follows.

The fluid shear stress is transferred to the surface rocks and through the riprap layer by the individual rocks. The gradation of the mixture influences the efficiency of this transfer. Well graded riprap transfers the load through large, medium and small particles. Because of the large number of particles in well graded riprap, there are many transfer points . The smaller particles fill in the voids between the larger particles. The load paths do not intersect the centers of the majority of the particles. Instead, the larger particles tend to transfer the load to the smaller particles tangentially. The small particles act like ball bearings. The bearing stress on the small particles is great enough to induce sliding between neighboring particles and instability is the result.

Poorly graded riprap has fewer transfer points than well graded riprap. The bearing stress is uniform throughout the layer because the particles are similar in size. Loads are transferred through the centers of the particles rather than tangentially. Since the bearing stress is transferred much more efficiently by poorly graded riprap, the overall stability is greater than well graded riprap.

Different failure mechanisms for poorly graded and well graded riprap have been observed. Poorly graded riprap withstands substantially larger flows, all other factors being equal, than well graded riprap, as shown in Figure 1. The failure of poorly graded riprap is much more sudden than well graded riprap. Well graded riprap tends to fill in voids in the post movement stage with small particles washed from above. This process is called healing. Several instances of large rocks being swept away only to have their spaces filled immediately by many smaller rocks from just upstream have been observed. After the movement stage, there is very little incidental movement in poorly graded riprap. The failure of poorly graded riprap begins with many surface rocks moving very suddenly. Within moments the entire riprap layer becomes mobile. Very little healing was observed in the poorly graded riprap tests. The suddenness of failure and the healing process both moderate in medium graded riprap mixtures.

The aftermath of a keystone failure is a deep longitudinal scour in the embankment. The flow, concentrated in a narrow reach of the embankment, scours down thru the riprap layer into the soil below. In many instances the scour is at a slight angle to the direction of flow. As the scour deepens, a greater percentage of the flow is captured and the unit discharge in the scour trench increases rapidly. The increase in unit discharge hastens the scour process. The shape of the scoured area is similar to a narrow "u" that opens downstream. The banks of the "u" are steep, often approaching ninety degrees. The aftermath of a mobile bed failure is different than the keystone failure. Large areas of the riprap layer are removed during a mobile bed failure. Mobile bed failures are characteristic of uniformly graded riprap and thus withstand substantially greater flows at failure. The particle transport capacity of the increased flow causes a large initial and sustained transport of the riprap layer.

Similar failure stages for riprap on side slopes with differing gradations have been identified by Ahmed (1989). Ahmed studied filter and gradation effects on side slope riprap stability. His observations included the movement of particles at relatively low flow rates to secondary and more stable positions. He also noted the vibration of the particles. Ahmed describes the threshold stage as the point when one particle is moved exposing some shielded particles. The failure stage followed the threshold stage and was characterized by a number of surface rocks moving and endangering the side slope stability. Ahmeds conclusions for side slopes match the conclusions

reached for embankment slopes. He identified an eight percent decrease in failure discharge for the well graded riprap ($C_u \sim 2.2$) as opposed to the poorly graded riprap ($C_u \sim 1.10$).

Conclusions

The stability of riprap protection on embankments and side slopes is significantly influenced by the gradation of the rock mixture. In general, poorly graded riprap demonstrates increased stability while well graded riprap is less stable than normal gradations for overtopping flows. Well graded riprap fails over a period of time as voids are filled with eroded material from upstream. The filling process is called healing. Failure of poorly graded riprap occurs very suddenly with very little healing. The designer should consider the ramifications of gradation specifications both in design and quality control during construction.

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Momentum and Kinetic Energy Coefficient Research - Ramp Flumes

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Abstract

In October 1984, Reclamation Hydraulic Branch staff members investigated the operation of the A-Canal ramp flume at Klamath Falls, Oregon. The ramp flume was completed in 1983 as a water measurement device. Extensive data were collected at the field site: (1) velocities (vertical profiles) across the channel at the entrance and throat and (2) water surface profiles through the ramp flume at varying discharges.

Velocity data were used to compute values of alpha (kinetic energy coefficient) and beta (momentum coefficient). Alpha values were used to analyze a ramp flume computer program that predicts head-discharge relationships in ramp flumes. The computer code uses an approximation of the kinetic energy coefficient to make adjustments in the water surface elevations calculated with an energy balance. Comparisons of the predicted head-discharge relationships determined by the computer program and the actual values computed from the Klamath Falls ramp flume data are made.

Introduction

The A-Canal is one of the principal supply facilities of the U.S. Bureau of Reclamation's Klamath Project, which provides irrigation water to over 200,000 acres of agricultural land in Klamath County, Oregon and Medoc and Siskiyou Counties, California. The A-Canal extends for miles from the southern end of Upper Klamath Lake (Link River Dam) in a southeastern direction through the City of Klamath Falls to the agricultural areas of the county (figure 1). The first 2,225 feet of the canal consists of a concrete lined section with a 13.5 foot bottom width, side slopes of 1/2 to 1 and a depth of 12 feet. The longitudinal slope is 0.0003674. Although the canal was historically designed for a maximum discharge of 1,500 ft^3/s, presently the maximum discharge is 1,050 ft^3/s due to concrete deteoriation.

In March 1982, Reclamation's Mid-Pacific Regional Office proposed a water measurement structure on the A-Canal. Two flume designs were considered: a Parshall flume and a ramp flume. The estimated cost of the ramp flume was approximately 40 percent of the cost of the Parshall flume and was selected as the water measurement device. The design parameters were developed in Reclamation's Denver Office Hydraulics Branch and the Mid-Pacific Regional Office developed the detailed construction design.

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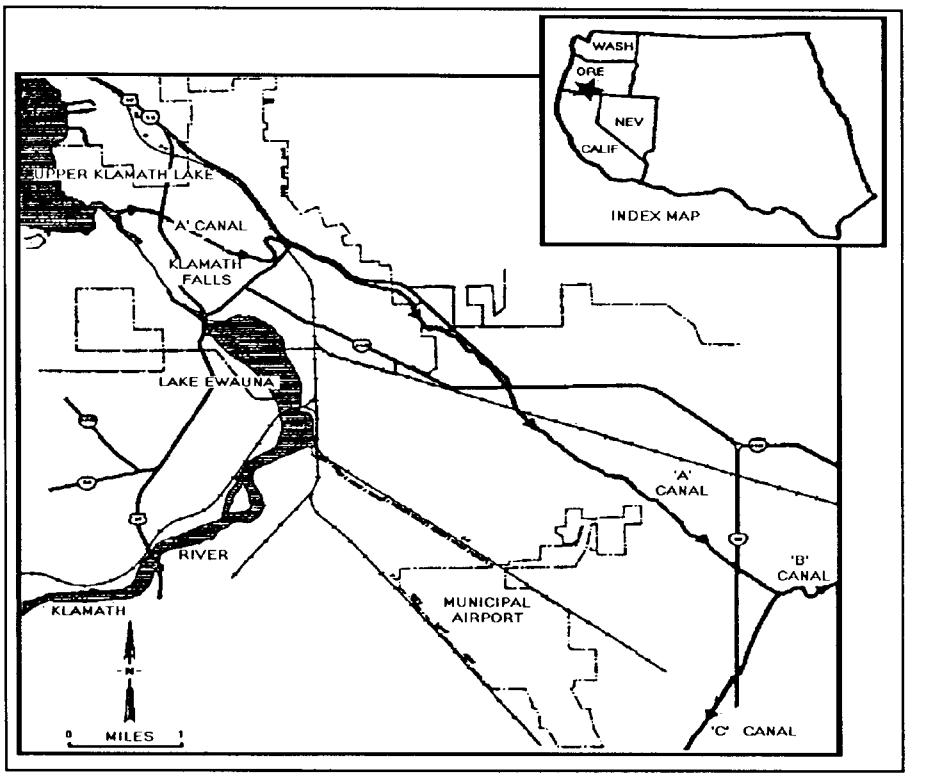

Figure 1 - Location of the A-Canal

Most critical flow devices are originally designed by laboratory calibration for accurate measurements because discharge is not theoretically predictable except through empirically derived coefficients. Ramp flumes are one type of critical flow water measurement device that does not require laboratory calibration. As long as critical flow occurs in the throat of the ramp flume, a rating table can be calculated with an error of less than 3 percent. A computer model has been developed for estimating flow through long-throated measuring flumes including ramp flumes (Clemmens et al, 1987; Replogle, 1974). The A-Canal ramp flume was designed with a maximum free flow discharge of 1,050 ft^3/s with an upstream depth of 13.35 feet and a head loss of 0.88 foot.

In October 1984, personnel from Reclamation's Hydraulics Branch collected velocity and water surface profile data at the A-Canal ramp flume. Velocity data were used for computing alpha (α) and beta (ß) values. The computed alpha values and water depths were used to evaluate the predicted head-discharge relationship of the computer model.

Theoretical Considerations

When velocities in open channel flow are nonuniform the velocity head is generally greater than the value computed using the mean velocity. When the energy principle is applied, the velocity head may be expressed as $\alpha V^2/2g$ (V= mean velocity, g=accleration due to gravity). Alpha (α) is known as the energy coefficient to correct the velocity head for nonuniform velocity distributions. Values for α vary between 1.03 and 1.36 (Chow, 1959). The nonuniform distribution of velocities also affects the computation of momentum. The momentum of the fluid passing through a channel section per unit time

is expressed as $\beta\gamma QV/g$ (γ = unit weight, Q = discharge). Beta (β) is known as the momentum coefficient or Boussinesq coefficient. The value of β varies between 1.01 and and 1.12 for straight prismatic channels (Chow, 1959).

The kinetic energy coefficient can be calculated from the following equation (Chow, 1959):

$$\alpha = \frac{\int v^3 dA}{V^3 A} \approx \frac{\Sigma v^3 \Delta A}{V^3 A} \qquad (1)$$

where v = point velocity
A = area

The momentum coefficient is computed from the following expression (Chow, 1959):

$$\beta = \frac{\int v^2 dA}{V^2 A} \approx \frac{\Sigma v^2 \Delta A}{V^2 A} \qquad (2)$$

The energy and momentum coefficients can be approximated by the following equations (Chow, 1959):

$$\alpha = 1+3\varepsilon^2-2\varepsilon^3 \qquad (3)$$

$$\beta = 1+\varepsilon^2 \qquad (4)$$

where

$$\varepsilon = \frac{v_{max} - 1}{V} \qquad (5)$$

v_{max} = maximum velocity
V = mean velocity

Replogle (1974) uses the energy coefficient, α, to adjust the energy computations using the average velocity in the section instead of the point velocities. For flows where the throat length of the flume is only one or two times the depth, α equals 1.0. At low flows where the throat length is 10 to 30 times the depth, a fully developed boundary layer is present and α probably equals 1.04. Replogle uses a value of α in the canal approach of 1.04, assuming flow is fully developed.

Replogle (1974) uses an equation based on Chow's approximation of α (equation 3) to calculate α in the throat section. His equation takes into account the shape factor (D/R, depth to hydraulic radius) and development of the boundary (L/R, crest length to hydraulic radius). The calculation of α in the throat is based on the following expression:

$$\alpha = 1+[3E^2-2E^3][1.5D_3/R_3-0.5][0.025L_3/R_3-0.05] \qquad (6)$$

E is based on the equation $E= 1.776C_f^{0.5}$, where C_f is the coefficient of total skin friction. The two factors that contain D/R and L/R are factors that account for channel shape and development of the boundary. In the flume throat, α varies from 1 to 1.85 as D/R ratio in the throat varies from 1 to 1.57. An increase in α will occur as L/R increases. This basically supports the assumption that fully develped flow will exist when the flume length exceeds 40R. As the length to hydraulic radius ratio increases, the deviation of α from unity will increase. When the the L/R ratio equals 2 or less, α will be close to 1.

Field Test

The field test was conducted by a hydrographer and engineer from Reclamation's Mid-Pacific office and three members of Reclamation's Hydraulic Laboratory in Denver. A schematic of the ramp flume is shown in Figure 2. Velocity measurements were taken at station 16+42 and station 16+77.91 for disharges of 286 ft^3/s, 586 ft^3/s, 906 ft^3/s and 1046 ft^3/s. Velocity measurements were taken at a 1-foot increments (throughout the depth) every 1.5 feet across the channel. Water surface elevations were taken at stations from the canal through the ramp flume to obtain water surface profiles (Figure 3).

The propellar size prevented the measurement of velocities near the boundary. Additional data were obtained by fitting a logarithmic velocity curve to the measured data based on the universal velocity law developed by Prandtl. A computer program was then written to calculate alpha and beta based on equations 1 and 2.

Results and Discussion

Alpha values varied from 1.067 to 1.099 in the canal. At the maximum discharge of 1,046 ft^3/s, alpha was 1.075. In the throat of the ramp flume, the variation in alpha was minimal, averaging 1.01. Beta values averaged 1.027 in the canal and 1.002 in the throat.

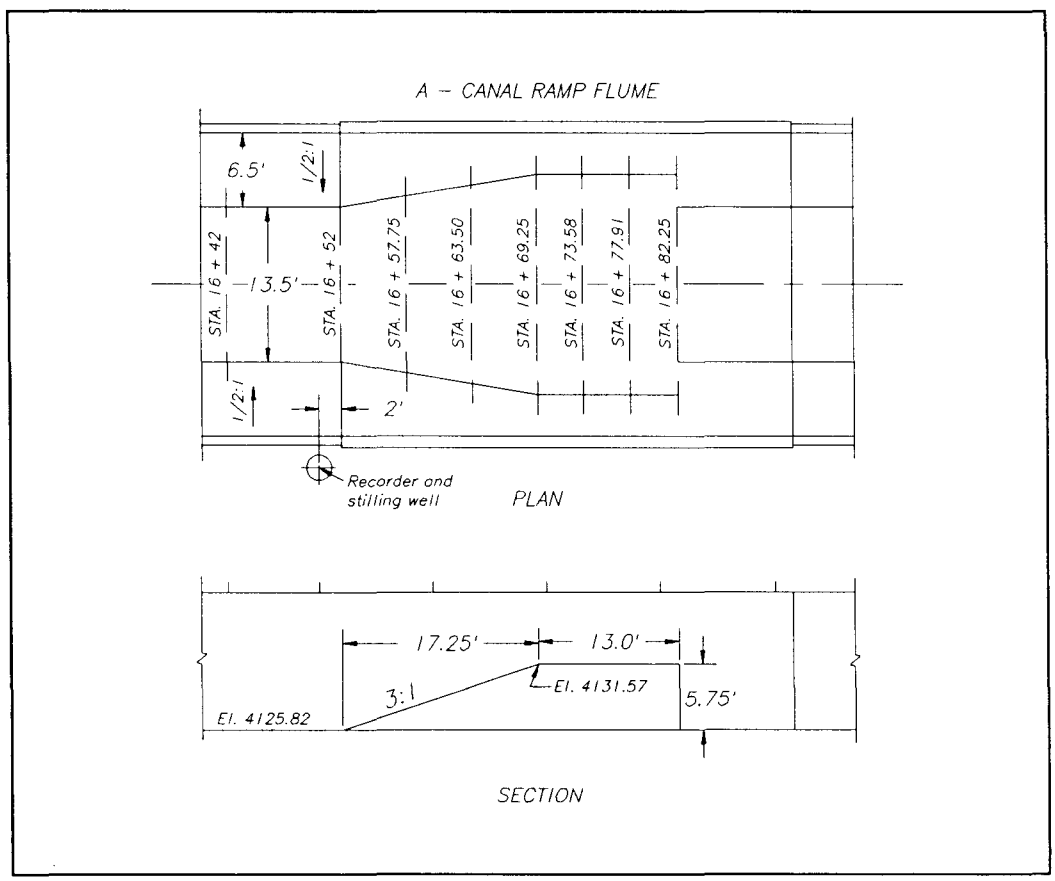

Figure 2 - A-Canal Plan and Section

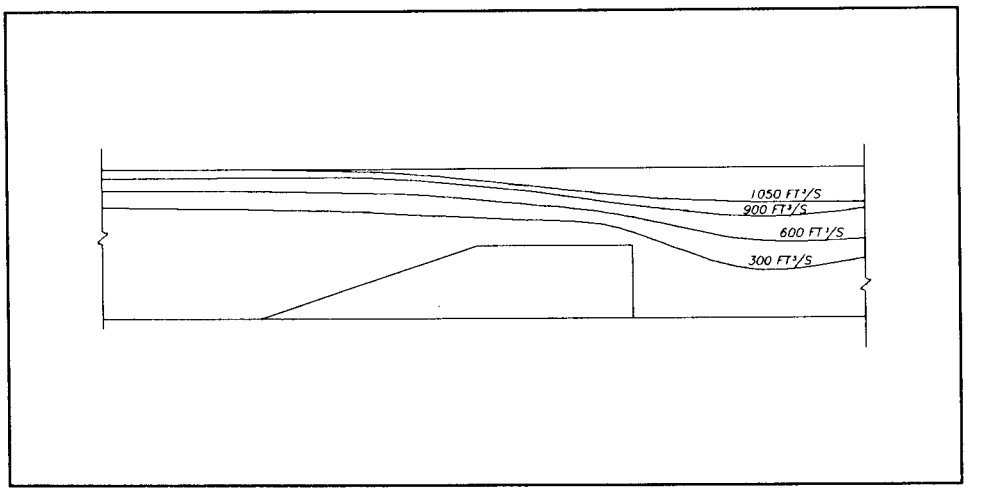

Figure 3 - Water surface profiles through A-Canal ramp flume.

Watts et al. (1967) conducted a study on alpha and beta values in the Poudre Supply Canal and the Horsetooth Feeder Canal as part of the Colorado Big Thompson Project. Velocity data were collected in seven bends of the canal. Bends 1-3 had a channel bottom width of 7 ft., and bends 4-7 had a bottom width of 12 ft. All sections were trapezoidal with a side slope of 1-1/4 to 1. Velocities were taken at 0.5-ft depths every 1.5 to 3.0 feet across the channel. Velocity data were measured with Price current meters at four sections through each bend at discharges ranging between 200 and 1,200 ft^3/s.

The results of the Watts study showed a variation in alpha between 1.026 and 1.084, averaging 1.04. Beta varied between 1.013 and 1.029 averaging 1.02. The results of Watts and the A-Canal ramp flume data are in good agreement. The A-Canal alpha values measured at the recorder (station 16+50, Fig. 2) varied from 1.068 to 1.098, within the range of the Watts study. The alpha values for the A-Canal at station 16+50 and the Watts data are greater than the alpha values in the throat because the flow accelerates through the ramp flume.

Comparison of the alpha values from the A-Canal, Poudre and Horsetooth canals with published data in Chow (1959) and Keulegan (1942) shows that the A-Canal, Poudre and Horsetooth canal alpha data are less. Chow states that alpha values should average 1.15 in straight regular channel; Keulegan predicts that alpha will average 1.10 in straight channels. The difference between the A-Canal, Poudre and Horsetooth Canal alpha values and alpha values shown in Chow and Keulegan is due to the selection of the velocity at the boundary. Chow and Keulegan did not use a semilog-velocity distribution near the wall. If the measured velocity and the zero velocity are averaged, then the alpha value for Watts' data would have been 1.15 instead of 1.04. Both the A-Canal and Watts data included a semilogarithmic velocity extrapolation at the boundary.

Replogle uses an average value of 1.04 for alpha in the approach channel based on Watts study. The A-Canal data supports a value of alpha in this range. Replogle and Clemmens compute an approximation of alpha (equation 6) based on a variation of Chow (equation 3). Clemmens computer program FLUME (1987) was run for two different cases based on the A-Canal data. The first computer run utilized the A-Canal ramp flume data and Replogle's approximation of alpha in the throat. The second computer run used alpha generated from the measured A-Canal velocity data. Both computer runs were compared to measured data (Figure 4). The variations in alpha do not significantly affect the prediction of head-discharge in the flume. The computer generated head-discharge relationships are in very good agreement with the measured head-discharge relationships obtained during the field trip.

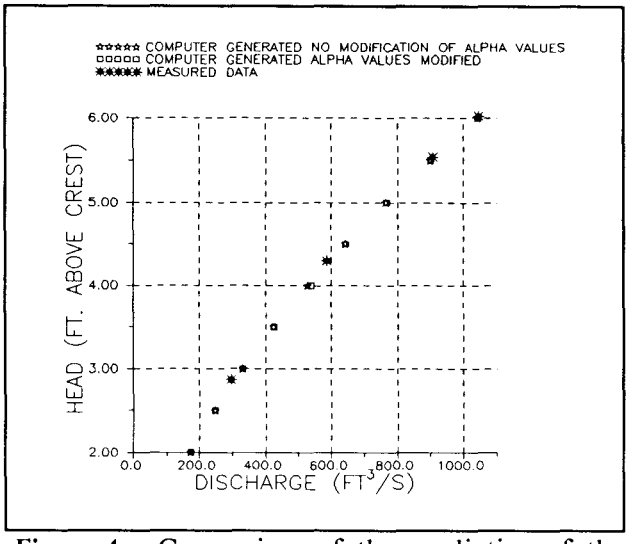

Figure 4 - Comparison of the prediction of the head-discharge relationship based on varying alpha in the throat for the A-Canal Ramp Flume

Conclusions

1. Alpha values determined from the A-Canal data are within the range of data collected in the Watts study. Both studies show the need to compute an approximation of the logarithmic velocity distribution at the boundary in order to prevent errors in alpha values of 10 percent or more.

2. The approximations of alpha to adjust average velocities in the computer program FLUME provide a good prediction of head-discharge in the A-Canal ramp flume. Variations from the measured data are less than 1 percent.

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OVERTOPPING PROTECTION FOR EMBANKMENT DAMS

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Chuck Henning[4]

Abstract

Modifications that allow safe overtopping during large flood events can be made to existing embankment dams. These modifications are on the leading edge of technology and require special investigations. This paper will describe the investigations under way by the Bureau of Reclamation for the overtopping protection at Arthur R. Bowman Dam.

Project Description

A.R. Bowman Dam (formerly Prineville Dam) is an earthfill structure on the Crooked River about 20 miles (32 km) upstream of Prineville, Oregon (fig. 1). The dam has a structural height of 245 feet (75 m), a hydraulic height of 159 feet (48 m), a crest length of 800 feet (244 m), and a crest width of 35 feet (11 m). The existing spillway has a 20-foot (6-m) long free overflow crest with a concrete-lined chute and stilling basin. The spillway design capacity is 8,120 ft^3/s (230 m^3/s). The outlet works is a concrete-lined tunnel controlled by two 6- by 4-foot (1.83- by 1.22-m) high pressure gates. The outlet discharges into the spillway stilling basin with a design capacity of 3,300 ft^3/s (93 m^3/s).

Dam Safety Investigations

As part of Reclamation's ongoing Safety Evaluation of Existing Dams program, hydrologic and hydraulic analyses were made for A.R. Bowman Dam. The probable maximum flood (PMF) developed for this dam has a peak inflow of 268,000 ft^3/s (7589 m^3/s) and a 15-day volume of 1,034,000 acre-feet (1.28×10^9 m^3). Flood routing studies determined the full PMF would overtop the dam by 21 feet (6.4 m) with overtopping occurring for any flood greater than 23 percent of the PMF. Minimal overtopping will likely cause failure of the existing

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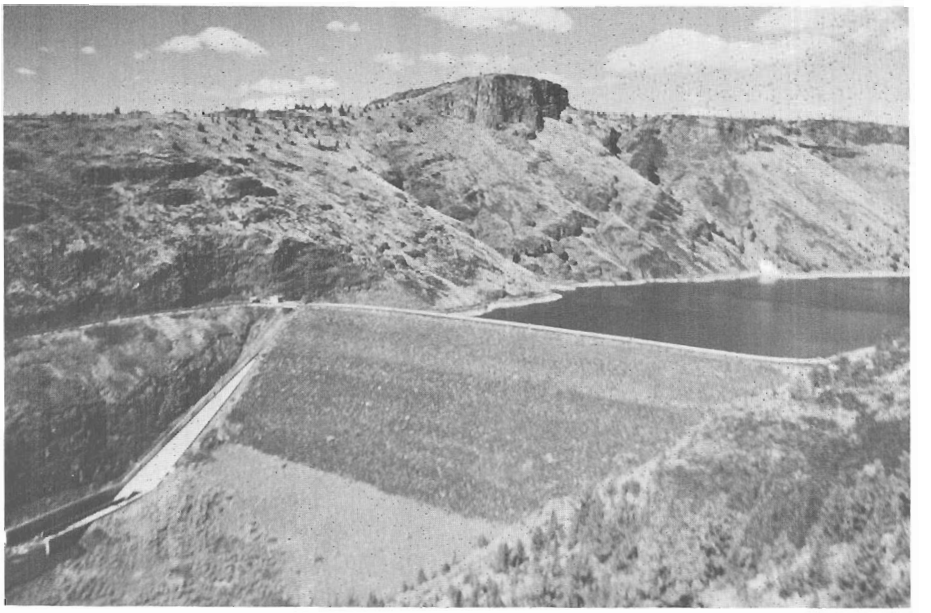

Figure 1. - A.R. Bowman Dam.

dam. Flood flows from this dam failure would inundate Prineville and other downstream habitations, and overtop and cause the failure of two downstream dams. Modifying A.R. Bowman Dam to mitigate these concerns is being accomplished under the Reclamation Safety of Dams program.

Corrective action studies were prepared for several structural modification alternatives including new spillways, raising the dam, and overtopping protection. Overtopping protection was considered the most economical alternative and is being pursued by Reclamation.

Four conceptual design alternatives to protect the dam embankment and foundation from erosion during overtopping are presently under development. These alternatives include one providing a reinforced rockfill facing and three utilizing concrete materials for the facing. These alternatives and associated cost estimates will be evaluated to determine the most appropriate method of overtopping protection to carry into final design.

Reinforced Rockfill Overtopping Protection Alternative

This alternative consists of placing a layer of steel-reinforced rockfill [2-foot (0.6-m) maximum diameter rock] on the existing downstream face and adjacent abutments of the dam. Protection for the dam toe would be provided by a roller-compacted concrete (RCC) apron. Reinforced-rockfill facings have been used on numerous dams and large cofferdams as an effective means of providing overtopping protection. However, applying this facing to an existing dam requires special design considerations.

Although reinforcing steel can be used to tie the rockfill together and prevent the erosion of individual rocks, the rockfill as a unit would be unstable on the 2:1 slope of the dam when saturated. The reinforcing steel must be tied back into a more stable, unsaturated portion of the embankment. The conceptual design provides the stable tie-back zone by the addition of crushed rockfill and sandy gravel (fig. 2). The lower permeability of the sand and gravel zone will act as a seepage barrier. This will allow the pervious underlying crushed rock to remain drained and provide the anchorage necessary to maintain the stability of the rockfill facing.

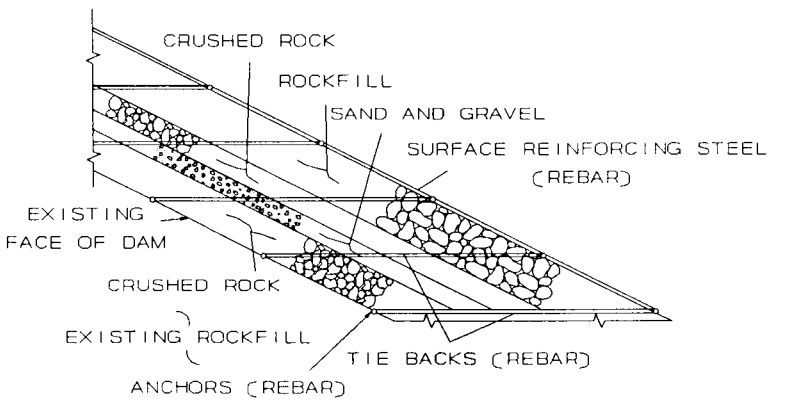

Figure 2. - Reinforced-rockfill protection (no scale).

Concrete Overtopping Protection Alternatives

The primary alternative being investigated for overtopping protection with concrete materials is placement of RCC in a stepped overlay on the downstream dam face (fig. 3). An RCC apron is planned to protect the toe. This alternative has been applied on other dams in this country, but not one as high as A.R. Bowman Dam or for such a large overtopping depth. The benefits of this alternative include simple construction procedures and energy dissipation along the downstream face which will reduce the size of the RCC apron.

The RCC stepped overlay must be designed to safely withstand uplift and hydrodynamic loads to remain stable and intact during all overtopping events. Temperature loads, shrinkage, freeze-thaw damage, and potential settlement of the underlying embankment must be evaluated. Cracking may occur during its lifetime so protective measures must be taken to prevent erosion of the underlying embankment material should overtopping flows enter these cracks. The gravel filter section (fig. 3) provides drainage behind the RCC to reduce the development of uplift loads. Hydraulic model studies are under way to provide data on this stepped overlay alternative.

Other concrete alternatives being considered for overtopping protection at A.R. Bowman Dam include a stepped surface on the downstream face using precast wedge-shaped blocks, and a reinforced

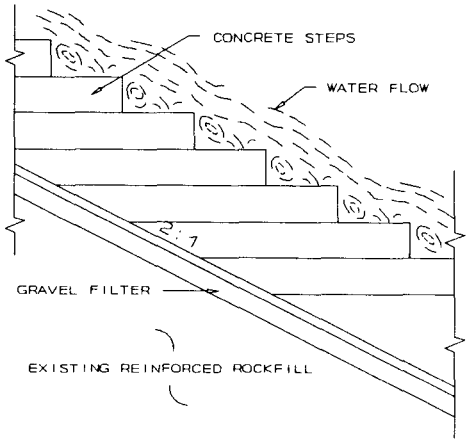

Figure 3. - Concrete overtopping protection (no scale).

concrete chute-type structure on the downstream face. The latter alternative requires a smaller volume of concrete material on the dam but requires a larger energy dissipation structure at the dam toe and more care in the chute design and construction.

Hydraulic Model Studies

Reclamation is currently conducting research on the hydraulic design of stepped spillways for high head structures. Research on embankment dam overlays will develop design criteria for optimizing step geometry for energy dissipation and venting embankment drainage. Enhancing embankment drainage will relieve uplift pressures, reduce drainage system requirements, and increase stability.

Test Facility

The present laboratory facility consists of a 1:12 scale, 2:1 sloping flume (variable to 4:1), 1.5 feet (0.5 m) wide, for studying overtopping of embankment dams up to 165 feet (50 m) high (fig. 4). The flume has the capability to study overtopping heads of 30 feet (9 m) and unit discharges of 500 ft^3/s-ft (46.5 m^3/s-m). The present step configuration is the "standard" 2-foot (0.6-m) step height with horizontal tread. A tailbox at the toe of the flume provides tailwater and returns flow to the main channel. Optimizing of the step geometry for A. R. Bowman's overtopping range of 5-21 feet (1.5-6.4 m) is being emphasized.

Initial Data

Pressure data on the step faces have been collected to determine locations for providing embankment drainage. Pressures were measured on both the vertical and horizontal faces of the steps at three measurement stations, upper, middle, and lower, along the slope. At each station, two steps are instrumented, each with 11

Figure 4. - Overall view of overtopping embankment flume facility, H = 5 ft (1.5 m), q = 33.96 ft^2/s (3.15 m^2/s).

piezometer taps, for 66 total. The average pressure for each piezometer tap was recorded with a computer data acquisition system and used to plot pressure profiles over the steps at each station.

The pressure profiles are plotted over two steps from each station for the appropriate overtopping head (fig. 5). Each piezometer tap location (Nos. 1-22) is indicated on the steps. The pressure profiles indicate the jet impact on the downstream end of the step tread and an area of lower pressure in the offset below the pitch line of the steps where an eddy forms from recirculation of the impacting jet. The approximate flow depth, which varied little down the stepped face, is also shown for comparison. The jet impact and eddy circulation provides energy dissipation (Houston, 1988) and the offset area below the step pitch line will provide an opportunity to locate embankment drains (Clopper, 1989).

Pressure profiles for heads of 5 to 21 feet (1.5 to 6.4 m) indicate reduced jet impact with each station down the slope for 5 feet (1.5 m) of overtopping. With 21 feet (6.4 m) of overtopping, an increase in jet impact is seen between the upper and middle stations but a decrease results between the middle and lower stations. These initial data show excellent promise for the application of overtopping protection for a high embankment dam.

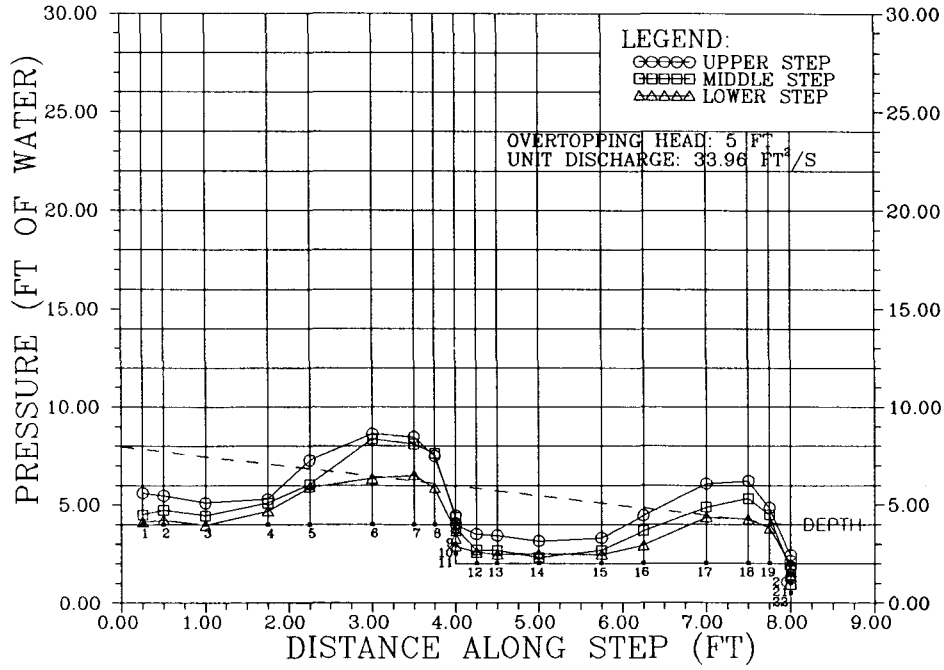

Figure 5. - Pressure profiles of stepped spillway face at three stations along slope (1 ft = 0.3048 m, 1 ft of water = 2.99 kPa, 1 ft^2/s = 0.093 m^2/s).

Future Investigations

A three-dimensional hydraulic model is under construction to determine effects of the converging walls down the dam face, with respect to flow conditions at the dam abutments, and energy dissipation or stilling basin requirements at the toe. Further research plans include performing near-prototype tests to study the effects of aeration, dynamic pressures, and embankment drainage on a large scale.

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Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

A free pamphlet is available from the Bureau entitled "Publications for Sale." It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-7923A, PO Box 25007, Denver Federal Center, Denver CO 80225-0007.