

RECORD OF PROCEEDINGS

O'Brien Ptg. & Stat. 90912

MINUTES OF THE PUEBLO CONSERVANCY DISTRICT SPECIAL MEETING ON JANUARY 19, 1987

DIRECTORS PRESENT: G. Sandstrom and P. Kelly

OTHERS PRESENT : R. Koester

ABSENT MEMBERS : K. Farley and L. Ward

OTHERS PRESENT : Bill Taggert and John Pflaum of
McLaughlin Water Engineers, Ltd.

The purpose of the meeting was to have a special report from the Engineers on the progress and problems of a leak in the grade control structure construction on Santa Fe.

McLaughlin gave a verbal explanation of their report letter of January 19, 1987, (copy attached), explaining terms and key points of concern of excessive flow through a portion of the sheet piling.

The recommended solutions to the condition, other than to do nothing, and be vulnerable to a rare flood where damage could occur, are as follows:

1. Employ pressure relief members (approximately 48 flap valves) downstream in the larger rock.
2. Filling upstream of the dam with clayey impervious material to cover the rock to reduce seepage under the dam.

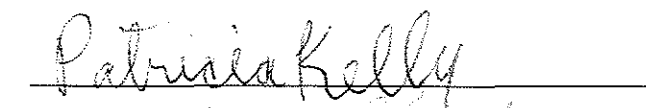
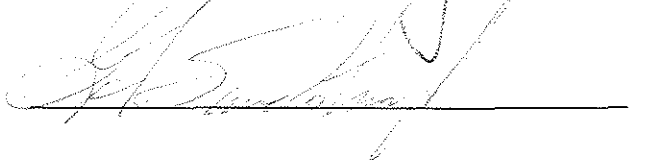
It was recommended that valves be installed, and if economical filling can be accomplished, both operations are to be done.

The Board indicated its desire to insure full protection of the dam with remedial construction work for the leak. How and who is to pay for construction was to be discussed on January 23, at 9:30 a.m., with January 30th at 8:30 a.m. as backup date.

There being no further business, the meeting adjourned at 10:45 A.M.


Secretary - Treasurer

APPROVED:



McLaughlin Water Engineers, Ltd.

January 19, 1987

2420 Alcott St., Denver, Colorado 80211 (303)458-5550

Mr. Raymond F. Koester
Engineering Administrator
The Pueblo Conservancy District
111 East Fifth Street
P. O. Box 234
Pueblo, Colorado 81002

RONALD C. McLAUGHLIN
HALFORD E. ERICKSON
DOUGLAS T. SOVERN
WILLIAM C. TAGGART
ROBERT L. CARLEY
WILLIAM R. KENDALL
CHARLES L. HARDT
RALPH L. TOREN

JOHN Y. McLANE
GENE A. BURRELL
MICHAEL E. MERCER
JOHN M. PFLAUM
MICHAEL R. GALUZZI
RONALD E. DAVORAK
SCOTT E. LEHMAN
KENNETH D. ZIEBART
MARY ANNE BLISH

HAROLD ROBERTS
JACK W. STEINMEYER
LEANDER L. URMY

Dear Mr. Koester:

This letter summarizes the activities, measurements and analyses we have performed on the Santa Fe Avenue Dam during past few weeks.

Initial Investigation

During the finishing stages of construction of the dam, our field inspector Mr. Greg Chol found a small amount of water leaking at an isolated point on the face of the dam. This was perceived at the time to be a possible cause for concern. Immediately we directed installation of piezometers near the location of the leak and at a few other locations on the face of the dam. The initial readings indicated higher piezometric head at the location of the leak compared to the piezometric head at other locations having the same elevation.

The construction records were reviewed and it was observed that sheet pile driving encountered problems in the area where the piezometric head was found higher and where the leak was occurring. In order to investigate the cause of the problem in further detail, we directed installation of piezometers along sections perpendicular to the longitudinal axis of the dam. The locations of all the piezometers installed are shown in Figure 1. The piezometer readings are summarized in Table 1. The piezometer readings were plotted along each section perpendicular to the axis of the dam and parallel to the dam axis. They are shown in Figures 2 through 4.

As can be seen from the piezometer readings, the readings have consistently gone down in all locations, eventhough the upstream water surface has varied from elevation 53.03 to elevation 55.00 (top of dam).

However, the readings have remained relatively high in the observed problem area as illustrated in Figure 4.

ASPEN
(303)925-1920

CASTLE ROCK
(303)688-5599

LAKE DILLON
(303)468-2141

TULSA
(918)664-2741

The consistent high piezometer readings at the trouble spot indicate that the opening through or under the upstream row of sheet piles is probably more than in the other areas of the dam. The excess flow here apparently travels laterally under the dam and discharges at the many weepholes on the downstream face. This explains the gradient away from the trouble area and could possibly indicate reduced capacity thru the weeps on the downstream row of sheet pile.

However, due to the coincidence of high piezometer readings in the area, where problems were encountered in driving the sheet piles (use of diesel hammer instead of vibratory hammer), the data implies the possibility that sheet piles might have buckled, a joint partially separated, a hole was cut (we are not aware of any), and/or the pile did not penetrate to bedrock in that area. There is always the possibility of boulders or cobbles obstructing the penetration of the sheet piles.

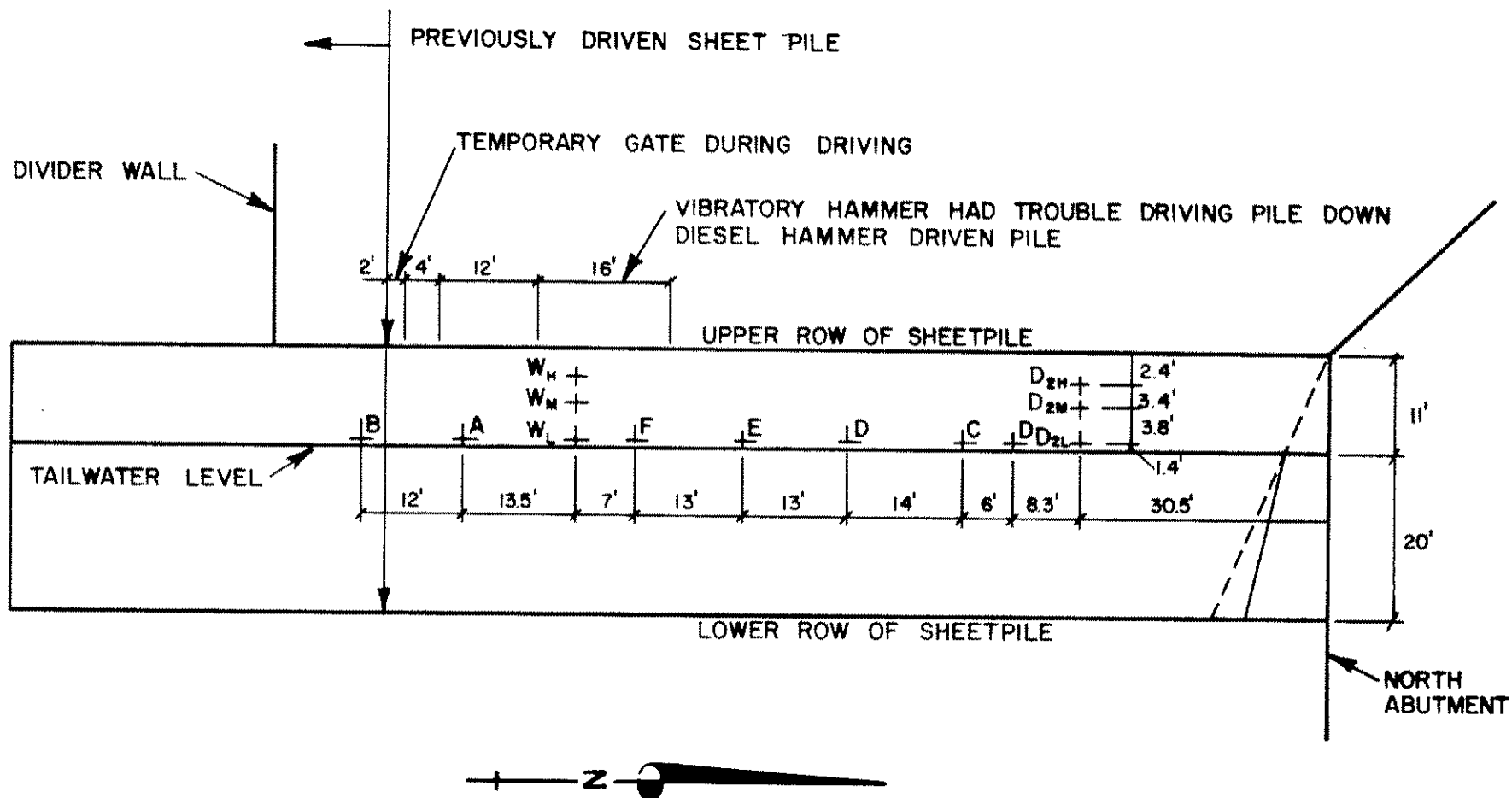


FIGURE 1
PIEZOMETER LOCATION
DRAWING



MWE

McLaughlin Water Engineers, Ltd.

TABLE 1

Project Title Aransas River Grade Control / Boat Launch Project No. 83-113.00C Date 1-16-87
 Subject Figure 5 Piezometer Readings Designed GRC Page 1

U/S WATER ELEV.'	TEST HOLE DATA WATER SURFACE ELEVATION													TIME DATE
	TEST HOLE	A	W _L	W _M	W _H	F	E	D	C	D ₁	D _{2L}	D _{2M}	D _{2H}	
	GROUT ELEV	49.46	49.46	51.88	53.71	49.46	49.46	49.46	49.46	49.46	49.46	51.88	53.71	
54 ¹²	—	—	50 ³¹	—	—	—	—	—	—	48 ⁷⁶	48 ⁷⁶	—	—	2:00 PM 12/23/86
53 ⁸³	49 ⁰⁴	49 ⁰⁴	49 ⁷²	50 ⁰⁵	50 ⁴²	—	—	—	—	48 ⁷⁶	48 ⁸⁸	48 ⁸⁸	NR	2:00 PM 12/24/86
55 ⁰⁰	49 ⁰⁴	49 ²⁵	49 ⁶⁶	49 ⁸³	50 ³⁰	49 ²⁵	49 ³⁰	49 ²¹	49 ¹²	49 ¹⁵	48 ⁷⁶	48 ⁸⁸	NR	10:00 A 12/19/86
55 ⁰⁰	48 ⁷⁶	48 ⁷⁶	49 ³⁴	49 ⁴⁸	NR	49 ¹⁰	49 ¹⁰	49 ⁰⁰	48 ⁸⁷	48 ⁷²	48 ⁷²	NR	NR	1:30 P 12/29/86
55 ⁰⁰	48 ⁷²	48 ⁷⁶	49 ³³	49 ⁴⁶	NR	49 ⁰⁸	48 ⁷⁶	48 ⁷⁶	48 ⁸³	48 ⁷⁵	48 ⁶²	NR	NR	3:00 P 12/29/86
55 ⁰⁰	48 ⁷²	48 ⁷⁶	49 ³¹	49 ³⁸	NR	49 ⁰⁸	48 ⁷⁶	48 ⁷⁶	48 ⁸³	48 ⁷⁵	48 ⁶²	NR	NR	4:00 PM 12/29/86
55 ⁰⁰	48 ⁷²	48 ⁸⁸	49 ²²	49 ³⁸	NR	48 ⁷⁶	48 ⁷¹	48 ⁸³	49 ⁷²	48 ⁶²	48 ⁶²	NR	NR	8:00 A 12/30/86
55 ⁰⁰	48 ⁷²	48 ⁸⁸	49 ²²	49 ³⁸	NR	48 ⁷⁶	48 ⁷¹	48 ⁸³	49 ⁷²	48 ⁶²	48 ⁶²	NR	NR	11:00 A 12/31/86
55 ⁰⁰ ²	48 ⁷¹	48 ⁷²	49 ⁰⁰	49 ¹³	NR	48 ⁸⁰	48 ⁷¹	48 ⁶²	48 ⁶²	48 ⁶²	48 ⁶²	NR	NR	12:00 N 1/6/87

NOTES:
 1. TAILWATER AT ELEV. 48⁷⁸ UNO
 2. TAILWATER AT ELEV. 48⁶⁶.

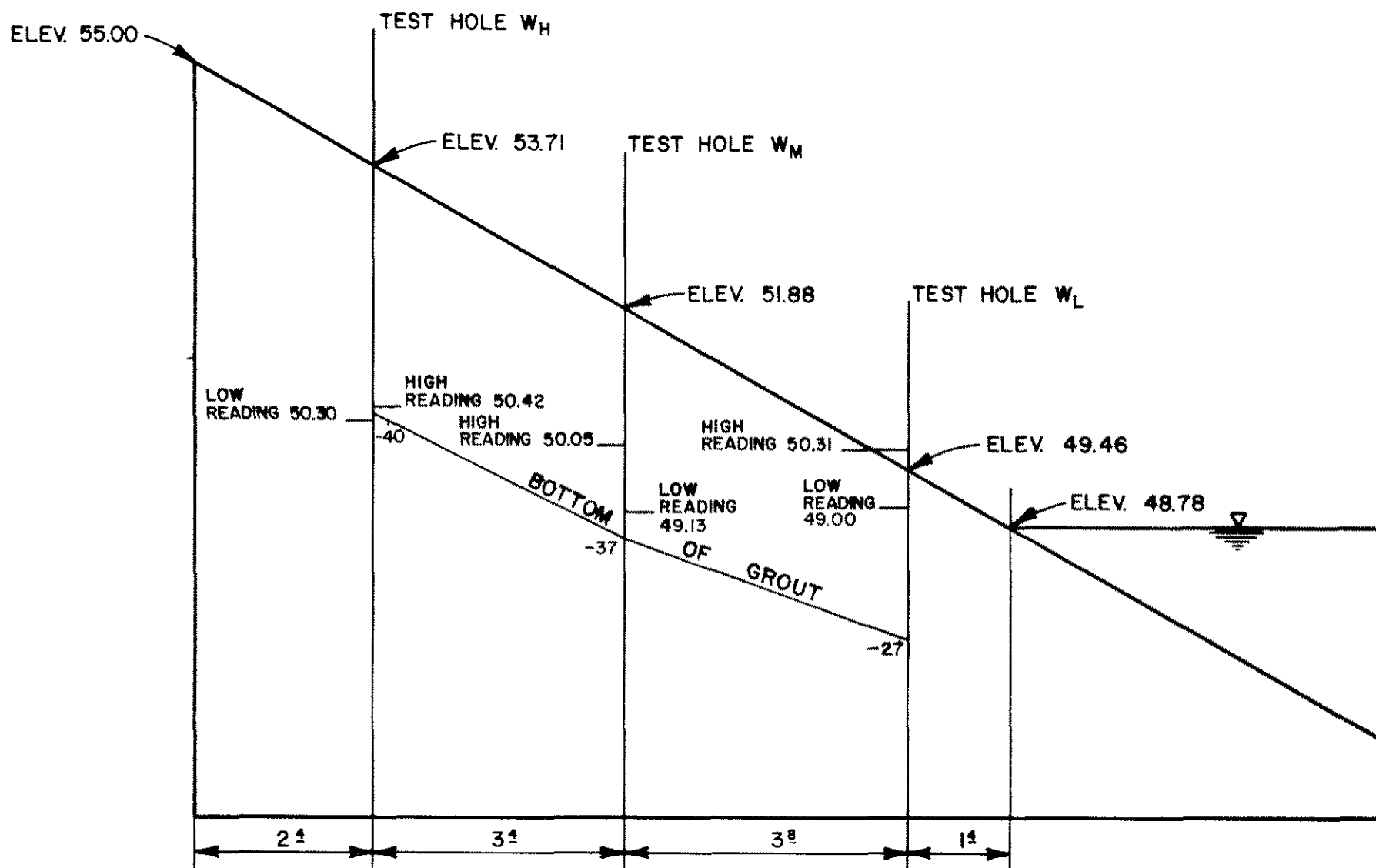


FIGURE 2
CROSS SECTION
TEST HOLES W

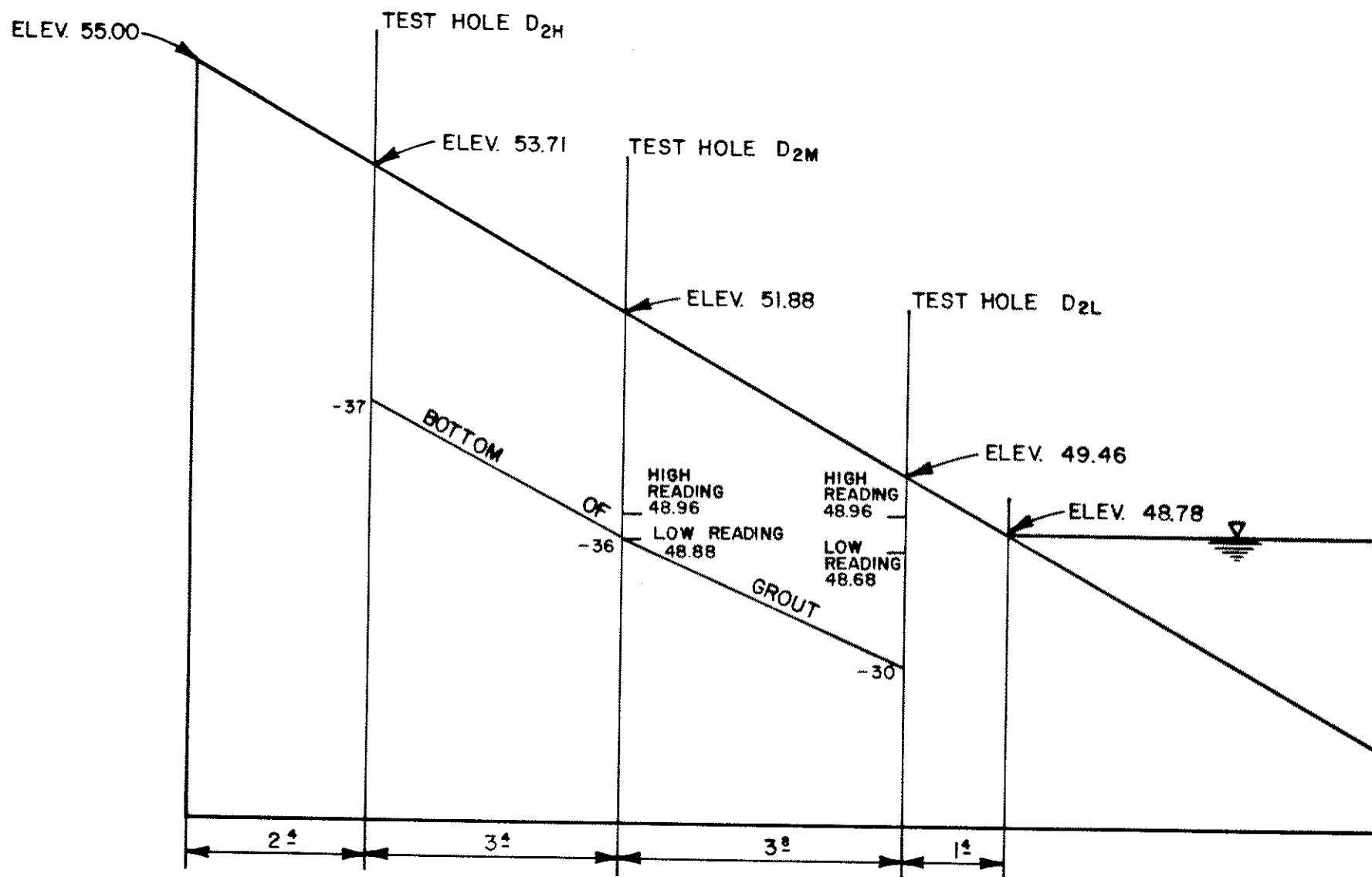


FIGURE 3
CROSS SECTION
TEST HOLES D₂

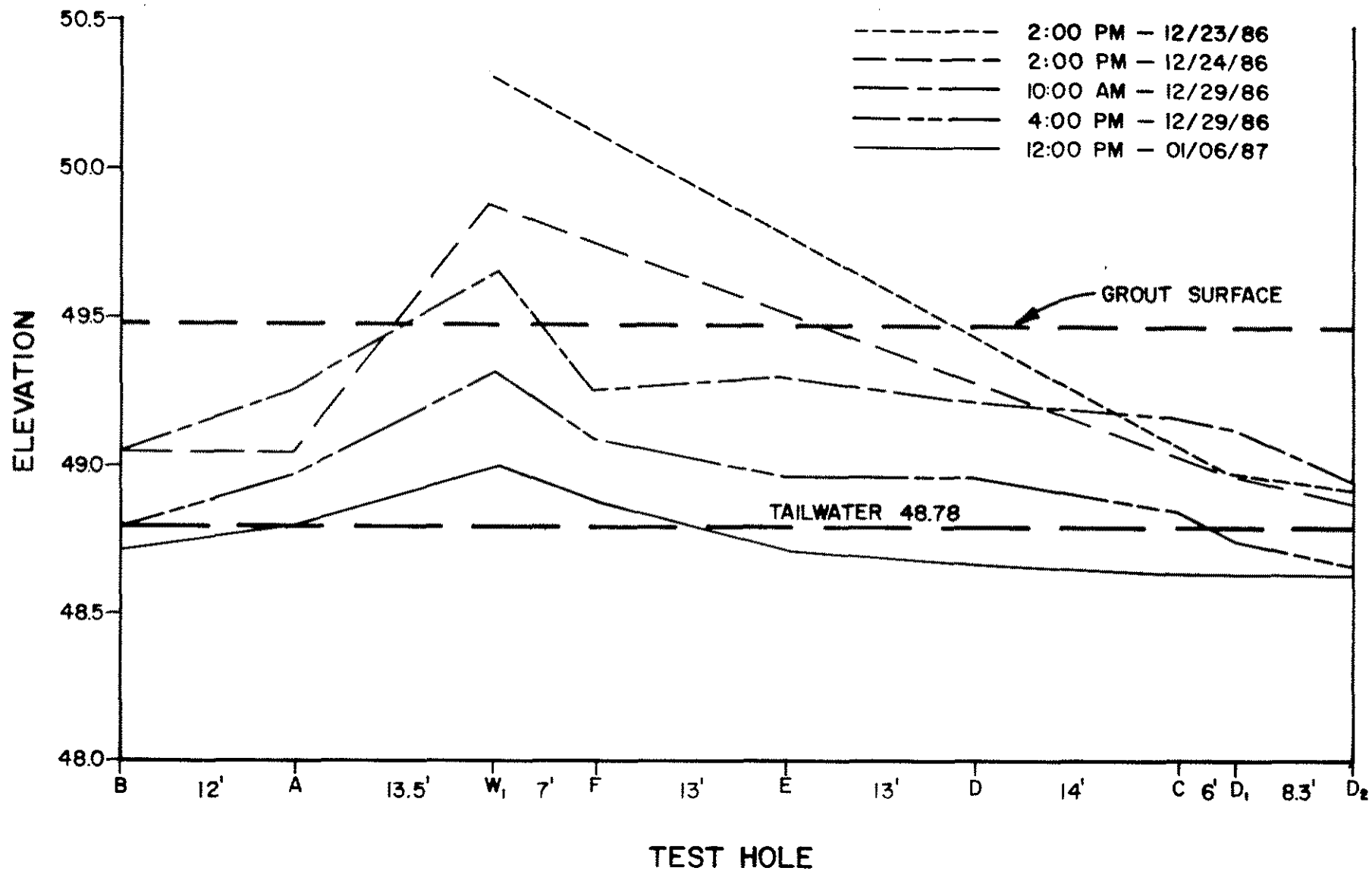


FIGURE 4
LATERAL INTERPRETATION
OF PIEZOMETER DATA

Analysis

This type of structure (grouted boulders) encounters uplift pressure as the critical force that leads to instability. Basically the inherent forces that lead to stability are the thickness of the grouted rock and its resultant mass, and the weight of the water flowing on top. Other forces or situations which can cause or lead to instability are the dynamic impact and shear forces of the flowing water, and the possibility of the flow separating from the face of the dam such that the weight of water is not on top of the slab and possible pressure fluctuations related to such phenomena.

Basically, the present situation is stable, although we do not like the higher pressures (or water levels) under this area. The design intent was what is occurring in the other sections. That is the water level under the dam is the same as downstream river level, indicating free drainage and total pressure relief. We would like to rectify the situation, but first it is appropriate to review other conditions.

Extrapolation of Field Data to Other Hydraulic Conditions

The concern of uplift pressures during other conditions was paramount. The existing data was analyzed by several methods, to see if reasonable parameters and phenomena could be assumed which gave similar results. Once reasonable phenomena and related parameters were determined, extrapolation could be made to other conditions.

This was done, and the following general phenomena were adopted. The flow through the soils under the dam was thought to be best described as flow through porous media, much like normal groundwater problems and seepage analysis under large earth dams. The rows of sheet pile were treated as a hydraulic flow limiting device such as a submerged orifice. This is really a complex phenomenon because we don't really know the open area thru the pile joints, possible defects and their location. Also the openings have some area filled by media (gravel, sand, etc.). Thus reference is made to flow through some equivalent open area. On the downstream row of pile we do know the area of the weep holes, but we don't know the exact open area of the joints. The sheet pile joint openings will vary because as it is driven the steel surfaces inside the joint have play and thus in some areas the joints may be totally closed or partially open.

late in the process some new computer software was found which utilize the same approach. Basically, we could assume reasonable parameters which would approximate field conditions. It clearly indicated more open area on the upstream row of sheet pile in the problem area.

However, the simulation assumes parameters for the open area at each pile, how much the opening is obstructed, what the permability of the soil is, and that the soil is homogenous to the shale bedrock. None of these variables have been scientifically isolated and determined, and practically they would be impossible to determine in this case. However there is a (large) range of permability that can be assumed and a fairly narrow range of reasonable flow gradients under the dam based on the field measurements. Also there is no reason to assume wide variation in soils characteristics or sheet pile openings on the downstream row of pile.

Therefore, we could assume a range in permabilities which would lead to various results in terms of the open area. Table 2 presents the key analysis data.

TABLE 2

**Assumed Hydraulic Parameters and
Results for Seepage Analysis of
Existing Situation**

	<u>At Trouble Section</u>	<u>At Section Near Dam (D2)</u>
Under flow (GPM per 10 feet of dam)	7	2.7
Assumed Permeability (ft/day)	100	265
<u>Upstream Sheet Pile Opening Data</u>		
Net Opening (sq. inches per 10 feet of dam)	0.31	0.09
Range of Gross Opening (sq. inches per 10 feet of dam)	1.0 to 10.0	.3 to 3.0
<u>Downstream Sheet Pile Opening Data</u>		
Net Opening (sq. inches per 10 feet of dam)	1.08	1.08
Range to Gross Opening (sq. inches per 10 feet of dam)	3.6 to 36	3.6 to 36

Basically, this qualitatively indicates the larger opening on the upstream row of piles at the trouble section. All numbers indicate range and proper relationship rather than a scientifically exact number. Put into further perspective, the weep opening in the downstream row of sheet pile was specified at the equivalent of 18.6 square inches per 10 feet of dam.

These parameters were then used for other hydraulic conditions, such as typical spring runoff and 100-year flood, with various channel conditions downstream.

The resulting pressure diagrams illustrate that troublesome uplift forces could occur as flood discharge increases, particularly near the 100-year flood. This situation is complicated by the potential separation of the flow over the dam.

This phenomenon deserves further explanation. As flow increases over a dam, and depending on the shape of the dam and the steepness of its slope, the flow can literally leap free, and project out away from the dam. The related conflict is that if the pressure underneath is increased, and the weight of water is removed above, the stability is worsened. Ultimately this jet, called the nappe, will contact the face of the dam or the pool downstream. The surface of the dam is intentionally rough and projects into this area, causing diversion of flow into the potentially open area and thus maintaining weight on top. Also the high tailwater elevations and the lack of an air source tend to pull the Nappe back onto the grouted rock surface. The problem is that the exact pressure on top from the overflow cannot be determined, and qualitatively the pressure underneath makes the problem worse.

Solution Options

We have evaluated the probable ranges of forces with our structural engineer. For present conditions and the lower range of flows there will be isolated locations of uplift forces (meaning that there is portion of the dam that experiences a net force up while the larger adjoining areas experience the desired situation of net forces down). These areas are small and within the probable structural capability of the dam.

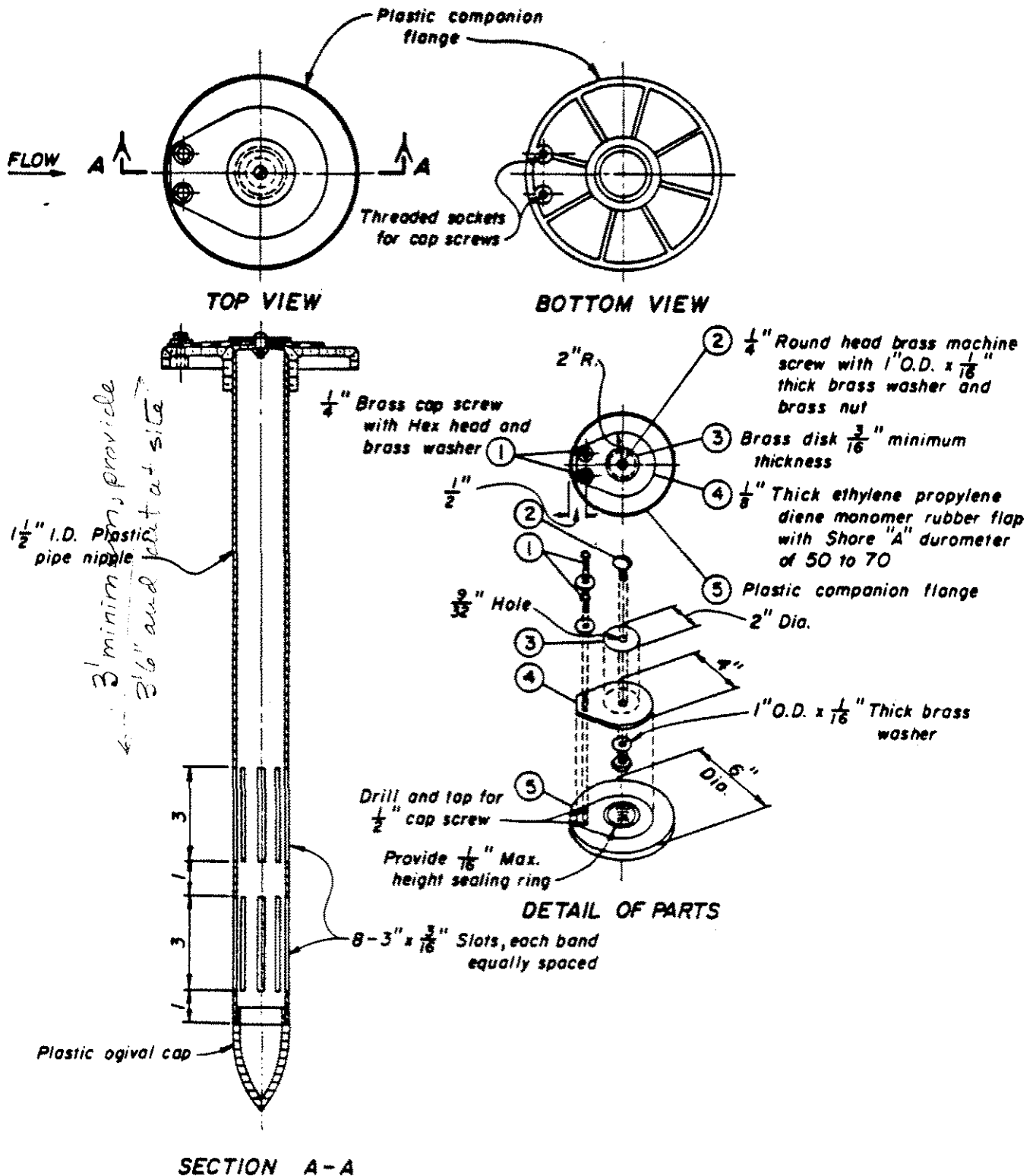
However, the condition for larger floods requires review. We believe, based on judgement and experience, that the possibility of the nappe lifting completely free is unlikely. Also, one of the inherent advantages of large grouted rock work is that cracking is anticipated, and these cracks provide a pressure relief.

Unfortunately, this is not a quantifiable parameter. One could risk doing nothing, but the risk is such that in a rare flood, damage could occur.

The two basic concepts which can be used, (probably together) are (1) pressure relief and (2) measures to insure that flow separation does not occur or is greatly reduced.

Pressure relief is the recommended minimum option. Several options were explored, but the Bureau of Reclamation has a flap weep valve which they use for relief of pressure under linings. We visited several people at the Bureau, reviewed several technical papers, and interviewed the likely manufacturer. A copy of their diagram is attached. These should adequately relieve the pressure. They will need to be installed at intervals horizontally (to relieve inflow all along the dam) and vertically because the desired relief location varies with different tailwater and nappe locations. The flap is necessary to prevent the introduction of flow into the subgrade for lower flow levels (the water could enter a given weep on the top of the dam and then could pressurize the subgrade). Maintenance would be required, but the Bureau's experience has been minimal. We would expect more maintenance because of the potential for vandalism and the rugged location.

The construction is simple, requiring drilling of a hole, placing the drain, gluing the cover flange and grouting in place. The manufacturer quoted \$22 per unit and suggests an installation cost of \$5 to \$20 per unit. The thicker grout layer and difficulty in getting the drain to go into the subgrade soil makes us think costs could be higher (on the order of 75 to \$100 apiece). Generally we recommend installation of two rows at five foot spacing at elevation 52 (plus or minus one foot) and elevation 50 (plus or minus one foot). They should be installed downstream of the larger rock where there is better protection. This would require 48 units or a budget of \$3,600 to \$4,800. These drains would both help to rectify the problem area and generally be an improvement of the dam.



FLAP VALVE WEEP

Figure 14

The second concept includes filling upstream of the dam with clayey impervious material which would be partially covered with rock (see Figure). This would reduce seepage, thus uplift pressure under the dam. And it would lower the potential for flow separation over the dam, theoretically reducing the profile of the nappe so that it has to contact most of the rock on the dam. This is estimated to cost \$8,000, and will also help to rectify the problem and generally be an improvement to the dam.

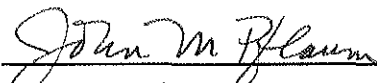
We are available to discuss these options with you.

Sincerely,

McLaughlin Water Engineers, Ltd.

A handwritten signature in cursive script, reading "William C. Taggart", written over a horizontal line.

William C. Taggart, P.E.

A handwritten signature in cursive script, reading "John M. Pflaum", written over a horizontal line.

John M. Pflaum, P.E.

WCT:nh

Current

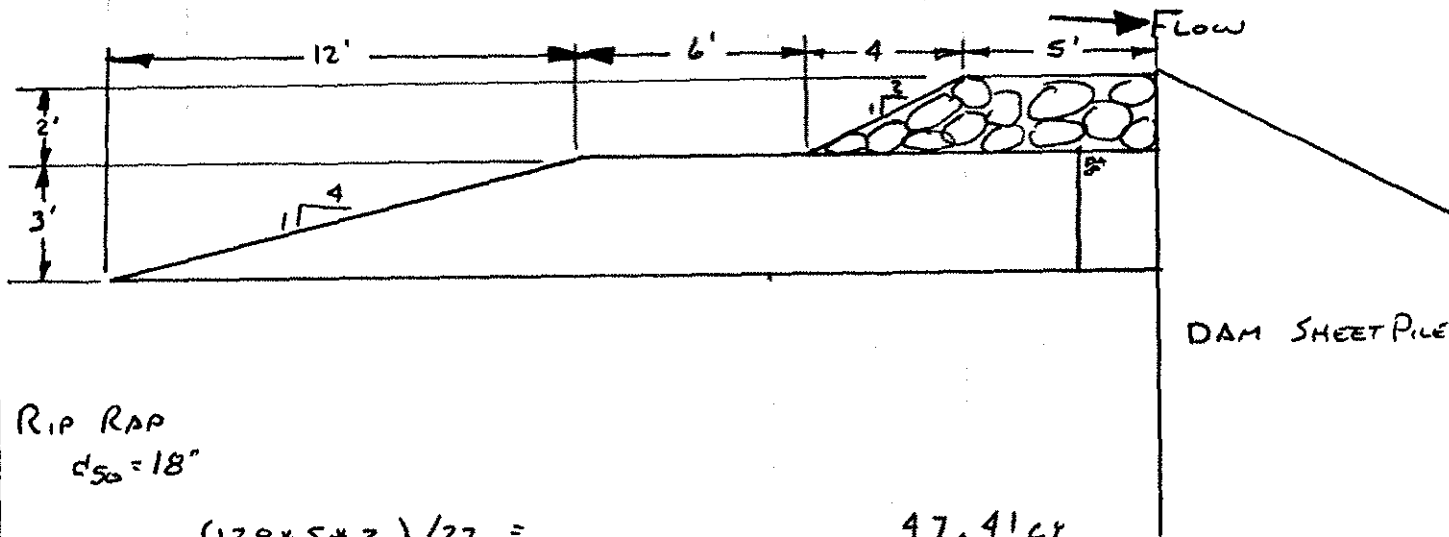


MWE

McLaughlin Water Engineers, Ltd.

Project Title ARKANSAS RIVER GRADE CONTROL & BOAT LIFT Project No. 83-113.000 Date 1-16-87

Subject COST ESTIMATE FOR CLAY & RIP-RAP Designed GRC Page 1/1



RIP RAP

$d_{50} = 18"$

$$(128 \times 5 \times 2) / 27 =$$

$$47.41 \text{ cy}$$

$$(128 \times 4 \times 2 \times \frac{1}{2}) / 27 =$$

$$18.96 \text{ cy}$$

$$(5 \times 5 \times 2 \times \frac{1}{2}) / 27 =$$

$$.46 \text{ cy}$$

$$((9+5)/2 \times 4 \times \frac{1}{2}) / 27 =$$

$$+ .52 \text{ cy}$$

$$63.35 \times 1.7 = 107.76 \text{ TON}$$

$$\$21.20 \times 107.76 = \$2360$$

CLAY FILL

$$(128 \times 15 \times 3) / 27 =$$

$$155.68 \text{ cy}$$

$$(128 \times 12 \times 3 \times \frac{1}{2}) / 27 =$$

$$85.33 \text{ cy}$$

$$(15 \times 15 \times 3 \times \frac{1}{2}) / 27 =$$

$$12.50 \text{ cy}$$

$$(15 \times 12 \times 3 \times \frac{1}{2}) / 27 =$$

$$10.00 \text{ cy}$$

$$263.51 \text{ cy}$$

$$\$5/\text{cy} \times 263.51 = \$1,316$$

DEWATER

\$4000

USING UNIT PRICES FROM BID.

RIP RAP

2,360

CLAY FILL

1,316

\$7,676

USE \$8000



MWE

McLaughlin Water Engineers, Ltd.

Project Title ARKANSAS RIVER GRADE CONTROL & BOAT CHUTE Project No. 83-113.002 Date 1-16-87
Subject CHRONOLOGY OF EVENTS IN PUEBLO Designed GRC Page 1/3

DEC. 22, 1986 I NOTICED A SMALL LEAK IN THE GROUTED DAM FACE. I CALLED DENVER TO TALK WITH JOHN PFLAUM WHO WAS OUT OF THE OFFICE. I TALKED TO BILL TAGGART, EXPLAINING WHERE THE LEAK WAS AND THE APPROXIMATE FLOW. HE TOLD ME TO HAVE THE CONTRACTOR DRILL 2 HOLES IN THE DAM FACE. 1 TEST HOLE WAS TO BE WHERE THE WATER WAS LEAKING AND 1 TEST HOLE WHERE IT WAS DRY. I TALKED WITH RAY KOSTER WITH THE PUEBLO CONSERVANCY DISTRICT, ABOUT THE PROBLEM AND THE TEST TO BE CONDUCTED.

I WENT TO BASSETT CONSTRUCTION CO OFFICES TO TALK TO BOB BASSETT ABOUT THE PROBLEMS, AND THE MEASURES WHICH WOULD BE REQUIRED. I TALKED WITH BOB BASSETT Supt., JOHN BASSETT AND JOE MARINO PROJECT MANAGERS. THEY EXPRESSED CONCERN ABOUT THE LIABILITY AND THE COSTS OF THE TEST MEASURES.

DEC 23, 1986 I CALLED AND TALKED TO JOHN PFLAUM EXPLAINING THE PROBLEM AND THE TEST BILL WOULD LIKE TO MAKE.

AT ABOUT 12:30PM THE CONTRACTOR DRILLED 3 HOLES (W₁, D₁, D_{2L} ON FIG 1) IN THE DAM FACE. THE WATER RAISED TO ELEVATION 50³¹ IN TEST HOLE W₁ AND IN TEST HOLE D₁ AND D_{2L} IT WAS 6" BELOW THE GROUT SURFACE. THE CONTRACTOR DRILLED THE THIRD TEST HOLE BECAUSE HE THOUGHT THAT HE HAD HIT A TIE BEAM WITH THE SECOND HOLE.

I CALLED THE OFFICE AND GAVE THEM THE INFORMATION OVER THE PHONE. THEY ASKED IF I COULD GET THEM THE INFORMATION IN THE OFFICE BY WEDNESDAY MORNING. I DROVE TO DENVER TO GET THEM THE INFORMATION.

DEC 24, 1986 I DROVE TO PUEBLO TO DRILL 6 MORE HOLES (A, B, WM, WH, DM, D_{2H}) BEFORE THE GROUT POUR SCHEDULED FOR 8:00AM.

DEC 25, 1986 HOLIDAY

DEC 26, 1986 I WAS IN THE OFFICE IN DENVER DISCUSSING THE PROBLEM AND PIEZOMETER READINGS WITH BILL TAGGART AND JOHN PFLAUM. THEY DECIDED THAT 4 ADDITIONAL HOLES WERE REQUIRED (F, E, D, C).

DEC 29, 1986 IN PUEBLO THE CONTRACTOR DRILLED THE FOUR HOLES AND I TOOK SOME ADDITIONAL READINGS (FIG 5). JOHN PFLAUM CALLED JOE MARINO TO INFORM HIM THAT THE INSPECTION COSTS HAD BEEN SUSPENDED FROM DECEMBER 22, 1986 UNTIL A SOLUTION WAS DECIDED ON.

DEC 30/31, 1986 I TOOK MORE PIEZOMETER READINGS.

JAN 6, 1987 I DROVE TO PUEBLO TO TAKE CROSS SECTIONS OF THE GROUTED ROCK FACE AT APPROXIMATELY 20' INTERVALS. I DROVE TO DENVER AND PLOTTED THE CROSS SECTIONS UP.