RWCD Reach 1 and 2 (EMF Reach 1 and 2)

Stability Design

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RWCD 1 & 2
EMF Reach 1 & 2

ATTACHMENT 11

STABILITY DESIGN

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1. Design Approach--Tractive Power

The tractive power approach as found in Chapter 6, Technical Release No. 25, was used as a basis for the design of the RWCD Floodway, Reach I.

2. Flaxman Data--Tractive Power Based on Average Tractive Stress

Based on field observations and data from 12 ephemeral and perennial channel reaches located in 6 western states (12 reaches from apparently 9 natural channels), Flaxman[1,2,3,4] presented in graphical form a correlation between tractive power (τV) and the saturated total unconfined compressive strength (q_u) of undisturbed channel boundary material. Figure 1, the Figure 2 of Reference 1, shows his line of demarcation between two separate zones, one for eroding and the other for non-eroding channel reaches. For the 12 reaches studied, 5 were eroding and 7 were stable. The line of demarcation, defined in TR-25 as the S-line, is obviously highly subjective.

The average tractive stress was used to compute τV,

$$\tau_{ave} V = (\gamma R S_e) V \qquad (1)$$

where R is the hydraulic radius of the representative channel section for that reach for flow at the high water mark; S is stated by Flaxman to be the channel slope (probably the energy slope Se); V is the average velocity.

Figure 2 is a replot of Flaxman's Figure 2, the demarcation S-line having the same slope and ordinate-intercept. Based on changes listed in his Closure[3], point 13 has been deleted and point 6 has been replotted using a velocity of 8.25 rather than 6.34 fps. Also, point 8 is not consistent between Flaxman's Table 1 and Figure 2 in Reference 1; point 8 has been replotted using the $\tau_{ave} V$ of the table. Refer to Table 1.

Flaxman[4] did list the base flow conditions for 5 of the 12 test reaches and these are indicated on Figure 2. Three were perennial and two were ephemeral. Reference 4 gives additional information on soil properties, including the plasticity index, for these same five reaches.

Flaxman's S-line is a straight line on semi-log graph paper indicating an exponential relationship between $\tau_{ave}V$ and q_u.

3. Flaxman Data--Tractive Power Based on Reference Tractive Stress for Coarse-Grained Discrete

Flaxman's Closure[3] lists recomputed values of τV using the reference tractive stress of TR-25 for coarse-grained discrete soils,

$$\tau_\infty V = (\gamma d S_t) V \qquad (2)$$

where d is the flow depth and S_t is the part of the total S_e apportioned to the channel boundary material evaluated by,

$$S_t = \left(\frac{n_t}{n}\right)^2 S_e \qquad (3)$$

where n_t is the particle (grain) resistance coefficient and n is the total channel resistance coefficient of Manning.

The recomputed $\tau_\infty V$ values are shown in Table 2 and plotted graphically in Figure 3, the latter graph reproduced from an earlier edition of TR-25. Note that the abscissa is $\frac{q_u}{2}$. Of the 12 recomputed Flaxman data values, 8 match exactly the hollow circled points of the earlier edition of TR-25; 2 others are a close match; only points 7 and 9 do not match.

It would appear that the original Flaxman data was the basis for the demarcation S-line of Figure 3. Note that five additional points are shown in this earlier TR-25 edition. While the origin of these added points is unknown to the writer, they may be from channels also studied by Flaxman[4]. Reference 4 lists other tested channels but the data is not in a form which permits computing $\tau_\infty V$. How many of these extra plotted points were ephemeral and how many perennial could not be ascertained.

The S-line is a straight line on log-log graph paper indicating a power relationship between $\tau_\infty V$ and $\frac{q_u}{2}$.

Going back to the 12 Flaxman test reaches, it can be seen that the values of the hydraulic radius (R), Table 1, differ from their related flow depths (d), Table 2. The test reaches apparently had limited b/d ratios. In addition, it is difficult to imagine a natural channel having a set water depth (d). Possibly, the stated d is really the computed mean depth (d_m),

$$d_m = \frac{a}{T} \tag{4}$$

where a is the wetted area and T is the width of the water surface.

4. Flaxman Data--Tractive Power Based on Reference Tractive Stress for Fine-Grained Soils

The current edition of TR-25 computes τV using the reference tractive stress for fine-grained soils,

$$\tau_{R_t} V = (\gamma R_t S_e) V \tag{5}$$

where R_t is the hydraulic radius associated with particle (grain) roughness and S_e is the energy slope.

Table 3, shows the recomputed $\tau_{R_t} V$ values for the 12 tested channels of Flaxman. Note that D_{75} was used to compute τ_{R_t}; D_{65} is the correct grain size to use but these values were not available.

Note that Figure 4, the current TR-25 edition, duplicates the graph of Figure 3 from the earlier TR-25 edition. It has the same slope, ordinate intercept, and regression equation. Values of τV versus $\frac{q_u}{2}$ at the S-line are equivalent, indicating that $\tau_\infty \approx \tau_{R_t}$. If true, this is most interesting.

Tables 2 and 3 show that for 8 computed points, the magnitudes of τ_∞ are close to those for τ_{R_t}. Points 7 through 10 differ appreciably in magnitude.

The 12 recomputed $\tau_{R_t} V$ values plotted in Figure 4, Figure 6-15 of TR-25, show that point 7, marked erosive, sticks out like a sore thumb. Possibly the data for points 7 through 10 have been massaged. Were any of the five additional data points mentioned in Figure 3 designated E and did any of these plot in the stable zone? SCS does not reference much of the literature it uses.

5. Theurer's Contention that the RWCD Floodway, Reach I, is Overstressed

In a report dated 23 May 1985, Theurer to Arrington, page 2 states:

> 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.

Let us assume for a moment that Theurer's hypothesis, that the grains have no influence on the flow resistance, is correct.

For Reach I, the design Q = 8700 cfs, b = 200 ft, Z = 3:1, $S_o = 0.0015 = S_e$, n = 0.027, d = 6.01 ft, b/d = 33.3, R = 5.50 ft, V = 6.64 fps, Temperature = 75° F. In terms of the water force,

$$\tau_{ave} = \gamma R S_e = 62.4 \times 5.50 \times 0.0015 = 0.515 \quad \underline{\text{and}}$$

$$\tau_{ave} V = 0.515 \times 6.64 = 3.422.$$

Using the S-line of Figure 6-15 in TR-25, Figure 4, at $\frac{q_u}{2} = 350$ results in an allowable (boundary material resistance) $\tau V = 0.74$. Theurer claims the channel is overstressed, the actual τV being more than 4.5 times the allowable τV --- $\frac{3.422}{0.74} = 4.62$.

In terms of the water force, the Arizona design staff computed a $\tau V = 0.1047 \times 6.64 = 0.695$ based on the bed governing---

$$\tau_b = \frac{\tau_b}{\tau_{R_t}} \times \tau_{R_t} = 1.0 \times 0.1047$$

(whereas $\tau_s = \frac{\tau_s}{\tau_{R_t}} \times \tau_{R_t} = 0.8\ \tau_{R_t} = 0.8 \times 0.1047$).

With the actual $\tau_b V = 0.695$ less than the allowable $\tau V = 0.74$, the channel would not be overstressed.

Theurer errs in that by using $\tau_{ave} = 0.515$ and $\tau_{ave} V = 3.422$, he should have used the Flaxman curve, Figures 1 and 2 at a $q_u = 700$, rather than at a $\frac{q_u}{2} = 350$. Using $q_u = 700$ results in an allowable $\tau V = 5.85$. Thus, the actual $\tau V = 3.422$ is less than the allowable $\tau V = 5.85$ and the channel would not be overstressed.

Even if Theurer's hypothesis were correct, the channel boundary would not be overstressed. His hypothesis has merit, however, and will be discussed later in this Attachment, Section 8.

6. Partitioning of the Total Tractive Stress--Different Methods to Compute the Reference Tractive Stress

The computation of τV exerted by the water on the earth channel material depends on

- the magnitude of the reference tractive stress, and,
- the magnitude of the channel geometry correction factors used to evaluate the maximum water stress on the bed and on the banks.

The total τ exerted by the water on a channel section can be divided into two parts

$$\tau = \tau' + \tau'' \qquad (6)$$

where τ' is the stress attributed to grain resistance (that portion of the total stress acting to dislodge the channel material), and τ'' is the

remaining stress attributed to bed forms, debris, vegetation and other factors.

One way to divide the total shear is

$$\tau = \gamma RS_e = \gamma RS' + \gamma RS'' \qquad (7)$$

where S' is the energy slope associated with grain roughness and S" is the remaining shear. To isolate the water force acting to move channel boundary material for coarse-grained discrete (6.35mm < D_{75} < 127mm), TR-25 uses the reference tractive stress $\tau_\infty = \gamma dS_t$, the τ_∞ in an infinitely wide channel. This τ_∞ is then multiplied by the channel geometry factors to compute the maximum τ_b on the bed and the maximum τ_s on the banks. The method is based on Lane and the USBR[5].

Another way to divide the total stress is

$$\tau = \gamma RS_e = \gamma R'S_e + \gamma R''S_e \qquad (8)$$

where R' is the hydraulic radius associated with grain resistance and R" is the hydraulic radius to account for all other factors. For fine-grained soils (D_{75} < 6.35 mm), TR-25 uses the reference tractive stress $\tau_{R_t} = \gamma R_t S_e$. This τ_{R_t} is then multiplied by a different set of channel geometry correction factors to compute the maximum τ_b and τ_s exerted by the water. The method is based on the work of Vanoni and Brooks[6]. Figure 6-9 of TR-25 is from that reference and is based on their flume data. The maximum b/d ratio used was 15/1. The tested sand sizes ranged from a D_{65} of 0.094 to 0.191 mm. Figure 6-10 of TR-25 is an extension of the curves beyond the original range of the Vanoni and Brooks data (for $\frac{V}{\sqrt{gK_s S_e}} > 1000$) and was intended to cover the fine-grained channel material. The D_{65} channel material on the RWCD Floodway, Reach I, and the recomputed values of the Flaxman data, Table 3 made use of Figure 6-10 only. The Arizona staff used a liner D_{65} = 0.03 mm and the Flaxman D_{75} ranged from 0.0415 through 0.1341 mm, point 8 being an exception with a D_{75} = 0.7327 mm. While the TR-25 computation procedure

for fine-grained soils is fairly simple, it is difficult to physically picture R_t. The tractive power method uses the fine-grained τ to compute τV, the τ being the larger of the τ_b or τ_s.

7. Channel Geometry Correction Factors

It appears that the S-line between erosive and non-erosive behavior in Figure 6-15 of the current TR-25 edition (Figure 4) was based on the reference tractive stress τ_{R_t}. The graph and S-line, in turn, would appear to be a duplicate of Figure 3 from the older TR-25 edition based on the reference τ_∞. Values of τV versus $\frac{q_u}{2}$ at the S-line are equivalent, indicating that $\tau_{R_t} \approx \tau_\infty$. This raises the questions:

- If indeed $\tau_{R_t} \approx \tau_\infty$, why do we have two methods to compute a reference tractive stress?
- If indeed $\tau_{R_t} \approx \tau_\infty$, why do we use two entirely different sets of channel geometry factors--one set for coarse-grained and another set for fine-grained--each giving very different magnitudes?
- Is $\tau_{R_t} \approx \tau_\infty$? Should $\tau_{R_t} \approx \tau_\infty$?

The correction factors depend on b/d and Z. Using b/d = 5 and Z = 2, for an example, Figures 6-3 and 6-4 of TR-25 give coarse-grained correction factors

$$\frac{\tau_b}{\tau_\infty} \approx 0.98 \text{ and } \frac{\tau_s}{\tau_\infty} \approx 0.78$$

whereas Figures 6-13 and 6-12 of TR-25 for fine-grained give

$$\frac{\tau_b}{\tau_{R_t}} \approx 1.35 \text{ and } \frac{\tau_s}{\tau_{R_t}} \approx 1.05$$

If $\tau_\infty = \tau_{R_t}$, the actual τ_b value for fine-grained would be $\frac{1.35}{0.98} = 1.38$ greater than for coarse-grained and the actual τ_s for fine-grained would be $\frac{1.05}{0.78} = 1.35$ greater than for coarse-grained. The actual τ values for the fine-grained would be more conservative, if $\tau_\infty \approx \tau_{R_t}$.

It is necessary to study the origin of these correction factors for channel geometry.

The coarse-grained curves of TR-25 cover a b/d from 0 to 10 and Z's = 1.5 and 2.0; the curves are from Lane[5] and Olsen and Florey[7]. The fine-grained curves of TR-25 cover a b/d from 0 to 10 and Z's = 1.5, 2, 3, 4 and 6; these curves come from HRB-108[8]. HRB-108 extrapolated the USBR results to the flatter slopes Z = 3, 4 and 6. This writer has compared the $\frac{\tau_b}{\tau_s}$ ratio at Z = 1.5 and 2.0 over a range of b/d from 1 through 10. For all practical purposes, the USBR $\frac{\tau_b}{\tau_s}$ values equal the $\frac{\tau_b}{\tau_s}$ values from HRB-108. And indeed they should. The HRB-108 curves show a $\frac{\tau_b}{\tau_s} \approx 0.85$ for $\frac{b}{d} > 10$ at Z = 3. The Arizona staff used a $\frac{\tau_b}{\tau_s} = 0.80$ based on $\frac{\tau_b}{\tau_{R_t}} = 1.0$ and $\frac{\tau_s}{\tau_{R_t}} = 0.8$ for their $\frac{b}{d} = 33.3$ at Z = 3.0.

There is, however, one last item. The HRB-108 curves use a reference τ_{ref}

$$\tau_{ref} = \tau_{ave} = \gamma R S_e \qquad (9)$$

to define the correction factors $\frac{\tau_b}{\tau_{ave}}$ and $\frac{\tau_s}{\tau_{ave}}$, duplicated in TR-25 as Figures 6-12 and 6-13. But TR-25 substitutes R_t for R to compute the reference τ_{R_t} for fine-grained and then multiplies τ_{R_t} by the correction factors $\frac{\tau_b}{\tau_{R_t}}$ and $\frac{\tau_s}{\tau_{R_t}}$.

If $\tau_{R_t} \neq \tau_\infty$ for fine-grained soils, then the use of the two entirely different correction factor curves would make sense. This writer compared the reference τ values for a limited selection of channel geometry b/d and Z values and grain D_{75} values. As shown in the schematic of Figure 5, the D_{75} intersection was approximately 0.30 mm, the $\tau_{R_t} < \tau_\infty$ for larger D_{75} and the $\tau_{R_t} > \tau_\infty$ for smaller sizes. The same trend occurred for the actual τ, both for τ_b and τ_s. The D_{75} intersection was around 3 to 5 mm. A more detailed sensitivity analysis over a much wider range of b/d, Z, and D75 (D65) is needed.

8. Wide Smooth Channels in Earth Material

Theurer, in his report to Arrington--refer to Section 5--hypothesized that for a smooth channel boundary, the grains should have little or no influence on the flow resistance. This is correct, especially for wide channels where $b/d > 10$. The RWCD Floodway, Reach I, has a $b/d = 33.3$.

For a very wide, smooth channel, τ_∞ and τ_{R_t} should be equal, the computed $S_t = S_e$ and the computed $R_t = d$.

Using the older edition of TR-25, the computed S_t for the finer D_{75} grain sizes were often found to be unrealistically too low. For instance, Table 2 shows $\frac{S_t}{S_e}$ ranging from a low of 0.05 to a high of only 0.22. To boost the computed S_t to a greater value, to artificially raise S_t to S_e, designers set a lower limit on n_t. This writer has used an n_t from 0.02 to 0.03 to force the computed S_t closer to S_e.

It was this limitation when applied to fine-grained soils that prompted the change to the current TR-25 method using τ_{R_t} as a reference stress. For a smooth channel material, the "SMOOTH" curve of Figures 6-9 and 6-10, TR-25, comes into play. Indeed, all twelve recomputed Flaxman data points, Table 3, have their $\frac{V}{\sqrt{gK_s S_e}}$ and $\frac{V^3}{g\nu S_e}$ values intersecting at or near this "SMOOTH" curve which says $R_t \rightarrow R$. At the design flow of 8700 cfs, the Arizona design staff calculations show this intersection to be on the "SMOOTH" curve.

Maintenance was performed in Reach I around August 1984 including removal of vegetation and general--not spot--grading across the entire width of channel. This loose material was not removed. During the December 1984 flow, this debris quite possibly bunched together creating bed forms. The actual water τ could thus have been greater than that predicted for a smooth bed.

9. Saturated Undisturbed Samples

Flaxman used the total q_u of "saturated undisturbed channel boundary material" as the sole indicator of the soil-sediment erosive potential. Flaxman[1] does not give the percent saturation of the soil samples for the twelve data points of Figure 1. Geotechnical literature does state that the magnitude of $\frac{q_u}{2}$ for a given tested sample does decrease with percent saturation, a 95% or higher level normally considered "saturated" by the profession. Refer to Schematic A of Figure 6. In his Closure[3], Flaxman talks about nine of the samples, stating that 5 were at 100% saturation and 4 were at approximately 92% saturation.

To better understand this effect of percent saturation on the soil strength, assume that a channel is being designed and soil sample $\frac{q_u}{2}$'s were tested at 80% rather than 95% saturation or greater, the latter assumed to form the S-Line of Figure 6-15 in TR-25 (Figures 1 and 2). As seen in Schematic b of Figure 6, the allowable τV for 80% might permit a bed slope of $S_o = 0.0015$ but the true allowable τV is in reality lower. Thus, the $S_o = 0.0015$ would produce an actual water tractive stress greater than the allowable and the channel soil would be over-stressed.

Is the soil parameter, q_u or $\frac{q_u}{2}$, an adequate indicator to predict the ability of a channel material to resist erosion?

Would an ephemeral stream be expected to follow a different S-line than that of a perennial stream?

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FIGURE 1 – PLOT OF TAVE V VS q_u

indicated to be stable although the point plotted far above the line. In this instance, the estimated n-value was considerably in excess of the average, due to heavy vegetation in the vicinity of the sampled surface. Additional data are needed in order to establish the validity of tractive power as defined, or some similar indicator of the relative stress and the opposing shear strength as related to stability.

Fig. 3 shows Fig. 2 alined with data on permeability of the same samples, also plotted in relationship to tractive power. Three channels on which some

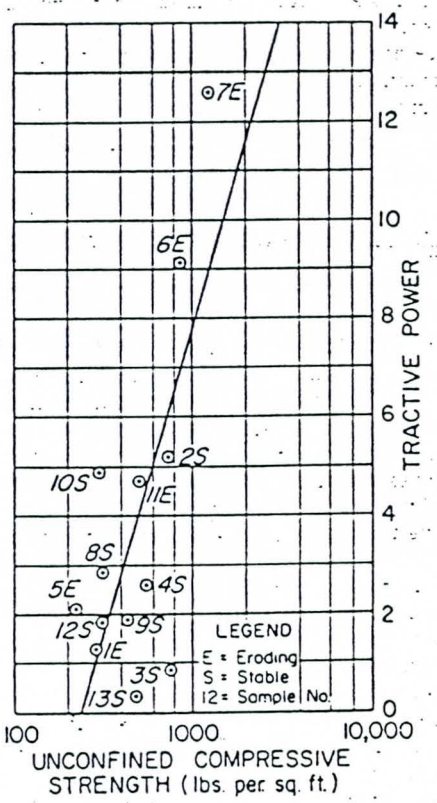

FIG. 2.—RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND TRACTIVE POWER

estimate of rates of erosion have been obtained are shown by the lines connecting sample numbers 5, 6, and 7. The stream measurements for the plotted points on Figs. 2 and 3 are given in Table 1. Data as shown on Fig. 3 can be graphically useful in evaluating the degree of channel deterioration that will occur if flows exceed the calculated stable limits.

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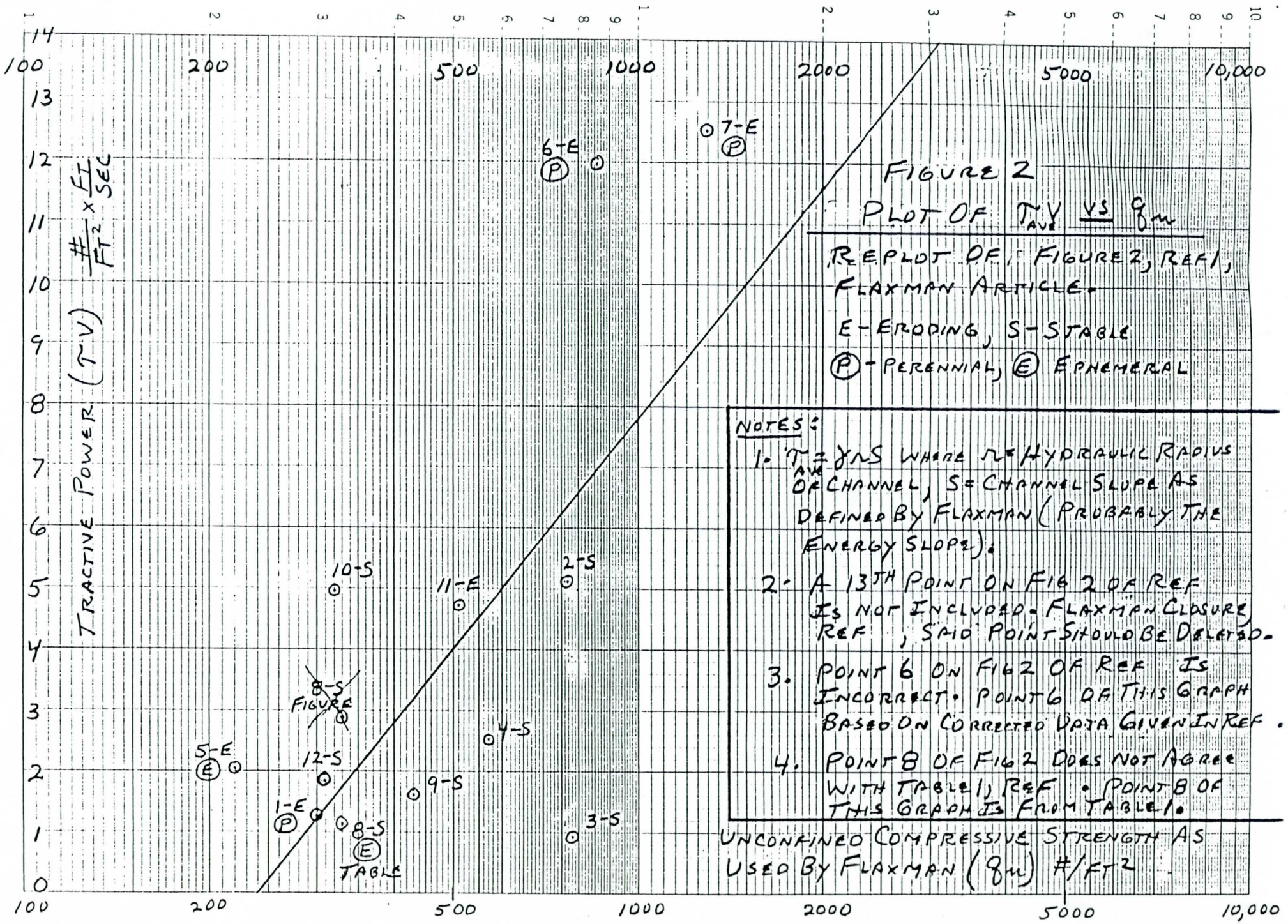


FIGURE 2
PLOT OF $\tau_{AVE} V$ VS q_u

REPLOT OF FIGURE 2, REF 1, FLAXMAN ARTICLE.

E - ERODING, S - STABLE

(P) - PERENNIAL, (E) EPHEMERAL

NOTES:

1. $\tau = \gamma R S$ WHERE R = HYDRAULIC RADIUS OF CHANNEL, S = CHANNEL SLOPE AS DEFINED BY FLAXMAN (PROBABLY THE ENERGY SLOPE).
2. A 13TH POINT ON FIG 2 OF REF IS NOT INCLUDED. FLAXMAN CLOSURE, REF , SAID POINT SHOULD BE DELETED.
3. POINT 6 ON FIG 2 OF REF IS INCORRECT. POINT 6 OF THIS GRAPH BASED ON CORRECTED DATA GIVEN IN REF.
4. POINT 8 OF FIG 2 DOES NOT AGREE WITH TABLE 1, REF. POINT 8 OF THIS GRAPH IS FROM TABLE 1.

TABLE 1 — FLAXMAN DATA — $\tau_{AVE} V$ USING τ_{AVE} & RSe

REFER TO GRAPH (FIGURE 2)

Sample and Site No. (1)	Stream (2)	(Use ASSe) Channel Slope, in feet per foot (3)	R Hydraulic Radius in feet (4)	V Average Velocity in feet per second (5)	REF. 1 — τ_{AVE} #/FT² (6)	REF 2	REF 1 — Tractive Power $\tau_{AVE} V$ (7)	REF 2	REF 2 — Total Unconfined Comp. Strength q_u (8)
1	Sutherlin Creek Douglas Co. Oreg.	0.00210	3.10	3.05	0.406		1.24		300
2	" "	0.00459	2.92	6.12	0.836		5.12		770
3	" "	0.00086	4.50	3.91	0.241		0.94		780
4	" "	0.00257	3.33	4.80	0.534		2.56		570
5	Magma Wash, Pinal Co., Ariz.	0.00250	2.72	4.83	0.424		2.05		220
6	Petawa Creek, Umatilla Co., Oreg.	0.01200	1.94	8.25 ~~6.34~~	1.453		11.99 ~~9.21~~		860
7	Wilson Creek, Kittitas Co., Wash.	0.00670	3.00	10.00	1.254		12.54		1300
8	Piner Creek, Sonoma Co., Calif.	0.00330	1.62	3.42	0.334		1.14		330
9	Adobe Creek, Lake Co., Calif.	0.00116	5.15	4.32	0.373		1.61		430
10	" "	0.00260	5.08	6.03	0.824		4.97		320
11	Willow Brook, Sonoma Co., Calif.	0.00200	4.76	7.95	0.594		4.72		510
12	Channel B, Alameda Co., Calif.	0.00300	2.35	4.19	0.440		1.83		310

$\tau = \gamma R S_e$ CALCULATIONS AGREE WITH REF. 2

AGREES WITH GRAPH VALUES

TABLE 2 — FLAXMAN DATA — $\tau_o V$ USING $(\tau_{REF} = \tau_o) = \gamma d S_t$

REFER TO GRAPH (FIGURE 3)

				REF 1	REF 3		TABLE 1 COL(8)/2	REF 3	COMPUTED					
Sample and Site No. (1)	Stream (2)	(USE AS Se) Channel Slope, in feet per foot (9)	V FPS (10)	m_t (11)	τ_o #/FT² (12)	$\tau_o V$ (13)	$q_w/2$ (14)	d FLOW DEPTH FT (15)	D_{75} IN (16)	D_{75} MM (17)	S_t (18)	S_t/S_e (19)	m_t/m (20)	m_t/m_e (21)
1	Sutherlin Creek Douglas Co. Oreg.	0.00210	3.05	0.0106	0.0307	0.0936	150	5.0	0.00499	0.1268	0.000098	0.047	0.0490	0.216
2	" "	0.00459	6.12	0.0106	0.148	0.905	385	4.7	0.00499	0.1268	0.000505	0.110	0.0320	0.331
3	" "	0.00086	3.91	0.0106	0.040	0.156	390	6.0	0.00499	0.1268	0.000107	0.124	0.0301	0.352
4	" "	0.00257	4.80	0.0091	0.0623	0.299	285	5.75	0.00200	0.0508	0.000174	0.068	0.0350	0.260
5	Magma Wash, Pinal Co., Ariz.	0.00250	4.83	0.010	0.0692	0.334	110	4.83	0.00352	0.0894	0.000230	0.092	0.0330	0.303
6	Petawa Creek, Umatilla Co., Oreg.	0.01200	8.25 ~~6.34~~	0.0088	0.1932	1.59	430	3.0	0.00163	0.0415	0.001032	0.086	0.0300	0.293
7	Wilson Creek, Kittitas Co., Wash.	0.00670	10.00	0.00936	0.550	5.50	650	6.0	0.00237	0.0601	0.001469	0.219	0.0200	0.468
8	Piner Creek, Sonoma Co., Calif.	0.00330	3.42	0.0142	0.1151	0.394	165	3.3	0.02665	0.7327	0.00056	0.169	0.0345	0.412
9	Adobe Creek, Lake Co., Calif.	0.00116	4.32	0.00946	0.0243	0.105	215	6.0	0.00252	0.0641	0.00006	0.056	0.0400	0.237
10	" "	0.00260	6.03	0.0102	0.074	0.446	160	6.0	0.00396	0.1007	0.000198	0.076	0.0370	0.276
11	Willow Brook, Sonoma Co., Calif.	0.00200	7.95	0.0107	0.174	1.38	255	7.0	0.00528	0.1341	0.000398	0.199	0.0240	0.446
12	Channel B, Alameda Co., Calif.	0.00300	4.19	0.0103	0.0697	0.292	155	4.3	0.00470	0.1067	0.000260	0.087	0.0350	0.294

$m_t = \frac{(D_{75})^{1/6}}{39}$ ∴ $D_{75} = (39 m_t)^6$

1" = 25.4 MM

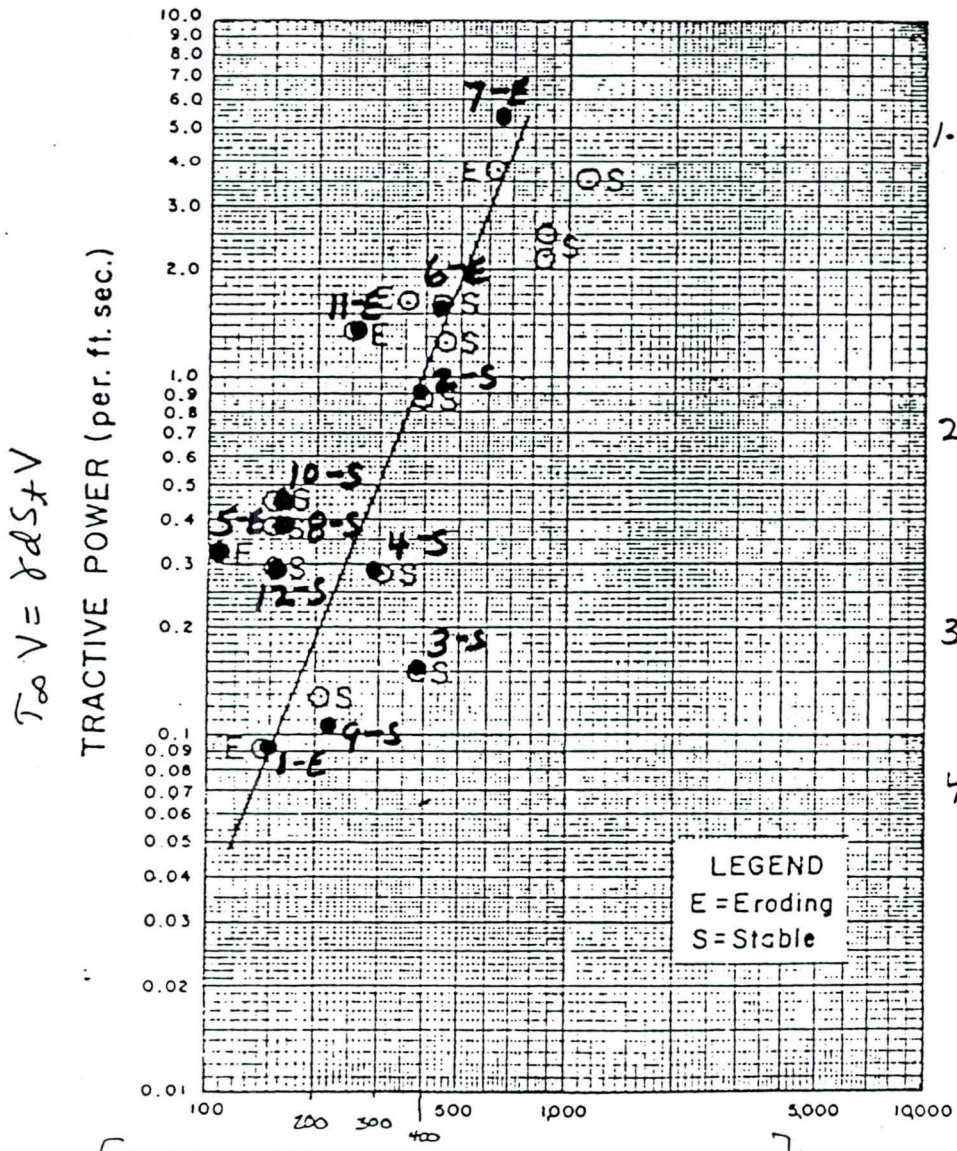
$\tau_o = \gamma d S_t$ ∴ $S_t = \frac{\tau_o}{\gamma d}$

$S_t = S_e \left(\frac{m_t}{m}\right)^2$ ∴ $m = m_t \left(\frac{S_e}{S_t}\right)^{1/2}$

FIGURE 3
PLOT OF $T_o V$ VS $\left(\frac{q_m}{2}\right)$

FLAXMAN DATA — GRAPH IN EARLIER EDITION OF TR-25

On Figure 6.1-1, plot tractive power $T_bV = 0.54$ and $T_sV = 0.43$ on the ordinate against the unconfined compressive strength of 410 lbs. per square foot on the abscissa. This channel is indicated to be stable since the plotting is to the right of the line, whereas a plotting to the left would indicate that erosion may occur, as based on field experience.



UNCONFINED COMPRESSIVE STRENGTH (lbs. per sq. ft.)

$q_m/2$

NOTES:

1. $T_{REF} = \gamma d S_f$ WHERE d = CHANNEL DEPTH. AND S_f = PARTICLE (GRAIN) ENERGY SLOPE.
2. POINTS 1, 2, 3, 4, 5, 6, 11, AND 12 MATCH EXISTING PLOTTED POINTS.
3. POINTS 8 AND 10 ARE A CLOSE MATCH.
4. POINTS 7 AND 9 DO NOT MATCH.

FIGURE 6·1-1
UNCONFINED COMPRESSIVE STRENGTH AND TRACTIVE POWER AS RELATED TO CHANNEL STABILITY

$q_o/2$

TABLE 3 — FLAXMAN DATA — τ_{REF} V USING $\tau_{R*} = \gamma R_* S_e$

REFER TO GRAPH (FIGURE 4)

Sample and Site No. (1)	Stream (2)	(Use As Se) Channel Slope, in feet per foot (22)	V FPS (23)	$\frac{V^3}{g\gamma S_e}$ (24)	D_{75} IN (25)	D_{75} FT (26)	$\frac{V}{\sqrt{g K_s S_e}}$ $K_s = D_{75}$ (23)	$\frac{V}{\sqrt{\tau/\rho}}$ (28)	τ_{R*} #/FT² (29)	τ_{R*} V (30)	$q_m/2$ #/FT² (31)	R_* FT (32)
		REF 1		COMPUTED	SEE TABLE 2		COMPUTED				TABLE 1 COL (8)	COMPUTED
1	Sutherlin Creek Douglas Co. Oreg.	0.00210	3.05	4.20×10^7	0.00499	0.000416	573.1	23.4	0.0330	0.101	150	0.2518
2	" "	0.00459	6.12	1.55×10^8	0.00499	0.000416	780.5	25.7	0.1100	0.673	385	0.3841
3	" "	0.00086	3.91	2.16×10^8	0.00499	0.000416	1152.0	25.6	0.0453	0.177	390	0.8441
4	" "	0.00257	4.80	1.34×10^8	0.00200	0.000167	1291.2	25.5	0.0687	0.330	285	0.4284
5	Magma Wash, Pinal Co., Ariz.	0.00250	4.83	1.40×10^8	0.00352	0.000293	994.5	25.2	0.0713	0.344	110	0.4571
6	Petawa Creek, Umatilla Co., Oreg.	0.01200	8.25 ~~6.84~~	1.45×10^8	0.00163	0.000136	1138.1	25.4	0.2047	1.689	430	0.2734
7	Wilson Creek, Kittitas Co., Wash.	0.00670	10.00	4.64×10^8	0.00237	0.000198	1530.0	27.3	0.2603	2.603	650	0.6226
8	Piner Creek, Sonoma Co., Calif.	0.00330	3.42	3.76×10^7	0.02885	0.00240	214.2	23.2	0.0422	0.144	165	0.2049
9	Adobe Creek, Lake Co., Calif.	0.00116	4.32	2.16×10^8	0.00252	0.00021	1542.5	26.4	0.0519	0.224	215	0.7170
10	" "	0.00260	6.03	2.62×10^8	0.00396	0.00033	1147.2	25.7	0.1068	0.644	160	0.6583
11	Willow Brook, Sonoma Co., Calif.	0.00200	7.95	7.80×10^8	0.00528	0.00044	1493.5	27.3	0.1645	1.308	255	1.318
12	Channel B, Alameda Co., Calif.	0.00300	4.19	7.61×10^7	0.00420	0.00035	720.6	24.5	0.0567	0.238	155	0.303

$g = 32.2$ FT/SEC²
ASSUME $T = 75°F$
$\gamma = 1.0\times10^{-5}$ FT²/SEC

K_s SHOULD BE D_{65} IN FT.

$\rho = 1.94$ SLUGS/FT³

$\tau_{REF} = \frac{V^2 \rho}{\left(\frac{V}{\sqrt{\tau/\rho}}\right)^2}$

$\tau_{R*} = \gamma R_* S_e$
$\therefore R_* = \frac{\tau_{R*}}{\gamma S_e}$

FIGURE 4
PLOT OF $\tau_{R \cdot x} \cdot V$ VS $\left(\frac{q_u}{2}\right)$

FLAXMAN DATA – GRAPH IN CURRENT EDITION OF TR-25

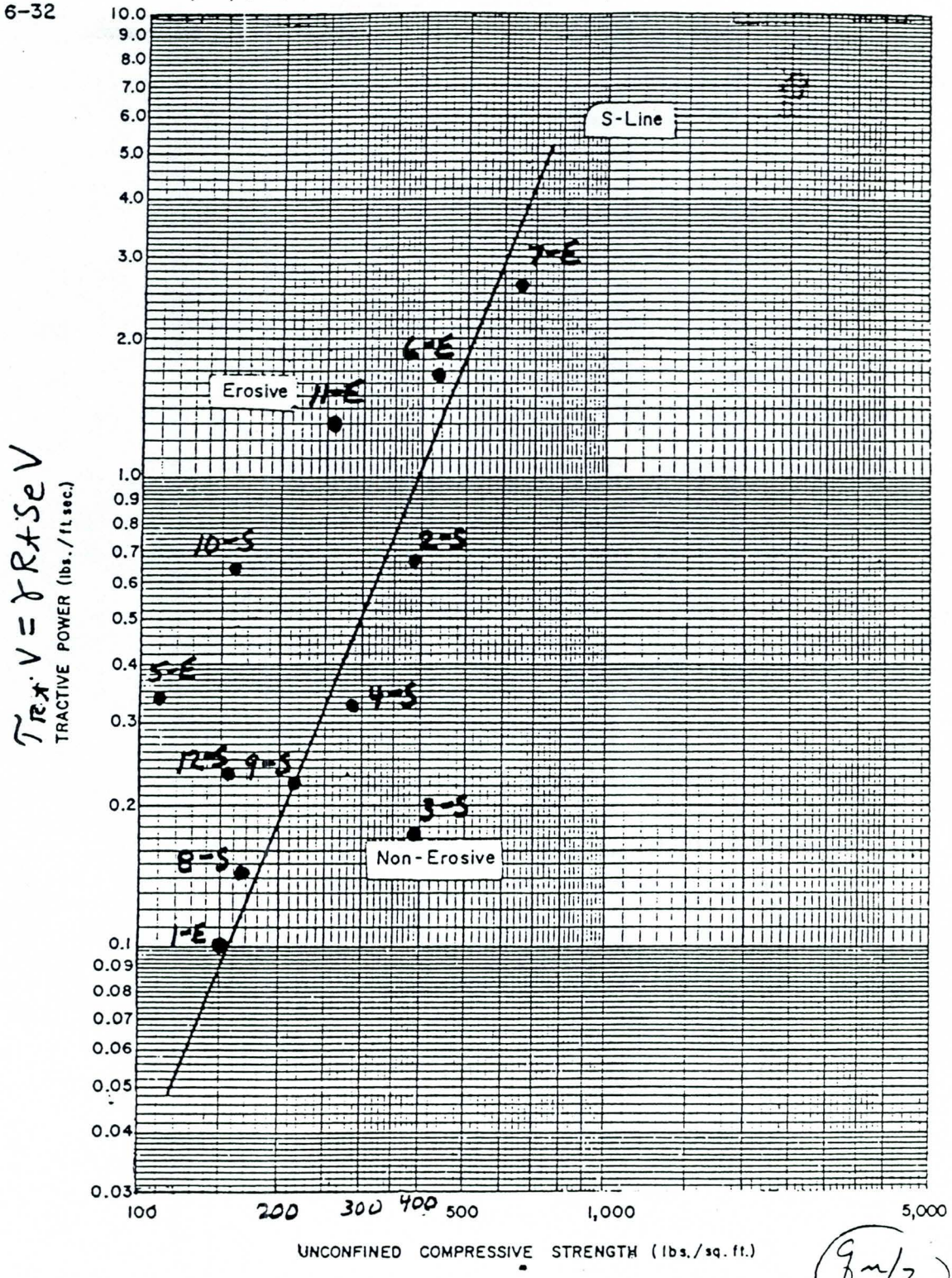


Figure 6-15

Unconfined Compressive Strength And Tractive Power As Related To Channel Stability

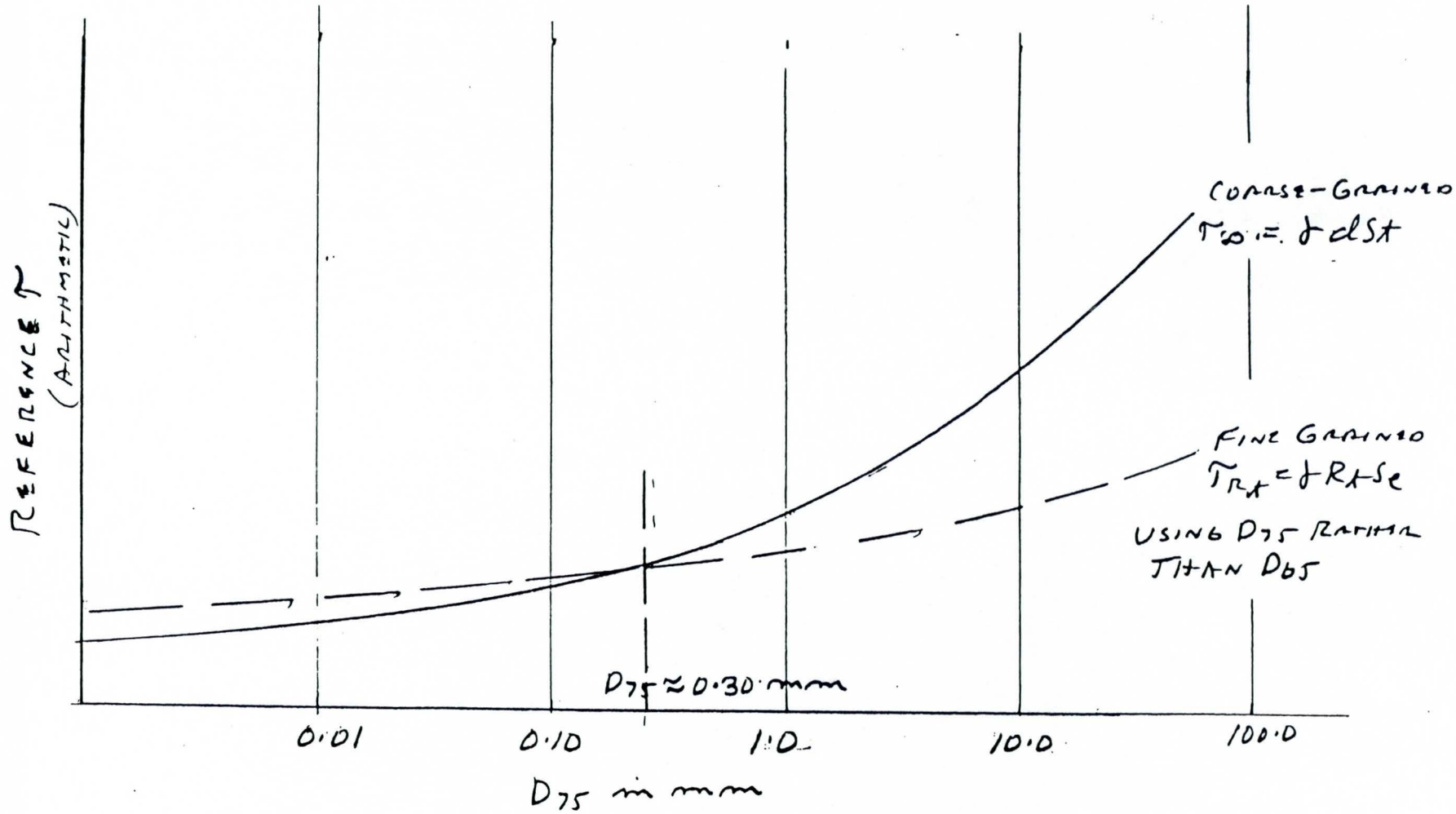
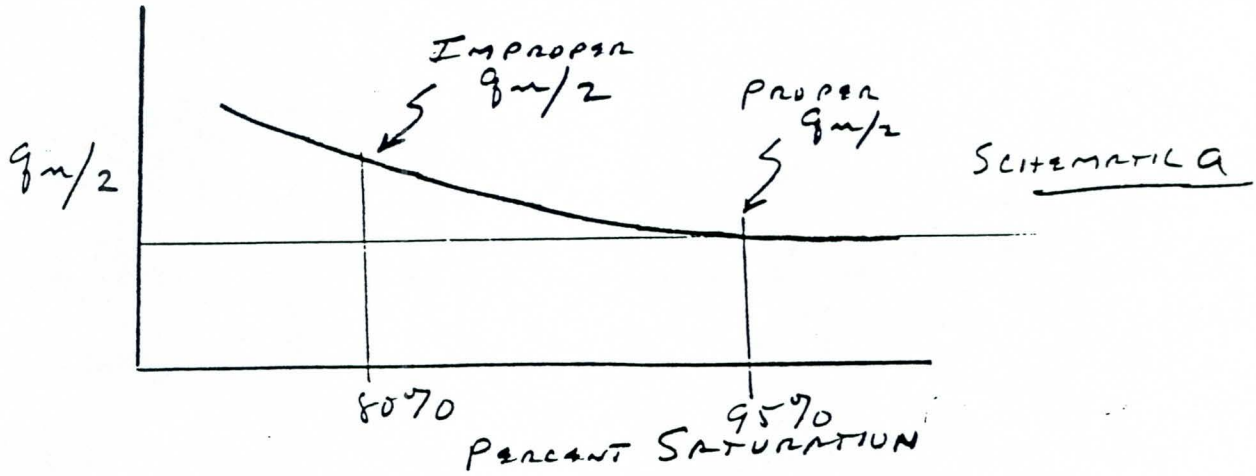
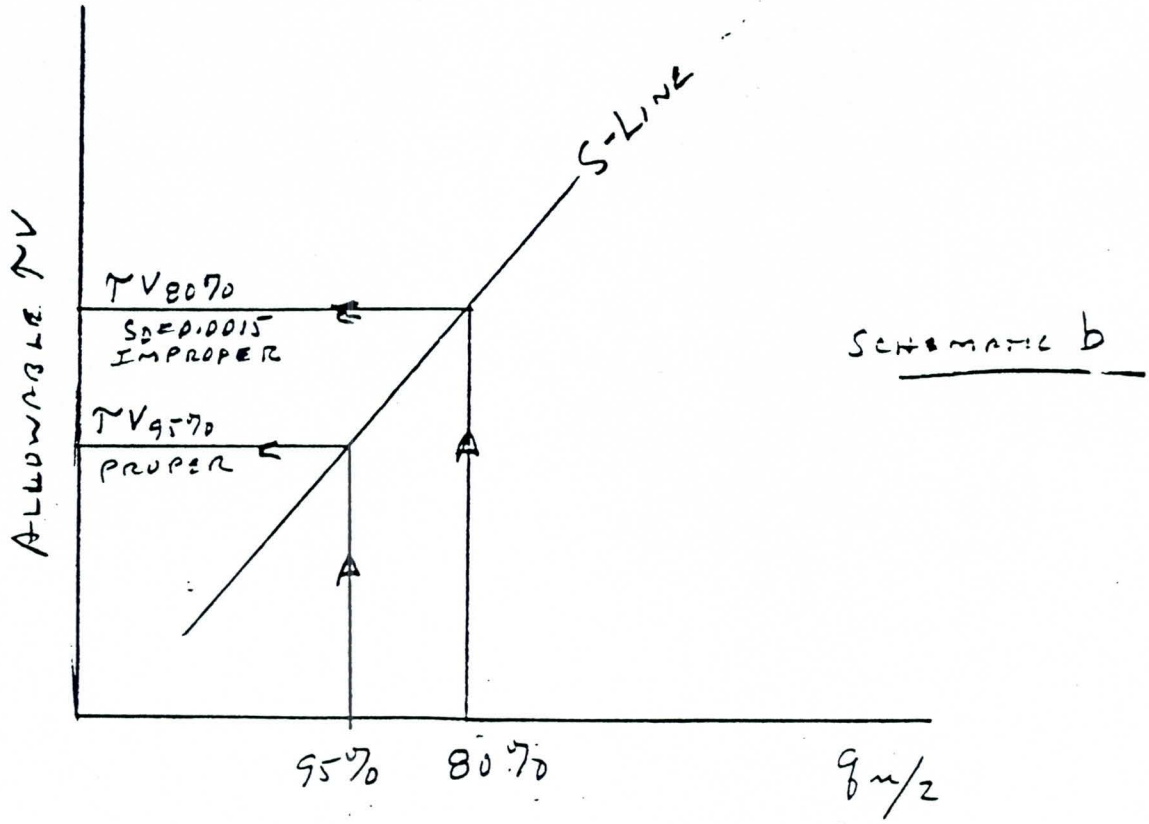


FIGURE 5 — SCHEMATIC OF RELATIVE TRACTIVE STRESS VS D75

FIGURE 6

EFFECT OF PERCENT SATURATION OF MEASURED $q_m/2$ OF CHANNEL MATERIAL





RWCD 1&2

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola Ave.
Suite 200
Phoenix, Arizona 85012

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

FDT:nn

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?

STATE		PROJECT RWCD Floodway		
BY [illegible]	DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT Hydraulic Analysis				SHEET 1 OF 5

Formulas Used:

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

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

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

$R = A/P$, ft

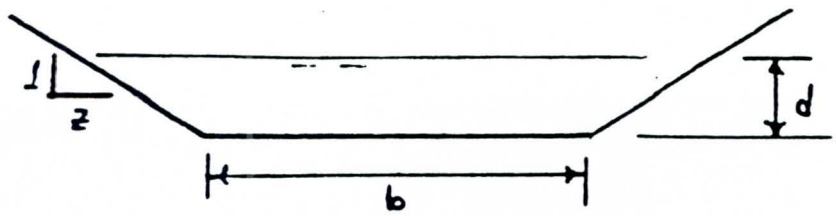
$\tau = \gamma R S_o$, $\#/ft^2$

$v = Q/A$, fps

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

$d_{50} \geq 11 \cdot d \cdot S_o$ (shields initiation of motion) , 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
	2326	200	3	0.0003	0.027	historical



SCS-ENG-823 Rev. 8-69 SOIL CONSERVATION SERVICE

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

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

capacity

Q cfs	n –	d ft.	R ft.	v fps	τ N/ft²	τ·v (ft·N/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 ch.	0.027	2.87	2.74	4.18	0.2568	1.073
[illegible]); 31%	0.030	3.05	2.91	3.91	0.2727	1.067
in 4 [illegible])	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
1 – S_f
2 – n
3 – d
4 – A

5 – P
6 – R
7
8
9

d, S_o, Q – Lbl A
n – RLS or Lbl B

DATE		PROJECT RCWD Floodway		
BY	DATE 5/23/85	CHECKED BY	DATE	JOB NO.
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	τ·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	9.28	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 stringer section to armor.

SCS-ENG-623 Rev. 8-69 SOIL CONSERVATION SERVICE

STATE		PROJECT RWCD Floodway		
BY [illegible]	DATE 5/23/85	CHECKED BY	DATE	JOB NO
SUBJECT Q16 vs max. S_o Analysis				SHEET 4 OF 5

$\tau \cdot v$ (ft-#/sec/ft²)

Q16 / S_o	43.5	40	30	20
0.0007				0.759
0.0006	1.304	1.211	0.934	0.647
0.0005	1.075	0.999	0.773	—
0.0004	0.848	0.788	0.611	—
0.0003	0.624	0.581	0.451	—
0.0002	0.404	—	—	—

	SKa	Riley
R.C.	1464+00	1195.5
C.C.	1160+22	1240.4
	30,378	44.9

S_o = 44.9 / 30378 = 0.00148

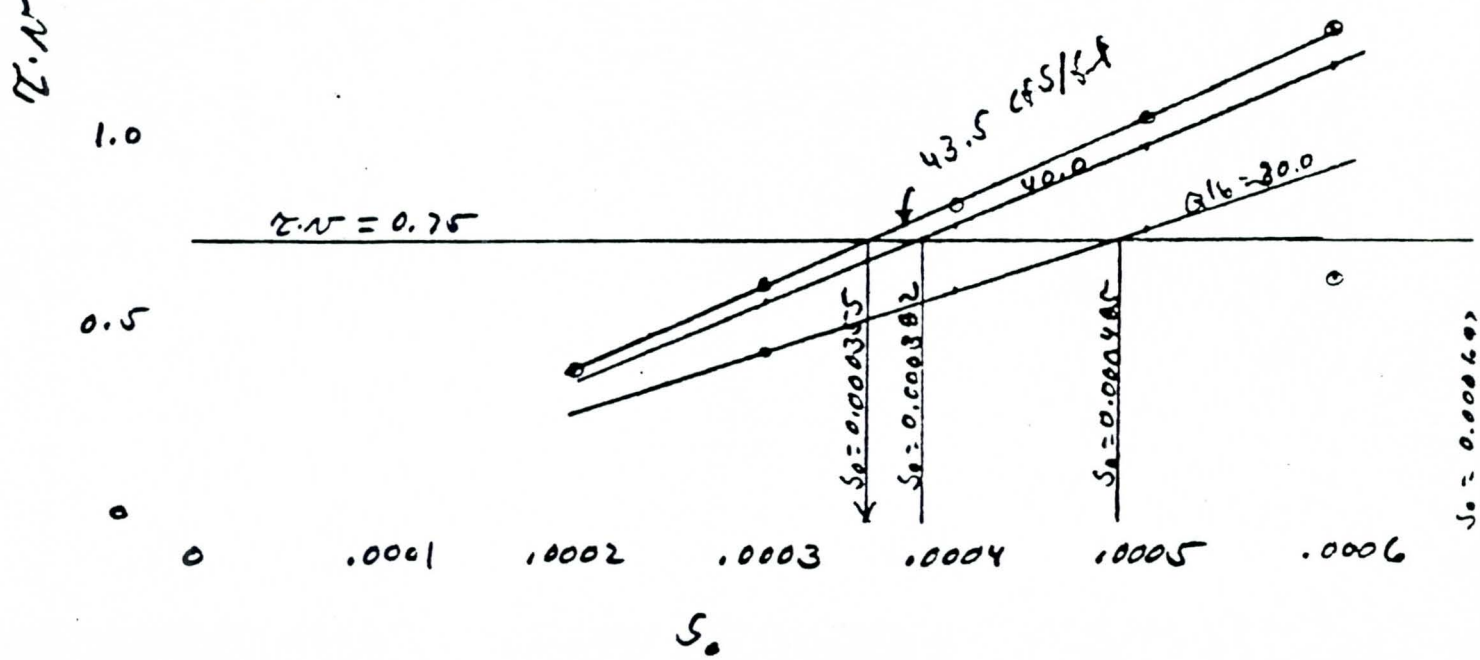
Δh_{OH} = 30378 * 0.000355 = 10.8'

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

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT RWCD		
BY [illegible]	DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT Armor material analysis				SHEET 5 OF 15

$d_{75} = 2''$

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

$d = 6.24'$

$R = 5.70'$

$v = 6.38$ fps

$\tau = 0.5332$ #/ft^2

$\tau \cdot v = 3.401$ ft-#/sec/ft^2

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

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

use min. 4" (larger)

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

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

$d_{max} = 3''$

United States
Department of
Agriculture

Soil
Conservation
Service

West National Technical Center
511 N. W. Broadway, Room 547
Portland, Oregon 97209-3489

RWCD 1 & 2

Subject: ENG - Stability and Erosion Analysis of RWCD Floodway, Reach 1 and Reach 2, Williams-Chandler WPP, Arizona

Date: June 21, 1985

To: Ralph Arrington, State Conservation Engineer, SCS, Phoenix, Arizona

File Code:

We have reviewed the findings prepared by Fred Theurer following his visit to the project and his discussions with your staff. Fred is to be commended on his forthright appraisal of the situation.

Significant questions have been raised by this report which need resolution.

1. The main problems identified are toe erosion of the channel banks, local scour downstream and deposition in rock lined sections, and rilling of the channel banks. It was concluded that general bed degradation is the cause of the toe erosion observed. The channel cross-sections taken to date document local scour and deposition downstream of the rock lined sections. Additional cross sections are necessary to verify that general bed degradation is occurring.

2. The assertion that the reference tractive stress procedure as used in TR-25 is in error or is inappropriate for the situation here is open to question. Reference tractive stress is a measure of the portion of the total resistance to flow, or energy loss due to turbulence, attributed to the bed grain roughness. The remainder of the losses are attributed to the larger scale turbulence resulting from bed forms, vegetation, debris, or other factors. Mannings Equation is universally used in practice for determination of energy loss in open channel design. The Mannings Equation "n" value selected is the index for total energy losses due to all factors. The reference tractive stress procedure uses the friction formula presented in USDA Technical Bulletin No. 1026 to determine losses due to grain roughness. This formula is based on the Von Karman velocity distribution theory with correction constants developed by Keuligan and covers the spectrum from smooth to turbulent flow.

With this approach, the hydraulic radius is the index of relative losses used and is divided into portions R' and R" representing losses due to particle and form roughness respectively. The total hydraulic radius for a given channel geometry, flow, and energy slope is dependent on the "n" value selected. The

design "n" values for as built and seasoned condition are selected by procedures outlined in NEH 5 or from other standard references.

3. The tractive stresses used for development of the tractive power design chart and procedure contained in the TR-25 are the tractive stresses with respect to the bed grain roughness. Use of this chart with tractive power based on total tractive stress is erroneous. The conclusion that computed total tractive power for Reach 1 is 4.5 times the allowable is therefore wrong.

4. In light of the above, the tractive power approach does not predict severe general bed degradation for this channel.

5. Dispersive soil behavior needs to be verified by both laboratory testing and field observation. Percent sodium vs. TDS in pore-water extract, cited in the report, is not a reliable basis alone for predicting dispersive soil behavior. Pinhole and "double hydrometer" testing should be considered as well.

Additional work is needed to determine the nature of the problem, whether excessive general bed erosion is occurring, the extent of influence of soil chemistry, and to recommend appropriate engineering solutions.

Other comments include:

1. At the time the floodway was designed and reviewed, the procedures for hydraulic design were extensively discussed. The tractive power procedure in its application here was recognized as a pioneering approach and that the resulting channel may have a relatively high maintenance requirement.

2. The photographs seem to indicate that the local scour downstream of the rock lined sections are in part due to concentration of flow between debris piles deposited in the rock riprap.

3. Erosion at the toe of the channel banks is a typical first mode of failure in a constructed earth channel. In many instances the channel can be stabilized by protecting the lower banks by installing riprap or other slope protection.

4. Low flows in a wide channel such as this can be expected to develop meandering low flow channels with resulting areas of attack and deposition.

5. Billing of exposed earth banks is typical for all construction in this climatic area. The cost of preventing it needs to be weighed against the problems it presents.

We believe an engineering investigation committee needs to be formed to study the appropriateness of current SCS channel design procedures as applied to this job. A field review by the committee needs to be conducted as soon as

possible and the sponsors encouraged to perform O and M following the field review as needed to prevent additional damage from other flood events.

Donald E. Wallin Acting

JACK C. STEVENSON
Head, Engineering Staff

cc:
Don Basinger, Director of Engineering
SCS, Washington, D.C.
Verne Bathurst, State Conservationist,
SCS, Phoenix, Arizona
Fred Theurer, Civil Engineer,
ARS, Fort Collins, Colorado

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola Ave.
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Phoenix, Arizona 85012

Aubry

FLOOD CONTROL DISTRICT
RECEIVED
JUN 15 '85

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

Date: May 23, 1985

	CH ENG		HYDRO
	ASST		LMgt
	ADMIN		SUSP
	C & O		FILE
I	ENGR		DESTROY
	FINANCE		
REMARKS			

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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STATE		PROJECT RWCD Floodway		
BY [illegible]	DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT Hydraulic Analysis				SHEET 1 OF 5

Formulas Used:

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

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

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

$R = A/P$, ft

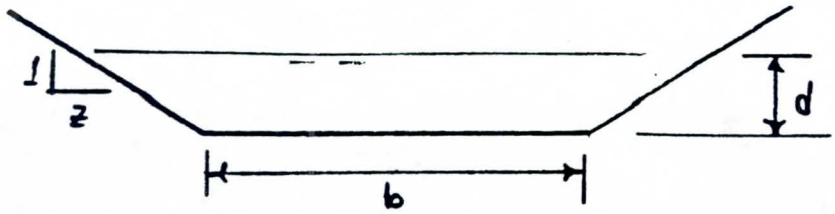
$\tau = \gamma R S_o$, $\#/ft^2$

$v = Q/A$, fps

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

$d_{50} \geq 11 \cdot d \cdot S_o$ (shields initiation of motion) , 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 LJT	DATE 5/21/85	CHECKED BY	DATE	JOB NO.
SUBJECT Flow Analysis				SHEET 2 OF 5

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

capacity

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
1 - S_f
2 - n
3 - d
4 - A

5 - P
6 - R
7
8
9

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

DATE		PROJECT RCWD Floodway		
BY	DATE 5/23/85	CHECKED BY	DATE	JOB NO.
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	τ·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

or $\tau_{all} = 4 \cdot d_{50} = 4 \cdot \left(\frac{4}{12}\right) = 1.33$ #/ft² $= 0.0369'$

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 stringers below to armor.

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STATE		PROJECT RWCD Floodway		
BY [illegible]	DATE 5/23/85	CHECKED BY	DATE	JOB NO
SUBJECT Q16 vs max. S_o Analysis				SHEET 4 OF 5

$\tau \cdot v$ (ft-#/sec/ft²)

S_o \ Q16	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	0.624	0.581	0.451	—
0.0002	0.404	—	—	—

	SKa	Riley
R.C.	1464+00	1195.5
C.C.	1160+22	1240.4
	30,378	44.9

S_o = 44.9/30378 = 0.00148

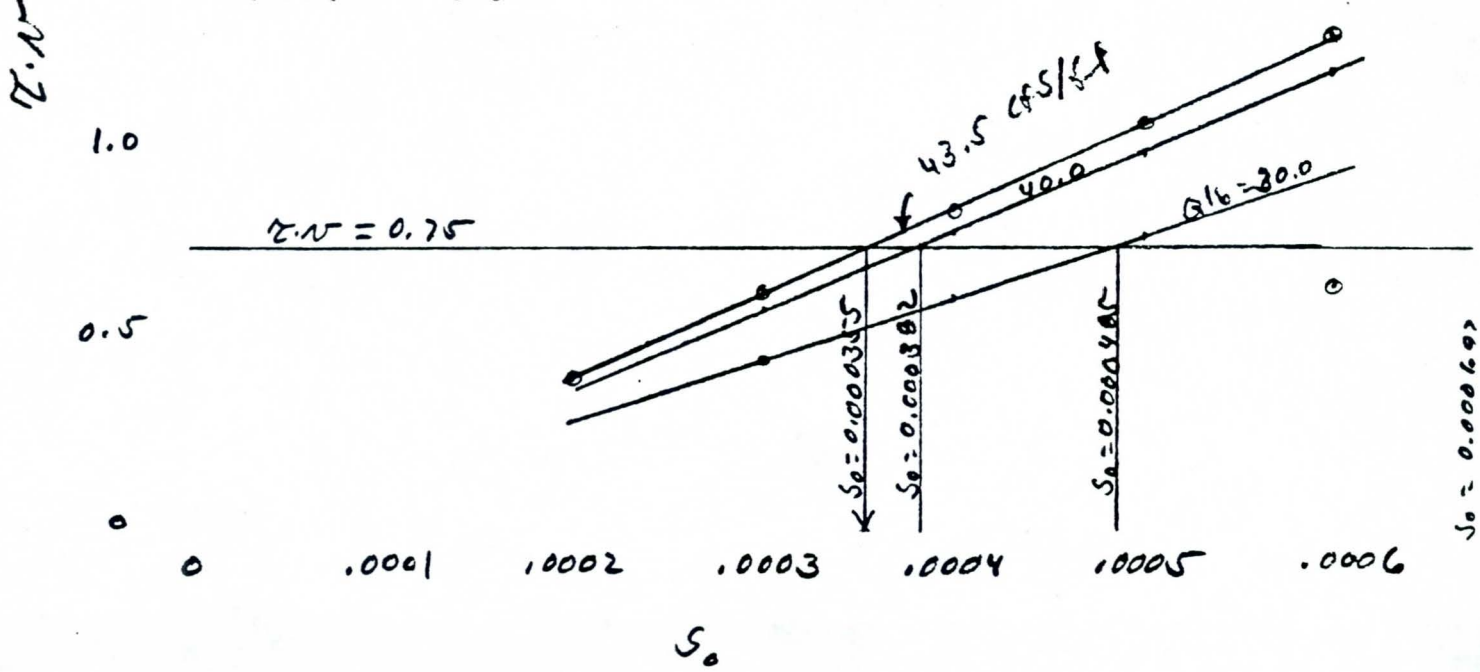
Δh_{oil} = 30378 * 0.000355 = 10.8'

Cut = 44.9 - 10.8 = 34.1'

three rock liners + 2 dip averaging = 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 15

$d_{75} = 2''$

$m = 2''/39 = 0.029$

$d = 6.24'$

$R = 5.70'$

$v = 6.38$ fps

$\tau = 0.5332$ #/ft^2

$\tau \cdot v = 3.401$ ft-#/sec/ft^2

shields) $d_{50} \geq 11 \cdot d \cdot S_o = 11 \cdot 6.24 \cdot 0.0015 = 0.103'$
$= 1.235''$
$= 31.4$ mm

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

use min. 4" layer

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

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

$d_{max} = 3''$