ENGINEERING REPORT

SADDLEBACK DIVERSION
HARQUAHALA VALLEY WATERSHED
MARICOPA COUNTY, ARIZONA

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U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

MARICOPA COUNTY, ARIZONA

February 1987

ABSTRACT OF ENGINEERING REPORT

General Description of Problem or Deficiency: Overtopping of grouted rock side inlets and breached earthen collector channels for a floodwater diversion.

Location: West of Saddleback Mountain, Harquahala Valley, Maricopa County, Arizona.

Type of Facility: Diversion providing agricultural flood protection.

Job Class: Class VII (NEM Part 501).

Size:

Diversion

Design Discharge = 1,120 - 6,060 cfs.
Bottom Width = 26 - 560 ft.
Side Slopes = 3:1 and 5:1
Excavation = 2 - 10 ft.

Side Inlets (17 grouted rock grade control structures)

Design Discharge = 85 - 1,020 cfs.
Drainage Area = 0.02 - 1.17 sq. mi.
Bottom Width = 10 - 60 ft.

Collector Channels

Design Discharge = 50 - 320 cfs.
Bottom Width = 10, 20, 30 ft.
Length = 197 - 2,510 ft.
Depth = 2.0 ft.

Problem Category:	Earth - external erosion, side inlets to diversion channel.	Site Name:	Harquahala Valley Watershed Saddleback Diversion
Practice Standard:	400 and 410	State:	Arizona

Date of Installation: 1981 to 1982

Saddleback Diversion is located on an alluvial fan. The diversion takes the principal spillway outflow from Saddleback FRS and the runoff from Saddleback Mountain and outlets these flows into an unnamed tributary to the major drainage of Centennial Wash. Grouted rock side inlets were placed at points of existing washes along the diversion channel to inlet intercepted runoff into the channel. Earthen collector channels extend perpendicular to the side inlets. They collect and divert overland flow to the side inlets.

The diversion, side inlets, and collector channels were designed for the 50-year 24-hour storm. On September 2, 1984, an intense storm produced peak discharges that exceeded design for the middle and lower reaches of the diversion. The diversion performed remarkably well, conveying up to twice design discharge without damage.

The storm changed the location of several washes and caused flow braiding and erosion on the alluvial fan above the diversion. Runoff water by overland flow and washes deposited sediment bars in the earthen collector channels. The channels were overtopped and breached. Fifteen of the 17 grouted rock side inlets were overtopped or flanked by erosion through the abutments or through the collector channels adjacent to the inlets.

The primary cause of the damage to the side inlets and collector channels was because the storm exceeded design for most of the problem areas. Responsibility is assigned to natural occurrences. There were, however, three areas of shortcomings in the design: (1) The side inlets were incorrectly sized and were too small even for the 50-year storm. (2) The side inlets and collector channels were designed and constructed without freeboard. (3) No allowance was made for sediment accumulation in the collector channels.

Incorrect sizing of the side inlets could have been prevented by a detailed review of design computations. The designer(s) followed established policy in not providing freeboard or accounting for sediment since the side inlets and collector channels are considered minor structures. Arizona SCS policy regarding freeboard and sediment for structures of this type should be re-examined.

Remedial Treatment 1/ The investigating engineers recommend the repair of inlet structures and breached collector channels, the construction of collector channels and dikes to cover the entire length of the diversion, the enlargement of the collector channels to accomodate some sediment accumulations, and adding additional side inlets at locations where new major washes now exist.

1/ For additional information contact:

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

Include a copy of abstract with request.

REPORT OF INVESTIGATION COMMITTEE

FOR

HARQUAHALA VALLEY

WATERSHED

SEPTEMBER 2, 1984 STORM OF SADDLEBACK DIVERSION

BY

Original Committee

William E. Payne, Jr.

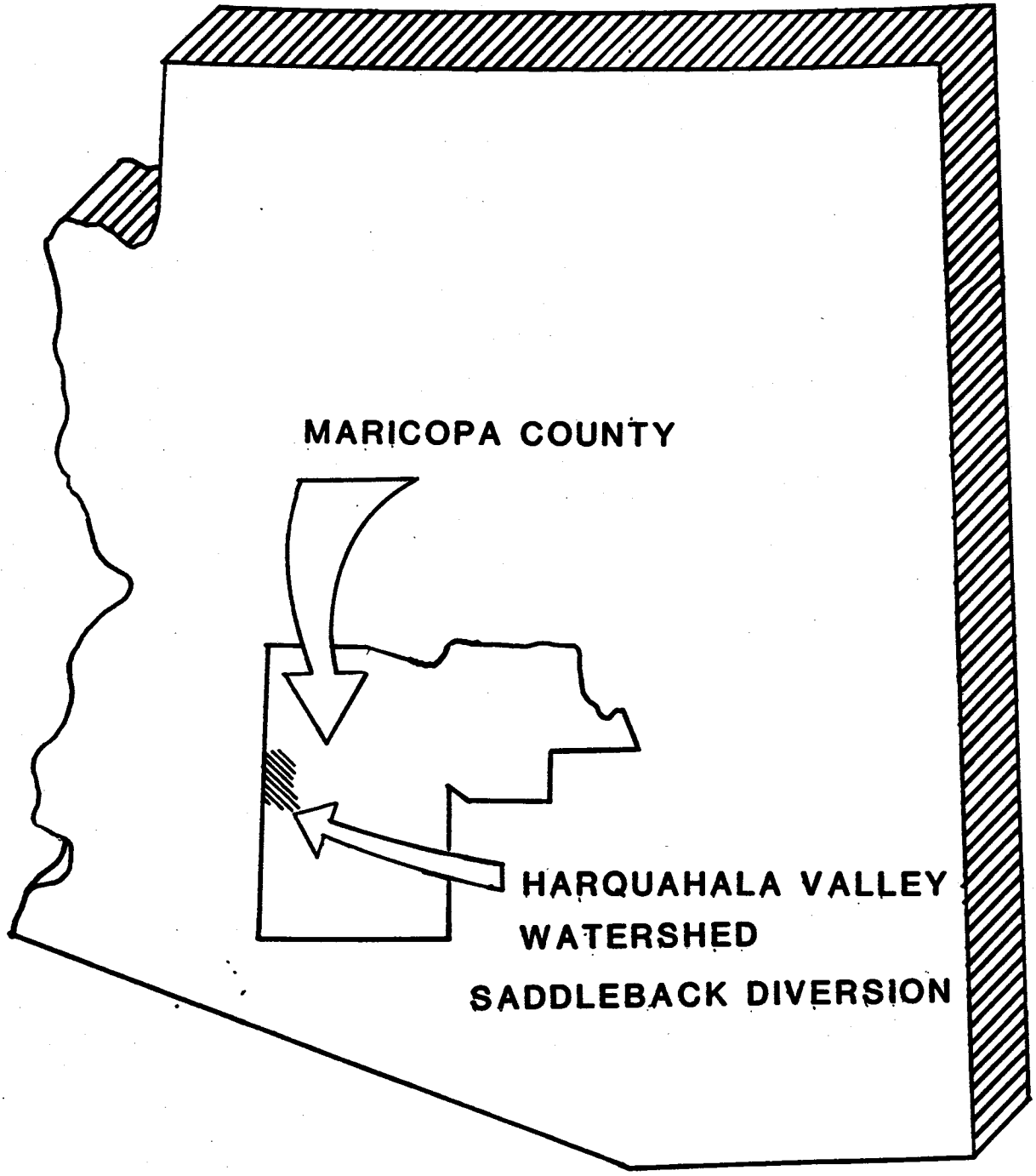
Herman N. McGill

Present Committee

John E. Weaver ,Chairman

Harry C. Millsaps ,Member

LOCATION MAP



ARIZONA

ENGINEERING REPORT
for
Saddleback Diversion
Maricopa County, Arizona

TABLE OF CONTENTS

	Page
Location Map	i
Project	1
Location	1
Site	1
Appropriation	1
General Description of Problem	1
Authority	1
Composition of Committee	1
INVESTIGATIONS	
Project Data	2
Damages	3
Committee Activities	3
Interviews	3
EVALUATION	
Design	4
Experience in Similar Design	4
Side Inlets	4
Collector Channels	4
Sediment	5
Construction	5
CONCLUSIONS	5
RECOMMENDATIONS	6
SUMMARY	7

TABLES

No.		Page
1.	Side Inlets - Comparison of Design, As-Built, and Storm Discharge	9
2.	Collector Channels - Comparison of Design and As-Built Discharges	10

PHOTOGRAPHS

		Page
1.	Station 7+00, Side Inlet #1, No Damage.	11
2.	Station 14+00, Side Inlet #2, Sediment Bars in Collector Channels Caused Overtopping of Dike.	11
3.	Station 19+69, Drop #1, Some Overbank Erosion.	12
4.	Station 29+00, Side Inlet #3, Some Rill Erosion Due to Overtopping.	12
5.	Station 41+61.6, Side Inlet #4, Erosion from Overtopping.	13
6.	Station 52+00, Side Inlet #5, Overtopped Collector Dike for Approximate 200 Feet.	13
7.	Station 61+25, Side Inlet #6, No Damage.	14
8.	Approximate Station 63+75, Collector Channel and Dike Overtopped, Severe Erosion.	14
9.	Station 80+12, Inlet #7, Severe Erosion on North Side from Overtopping.	15
10.	Station 80+20, Side Inlet #7, Looking West. Severe Erosion on South Side from Overtopping.	15
11.	Station 81+83, Drop #2, Erosion from Overtopping.	16
12.	Station 90+08.5, Side Inlet #8, Erosion from Overtopping on Both Sides.	16
13.	Approximate Station 99+00, A New Gully 5 Feet Deep and 10 Feet Wide and Severe Erosion from Overtopping.	17
14.	Station 108+14.5, Side Inlet #9, Overtopping of Collector Dikes.	17
15.	Station 108+14.5, Inlet #9, Erosion on North Side.	18
16.	Station 108+14.5, Inlet #9, Erosion on South Side.	18
17.	Station 121+26, Side Inlet #10, No Damage.	19
18.	Station 128+32, Side Inlet #11, Erosion from Overtopping.	19

Page

19. Approximate Station 133+00, Severe Erosion from Gullying. . 20

20. Station 144+82, Drop #3, Minor Rill Erosion. 20

21. Station 157+96, Side Inlet #12, Erosion from Overtopping on both sides. 21

22. Station 170+05, Side Inlet #13, Erosion from Overtopping on Both Sides. 21

23. Station 173+98, Side Inlet #14, Erosion from Overtopping on Both Sides. 22

24. Station 183+65, Side Inlet #15, Erosion from Overtopping on Both Sides. 22

25. Approximate Station 200+00, Severe Erosion from Gullying. 23

26. Station 202+00, Side Inlet #16, Erosion on South Side from Overtopping. 23

27. Station 220+80, Side Inlet #17, Erosion from Overtopping on Both Sides. 24

APPENDICES

No.

A. Hydrologic Report by Herman C. McGill, entitled "Saddleback Diversion, Study of September 2, 1984 2, 1984 Storm and Comments on McGill Report by Harry C. Millsaps (memo of 1-29-85)

B. As-Built Drawing, Sheet 20, Inlet Weir Details. Computations - Capacity of Side Inlets and Collector Channels.

C. Comments on 1st Engineering Report.
Sergent, Hauskins and Beckwith - February 12, 1986
Robert R. Koons, PE - February 2, 1986
PRC Engineering - March 11, 1986

D. Comments on Present Engineering Report.
Edward A. Adair, P.E. - April 17, 1987
PRC Engineering - April 27, 1987

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

PHOENIX, ARIZONA

February 1987

ENGINEERING REPORT

STORM OF SEPTEMBER 2, 1987 - SADDLEBACK DIVERSION

Project: Harquahala Valley Watershed Project

Location: Maricopa County, Arizona

Site: Saddleback Diversion

Appropriation: PL-566

General Description of Problem:

An intense storm on September 2, 1984 caused overtopping of grouted rock side inlet structures and breaching of collector channels that collect and divert overland flow to the side inlets. Damage consists of erosion around the abutments of the side inlets; collector channels filled with sediment, overtopped, eroded and breached.

Authority:

Letters dated January 29, 1985 and November 4, 1986 from Verne Bathurst, State Conservationist.

Composition of Committee:

This report was prepared by two separate committees. The original committee, appointed January 29, 1985 was:

William E. Payne, Jr., State Design Engineer, SCS, Phoenix, Arizona

Herman N. McGill, Hydraulic Engineer, SCS, Temple, Texas

On November 4, 1986, the present committee was appointed for the purpose of revising the original report and incorporating comments received from the West Technical Service Center (memo of May 22, 1986 from Donald Wallin). The present committee is:

John E. Weaver, Asst. State Conservation Engineer, SCS, Phoenix, Arizona (Chairman)

Harry C. Millsaps, Hydraulic Engineer, SCS, Phoenix, Arizona (Member)

INVESTIGATIONS

PROJECT DATA

Saddleback Diversion is a Class VII structure that is 4.73 miles long, constructed across an alluvial fan. The drainage area above the diversion is 8.6 square miles. The diversion takes the principal spillway outflow from Saddleback FRS and runoff from the intervening area and outlets into an unnamed tributary to Centennial Wash. The geometery of the diversion is a dike with a 12 foot top width and side slopes of 3 to 1 and 4 to 1. The channel has a variable bottom width with 3 to 1 and 5 to 1 side slopes. Approximately 1,900 feet of the channel at the upper end is rock riprapped. There are four grouted rock drop structures for grade control and velocity reduction within the channel.

There are 18 side inlets at the points where the diversion intercepts washes that drain the west slopes of Saddleback Mountain. Seventeen of these inlets are grouted rock grade control structures referred to as "side inlets". The No. 18 inlet is a earth channel outletting at channel grade. The grouted rock inlets vary in bottom width from 10-60 ft. The grouted rock is 2.0 ft. thick. The weir inlet is 2.0 ft. deep for all of the inlets. A detail drawing and other data on these structures is shown in Appendix B, which is a copy of sheet 20, As-Built Plans.

Collector channels, referred to on the plans, in the design file, and in the construction records as "lateral swales", were constructed to direct overland flow to the side inlets. These channels were 2.0 ft. deep, with 3:1 side slopes, and a bottom width of either 10, 20, or 30 ft. (see Appendix B, As-Built Drawing). The channels vary in length from 197 ft. at Inlet No. 4 to 2,510 ft. at Inlet No. 12.

The diversion, side inlets, and collector channels were designed for a 50-year, 24-hour storm. The design discharge for a specific collector channel was assumed to be 1/3 the design discharge of the side inlet.

Saddleback Diversion was designed by PRC Toups, a consulting engineering firm. Construction inspection was done by the firm of Sergent, Hauskins, and Beckwith.

DAMAGES

The 50-year, 24-hour design capacity for the diversion varies from 1120 cfs at the Saddleback FRS outlet to 6060 cfs at the diversion outlet. On Sunday, September 2, 1984 a storm event passed through the watershed in a northwest to southeast direction with an approximate duration of 4 hours. This storm event produced an estimated outflow of 739 cfs at the Saddleback FRS outlet and 12,355 cfs at the diversion outlet. The diversion functioned perfectly during and after the storm with the water level within one foot of the top of the diversion dike. There was very little or no erosion to the diversion dike or channel. The major erosion was around and between the side inlets into the diversion channel.

The runoff water from overland sheet flow and side washes caused sediment bar development in the collector channels which were overtopped by floodwaters.

Fifteen of the 17 side inlets were overtopped or flanked by erosion through the abutments or through the collector channels adjacent to the inlets. The storm produced discharges of up to twice the design discharges in the lower reaches of the diversion but, the side inlets and collector channel in the upper reaches, where the design discharge was not exceeded, were also damaged (see photographs).

COMMITTEE ACTIVITIES

Herman McGill, Hydraulic Engineer, Temple, Texas began the hydrologic study of the site on November 5, 1984 and sent the completed study to the Arizona State Office in January, 1985. A copy of Mr. McGill's report is attached (see Appendix A). Harry C. Millsaps , Hydraulic Engineer, Phoenix, Arizona, analyzed Mr. McGill's study for each side inlet in January, 1985. A copy of Mr. Millsaps analysis is attached (see Appendix A). William Payne, Design Engineer, Phoenix, Arizona, interviewed the SCS Government Representative for construction and reviewed the design and construction records, and prepared the original Engineering Report, dated December 1985.

The present committee was assigned in November 1986. This committee reviewed the work done by McGill and Payne, reviewed the design and construction records, and conducted additional interviews.

INTERVIEWS

Interviews were conducted with Tom Jayo, SCS Government Representative for construction; John Sullivan, SCS Construction Engineer; Ralph Arrington, State Conservation Engineer; Paul Monville, Government Representative for the design contract and the State Design Engineer at the time of design (telephone conversation); and Ash C. Patel, Designer, PRC Toups.

EVALUATION

DESIGN

Key elements of the design included the hydraulic sizing of the side inlet structures and the collector channels, sediment in the collector channels, and location of the side inlets.

Experience in Similar Design

Arizona SCS has experience with the design and construction of side inlets into a diversion channel on four previous projects. These are discussed below:

1. RWCD Reach 2 - there was some erosion at the side inlets on this project but the erosion resulted from soil piping, which did not occur in the coarse soils at Saddleback Diversion.

2. RWCD Reach 3 - this project has not been tested.

3. Signal butte Floodway - the inlets here have performed well except for ponding of water outside the right-of-way.

4. Spookhill Floodway - this project has experienced erosion damage very similar to that which occurred to the side inlets and collector channels at Saddleback Diversion. These damages were repaired by the project sponsor and an engineering report was not prepared. This committee did not conduct an in-depth comparison of the two projects.

Side Inlets

Details and design discharges for the grouted rock side inlets are shown on the As-Built Drawing sheet 20 in Appendix B.

The Design Report for this project (see pages 3.1-50 and 3.1-52 in Appendix B) shows that the design width of the inlets was determined by applying Mannings Formula to the natural wash upstream from the inlet. This method results in supercritical flow over the weir crest.

These inlets were constructed with no slope through the weir. The committee used the broad crested weir method (see Appendix B) to compute as-built capacities. This method shows that the actual capacity of all 17 weirs is less than 50-year discharge capacity. For example, the capacity of inlet No. 7 is only 47% of that needed for the 50-year discharge (see Table 1). The weirs were designed and constructed without freeboard.

Collector Channels

The collector channels, called "lateral swales" in the drawings and design file, are earthen channels constructed to divert overland flow to the

grouted rock side inlets. The channels had a bottom width of either 10, 20, or 30 ft. with a 2.0 ft. depth (see As-Built Drawing, sheet 20, in Appendix B).

The intent of the designers (see excerpt from Design Report, page 3.1-50 in Appendix B) was to size the channel capacity at 1/3 of the capacity of a corresponding side inlet.

The bottom slope of the collector channels is not shown on the drawings. The Committee determined the slope from the slope stake survey field books and computed the As-Built capacity of the collector channel (see Computations in Appendix B). A comparison of design and As-Built capacity is shown in Table 2. In summary, the As-Built capacity either equals or exceeds design capacity (1/3 Q of side inlets) except for inlets No. 1, 7, 9, and 17. The collector channels did not have any allowance for freeboard or sediment accumulations.

Sediment

Sediment bars across the collector channels caused overtopping and breaching of the collector dikes. Extensive attention was given in design to the subject of sedimentation for the main diversion channel but not for the collector channels. No allowances were made in the design for sediment. Hydraulic sizing assumed that the collector channels would carry the run-off without the need to increase capacity for sediment or added freeboard.

CONSTRUCTION

Construction records were reviewed. Construction was in accordance with plans and specifications. Construction operations did not have any bearing on the problems encountered.

CONCLUSIONS

The primary cause of the damage to the side inlets and collector channel was an intense storm that exceeded design discharge for most of the area where problems were encountered. The 50-year design capacity was exceeded at 13 of the 17 inlets (see Table 1). This storm caused flow braiding on the alluvial fan above the structures, erosion, and significant deposition of sediment in the collector channels. The primary responsibility is assigned to natural occurrences beyond the reasonable control of anyone involved with this project.

However, there were some damages to the inlets and collectors channels in the upper reaches of the diversion where the storm peaks were actually less than the design discharges. This occurred at Inlet No's. 2, 3, 5 & 8 (see Table 1). At Inlet No. 4, the storm peak was greater than the design discharge but

this was not due to the intensity of the storm, but to an increase in the drainage area assumed to be contributing to this inlet. The design drainage area was 0.14 square miles as compared to 0.29 square miles used for the storm. This apparent discrepency has not been reconciled as of this writing (see Millsap's report in Appendix A).

The as-built capacity of all 17 side inlets is less than the planned 50-year capacity because the weir width was determined by Mannings Formula instead of the weir formula (see Appendix B). The actual capacity versus 50-year discharge varies from 47% at Inlet No. 7 to 95% at Inlet No. 14. At Inlet No's. 2 and 3, the storm discharge exceeded the as-built capacity, but was approximately equal to or less than the 50-year design discharge (see Table 1). These two inlets might not have been damaged had they been designed correctly. Responsibility is with design, although it should be recognized that failure of most of the inlets would still have occurred even if they had been sized correctly, because of breaching of the collector channels adjacent to the side inlets.

The collector channels had adequate hydraulic capacity at the time of construction (see Table 2), but no allowance was made for sediment accumulation. Further, no freeboard was provided for either the collector channels or the side inlets. There does not appear to be any definite Arizona SCS policy regarding freeboard, or for accounting for sediment in the design of secondary type systems such as these side inlets and collector channels above a main diverison. The primary design (and review) attention is on the main diversion structure. Accounting for freeboard and sediment is at the option of the designers. Thus, the designers of Saddleback Diversion followed established procedures.

RECOMMENDATIONS

Undersizing of the side inlets during design should have been detected during the several reviews of the design calculations. More time and attention should be given to a detailed review to avoid a recurrence of this particular short-coming.

SCS policy/criteria that needs to be re-examined includes:

1. Freeboard requirements for side inlets.
2. Freeboard requirements for collector channels.
3. Sediment accumulation in collector channels.
4. Overdesigning collector channels to account for shifting, braided flow on alluival fans.

Recommended remedial treatments for Saddleback Diversion are:

1. All of the 17 side inlets are undersized. Decide if it is prudent to enlarge the structures to handle the 50-year discharge.

2. Repair collector channels. Construct additional channels and dikes

the entire length of the diversion. Enlarge the channels to accomodate some sediment.

3. Add side inlets in locations where new major washes now exist.

4. Repair existing side inlets.

SUMMARY

1. Saddleback Diversion was completed in 1982. On September 2, 1984, an intense storm produced peak discharges that exceeded design for the middle and lower reaches of the diversion. Peak discharge at the outlet was double design discharge.

2. The diversion performed almost perfectly, providing greater than design protection for downstream properties. Water levels in the diversion came within one foot of overtopping the diversion dike; without damage to the main diversion channel, diversion dike, or four large grouted rock grade control structures.

3. The intense storm changed the location of several side washes and caused flow braiding and erosion on the alluvial fan of the contributing drainage area.

4. Runoff water by overland sheet flow and washes deposited sediment bars in the collector channels. Almost all of the collector channels were overtopped and breached.

5. Fifteen of 17 grouted rock side inlets were overtopped and/or flanked by floodwaters. The structures are still structurally intact except for erosion around the abutments where the collector channels meet the side inlets.

6. Construction was in accordance with plans and specifications and had no bearing on the problems encountered.

7. The primary cause of the damage to the side inlets and collector channels was an intense storm that exceeded design for most of the problem areas. Responsibility is assigned to natural occurrences.

8. The as-built capacity of all 17 side inlets is less than 50-year capacity because the designer(s) did not use established design methods to size the structures. This undersizing might have contributed to damage at Inlet No.'s 2 & 3 where the storm exceeded as-built but not design capacities. Responsibility is with design. This problem could have been prevented by a detailed review of design computations.

9. The collector channels and side inlets were designed and constructed without freeboard. The addition of freeboard would probably not have

prevented failure of the collector channels because of the sediment bars across the channels. The designers followed established procedures. Arizona SCS policy regarding freeboard for these type structures should be re-examined.

10. No allowances, other than sediment removal during normal maintenance, were made for sediment accumulations in the collector channels despite the fact that floodwater had to turn 90° when they met the channels and one channel was 2,510 feet long.

11. The exceptional success of the main diversion should not be obscured or discounted by the problems encountered with the secondary side inlet and collector channel system. The design engineers for this project faced a very close benefit-cost ratio of 1.16:1. The watershed work plan did not have any provisions to handle side inflow other than sloping the upstream side of the main diversion channel on a 5:1 slope. The structural grouted rock side inlets and collector channels were added during design as a better, but costlier, method to handle side inflow. The designers took some risk in the design of the collector channels and side inlets (no freeboard and no allowance for sediment) since the main line of defence, the main diversion channel, was immediately downstream. The collector channels and side inlet were considered to be secondary, or minor, structures in the total project.

TABLE 1

ENGINEERING REPORT
SADDLEBACK DIVERSION
Comparison of Design, AS-Built, and Storm Discharges
for
Side Inlets

Inlet No.	50-Year Q (cfs)	Design Q (cfs)	As-Built Q (cfs)	Percent As-Built/ 50-year	Storm of 12-2-84 (cfs)	Was Inlet Damaged
1	773	780	578	75%	1/	No
2	325	360	245	69%	325	Yes
3	172	180	140	81%	184	Yes
4	182	180	140	77%	353	Yes
5	208	210	158	76%	123	Yes
6	177	180	140	79%	26	Minor
7	956	1020	447	47%	1024	Yes
8	150	205	140	82%	210	Yes
9	535	545	447	84%	860	Yes
10	535	565	360	67%	860	No
11	176	185	158	90%	236	Yes
12	797	880	447	56%	1655	Yes
13	449	470	272	61%	1670	Yes
14	147	180	140	95%	279	Yes
15	212	225	184	87%	448	Yes
16	790	800	491	62%	1774	Yes
17	790	805	447	57%	1637	Yes

1/ Not Determined

TABLE 2

ENGINEERING REPORT

SADDLEBACK DIVERSION

Comparison of Design and As-Built Discharge
for
Collector Channels

Inlet No.	Intended/ Design Q (cfs)	As-Built Q (cfs)	Was Channel Overtopped?
1	250	137	No
2	110	206	Yes
3	60	142	Yes
4	60	128	Yes
5	70	North = 142	
		South = 75	Yes
6	60	231	No
7	320	227	Yes
8	50	North = 139	Yes
		South = 160	Yes
9	180	167	Yes
10	180	North = 152	?
		South = 171	?
11	60	90	Yes
12	270	304	Yes
13	150	205	Yes
14	60	N/A	N/A
15	70	155	Yes
16	260	265	Yes
17	260	216	

1/ Equals 1/3 of design discharge for side inlets.

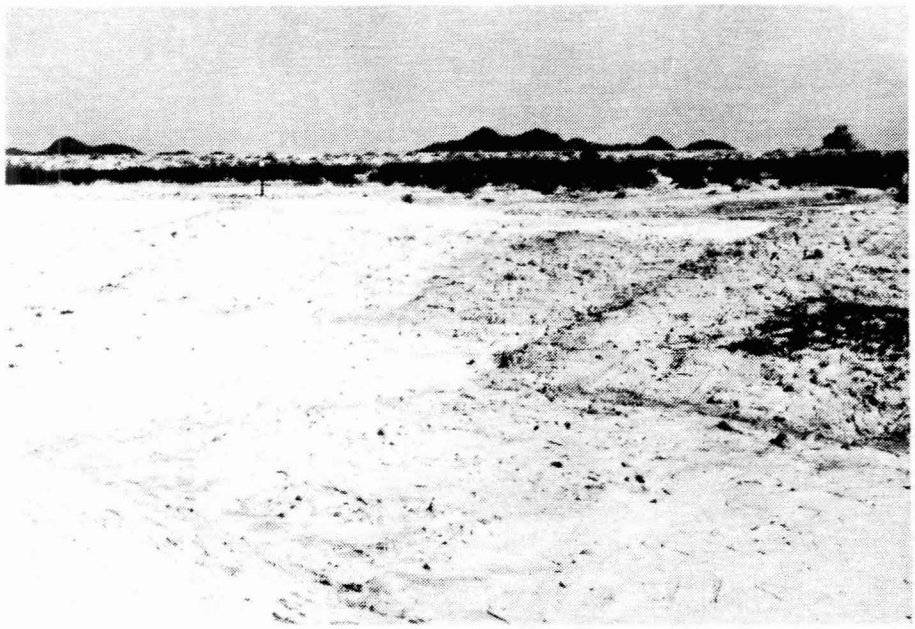


Photo No. 1

STATION 7+00 SIDE INLET #1
NO DAMAGE

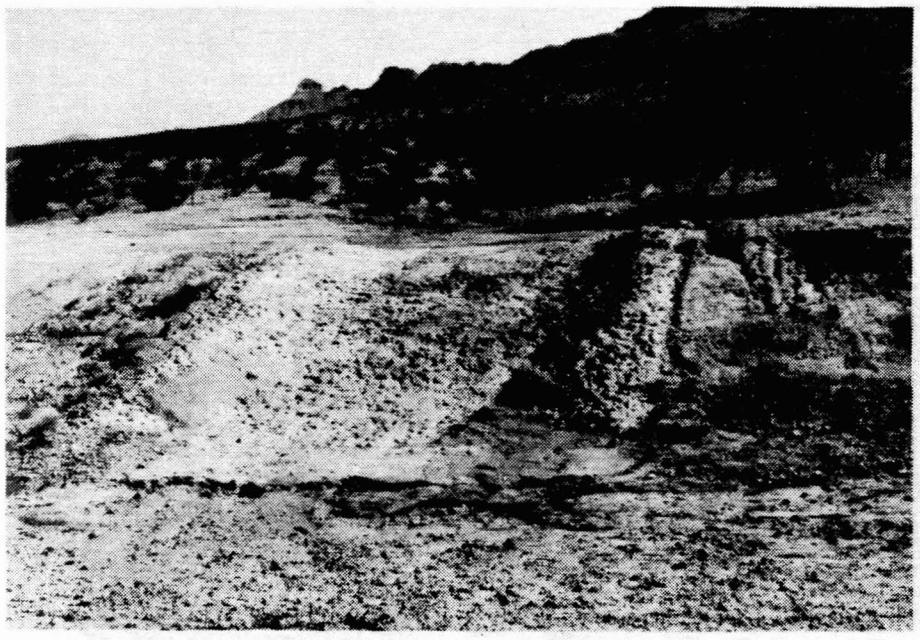


Photo No. 2

STATION 14+00 SIDE INLET #2
SEDIMENT BARS IN COLLECTOR CHANNELS
CAUSED OVERTOPPING OF DIKE

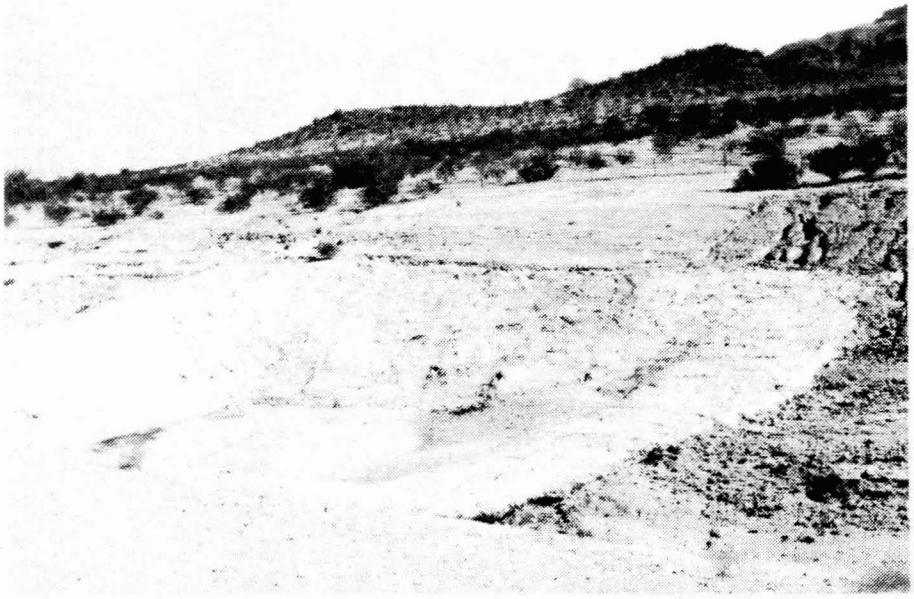


Photo No. 3

STATION 19+69 DROP #1
SOME OVERBANK EROSION

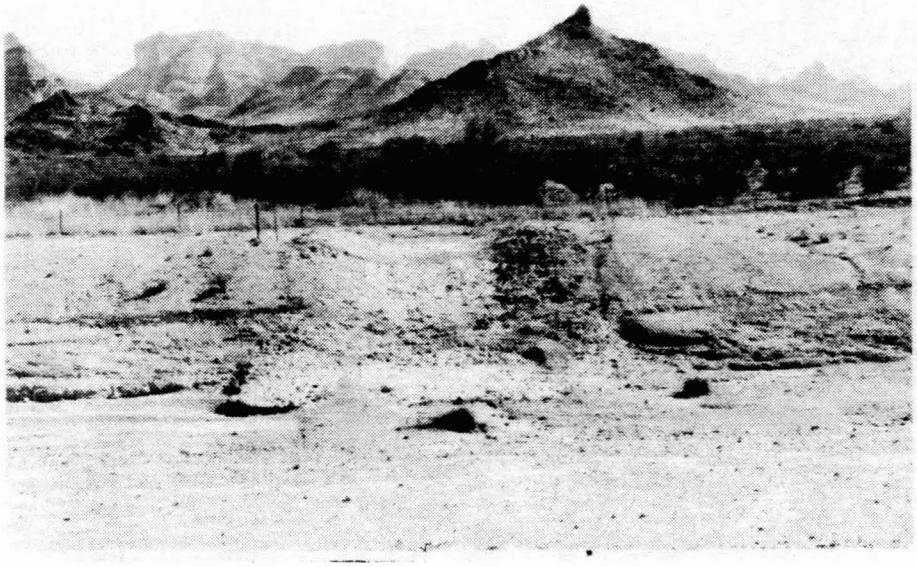


Photo No. 4

STATION 29+00 SIDE INLET #3
SOME RILL EROSION DUE TO OVERTOPPING

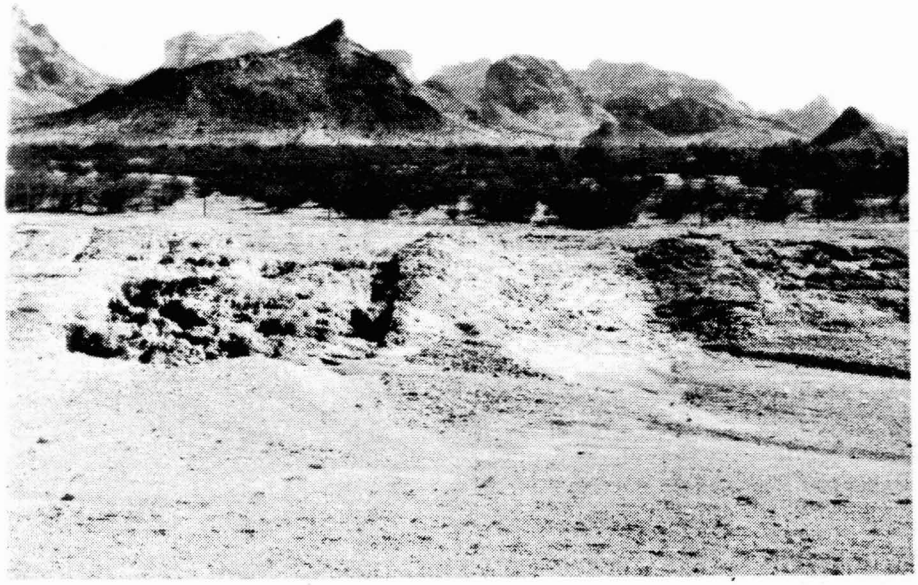


Photo No. 5

STATION 41+61.6 SIDE INLET #4
EROSION FROM OVERTOPPING

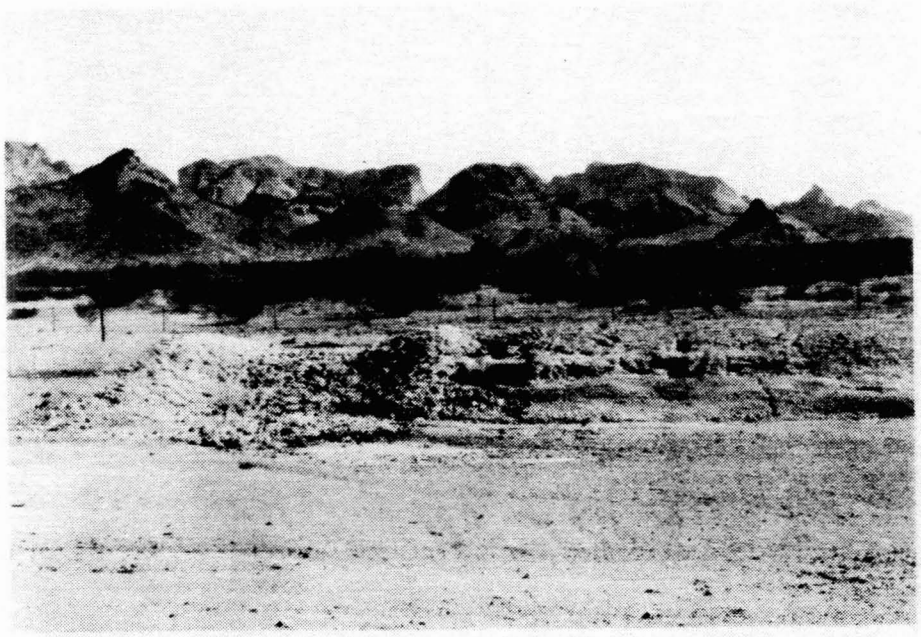


Photo No. 6

STATION 52+00 SIDE INLET #5
OVERTOPPED COLLECTOR DIKE
FOR APPROXIMATE 200 FEET.

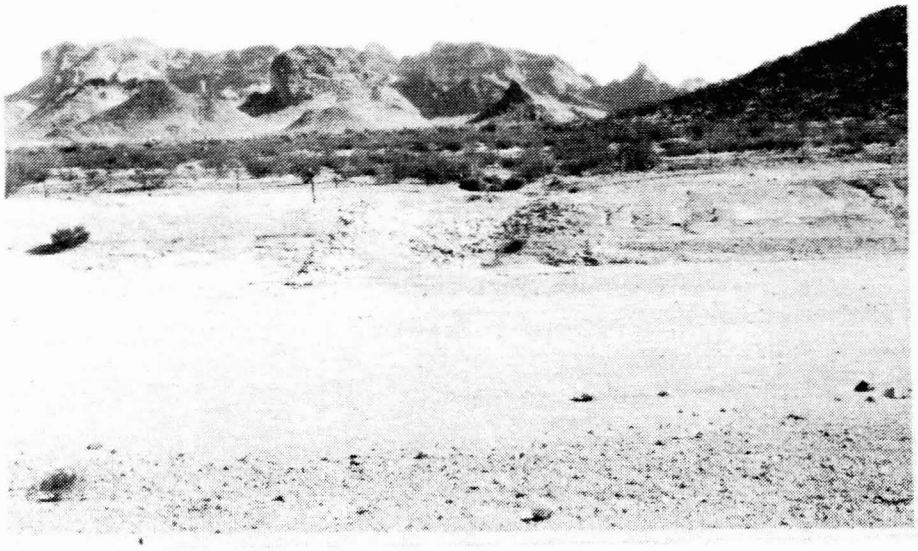


Photo No. 7

STATION 61+25 SIDE INLET #6
NO DAMAGE

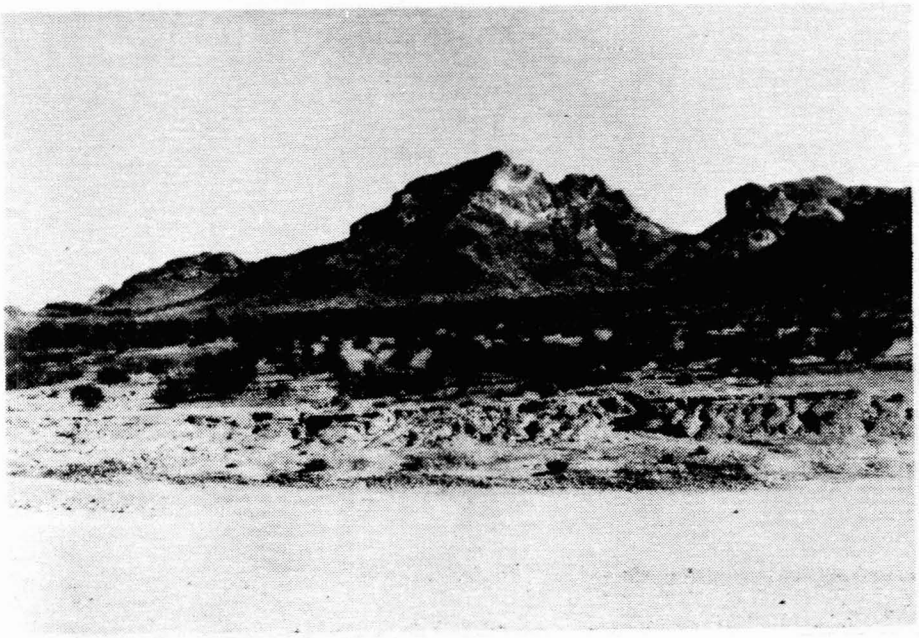


Photo No. 8

APPROXIMATE STATION 63+75
COLLECTOR CHANNEL AND DIKE OVERTOPPED
SEVERE EROSION

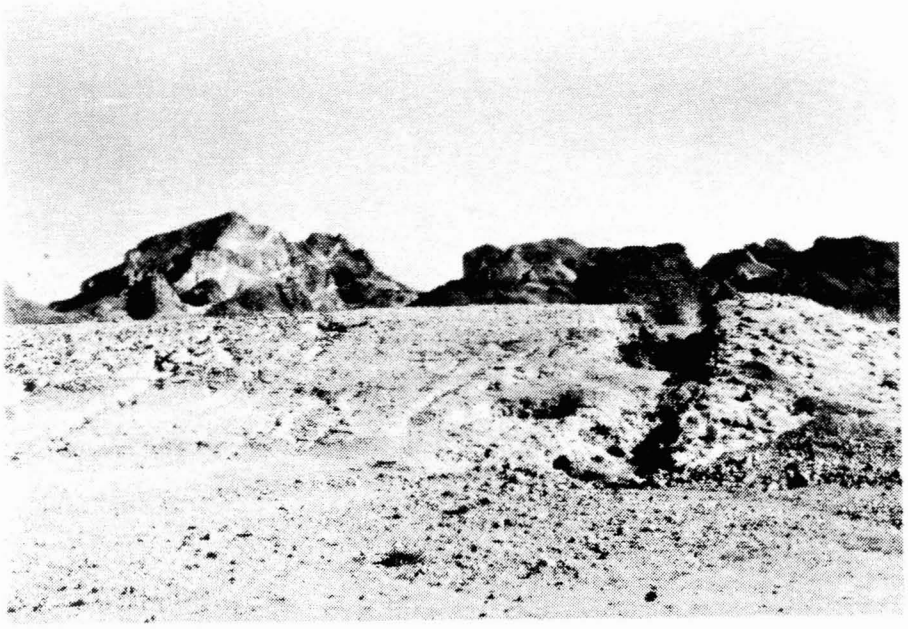


Photo No. 9

STATION 80+12 INLET #7
SEVERE EROSION ON NORTH SIDE FROM
OVERTOPPING

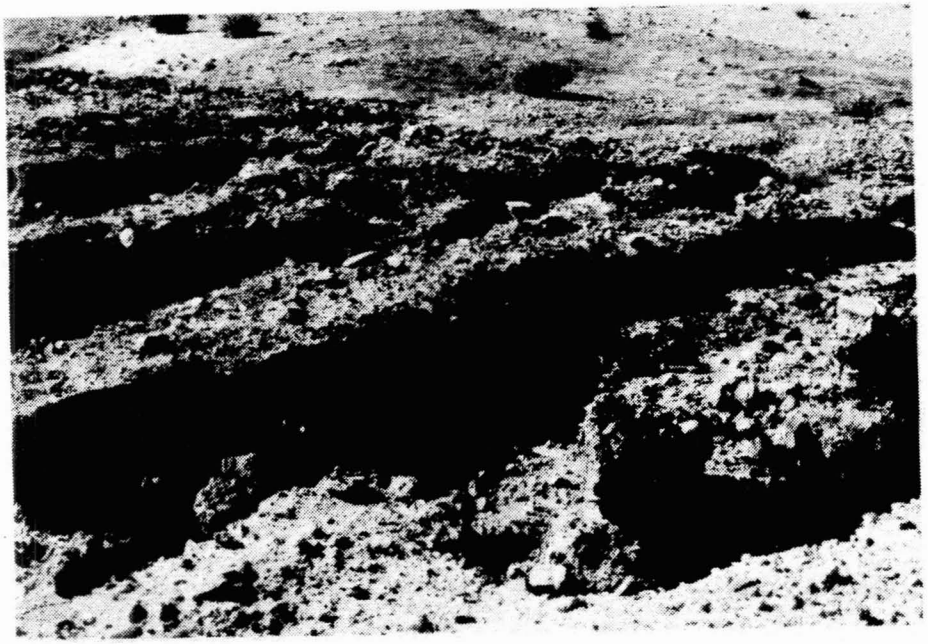


Photo No. 10

STATION 80+12 SIDE INLET #7 LOOKING
WEST. SEVERE EROSION ON SOUTH SIDE
FROM OVERTOPPING

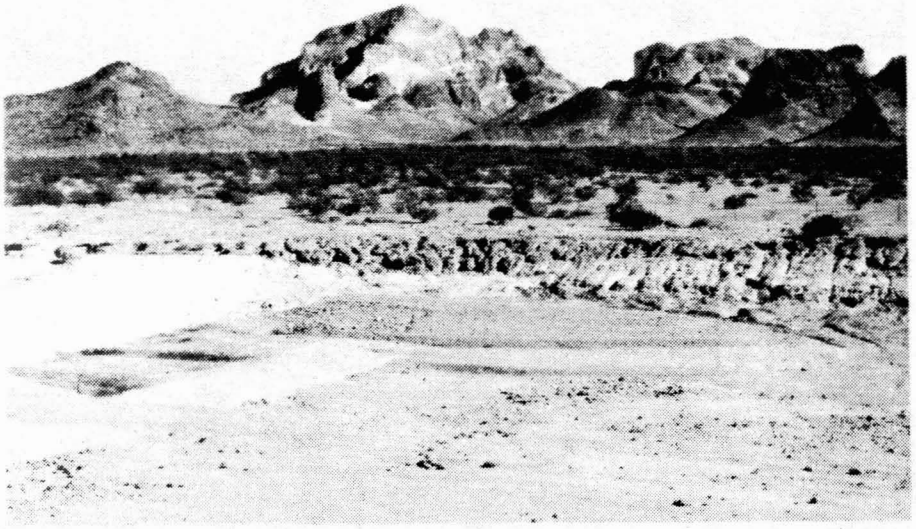


Photo No. 11

STATION 81+83 DROP #2
EROSION FROM OVERTOPPING

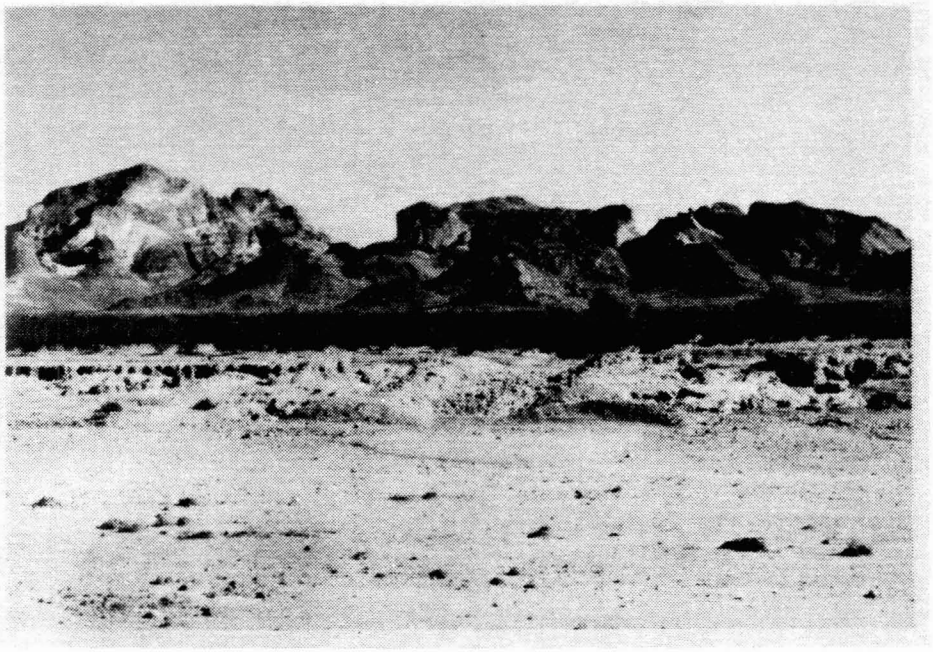


Photo No. 12

STATION 90+08.5 SIDE INLET #8
EROSION FROM OVERTOPPING
ON BOTH SIDES

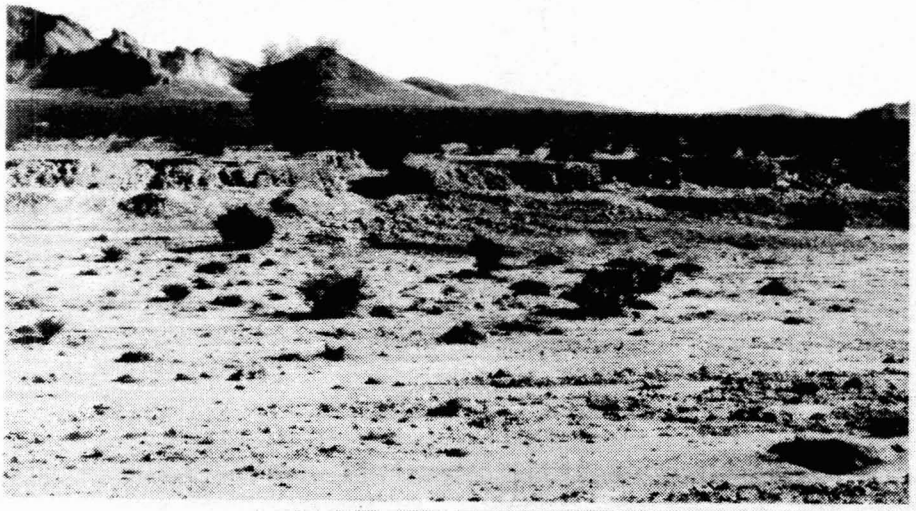


Photo No. 13

APPROXIMATE STATION 99+00
A NEW GULLY 5 FEET DEEP AND 10 FEET
WIDE AND SEVERE EROSION FROM
OVERTOPPING

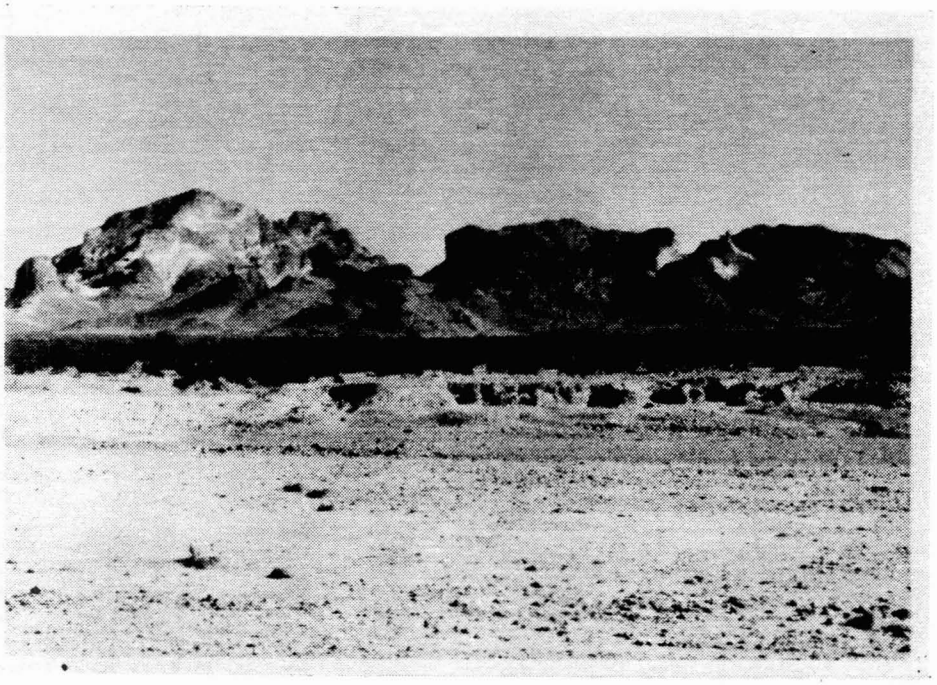


Photo No. 14

STATION 108+14.5 SIDE INLET #9
OVERTOPPING OF COLLECTOR DIKES

Photo No. 15

STATION 108+14.5 INLET #9
EROSION ON NORTH SIDE

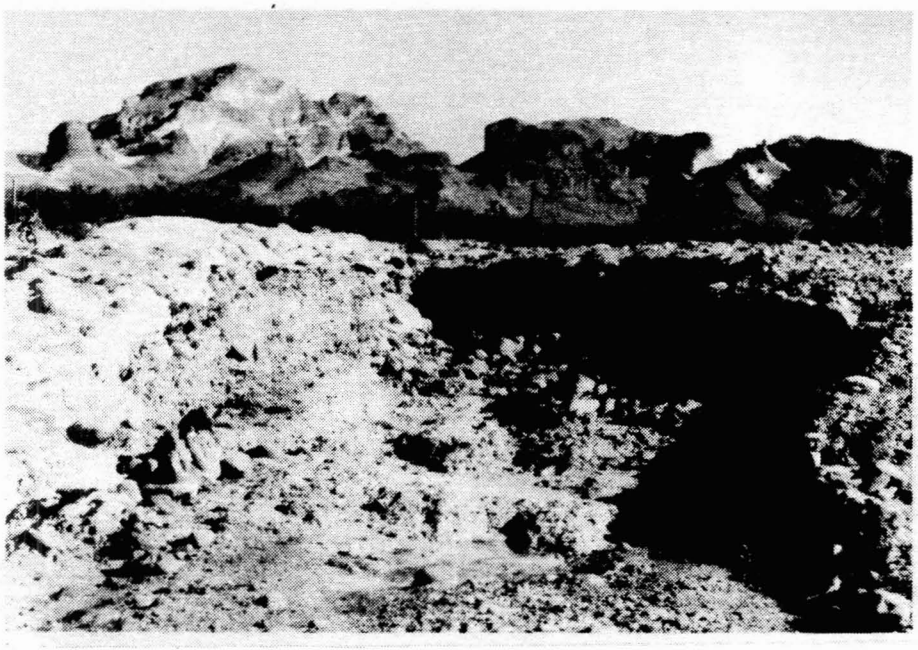


Photo No. 16

STATION 108+14.5 INLET #9
EROSION ON SOUTH SIDE

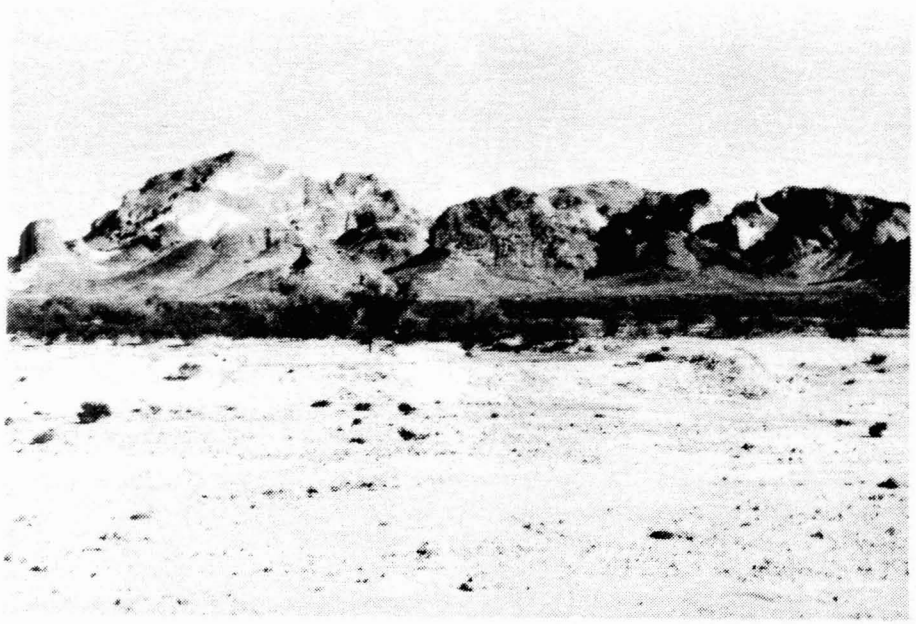


Photo No. 17

STATION 121+26 SIDE INLET #10
NO DAMAGE

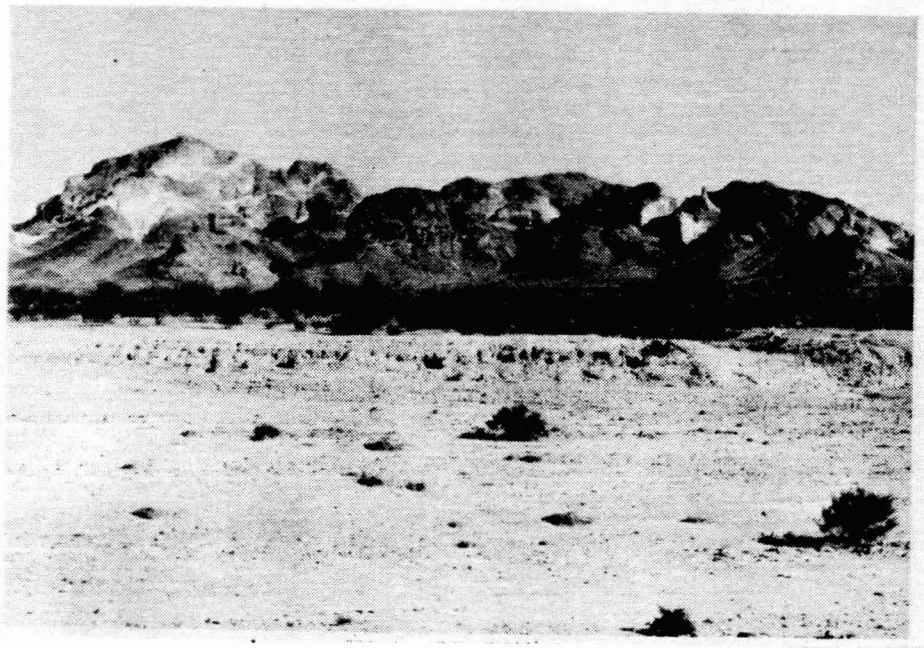


Photo No. 18

STATION 128+32 SIDE INLET #11
EROSION FROM OVERTOPPING

Photo No. 19

APPROXIMATE STATION 133+00
SEVERE EROSION FROM GULLYING

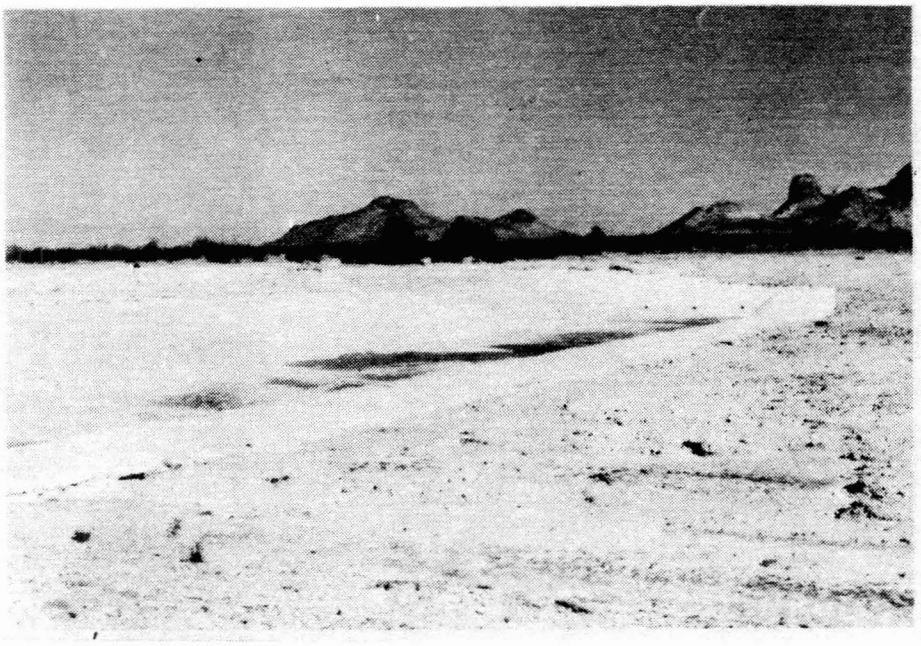


Photo No. 20

STATION 144+82 DROP #3
MINOR RILL EROSION

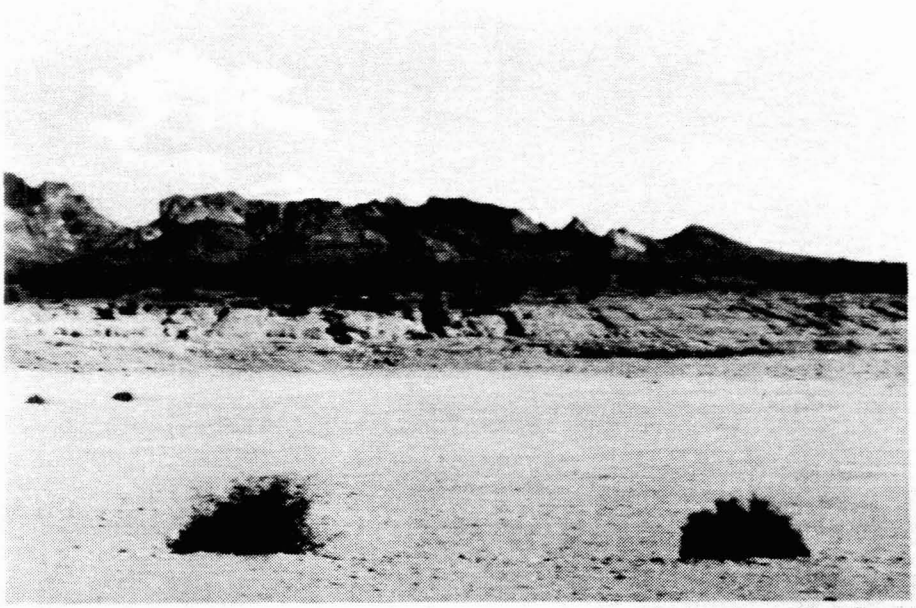


Photo No. 21

STATION 157+96 SIDE INLET #12
EROSION FROM OVERTOPPING
ON BOTH SIDES

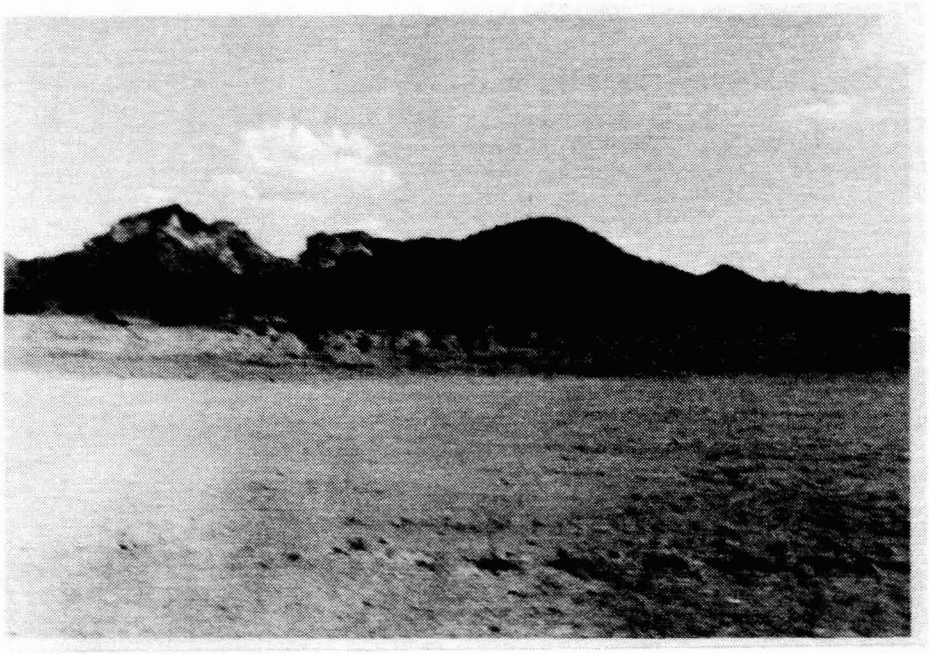


Photo No. 22

STATION 170+05 SIDE INLET #13
EROSION FROM OVERTOPPING
ON BOTH SIDES

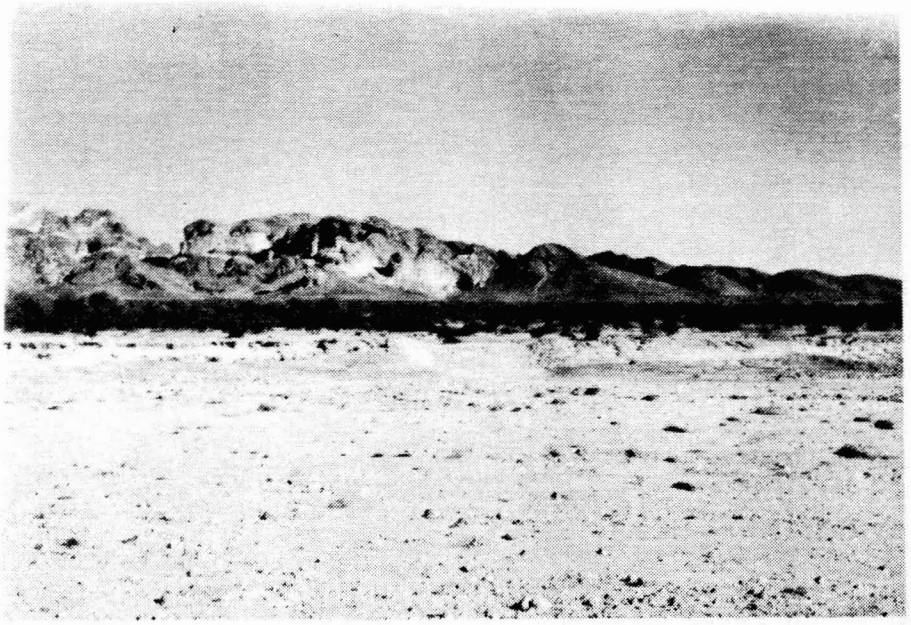


Photo No. 23

STATION 173+98 SIDE INLET #14
EROSION FROM OVERTOPPING
ON BOTH SIDES

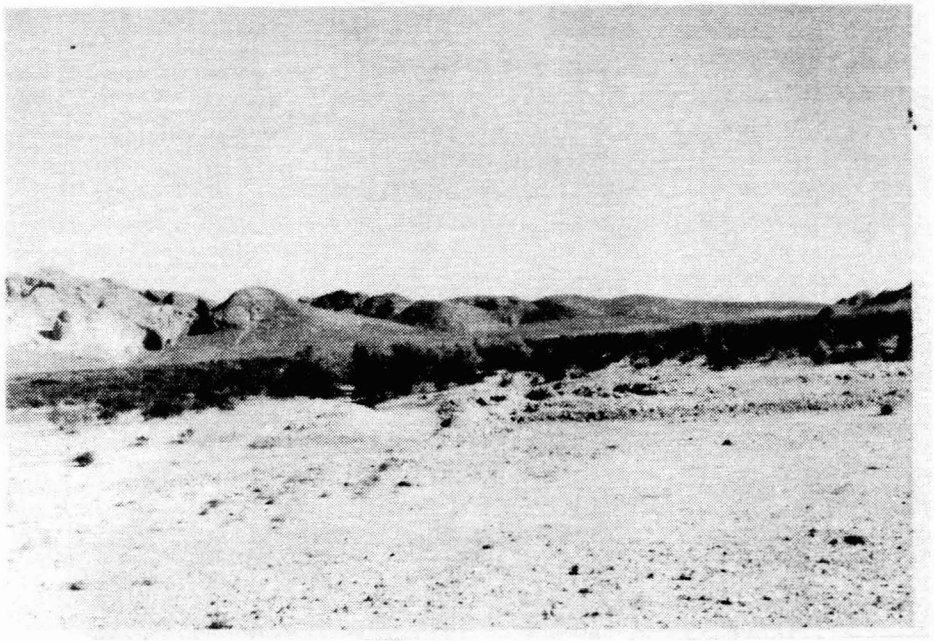


Photo No. 24

STATION 183+65 SIDE INLET #15
EROSION FROM OVERTOPPING ON
BOTH SIDES

Photo No. 25

APPROXIMATE STATION 200+00
SEVERE EROSION FROM GULLYING

Photo No. 26

STATION 202+00 SIDE INLET #16
EROSION ON SOUTH SIDE FROM
OVERTOPPING

Photo No. 27

STATION 220+80 SIDE INLET #17
EROSION FROM OVERTOPPING ON
BOTH SIDES

APPENDIX A

HARQUAHALA VALLEY WATERSHED
Saddleback Diversion
Study of September 2, 1984 Storm

Surveys

Surveys included carrying a level line along the upstream side of the channel. The elevation of high water marks in the diversion and side inlet channels were recorded. The location of side entrance problems were documented. Survey results are summarized on Table 1.

Rainfall

Figure 1 shows the rainfall amounts reported at locations near the watershed. The red values were obtained by work unit staff, the green during the study. Reported amounts range from 2 to 11 inches. There are no residences in the drainage area of Saddleback diversion, thus no rainfall information was available there. It appears, however, that the most intense part of the storm traveled from NW to SE and traversed the central and lower portion of the diversion watershed. High water marks at FRS 1 showed that the maximum stage was 4.8 feet over the principal spillway crest. The total capacity at this stage is 1732 acre-feet or .32 inches runoff from the watershed. The sediment pool capacity is 424 acre-feet or .08 inches. Because of the short duration of the storm, and small principal spillway capacity of FRS 1, rainfall above the site was not used in the Saddleback Diversion Study.

The storm distribution was taken from the recorder chart (Figure 2). The tabulated values were used because it was not possible to accurately read shorter time increments from the chart. It is probable that more intensive rainfall occurred during shorter time increments.

Runoff

Soil cover complex runoff curve number 90 was used to determine the runoff volume. This approximates the work plan value that was used for determining the 50-year design storm for the diversion. This appears high but runoff volume is not important for this study. The comparison of peak discharges and flow stage elevations determined by high water marks to the design values was the objective of the study.

Storm Evaluation Studies

Water surface profiles were prepared by Toups Corporation engineers using the Corps of Engineers HEC II program. The 10-yr and 50-yr frequency storms and the floodwater retarding structure principal spillway outflow discharge and elevation are recorded in Volume II of their design report. This did not provide enough information to make cross section hydraulic ratings. The planned cross section dimensions were input for the SCS WSP2 program. The planned dimensions were used

instead of the "as built" ones because differences were not significant and they were more readily available. The design engineer used .030 "n value" for the central portion of the channel and .035 for the bank sections. Because most of the flow is carried in the central segment and the bank segments flow differences between the use of .030 and .035 are small, one segment sections using .030 were entered in the WSP2 program.

The output from the WSP2 program was used in a TR-20 program to model the watershed. The convex routing method was used in the original study and in this study. The point that incremental drainage areas enter the diversion was altered based on a recent aerial photograph of the watershed and the location where it was noted that excessive inflow occurred. Selected time of concentrations are consistent with those used in the watershed plan. Table 2 is a schematic of the watershed showing incremental drainage areas and Tc values used in this study.

Five storms were routed through the watershed beginning with FRS 2. The Type II, 24-hour, 50-year storm (3.6 inches) was routed to evaluate the model. The resulting discharges and elevations compared favorably with those used in the plan except in the upper reaches of the diversion. It was later discovered that in this study it was assumed that an .84 square mile drainage area flowed into the FRS. The original plan entered this flow near the upstream end of the diversion. Because the discharge from the FRS is the principal contributor to the peak discharge in the upper reaches of the diversion, changes were not made.

Four additional storms using the 1984 storm time distribution were routed through the watershed. Aerial distributions were varied to obtain the best flow - high water mark relationship. Storm No. 5 appears to provide the best relationship, though the high water marks are generally higher than the generated elevations in the central and lower reaches of the watershed. This could be partly due to the reduced channel capacity caused by silt bars. Table 3 is a summary of these results. This shows that rainfall above the floodwater retarding structures probably averaged less than 3 inches and ranged from 5 to 7 inches below the FRS.

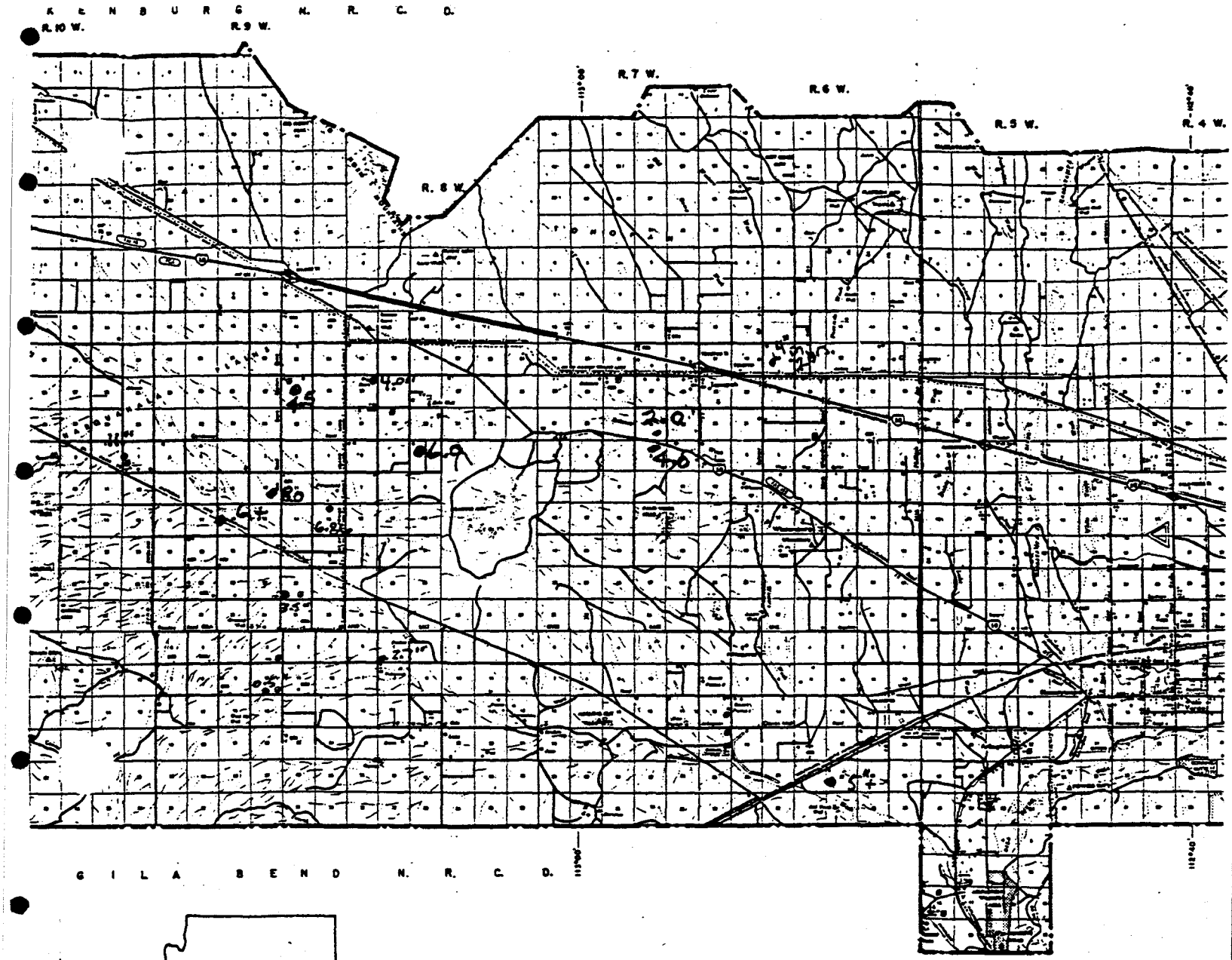
The incremental drainage area peak discharges for each of the five storms and the capacity of each side inlet structure are shown on Table 4. It is noted that Storm 1 discharges compare favorably to the capacity of the side inlet structures. This is expected in as much as the same storm and Tc were used. However, the location of side inlet inflow differs somewhat. A study of the drainage area flow patterns shows that the location of inlet flow is subjective.

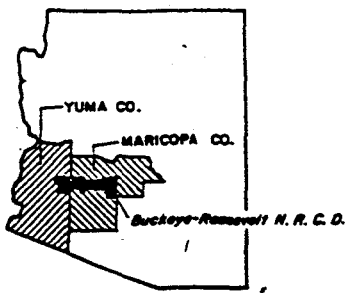
Generally, channels were used to divert flow to the inlet structure location. Flow in these were partially blocked by sediment bars resulting in over-bank flow before the inlet structure capacity was reached. In some locations these breached the bank between the training channel and diversion. This occurred in the upper reaches of the diversion where the design flow of the structure was not exceeded as well as in the central and lower reaches.

Conclusions

The study shows that the September 2, 1984 storm exceeded the 50-year frequency design storm in the central and lower reaches of the diversion. Flows, based on high water marks, were estimated to be about 2 times design flows near the diversion outlet. This did not appear to have caused significant damage to the diversion channel.

The location of inflow to the diversion is subjective, but a recent aerial photograph and onsite studies show that side inlet structures were not always located where needed. Also the limited capacity of the below natural ground channels used to divert flow to the side inlet structures were reduced by sediment bars resulting in over-bank inflow to the diversion. This caused damage to the upstream bank of the diversion and in some locations, left sediment bars in the diversion channel.





HARQUAHALA VALLEY
RAINFALL AMOUNTS
LABOR DAY STORM 1984
4hr. duration.

BUCKEYE-ROOSEVELT
NATURAL RESOURCE CONSERVATION DISTRICT
MARICOPA AND YUMA COUNTIES, ARIZONA

OCTOBER 1976

1 0 1 2 3 4 5 MILES
SCALE 1:280,000

TABLE 4

HARQUAHALA VALLEY WATERSHED
Saddleback Diversion

Peak Discharge from Incremental Drainage Areas

STATION	DESIGN Drainage Area Mi²	DESIGN Discharge cfs	TYPE II STORM DIST. Drainage Area Mi²	TYPE II STORM DIST. Storm 1 cfs	1984 STORM DISTRIBUTIONS Storm 2 cfs	1984 STORM DISTRIBUTIONS Storm 3 cfs	1984 STORM DISTRIBUTIONS Storm 4 cfs	1984 STORM DISTRIBUTIONS Storm 5 cfs	SIDE INLET Station	SIDE INLET Capacity cfs
1000	.840	773							700	780
1090			.27	362	250	250	250	325		
1500			.02	44	20	20	20	26	1390	360
2200	.293	355								
3000			.15	271	142	142	142	184	2900	180
4500			.29	471	272	353	353	353	4161	180
5500			.10	193	95	123	123	123	5200	210
6500			.02	44	20	26	26	26	6125	180
6800	.554	739								
7500			.53	512	451	591	732	732		
8500			.53	512	451	591	732	732	8012	1020
8700	1.075	956								
9500			.14	253	132	171	210	210	9408	205
11000			.60	643	524	766	847	847	10814	545
12200	1.304	1220							12126	535
12500			.61	653	533	779	861	861		
13000			.16	206	147	214	236	236	12832	85
16000			1.06	880	856	1390	1655	1655	15796	880
17000			1.07	888	864	1404	1670	1670	17005	470
17500			.16	204	147	235	279	279	17398	180
18000	2.323	1740								
18500			.26	320	235	377	448	448	18365	255
20500			1.17	894	917	1490	1774	1774	20200	800
22500			1.08	825	846	1375	1637	1637	22080	805
23500			.10	181	94	150	178	178		
23800	2.623	1792								
23900			.43	528	389	626	743	743		
24400	.4323	437								

NOTE: Storm 1 is 24-hour 50-year Type II Storm - 3.6 inches
Storm 2 is 1984 storm distribution - 4.00 inches
Storm 3 is 1984 storm FRS to Sta 3000 - 4.0"; Sta 3000 to Sta 9500 - 5.0"; Sta 9500 to Sta 13000 - 5.5"; Sta 13000 to Sta 23900
Storm 4 is 1984 storm FRS to 3000 - 4.0"; Sta 3000 to Sta 6500 - 5.0"; Sta 6500 - Sta 13000 - 6.0"; Sta 13000 to Sta 23900 - 7"
Storm 5 is 1984 storm FRS - 3.0"; FRS to Sta 6500 - 5.0"; Sta 6500 - Sta 13000 - 6.0"; Sta 13000 to Sta 23900 - 7"

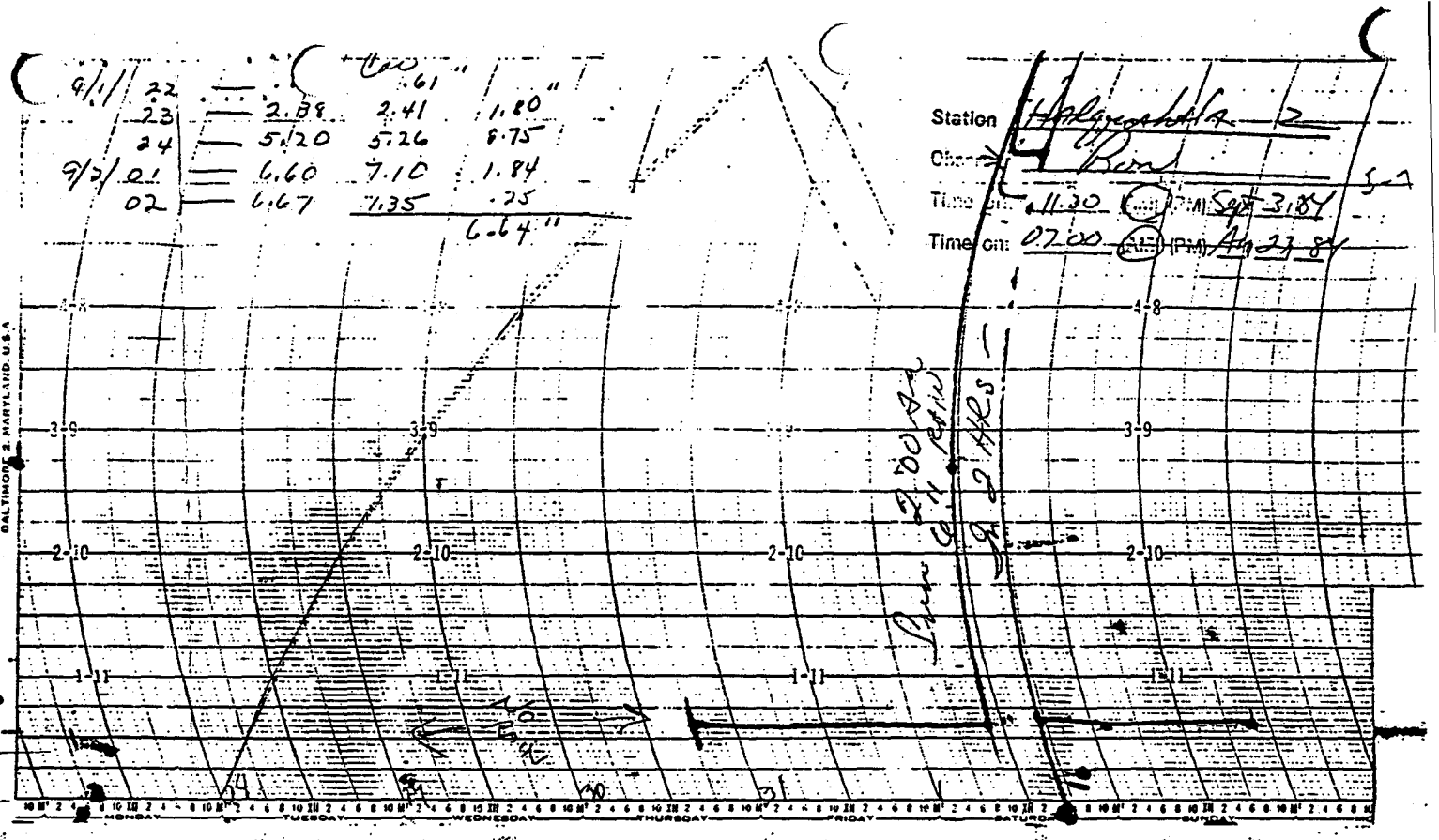

Station
Observer
Time on: 11:30 (AM) (PM) Sept 3 1984
Time on: 07:00 (AM) (PM) Aug 27 84
9/1 22 — 2.88 2.41 1.80"
23 — 5.20 5.26 8.75
24 — 6.60 7.10 1.84
9/2 01 — 6.67 7.35 .25
6.64"
.61"
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
12 INCH DUAL
UNIVERSAL RAIN GAGE
DELFORT INSTRUMENT COMPANY
BALTIMORE 2, MARYLAND, U.S.A.
PRINTED IN U.S.A.

Figure 2

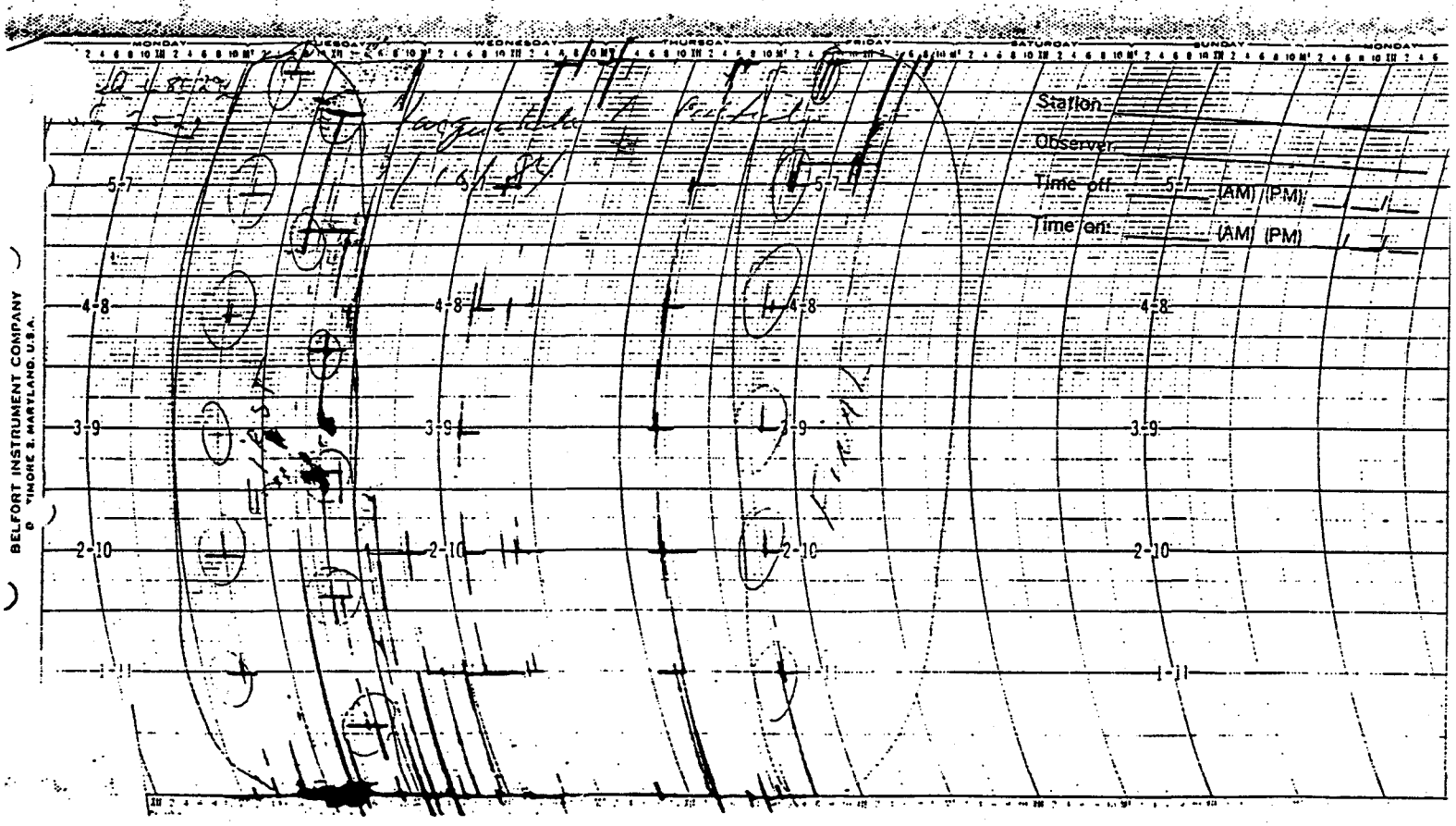

MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
MONDAY
Station
Observer
Time off: (AM) (PM)
Time on: (AM) (PM)
5-7
4-8
3-9
2-10
1-11
BELFORT INSTRUMENT COMPANY
BALTIMORE 3, MARYLAND, U.S.A.

TABLE 1
Harquahala Valley Watershed
Saddleback Diversion

Structure Damage Study - September 1984 Storm

Str 2	-	Sta 13+90	Structure did not overtop but overtopped training dike DS of structure.	
		Sta 15+00	HWM side entrance channel	1180.6
		Sta 15+00	HWM diversion	1175.8
Str 3	-	Sta 29+00	Structure Overtopped .4 foot	
		Sta 29+50	HWM side entrance channel	1171.1
		Sta 29+50	HWM diversion	1167.2
		Sta 31+00 to 35+00 overbank inflow to diversion		
Str 4	-	Sta 35+50	Begin side entrance channel	
		Sta 37+00	Flow over side entrance channel bank .4'	
		Sta 37+00	HWM diversion	1164.9
		Sta 38+50	Flow over entrance channel bank	
		Sta 41+00	HWM side entrance channel	1167.4
		Sta 41+00	HWM diversion	1164.3
		Sta 41+54	Side entrance channel bank breached	
		Sta 41+61	Str 4	
		Sta 42+50	HWM diversion	1163.8
		Sta 44+00	Side entrance channel bank overtopping	
		Sta 51+90	HWM side entrance	1164.2
			HWM diversion	1162.1
Str 5	-	Sta 52+00		
		Sta 52+50	Side entrance overtopped	
Str 6	-	Sta 61+25	Not overtopped	
		Sta 61+25	HWM diversion	1160.0
		Sta 63+50-66+00	Side entrance overtopped and breached in places.	
		Sta 68+50	Side entrance overtopped & breached	
		Sta 73+00	Side entrance channel begins overtopped in/out	
		Sta 77+00	HWM side entrance channel	1161.6
			HWM diversion	1157.1
		Sta 79+00	HWM diversion	1156.9

Str 7	-	Sta 80+12	Eroded around both abutments	
		Sta 86+00	Over bank flow to here narrow Terrace begins-breached at 86+80	
		Sta 88+00	HWM diversion	1151.1
		Sta 88+60	Terrace breach	
		Sta 89+00	Begin breach & severe bank erosion	
		Sta 90+00	Begin side inlet to Str 8	
		Sta 91+60	Begin side inlet embankment overtop	
		Sta 93+00	Severe bank erosion	
Str 8	-	Sta 94+08	Eroded around both abutments	
		Sta 101+00	Side entrance overtopped to here	
		Sta 103+00	Begin side entrance channel to Str 9 some flow around end	
		Sta 104+00	HWM diversion	1148.7
		Sta 105+00	Entrance overtopped severe bank erosion	
		Sta 107+00	Entrance overtopped-severe bank erosion	
Str 9		Sta 108+00	N. abutment overtopped	
		Sta 109+00	Sheet flow severe bank erosion	
		Sta 115+00	HWM diversion	1145.8
		Sta 117+00	Begin side inlet channel to Str 10	
		Sta 120+00	Side inlet channel overtopping begins	
			HWM diversion	1145.1
			HWM side inlet channel	1147.6
Str 10	-	Sta 121+00	Str 10	
		Sta 127+00	HWM diversion	1143.7
		Sta 127+00	Side inlet overtopped	
Str 11	-	Sta 128+30	Str overtopped	
		Sta 128+80	HWM side inlet	1146.3
		Sta 134+00	Side inlet channel overtopped	
		Sta 135+90	Side inlet channel breached	
		Sta 138+00	HWM diversion	1140.3
		Sta 139+40	Side inlet channel overtopped and breached-severe erosion	
		Sta 144+30	HWM diversion	1137.4
		Sta 147+00	HWM diversion	1132.5
			Side inlet channel overtopped and breached.	
		Sta 155+75	Side inlet channel overtopped and breached.	
			Side inlet channel blocked with gravel bar.	
Str 12	-	Sta 158+00	Breached around both abutments	
		Sta 161+00	Begin side inlet channel to Str 13	
		Sta 163+00	HWM diversion	1130.8
			Side inlet channel overtopping begins severe erosion.	

Str 13	-	Sta 170+00		
		Sta 171+20	Side inlet channel breached.	
Str 14	-	Sta 174+00	Washed around both abutments	
		Sta 175+00	HWM side inlet channel	1129.6
			HWM diversion	1127.7
		Sta 177+00	Severe erosion to here	
		Sta 179+00	HWM diversion	1126.9
		Sta 179+00	HWM side inlet channel	1128.7
		Sta 182+00	HWM side inlet channel	1127.3
			HWM diversion	1126.4
		Sta 182+70	Side inlet channel breached to Sta 183+50	
Str 15	-	Sta 184+00	Eroded both abutments	
		Sta 188+00	HWM diversion	1125.0
		Sta 190+00	Stream starting to cut into Div	
		Sta 195+00	Begin overtopping DS	
		Sta 196+00	HWM side inlet channel	1125.4
		Sta 196+30	Breach into diversion	
		Sta 197+50	End breach into diversion	
		Sta 198+00	HWM side inlet	1125.4
			HWM diversion	1123.7
		Sta 199+50	Begin breach into diversion	
		Sta 201+00	End breach into diversion	
			HWM side inlet	1123.3
			HWM diversion	1122.7
Str 16	-	Sta 202+00	The structure not referenced in notes and does not appear on photo. As built plans show it in place. It appears that the inlet may have been shaped and no structure built.	
		Sta 202+75	Cut into diversion 3' deep 30' wide	
		Sta 207+50	Begin breach into diversion	
		Sta 212+00	HWM side inlet	1121.7
			HWM diversion	1120.0
		Sta 219+00	HWM side inlet	1120.1
			HWM diversion	1118.6
Str 17	-	Sta 220+80	Eroded US abutment overtopped US & DS 25'	
		Sta 221+50	HWM side inlet	1120.0
			HWM diversion	1117.6
		Sta 230+00	HWM diversion	1115.5
		Sta 236+00	HWM diversion	1114.2
		Sta 237+00	HWM diversion	1113.7

TABLE 2
Harquahala Valley Watershed
Saddleback Diversion

1984 Storm TR-20 Schematic

	Station	Drainage Area		Tc
		Increment	Total	
FRS 2	0	29.56	29.56	3.26
Sec 1	1090	.27	29.83	.37
Sec 2	1500	.02	29.85	.09
Sec 3	3000	.15	30.00	.23
Sec 4	4500	.29	30.29	.28
Sec 5	5500	.10	30.39	.20
Sec 6	6500	.02	30.41	.08
Sec 7	7500	.53	30.94	.78
Sec 8	8500	.53	31.47	.78
Sec 9	9500	.14	31.61	.23
Sec 10	11000	.60	32.21	.67
Sec 11	12500	.61	32.82	.67
Sec 12	13000	.16	32.98	.41
Sec 13	14000	0	32.98	
Sec 14	15000	0	32.98	
Sec 15	16000	1.06	34.04	1.00
Sec 16	17000	1.07	35.11	1.00
Sec 17	17500	.16	35.27	.45
Sec 18	18500	.26	35.53	.52
Sec 19	19500	0	35.53	
Sec 20	20500	1.17	36.70	1.12
Sec 21	21500	0	36.70	
Sec 22	22500	1.08	37.78	1.12
Sec 23	23500	.10	37.79	.23
Sec 24	23900	.43	38.22	.53

TABLE 3
HANOHANALA VALLEY WATERSHED
Saddleback Diversion
Summary of TR20 Flood Routings

Station	TR20 Sec #	50-yr Storm HECII Disc. cfs	50-yr Storm HECII Elev. ft.	Storm No. 1 Disc. cfs	Storm No. 1 Elev. ft.	IMM 1984 Storm Elev. ft.	Storm No. 2 Disc. cfs	Storm No. 2 Elev. ft.	Storm No. 2 IMM Diff.	Storm No. 3 Disc. cfs	Storm No. 3 Elev. ft.	Storm No. 3 IMM Diff.	Storm No. 4 Disc. cfs	Storm No. 4 Elev. ft.	Storm No. 4 IMM Diff.	Storm No. 5 Disc. cfs	Storm No. 5 Elev. ft.	Storm No. 5 IMM Diff.
FWS 2		764	1189.4	755	1189.1	1182.9	802	1190.5		802	1190.5		802	1190.5		739	1188.6	
82		950	1179.1															
100					(1177.9)	1177.6		(1178.1)	-.5		(1178.1)	-.5		(1178.1)	-.5		(1177.8)	-.8
1090	1	1090	1177.1	762	1175.9		802	1176.1		802	1176.1		802	1176.1		739	1175.8	
1500	2	1160	1176.3	762	1175.0	1175.8	802	1175.1	.7	802	1175.1	.7	802	1175.1	.7	748	1174.9	.5
2950						1167.2		(1167.3)	-.1		(1167.3)	-.1		(1167.3	-.1			
3000	3	1370	1167.1	896	1167.6		835	1167.4		835	1167.4		835	1167.4		852	(1167.4)	-.1
3500		1370	1165.8															
3700					(1165.5)	1164.9		(1165.1)	-.2		(1165.1)	-.2		1165.1	-.2		1165.2	-.1
4000		1370	1164.7															
4100					(1164.5)	1164.3		(1164.1)	.2		(1164.2)	.1		(1164.2)	.1		(1164.3)	.2
4250					(1164.1)	1163.8		(1113.7)	.1		(1113.8)	0		(1113.8)	0		(1113.9)	.1
4500	4	1500	1163.5	1242	1163.3		1012	1162.9		1073	1163.0		1073	1163.0		1140	1163.1	
5000		1500	1162.3															
5200					(1161.6)	1162.1		(1161.2)	.9		(1161.4)	.7		(1161.4)	.7		(1161.5)	.8
5500	5	1500	1161.1	1311	1160.9		1069	1160.5		1172.6	1160.7		1172.6	1160.7		1244	1160.8	
6000		1500	1160.3															
6125					(1160.0)	1160.0		(1159.7)	.3		(1159.9)	.1		(1159.9)	.1		(1160.0)	.2
6500	6	1900	1159.2	1211	1158.5		1078	1158.2		1182	1158.4		1182	1158.4		1252	1158.5	
7000		1900	1158.1															
7500	7	1900	1157.2	1682	1156.5		1522	1156.4		1768	1156.6		1907	1156.7		1977	1156.8	
7700					(1156.0)	1157.1		(1155.8)	1.3		(1156.1)	1.0		(1156.2)	.9		(1156.3)	1.0
7900					(1154.4)	1156.9		(1154.2)	2.7		(1154.5)	2.4		(1154.7)	2.2		(1154.7)	2.2
8000		2350	1155.8															
8500	8	2350	1150.6	2137	1150.4		1962	1150.2		2350	1150.6		2628	1150.8		2698	1150.8	
8800					(1149.6)	1151.1		(1149.6)	1.5		(1150.0)	1.1		(1150.3)	.8		(1150.3)	.8
9000		2350	1149.6															
9500	9	2590	1148.6	2103	1148.2		2049	1148.2		2465	1148.5		2773	1148.8		2843	1148.8	
10000		2590	1147.7															
10400					(1146.8)	1148.7		(1146.8)	1.9		(1147.3)	1.4		(1147.6)	1.1		(1147.6)	1.1
10500		2860	1146.7															
11000	10	2860	1145.8	2583	1145.9		2591	1145.9		3100	1146.5		3572	1146.8		3640	1146.9	
11500		3250	1144.7		(1144.8)	1145.8		(1144.8)	1.0		(1145.4)	.4		(1145.7)	.1		(1145.7)	.1
12000		3250	1143.6		(1143.7)	1145.1		(1143.7)	1.4		(1144.4)	.7		(1144.7)	.4		(1144.7)	.4
12500	11	3250	1142.7	3044	1142.6		2996	1142.6		3902	1143.3		4376	1143.6		4440	1143.6	
12700					(1142.3)	1143.7		(1142.3)	1.4		(1143.0)	.7		(1143.3)	.4		(1143.3)	.4

Station	HEC-2 Sec #	50-yr Storm HECII Disc. cfs	50-yr Storm HECII Elev. ft.	Storm No. 1 Disc. cfs	Storm No. 1 Elev. ft.	IHM 1984 Storm Elev. ft.	Storm No. 2 Disc. cfs	Storm No. 2 Elev. ft.	Storm No. 2 IHM Diff.	Storm No. 3 Disc. cfs	Storm No. 3 Elev. ft.	Storm No. 3 IHM Diff.	Storm No. 4 Disc. cfs	Storm No. 4 Elev. ft.	Storm No. 4 IHM Diff.	Storm No. 5 Disc. cfs	Storm No. 5 Elev. ft.	Storm No. 5 IHM Diff.
13000	12	3500	1141.6	3105	1141.9		3107	(11419)		4070	1142.6		4565	1142.9		4627	1143.0	
13500		3500	1140.6															
13800					(1139.8)	1140.3		(1139.9)	.4		(1140.4)	-.1		(1140.7)	-.4		1140.7	-.4
14000	13	3740	1139.5	3051	1139.3		3101	1139.4		4059	1139.8		4552	1140.1		46.14	1140. 1	
14400		3740	1138.3															
14430					(1138.0)	1137.4		(1138.1)	-.7		(1138.5)	-1.1		(1138.8)	-1.4		(1138.8)	-1.4
14467		3740	1137.8															
14519		3740	1132.4															
14700					(1131.3)	1132.5		(1131.4)	1.1		(1131.9)	.6		(1132.1)	.4		(1132.1)	.4
15000	14	3970	1131.2	2994	1130.6		3094	1130.7		4049	1131.2		4543	1131.4		4605	1131.4	
15500		3970	1130.1															
16000	15	4320	1129.0	3792	1128.8		3916	1128.8		5404	1129.5		6170	1129.8		6228	1129.8	
16300					(1128.2)	1130.8		(1128.2)	2.6		(1129.0)	1.8		(1129.3)	1.5		(1129.3)	1.5
16500		4320	1127.9															
17000	16	4320	1126.9	4571	1126.9		4742	1126.9		6769	1127.8		7803	1128.1		7854	1128.1	
17500	17	4720	1125.8	4587	1125.7	1127.7	4847	1125.8	1.9	6943	1126.7	1.0	8005	1127.1		8060	1127.1	
17900					(1124.8)	1126.9		(1124.9)	2.0		(1125.8)	1.1		(1126.2)	.7		(1126.2)	.7
18000		4720	1124.8															
18200					(1124.1)	1126.4		(1124.3)	2.1		(1125.1)	1.3		(1125.5)	.9		(1125.5)	.9
18500	18	4720	1123.6	4601	1123.4		5019	1123.6		7229	1124.4		8350	1124.8		8404	1124.8	
18800					(1122.8)	1125.0		(1122.9)	2.1		(1123.8)	1.2		(1124.2)	.8		(1124.2)	.8
19000		5110	1122.5															
19500	19	5110	1121.4	4542	1121.2		5013	1121.3		7217	1122.2		8338	1122.5		8393	1122.5	
19800					(1120.7)	1123.7		(1120.8)	2.9		(1121.7)	2.0		(1122.1)	1.6		(1122.1)	1.6
20000		5110	1120.2															
20100					(1120.0)	1122.7		(1120.2)	2.5		(1121.1)	1.6		(1121.5)	1.2		(1121.5)	1.2
20500	20	5110	1119.2	5326	1119.3		5889	1119.5		8668	1120.4		10082	1120.9		10134	1120.9	
21000																		
21200					(1118.0)	1120.0		(1118.1)	1.9									
21500	21	5430	1117.0	5257	1117.2		5881	1117.5		8657	1118.6		10063	1119.1		10115	1119.1	
21900					(1116.3)	1118.6		(1116.5)	2.1		(1117.5)	1.1		(1118.0)	.6		(1118.0)	.6
22000		5430	1116.0															
22500	22	5750	1114.8	5939	1114.9		6678	1115.1		9979	1116.1		11649	1116.5		11700	1116.5	
23000		5750	1113.7		(1113.5)	1115.2		(1113.8)	1.4		(1114.8)	.4		(1115.2)	.2		(1115.2)	.2
23500	23	5750	1112.3	5891	1112.2		6738	1112.5		10070	1113.4		11758	1113.8		11810	1113.8	
23600			(1112.0)		(1111.9)	1114.2		(1112.2)	2.0		(1113.2)	1.0		(1113.6)	.6		(1113.6)	.6
23650		5970	1111.9															
23696		5970	1111.4															
23700					(1111.2)	1113.7		(1111.5)	2.2		(1112.5)	1.2		(1113.0)	.7		1113.0	.7
23711		5970	1110.9															
23900	24			5995	1101.8		7025	1102.2		10530	1103.3		12307	1103.8		12355	1103.9	

NOTE:
1. 50-yr storm discharge and elevation taken from Design Report Volume II, Page 3.1-27 to 33
2. FRS 2 spillway rating Hydrology Documentation supplement No. 1, Page 107
3. Storm No. 1 is 24-hour 50-year Type II storm 3.6 inches
   Storm No. 2 is 1984 Storm distribution -- 4.0 inches
   Storm No. 3 is 1984 storm distribution FRS to Sta 3000-4.0"; Sta 3000 to Sta 9500-5.0"; Sta 9500 to Sta 13000-5.5"; Sta 13000 to Sta 23900-6.0"
   Storm No. 4 is 1984 storm distribution FRS to Sta 3000-4.0"; Sta 3000 to Sta 6500-5.0"; Sta 6500 to Sta 13000-6.0"; Sta 13000 to 23900-7.0"
   Storm No. 5 is 1984 storm distribution FRS-3.0"; FRS to Sta 6500-5.0"; Sta 6500 to Sta 13000-6.0"; Sta 13000 to Sta 23900-7.0"
4. Elevations shown in parenthesis are interpolated values.

TABLE 4
HARDWAHALA VALLEY WATERSHED
Saddleback Diversion

Peak Discharge from Incremental Drainage Areas

STATION	DESIGN Drainage Area Mi²	DESIGN Discharge cfs	TYPE II STORM DIST. Drainage Area Mi²	TYPE II STORM DIST. Storm 1 cfs	1984 STORM DISTRIBUTIONS Storm 2 cfs	1984 STORM DISTRIBUTIONS Storm 3 cfs	1984 STORM DISTRIBUTIONS Storm 4 cfs	1984 STORM DISTRIBUTIONS Storm 5 cfs	SIDE INLET Station	SIDE INLET Capacity cfs
1000	.840	773							700	780
1090			.27	362	250	250	250	325		
1500			.02	44	20	20	20	26	1390	360
2200	.293	355								
3000			.15	271	142	142	142	184	2900	180
4500			.29	471	272	353	353	353	4161	180
5500			.10	193	95	123	123	123	5200	210
6500			.02	44	20	26	26	26	6125	180
6800	.554	739								
7500			.53	512	451	591	732	732		
8500			.53	512	451	591	732	732	8012	1020
8700	1.075	956								
9500			.14	253	132	171	210	210	9408	205
11000			.60	643	524	766	847	847	10814	545
12200	1.304	1220							12126	535
12500			.61	653	533	779	861	861		
13000			.16	206	147	214	236	236	12832	85
16000			1.06	880	856	1390	1655	1655	15796	880
17000			1.07	888	864	1404	1670	1670	17005	470
17500			.16	204	147	235	279	279	17398	180
18000	2.323	1740								
18500			.26	320	235	377	448	448	18365	255
20500			1.17	894	917	1490	1774	1774	20200	800
22500			1.08	825	846	1375	1637	1637	22080	805
23500			.10	181	94	150	178	178		
23800	2.623	1792								
23900			.43	528	389	626	743	743		
24400	.4323	437								

NOTE: Storm 1 is 24-hour 50-year Type II Storm - 3.6 inches
Storm 2 is 1984 storm distribution - 4.00 inches
Storm 3 is 1984 storm FRS to Sta 3000 - 4.0"; Sta 3000 to Sta 9500 - 5.0"; Sta 9500 to Sta 13000 - 5.5"; Sta 13000 to Sta 23900 -6.0"
Storm 4 is 1984 storm FRS to 3000 - 4.0"; Sta 3000 to Sta 6500 - 5.0"; Sta 6500 - Sta 13000 - 6.0"; Sta 13000 to Sta 23900 - 7"
Storm 5 is 1984 storm FRS - 3.0"; FRS to Sta 6500 - 5.0"; Sta 6500 - Sta 13000 - 6.0"; Sta 13000 to Sta 23900 - 7"

APPENDIX A
(Comments on McGill Report by
Harry C. Millsaps)

United States Department of Agriculture

Soil Conservation Service

USDA - Soil Conservation Service
201 East Indianola Ave., Suite 200
Phoenix, Arizona 85012

Subject: ENG - Harquahala Valley Watershed, Saddleback Diversion, Storm, of September 2, 1984

Date: January 29, 1985

To: Ralph M. Arrington, SCE
Phoenix, AZ

File Code: 210-13-15

I have reviewed the Special Study Report on the Saddleback Diversion, Harquahala Valley Watershed for the Storm of September 2, 1984 by Herman McGill, Temple, Texas, and have drawn the following conclusions concerning the adequacy of the design of the diversion and its side inlets.

In general, it appears that the diversion channel itself functioned as design with only minor damage, although the design discharges were exceeded by more than two times in its lower reaches. (Design Q = 5995 cfs at Station 239+00 as compared to an estimated Q = 12,355 cfs for the September 2, 1984 storm.) The same, however, cannot be said for the inlet structures and their respective collector channels which were designed to collect and discharge overland flows into the diversion channel. I have evaluated the functioning of each of the inlets and made the following observations.

Inlet No. 1

This inlet is located at Station 7+00 just north of Courthouse Road. The inlet has a drainage area of 0.84 square miles, but was inadvertently left out of Mr. McGill's study. Observation in the field, however, showed that there was little or no damage to this structure and it seemed to have functioned as design. From Mr. McGill's study it can be shown that the historical peak discharges in this area were generally less than the design discharges.

Inlet No. 2

This inlet is located at Station 14+00, and although it did not overtop, there was some damage to the abutements, and the training dike overtopped downstream of the structure. The drainage area estimated by Mr. McGill (0.27 Mi^2) agrees closely with that used in the design (0.29 Mi^2). The September storm peak was estimated at 325 cfs as compared with the design discharge of 360 cfs. Therefore, the downstream training dike should not have overtopped and probably needs to be raised.

Inlet No. 3

Inlet No. 3 is located at Station 29+00 and has a drainage area of 0.15 square miles based on Mr. McGill's study. This compares with D.A. = 0.13 square miles used in design. The design peak was Q = 180 cfs as compared to

the estimated storm peak Q = 184 cfs. The difference is only 4 cfs and yet the inlet was overtopped by 0.4 feet. Either the storm peak was larger than estimated or the inlet structure was not properly designed and installed.

Inlet No. 4

This inlet is located at Station 41+62 and has a drainage area of 0.29 square miles as estimated by Mr. McGill. This is more than twice the drainage area of 0.14 square miles used in design. The September storm peak was estimated at 353 cfs as compared to the design Q = 180 cfs. Mr. McGill estimated a design Q = 471 cfs for the 50-year, 24-hour storm. This latter discharge would be somewhat high since Mr. McGill used a CN = 90 as compared to CN = 88 based on average conditions and used in the design. However, the inlet would be considered underdesigned since there is a difference in the contributing drainage area.

Inlet No. 5

Inlet No. 5 has a drainage area of 0.10 square miles as estimated by Mr. McGill and is located at Station 52+00. The drainage area used in design was 0.16 square miles, with a design peak of 210 cfs. The September storm peak was estimated at only 123 cfs and yet the collector channel overtopped downstream of the structure. Also this inlet may be improperly located since the major channel in the drainage area is located about 100 feet downstream of the present structure, and the collector channel was overtopped near this location during the September storm.

Inlet No. 6

The drainage area for this inlet is questionable. The structure is located near a small butte at Station 61+25 and appears to have a very limited drainage area based on a large scale photograph (Scale 1" = 400') used by Mr. McGill. Mr. McGill estimated the contributing area at only 0.02 square miles as compared to 0.13 square miles used in the design. The design peak was 180 cfs as compared to only 26 cfs estimated for the historical storm. The structure itself did not overtop, but there was some overtopping of the collector channel downstream of the structure noted by Mr. McGill. There appears to be some kind of channel in this area as indicated on the large photo, but none is shown on the as-built plans. It is recommended that a collector channel be installed in this area if there is none; or if there is one, that it be enlarged.

Inlet No. 7

This inlet is located at Station 80+12 and has a total drainage area of 1.08 square miles as used in the design. The design discharge was 1020 cfs. Mr. McGill split the drainage into two equal areas (0.55 square miles each) with estimated design peaks of 512 cfs each for the 50-year, 24-hour storm. This would mean that a new inlet structure would need to be located upstream of Inlet No. 7. A logical location would be near Station 72+00 where a rather large channel intersects the diversion and near the point where the side channel was overtopped during the September storm. Some work may need to be done near the small butte located in this drainage area in order to insure that flow would be directed torward the proposed new inlet structure. It is

in this drainage area where the historical storm peaks first begin to exceed designed discharges. Mr. McGill estimated the September storm peaks at 732 cfs each for the two separate drainage areas.

Inlet No. 8

This inlet is located at Station 94+08 and was designed for 250 cfs from a 0.16 square mile drainage area. These compare to a drainage area of 0.14 square miles used by Mr. McGill with a historical storm peak of 210 cfs. Little or no damage should have occurred in this area, but this was not the case. Major overtopping and severe erosion did occur both upstream and downstream of the structure. It could have been that the historical storm peak was much larger than estimated, or the collector channels were under designed or did not function as designed due to sediment deposits within the channels. Although Mr. McGill used a 6.0" rainfall in this reach for the September storm, his computed elevations were still below the surveyed highwater marks for the diversion.

Inlet No. 9 and 10

The drainage areas for these two inlets are questionable, since both serve the same major composite drainage area. It is unknown and impossible to determine which inlet would receive the largest amount of storm runoff. It was assumed by both Mr. McGill and the designers that the total D.A. would be split evenly between the two structures or about 0.60 square miles each. The computed historical peaks for each area was about 860 cfs, and compares to design peaks of about 550 cfs each. Although major overtopping occurred in this reach, the design discharges appear to be adequate. Based on the observations made on the upstreams inlets, however, I would suggest that the collector channels be evaluated for capacity.

Inlet No. 11

The design of this inlet should have been pretty straight forward since it has a fairly well defined drainage area. Mr. McGill estimated D.A. = 0.16 square miles as compared to 0.24 square miles used in design. The structure is located at Station 128+32, and was overtopped during the September storm. This would be expected, since the storm peak (236 cfs) exceeded that used in design (185 cfs). I would judge that this structure is adequate from a hydrologic standpoint, although it hydraulics may need to be evaluated.

Inlet No. 12 and 13

The drainagae area for these two structures, like those for Inlets No. 9 and 10, are questionable, since there is a separation of flow near the point where the mountain canyon discharges onto the alluvial fan. Although both Mr. McGill and Toups assumed that the canyon flow would be divided equally between the two inlets, there is a major difference in the total drainage area used in the respective studies. Mr. McGill estimated a total drainage area of 2.13 square miles as compared to 1.66 square miles used by Toups. Both Mr. McGill and Toups used 1.06 square miles for Inlet No. 12, but on Inlet No. 13, Mr. McGill used 1.07 square miles as compared to 0.60 square miles used by Toups. Assuming that Mr. McGill's drainage areas are correct,

Inlet No 12 would be adequately designed, but Inlet 13 would be underdesigned. During the September 1984 storm, however, the estimated peak for even Inlet No. 12 was nearly doubled that of the design discharge. (Design Q = 880 cfs as compared to the estimated September storm peak of 1655 cfs.) For Inlet No. 13, the Design peak was 470 cfs as compared to the computed peak for the September storm of 1670 cfs or more than three times the design flow. Again it is suggested that the collector channels be evaluated for capacities.

Inlet No. 14

This inlet is located at Station 173+98 and has a fairly well defined drainage area, although some of the canyon flow from Inlets 12 and 13 drainage areas could be diverted into this inlet. Mr. McGill estimated the contributing area at 0.16 square miles as compared to 0.20 square miles used in the design. The design flow was estimated at 180 cfs and appears to be reasonable, although the September storm produced a discharge estimated at 279 cfs, which washed out both abutements and overtopped the diversion embankment.

Inlet No. 15

This inlet, located at Station 183+65, has a drainage area of 0.26 square miles based on Mr. McGill's study as compared to 0.31 square miles used in the design. This structure is again subject to excess flow from the canyon assumed to be contributing to Inlets No.'s 12 and 13. The design discharge was estimated at 255 cfs and appears to be adequate, although the capacity of the collector channel needs to be checked. The peak for the September storm was estimated at 448 cfs.

Inlet No. 16

This inlet is located at Station 202+00, with a drainage area of 1.17 square miles and a design discharge of 800 cfs. Although this structure was not referenced in Mr. McGill's field notes he did observe that severe erosion had occured both upstream and downstream of the structure with a 3' deep , 30'-wide channel being eroded in the embankment of the diversion near Station 202+75. The design discharge appears to be adequate although the historical peak was estimated at 1774 cfs.

Inlet No. 17

Inlet No. 17 is located at Station 220+00 and has a drainage area of 1.08 square miles as determined by Mr. McGill. This compares with a D. A. = 1.16 square miles used in design. The design peak was estimated at 805 cfs and appears to be adequate, although the September storm produced a peak of 1637 cfs. From the large photo, it can be noted that the drainage areas for both Inlets No. 16 and 17 are subjective, since they serve a common drainage area. The division where about half of the total drainage area contributes to each inlet is as good as any, and was used by both Mr. McGill and PRC Toups Corporation.

Inlet No. 18

This inlet is located at 241+75 but was not constructed and does not show on the as-built plans except as a "lateral swale". Some erosion, however, did occur at this location during the September storm, and it is recommended that an inlet structure be installed. The total drainage area was estimated at 0.43 square miles by both Mr. McGill and Toups.

Summary of Conclusions

Based on Mr. McGill's study, Inlet Structures No's. 4 and 13 are definitely underdesigned due to differences in drainage areas. There are also some questions as to adequacy of the collector channels on most structures with reference to their designed capacities. There is also a need for a structure at Station 241+75 or Inlet No 18, although this structure does not show on the as-built plans. It is proposed that a new structure be located near Station 72+00 in the reach between Inlets No.'s, 6 and 7.

Submitted by:

Harry C. Millsaps

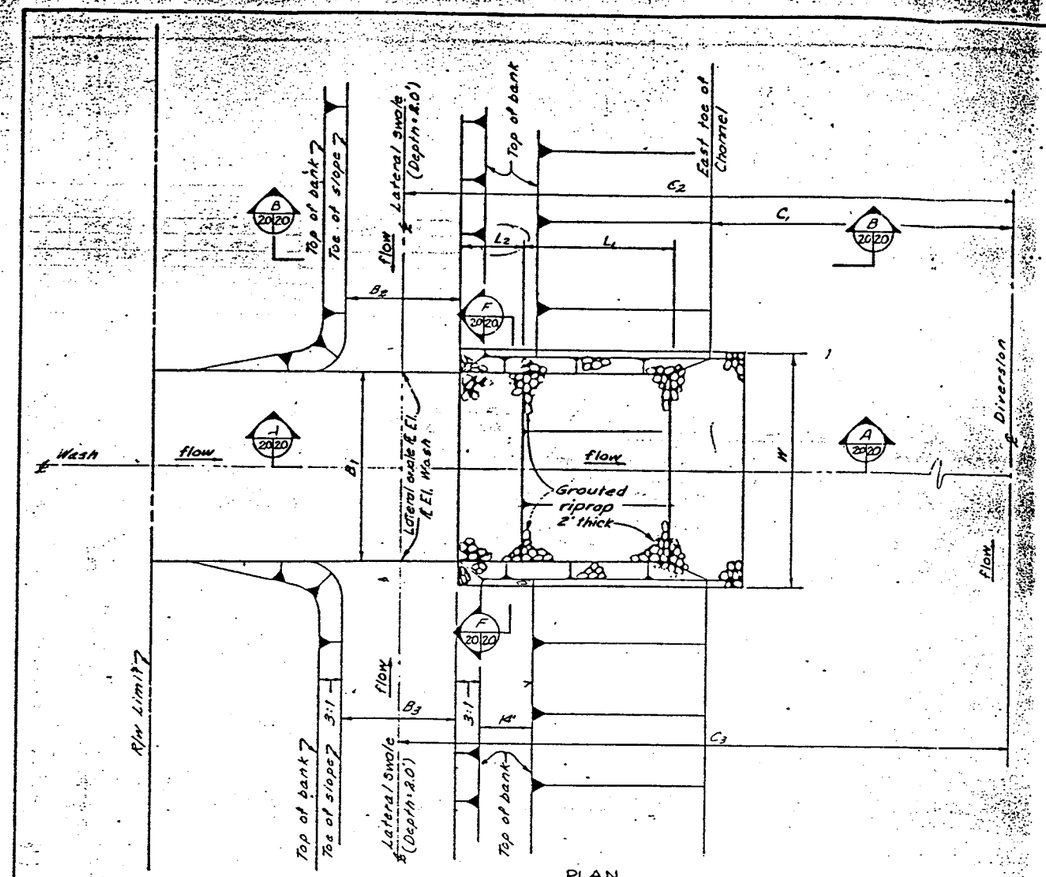
Harry C. Millsaps
Hydraulic Engineer

APPENDIX B

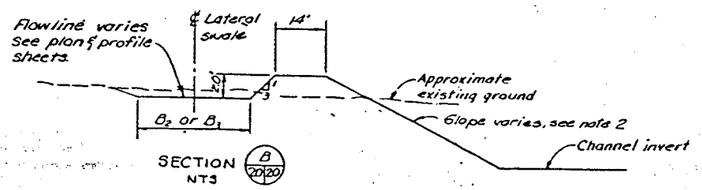
APPENDIX B
INLET WEIR DETAILS

C_1	C_2	C_3	L_1	℄ STATION	Location Point	Approx Station Along ℄	℄ Lateral Swale Elev.	℄ ** Wash Elev.	Schedule Of Inlet Structures Dimensions (Ft) B_1	B_2	B_3	C_1	C_2	C_3	L_1	L_2	W	Q cfs
12.5	—	71.2	25.7	7+00	1*	7+00	1179.00	1179.00	60	—	10	12.5	—	72	31	20	76	780
15.0	—	82.4	32.0	13+90	2	14+00	1178.00	1178.00	22	—	10	15	—	83	38	20	39	360
18.5	—	86.0	32.5	29+00	3	29+00	1163.50	1163.50	10	—	10	18.5	—	86	37.5	20	26	180
29.53	87.13	87.13	22.5	41+60.6	4	41+30	1165.00	1165.00	10	10	10	29.5	87	87	29	20	26	180
58.63	91.63	91.63	18.0	52+00	5	53+00	1162.00	1162.00	12	10	10	58.5	91	91	18	20	28	210
44.72	—	98.97	17.3	61+26	6	61+25	1160.00	1160.00	10	—	10	[illegible]	—	97	18	20	26	180
63.35	—	129.5	26.0	80+12	7	80+00	1158.00	1158.00	45*	—	20*	[illegible]	—	[illegible]	[illegible]	20	61	1020
71.57	129.7	129.7	23.0	94+08.5	8	94+00	1150.00	1150.00	10	10	10	71.5	129.5	[illegible]	[illegible]	20	26	205
79.85	—	140.0	25.0	108+14.5	9	108+00	1147.50	1147.50	45	—	10	[illegible]	—	145	[illegible]	20	61	545
89.65	144.9	144.9	20.2	121+26	10	121+00	1144.00	1144.00	35	10	10	90	[illegible]	[illegible]	[illegible]	20	51	565
94.95	—	157.2	27.3	128+32	11	128+00	1144.00	1144.00	[illegible]	—	10	[illegible]	—	[illegible]	[illegible]	20	29	135
115.0	—	190.0	30.0	157+96	12	158+00	1132.00	1132.00	45	—	30	[illegible]	—	[illegible]	[illegible]	20	61	330
121.4	—	189.7	28.3	170+05	13	170+00	1129.00	1129.00	25	—	20	[illegible]	—	[illegible]	[illegible]	20	41	470
123.5	—	—	22.6	173+98	14	174+00	—	1127.00	10	—	—	[illegible]	—	[illegible]	[illegible]	20	26	180
128.6	—	181.8	18.3	183+65	15	183+50	1124.00	1124.00	15	—	10	[illegible]	—	[illegible]	[illegible]	20	31	255
138.3	—	196.7	16.6	202+00	16	202+00	1120.00	1120.00	50	—	20	[illegible]	—	[illegible]	[illegible]	20	66	620
145.6	—	209.6	23.8	220+80	17	221+00	1117.00	1117.00	45	—	20	[illegible]	—	[illegible]	[illegible]	20	61	805
					18***	224+00	1106.0	1106.0	—	—	10	—	—	820	—	—	—	25

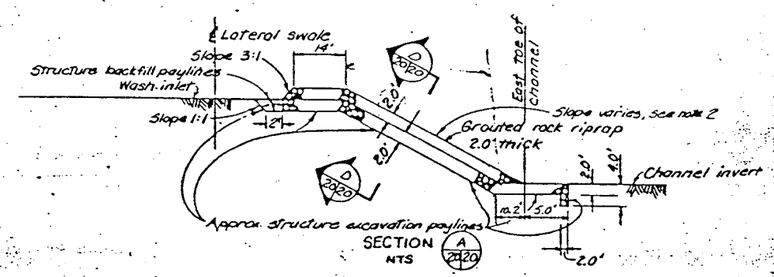
* Structure No. 1 is located in grouted riprapped section of diversion.
** ℄ Elevation may require compacted fill to provide two feet depth inlet weir cross section. Compacted fill to be in accordance with specifications.
*** Structure No. 18 consists of a lateral swale only. No inlet structure is provided.



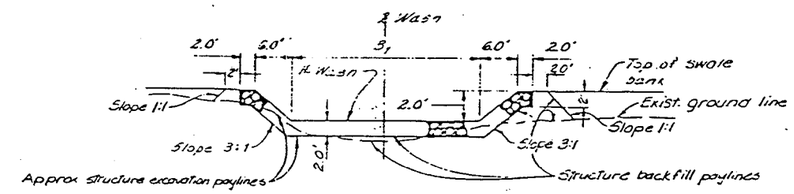
PLAN
NTS



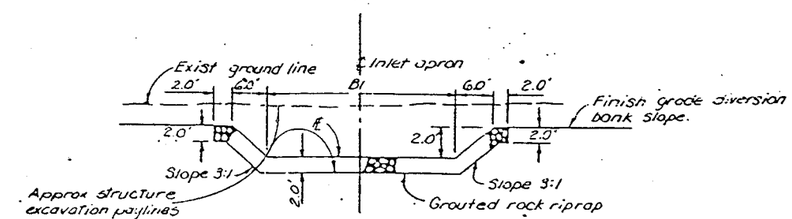
SECTION (B 20/20)
NTS



SECTION (A 20/20)
NTS



SECTION (F 20/20)
NTS



SECTION (D 20/20)
NTS

NOTE:
1. Lateral swale stationing, as shown on sheets 5 to 11, are approximate. Actual length of lateral swale to be determined in the field at the time of construction by the engineer.
2. Slope is 4:1 for location Pt. No. 1 and 5:1 for all others.

AS BUILT
DATE MAY 1982

INLET WEIR DETAILS

SADDLEBACK DIVERSION
HARQUAHALA VALLEY W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed C.E.S.	Date 9-78	Approved by	
Drawn J.B., L.J.C.	9-78	Title	
Traced		Sheet No. 20 of 20	Drawing No. 7-E-24040
Checked A.P.	2-79		

TOUPS CORPORATION

CONSULTING ENGINEERS AND PLANNERS

APPENDIX B
(Excerpt from Design Report)

BY JB DATE 9/26/78 CLIENT U.S. Soil Conservation Service SHEET NO. ____ OF ____

CHECKED ____ DATE ____ JOB Saddleback FRS & Div. Chl. JOB NO. 3043-000

Weir Inlets - 50-YR Design Discharge Estimation

TR-20 AREA (Schematic)	AREA SQ MI.	CFS Q	SUB AREA	AREA SQ. MI.	cfs Q (Design Weir)	Approx. Q/2	Approx Q/3
7c (029)	0.84	773	1	0.84	773	400	250
7A (028)	0.293	355	2	0.293	355	200	110
7B (031)	0.554	739	3	0.129	172	100	60
			4	0.136	182	100	60
			5	0.133	177	100	60
			6	0.156	208	100	70
7c (034)	1.075	956	7	1.075	956	500	320
8A (037)	1.304	1220	8	0.160	150	100	50
			9	0.572	535	250	180
			10	0.572	535	250	180
8B (040)	2.323	1740	11	0.235	176	100	60
			12	1.064	797	400	270
			13	0.228	171	100	60
			14	0.600	449	200	150
			15	0.196	147	100	50
9A (043)	2.623	1792	16	0.310	212	100	70
			17	1.156	790	400	260
			18	1.157	790	400	260
9B (046)	0.432	437	19	0.432	437	200	150

* Swale Design Q

SADDLE BACK DIVERSION CHANNEL 10-21-78 CES

LATERAL WASHES SECTIONS THRU EAST BANK 3.1-52

PT ① [a] AREA 7_c 0.84 SM Q_{50} = 773 CFS

CHANNEL THRU BANK

APPENDIX B
(Excerpt from Design Report)

B= 60' Z= 2 n= 0.03 D= 2' S= 0.0067

V= 6.1 Q = 782 CFS

PT ② AREA 7A 0.29 SM Q_{50} = 355 CFS

B= 22' Z= 2 n= 0.03 D= 2' S= 0.01

V= 7.0 Q= 364

PT ③ AREA 0.129 SM Q_{50} = 172

B= 10' Z= 2 n= 0.03 D= 2' S= 0.01

V= 6.4 Q= 180

PT ④ AREA 0.136 SM Q_{50} = 182

B= 10' Z= 2 n= 0.03 D= 2' S= 0.01

V= 6.4 Q= 180

PT ⑥ AREA 0.133 SM Q_{50} = 177

B= 10' Z= 2 n= 0.03 D= 2' S= 0.01

V= 6.4 Q= 180

PT ⑤ AREA 0.156 SM Q_{50} = 208

B= 12' Z= 2 n= 0.03 D= 2' S= 0.01

V= 6.6 Q= 210

PT ⑦ AREA 1.075 SM Q_{50} = 956

B= 45' Z= 2 n= 0.03 D= 2' S= 0.02

V= 10.4 Q= 1022

PT ⑧ AREA 0.16 SM Q_{50} = 150

B= 10 Z= 2 n= 0.03 D= 2' S= 0.0133

V= 7.4 Q= 207

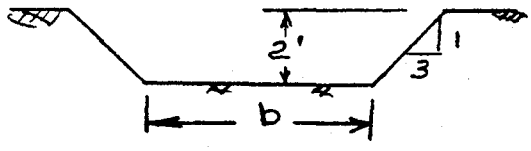
[a] See sht. 3.1-55 for approx. weir inlet location

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE ARIZONA		PROJECT SADDLEBACK DIVERSION		
BY J. Weaver	DATE 1-11-86	CHECKED BY	DATE	JOB NO.
SUBJECT Engineering Report				SHEET 1 OF 5

Compute As-Built Capacity of Side Inlets



Weir flow

EQUIVALENT RECTANGULAR SECTION

$W = b + Z(d)$
$W = b + 3 \cdot 2$
$W = b + 6$

RECTANGULAR BROAD-CRESTED WEIR

$Q = 3.1\, W H^{3/2}$

$W = b + 6$
$H = 2$
$H^{3/2} = 2.828$

Inlet #	b	As-Built Q	50-year Q	% As-Built/50-year
1	60	578	773	75 %
2	22	245	325	69
3	10	140	172	81
4	10	140	182	77
5	12	158	208	76
6	10	140	177	79
7	45	447	956	47
8	10	140	150	82
9	45	447	535	84
10	35	360	535	67
11	12	158	176	90
12	45	447	797	56
13	25	272	449	61
14	10	140	147	95
15	15	184	212	87
16	50	491	790	62
17	45	447	790	57

TOUPS CORPORATION

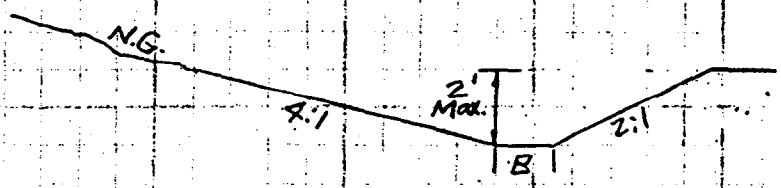
CONSULTING ENGINEERS AND PLANNERS

BY JB DATE 9/27/78 CLIENT US Soil Conservation Service SHEET NO. ____ OF ____

CHECKED ____ DATE ____ JOB Saddleback FRS & Div. Ch1 JOB NO. 3043-000

Lateral Swale Design Table for subareas (1-19) contributary

50 yr frequency storm runoffs.



TYPICAL SECTION
(LOOKING DOWNSTREAM)

Z:1 = 3 (Average)
B = Varies
n = 0.03
d = 2' (Max.)
S = 0.003%

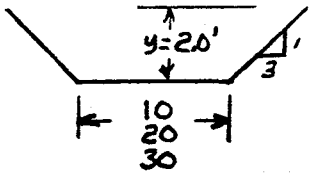
B	d	Q	V	Dist out from top of 2:1 slope
0.0001	1	4.95	1.65	+6'
10	1	30	2.33	+16
20	1	57	2.48	+26
30	1	84	2.54	+36
40	1	111	2.58	+46
0.0001	2	31	2.62	+12'
10	2	109	3.42	+22'
20	2	192	3.70	+32'
30	2	277	3.85	+42'
40	2	362	3.94	+52'
50	2	448	4.00	+62'
DITCH TO PT 1 10	2	141	4.4	S = 0.005 Z = 3 (AV.)

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT		
ARIZONA		SADDLEBACK DIVERSION		
BY J. Weaver	DATE 1-8-86	CHECKED BY	DATE	JOB NO.
SUBJECT ENGINEERING REPORT				SHEET 2 OF 5

AS-BUILT CAPACITY OF LATERAL SWALES



$n = 0.03$

$$Q = \frac{1.49 A R^{2/3} \sqrt{S}}{n}$$

general formulas

b=10'

$A = (b+zy)(y) = [10+3(2)]2 = 32$

$p = b + 2y\sqrt{1+3^2} = 10 + 4(3.16) = 22.64$

$R = A/p = 32/22.64 = 1.41$

$R^{2/3} = 1.26$

$$Q = \frac{(1.49)(32)(1.26)\sqrt{S}}{.03} = \underline{\underline{2002\sqrt{S}}}$$

b=20'

$A = (b+zy)(y) = (20+6)(2) = 52$

$P = 20 + 4\sqrt{10} = 32.65$

$R = A/p = 1.59$

$R^{2/3} = 1.36$

$$Q = \frac{(1.49)(52)(1.36)\sqrt{S}}{0.03} = \underline{\underline{3512\sqrt{S}}}$$

b=30'

$A = (30+6)(2) = 72$

$P = 30 + 4\sqrt{10} = 42.65$

$R = A/P = 1.69$

$R^{2/3} = 1.418$

$$Q = \frac{(1.49)(72)(1.418)\sqrt{S}}{0.03} = \underline{\underline{5070.8\sqrt{S}}}$$

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE	PROJECT		
ARIZONA	SADDLEBACK DIVERSION		
BY J. WEAVER / DATE 1-8-86	CHECKED BY	DATE	JOB NO.
SUBJECT ENGINEERING REPORT			SHEET 3 OF 5

COMPUTE AS-BUILT SLOPES OF LATERAL SWALES

Channel slopes are not shown on drawings. Obtain slopes from slope stake field books.

	Weir Inlet		Stations	L	Δ ELEV	Slope
Field Book #4	1		4+00 to 7+00	300'	1.4'	0.0047
	2		10+60 to 13+90	330'	3.5	0.0106
	3		26+00 to 29+00	300'	1.5'	0.005
	4	N	35+50 to 41+63	613'	2.5'	0.004
		S	41+63 to 43+60	197'	0.8'	0.004
Field Book #3	5	N	50+00 to 52+00	200'	1.0	0.005
		S	52+00 to 59+00	700'	1.0	0.0014
	6		59+00 to 61+25	225'	3.0'	0.0133
	7		73+00 to 80+13	713	3.0'	0.0042
	8	N	88+50 to 94+08	630	3.0'	0.0048
		S	94+09 to 97+20	311	2.0'	0.0064
	9		104+00 to 108+17	417	2.9	0.007
Field Book #2	10	N	117+80 to 121+25	345'	2.0	0.0058
		S	121+25 to 124+00	275'	2.0	0.0073
	11		125+10 to 128+00	300'	0.6	0.002
	12		133+40 to 158+50	2,510'	9.0	0.0036
	13		161+20 to 170+00	880'	3.0'	0.0034
Field Book #1	14					
	15		177+00 to 183+65	665'	4.0'	0.006
	16		195+00 to 202+00	700'	4.0	0.0057
	17		210+50 to 221+00	1,050'	4.0'	0.0038
	18		237+13 to 241+75	462'	0 (FLAT)	0.0000

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE	PROJECT		
ARIZONA	SADDLEBACK DIVERSION		
BY J. Weaver / DATE 1-8-86	CHECKED BY	DATE	JOB NO.
SUBJECT Engineering Report			SHEET 4 OF 5

As-Built Capacity of Lateral Swales

inlet	Swales bottom Width	slope		Q cfs
1	10	.0047	$2002\sqrt{S}$	= 137
2	10	.0106	"	= 206
3	10	.005	"	= 142
4 N	10	.004	"	= 128
S	10	.004	"	= 128
5 N	10	.005	"	= 142
S	10	.0014	"	= 75
6	10	.0133	"	= 231
7	20	.0042	$3512\sqrt{S}$	= 227
8 N	10	.0048	$2002\sqrt{S}$	= 139
S	10	.0064	"	= 160
9	10	.007	"	= 167
10 N	10	.0058	"	= 152
S	10	.0073	"	= 171
11	10	.002	"	= 90
12	30	.0036	$5070.8\sqrt{S}$	= 304
13	20	.0034	$3512\sqrt{S}$	= 205
14	—			
15	10	.006	$2002\sqrt{S}$	= 155
16	20	.0057	$3512\sqrt{S}$	= 265
17	20	.0038	$3512\sqrt{S}$	= 216
18	10	FLAT	$2002\sqrt{S}$	= ? S=0.00%

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE ARIZONA		PROJECT SADDLEBACK DIVERSION	
BY J. Weaver	DATE 1-9-86	CHECKED BY	DATE	JOB NO.
SUBJECT Engineering Report			SHEET 5 OF 5

COMPARE DESIGN & AS-BUILT Q FOR LATERAL SWALES

inlet	design inlet Q	design swale Q/3 1/	As-built Q 2/
1	773	250	137
2	355	110	206
3	172	60	142
4	182	60	128
5	208	70	N = 142 S = 75
6	177	60	231
7	956	320	227
8	150	50	N = 139 S = 160
9	535	180	167
10	535	180	N = 152 S = 171
11	176	60	90
12	797	270	304
13	449	150	205
14	147	60	—
15	212	70	155
16	790	260	265
17	790	260	216
18	790	260	? S = 0.00%

1/ SWALE Q = INLET Q/3. SEE DESIGN FILE, p. 3.1-50

2/ MAX. CAPACITY AT 2.0' depth. No freeboard.

APPENDIX C

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY

B. DWAINE SERGENT, P.E.
LAWRENCE A. HANSEN, PH.D., P.E.
RALPH E. WEEKS, P.G.
DARREL L. BUFFINGTON, P.E.
DONALD VAN BUSKIRK, P.G.
JOHN B. HAUSKINS, P.E.
DALE V. BEDENKOP, P.E.
DONALD L. CURRAN, P.E.
J. DAVID DEATHERAGE, P.E.
MICHAEL R. RUCKER, P.E.
GEORGE H. BECKWITH, P.E.
ROBERT W. CROSSLEY, P.E.
DONALD G. METZGER, P.G.
JONATHAN A. CRYSTAL, P.E.
PAUL V. SMITH, P.G.
ROBERT D. BOOTH, P.E.
NORMAN H. WETZ, P.E.
ROBERT L. FREW
ALLON C. OWEN, JR., P.E.

February 12, 1986

Mr. David O. Lambson
Contracting Officer
United States Department of Agriculture
Soil Conservation Service
201 East Indianola
Suite 200
Phoenix, Arizona 85012

Re: Investigation Report
Saddleback Division
Harquahala WPP

Dear Mr. Lambson,

I apologize for not replying sooner to your request of January 9, 1986. Robert R. Koons, P.E., is no longer employed by Sergent, Hauskins & Beckwith Geotechnical Engineers, Inc. (SHB) and I felt he could best perform this review. By the time I got the information to him, and he discussed the project with your staff, this much time had passed.

I have attached Mr. Koons' letter, which is self-explanatory. If you have questions or if we can be of service, please call.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By _____
B. Dwaine Sergent, P.E.
President

Copies: Addressee (1)

REPLY TO: 3940 W. CLARENDON, PHOENIX, ARIZONA 85019

PHOENIX (602) 272-6848 | ALBUQUERQUE (505) 884-0950 | SANTA FE (505) 471-7836 | SALT LAKE CITY (801) 266-0720 | EL PASO (915) 778-3369

Robert R. Koons, P.E., Inc.
Construction Engineering Services

February 3, 1986

B. Dwaine Sergent
Sergent, Hauskins & Beckwith
3940 W Clarendon Ave
Phoenix, AZ 85019

Ref: Investigation Report Saddleback Div., Harguahala WPP

Dear Mr. Sergent:

Responding to a request from SCS, I have reviewed their Engineering Report prepared to investigate possible deficiencies in the structures following a storm in September, 1984. My comments are related to construction and inspection services provided by Sergent, Hauskins & Beckwith.

The report was reviewed without the benefit of a set of plans or construction photographs. As the construction was completed nearly four years ago, I can rely only on the memory of the work as I recall seeing it.

The investigators apparently made a thorough analysis of the hydrology and hydraulics for the particular storm event in question. The inherent vagaries of these studies became obvious as the investigators did not exactly agree with the designers in calculating of drainage areas for the various structures. Rainfall distribution pattern is unknown and inflow at the various structures was estimated by methods unknown to me. Mr. Millsaps concluded that two of the Inlet Structures were underdesigned and he questions the adequacy of the collection channels on most of the structures.

4645 S. Lakeshore Dr., Suite 7 • Tempe, AZ 85282 • (602) 820-3104

The report reveals no deficiency or fault in construction or materials. Inasmuch as I did not inspect the damange following the storm, I cannot comment on the quailty or performance of the structures or channels. But again considering the variables inherent in hydrolgy studies as well as in construction of the lateral collection channels, it may be nearly impossible to predit exactly the performance of these structures. Even with the knowledge gained from this study the exact performance for an event with a somewhat different rainfall distribution pattern would be difficult, in my opinion.

In conclusion, and with only the limited information available to me, I can find no relationship between the construction and inspection services provided and the damage resulting from the September 1984 storm. My only recommendation would be to provide a greater saftey factor in design for those project features in which the variables in design and construction have the greatest impact on performance.

Very truly yours,

Robert R. Koons

Robert R. Koons, P.E.
President

PRC Engineering
4131 North 24th Street
Phoenix, AZ 85016
602-954-9191

prc
Planning Research Corporation

11 March 1986

Ralph M. Arrington
State Conservation Engineer
USDA, Soil Conservation Service
201 East Indianola, Suite 200
Phoenix, Arizona 85012

Dear Mr. Arrington:

First, I would like to apologize for the delay in responding to the letter of January 9, 1986 from Mr. David O. Lambson of your office. Basic research of the Saddleback Diversion Harquahala W.P.P. was required by our office since the engineers who originally worked on the project are no longer employed at PRC Engineering.

It is apparent that the September 2, 1984 storm which the Diversion Structure experienced was significantly greater than the 50-year 24-hour design storm. According to the post-storm analysis performed by the SCS, the main diversion channel operated with little or no problems during this event. However, some erosion damage did occur in several of the seventeen side inlets to the main channel.

It is our opinion that all of the side inlets were properly sized and located at the time of design. Eleven of the seventeen side inlets were relocated during construction varying from 2 to 82 feet according to the SCS post-storm analysis. As of yet, our office has not found documentation for relocating these side inlets. However, it is possible that the side inlets were adjusted in the field during construction for some reason; possibly to better fit natural drainage channel locations which existed at the time of construction.

During a storm of this intensity and magnitude it is common for alluvial fan type flows to migrate across the fan and establish new drainage channels and concentration points. Without the benefit of mapping pre- and post-flood drainage patterns this cannot be verified but, based on our present knowledge of alluvial fan hydrology it is probable that some drainage patterns did change during this storm.

Evidence of sediment deposition in the side inlets, as cited in the SCS post-storm analysis may have been responsible for reduced capacity and contributed to the over-topping of several side inlets. Sediment deposition may have occurred in the side inlets prior to the September 2, 1984 storm in sufficient quantity to reduce the capacity of the inlets. Also, because the September 2, 1984 storm was so intense it may have transported larger than design quantities of sediment into the side inlets.

One or a combination of both of these factors could have reduced the capacity of the side inlets and caused flows to over-top them.

After careful review of the post-storm document prepared by the SCS and our engineering files on the Saddleback Diversion Harqualala W.P.P., it is PRC Engineering's position that the structure performed at or beyond its design capabilities. We trust that you are satisified with the structure and look forward to working with the SCS in the future.

If you have any additional questions regarding this matter please do not hestitate to call me at 954-9191.

Sincerely,

PRC ENGINEERING, INC.

John Gerometta, P.E.
Associate Vice President

JG:smk

APPENDIX D

ENGINEERS, INC.
2255 N. 44th St. Suite 220 • Phoenix, Az. 85008 • Phone (602) 244-2566

Edward A. Adair, P.E.
R. Gerald Green, P.E.
Samuel E. Kao, PhD, P.E.

April 17, 1987

Mr. Ralph M. Arrington
State Conservation Engineer
UNITED STATES DEPARTMENT OF AGRICULTURE
Soil Conservation Service
201 East Indianola Avenue
Suite 200
Phoenix, Arizona 85012

Dear Mr. Arrington:

On March 25, 1987, I received a request to comment upon the damage investigations carried out by your office on the Saddleback Diversion in the Harquahala Valley Watershed. I have reviewed your engineering report dated February 1987 and concur with the comments submitted by PRC Engineering in their letter dated 11 March 1986.

In addition, I offer the following observations:

1. The inappropriateness of using the Mannings formula for the side inlet design was concluded by your office in favor of a weir formula. My examination of drawing no. 7-E-24040 reveals the intention of positive slope from lateral swales through side inlets and down the side slope of the diversion. Therefore, I am still of the opinion that the use of channel flow criteria is appropriate.

2. The effectiveness of flood protection structures is greatly dependent upon proper maintenance. This is especially true for those elements having no freeboard and no deposition capacity. In addition, if the upslope side of the diversion is located normal to an alluvial fan, side inlets and accompanying training channels and dikes may be left high and dry as a result of a localized previous storm event. Nothing was indicated in the report about the history of maintenance and more particularly the length of time since the last maintenance was carried out. References in the report were made to reductions in side inlet capacity due to sediment bars.

If I can be of further assistance, please do not hesitate to contact me.

Sincerely,

Edward A. Adair

Edward A. Adair, P.E.

EAA:pmm

PRC Engineering
4131 North 24th Street
Phoenix, AZ 85016
602-954-9191

prc
Planning Research Corporation

April 27, 1987

3043-000-00

Mr. Ralph M. Arrington
State Conservation Engineer
USDA, Soil Conservation Service
201 East Indianola, Ste 200
Phoenix, Arizona 85012

Dear Mr. Arrington:

PRC Engineering has finished reviewing the Engineering Report for the Saddleback Division Hanquahala Valley Watershed in Maricopa County, Arizona, dated February 1987. We appreciate the opportunity to review this document and make comment.

As we stated in our earlier letter dated March 11, 1986, none of the engineers or designers who participated on this project are currently employed by PRC Engineering. Lacking the first hand knowledge has put us at a disadvantage regarding the specifics of the design process and how design decisions were arrived. However, we feel the current Engineering Report addresses some of the elements brought to attention in our March 11, 1986 correspondence.

At this time we have no further comments concerning the revised Engineering Report. If you require any additional information, please do not hesitate to call me at 954-9191.

Sincerely,

PRC ENGINEERING, INC.

John W. Gerometta, P.E., L.S.
Associate Vice President
Director of Public Works

JG/js/87