REPORT OF INVESTIGATING COMMITTEE
FOR
WILLIAMS - CHANDLER WATERSHED
RWCD FLOODWAY, REACH I
PINAL COUNTY, ARIZONA

United States
Department of
Agriculture

Soil
Conservation
Service

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

October 21, 1986

Dan Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 W. Durango St.
Phoenix, AZ 85009

Dear Dan:

We are submitting for your information and use a copy of the engineering report prepared as a result of erosion and scour occurring within the RWCD Floodway Reach I and lower Reach 2, Williams-Chandler WPP. Design of repairs will be made consistent with the priorities that have been previously set.

We appreciate the maintenance work that has been completed by the FCD to minimize further damages to the bottom and banks of the floodway. Please keep us informed of any elements of consideration that must be made to keep the floodway operational or that may influence the schedule for repair.

Sincerely,

W. Wayne Killgore Acting For

Verne M. Bathurst
State Conservationist

Copies of the Engineering Report on file in ~~Engineering~~ Div.
C & O Div.

File: SL1.5

JW

United States
Department of
Agriculture

Soil
Conservation
Service

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

Subject: ENG - Engineering Report, RWCD Floodway, Reach I, Williams-Chandler WPP, Arizona

Date: August 15, 1986

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

File code:

We have reviewed the report and find it technically acceptable.

Donald E Wallin

DONALD E. WALLIN
Head, Engineering Staff

cc:
Paul J. Monville, Acting Head, Design Unit, Engineering Staff, WNTC
Donald L. Basinger, Director Engineering Division, SCS, Washington, D.C.

United States Department of Agriculture

Soil Conservation Service

USDA, Soil Conservation Service
201 E. Indianola, Suite 200
Phoenix, Arizona 85012

Subject: ENG: Engineering Report Review
RWCD Floodway Reach 1

Date: June 24, 1986

To: Ralph M. Arrington
State Conservation Engineer

File code: 210

I have reviewed the Engineering Report for RWCD Floodway Reach 1 from the design and construction viewpoints and have no comments.

John L. Sullivan

John L. Sullivan
State Construction Engineer

United States Department of Agriculture

Soil Conservation Service

P.O. Box 2890
Washington, D.C.
20013

Subject: ENG - Investigating Committee RWCD Reach I Report

Date: November 15, 1985

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

File Code:

Under separate cover, I am sending you ten copies (two in a envelope and eight in a box) of the investigating committee's final report for the RWCD Reach I, Williams-Chandler WPP. I am sending each of the committee members a copy of the report also.

In accordance with National Engineering Manual §504.06, you will need to coordinate the reviews required and the technical acceptance. When this is accomplished, please send each of the committee members a copy of all review comments so that we may attach them to our copies of the report. If there are any questions, please call me or any of the other members of the committee.

Thanks again for the help that you and your staff have given us. We sincerely hope that this report will be helpful to you.

John A. Brevard

JOHN A. BREVARD
Civil Engineer
Design Unit, Engineering Division

cc:
Donald L. Basinger, Director, Engineering Division, SCS, Washington, D.C.
Robert Pasley, Assistant Director for Engineering Technology Development, Engineering Division, SCS, Washington, D.C.
Edwin S. Alling, Head, Design Unit, Engineering Division, SCS, Washington, D.C.
Gary L. Conaway, Design Engineer, WNTC, SCS, Portland, Oregon
Dr. Lewis J. Mathers, Professor, Civil Engineering Department, Villanova University, Villanova, Pennsylvania
Charles H. McElroy, Soil Mechanics Engineer, SNTC, SCS, Fort Worth, Texas

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
PINAL COUNTY, ARIZONA
NOVEMBER 1985

ABSTRACT OF ENGINEERING REPORT

Excessive erosion of the compacted earth liner lead to erosion of the toe of the banks. Excessive scour of the channel liner occurs downstream of the riprap lined sections of the channel. Jug holes are very common in the diking along the channel. Excessive deposition is evident in the channel.

Location: Nine miles south of Chandler, Arizona.

Type of Facility: Single-purpose floodwater control channel.

Job Class: Class VII (NEM Part 501)

Size: Bottom width = 200 feet
Side slopes = 3:1
Depth = 8 - 8.25 feet

Date of Installation: November 1981

The channel bottom eroded excessively, and this caused toe erosion of the channel banks. The most severe toe erosion is a vertical cut of 4 to 4.5 feet. The soil liner of the channel also eroded immediately downstream of the riprap reaches with the maximum erosion depth being 3 to 3.5 feet. The erosion which occurred was associated with discharges which are much smaller than the design discharge.

The channel experienced some erosion and deposition before the December 1984 storm, but maintenance had been such that the channel was considered well maintained.

The concern is that if a small discharge could cause this damage, the design discharge might cause complete failure of the channel.

The principal cause of the excessive erosion is the soil material used in the channel liner. The soil is dispersive and moderately to highly plastic with a high cracking potential. The liner did crack extensively, and the channel flows penetrated these cracks to significant depths and created uplift pressures that lifted the soil blocks into the moving water. The dispersive clays accelerated this action since they tend to erode very easily.

A contributing cause of the erosion may be the maintenance grading operation which increased the loss of lining material and increased the sediment load because the loose material was left in the channel. The increased sediment load increased deposits which resulted in a flow meander pattern with increased attack.

At the time of this design, Arizona had not experienced problems with dispersed clays. Also, the designer believed that the dispersive clays were limited to the most downstream 1000 feet of the channel. The design did not consider the consequences of using highly dispersive clays or highly crack-prone clays as compacted liner material.

The tractive power procedure used in the design of this channel is basically untested. No documentation exists for the performance of channels with compacted earth liners designed using the tractive power procedure. The tractive power procedure is given in TR-25 with few cautions, and if Figure 6-1 shows it to be an acceptable method for the site conditions, it is taken as an acceptable alternative to the other design procedures in TR-25. Its inclusion in an SCS technical publication gives the user confidence in this procedure.

Remedial Treatment: For final remedial treatments used or for additional information contact: (Include copy of the abstract with request)

State Conservation Engineer
Soil Conservation Service
201 E. Indianola, Suite 200
Phoenix, Arizona 85012

Problem Category:	Earth - External Erosion In channels Dispersion	Site Name:	Williams-Chandler Watershed, RWCD Floodway, Reach I
Practice Standard:	582	State:	Arizona

REPORT OF INVESTIGATING COMMITTEE

FOR

WILLIAMS - CHANDLER WATERSHED

RWCD FLOODWAY, REACH I

PINAL COUNTY, ARIZONA

BY

John A. Brevard

John A. Brevard

Lewis J. Mathers

Dr. Lewis J. Mathers

Gary L. Conaway

Gary L. Conaway

Charles H. McElroy

Charles H. McElroy

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

PINAL COUNTY, ARIZONA

NOVEMBER 8, 1985

ENGINEERING REPORT

Project: Williams-Chandler Watershed

Location: Pinal County, Arizona

Name of Structure: R.W.C.D. Floodway - Reach I

Appropriation: WF-08

General Description of Problem: Based on this committee's observations and investigation, the main problems identified are:

1. Excessive toe erosion of the channel banks in the soil-lined portions of the floodway,

2. Local scour downstream from most of the rock-lined sections,

3. Rilling and gully development on most of the channel banks, and

4. Jug hole development on the crest and banks of the dikes all along Reach I, with the worst portions being the lower 1000 feet of the reach.

Authority

The committee appointments are contained in a letter dated June 28, 1985 from Verne M. Bathurst, State Conservationist, to Donald L. Basinger, Director Engineering Division, NHQ; Arthur B. Holland, Director NENTC; Jerry S. Lee, Director SNTC; and George C. Bluhm, Director WNTC. A copy of this letter is included as Attachment 1.

Composition of Committee

John A. Brevard, Civil Engineer, NHQ - Chairman
Gary L. Conaway, Hydraulic Engineer, WNTC
Dr. Lewis J. Mathers, Professor, Villanova University
Charles H. McElroy, Soil Mechanics Engineer, SNTC

INVESTIGATION

Site Inspection

The committee traveled to the floodway on Tuesday, July 30, 1985 to observe the problem areas. We inspected the floodway from approximately Station 1160 + 00 in Reach II to the downstream end of Reach I. Soil samples were taken at selected locations to gather additional data. Photographs were also taken to illustrate several key conditions and to serve as illustrations of the problems and their possible causes.

Review of Pertinent Records and Documents

The committee reviewed the following documents:

1. Workplan and supplements
2. Design report including the geological site investigation and soil testing records
3. Construction records
4. Report of sedimentation problems on RWCD Reaches I and II prepared by Aubrey C. Sanders, Jr., State Geologist, SCS, dated May 10, 1985 (Attachment 10)
5. Correspondence file
6. Report of stability and erosion analysis on RWCD Reaches I and II prepared by Dr. Fred Theurer, Soil Conservationist, SCS, Fort Collins, Colorado, dated May 23, 1985 (Attachment 2)
7. Correspondence prepared by the West NTC in response to Dr. Theurer's report dated June 21, 1985 (Attachment 3)
8. Soil testing report prepared by Sargent, Hauskins, and Beckwith - Consulting Soil and Foundation Engineers, Phoenix, dated June 29, 1978
9. Summary of soil testing done by University of Arizona in conjunction with a contractor's claim
10. Summary of water content and dry unit weights also prepared for the contractor's claim
11. As-built plans and construction specifications
12. Survey information comparing present channel cross-sections with as-built conditions
13. Chapter 6 of Technical Release No. 25, Design of Open Channels, October 1977
14. Two papers prepared by Elliott M. Flaxman:
    "A Method of Determining the Erosion Potential of Cohesive Soils" presented at the Symposium on Land Erosion, October 1962.
    "Channel Stability in Undisturbed Cohesive Soils", Journal of the Hydraulics Division, American Society of Civil Engineers, March 1963 and three discussions and Flaxman's closure

Interviews

The committee interviewed the following SCS personnel:

Ralph M. Arrington, State Conservation Engineer
John L. Sullivan, State Construction Engineer and Design Engineer for Reach I
Aubrey C. Sanders, Jr., State Geologist

Susanne M. Leckband, Civil Engineer
William E. Payne, Jr., State Design Engineer

Charles McElroy held discussions with the following personnel while supervising the field dispersion testing of the collected soil samples:

Robert H. Carr, Construction Inspector
Gary L. Mason, Civil Engineering Technician
Donald K. Hack, Civil Engineering Technician

Summary of the Facts

Reach I was completed about four years ago and has experienced three appreciable flows. The last flow occurred in December 1984 and produced a flow depth in the channel of about three feet. Based on Theurer's calculations (Attachment 2), the maximum discharge for this flow was approximately 2500 cfs or about 30% of the design discharge. This is estimated to be the maximum flow to date in the floodway. This discharge caused local scour immediately downstream of the riprap sections and general scour of the earth portions of the channel which resulted in scour at the toe of the banks. The maximum depth of localized erosion was about 3 1/2 feet and the maximum depth of general erosion was about 2 feet.

Since this erosion was considerably greater than expected for this discharge, at the request of Jack Stevenson, Head of Engineering Staff at the West NTC, and Ralph Arrington, SCE, Dr. Fred Theurer inspected the site, reviewed the design, analyzed the stability of Reaches I and II, and prepared a report on his findings. This report dated May 23, 1985 is included as Attachment 2. After reviewing Dr. Theurer's report, Jack Stevenson wrote to Ralph Arrington saying that "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." Also, Mr. Stevenson suggested that an investigation committee be formed. This letter is Attachment 3. Verne Bathurst, State Conservationist, appointed this committee to "study the appropriateness of current SCS channel design procedures as applied to this job..." and to prepare a report.

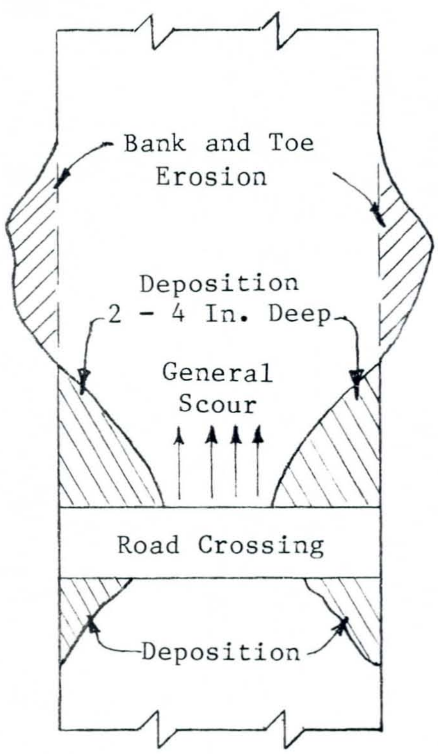
Description of designed channel.--The Reach I portion of the floodway is approximately 4.6 miles long and begins just downstream from the State Highway 87 bridge and outlets into the Gila River. The design consisted primarily of the excavation of natural materials from the channel and replacement to design grade with about 16,500 lineal feet of compacted earth liner (3 feet thick in the bottom) on both the bottom and sides of the floodway. The earth liner material is soil with a greater unconfined compressive strength than the naturally occurring soil. Approximately 4700 lineal feet of loose rock riprap (1 foot thickness) was used in the curved sections, at bridges, dip crossings, and transitions for additional erosion resistance. In other minor sections, the earth liner was used only on the sides and in some sections the exposed boundaries consisted of natural materials.

Committee visit to channel.--The committee observed the following on its visit to the channel:

1. General bed scour and deposition - General scouring of the bed is evident in many reaches of the channel. This scour occurs primarily in the earth sections of the channel which are on a slope of 0.0015 feet per foot. The maximum depth of general scour is about two feet at the toe of the slope and one foot on the channel centerline.

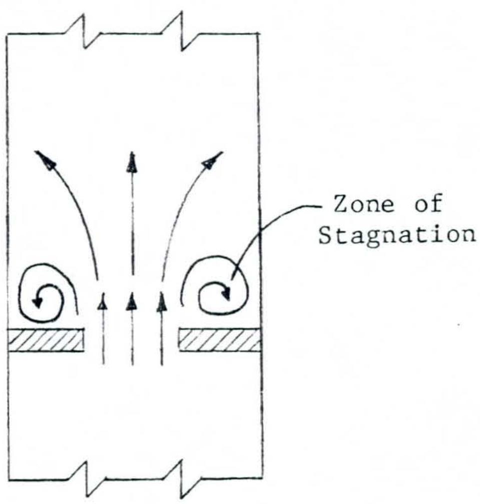
   Deposition also occurs in sections of the channel. The deposition is probably associated with the backwater effects from bridges and riprap sections; however, sections exist where both erosion and deposition occur.

   In sections of the channel, a great deal of sub-channelization is present. Three, four, or even more sub-channels were observed within the main channel width. Each probably caused localized high velocity currents that scoured the compacted earth lining, with sediment deposited bars acting as levees separating the sub-channels. The sub-channels showed signs of meandering. When located near the main channel bank, this meandering tendency increased the bank and toe erosion and created the cyclic erosion pattern on the main channel bank as the meandering sub-channel proceeded downstream.



Scour - Erosion - Deposition

LOW WATER ROAD CROSSING
AT STA. 1315 + 00



CHANNEL CONTRACTION

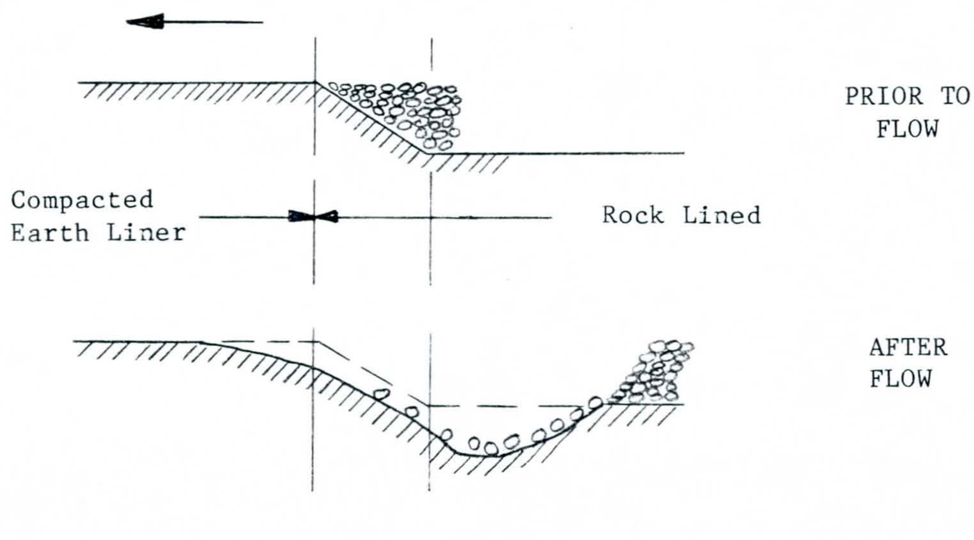
(Not to Scale)

2. <u>Bank toe erosion</u> - Long stretches of the bank toe exhibited bank failure by erosion. In uniform reaches of the earth channel, the general bed erosion appears to have caused the toe erosion, and, with but one observed exception, bank and toe erosion in Reach I occurred on one bank or the other but never on both banks at the same section. The exception was immediately downstream of the road crossing at Station 1315 + 00, where other factors may have contributed to the most severe toe erosion in the channel (a vertical cut of 4 to 4.5 feet). As shown in the sketch above, the crossing appears to have caused a contraction of the flow. General bed scour occurred in the central portion of the channel as if it were subjected to a concentrated jet of water, with sediment disposition in the zones of stagnation on either side of the jet, and subsequent bank erosion further downstream as the jet fanned out.

3. <u>Localized bed erosion</u> - Bed erosion occurs immediately downstream of most of the riprap reaches; however, the erosion is much greater below some riprap reaches than others. At the downstream end of the riprap reaches where soil erosion does occur, the erosion is not uniform across the channel section. In fact, it is "saw tooth" in that there are areas of deep erosion next to areas of basically no erosion. At some of these locations, the scoured soil surface is well below the riprap surface.

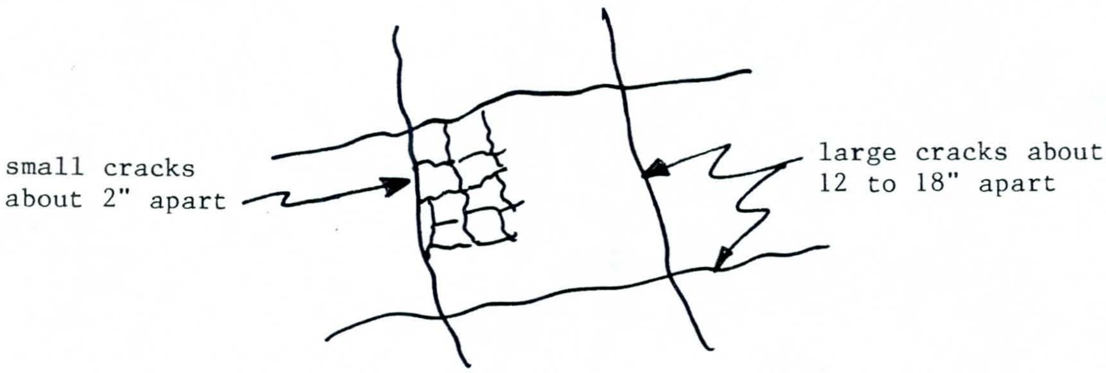
   In most locations, it is not obvious why the erosion occurs at one point but not at another. However, at some locations it is obvious that machinery was driven onto the riprap and depressed the riprap. Where the depression of the riprap is approximately parallel to the flow, scour of the downstream soil always occurs.

   There is no filter under the riprap lining. The flow has washed bed material from under the rock, and the rock has settled into the cavities. As shown in the sketch below, the drop in the rock surface creates a chute-like situation which heightens the erosive capacity of the water.



Bed erosion also occurs downstream of the curve sections of the channel due to the boundary shear stresses caused by the flow in the curved sections. For example, the erosion is approximately one foot deeper downstream of the inside of the double curve at Station 1249 + 80 and the 90 degree curve at Station 1414 + 96 than downstream of the outside of these curves.

4. Cracks in bed liner - Cracking of the bed liner was evident throughout the reach. These shrinkage cracks should be expected due to the moderately to highly plastic clay used as liner material and the very hot, dry climate. The sketch below shows a typical cracking pattern.



5. Riprap - The riprap sections appear to be stable with very little movement of the riprap by the flow. However, local subsidence of the riprap is evident especially at the downstream transition from riprap to soil.

   The riprap lining is one foot thick with no filter bedding beneath the lining. The soil beneath the riprap moves through the riprap allowing the obvious settlement.

6. Bank rilling - The channel banks are rilled and, in at least one section of the channel, the rills are 2 to 3 inches deep. However, most of the rilling is not that deep. The bank rilling at some locations has resulted in deposition on the bed at the toe of the bank.

7. Jug holes - Jug holes are evident on the crests and banks of the dikes, with the worst section being the lower 1000 feet of the reach. In the vicinity of Station 1241 + 50, numerous jug holes and a large piping failure, extending from the crest of the right dike into the channel, were noted. The jug holes were especially numerous in the diking above riprap sections.

The committee observed only minor erosion in the Reach II portion of the channel; therefore, the decision was made to limit this report to information pertaining to Reach I.

Site investigation.--Aubrey C. Sanders, Jr., SCS State Geologist, made the basic site investigation in 1972 and 1973. The investigation obtained a total of nine undisturbed and seven disturbed soil samples from the 4.6 mile reach for laboratory testing and analysis. Based on the test results, field logs, and observations, 15 stratigraphic units were identified.

The geology report revealed that the lower 1,000-foot reach of the floodway contained piping-prone materials. This conclusion was based on field observations of rather deep gullies and pipes. The geologist also stated that the type and location of any needed outlet structures (from the floodway into the Gila River) had not been determined at the time of the 1973 investigation.

Prior to design in 1978, an additional field investigation and sampling program were carried out for material evaluation in the vicinity of the outlet structure. This investigation included taking two undisturbed samples and twelve disturbed samples.

Soil testing.--In addition to the normal routine index tests, the Portland MTS ran unconfined compressive shear tests on six of the undisturbed samples and two of the disturbed samples taken during the first site investigation. Also, the MTS determined soluble salt contents on all samples except one. The soluble salt contents ranged from 0.3 to 3.0%. Double hydrometer dispersion tests were run on eight samples with results ranging from a trace to 60%. Attachment 4 summarizes these test results.

Based on the results of the 1973 testing program, six samples had plasticity indices of 15 or more. Based on the 1978 testing program, 7 of the 10 samples tested had plasticity indices of 15 or more. No laboratory testing was done to evaluate the cracking potential of the clayey soils.

The unconfined shear strength obtained for the undisturbed soils ranged from 120 to 860 psf. Two similar tests on the disturbed samples yielded unconfined compressive shear strengths of 356 and 713 psf at test dry unit weights equal to approximately 95% of their D698A maximum. The table below gives these results along with the degree of saturation.

Sample	PI	Unconfined Compressive Shear Strength (psf)	% Saturation
2431.1	41	864	87
402.1	9	280	88
2432.1	20	200	100
2404.2	17	396	91
2404.3	7	288	91
2408.1	12	120	92
2404.1*	9	713	89
2420.3*	9	356	86

* Remolded samples

A local Phoenix consultant ran index tests, ten unconfined compressive shear tests, and three pinhole dispersion tests. The pinhole tests indicated that two of the samples were nondispersed (ND-1) and one was dispersed (D-1). Attachment 5 summarizes these test results.

The ten unconfined shear tests performed by the consultant in 1978 were at water contents that ranged from 70% to 95% of saturation for the eight disturbed specimens and from 19% to 50% of saturation for the two undisturbed specimens. A percent saturation of 95% or higher is normally considered "saturated" by the geotechnical profession. Of the eighteen unconfined compression tests performed in 1973 and 1978, only 2 meet this criteria. Six were between 90 and 93 percent and six were between 85 and 90%. Three were tested at percent saturations below 70%. In addition, all of the disturbed specimens were tested at close to 100% of their maximum D698A dry unit weights.

Planning.--The January 1963 watershed work plan showed the floodway as a 7.2 mile channel with a bottom width of 100 to 110 feet, side slopes of 3 horizontal to 1 vertical, a bottom slope of 0.0005 feet per foot, and a design velocity of 4 feet per second at a design discharge of 4600 cubic feet per second. The April 1979 work plan Supplement No. 2 extended the floodway through the Gila River Indian Reservation to a Gila River outlet. The planned earth channel at its lower end had a 200 foot bottom width, 3 to 1 side slopes, 0.00155 feet per foot bottom slope, and a design velocity of 6.8 feet per second at a design discharge of 8700 cubic feet per second.

During planning, consideration was given to a fully riprap-lined channel and a concrete-lined channel, but both were eliminated because of economic feasibility. The compacted earth lining was chosen realizing that some savings in construction would be offset by higher maintenance costs. In fact, erosion of the earth lining was expected and a cost for replacing the lining is included in the planning economics.

Design.--The designed channel agrees with that in the work plan supplement except that the bottom slope is 0.00150 feet per foot. The stability design used the tractive power procedure as described in SCS Technical Release No. 25.

The designer also checked the earth lining using the allowable velocity and the tractive stress procedures given in TR-25. These checks showed the earth lining to be unstable.

The designer felt that the tractive power design should be used due to limited land rights and economics. The designer considered the tractive power approach acceptable, since it is in TR-25, although less conservative than the allowable velocity or tractive stress approaches.

Tractive power approach.--Elliott M. Flaxman (Flaxman 1962 and Flaxman 1963) introduced the tractive power design procedure. Flaxman developed an empirical relationship between the saturated unconfined compressive strengths of undisturbed soils and their tractive power based on actual field observations of erodibility on twelve, mostly perennial flow, stream reaches in six western states. The soils observed and tested were cohesive soils which did not act as discrete particles. The approach attempts to model the aggregate stability of saturated soils to account for the effects of cementation and related geologic processes.

Figure 6-15 in TR-25 shows this relationship with the "S-Line" separating erosive and non-erosive behavior.

Acknowledged or clearly inferred risks taken.--The only such risk taken was the use of the tractive power procedure for the stability design of the channel lining. This is very clearly stated in the final design review report from the West Technical Service Center (Attachment 6). This report says, "The tractive power procedure used in the design of this channel is based on the physical characteristics of the channel and the unconfined compressive strength of soils in a natural state (undisturbed). It's application to disturbed recompacted soils tested under laboratory conditions does not account for all the effects of the natural aging process and leaves some questions as to erosion resistance."

Further, the report states, "The maintenance required over the design life, however, is unknown and could be significant. As a condition of approval, the approving authorities must be aware and willing to accept the risk of high maintenance in lieu of the higher initial cost with more positive measures."

Conditions differing from those assumed in design.--An important design assumption concerns the type of soils to be used in construction of the compacted liner. Sample 2420.3 (730521) represents an SC (LL=31, PI=9) with 29% fines and an unconfined compressive shear strength of 356 psf. The other liner material tested classified as a CL (LL=29, PI=9) with 79% fines and had an unconfined compressive shear strength of 713 psf. No shear tests were run on the more plastic soils such as 2408.2 (LL=67, PI=39) or 2422.1 (LL=45, PI=26). During its inspection of the channel, the committee took nine samples of the soil in the liner. The testing of these samples shows the soils used in the liner to be more plastic than expected and to be dispersed. Attachment 7 shows these test results.

Based on field observations, the designer thought dispersion to be a problem only in the lower 1000 feet of floodway; therefore, additional dispersion testing was not thought to be necessary. Furthermore, Arizona had not experienced any documented problems in the field due to the presence or use of dispersed clays up to the time of the final design for Reach I.

The tractive power design approach is based on tests on cohesive, in-place materials in, to a large extent, channels with perennial flow. This floodway uses a liner composed of soils which are dispersive, thus, tending to act as discrete particles. The normal condition in this channel is a condition of no flow; thus, the soil materials in the channel are very dry and cracked.

Flaxman's tractive power procedure is also based on the saturated unconfined shear strength of soils. Only two of the tests of materials for this floodway were run at water contents of 95% saturation or more. Therefore, the design values of the saturated unconfined shear strength of the compacted liner material may have been overestimated.

Since dispersed soils do erode and act as discrete particles, Figure 6-1 of TR-25, "Channel Evaluation Procedural Guide", does not suggest that the tractive power procedure be used in evaluating the stability design of the channel.

Background of experience.--The Arizona personnel involved with the design of this floodway had little or no prior experience with the use of the tractive power design approach. They had confidence in the procedure because of its inclusion in TR-25.

Dispersive soils had not caused problems in other channels in the state so little concern was given to that possibility in this channel, except at the outlet end.

Construction specifications.--The construction specifications designated the soil materials for the compacted liner by general location. These soils were to consist of suitable CL's, ML's, and SC's as approved by the Engineer. The specifications required the soils used for the liner to have more than 15% fines and to be compacted to 95% of their maximum ASTM D698A dry unit weights at water contents ranging from 3 points below optimum to 1 point above.

Construction.-Records reveal that the liner was constructed in accordance with the specifications. Dry unit weights averaged greater than 98% of standard and placement water contents averaged about 2 points below optimum.

In an interview with John Sullivan, he indicated that he gave oral instructions to field construction personnel not to use CH's in the liner. Field personnel were looking for the more plastic materials.

The committee found no other significant items in the construction records that would affect the intended design or performance of the structure.

Maintenance actions prior to December 1984 storm.--The maintenance on the compacted earth lining was to grade the lining for the entire length of Reach I. The grading was done to level the channel bottom and to control vegetation. The banks were smoothed to remove rills. The loose material from the grading operation was left at the toe of the banks. This was expected to provide some protection to the banks.

Hand labor was used to cut the larger brush in the riprap sections; however, from the committee's observations, vehicles were driven on the riprap sections probably during maintenance.

No effort was made during maintenance to restore the compacted earth lining to its original thickness.

EVALUATION

When a compacted earth lining was selected for this channel, high maintenance costs were expected and were considered in the economic analysis. However, the channel has experienced only three principal flows, and all of them were much less than the design discharge, and, yet, significant erosion of the earth lining has occurred. This was not expected.

Possible Causes

Based on the facts gathered during this investigation, the following causes of the significant erosion are postulated:

Shrinkage cracks.--Shrinkage cracking occurred in the moderately to highly plastic clays used in portions of the liner due to dessication in an extremely hot and arid climate. The cracking of the compacted earth liner greatly reduced the erosion resistance of the material; in fact, the flow could very easily remove the surface of the liner.

Dispersive soils.--Dispersive clays were used in portions of the liner. Their use complicates our understanding of the erosion process because erosion occurs as "discrete" particles although the soils are highly cohesive. This uncertainty leads to difficulty in selecting an appropriate stability design procedure.

Tractive power procedure.--The stability design of this channel uses the tractive power procedure. Questions concerning its application to this channel are:

a. How does the tractive power procedure relate to remolded soils where the natural cementation and, perhaps, other geological forming processes have been destroyed except for prestresses built in during the compaction process?
b. What degree of saturation is acceptable in testing for the unconfined compressive shear strength of soils when using the tractive power procedure?
c. Should dispersed clays be treated as discrete particles?
d. How does the development of an extremely blocky structure due to shrinkage cracking affect stability?
e. Should design values of tractive power be selected from a line with erosive and non-erosive cases on each side of the line when a data base of experience with the use of this design procedure is very limited.

Maintenance operations.--The maintenance operations may have contributed to the erosion problems by leaving in the channel large quantities of loose material which is easily moved by the flow and deposited on riprap and in other parts of channel. These deposits on the riprap reduce the friction factor for the riprap sections, thus, discharge velocity is increased; and the deposits encourage vegetation which ultimately concentrates flow between vegetated areas. The deposits on the earth liner sections encourage a meandering flow pattern which leads to erosion of the earth liner. The deposited bars also add to the frictional effects.

One additional detail of the maintenance which may have aggravated the situation was the pushing of the loose material during grading to the toe of the bank. This restriction of the channel for low flows may have also contributed to erosion.

Road crossings.--The large amount of erosion immediately downstream of the riprap road crossing at Station 1315 + 00 appears to be the result of a constriction of the flow at the crossing and then an expansion of the flow with the associated eddying below the crossing.

Transitions from riprap to earth.--The localized scour immediately downstream of the riprap sections of the channel may be caused by the turbulence created when the flow passes from the riprap to the earth lining. In some instances, large vegetation in the riprap and vehicle tracks in the riprap may have caused flow concentrations and turbulence which caused the observed scour patterns. Another possibility is the lowering of the riprap surface due to movement of the bedding material through the rock with the resulting settlement.

The unsummetrical distribution of boundary shear stresses downstream of channel curves probably increased the erosion depths on the edges of the channel just downstream from the inside of curves (Ippen, Drinker, Jobin, and Shemdin 1962).

Evaluation of Basic Data

Geology and site investigation.--The original site investigation and sampling program were consistent with SCS policy in effect at that time and were adequate to represent the engineering behavioral properties of both the insitu foundation materials and those used in the compacted liner. However, only seven disturbed samples to represent 440,000 cubic yards of lining is, at best, a minimum number of samples.

Soil testing.--The evaluation of the soil testing program is divided into three parts for ease of discussion.

Dispersion.--During the committee's inspection of the floodway, nine samples were taken for laboratory testing. The main purpose was to determine if dispersive clays are present. Attachment 7 shows the test results. Dispersive clays were suspected because of the numerous jug holes in the dikes along the channel and a large piping failure extending from the crest of the right dike into the channel in the vicinity of Station 1241 + 50.

Due to the presence of soluble salts, oven drying was controlled at 60° C. The soils contained 63 to 94% fines and classified as CL and CH's. All of the samples had plasticity indices of 15 or more with four being greater than 20. Double hydrometer, crumb, and pinhole dispersion tests were run on all of the samples. The results clearly indicate that six of the nine samples are highly dispersed.

An earlier contract claim, regarding the channel, required additional soil testing, and this data was also reviewed and is shown as Attachment 8. It included values of soluble salts obtained from saturated extracts. The amount of sodium (Na), calcium (Ca), potassium (K), and magnesium (Mg) are determined in units of MEQ per liter. These amounts of each are added together to obtain the total soluble salts, and then the percentage of sodium is calculated. Sherard (Sherard 1972 and Sherard 1977) developed a plot showing an empirical relationship between the percent sodium, total soluble salts, and dispersion. The data obtained from 14 samples was plotted and is shown as Attachment 9. The plot is divided into three areas indicating the degree of dispersion. One of the samples plots in the nondispersed zone, four in the transition (or borderline) zone, and nine plot in the dispersed zone. This data also tends to confirm the presence of dispersed clays.

Based on the results of laboratory tests and our field observations, there seems to be no doubt that dispersive clays are present and were used in the construction of the liner.

Dispersive clays tend to act as discrete particles instead of aggregates as do normal clays. Due to their very small particle size and lack of particle attraction, the particles are considered as discrete particles and, thus, display very highly erosive behavior.

Cracking.--A detailed analysis of the Atterberg limit data revealed the following information with respect to plasticity for both the undisturbed and disturbed soil samples:

Testing	No. of tests run	No. of Samples with PI* < 15	No. of Samples with PI > 15
MTS, Portland (1973)	16	10	6
Phoenix Consultant (1978)	11	5	6
Contract Claim (?)	14	2	12
SML, Ft. Worth (1985)	9	2	7

* PI = Plasticity Index

About 60% of the samples tested had plasticity indices (PI) greater than 15. Many geotechnical engineers consider PI as a good indication of shrink-swell behavior. See Attachment 7A for one generalized relationship. Eighteen, or almost 40%, had PI's of 20 or more. This raises the question as to the clay mineralogy of these soils.

Again, Attachment 8 also gives the results of x-ray diffraction scans made on the 2 micron size for 14 samples. All contained at least medium amounts of montmorillonite with 12 containing a large amount. Montmorillonite is a clay mineral that exhibits high shrink-swell behavior. In particular, soils containing large amounts of this mineral could be expected to experience a large volume decrease (shrinkage) upon drying. Due to the very hot and dry climatic conditions in this part of Arizona, it seems logical to assume that at least some of the clays used to construct the liner would dry and develop shrinkage cracks. The committee noted the presence of many such cracks during our visit to the site.

This cracking extends at least two feet deep in places and results in a very blocky structure. Flow in the channel easily penetrates these cracks and creates uplift pressures with a resulting "plucking" of the soil blocks.

Unconfined shear strength.--The unconfined compression testing program carried out by the Portland MTS consisted of only two tests. One test represented a CL (LL = 29, PI = 9) with 79% fines and one test represented an SC (LL = 31, PI = 9) with 29% fines. No tests were run on sample 2408.2, a CH with a LL of 67 and a PI of 39, nor on the other samples that represented soils with less than 15% fines. The reasoning in the laboratory was probably based on the assumption that these types of soils are not suitable for a liner; and, therefore, unconfined compression tests on them were unnecessary.

As previously stated, the use of the tractive power procedure as developed by Flaxman is based on the saturated unconfined shear strength of undisturbed soils. However, only two of the eighteen unconfined compression tests were run at water contents of 95% saturation or more. The two tests in the 1973 testing program on the recompacted specimens were 86% and 89% saturated. Therefore, it seems logical that the saturated unconfined shear strength of the compacted liner materials used in design may have been overestimated.

The committee believes that q_u tests on only two compacted samples (1973 testing program) to represent over 16,500 feet (440,000 cubic yards) of channel liner is inadequate. The 1978 testing represented the conditions at the outlet end only and was not applicable to the rest of Reach I.

Evaluation of Design

The tractive power procedure used in the design of this channel conforms to that given in Chapter 6 of TR-25. By appearing in TR-25, the procedure has SCS approval as an accepted stability design procedure. However, as noted earlier in this report, the tractive power procedure is based on testing of insitu soils, not soils from a compacted earth liner. The designer assumed this risk since the impact of the differences between Flaxman's conditions and this floodway condition are difficult to assess.

The tractive power procedure is basically untested. Flaxman's original data base has not been expanded. There is no readily available information on the functioning of tractive power designed channels with essentially rigid boundaries.

The design is based on a tractive power value associated with an unconfined compressive shear strength for a soil expected to be used as liner material. The tractive power value was taken from figure 6-15 of TR-25 and is on the "S-Line" which has erosive and non-erosive cases on each side of the line. Again, the designer assumed a risk in selecting a tractive power value on the S-Line. A more conservative approach would have been to select a smaller tractive power value, one more obviously in the non-erosive area of figure 6-15. For a more detailed discussion of the tractive power procedure, see Attachment 11.

At the time of this design, Arizona had not experienced problems with dispersed clays. The designer also believed that the dispersive clays were limited to the downstream 1000 feet of channel. The design did not consider the consequences of using highly dispersive clays or highly crack-prone clays as compacted liner material. The two specification requirements for the soils used in the liner was that they have more than 15% passing the No. 200 sieve and be from specified locations. The specifications should have been more restrictive to prevent the use of moderately to highly plastic clays in the liner.

Attachment 6 shows that at least some of the assumed risk was taken because of land rights and economic restraints.

Theurer's contention that the channel liner was overstressed is not true. See section 5 of Attachment 11.

Evaluation of Construction Operations

The construction operation appears to have complied with the critical provisions of the contract specifications. The specifications and the inspection program were adequate.

The construction operation did not have a critical bearing on the erosion occurring in this channel.

CONCLUSIONS

The committee concludes that

1. The major cause of the excessive erosion was the use of dispersive, moderately to highly plastic soils with a high cracking potential in the compacted liner. The liner did crack extensively, and the channel flows penetrated these cracks to significant depths and created uplift pressures that lifted or "plucked" the soil blocks into the moving water. The presence of dispersed clays accelerated this "plucking" action since they tend to erode very easily, even in the presence of relatively still water. This quick erosive behavior prevented the drying cracks from swelling shut.

2. The maintenance operation of grading the lining increased the loss of lining material and, because the loose material was left in the channel, increased the sediment load. The increased sediment load increased deposits which resulted in a flow meander pattern with increased attack.

3. Special design measures are needed to prevent erosion of dispersed clays. The use of chemical additives, such as hydrated lime; riprap with an appropriately designed filter; or other alternatives should be considered. Normal channel design procedures do not apply.

4. Rainfall penetrating drying cracks in the dispersed clays probably caused the numerous jug holes in the crests of the dikes and on the banks.

5. The rills and gullies present on the channel banks do not appear to be excessive for this climatic condition. However, any such surface disturbances in dispersed clays will continue to enlarge.

6. The use of the tractive power procedure is questionable since it appears that the performance of channels designed by this procedure is not well documented. Certainly, the performance of channels with compacted earth liners designed using the tractive power procedure is not documented. While this was recognized in the design and approval process, no special operation and maintenance requirements were given to the sponsor.

7. Probably some degree of deficiency would have occurred regardless of which TR-25 channel stability procedure was used for design. This conclusion is based on the fact that untreated dispersed clays were used in the liner, and these soils can erode even in practically still water.

8. The unconfined compressive shear strengths used in the tractive power design were probably overestimated due to the relatively low degrees of saturation of the test specimens obtained in the shear testing program.

9. The riprap probably does not extent enough downstream of the curve sections to protect the soil liner from the curve-induced boundary shear stresses.

The committee has determined that several areas share responsibility for the deficiency.

Planning	- Very restrictive land rights were obtained thus limiting the possibilities at the design stage.
Investigation	- More samples of possible liner materials may have resulted in a determination that a large portion of the material was more plastic than thought with the samples obtained.
Design	- Considering the soil materials, harsh weather conditions, unproven design procedure, and inadequate specifications, the stability design is somewhat unconservative and may have accentuated the erosion damage.
Maintenance	- The maintenance operations probably contributed to the erosion occurring in the channel, specifically, the grading operation which removed additional compacted earth liner material and left the loose soil material in the channel, providing material for deposition.

The committee believes that the three flood events, which have occurred since the channel was completed, were not excessive or more frequent than expected.

RECOMMENDATIONS

The committee recommends the following regarding the tractive power procedure.

1. Due to the limited data base for this procedure, technical materials should caution potential users. Criteria should require that the maximum velocity and maximum tractive stress for the tractive power designed channel be compared with the allowable velocity and allowable tractive stress, respectively. Values for these two parameters which greatly exceed the allowables should raise a concern for the design.

2. Figure 6-15 of TR-25 should be modified by replacing the "S-Line" with a band which would result in more conservative designs. The band would graphically signify the uncertainty of the procedure.

3. The tractive power procedure should require as input unconfined compressive shear strengths based on tests conducted at 95% of saturation or higher.

4. The Engineering Division, NHQ should conduct a survey to determine the performance of existing channels designed using the tractive power procedure. If possible, data should be collected for channels in both natural soils and compacted soils and for both ephemeral and perennial flow conditions. The collected data should be used to substantiate or revise figure 6-15 of TR-25.

Other recommendations are:

1. Designers should tighten specifications for the use of soils with high cracking potential in channel liners constructed in areas where prolonged periods of hot, dry weather are probable.

2. Arizona should initiate a dispersion testing program for all future and ongoing projects, including those in the design or construction phase. The testing program should include the double hydrometer, crumb, and pinhole tests as a minimum.

3. Arizona should closely monitor other completed reaches of the RWCD floodway to determine if special protective measures are needed. On others in the design or construction phase, consideration should be given to modifying the design procedures to consider both the applicability of the tractive power procedure and the possible presence of dispersed clays.

4. Maintenance operations should not leave large quantities of loose materials in the channel. Vegetation control should be accomplished with herbicides.

ATTACHMENTS

1 - Letter appointing investigation committee dated 6-28-85

2 - Dr. Fred Theurer's report dated 5-23-85

3 - Letter from Stevenson to Arrington dated 6-21-85

4 - Summary of 1973 laboratory soil testing data

5 - Summary of 1978 laboratory soil testing data

6 - Final design review and transmittal letter dated 10-10-79

7 - Summary of 1985 laboratory soil testing data

7A - Prediction of shrink-swell class based on PI and % < 2 micron

8 - Summary of soil testing data compiled for contractor's claim

9 - Relationship of pore water salts and dispersion

10 - Letter from Sanders to Arrington dated 5-10-85

11 - Dr. Lewis Mather's comments on Stability Design

12 - Photos

13 - Profiles comparing as-built conditions with present conditions

14 - References

Franke writing → McElroy

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola, Suite 200
Phoenix, Arizona 85012

Subject: ENG: Investigating Committee RWCD Reaches I and II

Date: June 28, 1985

To: Don Basinger, Director Engineering Division, N.O.
Arthur B. Holland, Director NETC
Jerry S. Lee, Director SNTC
George Bluhm, Director WNTC

File Code: 210

As provided in the NEM 504.03 an investigating committee is being formed to prepare a report for the RWCD Reaches I and II, Williams-Chandler WPP, Arizona. The team will study the appropriateness of current SCS channel design procedures as applied to this job which has recently suffered erosion damage to the earth lined reaches.

We request the following team assignments:

John A. Brevard, NHQ - Chairman
Dr. Lewis Mathers - Professor Villanova University
Charles H. McElroy - SNTC
Gary L. Conaway - WNTC

As approved the team chairman will initiate study actions.

We are submitting to team members under separate cover the following:

1. "As-built" Drawings - RWCD Reach I
2. Preliminary Investigative Report - (5/23/85)
3. WNTC Review Comments - (6/21/85)

State coordination will be handled through Ralph Arrington, State Conservation Engineer - (FTS 261-5152) or (COMM. 241-5152).

Where travel budgets are insufficient, arrangements may be made through Bill Osterquist, SAO.

Ralph M Arrington Acting
Verne M. Bathurst
State Conservationist

cc: Jack Stevenson, Head Engineering Staff WNTC
Robert A. Frank, Head Engineering Staff SNTC
Ed Alling, Head Design Unit, NO
Lloyd E. Thomas, Head Engineering Staff NENTC

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.

<u>Observations</u>

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?

COPY TO SSWG MEMBERS
(SSWG MEMBERS NOTE
2nd PAGE ITEM FOR DISC.
@ OUR NEXT MTG.)

BCS

December 9, 1994

DEC 1994
Flood Control Planning & Development Division

John Wallace, P.E.
Pima County Flood Control District
201 N. Stone Ave., Suite 400
Tucson, AZ 85701

RE: Ungaged Watersheds State Standard

Dear John:

The following summarizes my understanding of the key points of discussion at the kick-off meeting for the Ungaged Watershed State Standard Study. The kick-off meeting with the State Standards Work Group was held on December 1, 1994 at the Flood Control District of Maricopa County.

- John Wallace will act as project manager for this study, and will be responsible for forwarding correspondence to other SSWG members, and relaying comments and information to Jon Fuller.

- The project schedule in the BCS proposal was adopted.

The group discussed the intent of the study in some detail and reached several conclusions. There are two overall objectives for the study. The first objective is to recommend a methodology for estimating peak discharges in Arizona. The methodology should have the following characteristics:

- It should be based on existing procedures. No new procedures need to be developed as part of this study. The goal of the agency contact and literature search are find methodologies are in use now, and what methodologies work well. However, if no existing methodologies are found to be acceptable, and a new methodology is needed, then this need shall be noted.

- The methodology should be simple enough so that it can be applied by staff who have minimal technical training, but should be not be so simplistic that it is overly conservative.

- The methodology is intended to fill a gap in SS#2 between the simplistic Level 1 approach, and the detailed hydrologic modeling of Level 3.

- The methodology should be appropriate for each region in which it is to be applied. A number of methodologies may be required to provide coverage of the entire state.

- A summary of existing methodologies and their area of applicability should be prepared as an aid in selecting appropriate methodologies.

The second objective of the study is to provide a means for verifying or checking the results of detailed hydrologic modeling studies.

- Exhaustive verification of detailed hydrologic modeling methodologies is not part of this study. Instead, we will outline procedures for using the methodologies recommended for verifying hydrologic modeling results. Specifically, we will develop procedures for verifying outlier results (peak discharge estimates that fall above or below regional regression equation predictions).

- A reference list of published literature, modeling summaries, gage data, regional regression equations, and other information will be assembled and categorized as a resource for floodplain managers and hydrologic modelers. These resources can be used to help verify modeling results for a broad range of watershed types. Examples of specific watersheds types that can be difficult to verify modeling results include flat agricultural areas, high elevation forests, and areas with extremely high losses (cinders, sand dunes, etc.).

There are two issues that SSWG may wish to discuss at our next progress meeting on January 6, 1994. First, there will be some watershed types that will have very little or no existing peak discharge information. For these areas, engineering judgment will need to be applied to some other (more conservative) methodology. Second, every existing methodology will over- or under-predict any specific watershed to some degree. For some watersheds, results will be far from accurate due to unique conditions. I have some thoughts on these two issues, but would like input from SSWG.

Finally, it would be very helpful if SSWG will provide the most up to date list of contacts for Arizona floodplain communities, and names of any specific individuals that SSWG members feel that I should contact. Dick French and I are excited about working on this project with you and SSWG.

Sincerely,

Benchmark Consulting Services

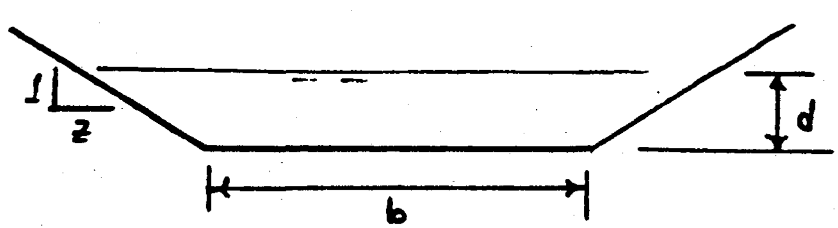
Jonathan Fuller, P.E.

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

Formulas Used:

$$Q = (1.486/n)\, A R^{2/3} \sqrt{S_o} \quad , \quad cfs$$
$$A = (b \cdot d) + (z \cdot d^2) \quad , \quad ft^2$$
$$P = b + d\sqrt{1+z^2} \quad , \quad ft$$
$$R = A/P \quad , \quad ft$$
$$\tau = \gamma R S_o \quad , \quad \#/ft^2$$
$$v = Q/A \quad , \quad fps$$
$$\text{tractive power} = \tau \cdot v \quad , \quad ft\text{-}\#/sec/ft^2$$
$$d_{50} \geq 11 \cdot d \cdot S_o \;\; \text{(shields initiation of motion)} \quad , \quad ft.$$

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



STATE	PROJECT RCWD Floodway		
BY [illegible] DATE 5/21/85	CHECKED BY	DATE	JOB NO.
SUBJECT Flow Analyses			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 lb/ft²	τ·v (ft·lb/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 [illegible]	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 (16% & 50%)	0.015	1.49	1.45	4.93	0.1361	0.670

$\tau = \gamma R S_o$

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

MPUTATION SHEET
-ENG-523 Rev. 8-69

SOIL CONSERVATION SERVICE

ATE		PROJECT RCWD Floodway		
[illegible]	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	$\tau \cdot v$ 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.

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE	PROJECT RWCD Floodway			
BY	DATE 5/23/85	CHECKED BY	DATE	JOB NO
SUBJECT Q/b vs max. S_o Analysis				SHEET 4 OF 5

$\tau \cdot v$ (ft-#/sec/ft²)

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	0.624	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

$S_o = 44.9/30378 = 0.00148$

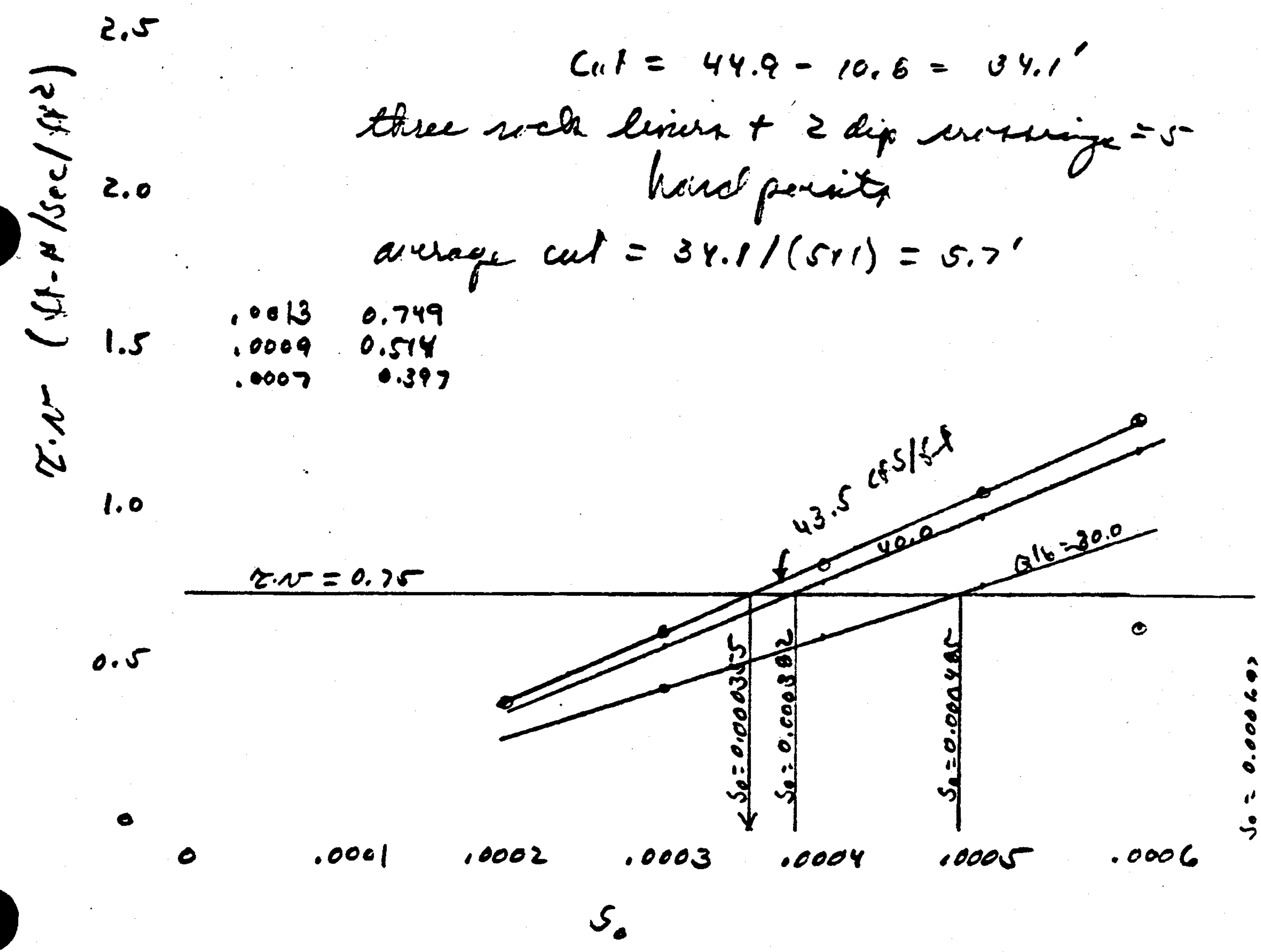
$\Delta h_{all} = 30378 \times 0.000355 = 10.8'$

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 JDR DATE 5/24/85	CHECKED BY	DATE	JOB NO.
SUBJECT armor material analysis			SHEET 5 OF 15

$d_{75} = 2''$

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

$d = 6.24'$

$R = 5.70'$

$v = 6.38$ fps

$\tau = 0.5332$ #/ft^2

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

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

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

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

$d_{max} = 3''$

United States
Department of
Agriculture

Soil
Conservation
Service

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

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. Rilling 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. Walker 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

SCS-ENG-354
Rev. 3-70
File Code ENG-13-18

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS
LABORATORY DATA
Sheet 1 of 2

RWCD, Reach I, Arizona

1973 Testing

Testing performed by the M.T.S., SCS, West NTC, Portland, Oregon

Laboratory Sample Number	Field Number	Location and Description	Depth	Field Classification	Mechanical Analysis (grain size distribution expressed as percent finer by dry weight): Fines .002 mm	.005	.02	.05	#200 .074	Sand #100	#60	#40	#20	#10	#4	Gravel 3/8"	1/2"	3/4"	1"	1 1/2"	3"	Atterberg Limits LL	PI	Unified Classification	Soluble Salts %	Dispersion %	Moisture-Density Relationships (☐ Standard ☐ Modified) Curve No.	Max γ_d pcf	Opt. w %	Undisturbed Sample Data γ_d g/cc	w %	G_s	Special Tests C_s	$q_u/2$ (psf)	Test γ_d	% sat.
528	2431.1	Sta. 1416+00 (UND)	6-8		—	80	90	93	94						100							81	41	MH	2.9	—				?	?	2.81		864	1.38 gm/cm^3	87
502	402.1	Sta. 1452+00 (UND)	4-5		—	25	53	77	88	90	95	97	97	99	100							33	9	ML	0.4	3.2				1.44	12.7	2.68		280	1.44 gm/cm^3	88
045	2432.1	Sta. 1400+00 (UND)	4-4.2		20	24	41	60	69	75	93	97	99	99	100							37	20	CL	0.6	43				1.31	9.8	2.65		200	1.81 gm/cm^3	100
511	2404.2	Sta. 1330+00 (UND)	4-5		—	49	77	88	93						100							46	17	ML	2.4	3						2.78		396	1.38 gm/cm^3	91
512	2404.3	Sta. 1330+00 (UND)	8-9		22	35	67	83	90									100				37	7	ML	3.0	Tr.				1.39	10.6	2.79		288	1.39 gm/cm^3	91
040	2408.1	Sta. 1280+00 (UND)	8-8.2		19	25	58	79	89	92	98	99	99	99	100							36	12	CL-ML	0.6	37				1.29	10.5	2.72		120	1.29 gm/cm^3	92
044	2422.1	Sta. 1245+00 (UND)	4-4.3'		41	49	62	76	83	85	93	95	99	100								45	26	CL	2.0	60				1.48	12.4	2.73				
519	2420.1	Sta. 1226+00 (UND)	7-8		—	9	10	17	20	23	33	43	60	90	94			100				37	12	SM-1	—	Tr.										
520	2420.2	Sta. 1226+00 (UND)	8-9		—	—	14	30	37	42	58	65	75	91	93					100		48	21	SC	0.4	Tr				1.90	3.0	2.76				
509	2401.1	Sta. 1360+00 (DIST)	8-10		—	—	—	9	12	19	57	85	99					100				—	NP	SM-1	0.5	—						2.70				
510	2404.1	Sta. 1330+00 (DIST)	2-6		—	42	59	73	79	82	91	93	97	99	100							39	9	CL-1	0.3	—		101.9	20.7			2.74		713	95.1	89
041	2408.2	Sta. 1280+00 (DIST)	11-12		—	82	92	94	95	96	97	97	99	99	100							67	39	CH	2.8	—						2.86				
524	2423.1	Sta. 1257+00, 100'L (DIST)	8-9		—	—	—	—	5	8	50	86	97	99	100							—	NP	SP-SM	0.3	—										
521	2420.3	Sta. 1226+00 (DIST)	2-6		—	—	10	22	29	32	43	49	59	75	87					100		31	9	SC	0.4	—		121.8	10.2			2.70		356	117.5	86
522	2421.1	Sta. 1220+00 (DIST)	0-6		—	—	14	26	32	36	52	65	79	93	97	100						45	12	SM-1	0.4							2.69				

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SOIL CONSERVATION SERVICE

Laboratory Sample Number	Field Number	Location and Description	Depth	Field Classification	Mechanical Analysis – Grain Size Distribution Expressed as Percent Finer by Dry Weight: Fines .002 mm	Fines .005	Fines .02	Fines .05	Fines #200 0.074	Sand #140 0.105	Sand #60 0.250	Sand #40 0.42	Sand #20 0.84	Sand #10 2.0	Sand #4 4.76	Gravel 3/8" 9.525	Gravel 1/2" 12.7	Gravel 3/4" 19.05	Gravel 1" 25.4	Gravel 1 1/2" 38.1	Gravel 3" 76.2 mm	Atterberg Limits LL	Atterberg Limits PI	Unified Classification	Soluble Salts %	Dispersion %	Moisture-Density Relationships ☐ Standard ☐ Modified: Curve No.	Max γ_d pcf	w_o %	Undisturbed Sample Data γ_d g/cc	w_o %	G_s	Special Tests				
523	24212	Sta. 1220+00 (DIST)	6-10		–	–	–	–	9	10	19	33	50	66	81	90	92				100	–	NP	SW-SM	04	–											

SCS-ENG-354
REV. 3-70
FILE CODE ENG-13-18

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS
LABORATORY DATA
Sheet 1 of 2

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION: RWCD, Reach I, AZ (1978 Testing *)	DEPTH FT	FIELD CLASSIFICATION	Fines .002 mm	Fines .005	Fines .02	Fines .05	Fines #200 .074	Sand #140 .105	Sand #80 .250	Sand #40 .42	Sand #20 .84	Sand #10 2.0	Sand #4 4.76	Gravel 3/8" 9.525	Gravel 1/2" 12.7	Gravel 3/4" 19.05	Gravel 1" 25.4	Gravel 1 1/2" 38.1	Gravel 3" 76.2 mm	Atterberg Limits LL	Atterberg Limits PI	UNIFIED CLASSIFICATION	SOLUBLE SALTS %	DISPERSION %	Moisture-Density Relationships ** (■ Standard □ Modified) Curve No.	Max γ_d pcf	w_o %	Undisturbed Sample Data γ_d g/cc	w_o %	G_s	Special Tests C=	$\frac{q_u}{2}$	Test γ_d	% Sat	Pinhole Test
3903-1		3406.4 (Dist)	5						96			98		99	100							41	15	CL-ML				103	20.5			2.72		3940	103.4	88	
3903-5		3411.2 (Dist)	1.5						48			93		99	100							27	9	SC				115.5	12.7			2.72		5620	119.8	90	
3903-7		3411.4 (Dist)	4						70			95		100								44	19	CL-ML				109.6	17.8			2.68		4220	110.5	90	
3903-9		3403.2 (Dist)	2-4						43			88		97	100							29	12	SC				118.9	12.5			2.69		3260	115.4	70	
3903-13		3406.2 (Dist)	1.5						68			98		100								43	25	CL				107.2	18.3			2.72		3620	109.9	95	
3903-16		3412.2 (Dist)	4-6.5						94			100										40	16	CL				104	20.6			2.72		2560	103.2	88	
3903-17		301.1 (Dist)	1-4						84			99		100								36	18	CL				103.4	21.4			2.69		3120	105.7	93	ND-1
3903-18		302.1 (Dist)	2-10						91			100										41	17	CL													ND-1
3903-19		302.2 (Dist)	12-15						16			88		97	100							–	NP	SM													
3903-20		303.1 (Dist)	5-10						92			99		100								50	28	CL-CH				98.8	21.2					4980	99.0	81	ND-3 / D-1
3903-21		303.2 (Dist)	15-16.5						8			79		99	100									SP-SM													
3903-24		303.11 (Dist)	30-30.5						19			42		62	70	80		88	92	95	100	23	2	SM													

*Testing performed by Sergent, Hauskins & Beckwith
Consulting Soil & Foundation Engineers
Phoenix, AZ

**ASTM D698, Method C

SCS-ENG-354
Rev. 3-70
File Code ENG-13-10

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 2 of 2

Laboratory Sample Number	Field Number	Location and Description: RWCD, Reach I, AZ (1978 Testing)	Depth FT.	Field Classification	Mechanical Analysis (Grain size distribution expressed as percent finer by dry weight) Fines .002 mm	.005	.02	.05	#200 .074	Sand #140 .105	#60 .250	#40 .42	#20 .84	#10 2.0	#4 4.76	Gravel 3/8" 9.525	1/2" 12.7	3/4" 19.05	1" 25.4	1 1/2" 38.1	3" 76.2 mm	Atterberg Limits LL	PI	Unified Classification	Soluble Salts %	Dispersion %	Moisture-Density Relationships ☐ Standard ☐ Modified: Curve No.	Max γ_d pcf	w_o %	Undisturbed Sample Data: γ_d g/cc	w_n %	G_s	Special Tests: C =	$\frac{S_u}{2}$	Test γ_d	% Sat	
3903-10		3405.1 (U)	3																													2.70		1040	97.4	19	
3903-12		3406.1 (U)	1.5																															4320	106	49.5	

United States Department of Agriculture
Soil Conservation Service
West Technical Service Center
511 NW Broadway, Rm. 510
Portland, Oregon 97209

FMW
JLB

SUBJECT: EN - 40-13 - Design - Williams-Chandler W/S, RWCD Floodway, Reach 1 - Final Design, Arizona

DATE: October 10, 1979

TO: Thomas G. Rockenbaugh, State Conservationist
SCS, Phoenix, Arizona

Attached are three copies of the Final Design Review Report. This is in accordance with my memo of 9/28/79.

C.E. Stearns acting
STANLEY N. HOBSON
Head, Engineering Staff

Attachments

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

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Engineering Staff
Portland, Oregon
October 10, 1979

Did we get a copy of this or were we made aware of the maintenance cost?

FINAL DESIGN REVIEW REPORT

Job : RWCD Floodway, Reach 1
Project : Williams-Chandler Watershed
Location : Maricopa and Pinal Counties, Arizona
Authority: PL-566 (08)
Phase : Final Design

Summary: The methods used to evaluate the compacted earth channel are reasonable and should provide a stable floodway.

The maintenance required over the design life, however, is unknown and could be significant. As a condition of approval, the approving authorities must be aware and willing to accept the risk of high maintenance in lieu of the higher initial cost with more positive measures.

The drawings and specifications have been well prepared and require only minor corrections.

Description of Job: This job consists of a floodway channel and a rock riprapped outlet into the Gila River. This channel represents the lower reach of the RWCD floodway project.

The work consists of approximately 4.6 miles of channel that has 4700 feet of riprap protection and 16,500 feet of compacted earth lining. An irrigation canal crossing is provided under the floodway by means of a reinforced concrete pipe siphon.

Purpose of Review: The review was made to determine adequacy of the design and to determine if the drawings and specifications were complete.

Scope of Review: The following material, prepared by the state design staff, was reviewed:

1. Construction Drawings, Reach 1, 24 sheets, dated 8/79
2. Final Design Construction Specifications, Reach 1, dated 8/79
3. Final Design (Report & Calculations), Reach 1, dated 8/79
4. Soils Correlation, Final Design, Reach 1, dated 8/79

Basis of Review: The following reference material was used in conducting this review:

1. Preliminary Design Review Report, dated 6/1/78
2. Technical Release No. 25
3. National Engineering Handbook No 5
4. Rock Riprap Design Procedure, TSC Advisory EN PO-18, May 1, 1974
5. Design of Small Canal Structures, Bureau of Reclamation, 1974
6. Channel Stability in Undisturbed Cohesive Soils, Journal of the Hydraulics Division, ASCE, March 1963

Review Comments

The final review of this job was completed during a joint review conference in Portland with John Sullivan and Paul Monville of the state staff, and WTSC personnel during the week of September 24-28, 1979. A round-table conference with discussion of the job and major review comments below was held Friday morning (9/28) with the following personnel present:

Arizona State Office

Ralph Arrington, State Conservation Engineer
Paul Monville, Civil Engineer
William Payne, Civil Engineer
John Sullivan, Civil Engineer

WTSC, Portland

Stanley N. Hobson, Head, Engineering Staff
Richard M. Matthews, Head, Design Section
Leland M. Saele, Supervisory Civil Engineer

Washington, D. C.

Robert Pasley, Asst. Director, Engineering Division

In general, the design and supporting calculations have shown good engineering judgment. The Design Report, however, is brief, leaving many unanswered questions for reviewers not initially familiar with the job. We believe that as a minimum, the report should include statements on the design of unique features, design procedures that were ruled out because of cost, etc., as well as general assumptions. We refer you to NEM 511.11(b) for guidance in preparing the design review report. We believe a good design report is essential on a project such as this where design extends over several years.

A. Floodway Design

The tractive power procedure used in the design of this channel is based on the physical characteristics of the channel and the unconfined compressive strength of soils in a natural state (undisturbed). Its application to disturbed recompacted soils tested under laboratory conditions does not account for all the effects of the natural aging process and leaves some question as to wear erosion resistance. However, in view of the land rights and economic restraints placed on this job, the designers' options are very limited. Confidence in the design is largely dependent on the degree of conservatism taken in application of the tractive power procedure. We believe the designers have used reasoned conservatism in preparing this design as indicated by the following:

1. The unconfined compressive strength of the compacted fill is nearly double the value indicating stability, as per Figure 6-15, TR-25.

2. The natural (undisturbed) condition of soils used for the compacted fill exhibit unconfined compressive strengths within the stable range as per Figure 6-15, TR-25.

3. Where the existing soils in the bottom of channel were inadequate, but side slopes adequate, the entire perimeter was relined to avoid points of discontinuity.

In addition to the above, we recommend that a monitoring program be initiated to determine the effects of weathering, and channel flow on the compacted fill. As a minimum, we suggest the program include the following:

1. Annual inspections of the channel after completion of construction to visually note conditions of the entire channel and to determine areas of degradation.

2. Three to five undisturbed soil samples for testing from selected areas as construction proceeds, during the first two annual inspections and thereafter following significant flows as determined by the results of previous evaluations. The tests should include moisture, density, permeability, and the unconfined compressive strength.

3. A report on the channel inspections, along with results of the soil tests, should be prepared. A copy of the reports should be sent to the TSC. This report should include an evaluation of the channel and recommendations for maintenance if necessary.

Since the durability of the compacted earthfill channels is not as positive as more permanent linings such as concrete, we recommend an operation and maintenance plan to account for the uncertainties. The plan should provide for replacement of areas of localized and general degradation as determined necessary through evaluation of the monitoring program. Type of replacement material to be used will be determined as part of the evaluation process (see NEM 511.11(b)(17)).

B. Drawings

The drawings were well prepared. Changes to the drawings mainly consisted of clarification of work limits and pay lines. All changes were noted in red on a copy of the drawings.

C. Specifications

The specifications were well prepared. The major change consisted of replacing Interim Specification 200, Grouted Rock Riprap and One-Time Use Specification 400, Seeding with SCS Standard Specifications 62 and 6, respectively.

We recommend that the construction details for Specification 6, Seeding include:

1. Prepared seeding dates to take advantage of local rainfall patterns.

2. Seed mix number 2 should include one pound of pure live 'Zorro' fescue seed per acre for grass cover.

James H Dunlap
Submitted
for Lee Steele

C E Maynard
Recommended

C E Maynard Acting
Approved
for RMMatthews

10/10/79
Date

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

1985 Testing

SOIL MECHANICS
LABORATORY DATA

SCS-ENG-354
3-70

JOB NO. 5018

Sheet 1 of 1

ARIZONA
WF-08
RWCD Reach I

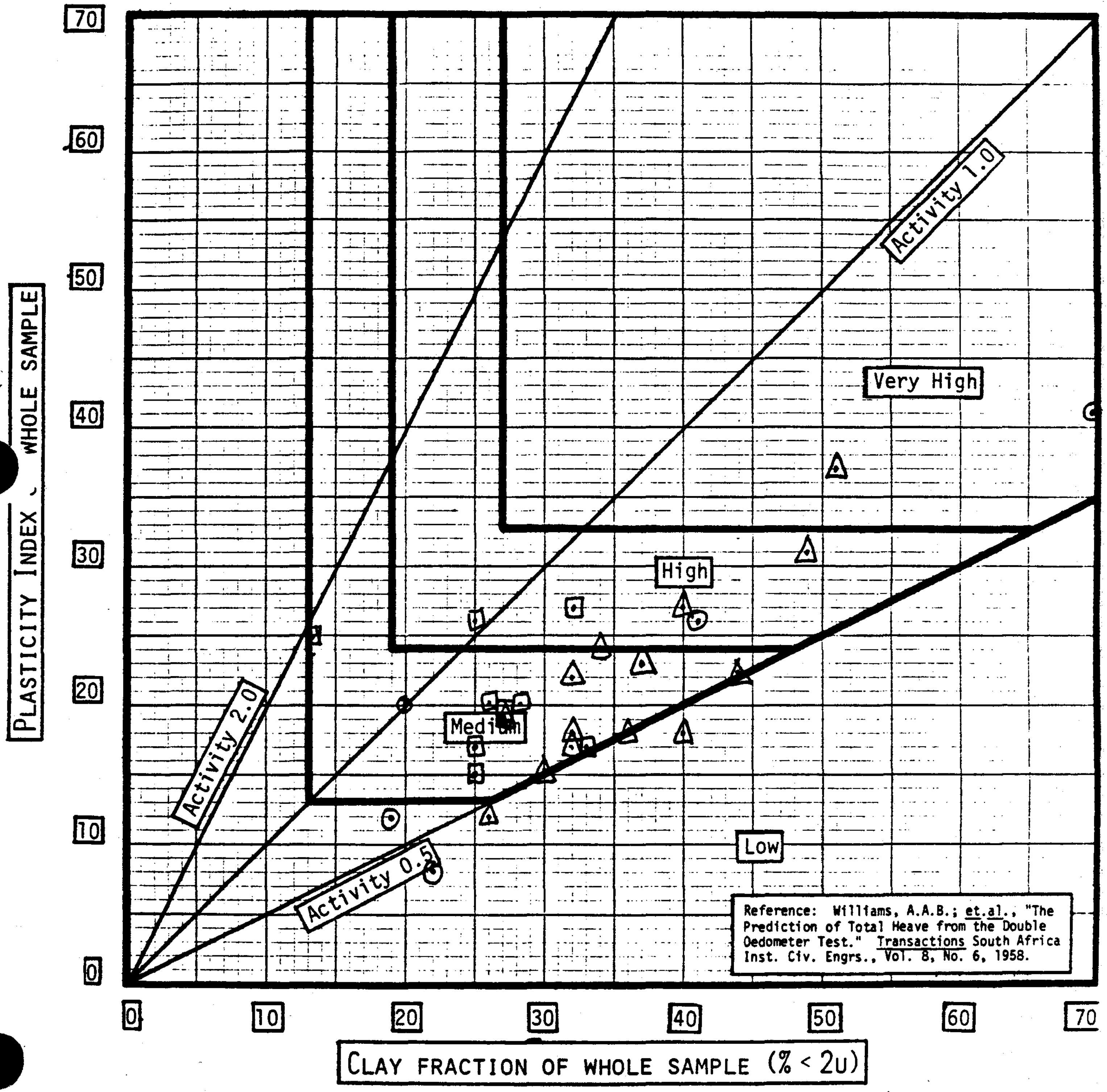
Max. Ht. Vol. Stru. Class

LABORATORY SAMPLE NUMBER FY-85	FIELD NUMBER	LOCATION AND DESCRIPTION	DEPTH	FIELD CLASSIFICATION	MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT: FINES .002 mm	FINES .005	FINES .02	FINES .05	FINES #200 0.074	SANDS #140 0.105	SANDS #60 0.250	SANDS #40 0.42	SANDS #20 0.84	SANDS #10 2.0	SANDS #4 4.76	GRAVEL 3/8" 9.525	GRAVEL 1/2" 12.7	GRAVEL 3/4" 19.05	GRAVEL 1" 25.4	GRAVEL 1 1/2" 38.1	GRAVEL 3" 76.2 mm
2389	1	1201+00 Lined section, US of Hwy 87 bridge, 75' from Rt toe toward centerline in bottom of channel			33	42	50	67	71					100							
2390	2	Sta. 1225 toe, unlined section channel side			25	37	57	68	76					100							
2391	3	1241+50 Rt crest of dike			27	34	43	56	63					100							
2392	4	Sta. 1250, 10' DS on embankment of riprap section taken from sides of deep gully	1-1.5'		25	36	49	61	69					100							
2393	5	Sta. 1264+00, Vertical force in toe of rt. levee, surface cracked 1" apart	15"		13	55	69	83	94					100							
2394	6	Sta. 1275+00 Right bank vertical cracks top to bottom			32	44	65	76	82					100							
2395	7	Sta. 1293+00, Toe of slope. No apparent erosion cracking not quite as bad	6"		26	38	49	61	69					100							
2396	8	1454+00, end of channel, left edge of left dike, 0"-1' in sides of jughole			28	49	56	70	81					100							
2397	9	1317+00, left bank-4' to 5' cut a toe of slope, from exposed face of cut, 2' deep			25	58	68	80	87					100							

LABORATORY SAMPLE NUMBER FY-85	ATTERBERG LIMITS L.L.	ATTERBERG LIMITS P.I.	UNIFIED CLASSIFICATION	SOLUBLE SALTS %	DISPERSION %	MOISTURE - DENSITY RELATIONSHIPS ☐ STANDARD ☐ MODIFIED: CURVE NO.	DRY DENSITY γ lb LBS/FT³	OPTIMUM H_2O w%	UNDISTURBED SAMPLE DATA: γ DRY UNIT WEIGHT g/cc	H_2O w%	G_s	SPECIAL TESTS: PINHOLE	CRUMB 1hr 4hr	pH	Resistivity Ohms-cm	
2389	43	17	CL	<0.5	5					4.4		ND-1	1-1 / 1-1	7.17	1260	
2390	33	15	CL	1.52	36					4.5		ND-4	1-1 / 1-1	7.23	100	
2391	36	19	CL	0.56	59					2.5		D-2	1-1 / 1-1	7.71	304	
2392	35	17	CL	<0.5	6					2.7		ND-1	1-1 / 1-1	7.30	396	
2393	48	25	CL	3.89	56					3.5		D-1	2-4 / 2-4	7.44	100	
2394	45	22	CL	3.22	67					3.3		D-1	1-4 / 2-4	7.53	100	
2395	33	15	CL	0.78	5					2.9		ND-1	1-1 / 1-1	7.62	292	
2396	40	20	CL	2.66	49					3.4		D-1	1-4 / 1-4	7.41	92	
2397	51	26	CH	3.86	38					4.6		D-2	1-1 / 1-1	7.14	52	

PREDICTION OF SHRINK-SWELL CLASS
BASED ON PI AND % < 2 MICRON

WILLIAMS - CHANDLER W/S, RWCD REACH I
ARIZ.



⊙ 1973 Testing
△ Contract Claim Testing
⊡ 1985 Testing

8/85

SCS-MGT-7
REV. 1-71

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Tabular Computations

RWCD - Reach I claim

(low)

SCS Sample #	U of A Sample #	USDA Class.	Unif. Class. 60°C 110°C	Gypsum %	$CaCO_3$ % (equiv.)	% clay <.002 mm	% silt .002 - .05 mm	% Sand .05 - 2 mm	0.05 - 0.1 mm	S 0.1 - 0.25 mm	A 0.25 - 0.5 mm	N 0.5 - 1 mm	D 1.0 - 2.0 mm	pH	Sol. Salts (1:5) %	PI 60°C 110°C	LL 60°C 110°C
1	2977	clay	CL	10.3	7.46	44.3	33.2	22.5	2.93	3.70	4.14	5.37	6.35	7.9	0.8024	22	42
			CL												8024	25	46
2	2979	clay loam	CL	9.7	4.80	37.3	38.0	24.6	2.46	4.44	5.22	6.83	5.65	7.8	0.7466	23	41
3	2985	clay loam	CL	6.0	8.78	39.7	34.0	26.4	2.51	4.56	4.94	6.94	7.44	8.15	0.5944	18	38
4	2976	loam	CL	5.2	7.11	27.0	29.5	43.5	4.14	10.16	12.62	10.16	6.42	8.25	0.2487	19	35
5	2973	clay loam	CL	8.5	8.20	32.0	42.6	25.4	2.84	5.18	5.23	6.71	5.43	7.8	0.4896	22	37
6	2983	clay	CH	2.7	8.34	51.3	34.7	14.0	1.45	1.95	2.58	3.99	4.02	9.0	0.1372	37	58
			CH													40	61
7	2981	loam	CL	4.0	7.70	25.8	33.6	40.5	5.34	11.60	10.58	8.49	4.42	8.6	0.2235	12	26
8	2982	clay loam	CL	4.2	9.63	32.7	30.7	36.6	7.13	8.87	7.88	7.19	5.53	8.6	0.1842	17	32
9	2978	clay loam	CL	4.2	10.47	36.4	32.0	31.6	3.82	6.65	8.50	7.38	5.26	8.5	0.2794	18	34
10	2986	clay loam	CL	4.0	14.59	39.7	35.7	24.6	2.80	4.88	5.88	5.97	4.70	8.35	0.3531	27	45
11	2980	clay loam	CL	4.2	11.08	34.2	32.0	33.7	3.61	7.74	9.01	8.18	5.16	8.2	0.2925	24	40
12	2984	clay loam	CL	6.4	12.31	30.4	45.8	23.8	3.59	4.46	4.07	5.71	5.73	8.3	0.4511	15	31
13	2974	si. c. loam	CL	7.5	9.44	33.5	50.2	16.3	3.08	3.06	3.03	3.85	3.27	8.0	0.5494	17	35
14	2975	clay	CH	2.7	9.46	49.0	37.0	14.0	1.26	2.05	2.89	4.17	3.63	8.95	0.1784	31	55
			CH													34	57

SCS-MGT-7
REV. 1-71

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Tabular Computations

RWCD - Reach I claim

zl le (+)	SCS Sample #	U of A Sample #	Ca mg/l meq/l	Mg mg/l meq/l	Na mg/l meq/l	K mg/l meq/l	Cl mg/l meq/l	SO_4 mg/l meq/l	HCO_3 mg/l meq/l	CO_3 mg/l meq/l	N_3 mg/l meq/l	PO_4-P mg/l meq/l	EC x 10^3	SAR	Gs 60°C 105°C	moisture content 100°F	140°F (60°C)	230°F (105°C)
	1	2977	1295	36	1290	13.9	1379	3946	49	0	15.44	0.05	7.92	9.65	2.523	18.3	21.3	24.0
045			64.62	2.96	56.11	0.355	38.90	82.16	0.802	—	1.103	.00156	saline		2.530			
	2	2979	1255	30	1177	10.3	1439	3479	54	0	21.53	0.05	7.70	8.97	2.527	19.1	21.8	24.5
153			62.62	2.47	51.20	0.263	40.59	72.43	0.884	—	1.538		saline		2.542			
	3	2985	551	19	1348	11.7	1368	2601	34	1.2	10.2	0.05	6.34	15.38	2.546	15.4	18.2	20.3
989			27.50	1.56	58.63	0.299	38.59	54.15	0.556	0.040	0.729		saline	alkali	2.566			
	4	2976	232	12	571	12.7	333	1275	49	0	2.62	0.06	2.91	9.91	2.579	16.5	18.1	19.5
735 NS ALK			11.58	0.99	24.84	0.325	9.39	26.55	0.802	—	0.187				2.585			
	5	2973	949	24	634	16.9	256	2965	49	0	2.01	0.1	4.64	5.55	2.602	18.9	21.0	23.0
342			47.36	1.97	27.58	0.432	7.22	61.73	0.802	—	0.144	0.00312			2.580			
	6	2983	17	2	483	2.3	466	344	49	4.8	3.7	0.05	2.04	29.52	2.521	21.6	25.4	27.6
079 NS ALK			0.85	0.16	21.01	0.059	13.14	7.16	0.802	0.160	0.264				2.547			
	7	2981	113	7	698	17.2	790	565	37	2.4	5.52	0.05	3.28	17.22	2.578	15.3	16.7	17.7
020 NS ALK			5.64	0.58	30.36	0.440	22.28	11.76	0.605	0.080	0.394				2.607			
	8	2982	89	6	574	12.9	635	476	37	4.8	7.31	0.05	2.77	15.9	2.580	19.6	21.2	22.4
230 NS ALK			4.44	0.49	24.97	0.330	17.91	9.91	0.605	0.160	0.522				2.597			
	9	2978	107	8	784	7.4	724	1112	39	4.8	4.79	0.05	3.90	19.69	2.588	20.3	22.5	24.2
145 NS ALK			5.34	0.66	34.10	0.045	20.42	23.15	0.638	0.160	0.342				2.577			
	10	2986	168	14	1004	7.7	1065	1220	41	2.4	8.4	0.05	4.87	20.0	2.565	19.6	21.7	23.4
397			8.38	1.15	43.67	0.197	30.04	25.40	0.671	0.080	0.60				2.596			
	11	2980	215	14	715	11.5	718	1208	34	2.4	6.7	0.05	3.85	12.76	2.588	20.2	22.2	23.8
.74 ?			10.73	1.15	31.10	0.294	20.25	25.15	0.556	0.080	0.479				2.584			
	12	2984	521	14	838	12.5	660	2421	37	2.4	4.76	0.06	4.64	9.89	2.578	21.4	23.4	25.2
720			26.00	1.15	36.45	0.320	18.62	50.41	0.605	0.080	0.340				2.528			
	13	2974	808	22	1001	11.3	883	2701	61	0	6.34	0.06	5.89	9.49	2.554	21.7	24.2	26.1
759			40.32	1.81	43.54	0.289	24.91	56.23	0.998	—	0.453				2.569			
	14	2975	19	2	626	2.9	272	795	54	12	1.03	0.1	2.43	36.51	2.534	29.0	33.0	35.2
414 NS ALK			0.95	0.16	27.23	0.074	7.67	16.55	0.884	0.399	0.074				2.549			
	BIA	EC	4-8	slight	8-16	mod.	16+	severe										
		SAR	6.5-13	"	13-37.5	"	37.5+	"										

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REV. 1-71

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Tabular Computations

RWCD-Reach I claim

SCS Sample #	U of A Sample #	<2µm x-ray diffraction scan - U. of A. Mt	Mi	K	P	Ca	Q	F	Vm	Cl	Inter.
1	2977	4+	1	2	1	—	—	—	0	0	0
2	2979	4	2	2	2	—	—	—			
3	2985	4+	2	2	1	—	—	—			
4	2976	4	2	2	1	—	—	—			
5	2973	4	2	2	1	—	—	—			
6	2983	4+	1	1	1	—	—	—			
7	2981	3+	2	2	2	1	1	—			
8	2982	3	3	2	2	1	1	—			
9	2978	4	2	2	2	—	—	—			
10	2986	4	1	1	1	—	—	—			
11	2980	4+	2	2	2	—	—	—			
12	2984	4	2	2	—	1	—	1			
13	2974	4	2	2	1	—	—	—			
14	2975	4+	1	1	1	—	—	—			

Mt - Montmorillonite
Mi - Mica
K - Kaolin
P - Palygorskite
Ca - Calcite
Q - Quartz
F - Feldspar
Vm - Vermiculite
Cl - Chlorite
Inter. - Interstratified

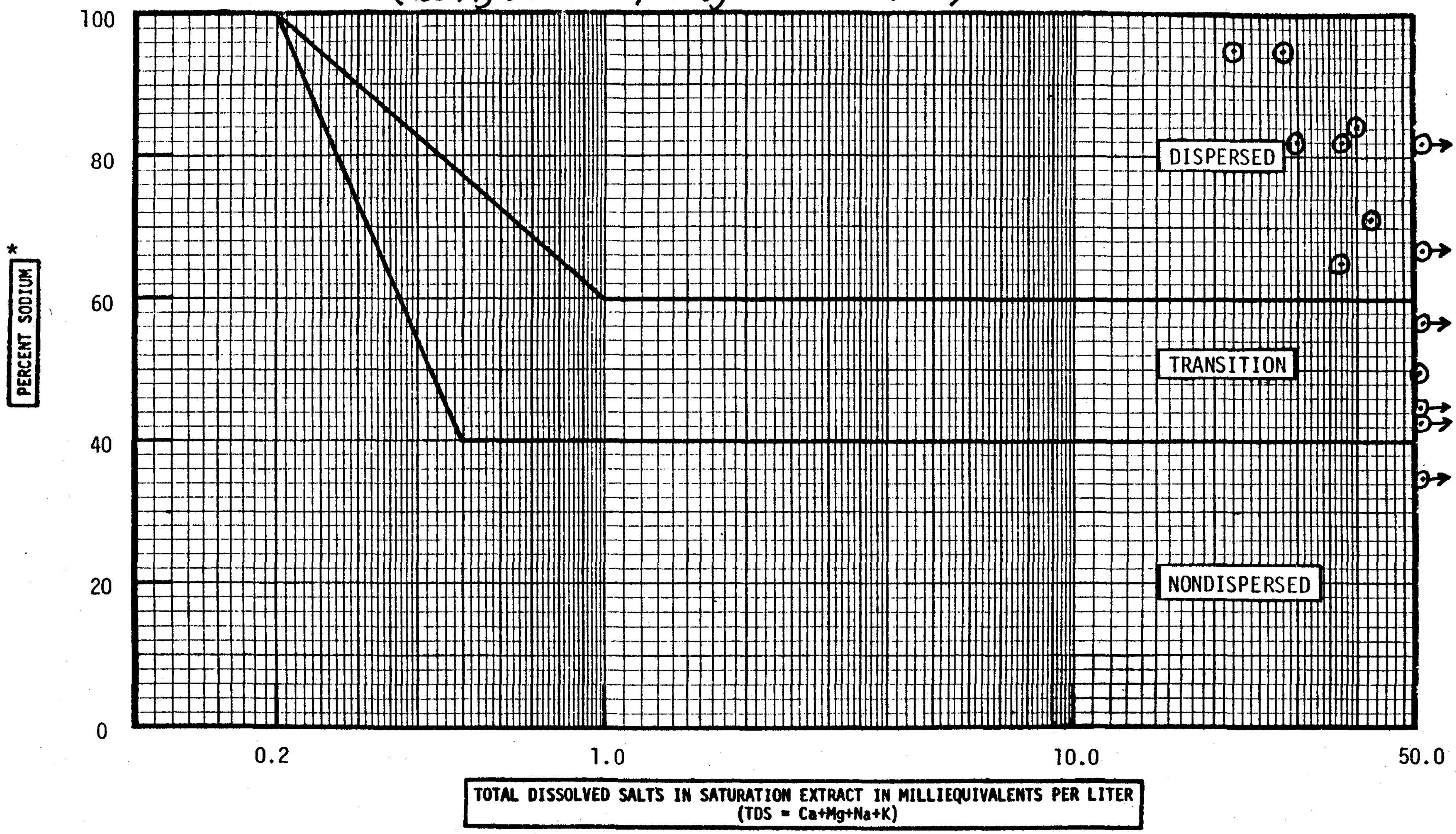
X-ray diffraction of whole soil (< 2mm) dominated by peaks from quartz and feldspars. Mica minerals and calcite were also detected. D.H. U of A

5 = Dominant
4 = Large Amt.
3 = Medium Amt
2 = Small Amt.
1 = Trace Amt.
0 = Looked for, but not det.

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	RELATIONSHIP OF PORE WATER SALTS AND DISPERSION

PROJECT and STATE	BY	DATE
Williams-Chandler W/S, RWCD, Reach I, Arizona	cthmc	8/85

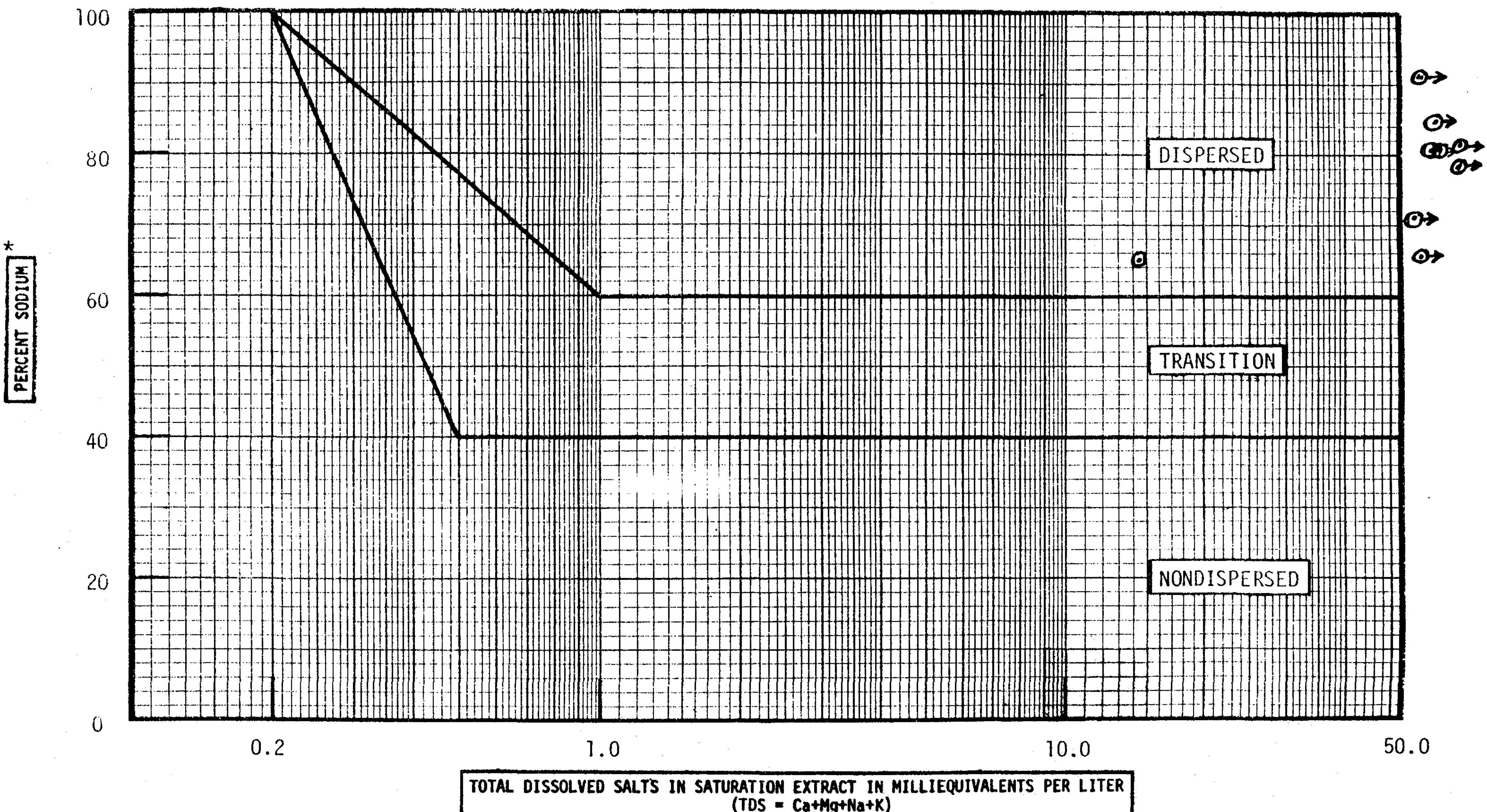
(Testing on 14 samples by Univ. of Arizona)



* Percent Sodium = $\frac{Na(100)}{TDS} = \frac{Na(100)}{Ca+Mg+Na+K}$ (all measured in milliequivalents per liter of saturation extract)

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	RELATIONSHIP OF PORE WATER SALTS AND DISPERSION

PROJECT and STATE	BY	DATE
Williams-Chandler W/S, RWCD, Reach I, Ariz.	cHHC	9/13/85



* Percent Sodium $= \frac{Na(100)}{TDS} = \frac{Na(100)}{Ca+Mg+Na+K}$ (all measured in milliequivalents per liter of saturation extract)

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	CHEMICAL TEST RESULTS
PROJECT and STATE Williams - Chandler W/S RWCD Reach I Arizona	DATE 8-22-85	JOB NO. 5018

SML NO. F85	SSIL LAB. NO.	SATURATED PASTE RESIST. ohm-cm	SATURATED PASTE pH	SATURATED PASTE H_2O %	Na SAR	TOTAL SOLUBLE SALTS	SATURATION EXTRACT MEQ/LITER Ca	Mg	Na	K	TOTAL	% Na	DISPERSION CLASS.
2389	85Q2861	1260	7.17	56.72	6	528	3.52	1.23	9.51	.30	14.56	65.3	D
2390	2862	100	7.23	63.91	55	12,450	31.45	13.30	259.38	.25	304.38	85.2	D
2391	2863	305	7.71	69.96	37	3,200	3.34	2.93	65.05	.05	71.37	91.1	D
2392	2864	395	7.30	54.73	12	2,120	16.30	4.10	39.90	.33	60.63	65.8	D
2393	2865	100	7.44	68.95	46	12,980	37.85	16.50	239.08	.68	294.11	81.3	D
2394	2866	100	7.53	72.32	48	13,830	32.60	19.30	246.38	.55	298.83	82.4	D
2395	2867	290	7.62	54.21	20	3,860	24.55	6.99	79.04	.57	111.15	71.1	D
2396	2868	90	7.41	54.74	55	15,130	62.85	17.45	350.84	.59	431.73	81.3	D
2397	2869	50	7.14	67.58	62	28,320	89.88	50.05	514.43	.46	654.82	78.6	D

Financial Project WF-08 Activity Code 37 State Worked 48 Location Code 0028

Sheet 1 of 1

United States
Department of
Agriculture

Soil
Conservation
Service

201 East Indianola Ave.
Suite 200
Phoenix, Arizona 85012

Subject: ENG-RWCD Floodway-Reaches I & II
Sedimentation Problems

Date: May 10, 1985

To: Ralph M. Arrington, SCE

File Code: 210

On May 2, 1985 I inspected that portion of the RWCD, Reaches I and II, which is downstream of the concrete chute section which ends at approximately station 1160+00.

The general overall condition which exists from station 1160+00 to the juncture of the RWCD with the Gila River is one of alternating channel bottom scour and sediment deposition. Sediment deposition occurs intermittently throughout the reaches. The depositional features take on various forms, ranging from long, low longitudinal bars to small mound-like deposits around vegetation and debris. In some areas the mound-like deposits are highly concentrated and create a very rough channel bottom condition much like a ripple and dune bed form. In many instances the longitudinal bars have been breached, thereby creating channel crossings during low flow conditions.

Channel scour is in the form of both sheet scour and localized scour. Scouring of the channel bottom is evident intermittently throughout the area inspected. It is difficult to estimate the thickness of compacted earth lining which has been removed by sheet scour by visual inspection of the surface. Scour holes which have developed as a result of localized scouring are as much as 2 feet deep. For example, a scour hole located approximately 75 feet from the right toe at station 1179+00 is about 8 feet wide, 13 feet long and 2 feet deep.

Several conditions exist which may have contributed to scour initiation. These include but are not necessarily limited to the following:

1. Accumulation of sediment deposits in the channel, thereby causing flow constrictions and accelerated velocities.

2. Accumulation of debris and vegetation, thereby restricting flow and causing accelerated velocities around these obstructions.

3. Dessication cracking of compacted earth lining thereby increasing vulnerability to erosion.

4. Deficiency in supply of sediment for transport.

Dessication cracks are present in the compacted earth lining in areas where it is not covered by sediment. This makes the blanket more susceptible to erosion.

A bedload transport analysis of this reach was made in 1975. This analysis showed that the 10 year (Q 10) flow event would be capable of transporting 2528 tons of bedload sized material. Larger flows, with longer durations increase the transport capacity. For example, the calculated transport capacity for the Q 100 for bedload sized material is 6935 tons. No bedload material is available for transport in this reach except materials which may be detached from the channel floor or banks. Therefore, if their erosion resistance is decreased the flows may be adequate to create erosion problems.

It was also noted that at intermittent locations the bank slope toes are being eroded. In some places the toes have been eroded to the point where vertical banks as high as 3.5 feet have been cut.

It is concluded that unless remedial maintenance measures are undertaken soon the channel will continue to be damaged by future flows. The rate of damage is expected to accelerate.

Further evaluation of present conditions is recommended. In order to determine the amount of scour, bank erosion, and sediment deposition I recommend that channel cross sections be surveyed prior to any maintenance work being performed, provided this can be accomplished in the near future. I do not believe that a large number of cross sections are required, if the locations are carefully chosen in the field by someone who understands the survey objectives. It would also be desirable to have the assistance of the WNTC sedimentation geologist in evaluating this channel reach. Annual inspections should include a very thorough appraisal of the above mentioned factors so that maintenance can be scheduled on a timely basis in order to prevent further deterioration.

It is recommended that the erosion of the toes of the bank slopes be repaired as soon as possible. A large channel flow could be very damaging if this remains unattended.

Aubrey C Sanders Jr.

Aubrey C. Sanders, Jr.
State Geologist

ATTACHMENT 11

STABILITY DESIGN

1. Design Approach--Tractive Power

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

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

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

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

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

7. Channel Geometry Correction Factors

8. Wide Smooth Channels in Earth Material

9. Saturated Undisturbed Samples

Lewis J. Mathers, PhD, PE
17 September 1985

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,

$$\tau_{\infty} V = (\gamma d S_t) V \tag{2}$$

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,

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

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,

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

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

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 <u>1 and 2</u> 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. <u>Partitioning of the Total Tractive Stress--Different Methods to Compute the Reference Tractive Stress</u>

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

- the magnitude of the reference tractive stress, <u>and</u>,
- 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

$$\tau = \tau' + \tau'' \tag{6}$$

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

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

One way to divide the total shear is

$$\tau = \gamma R S_e = \gamma R S' + \gamma R S'' \tag{7}$$

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 $\leq D_{75} \leq$ 127mm), TR-25 uses the reference tractive stress $\tau_\infty = \gamma d S_t$, 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

$$\tau = \gamma R S_e = \gamma R' S_e + \gamma R'' S_e \tag{8}$$

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

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

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

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

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}

$$\tau_{ref} = \tau_{ave} = \gamma RS_e \tag{9}$$

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 D75 intersection was approximately 0.30 mm, the $\tau_R < \tau_\infty$ 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 overstressed.

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_m

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

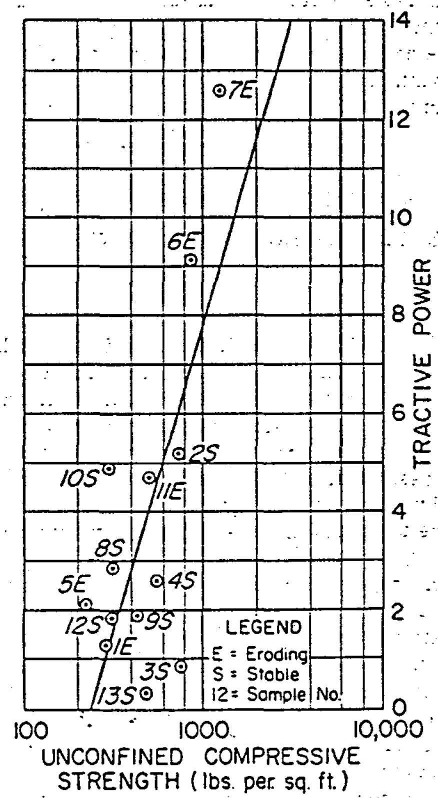


FIG. 2.—RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND TRACTIVE POWER

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.

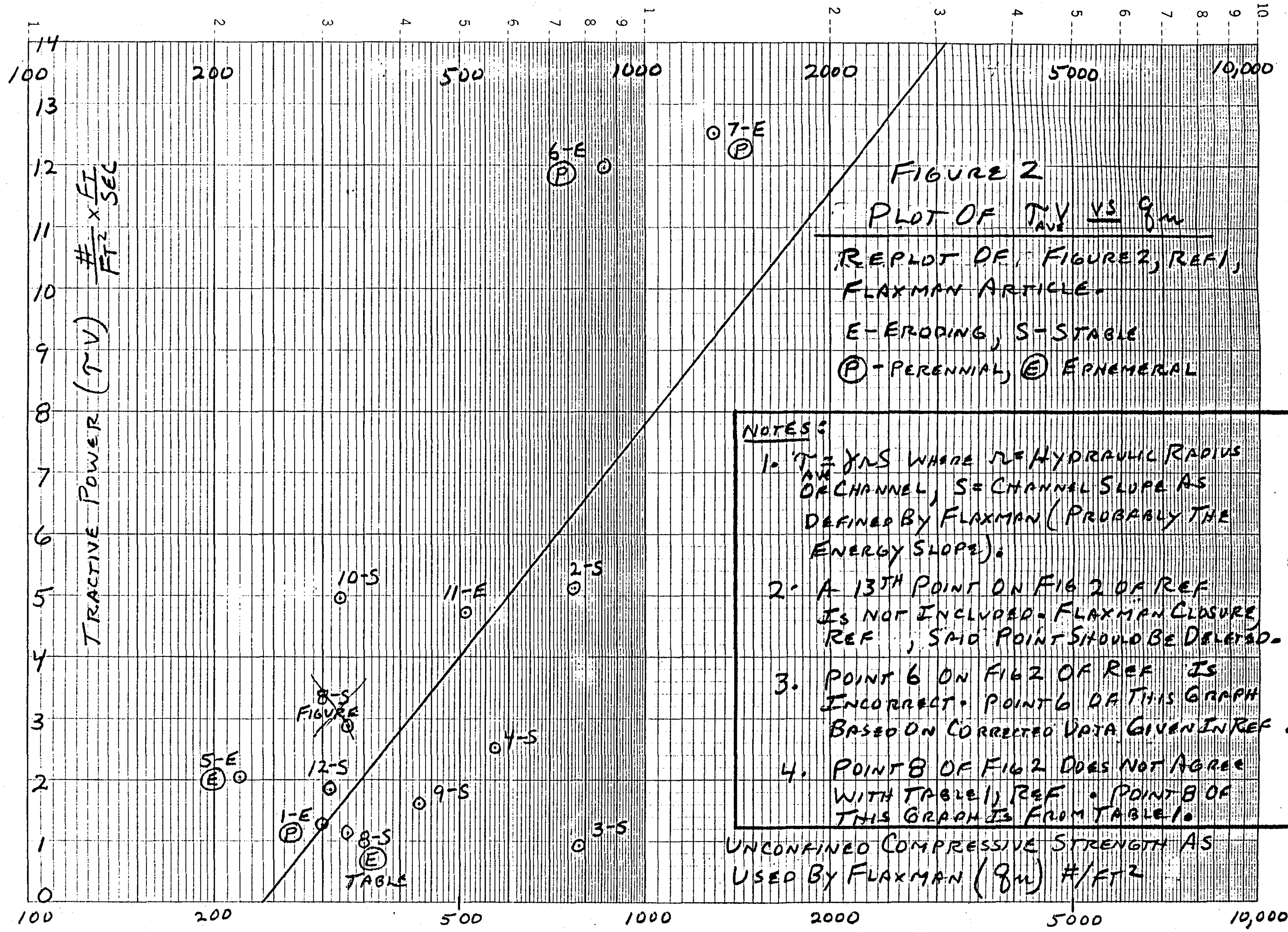


FIGURE 2

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. $\tau_{AVE} = \gamma R S$ 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 $\tau_{AVE} = \gamma R S_e$

REFER TO GRAPH (FIGURE: 2)

		Ref. 1			Ref 2	Ref 1	Ref 2
Sample and Site No. (1)	Stream (2)	(Use As S_e) Channel Slope, in feet per foot (3)	R Hydraulic Radius in feet (4)	V Average Velocity in feet per second (5)	τ_{AVE} #/FT^2 (6)	Tractive Power $\tau_{AVE} V$ (7)	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 — $\tau_\infty V$ USING $(\tau_{REF} = \tau_\infty) = \gamma d S_t$

REFER TO GRAPH (FIGURE 3)

Sample and Site No. (1)	Stream (2)	REF 1: (Use As S_e) Channel Slope, in feet per foot (9)	REF 1: V FPS (10)	REF 3: m_t (11)	REF 3: τ_∞ #/FT² (12)	REF 3: $\tau_\infty V$ (13)	TABLE 1 COL(8)/2: $g_m/2$ (14)	REF 3: d FLOW DEPTH FT (15)	COMPUTED: D_{75} IN (16)	COMPUTED: D_{75} MM (17)	COMPUTED: S_t (18)	COMPUTED: $\frac{S_t}{S_e}$ (19)	COMPUTED: m (20)	COMPUTED: $\frac{m_t}{m_e}$ (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	0.1101	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	0.02885	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

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

$\therefore D_{75} = (39 m_t)^6$

$1'' = 25.4$ MM

$\tau_0 = \gamma d S_t$

$\therefore S_t = \frac{\tau_0}{\gamma d}$

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

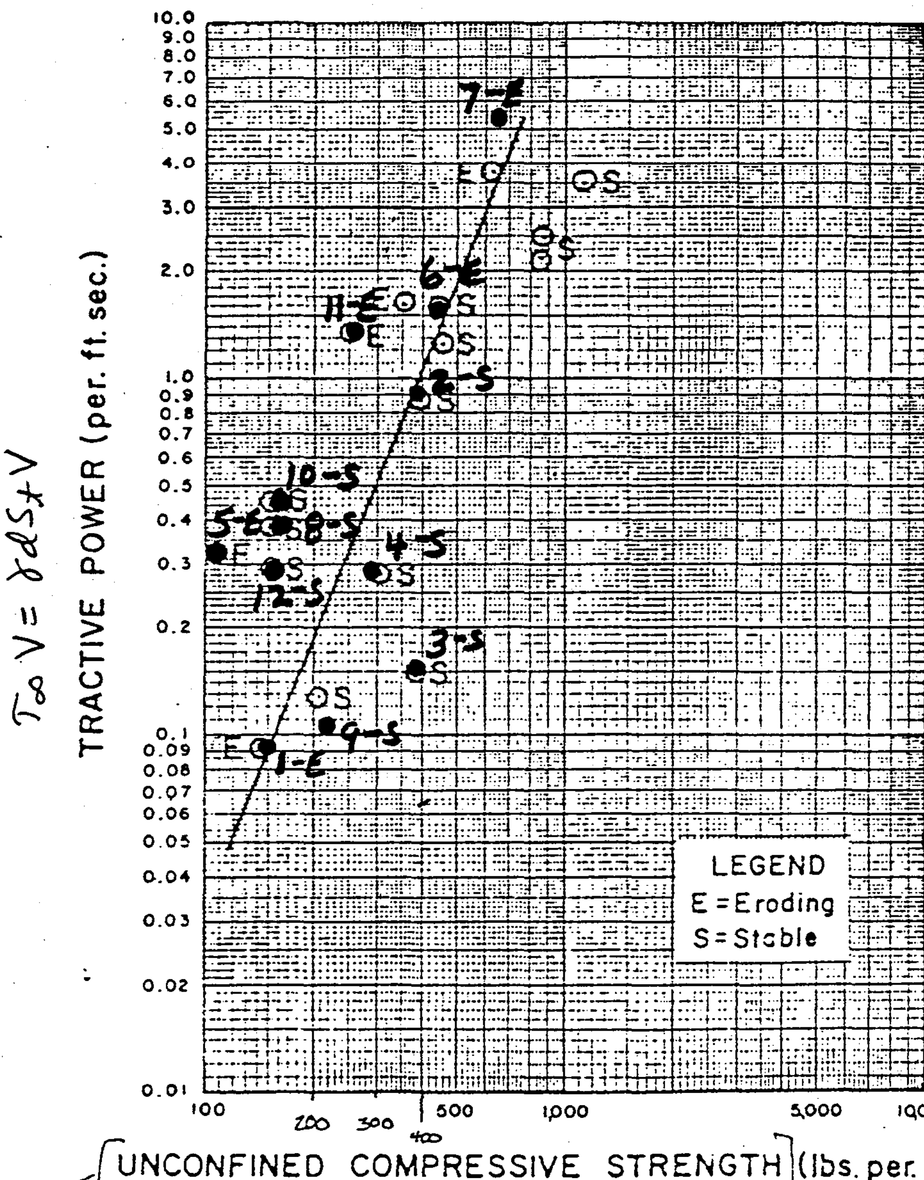
$\therefore m = m_t \left(\frac{S_e}{S_t}\right)^{1/2}$

FIGURE 3

PLOT OF $T_\delta V$ VS $\left(\frac{q_u}{2}\right)$

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.



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.

$q_u/2$

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

$q_u/2$

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

REFER TO GRAPH (FIGURE 4)

		REF 1		COMPUTED	SEE TABLE 2		COMPUTED				TABLE 1 COL (8)/2	COMPUTED
Sample and Site No. (1)	Stream (2)	(USE AS S_e) Channel Slope, in feet per foot (22)	V FPS (23)	$\frac{V^3}{g\nu S_e}$ (24)	D_{75} IN (25)	D_{75} FT (26)	$\frac{V}{\sqrt{g K_s S_e}}$ $K_s = D_{75}$ (25)	$\frac{V}{\sqrt{\tau/\rho}}$ (28)	τ_{R*} #/FT² (29)	$\tau_{R*}\,V$ (30)	$q_m/2$ #/FT² (31)	R_* FT (32)
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	0.6563
11	Willow Brook, Sonoma Co., Calif.	0.00200	7.95	7.80×10^8	0.00526	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

$g = 32.2\ FT/SEC^2$
ASSUME $T = 75^\circ F$
$\nu = 1.0\times10^{-5}\ FT^2/SEC$

K_s SHOULD BE D_{75} IN FT.

$\rho = 1.94\ SLUGS/FT^3$

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

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

FIGURE 4
PLOT OF $T_{R*} V$ VS $\left(\frac{q_u}{2}\right)$

FLAXMAN DATA – GRAPH IN CURRENT EDITION OF TR-25

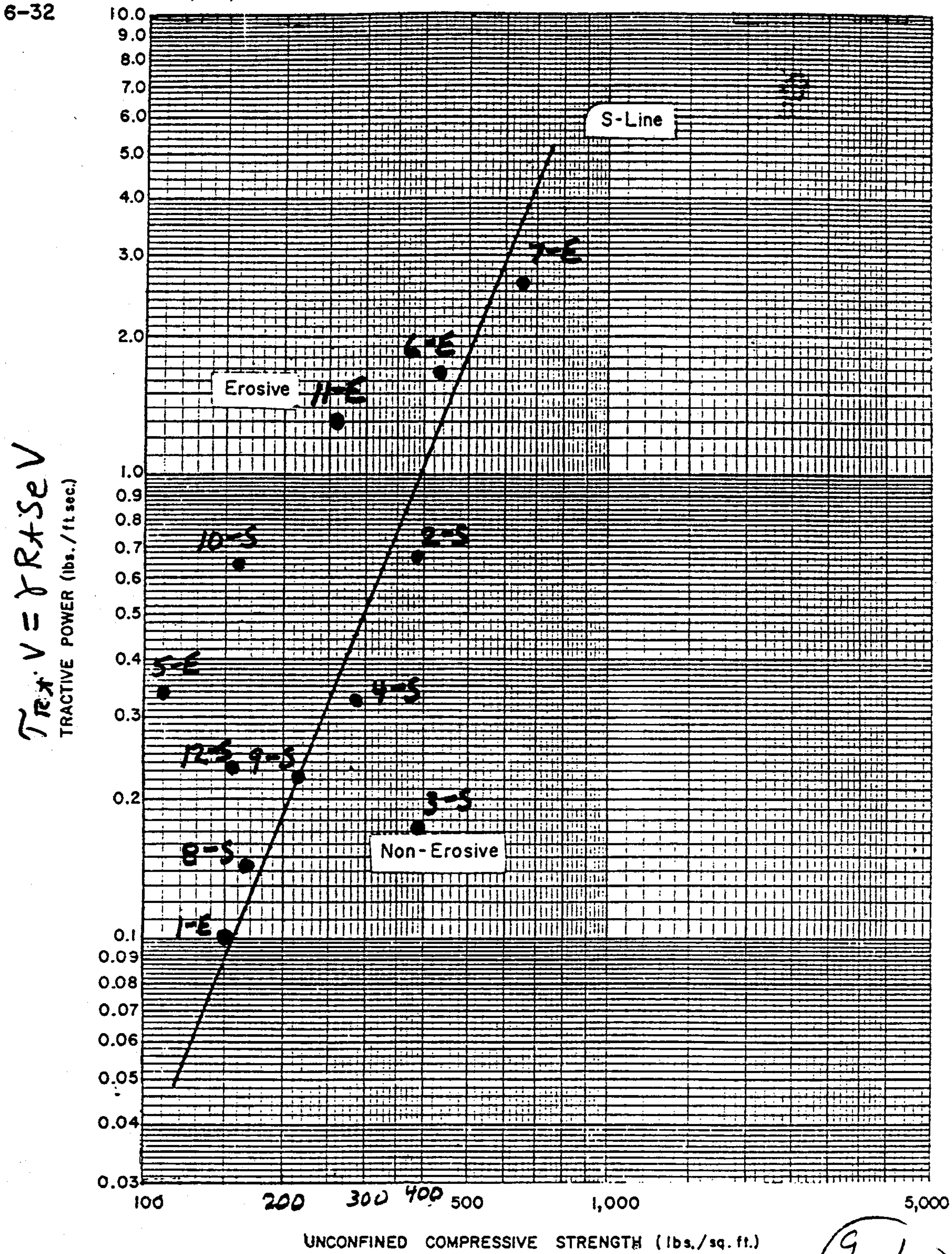


Figure 6-15
Unconfined Compressive Strength And Tractive Power As Related To Channel Stability

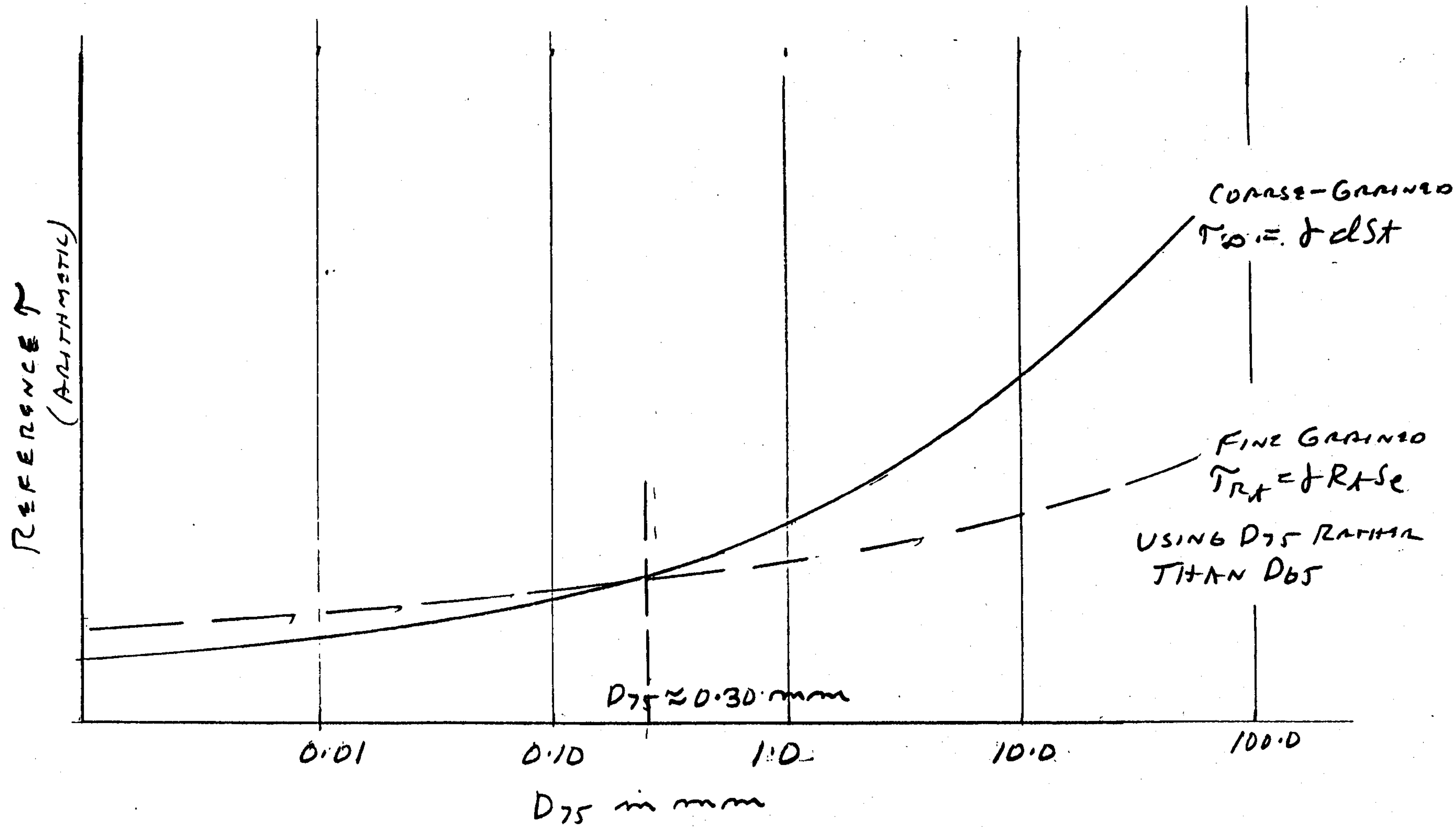
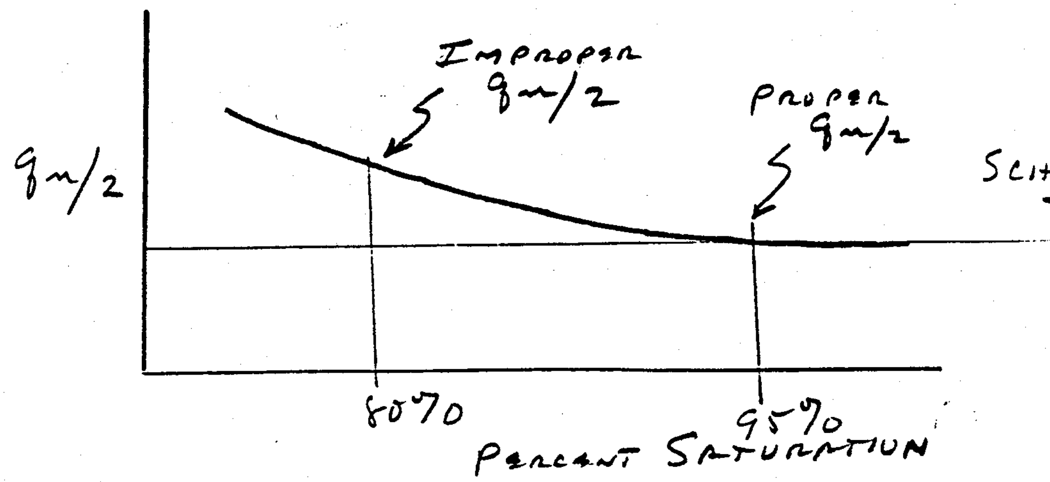


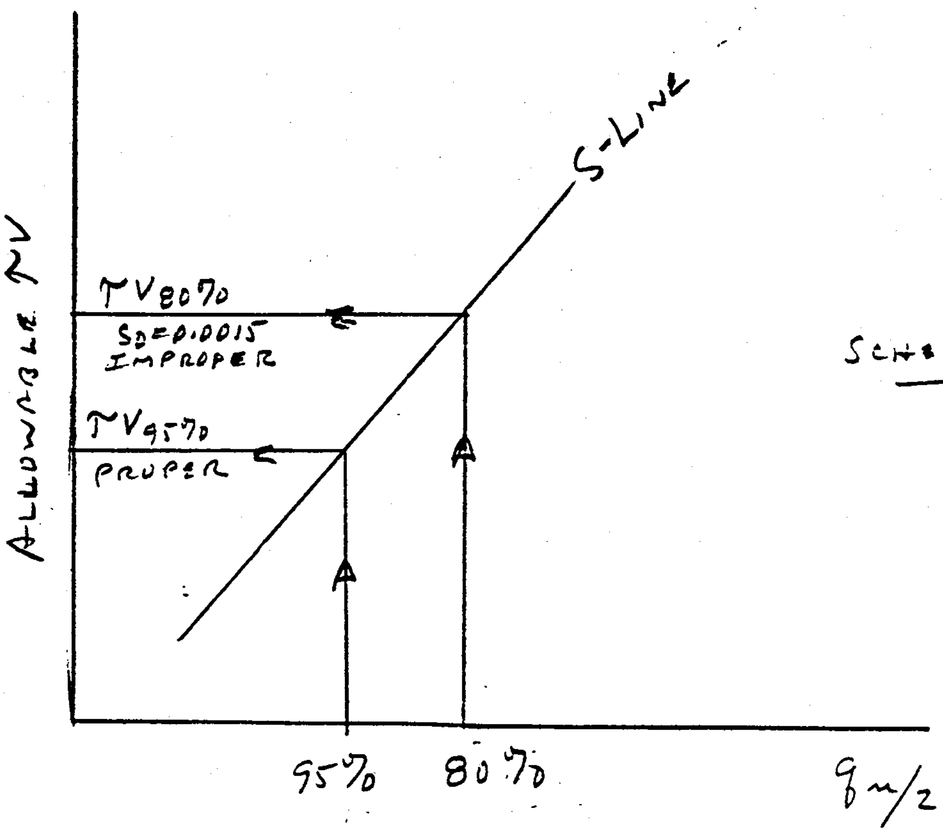
FIGURE 5 — SCHEMATIC OF RELATIVE TRACTIVE STRESS VS D_{75}

Figure 6

Effect Of Percent Saturation Of Measured $q_u/2$ Of Channel Material



Schematic a



Schematic b

Photo #1 - Shows holes exposed by excavation for repair of jug holes in dike area - Sta 1241 + 50

Photo #2 - Looking upstream in riprap-lined section showing sand bars - Sta 1241 + 50

Photo #3 - Shows close-up of riprap and deposition - Sta 1241 + 50

Photo #4 - Looking upstream at localized scour (2.5 to 3 foot deep) at downstream end of riprap section - Sta 1250 + 00

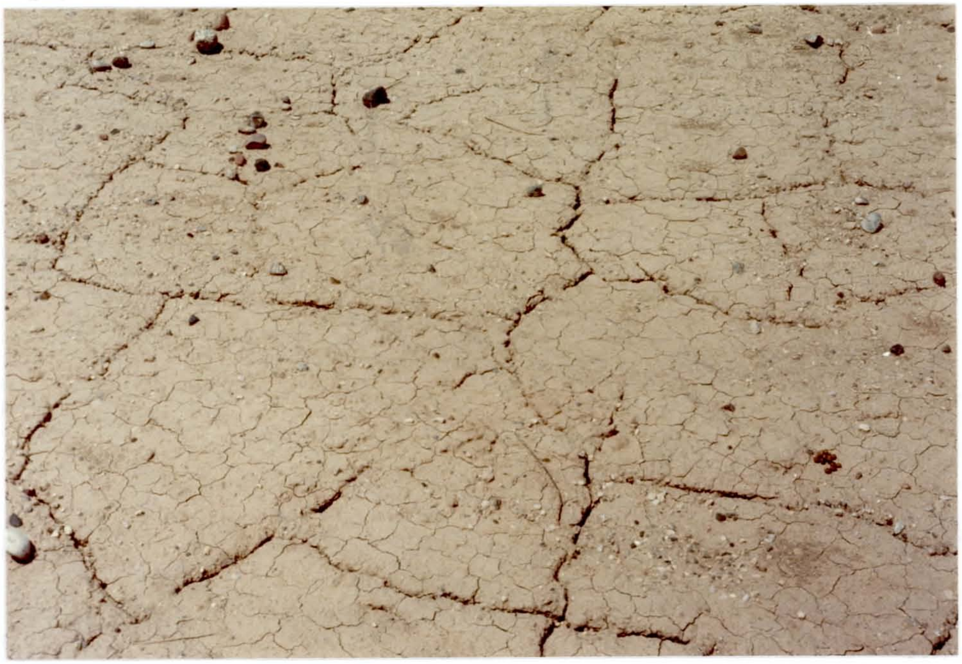


Photo #5 - Shows typical cracking pattern in earth liner - just downstream of Sta 1250 + 00

Photo #6 - Shows bank toe erosion - about 3 foot cut

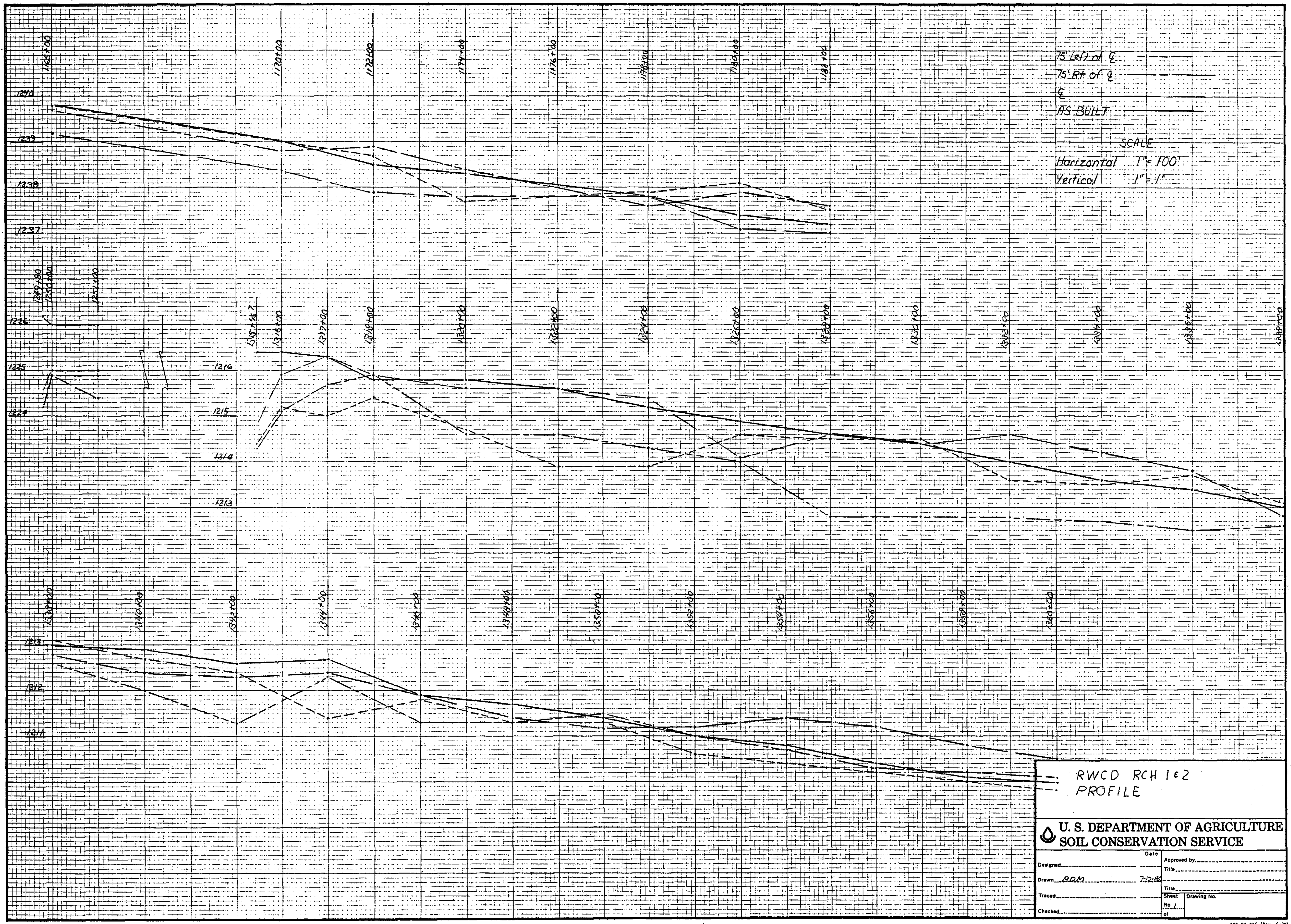
Photo #7 - Looking downstream at right bank - same location as photo #6

Photo #8 - Shows rills on channel bank - 2 to 3 inches deep

Photo #9 - Looking downstream showing sub-channel in main channel -
Sta 1317 + 00

Photo #10 - Shows local scour of soil liner immediately downstream of riprap road crossing - Sta 1317 + 00

Photo #11 - Shows jug hole in crest of dike at downstream end of Reach I


75' Left of ℄
75' Rt of ℄
℄
AS-BUILT
SCALE
Horizontal 1" = 100'
Vertical 1" = 1'
1240
1239
1238
1237
1226
1225
1224
1216
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RWCD RCH 1 & 2
PROFILE
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
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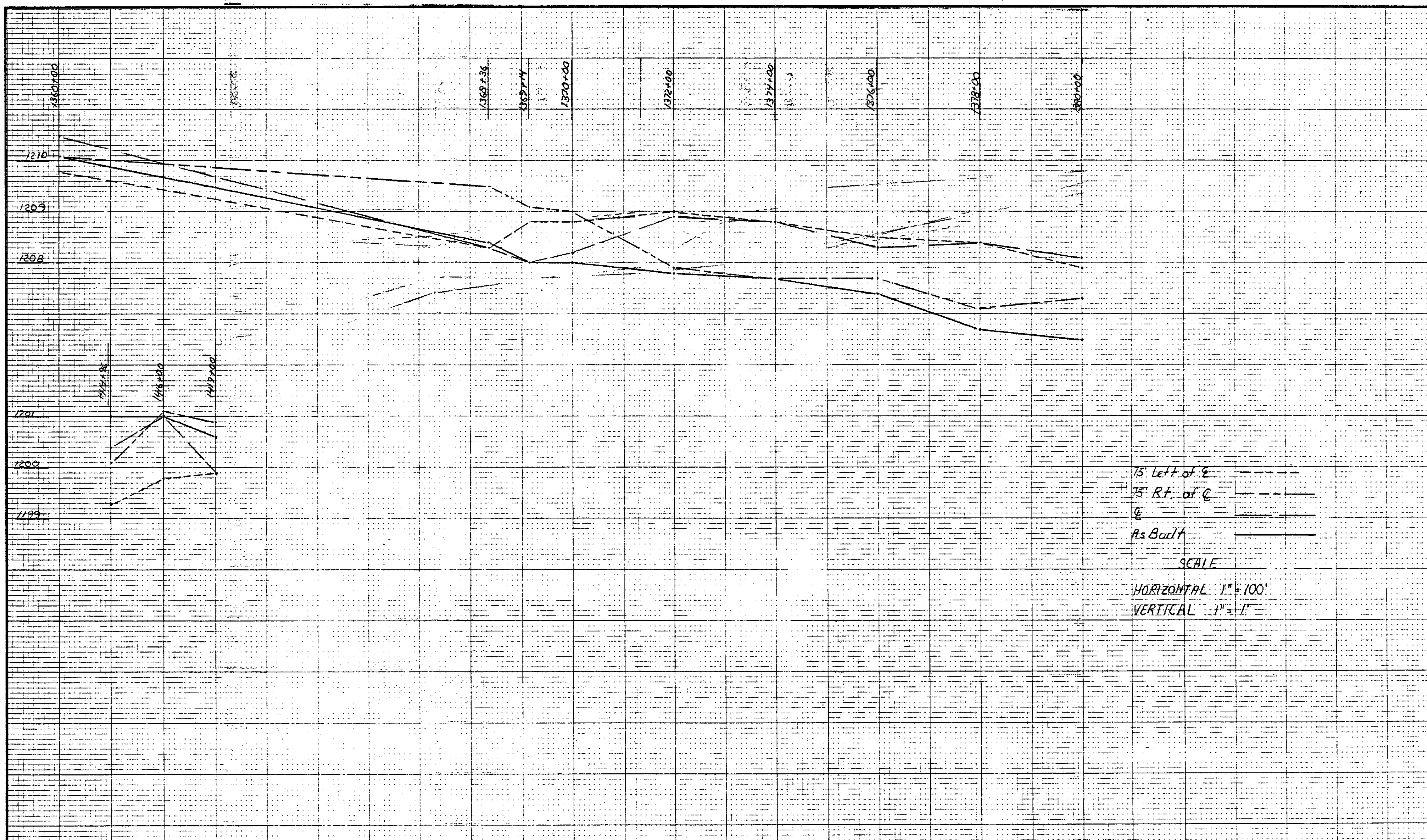

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

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Drawn RDM	7-15-85	Title
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Checked		Sheet No 2 of — Drawing No.

SCS-E4-315 (Rev. 6-79)

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