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Soil Conservation Service

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Subject: ENG-Stability and Erosion Analysis of the RWCD Floodway Reaches 1 and 2

Date: May 23, 1985

To: Ralph M. Arrington, State Conservation Engineer

File Code 210

Introduction

At yours and Jack Stevenson's request, I have observed reaches 1 and 2 in the field, reviewed the design, analyzed the stability of reaches 1 and 2, and prepared this report. Attached are some notes from the field investigation made on 5/20/85. Also attached are copies of the calculations I used for the analysis. The points mentioned in the notes were considered in the analysis and the preparation of this report.

Problem Definition

Two separate and distinct problems were apparent from the field review and were subsequently confirmed by the analysis. The first problem is general bed degradation leading to toe erosion of the levees. Local scour also was prevalent immediately below each rock-lined section. The second problem is the continuous rill erosion along the levees in reaches 1 and 2 with some additional gullying and jugging in reach 2. These two problems raised three questions: (1) Was the bed degradation and erosion by flow in the floodway? (2) Was the rill, gully, and jugging in the levees caused by raindrop splash and subsequent surface erosion? (3) Is dispersive soil a factor in questions 1 and 2?

The floodway is 4 years old. There have been 3 flows through the floodway since construction. The last flow was in December 1984 and it approached a depth of 3 feet in the earth-lined sections. This last flow is estimated to be the maximum flow to date in the floodway. My analysis would indicate this to be approximately 2500 cfs (d=2.9'). The design discharge for the floodway is 8700 cfs. Therefore, the maximum historical stress was approximately 29% of the design discharge.

Reach 2, above the concrete chute, has a much flatter gradient (S_o=0.0003). The same historical flows (Q=2500 cfs) produced a maximum tractive power of 0.20 ft-#/sec/ft^2 as opposed to the 0.75 ft-#/sec/ft^2 allowable. There was no evidence of bed erosion, toe erosion along the levees, or sediment deposition in the rock-lined section immediately above the concrete chute. However, there was evidence of bed and levee toe erosion immediately below the concrete chute where the bed slope was steep (S_o=0.0015)and subsequent deposition in the downstream rock-lined sections.

Existing survey cross-sections were requested beginning at station 1380+99 downstream through station 1398+76 at all as-built cross-sections. The upstream cross-section is at the upstream edge of a rock-lined section. The downstream cross-section is approxiately 200+ feet downstream of the downstream edge of the same rock-lined section. The purpose of these cross-sections was for comparision to the as-built cross-sections to determine deposition within the rock-lined section and to estimate bed erosion immediately downstream from the rock-lined section.

Bed erosion consists of two parts: (1) general scour, indicating bed degradation; and (2) local scour immediately below the rock-lined section caused by the water as it accelerates coming off of the rock lining.

Only 560 feet separated the downstream edge of this particular rock-lining and the upstream edge of the next rock-lining. There was not much opportunity for general bed erosion between these two rock-lined sections because of the increased stage due to the downstream rock-lined section. Additional existing cross-sections below other "hard points" are needed to confirm the general bed erosion problem. However, the deposition within the rock-lined section averages approximately 1 foot. The source of this sediment must have originated within the floodway below the concrete chute (sta 1160+22). If so, the additional surveys will show that significant bed erosion already has occurred.

Fourteen samples of the soil were taken between stations 1242+00 and 1435+00 for purposes associated with the recent contractor's claim. A chemical analysis was made of these 14 samples. Table 1 is a summary of the sodium and total salt content found in these samples. 40% sodium to total salt is supposed to be an indicator of a potential dispersion problem. 60% sodium to total salt content is supposed to indicate that there is a dispersion soil problem. All samples but one were greater than 40%; 9 were greater than 60% as was the aggregate of all. My conclusion is that there is a dispersion problem. Testing of lime treatment for these specific soils should be done before this method is used.

A review of the design procedures shows that the Arizona design staff followed TR-25. The design used the tractive power approach; however, the tractive stress procedure is an integral part of the tractive power approach. Unfortunately, the tractive stress procedure recommended in TR-25 for fine-grained materials is in error. The tractive stress procedure for fine-grained materials assumes that the energy loss is divided between work done on the boundary and energy losses to other causes. This is not true for a fixed-boundary plain-bed analysis, which is the situation for the RWCD Floodway. There are essentially no other causes for energy loss except the fixed, plain bed.

The allowable tractive power for the design of the RWCD Floodway is 0.75 ft-#/sec/ft^2 (unconfined compressive strength=350 #/ft^2). The floodway has already been stressed at approximately Q=2500 cfs to a tractive power greater than 1.07 ft-#/sec/ft^2. This is more than 43% greater than the allowable. The tractive power attacking the boundary, assuming the entire energy is working on the boundary, for the design discharge of Q=8700 cfs would be 3.4 ft-#/sec/ft^2. This would be more than 4.5 times the allowable.

TABLE 1. CHEMICAL DISPERSION ANALYSIS

Sample No.	Sta	Na Meq /L	Na,Cl,Mg K, Meq /L	Na %
1	1343+00	56.11	124.045	45.23
2	1365+00	51.20	116.553	43.93
3	1338+00	58.63	87.989	66.63
4	1271+00	24.84	37.735	65.83
5	1277+00	27.58	77.342	35.66
6	66+00	21.01	22.079	95.16
7	1242+00	30.36	37.020	82.01
8	1261+00	24.97	30.230	82.60
9	1252+00	34.10	40.145	84.94
10	1250+40	43.67	53.397	81.78
11	1250+50	31.10	43.274	71.87
12	1375+00	36.45	63.920	57.02
13	1401+00	43.54	85.959	50.65
14	1435+00	27.23	28.414	95.83
		510.79	848.102	60.23

It is reasonable to assume that general bed erosion has occurred. It will be checked by determining the difference between existing and as-built sections. The existing section have been stressed nearly 50% greater than the maximum allowable. That fact that this has occurred for a discharge less than 30% of the design discharge, suggests that general bed degradation at design discharges would be massive, endangering the integrity of the levees.

The distance between the downstream end of the concrete chute and (sta 1160+22) and upstream end of the rock chute (sta 1464+00) is 30,378 feet with a drop of 44.9 feet. The compacted earth-lining can safely withstand only 10.8 feet of that fall. There are 4291 feet of rock-lined sections that safely removed 6.4 feet of the drop. The remaining 27.7 feet of fall must be safely withstood or that much accumulative bed degradation can be eventually expected with subsequent downstream deposition that will encroach on the design capacity.

Other Reaches

Reach 3 was checked for the tractive power design. The same fine-grained tractive stress procedure was used. However, the resulting design, although underestimating the actual design tractive stress, overdesigned the required tractive power. However, there are some clean sand stringers present in the floodway. These sand stringers will armor within 0.5' at the design discharge.

Table 2 can be used to determine the maximum allowable bed slope for any given Q/b. It is based upon an allowable tractive power of 0.75 ft-#/sec/ft^2 which is associated with an unconfined compressive strength of 350 #/ft^2.

Table 2. Maximum So vs Q/b

Q/b (cfs/ft)	So (ft/ft)
43.5	0.000355
40.0	0.000382
30.0	0.000485
20.0	0.000692
10.0	0.001303

Possible solutions

The Arizona Engineering Staff provided the following cost estimates:

(1) Armor material (D_{60} not less than 1.6 inches and a D_{50} not less than 1.2 inches with not more than 20% fines) be used in a 6 inch layer across the entire floodway and up the inside of each levee to a height of 6.5 feet. The armor material is to be placed beginning at the downstream end of the concrete chute (station 1160+22) and ending at the rock chute at the confluence with the Gila River (station 1464+00) excluding any existing rock-lined section. An estimated cost for this work is $1,000,000.

2. Armor material be placed across the entire cross-section, from outside toe of left levee to outside toe of right levee, between the rock and concrete chutes. Also, across the levees on each side from the concrete chute (station 1160+22) to the upstream end of reach 2. An estimated cost for this $1,400,000.

3. Two alternate lime treatments for the dispersed soils:

   a. Levees only. The cost estimate is $470,000.

   b. Levees and floodway. The cost estimate is $1,400.000.

Conclusions

There are no bed erosion problems in reach 2 upstream from the concrete chute. There are serious bed and subsequent levee toe erosion problems in reaches 1 and 2 below concrete chute. The rilling along the levees in reaches 1 and 2 seemed serious to me; but, then I am not experienced with this environment.

(1) The earth-lined section of reaches 1 and 2 of the RWCD floodway is greatly under-designed with respect to the tractive stress that would be placed upon it by the design discharge and will continue to erode at much smaller discharges unless protective measures are taken.

(2) Sediment from floodway erosion will be deposited in the rock-lined sections and will continue to encroach on the design capacity. The level of protection will eventually become seriously impaired.

(3) Potentially dispersive soils are present within the boundary materials in reaches 1 and 2 of the RWCD Floodway.

Recommendations

(1) The RWCD Floodway be protected against the design flow from the concrete chute (sta 1160+22) down to the rock chute confluence with the Gila River (sta 1464+00)

(2) Protection be provided to the levees, if necessary.

(3) Recognition be given to the presence of potential dispersive soils.

Ralph, I enjoyed working with you and your staff. I am sorry it had to be under such alarming circumstances. I appreciated the opportunity to be able to speak with such candor to Verne Bathurst, State Conservationist. I greatly appreciated working directly with Aubrey Sanders, Bill Payne, John Sullivan, Susanne Leckband, and Neomi Nielsen.

Fred D. Theurer

Dr. Fred Theurer
Soil Conservationist

cc: Wendell D. Moody, Assistant Director of Engineer
Jack C. Stevenson, Head Technology Staff WNTC

Field Investigation Notes 5/20/85: RWCD Floodway Reaches 1,2,& 3.

Ralph, Aubrey and Susanne took me to the field to see Reaches 1, 2, & 3 of the RWCD Floodway. We started at the extreme downstream end of the floodway at its confluence with the Gila River. We worked our way upstream through Reach 2 into the new construction area of Reach 3. The following general observations were noted.

Observations

1. Severe rill erosion was evident on both levees. Deposition from this rill erosion was evident at the toes of the slope wherever erosion of the toes was not evident. Question: Is dispersion a problem?

2. Erosion at the toes of the levee were noted starting immediately and for some distance downstream from the rock lined sections and coincedent with low flow channels adjacent to the levees. However, toe erosion was not always evident wherever low flow channels are adjacent to the levees such as when immediately upstream from the rock lined sections.

3. Severe local scour was always evident downstream of rock lined sections. If the top of the rock lined sections were placed at grade, then severe sheet erosion also had to occur because the rock liners appeared to be better than a foot above current grade. Also

deposition was evident in the rock lined sections throughout the upstream portions. The downstream rock line section (first 50') did have deposition.

4. The downstream section of the reach immediately below the concrete chute appear to have evidence of as much as 18 inches of erosion.

It would appear to be important to investigate the chemistry of the earth lined sections used in Reach 1. Clay is present in the earth lined section. These clays range from kaolinite through montmorillonite (which are the platelike structures) and polygorahite clays (tubular structure). The clay ranges from no shrink-swell behavior into significant shrink-swell relationships. Furthermore if sodium is present in a form that could cause dispersion, the silt-clay earth lined sections would then be highly erodable. The reaches above Reach 1, (Reaches 2 and 3), appear to have a more sandy composition. If so the tractive stress approach would certainly be applicable.

It is important that we check the thinking of the Design Engineer regarding the tractive power approach. Secondly, it is important that we investigate the resistance analysis regarding the soils used for the earth lining in Reach 1.

Pictures were taken of Reach 1 and 2, picture number 13 was taken upstream of the bridge at station 1367+66. It was taken to show the deposition in the beginning of that rock lined section. Picture number 14 was taken at station 1335+00 to show the severe toe erosion that occurred upstream of the second rock lined section. Pictures 15 through 22 were taken subsequently as we moved upstream.

In order to determine the amount of erosion that has occurred since the Reaches 1 and 2 were constructed, I would suggest that surveyed sections be taken at the same location as the as-built sections beginning at station 1399+00 through station 1434+60. These sections would begin 200 foot upstream of the second rock lined section proceed through the deposition in the second rock lined area to the 200 foot downstream of the first rock lined area. This should give us a typical erosion-rate and deposition picture of the rock lined areas. The surveyed sections should be taken coinciding with the as-built sections.

It may be necessary to determine the remaining thickness of the earth lined areas by coring the earth-lined material. Furthermore, disturbed samples may be taken of the earth-lined areas in order to determine (1) the chemistry of the earth-lined materials and (2) the resistance properties of the earth-lined materials.

Query: What was the quantitative value of using the earth lined materials in lieu of the existing materials found in grade?

Observation: At the design discharge of 8700 cfs (1% chance), a stability analysis (n = 0.027) shows that the depth of flow would be 6.01 ft. and flow velocity would be 6.64 fps. For a bottom width of 200 foot this would be better than 43 cfs per foot of width. Also the tractive stress would be in the neighborhood of 0.52 pounds per square foot and tractive power greater than 3.4 ft-#/sec/ft^2. Both the unit discharge and the actual tractive power would appear to be very high for ML, CL, and SC materials; the design allowed only 0.74 ft-#/sec/ft^2.

Question: Would such materials withstand such high stress for any length of time? Subsequent question. If not what could be done about the existing design?

All rock lined sections below the concrete chute had no bedding beneath the 1 foot thick rock lining. The rock lining in some areas had the appearance of settling, as if the fines beneath the rock lined were being removed. This phenomena was not observed immediately above the rock concrete chute. It had bedding beneath the rock lining. If the rock lining is to serve as hard points in the channel (that is to serve as hinge points within the channel), then the rock lining would have to be prevented from settling. Otherwise the rock lining would continue to bury itself into the fine-grained material beneath it.

Stress the importance of determining the chemistry of the material used in the channel. Especially the material used as the earth lining. Second, stress the tractive stress analysis. Third, determine the concepts behind the use of the tractive power analysis. Determine who in SCS is a proponent of the use of the tractive power. Talk with Lee Saeles and Cliff Deal at the WNTC. Talk with Jim Talbot regarding dispersive soils. Talk with Dave Ralston regarding the use of the tractive stress, tractive power, and dispersive soils. Talk with Jack Stevenson regarding the potential seriousness of the erosion problem in Reaches 1 and 2. Talk with someone on the Arizona State Staff that is familiar with the soils properties that were used in Reaches 1 and 2. By this I mean speak with a Soil Scientist who may know of the chemistry of the soils that were used.

Talk with Aubrey regarding a tractive stress analysis in the materials found in Reach 3. These materials appear to be sandy as opposed to silt or clays. The plus 15% gravels may serve as a armor layer if necessary. However, the materials used in the levees of the upstream portion of Reach 2, that is above the rock chute, appear to be silts and blew out at the rock chute inlets coming into the channel.

Observation: The rock liner above the concrete chute has no evidence of any deposition or scour that would cause settling of the rock. There is a bedding beneath this rock liner. The concrete chute below the rock liner shows absolutely no evidence of any deposition. Query: If there is no deposition in the vicinity of the concrete chute and above, what is the source of deposition in the downstream reaches?

Question: How do you treat dispersive soils? Is lime used? Solid or liquid forms, or both?

Question: How expensive would it be to use a rock liner throughout the entire length of Reach 1? Include a bedding.

If a potential severe erosion problem exists, would not the levee (that portion above ground) be the most hazarduous?

Tractive stress in transport capacities need to be determined for Reaches 1, 2 and 3. The potential for armor in Reach 3 should also be analyzed. Two n values should be checked (1) for the bare earth channel and (1) for the rock lined portions. Sediment transport should be calculated using suspended load formulas such as the Einstein Bedload Function.

Historical hydrologic stress should be ascertained. Check with County officials also. Estimates of duration of flow also need to be made. This information should be coupled with sediment transport calculations to determine the volume of sediment entering the Gila River. Compare these estimates with the difference between existing cross sections and as-builts to calibrate the sediment transport model. Determine the amount of sediment that would move at the design discharge.

Observation: The high water mark in the concrete chute appears to be at the mid-point of the weep holes.

Question: If the rock grouted waterway inlets blew out in the upper end of Reach 2, would the same potential exist at the design discharge within the levees outwarded into the fields (considering the rilling that has already occurred)?

Question: Is there any reason to believe that the soils used in Reaches 1 & 2 do not behave as discreet particles? And if so, then the transport rate would also be a function of how rapidly the lining material could be peeled away.

Thought, talk to State Hydrologist to determine frequency-discharge information.

Remember to emphasize that the new cross sections should be plotted with respect to the as-built cross sections to determine the amount of erosion.

Question: Could the deposition that is immediately downstream of the rock chute in to the Gila River have originated from the floodway instead of the Gila River?

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SOIL CONSERVATION SERVICE

STATE		PROJECT RWCD Floodway		
BY	DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT Hydraulic Analyses				SHEET 1 OF 5

Formulas Used:

$$Q = (1.486/n)\, A R^{2/3} \sqrt{S_o} \quad , \quad cfs$$

$$A = (b \cdot d) + (z \cdot d^2) \quad , \quad ft^2$$

$$P = b + d\sqrt{1+z^2} \quad , \quad ft$$

$$R = A/P \quad , \quad ft$$

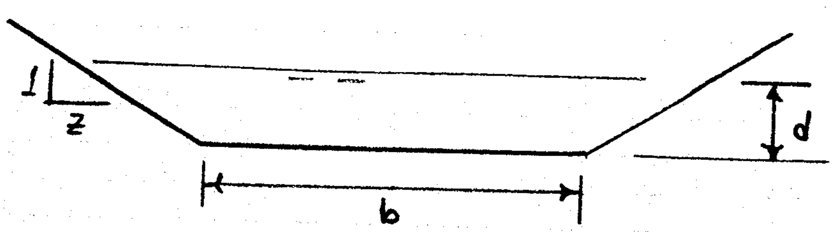
$$\tau = \gamma R S_o \quad , \quad \#/ft^2$$

$$v = Q/A \quad , \quad fps$$

$$\text{tractive power} = \tau \cdot v \quad , \quad ft\text{-}\#/sec/ft^2$$

$$d_{50} \geq 11 \cdot d \cdot S_o \ \text{(shields initiation of motion)} \quad , \quad ft.$$

Reach No.	cfs Q	ft b	— z	ft/ft S_o	— n	remarks
1 & 2 below conc. chute	8700	200	3	0.0015	0.027	design
	2500	200	3	0.0015	0.027	historical
2 above conc. chute	8700	200	3	0.0003	0.027	design
	2500	200	3	0.0003	0.027	historical
3	8100	200	3	0.0003	0.027	design
	2328	200	3	0.0003	0.027	historical



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STATE	PROJECT RCWD Floodway		
BY [illegible] DATE 5/21/85	CHECKED BY	DATE	JOB NO.
SUBJECT Flow Analysis			SHEET 2 OF 5

n-0.0[illegible] changes capacity to

Reach 1 : $b = 200'$, $z = 3$, $S_o = 0.0015$

Q cfs	n –	d ft.	R ft.	v fps	τ #/ft²	τ·v (ft-#/s)/ft²
8700	0.015	4.24	3.98	9.64	0.3725	3.590
	0.020	5.03	4.67	8.04	0.4370	3.513
(1%)	0.025	5.74	5.28	6.98	0.4939	3.446
	0.027	6.01	5.50	6.64	0.5150	3.422
	0.030	6.39	5.83	6.21	0.5453	3.387
	0.035	7.00	6.33	5.63	0.5925	3.334
	0.040	7.57	6.80	5.16	0.6363	3.286
2500	0.015	2.02	1.96	6.00	0.1832	1.100
	0.020	2.40	2.31	5.03	0.2162	1.087
(9% in	0.025	2.74	2.63	4.38	0.2457	1.077
any one	0.027	2.87	2.74	4.18	0.2568	1.073
year); 31%	0.030	3.05	2.91	3.91	0.2727	1.067
in 4 years)	0.035	3.35	3.18	3.56	0.2976	1.058
	0.040	3.62	3.43	3.27	0.3209	1.050
2000	0.015	1.77	1.72	5.51	0.1610	0.886
	0.020	2.10	2.03	4.62	0.1902	0.878
(12% &	0.025	2.40	2.31	4.02	0.2162	0.870
	0.027	2.51	2.41	3.84	0.2260	0.867
40%)	0.030	2.67	2.56	3.59	0.2401	0.863
	0.035	2.93	2.80	3.27	0.2622	0.857
	0.040	3.17	3.02	3.01	0.2828	0.851
1500	0.015	1.49	1.45	4.93	0.1361	0.670
(16% & 50%)						

$\tau = \gamma R S_o$

0 - Q	5 - P
1 - S_f	6 - R
2 - n	7
3 - d	8
4 - A	9

d, S_o, Q - Lbl A
n - RLS or Lbl B

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SUBJECT Reach 3 Flow Analysis			SHEET 3 OF 5

Reach 3 : $b = 200'$, $z = 3$, $S_o = 0.0003$

Q cfs	n –	d ft.	R ft.	v fps	τ #/ft²	$\tau \cdot v$ ft-#/sec/ft²
8700	0.015	6.82	6.19	5.78	0.1158	0.670
	0.020	8.08	7.21	4.80	0.1350	0.649
	0.027	9.62	8.44	3.95	0.1580	0.624
	0.035	11.18	9.65	3.33	0.1806	0.602
8100	0.015	6.54	5.95	5.64	0.1115	0.628
	0.020	7.75	6.94	4.68	0.1300	0.609
	0.027	9.23	8.13	3.86	0.1522	0.587
	0.035	10.73	9.30	3.25	0.1741	0.566
6900	0.015	5.95	5.46	5.32	0.1022	0.544
	0.020	7.05	6.38	4.42	0.1193	0.528
	0.027	8.41	7.48	3.65	0.1400	0.510
	0.035	9.78	8.56	3.08	0.1603	0.493
2500	0.015	3.26	3.10	3.65	0.0581	0.212
	0.020	3.87	3.65	3.05	0.0683	0.208
	0.027	4.63	4.32	2.53	0.0808	0.204
	0.035	5.39	4.98	2.14	0.0932	0.200
2328	0.015	3.13	2.98	3.55	0.0558	0.198
	0.020	3.71	3.51	2.97	0.0656	0.195
	0.027	4.43	4.15	2.46	0.0777	0.191
	0.035	5.17	4.79	2.09	0.0896	0.187

resistance ;

cohesive soils $c = 350$ #/ft² – $\tau \cdot v < 0.74$ ft-#/sec/ft²

discrete particles : $d_{50} = 1.2$ mm $\tau < 0.043$ #/ft²

rock rip rap : $d_{50} \geq 11 \, d S_o = 11 \cdot (11.18) \cdot 0.0003 = 0.4''$ $= 11.25$ mm $= 0.0369'$

or $\tau_{all} = 4 \cdot d_{50} = 4 \cdot \left(\frac{4}{12}\right) = 1.33$ #/ft²

armor : $d_m = 11 \cdot 9.62 \cdot 0.0003 = 0.0317'$
$m = 92\%$ from log data (sieve analysis)

$$D_d = d_m \left(\frac{100}{100-m}\right) = \frac{0.0317}{100-92} = 0.40'$$ depth of streambed erosion to armor.

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STATE | PROJECT RWCD Floodway
BY [illegible] | DATE 5/23/05 | CHECKED BY | DATE | JOB NO
SUBJECT Q/b vs max. S_o Analysis |

$\tau \cdot v$ (lb-ft/sec/ft^2)

S_o \ Q/b	43.5	40	30	20
0.0007				0.759
0.0006	1.304	1.211	0.936	0.647
0.0005	1.075	0.999	0.773	—
0.0004	0.848	0.788	0.611	—
0.0003	0.627	0.581	0.451	—
0.0002	0.404	—	—	—

	Sta	Elev
R.C.	1464+00	1195.5
C.C.	1160+22	1240.4
	30,378	44.9

S_o = 44.9/30378 = 0.00148

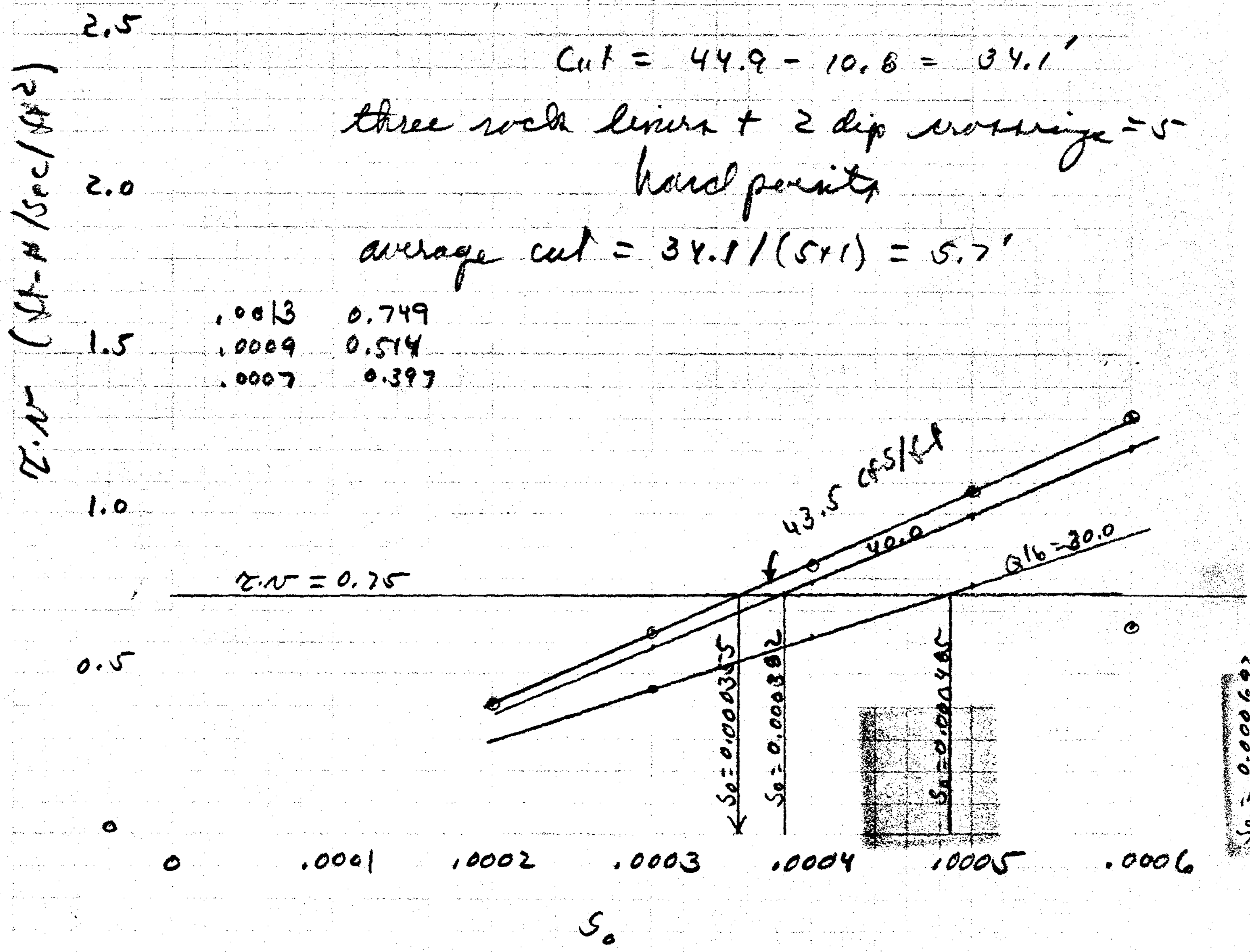
Δh_{all} = 30378 × 0.000355 = 10.8'

Cut = 44.9 − 10.8 = 34.1'

three rock liners + 2 dip crossings = 5 hard points

average cut = 34.1/(5+1) = 5.7'

.0013	0.749
.0009	0.514
.0007	0.397



0.397

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STATE		PROJECT RWCD		
BY [illegible]	DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT Armor material analysis				SHEET 5 OF 5

$d_{75} = 2''$

$n = 2^{1/6}/39 = 0.029$

$d = 6.24'$

$R = 5.70'$

$v = 6.38$ fps

$\tau = 0.5332\ \#/ft^2$

$\tau \cdot v = 3.401$ ft-#/sec/ft²

(Shields) $d_{50} \geq 11 \cdot d \cdot S_0 = 11 \cdot 6.24 \cdot 0.0015 = 0.103' = 1.235'' = 31.4\ mm$

(from #108) $d_m = \tau/4 = 0.5332/4 = 0.1333' = 1.60''$

use min. 4" layer

$D_d = d_m \left(\frac{100}{100-m} \right)$

$m = 100 - 100 \left(d_m / D_d \right) = 100 - 100 \left(1.60 / 4 \right) = 60\ \%$

$d_{max} = 3''$