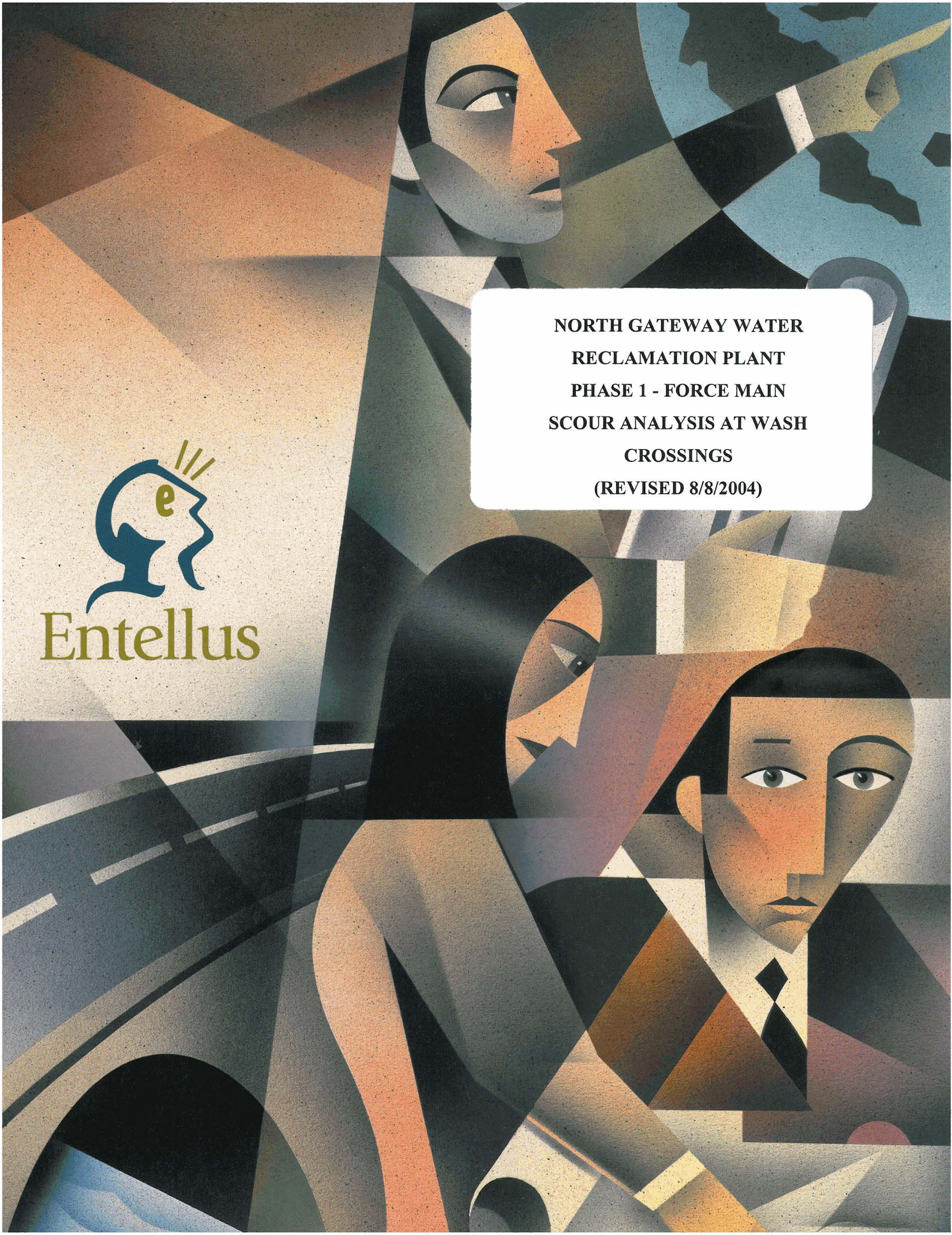

NORTH GATEWAY WATER RECLAMATION PLANT PHASE 1 - FORCE MAIN SCOUR ANALYSIS AT WASH CROSSINGS

(REVISED 8/8/2004)

Entellus

NORTH GATEWAY WATER RECLAMATION PLANT PHASE 1 - FORCE MAIN SCOUR ANALYSIS AT WASH CROSSINGS (REVISED 8/8/2004)

Project No. WS85090003

Entellus Project No. 910.005

April 2004, Revised August 2004

Prepared by:

Intelligent Engineering

Environmental Solutions

Registered Professional Engineer (Civil)
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ARISTIZABAL
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NORTH GATEWAY WATER RECLAMATION PLANT
PHASE 1 - FORCE MAIN
SCOUR ANALYSIS AT WASH CROSSINGS
EXECUTIVE SUMMARY

Damon S. Williams and Associates (DSWA) has designed a 24" force main from Cave Creek Road to a lift station located ¾ of a mile northwest of Interstate 17 and the Central Arizona Project Canal (CAP Canal) crossing. Entellus was retained to perform scour analyses at various wash crossings along the alignment.

The Arizona Department of Water Resources *State Standards Attachment 5-96*, was the main source used to estimate the scour depths, unless a more adequate analysis was found from previous studies. The table below shows a description and scour depth estimate for each wash crossing.

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Crossing No.	Crossing Description	Crossing Location	Estimated Scour Depth (ft)
1	Cave Creek Tributary	Approximately ¾ mile west of Cave Creek Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses a tributary to Cave Creek Wash	Negligible
2	Cave Creek Wash	Approximately 1000 feet west of Cave Creek Dam Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses Cave Creek Wash	Bedrock (3.0 - 5.0)
2a	Emergency Spillway Wash	Approximately 200 feet west of Cave Buttes Dam Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses a wash from the emergency spillway of Cave Buttes Dam	1.5
3	CAP Overchute	Just north of the CAP Canal, at approximately the 15th Avenue alignment, and approximately ½ mile north of Happy Valley Road where the force main alignment crosses a wash that feeds into a CAP Canal overchute	1.3
4	Sonoran Wash	Approximately ¾ north of the CAP Canal, near the 26th Avenue alignment where the force main crosses Sonoran Wash	5.0

NORTH GATEWAY WATER RECLAMATION PLANT

PHASE 1 - FORCE MAIN

SCOUR ANALYSIS AT WASH CROSSINGS

TABLE OF CONTENTS

SECTION 1: PROJECT DESCRIPTION

Damon S. Williams and Associates (DSWA) has designed a 24" force main from Cave Creek Road to a lift station located ¾ of a mile northwest of Interstate 17 and the Central Arizona Project Canal (CAP Canal) crossing. The force main alignment is approximately eight miles long and is shown on the copy of **Reference 4,** located in the pocket at the end of this report. Entellus was retained to perform scour analyses at various wash crossings along the alignment. A copy of **Reference 4** has been annotated with the locations of these crossings. The following is a description of the crossings:

Crossing No. 1 (Cave Creek Tributary Wash)

Located approximately ¾ mile west of Cave Creek Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses a tributary to Cave Creek Wash.

Crossing No. 2 (Cave Creek Wash)

Located approximately 1000 feet west of Cave Creek Dam Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses Cave Creek Wash.

Crossing No. 2a (Emergency Spillway Wash)

Located approximately 200 feet west of Cave Buttes Dam Road, and ¼ mile south of the Happy Valley Road alignment, where the force main alignment crosses a wash from the emergency spillway of Cave Buttes Dam.

Crossing No. 3 (CAP Overchute Wash)

Located just north of the CAP Canal, at approximately the 15th Avenue alignment, and approximately ½ mile north of Happy Valley Road where the force main alignment crosses a wash that feeds into a CAP Canal overchute.

Crossing No. 4 (Sonoran Wash)

Located approximately ¾ north of the CAP Canal, near the 26th Avenue alignment where the force main crosses Sonoran Wash.

SECTION 2: METHOD DESCRIPTION

2.1 General Methodology

The Arizona Department of Water Resources *State Standards Attachment* 5-96 (SSA 5-96) (**Reference 6),** was the main source used to estimate the scour depth, unless a more adequate analysis was found from previous studies. The specific procedures for determining scour depths varied for each crossing, and are summarized below.

2.2 Method at Crossing Nos. 1, 2, 2a and 3

No relevant existing scour analyses were found for crossings at the Cave Creek Tributary Wash, the Cave Creek Wash, the Emergency Spillway Wash, or the CAP Overchute Wash. The scour depths at these locations were determined in accordance with the procedures outlined in the SSA 5-96 (**Reference 6**). For convenience, relevant portions of the SSA 5-96 have been reproduced and included in **Appendix F**. Three levels of analysis are described in the SSA 5-96. The level 3 analysis was only considered for the crossing at Cave Creek Wash, because for the other crossings, the analyses of levels 1 and 2 appeared to be adequate for the purposes of this project. The level 3 analysis of the crossing at Cave Creek Wash is discussed further in **Section 4.2**. Levels 1 and 2 analyses were performed at Cave Creek Tributary Wash, Cave Creek Wash, and the Emergency Spillway Wash (Crossing Nos. 1, 2 and 2a). Some data required to complete the level 2 analysis was not available for the wash leading to the CAP overchute (Crossing No. 3). Therefore, only the level 1 analysis was performed at this crossing. Calculations for the analyses of Crossing Nos. 1, 2, 2a, and 3 are included in **Appendices B, C, G and D**, respectively. Various parameters estimated in order to complete the analyses are described in **Section 3**.

2.3 Method at Sonoran Wash (Crossing No. 4)

In 2001, the Flood Control District of Maricopa County completed the Skunk Creek Watercourse Master Plan. *Attachment 6, Lateral Stability Analysis Report* **(Reference**

7) of this document included scour data for the Sonoran Wash equivalent to a level 2 analysis. The results of this analysis were used to determine the scour potential at the force main Crossing of the Sonoran Wash (Crossing No. 4). Relevant portions of the Skunk Creek WMP were reproduced and included in **Appendix E**. For the scour analysis, the Skunk Creek WMP divided the Sonoran Wash into six different reaches. Crossing No. 4 is located in Reach 2, at approximately cross-section 1.33.

SECTION 3: PARAMETER ESTIMATION

3.1 Design Flows

3.1.1 Flow at the Cave Creek Tributary Wash (Crossing No. 1)

The contributing area to Crossing No. 1 was based on data obtained from the 1993 *Cave Creek Watershed, Vol. 1.7 Arizona Canal Diversion Channel Area Drainage Master Study ACDC/ADMS Phase 1, Hydrology Report* (**Reference 1**). This study does not have a concentration point at a location where the flows could be obtained directly. In order to estimate the 100-year flow, Entellus delineated the contributing area to this crossing (see **Appendix B**), and used a best-fit curve generated from results obtained from the Cave Creek ADMS results to estimate the flow at the crossing. The following are the results of these estimates:

- Area = 0.545 sq. miles
- Flow = 440 cfs

3.1.2 Flow at the Cave Creek Wash (Crossing No. 2)

The flow at Crossing No. 2 was obtained from Sheet 11of 11 of the 1991 *Middle Cave Creek Floodplain Delineation Study* (**Reference 2**). This document will be referred to as the *Cave Creek FDS*. The *Cave Creek FDS* sheet 11 shows a summary of flows for reaches along the watercourse. Crossing No. 2 falls into the reach identified as "Cave Creek above the Central Arizona Project Canal." The crossing is located approximately at the cross section labeled 26.784. The information from the *Cave Creek FDS* sheet 11 shows that the estimated 100-year peak flow at Crossing No. 2 is 2,900 cfs. A copy of the *Cave Creek FDS* sheet 11 is included in **Appendix C**.

3.1.3 Flow at the Emergency Spillway Wash (Crossing No. 2a)

The flow at Crossing No. 2a was developed using the rational method, as it is described in the *Draft Hydrology Manual* **(Reference 5)**. The flow developed represents the 100-year flow, and therefore does not include flows from the emergency spillway. The development of the peak flow has been documented in **Appendix G**. The peak flow is unusually high for the amount of contributing area. This is because the contributing area has a very steep slope (over 5% grade). The following are the results of these estimates:

- Area = 0.22 sq. miles
- Flow = 490 cfs

3.1.4 Flow at the CAP Overchute Wash (Crossing No. 3)

No previous studies were found that could be used to estimate the flow at Crossing No. 3. Since no data was available, the capacity of the overchute was used to estimate the scour depth. Typically, CAP overchutes are designed to pass the 50-year storm event, and the impoundment area is designed for the 100-year storm event. Using the capacity of the CAP overchute at full impoundment conditions (water surface at top of embankment) is a conservative assumption since under these conditions the crossing would be within the impoundment area. The size and configuration of the overchute were measured in the field. The capacity of the CAP overchute was estimated using the FHA Inlet Control Monogram. The flow used to estimate scour depth for this crossing is 100 cfs. The overchute modeling parameters and capacity analysis are included in **Appendix D**.

3.1.5 Flow at the Sonoran Wash (Crossing No. 4)

The flow at crossing No. 4 corresponds to concentration point C010 of the Sonoran Wash HEC-1 models from the Skunk Creek WMP. Table 3-3-16 from *Attachment 3: Hydrology* **(Reference 3)**, of the *Skunk Creek WMP* summarizes the flows. Relevant copies of this report are included in **Appendix E**. Based on the data obtained from this report, the Sonoran Wash flow at the force main crossing is 9,800 cfs.

3.2 Bed Material Size Distribution

3.2.1 Bed Material at Crossing Nos. 1, 2, and 2a

DSWA provided a copy of the grain size distribution analysis performed by AMEC at Cave Creek Wash near the crossing of the force main. This analysis was included as part of the *Geotechnical Investigative Report by AMEC* **(Reference 9)**, hereinafter referred to as the *Geotechnical Report.* No additional particle size information was provided. However, from field observations it appears that this gradation is typical of the washes in the area. Therefore, it has been assumed that the gradation curve from AMEC is representative of the washes near the Cave Creek Wash, and it has been used for the scour analysis at Crossing Nos. 1, 2, and 2a. Copies of the gradation curve are included in **Appendices B, C, and G**.

3.2.2 Bed Material at the CAP Overchute Wash (Crossing No. 3)

The bed material size distribution was not available for Crossing No. 3. However, the channel material was observed in the field and consists of cobbles and boulders.

3.2.3 Bed Material at the Sonoran Wash (Crossing No. 4)

Table 5-2 from **Reference 7** summarizes sediment sampling results for the various reaches of Sonoran Wash. Crossing No. 4 is within Reach 2. This information was used in the Skunk Creek WMP to determine the armoring and scour potential. A copy of the table has been included in **Appendix E**.

3.2.4 Bedrock Depth

The *Geotechnical Report* documents estimates of the bedrock depth at various stations along the force main alignment. DSWA used the estimates to develop a profile of the bedrock surface, which was included on the plan & profile sheets for the force main. This information was used for the further analysis described in **Section 4.2**.

3.3 Channel Geometry

3.3.1 Channel Geometry at the Cave Creek Tributary Wash (Crossing No. 1)

The wash geometry and slope at Crossing No. 1 were obtained from field surveys provided by DSWA. Plots of these cross sections are included in **Appendix B**.

3.3.2 Channel Geometry at the Cave Creek Wash (Crossing No. 2)

The wash geometry and slope at Crossing No. 2 were determined using the HEC-2 model cross sections from the *Cave Creek FDS* and surveyed cross sections provided by DSWA. Plots of these cross sections are included in **Appendix C**.

3.3.3 Channel Geometry at the Emergency Spillway Wash (Crossing No. 2a)

The wash geometry and slope Crossing No. 2a were determined using contours from Sheet 11of 11 of the *Cave Creek FDS*. A portion of the *Cave Creek FDS* sheet 11, along with documentation of the geometry and slope development have been included in **Appendix G**.

3.3.4 Channel Geometry at the CAP Overchute Wash (Crossing No. 3)

The wash geometry at Crossing No. 3 was not available. However, the wash was observed during a field investigation. An approximate sketch of the wash geometry is included in **Appendix D**.

3.3.5 Channel Geometry at the Sonoran Wash (Crossing No. 4)

The geometry of the Sonoran Wash is included in the Skunk Creek WCMP. The geometry was plotted and is included in **APPENDIX E**.

SECTION 4: RESULTS

4.1 Level 1 and 2 Analysis Results

4.1.1 Cave Creek Tributary Wash (Crossing No. 1)

The results of the scour analysis at Crossing No. 1 show that the wash is very well armored, and significant scour degradation is not likely. This is common in washes that historically conveyed large amount of flows, but presently carry much smaller amounts. Such is the case at this wash crossing because several dams and levees constructed upstream have significantly reduced flows from their historic amounts. The scour estimates for Crossing No. 1 are documented in **Appendix B**.

4.1.2 Cave Creek Wash (Crossing No. 2)

The level 1 and 2 analyses of the scour at Crossing No. 2 show that the wash armors itself after about 1/3 feet of degradation, and significant scour is not likely. Once again, this is common in washes that historically conveyed large amount of flows, but presently carry much smaller amounts. Such is the case at this wash crossing because it is located directly downstream of the Cave Buttes Dam. However, the conditions at the crossing warranted the use of further analysis techniques that are described in **Section 4.2**. The scour estimates for Crossing No. 2 are documented in **Appendix C**.

4.1.3 Emergency Spillway Wash (Crossing No. 2a)

The results of the levels 1 and 2 analyses show that the scour depth at Crossing No. 2a will be approximately 1.5 feet. The 100-year water velocity is very high at the crossing, which would typically cause deeper scouring. However, the large particle sizes of the bed material cause the wash to armor itself, thus limiting the scour depth to 1.5 feet. The scour estimates for

Crossing No. 2a are documented in **Appendix G**.

Part of the scour analysis for Crossing No. 2a also included the examination of the bedrock depth. The *Geotechnical Report* was used to determine the bedrock depth at various locations along the force main alignment. Review of this data revealed that the depth to bedrock at Crossing No. 2a varied between only 4 feet and 5 feet. The plans provided by DSWA show the force main below the bedrock at Crossing No. 2a (see **Appendix G** for a copy of the plan & profile sheet from the DSWA plans). Because significant erosion of the bedrock is not likely, the wash crossing will not scour below the bedrock depth.

4.1.4 CAP Overchute Wash (Crossing No. 3)

The results of the level 1 scour analysis at Crossing No. 3 indicate a scour depth of approximately 1.3 feet. There was not enough data available at the time of this analysis to perform a level 2 analysis. Based on the bed material observed in the field, it is likely that the level 1 scour analysis yields an overestimation of this wash's actual scour potential. The scour estimates for Crossing No. 3 are documented in **Appendix D**.

4.1.5 Sonoran Wash (Crossing No. 4)

The results of the scour analysis at Crossing No. 4 predict that the general wash scour depth will be 3.8 feet. This estimate does not include long-term scour. Using the slope equilibrium data supplied in the Skunk Creek WMP, the long-term degradation has been estimated to be approximately 1 foot. Therefore, the total scour at Crossing No. 4 is approximately 5 feet. Details of this analysis are included in **Appendix E**.

4.2 Further Analysis of Crossing No. 2

The level 1 and 2 analyses give a general estimate of what the scour depths could be.

In some cases, it is necessary to carry out a level 3 analysis in order to determine the scour depths with more accuracy. The Cave Creek Wash Crossing No. 2 is downstream from the Cave Buttes Dam principle spillway. At times, the spillway has carried flow for several days. This flow is likely to carry little or no sediment at all, and thus the scour conditions at the crossing are likely to be "clear-water." The analyses originally performed at Crossing No. 2 may not adequately apply to "clear-water" sustained flow. Therefore, further analysis was needed in order to determine the scour depth at Crossing No. 2.

Various steps of a level 3 analysis were started for the Cave Creek Wash crossing. These steps included the examination of historical data, such as precipitation and stage gages. Another step of the level 3 analysis is to develop a sediment transport model. In order to develop the model, the HEC-2 model from the *Cave Creek FDS* was imported into HEC-RAS, and modified in order to model the crossing. However, before the computer modeling had been completed, the *Geotechnical Report* was provided by DSWA. In this report, the depth to bedrock was determined at various locations along the force main alignment. Review of this data revealed that the depth to bedrock at Crossing No. 2 varied between only 3 feet and 5 feet. The plans provided by DSWA show the force main below the bedrock at Crossing No. 2 (see **Appendix C** for a copy of the plan & profile sheet from the DSWA plans). Because significant erosion of the bedrock is not likely, the wash crossings will not scour below the bedrock depth. At Crossing No. 2, it is likely that the sustained discharge from the principle spillway will erode the entire sediment layer down to the bedrock. Therefore, the scour depth at Crossing No. 2 has been assumed to be the bedrock depth.

4.3 Summary of Results

Table 1 summarizes the results and some of the parameters used to estimate scour depths.

Table 1: Scour Parameters and Results

Crossing No.	Crossing Description	Drainage Area (sq miles)	Scour Flow (cfs)	Level of Analysis Performed	Flow Source	Estimated Scour Depth (ft)
1	Cave Creek Tributary	0.5	440	1, 2	Middle Cave Creek FPD (modified)	Negligible
2	Cave Creek Wash	2.5	2900	1, 2, 3(Partial)	Middle Cave Creek FPD	Bedrock (3.0 - 5.0)
2a	Emergency Spillway Wash	0.2	490	2	Rational Method	1.5
3	CAP Overchute	N/A	100	1	Overchute Capacity	1.3
4	Sonoran Wash	13.4	9,825	2 equivalent	Sonoran Wash FPD	5.0

APPENDIX A. REFERENCES

List of References

1 *Cave Creek Watershed, Vol. 1.7 Arizona Canal Diversion Channel Area Drainage Master Study ACDC/ADMS Phase 1, Hydrology Report*, Kaminski-Hubbard Eng., Inc. 1993.

2 *Middle Cave Creek Floodplain Delineation Study*, Burgess & Niple, Inc. 1991.

3 *Skunk Creek Watercourse Master Plan, Attachment 3, Hydrology*, Tetra Tech, Inc. 2001.

4 *Figure 1 - Force Main Alignment North Gateway Water Reclamation Plant Phase 1 – Residual Pump Station and Force Main Project No. WS85090003*, Damon S. Williams Associates, LLC, 2004.

5 *Draft Hydrology Design Manual for Maricopa County, Arizona, Volume-1 Hydrology*, Flood Control District of Maricopa County, 2003.

6 *Arizona State Standards for floodplain Management, State Standards Attachment 5-96* (SSA 5-96), Arizona Department of Water Resources, 1996.

7 *Skunk Creek Watercourse Master Plan, Attachment 6, Lateral Stability Assessment Report*, JE Fuller/Hydrology and Geomorphology, Inc. 2001.

8 *Floodplain and Floodway Delineation for Sonoran Wash*, Tetra Tech, Inc. and Stantech Consult., Inc. 2001.

9 *Geotechnical Investigation Report, North Gateway Water Reclamation Plant, Residual Pump Station & Force Main Design Services*, AMEC Earth and Environmental, December 17th, 2003.

10 *AFMA Scour Short Course, Scour Analysis for Small to Mid-Size Desert Washes*, Arizona Floodplain Management Association, 1/27/2004

11 *North Gateway Water Reclamation Plant Phase 1 - Residuals Pump Station and Force Mains, Segment 5 - Salt River Project Easement Corridor - Plan & Profile, STA 64+50 to STA 69+50*, Damon S. Williams Associates, L.L.C., July 21st. 2004

APPENDIX B. CAVE CREEK TRIBUTARY, CROSSING NO. 1.

- Level 1 and 2 Analysis
- Copy of Plate 12, ACDC /ADMP Phase 1, Cave Creek Hydrology Study
- Flow vs. Area for ACDC Study 100-year 6-hr Output
- Contributing Area Estimates
- Cross-Section Plots of Survey Data
- Grain-size Distribution of Cave Creek Wash
- Copies of SSA 5-96 Relevant Pages

FORCE MAIN CROSSING #1 @ CAVE CREEK TRIBUTARY

LEVEL 1: SCOUR DEPTH ESTIMATION

Area = 0.545 Sq. Miles (Sheet 5 of 11)
Q_{100} = 440 cfs (from Flow vs Area table - ~~ADOT~~ (Sheet 4 of 11)

General Degradation, $d_{gs} = 0.157(440)^{.4} = 1.8$ feet — SSA 5-96
Long Term Degradation, $d_{lt} = .02(440)^{.6} = .8$ feet — SSA 5-96
Total Scour = 2.6 feet — SSA 5-96

LEVEL 2: RESISTANCE TO DEGRADATION

Erodibility Evaluation

Allowable Velocity Approach

The estimation of the basic velocity is identical to that of Crossing #2, because it has been assumed that both washes have the same bed material distribution.

Therefore, the basic velocity @ crossing #1 = 12.8 fps ✓

(see Appendix C for calculations)

Correction Factors:

Curve radius is negligible, $C_o = 1$ ✓
H:V Estimated bank slope from cross section = $\frac{30}{3} = 10$ ✓
From Figure 3 of Reference 6, $C_b = .82$ ✓ (Sheet 10 of 11)
From Normal Depth Calculations, Depth = 2.26 ft ✓ (Sheet 6 of 11)
From Figure 4 of Reference 6, $C_d = .95$ ✓ (Sheet 11 of 11)
Maximum Allowable Velocity = (Basic Velocity)(C_o)(C_b)(C_d)
= (12.8)(1)(.82)(.95) = 10 fps
Velocity from normal depth = 3.1 fps < 10 fps

∴ Channel is not erosive using the allowable velocity approach. ✓

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JOB NAME Force Main Scour Analysis JOB NO. 910005

FORCE MAIN CROSSING #1 @ CAVE CREEK TRIBUTARY CONTINUED

— LEVEL 2 ANALYSIS CONTINUED —

Armoring Potential Evaluation

R = Hydraulic Radius = 1.0 ft, from normal depth — sheet 6 of 11

D_{90} = 240 mm, from Reference 9 — sheet 8 of 11

Channel Area = 141 ft², from normal depth — sheet 6 of 11

Wetted Perimeter = 142 ft, from normal depth — sheet 6 of 11

V = Velocity = 3.10 fps, from normal depth — sheet 6 of 11

ρ = density of water = 1.94 slugs/ft³ — SSA 5-96

γ_s = specific weight of sediment = 165 lb/ft³ — SSA 5-96

γ = specific weight of water = 62.4 lb/ft³ — SSA 5-96

Manning's n related to particle roughness:

$n = (D_{90}/1000)^{1/6}/26 = (240/1000)^{1/6}/26 = .030$ — SSA-5-96

Friction factor $f = 116.5 n^2/R^{1/3} = 116.5(.03)^2/(1)^{1/3} = .105$ — SSA-5-96

Particle Sheer Stress: $\tau_p = 1/8 f \rho V^2$

$\tau_p = 1/8(.105)(1.94)(3.10)^2 = .24$ lb/ft² — SSA 5-96

Critical Particle Size $D_c = \tau_p/[.047(\gamma_s - \gamma)]$

$D_c = .24/[.047(165-62.4)] = .05$ ft $= 15$ mm

D_c = 15 mm, from Reference 9, 98% of the particles are larger than D_c, $P_c = .98$ — Sheet 8 of 11

Armor thickness $= 2D_c = 30$ mm $= .1$ ft $= y_a$ — SSA 5-96

Depth of Degradation required for armoring to form:

$\Delta Z_a = y_a[(1/P_c) - 1]$ — SSA 5-96

$\Delta Z_a = .1[(1/.98) - 1] = .002$ ft

∴ Channel will armor itself with almost no degradation.

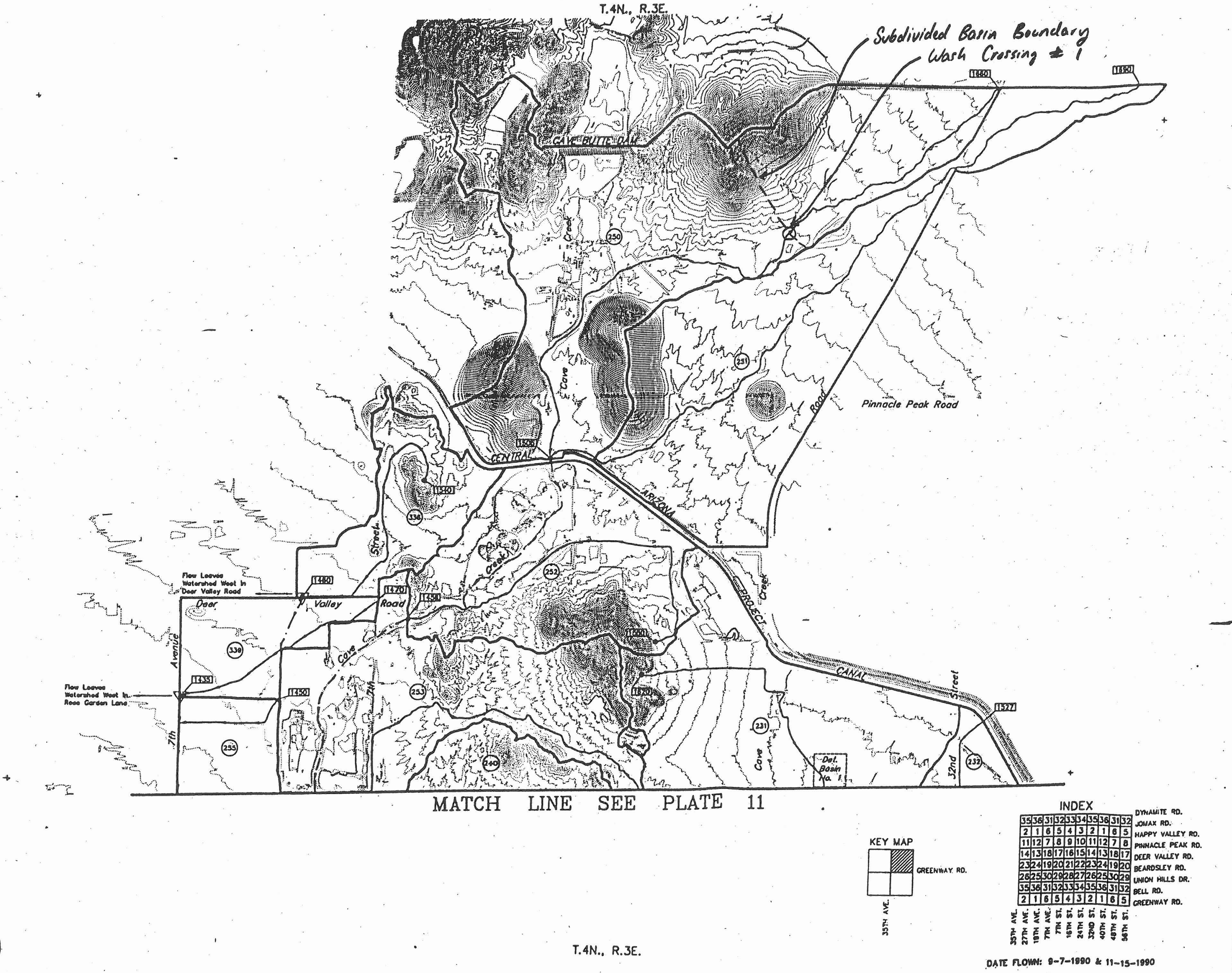
— CONCLUSION —

The force main at crossing #1 should ~~be buried~~ have ~~at~~ the minimum cover depth allowed - There is negligible scour.

Sheet 3 of 11

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

ACDC/ADMS PHASE 1 CAVE CREEK HYDROLOGY STUDY



LEGEND

- Drainage Basin Boundary
- (140) Drainage Sub-Basin Number
- Major Drainage Basin Concentration Point
- Drainage Sub-Basin Concentration Point
- ▽ Flow Diversion Point
- Routing Flow Path
- Length of Longest Watercourse
- [1514] Elevation Along Flow Path

FLOW ROUTING MAP
EXISTING CONDITIONS

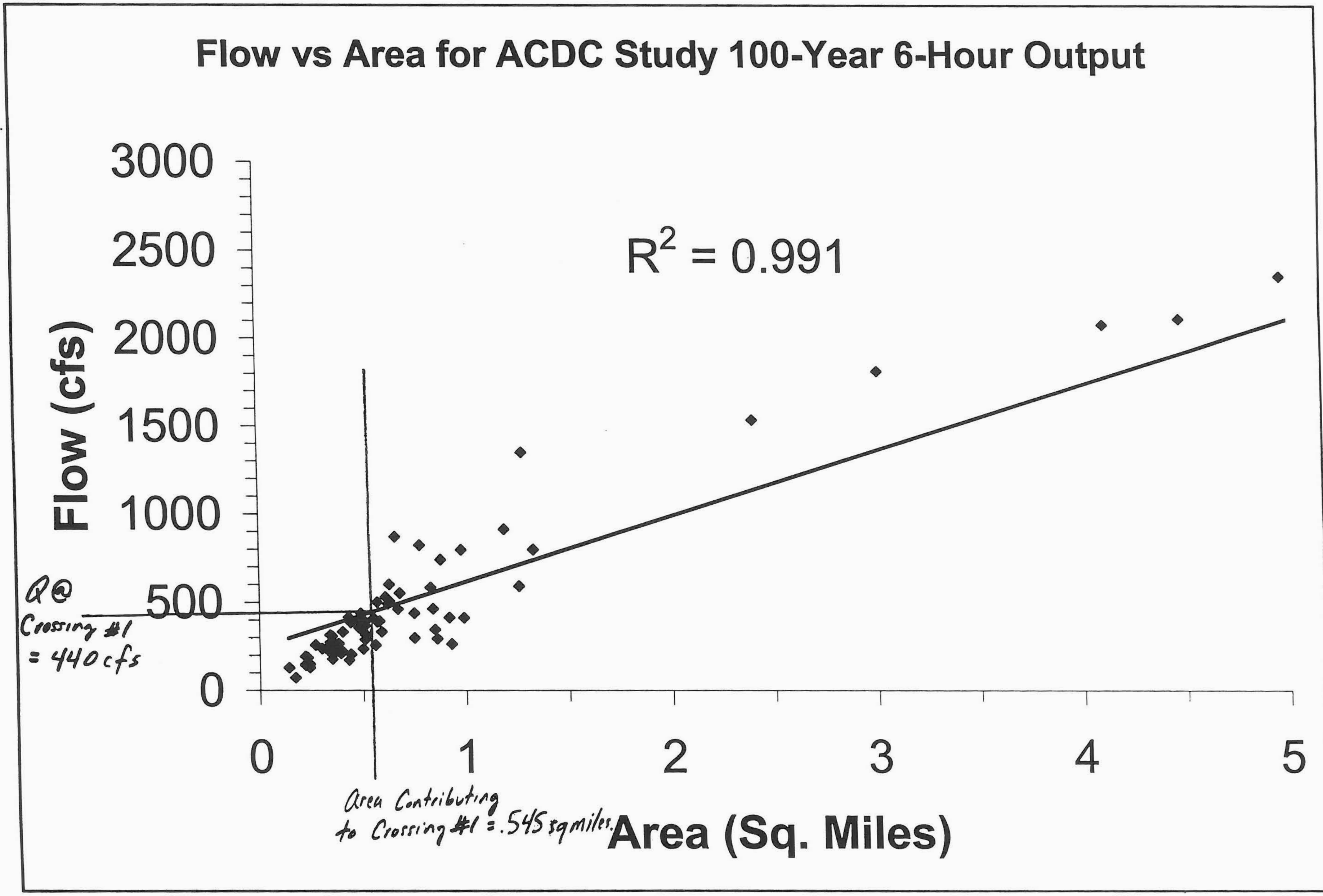
PLATE 12
MAY 21, 1993

KAMINSKI HUBBARD engineering, inc.
SURVEYING • CIVIL • HYDROLOGY
4550 N. BLACK CANYON HWY., SUITE C
PHOENIX, ARIZONA 85017
(602) 242-5588

N

SCALE
1200 600 0 1200 2400
FEET

CONTOUR INTERVAL 10 FEET

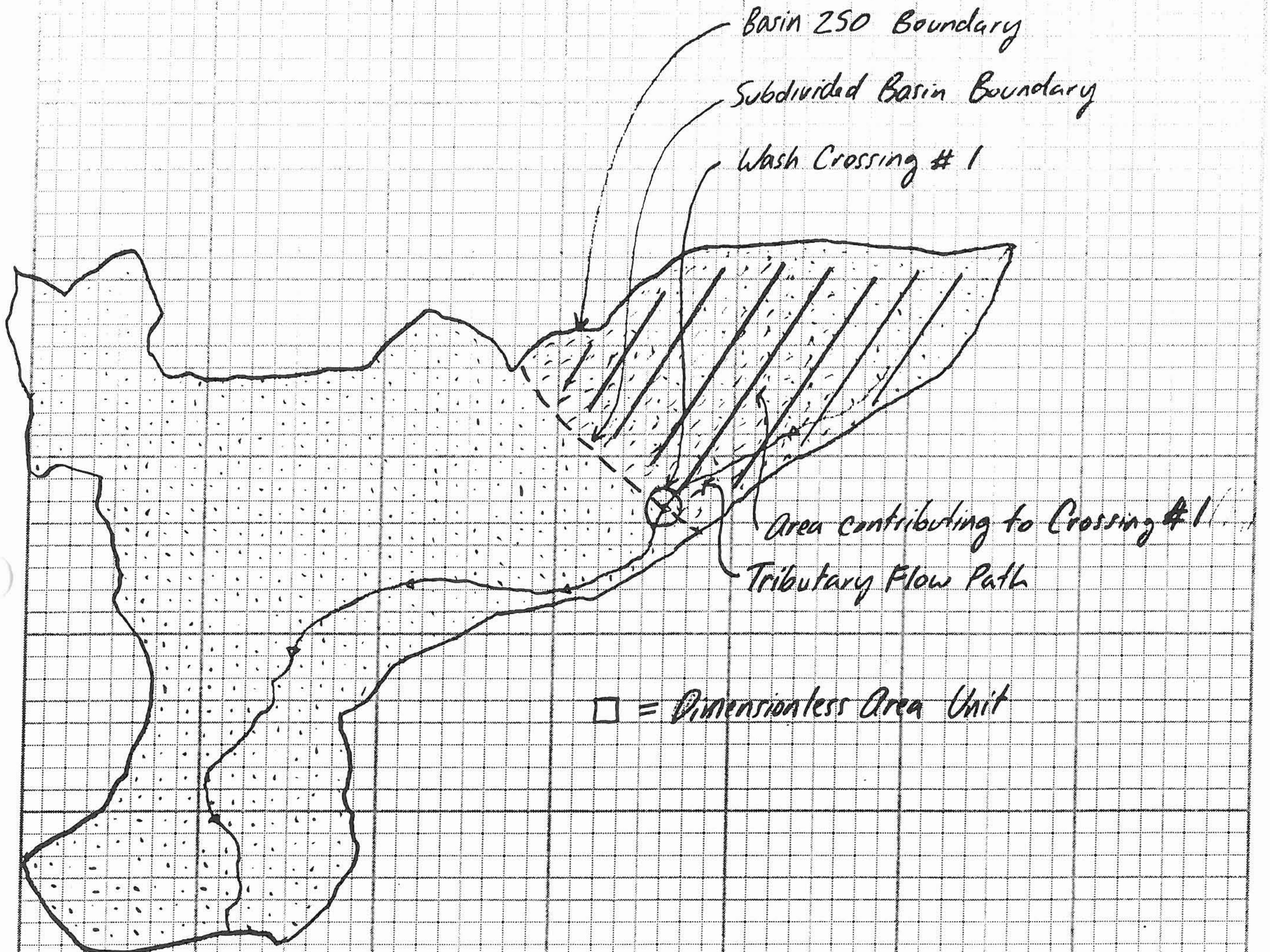


By: JS Date: 4/20/14
Checked: Date:
Sheet 4 of 11

CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 910005

DETERMINATION OF AREA CONTRIBUTING TO CROSSING # 1

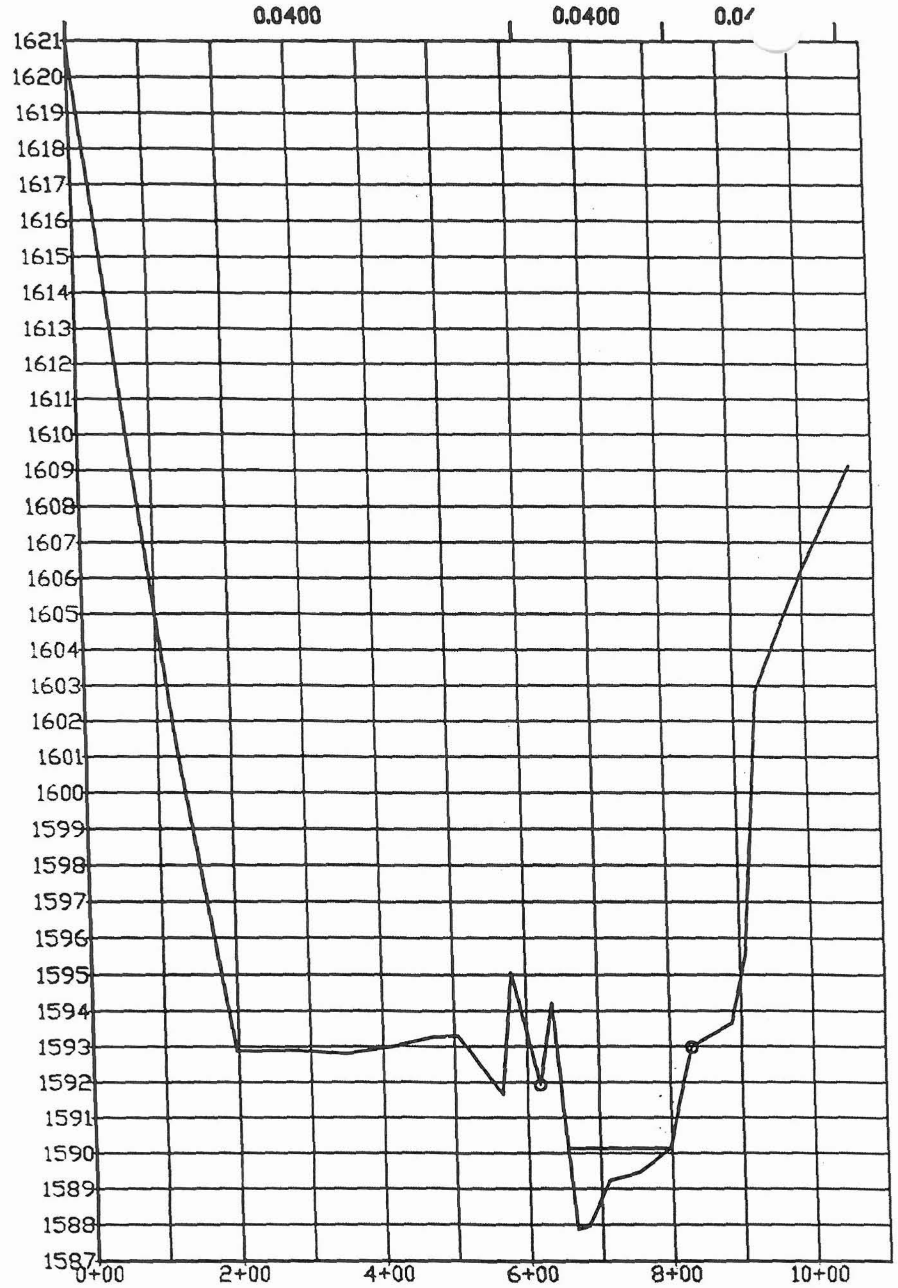


The original basin 250 has an area of 835 □

The subdivided area has an area of 190 □

Area basin 250 = 2.396 sq. miles (from ACDC Study HEC-1)

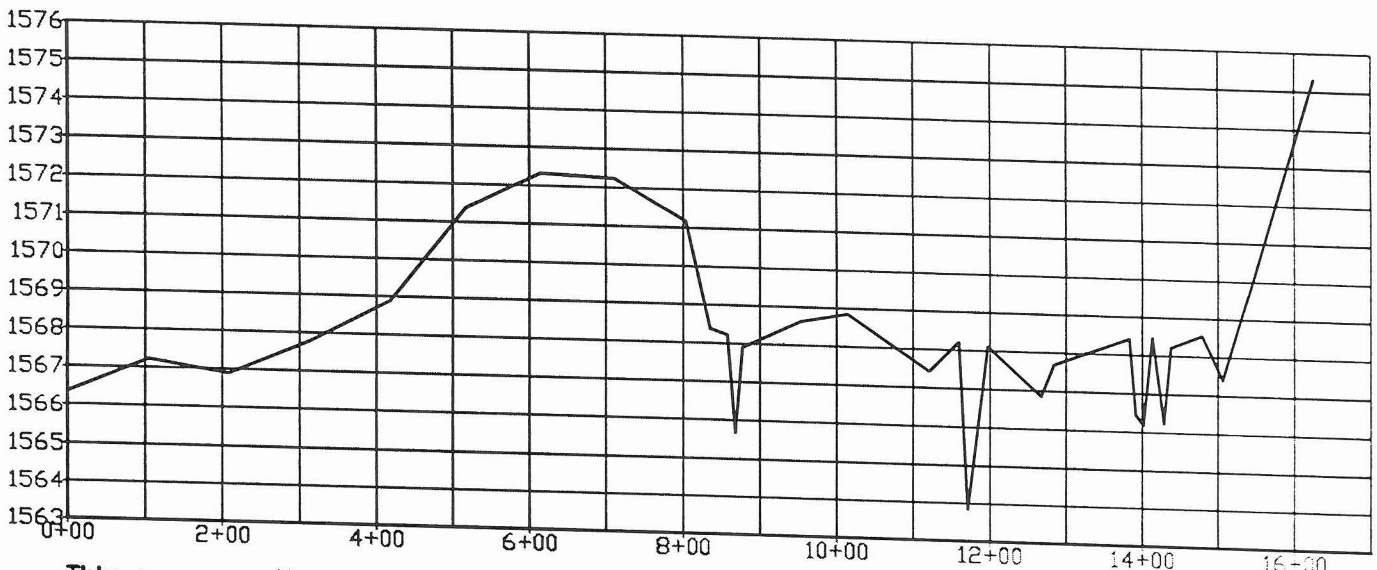
Area of the subdivided basin $= \frac{190}{835}(2.396) = .545$ sq miles



Normal Depth Results

Cross-Section:	2	
Elevation:	1590.13	ft MSL
Depth:	2.26	ft
Discharge:	440.00	cfs
Energy Gradient:	0.007	ft/ft
Froude Number:	0.3661	
Flow Regime:	Subcritical	
Flow Area:	140.89	sq ft
Average Velocity:	3.10	ft/s
Maximum Velocity:	3.10	ft/s
Composite n:	0.04	
Hydraulic Radius:	0.99	ft
Wetted Perimeter:	141.86	ft
Wetted Top Width:	141.58	ft
Critical Slope:	0.0529	ft/ft

This cross section was generated using survey data obtained from DSWA.
The cross section is approximately at Crossing #1.
The invert of this section is approximately at 1588 feet.
The slope was estimated using the downstream surveyed cross section.
Slope = (1588 - 1567) / 3000 = .007 ft/ft

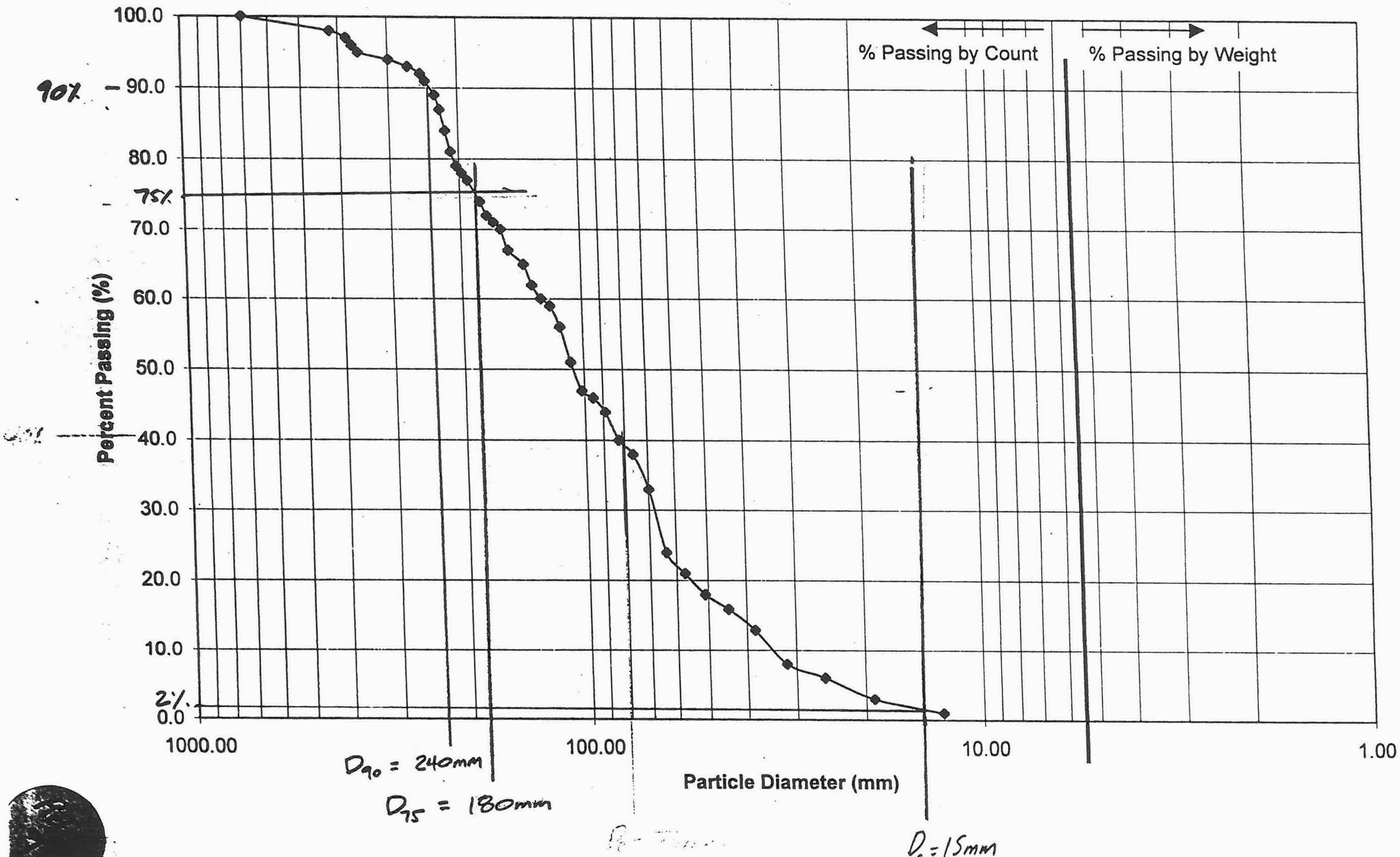


This cross section was generated using survey data obtained from DSWA.
The cross section is approximately 3000ft downstream from Crossing #1.
The invert of this section was assumed to be at 1567 feet.

Sheet 7 of 11

amec

Idealized Grain-size Distribution - Cave Creek Wash



Crossing #1 Bed Material

Gradation Analysis Crossing # 1 Bed material

amec

PROJECT:	NORTH GATEWAY WATER RECLAMATION	JOB NO:	3-117-001074
LOCATION:	HAPPY VALLEY ROAD AND 19TH AVENUE	WORK ORDER NO:	1
MATERIAL:	SOIL	LAB NO:	1
SAMPLE SOURCE:	SURFACE SAMPLE CAVE CREEK WASH	DATE SAMPLED:	12/08/2003

SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES (ASTM C136/C117)

MECHANICAL ANALYSIS

SIEVE SIZE	% PASSING
6 in / 152mm	100
4 in / 100mm	100
3 in / 75mm	100
2 in / 50mm	94
1 1/2 in / 37.5mm	83
1 1/4 in / 32 mm	77
1 in / 25 mm	69
3/4 in / 19 mm	59
1/2 in / 12.5 mm	47
3/8 in / 9.5 mm	42
1/4 in / 6.4 mm	34
#4, 4.75mm	29
#8, 2.36mm	19
#10, 2.00mm	16
#16, 1.18mm	11
#30, 0.60mm	7
#40, .425mm	6
#50, .300mm	6
#100, .150mm	5
#200, .075mm	5.0

NOTES:

Reviewed by: CW

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3232 W Virginia Avenue
Phoenix, Arizona 85009
(602) 272-6848

Crossing # 1

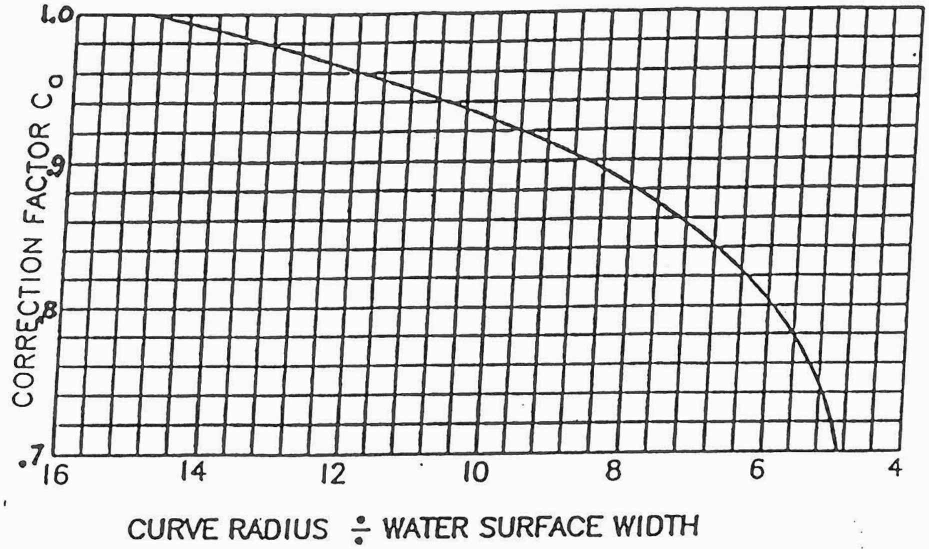

FIGURE 2

CORRECTION FACTOR C_a FOR CHANNEL ALIGNMENT

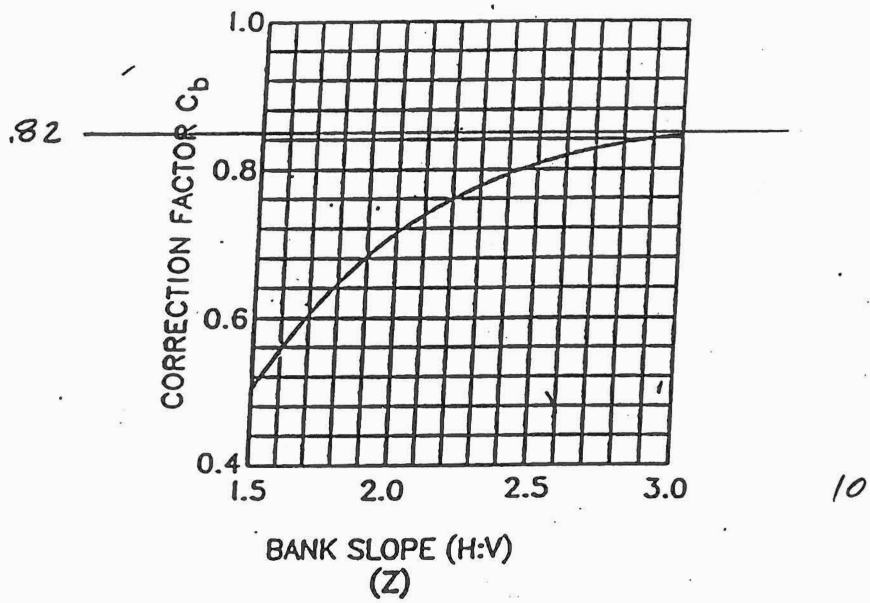

FIGURE 3

CORRECTION FACTOR C_b FOR BANK SLOPE

Crossing # 2

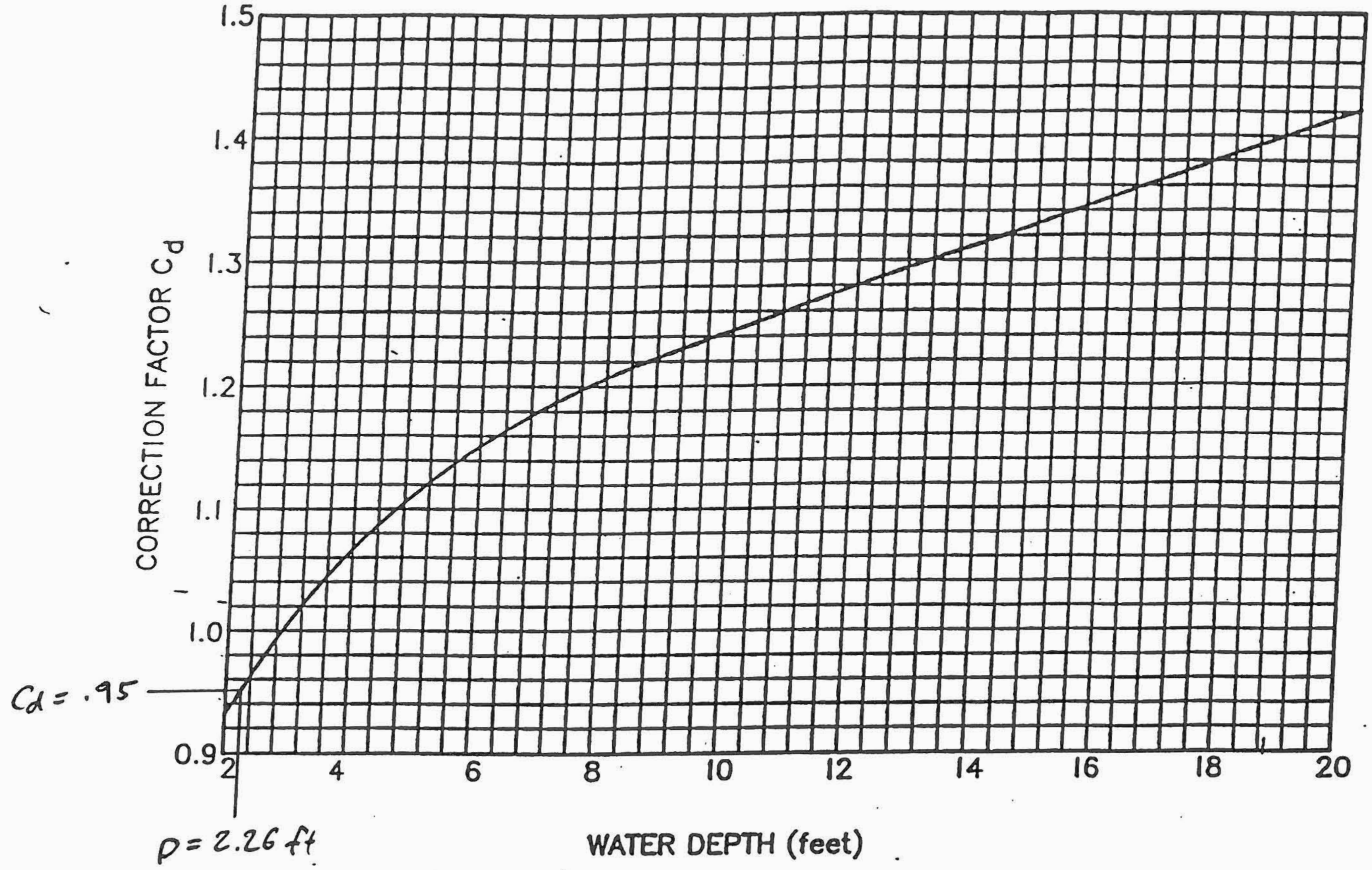

FIGURE 4

CORRECTION FACTOR C_d FOR DEPTH OF FLOW

APPENDIX C. CAVE CREEK WASH, CROSSING NO. 2

- Level 1 and 2 Analysis
- Cross Sections from Middle Cave Creek Floodplain Delineation
- Cross Section from Survey Data
- Grain-size Distribution of Cave Creek Wash
- Copies of SSA 5-96 Relevant Pages
- *Cave Creek FDS* Sheet 11(In Back Map Pocket)
- Plan Profile Sheet Showing Bedrock Profile at Crossing No. 2 (From DSWA)

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SHEET 1 OF 12
BY JS DATE 4/22/4
CHECK [initials] DATE 4/22/04

CLIENT OSWA

JOB NAME Force Main Scour Analyses JOB NO. 910005

FORCE MAIN CROSSING #2 @ CAVE CREEK WASH

LEVEL 1: SCOUR DEPTH ESTIMATION

Q_{100} = 2,900cfs (From Cave Creek Floodplain Delineation, Reference 2) Sheet 12A of 12

General Degradation, $d_{gs} = 0.157(2,900)^{.4}$ = 3.8 feet ✓
(straight channel)

Long Term Degradation, $d_{lts} = .02(2,900)^{.6}$ = 2.4 feet ✓

Total Scour = 6.2 feet ✓

LEVEL 2: RESISTANCE TO DEGRADATION

Erodibility Evaluation

Allowable velocity approach

D_{75} = 180 mm, from gradation curve for Cave Creek Wash (Reference 9) Sheets 9-11

From Figure 1 of State Standard (Reference 6), Sheet 10 of 12
Assuming Sediment Laden Flow,
Basic Velocity = 12.8 fps ✓

Correction Factors:
Curve Radius is negligible.
H:V Estimated Bank Slope from XS = $\left[\frac{(1551-1543)2}{10}\right]^{-1}$ = 7 ✓

From Figure 3 of State Standard (Reference 6), Sheet 11 of 12
Correction Factor C_b for bank slope = .82 ✓

From Normal Depth Calculations, Depth = 6.15 ft ✓ Sheet 4 of 12

From Figure 4 of State Standard (Reference 6), Sheet 12 of 12
Correction Factor Cd for depth of flow = 1.15 ✓

Maximum Velocity for no degradation = (Basic Velocity)(C_c)(C_b)(Cd)

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SHEET 2 OF 12
BY dS DATE 4/22/4
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CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 910005

FORCE MAIN CROSSING #2 @ CAVE CREEK WASH CONTINUED

Maximum Allowable Velocity = (12.8)(1)(.82)(1.15) = 12.1 fps ✓

Velocity from normal depth = 8.52 fps < 12.1 fps Sheet 4 of 12

∴ Channel is not erosive using the allowable velocity approach.

Armoring Potential Evaluation

R = Hydraulic Radius = 3.41 ft Sheet 4

D_{90} = 240 mm, from Reference 9 Sheet 8

Channel Area = 340 sq ft, from normal depth Sheet 4

Channel Wetted Perimeter = 99.7 ft, from normal depth sheet 4

Manning's n related to particle roughness

$n = (D_{90}/1000)^{1/6}/26 = (240/1000)^{1/6}/26 = .030$ SSA 5-96

Friction factor $f = 116.5\, n^2/R^{1/3} = 116.5(.03)^2/(3.14)^{1/3} = .072$ SSA 5-96

Particle Shear Stress: $\tau_p = 1/8\, f \rho V^2$ SSA 5-96

Where ρ = density of water = 1.94 slugs/ft³ SSA 5-96

V = Velocity = 8.52 fps sheet 4

$\tau_p = 1/8(.072)(1.94)(8.52)^2 = 1.27\ lb/ft^2$ SAA 5-96

Critical Particle Size: $D_c = \tau_p/[.047(\gamma_s - \gamma)]$ SAA 5-96

Where γ_s = specific weight of sediment

γ = specific weight of water

$D_c = 1.27/[.047(165-62.4)] = .26$ feet = 79 mm ✓

D_c = 79 mm, from Reference 9, 60% of the bed material is > D_c sheet 8

Armor thickness = $y_a = 2D_c$ = 158 mm = .52 ft. ✓

Depth of Degradation required for armoring to form:

$\Delta Z_a = y_a[(1/P_c) - 1]$

Where P_c is the decimal percentage of bed material that is larger than the critical particle size.

$\Delta Z_a = .52[(1/.60) - 1] = .35$ ft. ✓

CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 910005

FORCE MAIN CROSSING #2 @ CAVE CREEK WASH CONTINUED

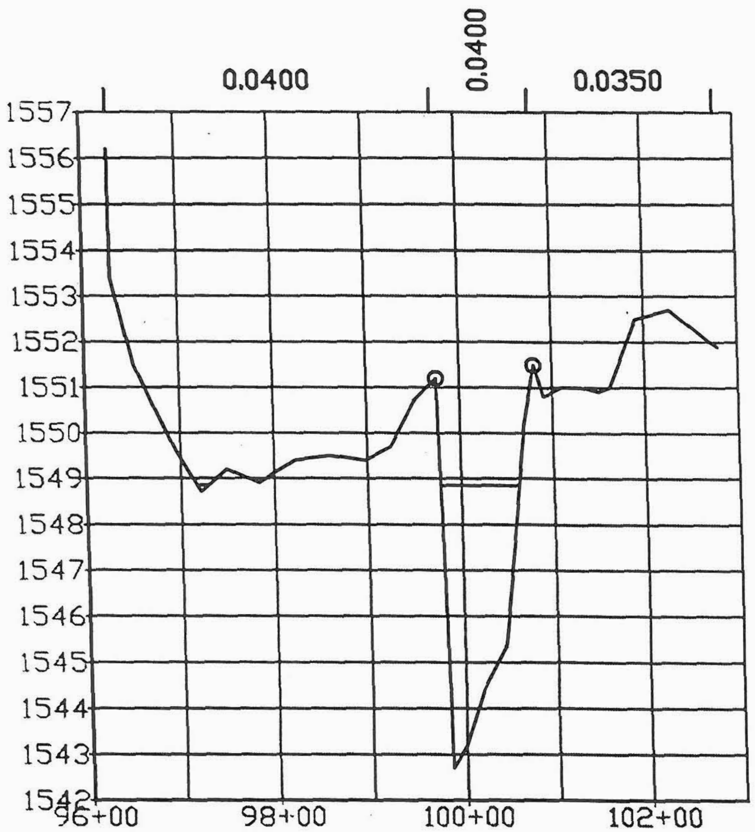
ΔZ_a = depth to armor = .35 ft.

∴ Channel will armor itself, and level 1 estimates of scour can be discarded.

CONCLUSION

The force main at crossing #2 should be buried at the minimum depth allowed + the armoring depth.

~~Burial Depth~~ = 3' (minimum allowed) + .35 (armoring depth)
Minimum Cover = 3.4 ft

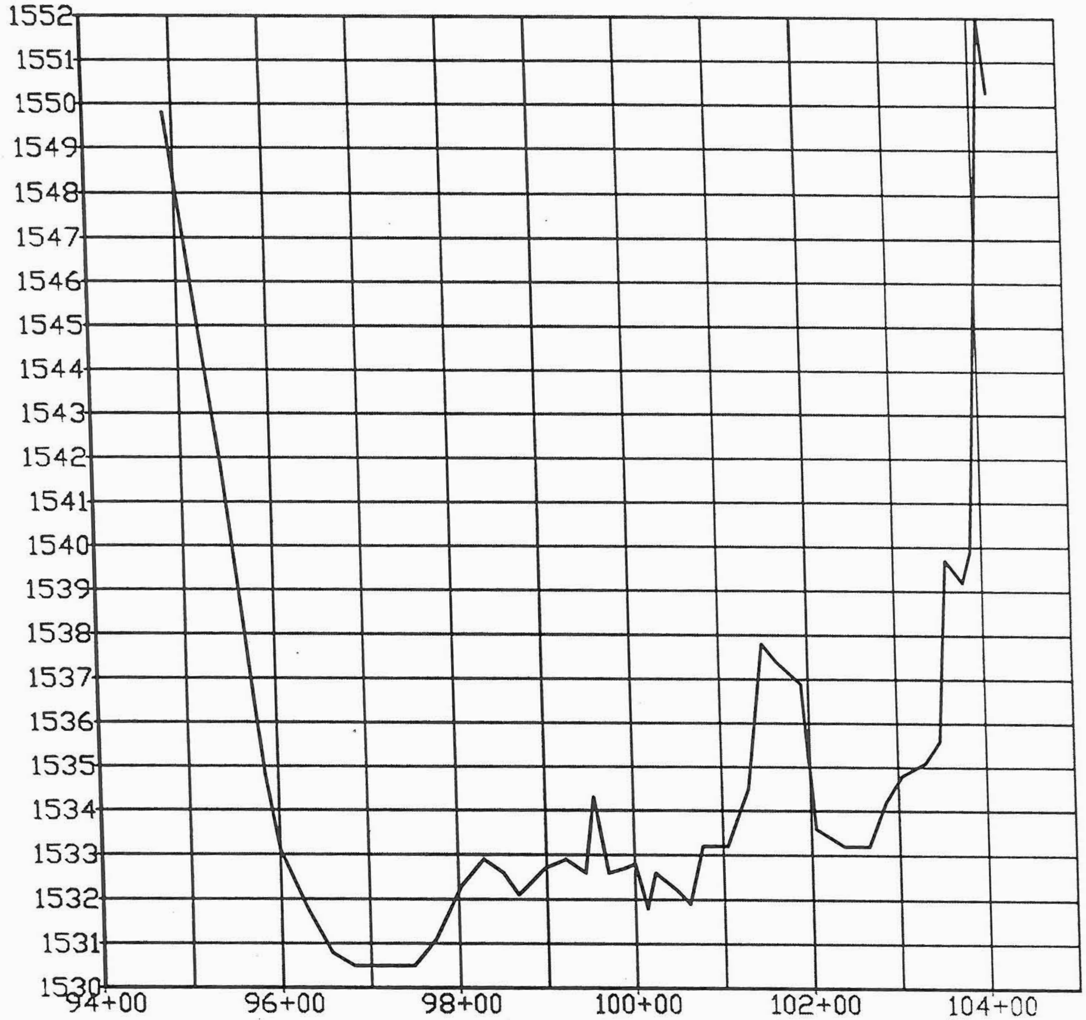


Normal Depth Results

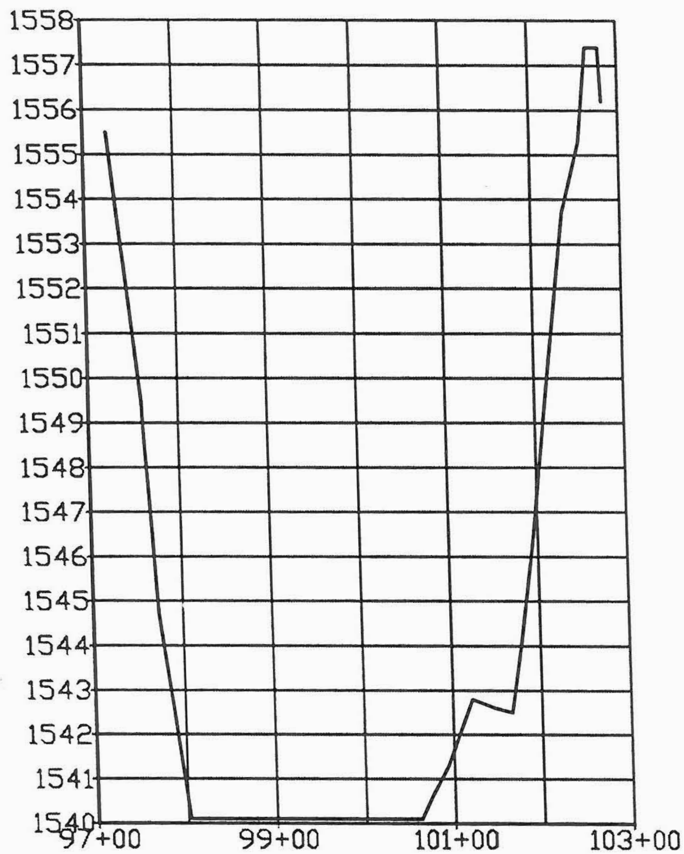
Cross-Section:	5	
Elevation:	1548.85	ft MSL
Depth:	6.15	ft
Discharge:	2900.00	cfs
Energy Gradient:	0.0086	ft/ft
Froude Number:	0.6065	
Flow Regime:	Subcritical	
Flow Area:	339.95	sq ft
Average Velocity:	8.52	ft/s
Maximum Velocity:	8.54	ft/s
Composite n:	0.04	
Hydraulic Radius:	3.41	ft
Wetted Perimeter:	99.74	ft
Wetted Top Width:	97.07	ft
Critical Slope:	0.0277	ft/ft

Cross Section from Middle Cave Creek Floodplain Delineation at RM26.784.
This cross section is at Crossing #2.
Upstream cross section RM26.883 and downstream cross section RM26.673 were used to estimate the slope at this section.
Slope = (1540-1530.5)/(533+586) = .0086 ft/ft

Sheet 4 of 12

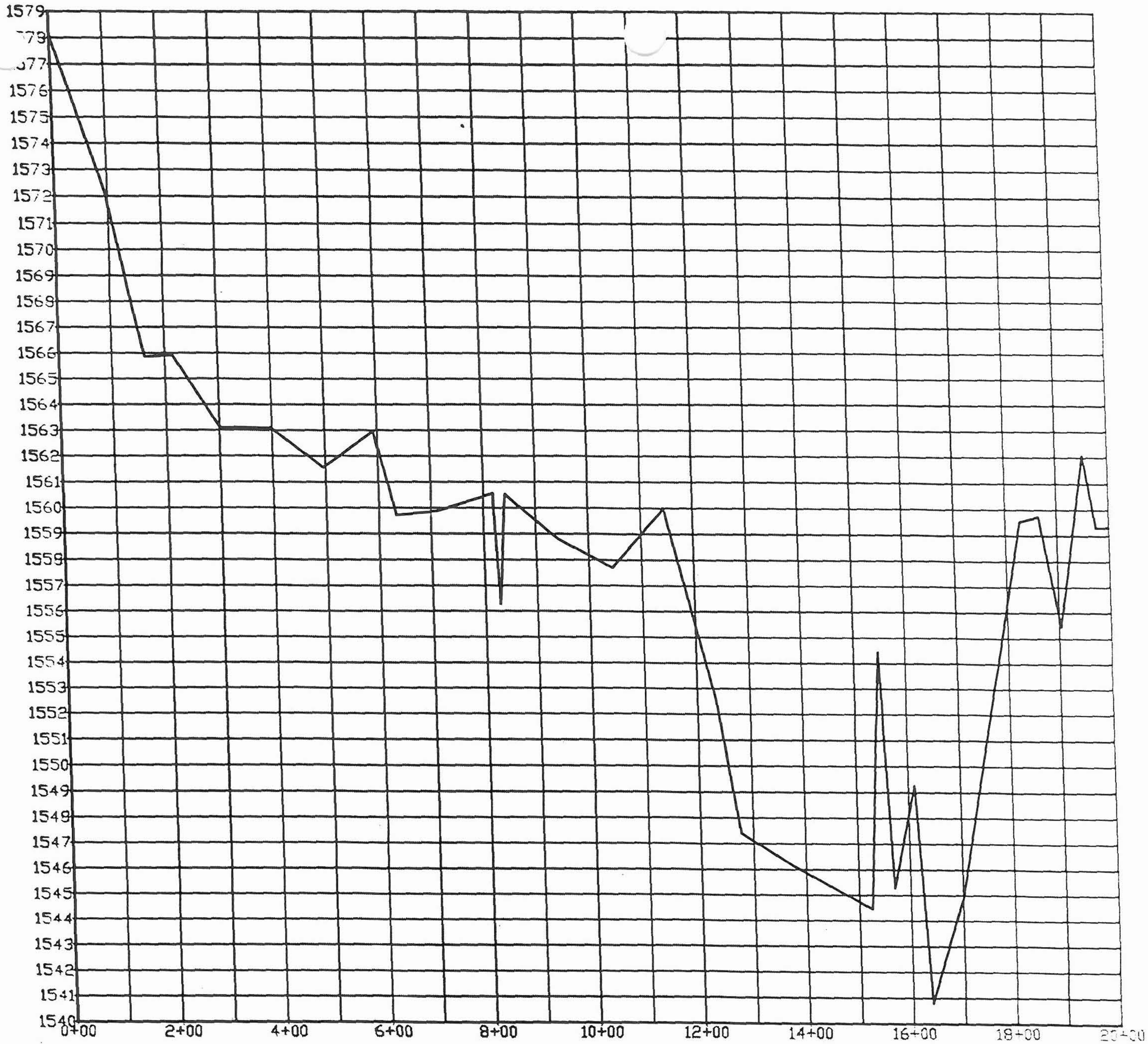


Cross Section from Middle Cave Creek Floodplain Delineation at RM26.673.
This cross section is 586 feet downstream from Crossing #2.
This cross section was used to estimate the slope at Crossing #2.
The invert elevation is at approximately 1530.5 ft.



Cross Section from Middle Cave Creek Floodplain Delineation at RM26.883.
This cross section is 533 feet upstream from Crossing #2.
This cross section was used to estimate the slope at Crossing #2.
The invert elevation is at approximately 1540.0 ft.

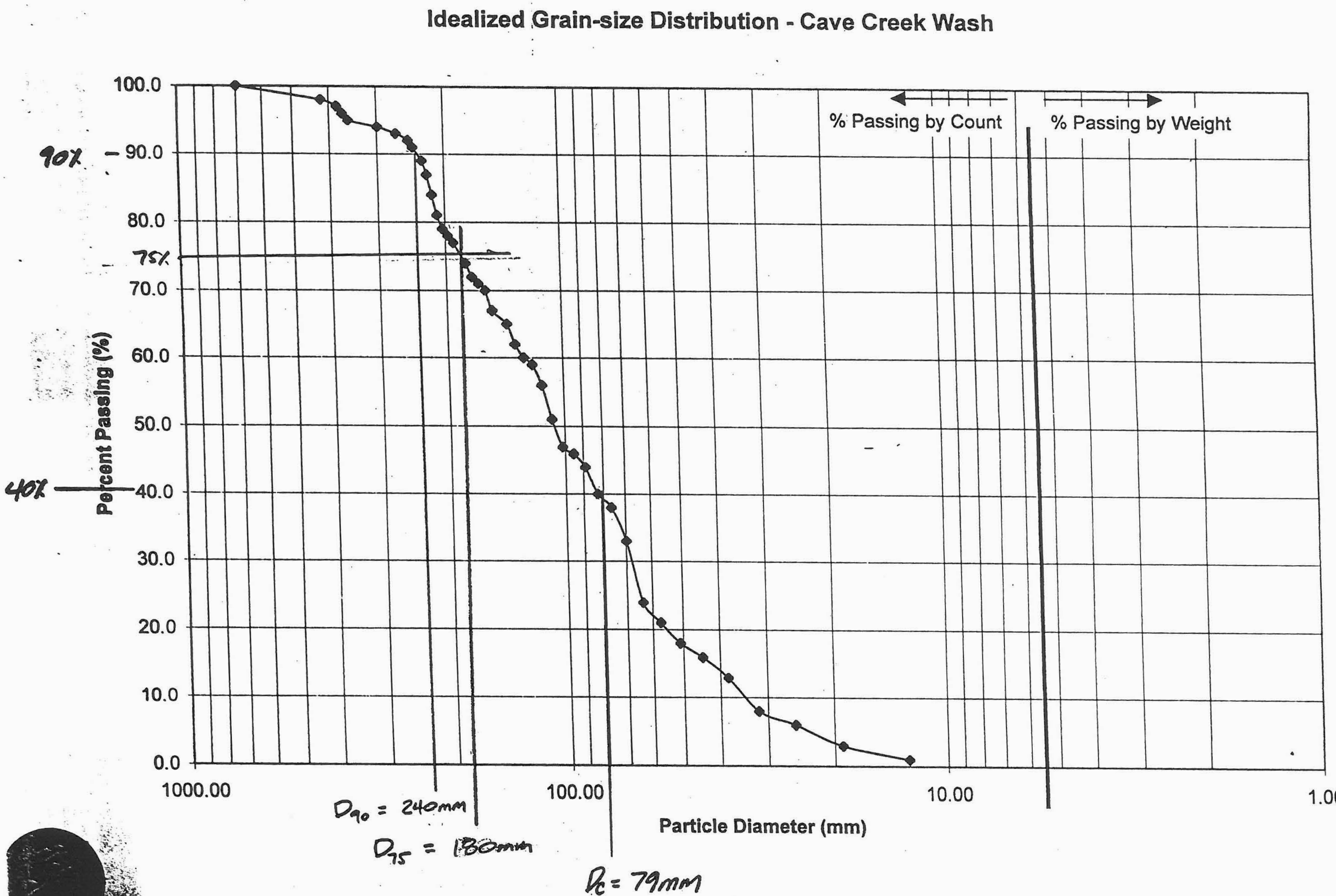
Sheet 6 of 12



This cross section was generated using survey data obtained from DSWA. The cross section is approximately at Crossing #2. The cross section was used to verify the data obtained from the Middle Cave Creek Floodplain Delineation Study HEC-2 model.

Sheet 1 of 16

amec



Bed Material at Crossing #2 Sheet 8 of 12

Gradation Analysis Bed material at Crossing # 2 Sheet 9 of 12

amec

PROJECT:	NORTH GATEWAY WATER RECLAMATION	**JOB NO:**	3-117-001074
LOCATION:	HAPPY VALLEY ROAD AND 19TH AVENUE	**WORK ORDER NO:**	1
MATERIAL:	SOIL	**LAB NO:**	1
SAMPLE SOURCE:	SURFACE SAMPLE CAVE CREEK WASH	**DATE SAMPLED:**	12/08/2003

SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES (ASTM C136/C117)

MECHANICAL ANALYSIS

SIEVE SIZE	% PASSING
6 in / 152mm	100
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3 in / 75mm	100
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#8, 2.36mm	19
#10, 2.00mm	16
#16, 1.18mm	11
#30, 0.60mm	7
#40, .425mm	6
#50, .300mm	6
#100, .150mm	5
#200, .075mm	5.0

NOTES:

Reviewed by: CW

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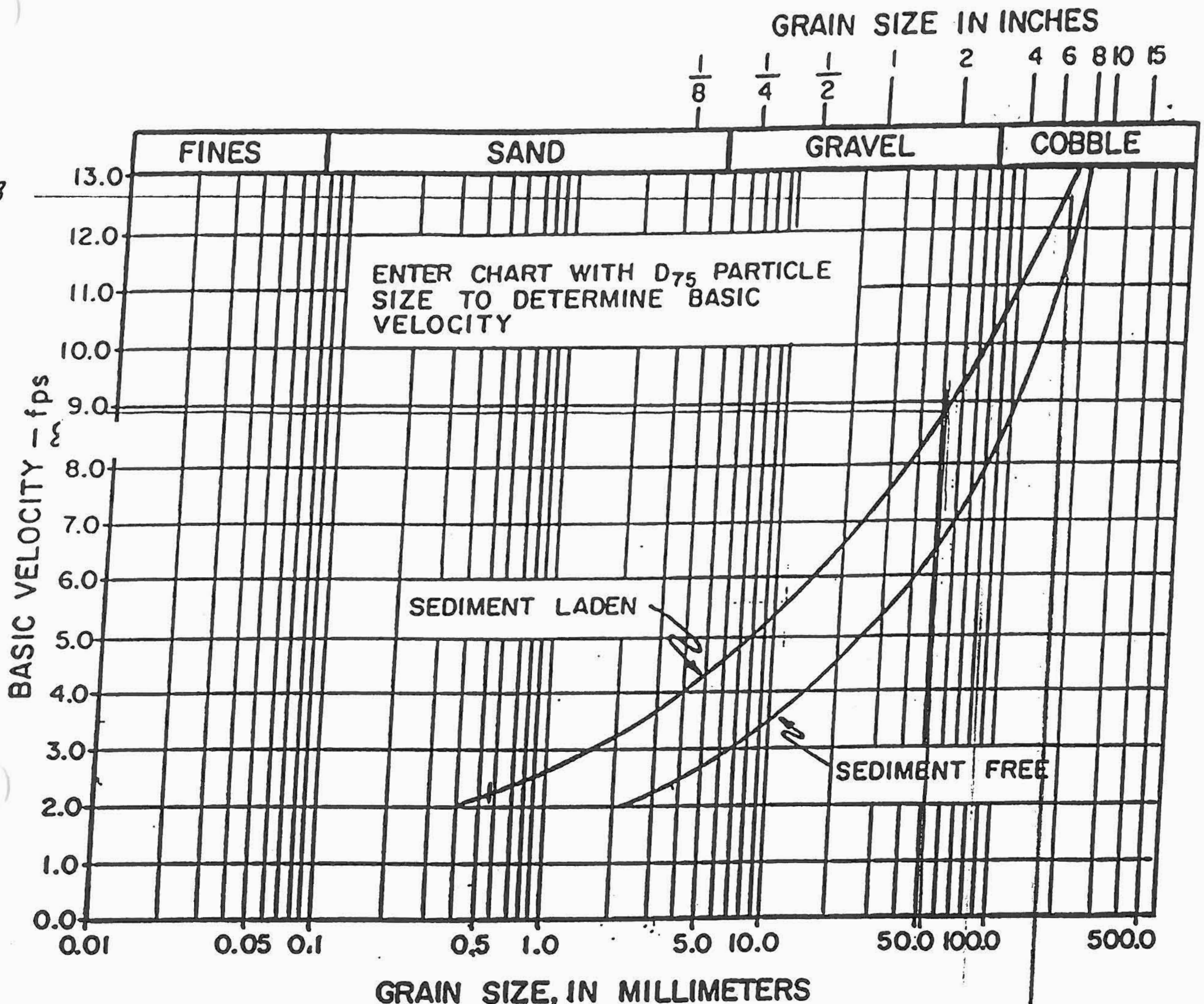
Phoenix, Arizona 85009
(602) 272-6848

Crossing #2 Determination of Basic Velocity

12.8

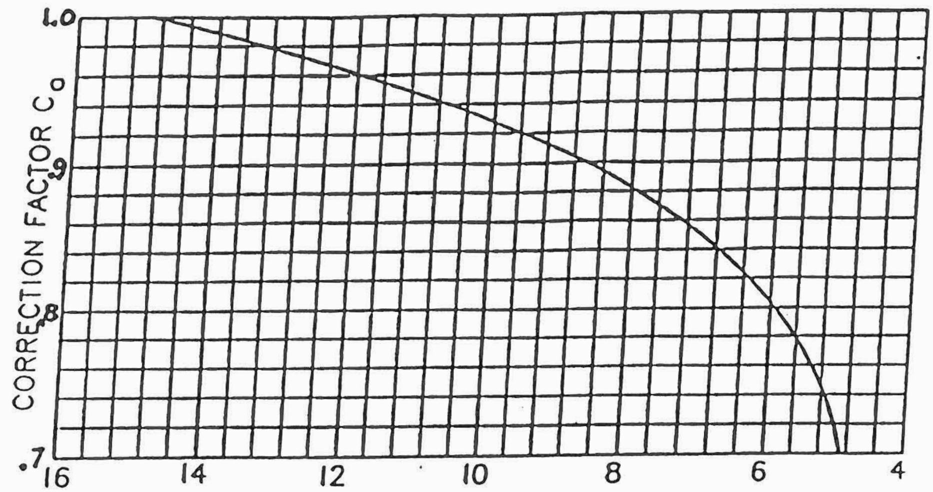


Basic Velocity = 12.8 fps

D_{75} = 180 mm

FIGURE 1

BASIC ALLOWABLE VELOCITY FOR EARTHEN CHANNELS



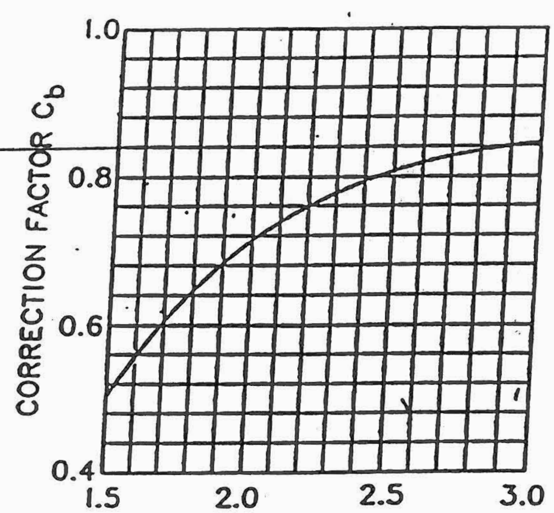
CURVE RADIUS ÷ WATER SURFACE WIDTH

C_a is negligible

FIGURE 2

CORRECTION FACTOR C_a FOR CHANNEL ALIGNMENT

.82



H:7 = 7

BANK SLOPE (H:V)
(Z)

$C_b = .82$

FIGURE 3

CORRECTION FACTOR C_b FOR BANK SLOPE

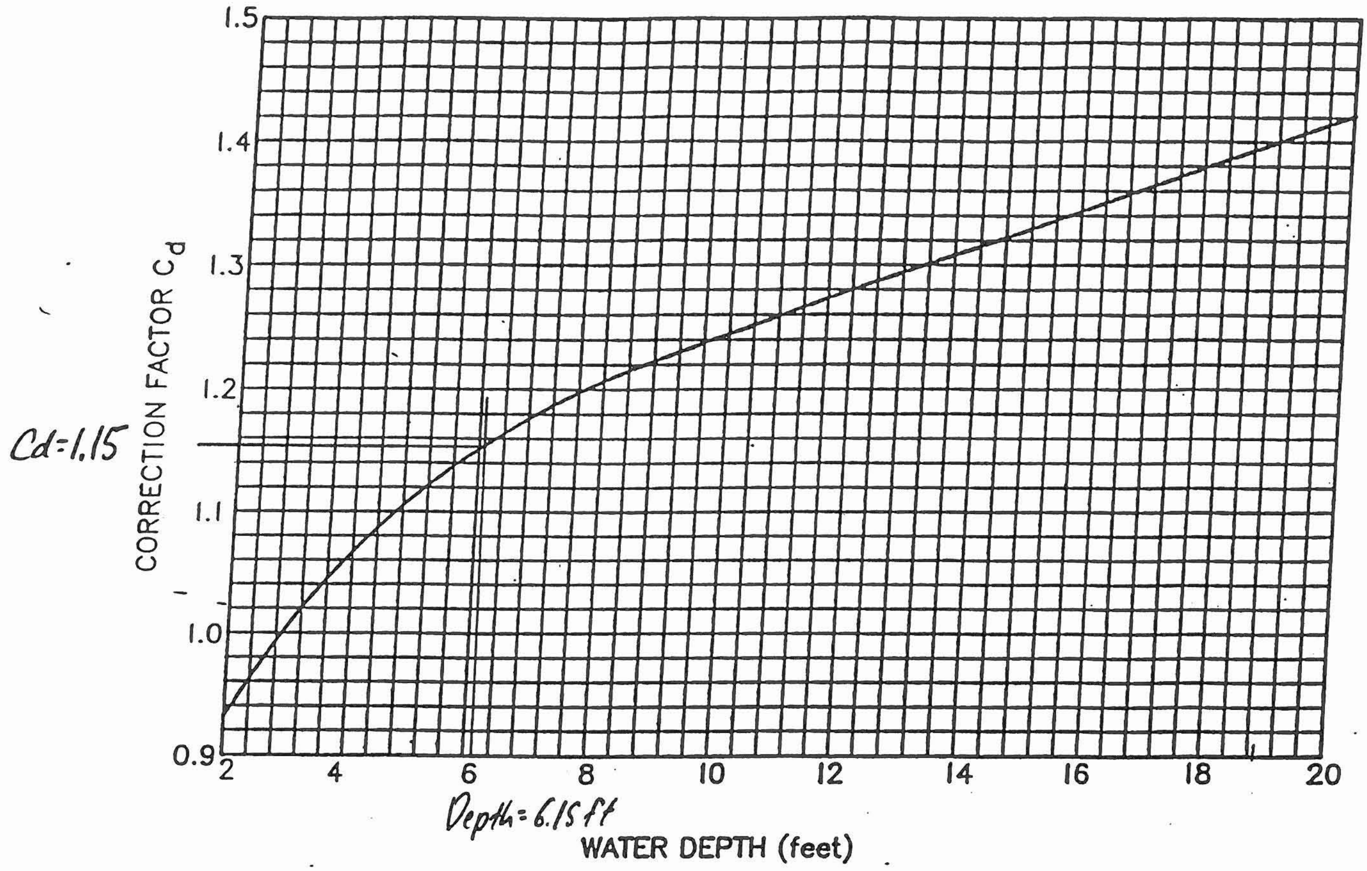
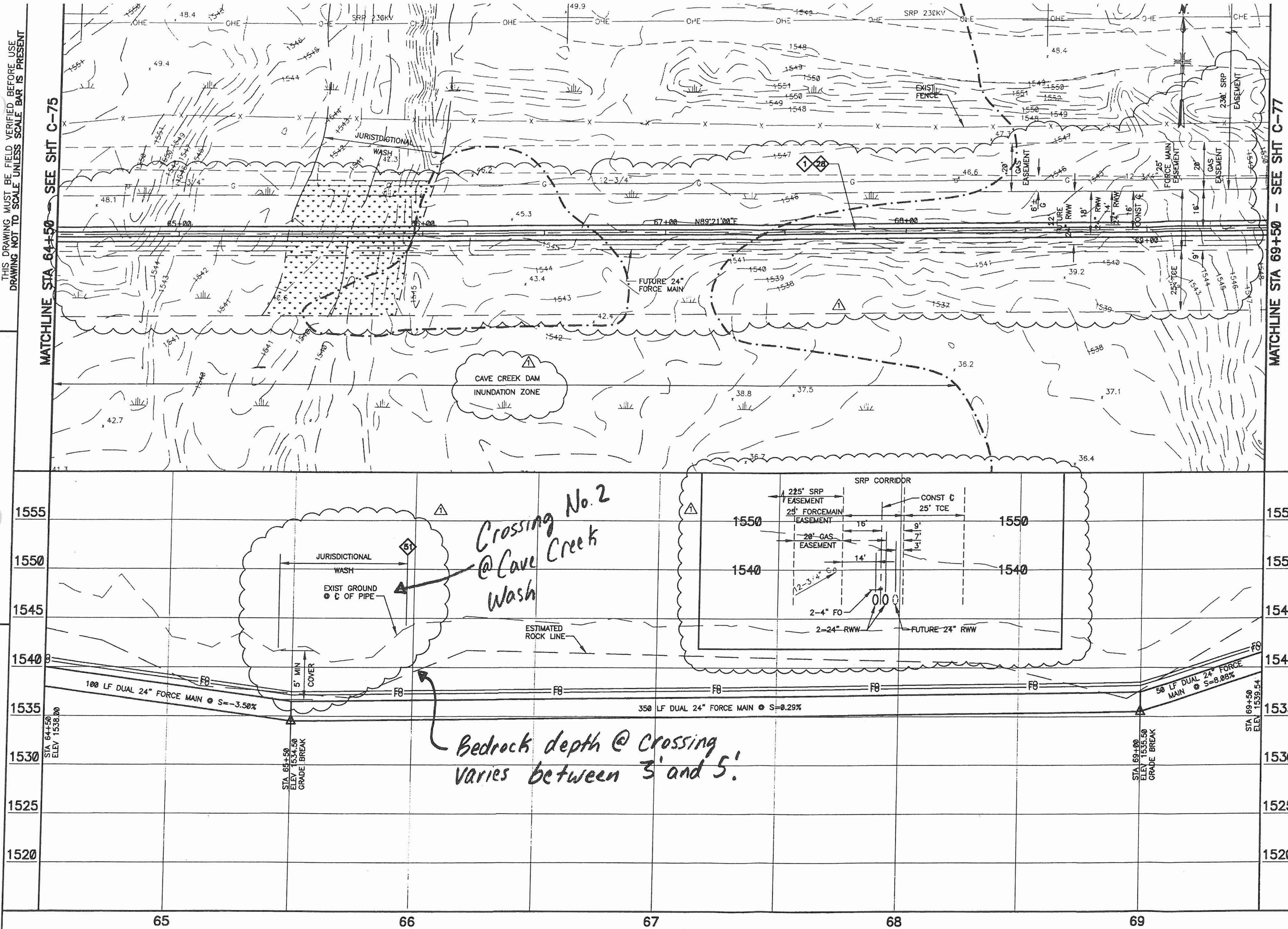


FIGURE 4

CORRECTION FACTOR C_d FOR DEPTH OF FLOW

THIS DRAWING MUST BE FIELD VERIFIED BEFORE USE
DRAWING NOT TO SCALE UNLESS SCALE BAR IS PRESENT



KEY NOTES

1. 24" FORCE MAIN — 1000 LF
28. 4" FO CONDUIT IN ACCORDANCE WITH INDUSTRY AND MFR GUIDE LINES AND APPLICABLE CODES. — 1000 LF
51. INSTALL 4" FO BELOW SCOUR DEPTH. PROVIDE MIN BURIAL DEPTH AS SHOWN.

FACILITY DRAWINGS
This drawing was supplied by a Consultant Engineer from a post construction project. The original construction drawing was modified based on information provided by the Contractor to provide the Record Drawing. The City does not warranty this drawing to be a complete and accurate portrayal of facilities as they exist in the field.

BRASS CAP IN HAND HOLE AT INTERSECTION OF CENTRAL AVE AND HAPPY VALLEY ROAD
STA:88+56.77, 21' L
NORTH:987126.52
EAST:652217.73
ELEV:1538.66

SCALE IN FEET 1"=5' VERT
SCALE IN FEET 1"=20' HORIZ

"PER CITY OF PHOENIX ORDINANCE G-4396, THESE PLANS ARE FOR OFFICIAL USE ONLY AND MAY NOT BE SHARED WITH OTHERS EXCEPT AS REQUIRED TO FULFILL THE OBLIGATIONS OF YOUR CONTRACT WITH THE CITY OF PHOENIX."

Ref. 11

DSWA DAMON S. WILLIAMS ASSOCIATES, L.L.C.

Registered Professional Engineer — CERTIFICATE NO. 30012 — JOHN H. MATTA — Civil — ARIZONA, U.S.A.

REVISIONS				
NO.	BY	DATE	CKD	REMARKS
1	SAR	06/04	RJK	REVISED PER ADDENDUM 1

DES DCM | DWN SAR/PGH | CKD RJK

City of Phoenix

CITY OF PHOENIX
WATER SERVICES DEPARTMENT
NORTH GATEWAY WATER RECLAMATION PLANT PHASE 1
RESIDUALS PUMP STATION AND FORCE MAINS

CIVIL
SEGMENT 5
SALT RIVER PROJECT EASEMENT CORRIDOR
PLAN & PROFILE
STA 64+50 TO 69+50

CITY PROJECT NO. WS85090003
DATE: MAY 2004
DWG NO. C-76
SHEET NO. 112 OF 159
CAD FILE: C-NGFM-PP77.dwg

FOR CITY OF PHOENIX USE ONLY - REFERENCE CID NUMBER: REV. | ENG. (CO.) | PROJ. NO. | PROJ. NAME | DATE | CAD FILE NAME | DWG NUMBER | REMARKS

Dwg File: U:\Cadd\030370\dwg\030372\C-NGFM-PP77.dwg Modified: Jul 02, 2004 - 10:34am Plotted: Jul 08, 2004 - 10:25am

APPENDIX D. CAP OVERCHUTE, CROSSING NO. 3

- Capacity and Level 1 Analysis
- Sketch of Channel
- Crossing Photos
- FHA Inlet Control Monogram for Concrete Pipe Culverts

CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 91005

FORCE MAIN @ CROSSING #3: ESTIMATION OF SCOUR

100-Year Peak Flow

Since no study was available to obtain the flow at this crossing, the scour analysis was performed using the estimated capacity of the CAP culvert.

Estimation of Culvert Capacity

The capacity of the culvert overchute was estimated using the following assumptions:

- The headwater at the culvert inlet is at the depth of overtopping into the canal. This depth was estimated to be 9 feet during the field visit.
- The culvert is operating with inlet control.

Using the nomogram, the culvert capacity was found to be: 100 cfs.

LEVEL 1 SCOUR ANALYSIS

$Q = 100 cfs$

General Degradation, $d_{gs} = 0.157(100)^{.4} = 1.0 ft$

Long term Degradation, $d_{lts} = .02(100)^{.6} = .32 ft$

Total Scour $= 1.3 ft$

Conclusion

Basic analysis shows the scour to be 1.3 ft. Therefore, ~~the burial depth~~ minimum cover should be 4.3 ft at crossing #3.

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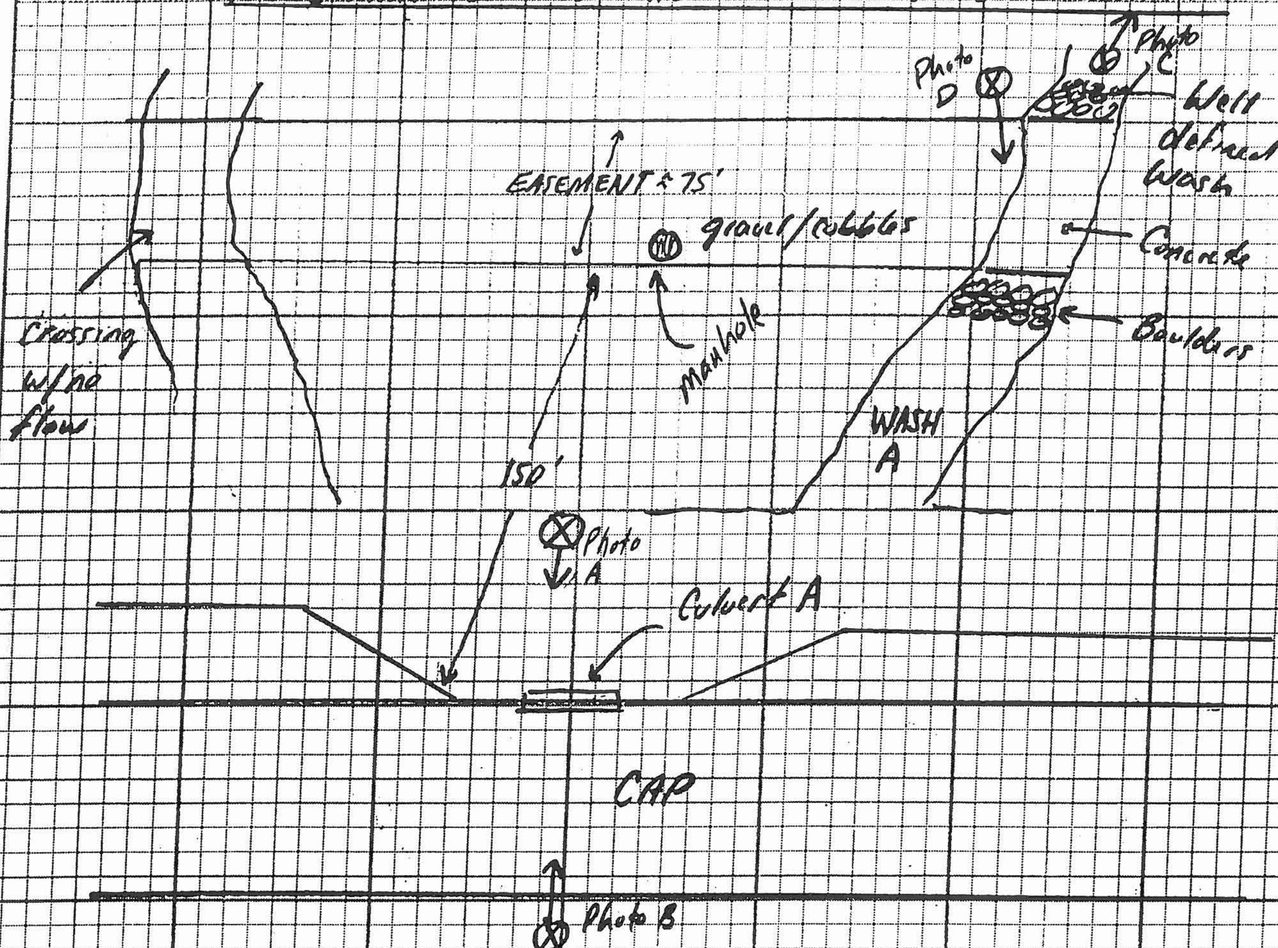
SHEET 2 OF 3
BY JJ DATE 4/22/04
CHECK HAD DATE 4/23/04

CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 910005

FORCE MAIN & CROSSING #3: CAP Culvert: FIELD OBSERVATIONS



WASH A: - 36' total width
- 17' Bottom Width
- Recent Flows
- Scour Protection
- channel - large cobbles and sand
- bank - large cobbles
- mild slope

CULVERT A: - Approx 36" RCP with flared headwall @ 15°
- Overtopping ± 4' above top of pipe
- 2' depression around invert
- Wash A is main contributor of flow
- Good condition

OTHER NOTES:

AMEC TEST BORING STAKE FOUND STA 102+40

Photo A: CAP Overchute Inlet at Crossing #3

Photo B: CAP Overchute Outlet at Crossing #3

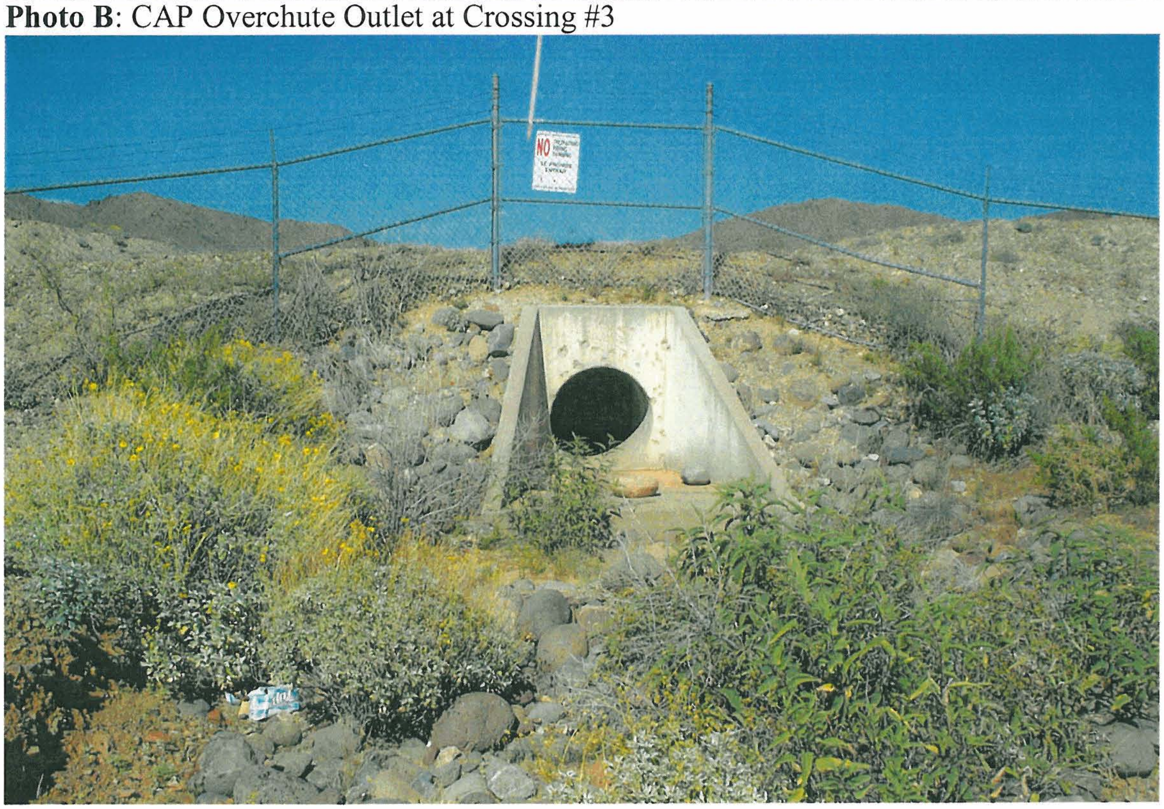
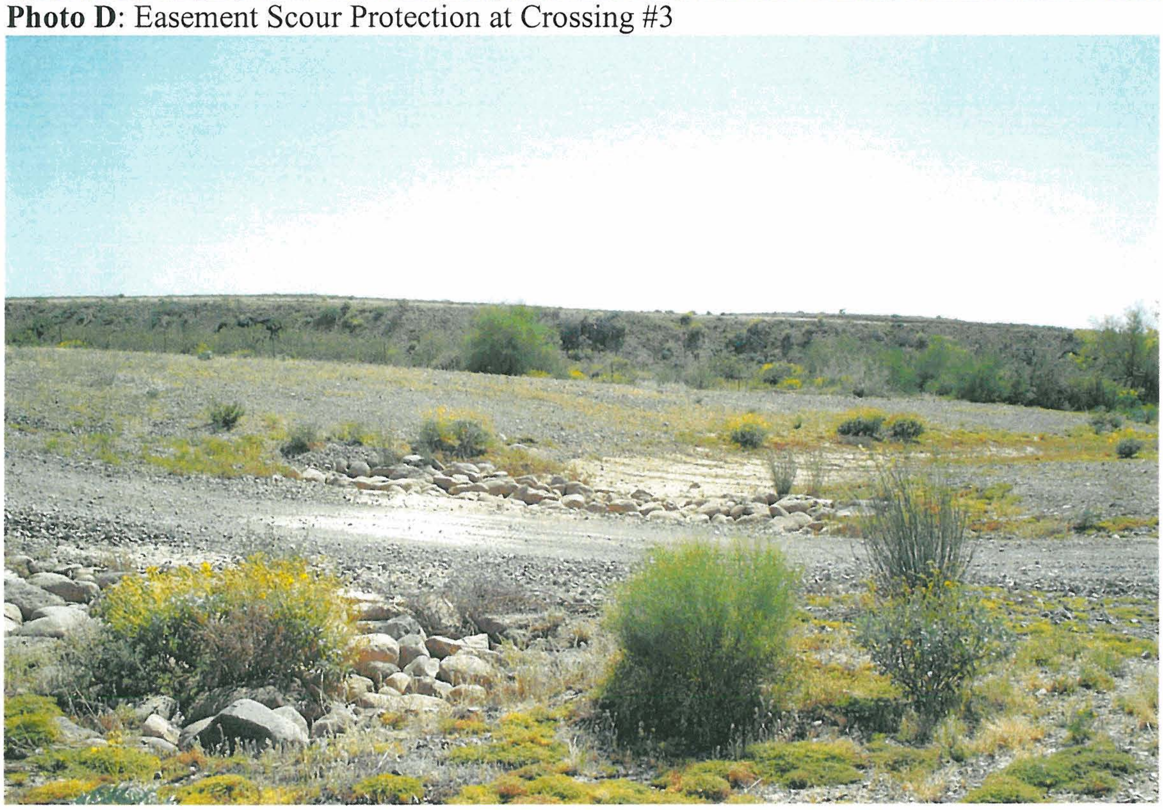


Photo C: Main Channel and Overbanks Upstream of Crossing #3

Photo D: Easement Scour Protection at Crossing #3



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Crossing # 3

SHEET 3 OF 3

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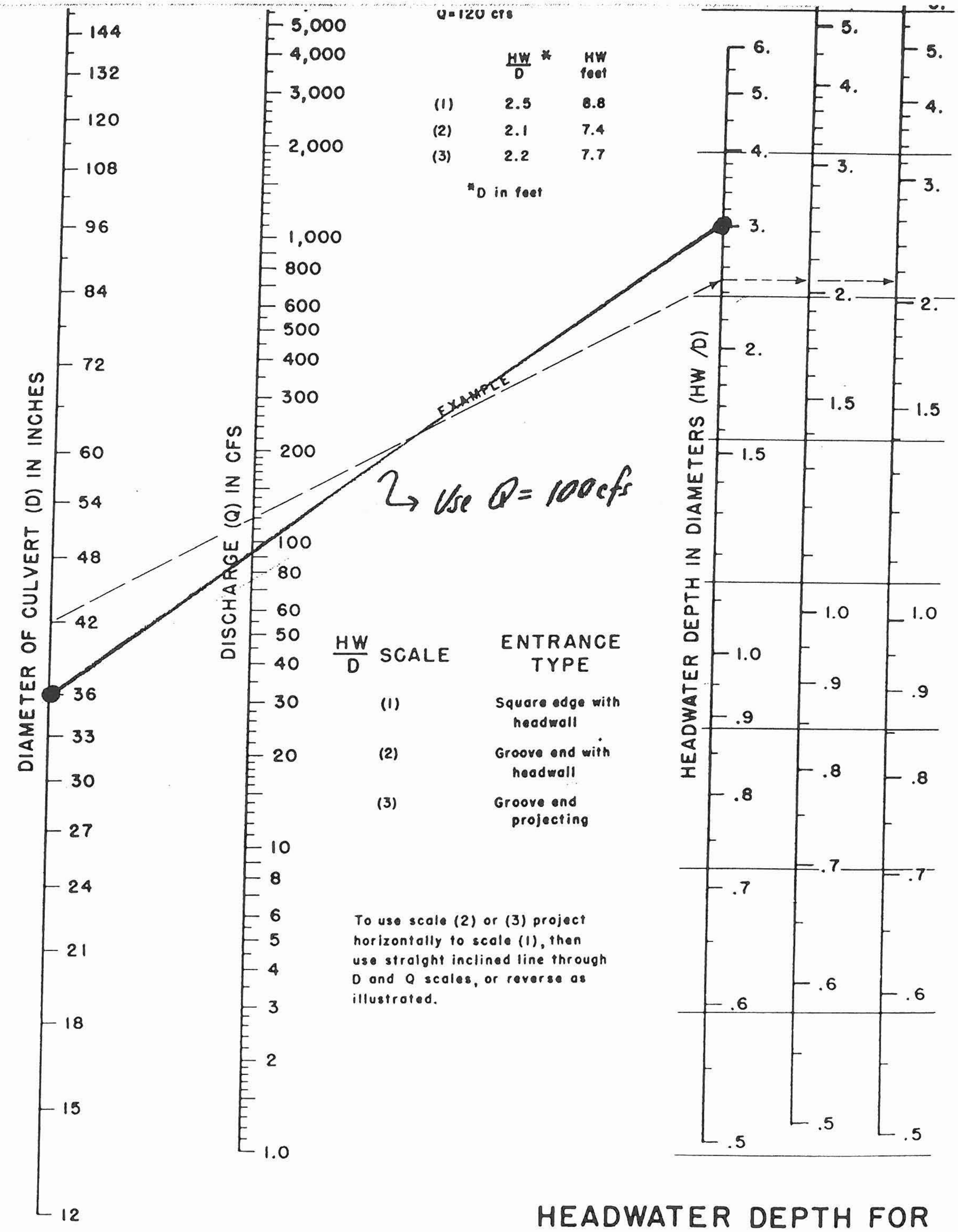
CLIENT DSWA

JOB NAME Force Main Scour Analysis JOB NO. 91000S

HW/D = 9/3 = 3



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

APPENDIX E. SONORAN WASH, CROSSING NO. 4

- Summary of Results
- Cross-sections from Skunk Creek WCMP
- Copies of Relevant Pages from the *Skunk Creek WCMP Attachment 6 – Lateral Stability Analysis* **(Reference 7)**

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SHEET 1 OF 17
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CLIENT DWSA

JOB NAME Force Main Scour Analysis JOB NO.

SUMMARY OF SCOUR RESULTS FROM SKUNK CREEK WMP

Ground Geometry (Sheet 2)

Crossing #4 is at approximately RM 1.33. The geometry at this section was imported from the Skunk Creek WMP HEC-RAS model and plotted. This plot is sheet 2 of this appendix.

Bed Material (Sheet 3)

Flow (Sheet 4)

Scour Estimate (Sheets 5-12)

Armoring Potential (Sheets 13-16)

Reach locations Sheet 17

Long term degradation

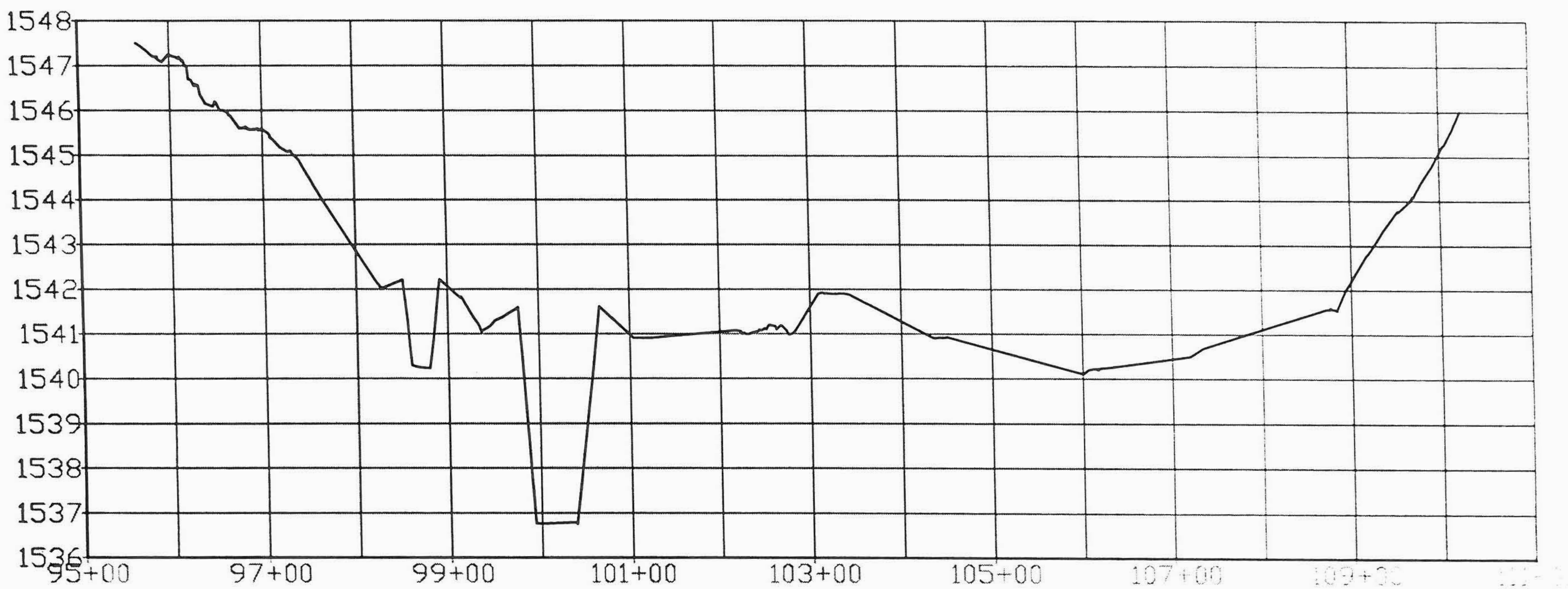
-1.0 ft / 5000 ft slope adjustment Table 5-18
Sheet 8 of 17

the CAP is a slope control for this reach

the crossing is approximately 1 mile from the overchute

Long term degradation ≈ 1.0 ft.

Total Scour = 3.8 + 1.0 ≈ 5.0 ft.



Cross Section from Skunk Creek WMP, Sonoran Wash at RM1.33
This cross section is at Crossing #4.
Hydraulic data at this section was extracted from the Sonoran Wash HEC-RAS output file.

Sheet 2 of 17

land on the larger grains because of their higher profile on the bed. The smaller clasts typically are visible beneath the stretched tape and are better recorded using the stretched tape method.

Results

Sediment samples were obtained for both Skunk Creek and Sonoran Wash. Sieve samples were collected at each of the channel soil pit locations. Boulder counts were completed at approximately 2,000-foot intervals at each of the channel field sections. The sediment distributions applicable to each reach were plotted on a standard sediment sampling data form and a best-fit distribution was selected by eye. The recommended sediment distributions shown in Table 5-2 were used for the engineering and geomorphic analyses presented in Chapters 4 and 5 of this report.

Table 5-2. Skunk Creek Watercourse Master Plan
Sediment Sampling Results

Reach	Mean (mm)	D90 (mm)	D84 (mm)	D50 (mm)	D16 (mm)	D10 (mm)	Max (mm)	Min (mm)
Skunk Creek								
1	28	82	58	5	0.5	0.4	213	0.2
2	30	108	66	6	0.8	0.5	225	0.1
CFR Hwy	31	128	65	6	0.4	0.2	317	0.1
3	37	109	59	6	0.6	0.4	443	0.2
4	32	93	64	9	1.2	0.9	312	0.2
5	36	92	62	9	1.7	1.5	366	0.4
6	40	118	73	16	1.4	0.7	409	0.2
NR Rd.	33	80	68	15	2.0	0.5	335	0.1
Supply	68	229	166	9	2.0	2.0	655	0.4
Sonoran Wash								
1	6	15	12	4	0.2	0.2	43	0.1
2	26	67	56	5	0.6	0.3	255	0.1
3	55	150	123	29	4.6	2.0	328	0.1
4	19	56	41	6	0.2	0.1	142	0.1
5	100	244	213	61	12.1	6.1	549	0.2
6	48	120	99	29	3.6	1.8	308	0.1
Entire	37	96	79	19	2.8	1.4	245	0.1

Crossing #4 is in Reach 2

Notes:
1. Use entire reach average for Sonoran Wash subreaches 3, 4, 5, & 6 since samples didn't get both pools and riffles
2. CFR Hwy – Carefree Highway Bridge NR Rd. – New River Road Bridge

The plots of sediment size distribution shown in Figure 5-45 reveal several trends. First, there is a significant difference in size of the bed materials in riffles compared to the size of the bed materials in pools, as shown by the plots of the data from Skunk Creek. Therefore, sediment-related engineering analyses are highly dependent on whether bed samples are obtained from pools or riffles. Second, the data from Sonoran Wash indicate that mean sediment size varies by about an order of magnitude over the study length.

Sediment sampling data and additional plots of bed sediment distributions are provided in Figure 5-45 and in Appendix C.

Table 3-3-16: Summary of Peak Discharge Results for the Sonoran Wash HEC-1 Models, continued

HEC-1 ID	Existing Drainage Area (sq. mi.)	Time of Peak								Peak Discharge							
		Existing				Future				Existing				Future			
		2-Year (hrs)	10-Year (hrs)	25-Year (hrs)	100-Year (hrs)	2-Year (hrs)	10-Year (hrs)	25-Year (hrs)	100-Year (hrs)	2-Year (cfs)	10-Year (cfs)	25-Year (cfs)	100-Year (cfs)	2-Year (cfs)	10-Year (cfs)	25-Year (cfs)	100-Year (cfs)
U15	0.58	12.17	12.17	12.17	12.17	12.08	12.08	12.08	12.08	362	674	827	1,083	563	890	1,062	1,356
U13A	0.35	12.25	12.25	12.25	12.25	12.17	12.17	12.17	12.17	127	267	343	472	152	306	388	525
Concentration Points																	
C002L	2.57	12.25	12.33	12.33	12.33	12.92	12.75	12.50	12.33	1,068	2,005	2,498	3,267	120	565	1,798	3,454
C002	4.99	12.25	12.33	12.33	12.33	12.92	12.92	12.50	12.33	2,008	3,916	4,892	6,492	121	857	3,295	7,246
C003L	5.68	12.50	12.50	12.42	12.42	13.50	13.08	12.75	12.58	1,882	3,772	4,829	6,303	74	644	2,227	5,695
C003	7.70	12.50	12.42	12.42	12.33	13.50	12.50	12.75	12.50	2,241	4,780	6,235	8,359	74	855	2,477	6,861
C007L	8.69	12.75	12.67	12.67	12.58	14.25	13.00	13.00	12.83	2,063	4,423	5,754	8,039	49	755	2,308	5,856
C006L	0.76	12.25	12.25	12.25	12.25	13.67	13.25	13.17	12.33	197	408	515	693	36	132	225	613
C006R	1.66	12.33	12.25	12.25	12.25	0.00	13.67	12.75	12.33	506	1,209	1,608	2,274	0	33	437	1,579
C006	2.42	12.33	12.25	12.25	12.25	13.67	13.33	12.75	12.33	667	1,599	2,100	2,938	36	132	527	2,160
C007	11.11	12.67	12.58	12.50	12.50	14.25	13.00	13.00	12.75	2,338	5,130	6,785	9,664	65	766	2,539	6,671
C010L	12.14	13.33	13.08	13.00	12.92	15.00	13.42	13.67	13.17	2,044	4,644	6,369	9,203	52	731	2,176	5,889
C010R	1.24	12.50	12.33	12.25	12.25	0.00	13.08	12.33	12.17	432	964	1,229	1,673	0	56	392	1,264
C010	13.38	13.33	13.00	12.92	12.83	15.00	13.42	13.67	13.17	2,098	4,852	6,712	9,825	52	717	2,241	6,098
Diversion Operations																	
DTU1	0.81	---	---	---	---	12.08	11.92	11.83	11.75	---	---	---	---	536	565	465	450
DU1	0.81	---	---	---	---	12.17	12.08	12.08	12.08	---	---	---	---	299	1,020	1,269	1,685
DTU2	1.76	---	---	---	---	12.25	12.25	12.25	12.17	---	---	---	---	1,339	2,188	2,625	3,371
DU2	1.76	---	---	---	---	0.00	12.92	12.42	12.25	---	---	---	---	0	263	1,338	2,951
DTU3	1.61	---	---	---	---	12.25	12.25	12.25	12.25	---	---	---	---	1,062	1,784	2,151	2,500
DU3	1.61	---	---	---	---	0.00	12.83	12.50	12.33	---	---	---	---	0	331	1,354	2,733
DTU4	0.82	---	---	---	---	12.25	12.25	12.17	12.08	---	---	---	---	470	886	1,107	1,272
DU4	0.82	---	---	---	---	0.00	12.42	12.33	12.25	---	---	---	---	0	498	933	1,474
DTU6	0.69	---	---	---	---	12.08	12.08	12.08	12.00	---	---	---	---	429	800	989	1,113

Crossing #4

Ref. 3

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Table 5-16. Skunk Creek Watercourse Master Plan Scour Estimates – Skunk Creek (ft.)							
Reach	Total Zt	General Zgs	Antidune Za	Bend Angle	Bend Zbs	Local Zls	Thalweg Zlft
Q100							
SR	3.6	-1.3	2.8	19.4	0.4	0.0	1.0
NR	3.1	-2.1	2.2	19.4	0.3	0.0	1.0
6	3.7	-1.3	3.0	19.4	0.3	0.0	1.0
5	4.0	-1.4	3.4	19.4	0.4	0.0	1.0
4	4.0	-0.9	3.4	18.1	0.3	0.0	1.0
3	3.6	-0.2	3.5	11.4	0.0	0.0	1.0
CFR	4.2	-0.7	4.3	11.4	0.0	0.0	1.0
2	3.8	-0.3	3.8	15.9	0.0	0.0	1.0
1	4.7	-0.2	2.8	20.5	0.6	0.0	1.0
Q10							
SR	3.2	-1.1	2.4	19.4	0.3	0.0	1.0
NR	2.6	-2.2	1.5	19.4	0.2	0.0	1.0
6	3.1	-1.4	2.3	19.4	0.3	0.0	1.0
5	3.1	-1.9	2.3	19.4	0.3	0.0	1.0
4	3.5	-0.8	2.9	17.8	0.3	0.0	1.0
3	3.0	-0.5	2.6	11.4	0.0	0.0	1.0
CFR	2.9	-0.6	2.5	11.4	0.0	0.0	1.0
2	3.2	-0.5	3.0	15.9	0.0	0.0	1.0
1	4.1	-0.3	2.4	20.5	0.5	0.0	1.0
Q2							
SR	1.8	-0.7	0.6	19.4	0.1	0.0	1.0
NR	1.7	-1.6	0.4	19.4	0.1	0.0	1.0
6	1.8	-0.7	0.6	19.4	0.1	0.0	1.0
5	1.9	-0.9	0.8	19.4	0.1	0.0	1.0
4	1.9	-0.7	0.8	17.8	0.1	0.0	1.0
3	1.7	-0.6	0.6	11.4	0.0	0.0	1.0
CFR	1.6	-0.8	0.4	11.4	0.0	0.0	1.0
2	1.8	-0.6	0.7	15.9	0.0	0.0	1.0
1	2.0	-0.6	0.7	20.5	0.1	0.0	1.0
Note: Long-term and local scour not included in estimate of total scour.							

Sonoran Wash. Scour estimates for Sonoran Wash are shown in Table 5-17. The total scour depths are not significantly less than for Skunk Creek due to the generally lower width/depth ratio in Sonoran Wash, which results in higher unit discharge in the channel, flow depths, and velocities. Predicted scour depths for the 2-year event are about equal to the depth of the clay-rich sublayer observed in the excavated channel soil pits. No evidence for scour in the range of the estimated total scour depths for the 10- and 100-year events was observed in the field, indicating that the total scour depths in Table 5-17 are probably overestimated for the large floods or that bend scour has not been a significant component of the total scour in the past. The similarity of the 10- and 100-year total scour estimates is due to negligible increase in channel discharge for flows greater than the 10-year event. That is, flows greater than the 10-year peak discharge tend to inundate the floodplain, and do not significantly increase the depth and velocity of flow in the main channel.

Scour Depth @ Crossing #4 w/o long term Scour

Table 5-17. Skunk Creek Watercourse Master Plan Scour Estimates - Sonoran Wash (ft.)							
Reach	**Total Zt**	**General Zgs**	**Antidune Za**	**Bend Angle**	**Bend Zbs**	**Local Zls**	**Thalweg Zlft**
Q100							
6	3.8	-1.1	2.9	20.8	0.5	0.0	1.0
5	4.2	-1.1	3.4	20.8	0.6	0.0	1.0
4	4.0	-1.2	3.1	20.8	0.5	0.0	1.0
3	4.2	-1.2	3.4	20.8	0.6	0.0	1.0
2	3.8	-0.7	2.8	20.8	0.5	0.0	1.0
1	4.6	-0.3	3.0	20.8	0.6	0.0	1.0
Q10							
6	3.5	-1.1	2.6	20.8	0.4	0.0	1.0
5	3.9	-1.1	3.0	20.8	0.5	0.0	1.0
4	3.7	-1.2	2.7	20.8	0.5	0.0	1.0
3	3.9	-1.2	3.0	20.8	0.5	0.0	1.0
2	3.6	-0.8	2.5	20.8	0.5	0.0	1.0
1	4.0	-0.4	2.6	20.8	0.5	0.0	1.0
Q2							
6	1.8	-0.7	0.6	20.8	0.1	0.0	1.0
5	2.2	-0.6	0.7	20.8	0.4	0.0	1.0
4	1.9	-0.6	0.7	20.8	0.1	0.0	1.0
3	1.9	-0.6	0.8	20.8	0.1	0.0	1.0
2	1.9	-0.6	0.7	20.8	0.1	0.0	1.0
1	1.8	-0.5	0.5	20.8	0.1	0.0	1.0
Note: Long-term and local scour not included in estimate of total scour.							

Long-Term Scour. The long-term scour component is the progressive scour that occurs over long time periods, rather than in response to a single flow event. Long-term scour was estimated from the following types of data:

- Field estimates of recent scour
- Interpretation of longitudinal profiles
- Interpretation of historical maps and photographs
- Interpretation of the ages of geomorphic surfaces
- Comparison of equilibrium and existing channel slopes

The first of four of the types of data listed above were described in Chapters 3 and 4 of this report. Field data were described in Chapter 4, and consisted of qualitative estimates of whether the channel had recently scoured or filled, and the depth of recent long-term scour. Longitudinal profiles were described in Chapters 3 and 4, and were used to estimate whether the bed elevation had moved up or down during the period of record. Geomorphic mapping of stream terraces was used to establish the net channel bed adjustments over the past 10,000 to 700,000 years. A summary of these data is shown in Table 5-18.

Predictions of the magnitude of long-term degradation or aggradation can also be made by comparing the predicted equilibrium slope with the existing channel slope (Tables 5-12 and 5-13). The slope difference multiplied by a stream reach length is the amount of adjustment in bed elevation at the upstream end of the reach. Reach lengths of 1,000 and 5,000 feet were used for the predictions shown in Table 5-18. Typically, long-term scour estimated using equilibrium slope is measured from the closest point of permanent grade control. However, because the only permanent grade control in the study area is at the CAP overchutes, as the distance from the CAP increases the predicted long-term scour depth would become unreasonably large. Therefore, the estimates were based on reach lengths of 1,000 and 5,000 feet to illustrate the potential range of channel responses to long-term slope adjustment.

Summary. General and long-term scour estimates for the streams in the study area indicate that moderate scour should be expected for Skunk Creek, especially in channel bends. Somewhat lower scour depths should be expected for Sonoran Wash. When scour occurs, it undermines the channel banks and increases the rate of lateral erosion. Therefore, the greatest amount of scour-induced bank erosion in the study area should be expected at channel bends, near obstructions, or where the channel has been excavated. Estimated bank erosion distances should be adjusted upward where bed scour is significant.

Table 5-18. Skunk Creek Watercourse Master Plan Summary of Long-Term Degradation/Aggradation Data Sources										
Reach	Field Assessment	Longitudinal Profile Comparison	Archaeological & Historic Data (ft/yr)	Geologic Mapping (ft/yr)	Equilibrium Slope Adjustment – Reach Length (ft)					
					Q100		Q10		Q2	
					1000 ft	5000 ft	1000 ft	5000 ft	1000 ft	5000 ft
Skunk Creek										
SR	Degradation	Degradation	No information	< -0.0001	-3.3	-16.3	-2.3	-11.3	4.3	21.3
NR	Aggradation	Mixed	+ 4 ft. since 1996	< -0.0001	4.2	20.8	4.3	21.3	6.9	34.7
6	Mixed	Degradation	No information	< -0.0001	1.1	5.6	1.3	6.5	4.5	22.4
5	Aggradation	Mixed	No information	< -0.0001	-2.8	-13.8	-3.3	-16.5	-0.5	-2.6
4	Degradation	Mixed	No information	< -0.0001	-2.4	-12.0	-1.8	-9.2	0.5	2.6
3	Mixed	Mixed	No information	< -0.0001	-2.6	-12.9	-2.4	-11.9	0.4	1.8
CFR	Aggradation	Mixed	+ 3 ft since 1977	< -0.0001	-0.1	-0.6	0.5	2.3	3.3	16.6
2	Stable	Mixed	No information	< -0.0001	-2.5	-12.6	-2.3	-11.7	0.0	-0.1
1	Mixed	Mixed	No information	< -0.0001	-1.0	-4.8	-0.9	-4.3	2.2	10.9
Sonoran Wash										
6	Stable	Unclear	No information	< -0.0001	4.3	21.6	4.5	22.4	10.0	50.1
5	Stable	Unclear	No information	< -0.0001	5.6	27.8	5.6	28.2	10.2	51.1
4	Stable	Unclear	No information	< -0.0001	4.2	20.9	4.3	21.6	9.1	45.6
3	Stable	Unclear	No information	< -0.0001	4.5	22.5	4.6	23.2	8.5	42.7
2	Degradation	Unclear	No information	< -0.0001	-0.2	-1.0	-0.2	-1.0	2.0	10.2
1	Aggradation	Unclear	No information	< -0.0001	0.8	4.1	1.0	4.8	2.3	11.7
Source of Data	Chapter 4 Appendix A	Chapter 3 Fig. 3-15 to 19	Chapter 3	Chapter 4 Fig. 4-76	Values computed by the following equation: (equilibrium slope – existing channel slope) x reach length (ft.)					

Crossing #4, long term scour ≈ 1 foot (3/4 of a mile upstream from CAP)

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Comparison of Armoring, Scour, and Equilibrium Slope Predictions.
Channel degradation can be prevented by armoring of the channel bed, by achieving a non-scouring stable slope, or by physical barriers to scour such as bedrock or artificial grade control. A comparison of the armoring, scour and equilibrium slope estimates described in the previous sections of this chapter is provided in Tables 5-19 and 5-20. The possible slope adjustment, or depth of long-term scour caused as the channel adjusts to stable slope, was estimated by multiplying the difference in the predicted (regime) and existing channel slopes by a specified reach length of 1,000 or 5,000 feet. The latter two distances were selected based on the length of typical pool and riffle sequence as well as on the reach lengths used for this study. The distances are intended to illustrate the order of magnitude of vertical change possible due to slope adjustments, rather than a specific prediction of long-term scour at any specific point in the study reach. Actually long-term changes will depend on a variety of site-specific variables.

The "Armor v. Scour" and "Armor v. Slope" columns in Tables 5-19 and 5-20 indicate whether total or long-term scour will be limited by armoring. That is, if the predicted depth of general scour (column 3 in Tables 5-19 and 5-20) is less than the depth of scour required to form an armor layer, scour will not be limited by armoring at that flow rate. Similarly, if the difference between the predicted and existing channel slope is too small to cause long-term scour greater than the depth of scour required to form an armor layer, long-term scour will not be limited by armoring. A "no" code indicates that scour will not be limited by armoring. A "yes" code indicates that scour will be limited by an armor layer.

Skunk Creek. As shown in Table 5-19, armoring generally will not prevent long-term degradation (last column) on Skunk Creek where it is predicted by the equilibrium slope analysis, except in the supply reach upstream of the New River Road bridge. Short-term or single event scour will be prevented by armoring in reach 1, and upstream of reach 3 during the 2- and 10-year events, and upstream of reaches 5 during the 100-year event. Bank stability will be most impacted by bed scour during the largest floods.

General Scour

Scour is defined as any lowering of the channel bed elevation that occurs as a result of flowing water. Scour can be caused by changes in the sediment transport capacity of a channel during the passage of a flood wave (general scour), by the formation of bed forms (dune, anti-dune, thalweg scour), by velocity currents around channel bends (bend scour), by local flow obstructions (local scour), or by gradual adjustments to changes in channel morphology (long-term scour).

Methodology. General scour for Skunk Creek and Sonoran Wash was estimated using procedures outlined in the City of Tucson's *Standards Manual for Drainage Design and Floodplain Management - Chapter VI - Erosion and Sedimentation* (1989; hereafter, "the COT Manual"). Depth of scour in a stream is given in the COT Manual:

$$Z_t = 1.3\,(Z_{gs} + \tfrac{1}{2} Z_a + Z_{ls} + Z_{bs} + Z_{lft})$$

where:

Z_t	= Design scour depth, excluding long-term degradation or aggradation (ft)
Z_{gs}	= General scour depth (ft)
Z_a	= Anti-dune trough depth (ft)
Z_{ls}	= Local scour depth (ft)
Z_{bs}	= Bend scour depth (ft)
Z_{lft}	= Low-flow thalweg depth (ft)
1.3	= Safety factor to account for nonuniform flow distribution

General scour, Z_{gs}, is the component of scour that represents the mobile portion of the bed-material of the channel bottom. General scour was estimated using the following equation:

$$Z_{gs} = Y_{max}\,[(0.0685\,V_m^{0.8})/(Y_h^{0.4}\,S_e^{0.3})-1]$$

where:

Z_{gs}	= General scour depth (ft)
V_m	= Average velocity of flow at design discharge (ft/sec)
Y_{max}	= Maximum depth of flow at design discharge (ft)
Y_h	= Hydraulic depth of flow at design discharge, (ft)
S_e	= Energy slope (ft/ft)

Where Z_{gs} was determined to be negative, the general scour component was assumed to be zero.

Anti-dune trough depth, Za, is the component of scour caused by movement of dune shaped bed forms along the bottom of the channel. The anti-dune trough depth was estimated using the following equation:

$$Z_a = 0.0137\,V^2_m$$

where:

V_m = Average velocity of flow at design discharge (ft/sec)

The anti-dune trough depth is limited to a maximum of ½ the flow depth. Anti-dunes were observed on portions of Skunk Creek during the small flood which occurred on July 15, 1999. Therefore, it was assumed that antidunes could form in any part of the study reach, except in riffles or in the reaches with the coarsest bed sediments.

Low-flow thalweg scour, Z_{lft}, occurs if a small channel forms to convey minor flows within the main channel of a stream. Typically, a low-flow thalweg forms on large streams with a high width to depth ratio and with mobile bed sediments. No physical evidence of formation of a distinct low flow thalweg was observed on Skunk Creek or Sonoran Wash, either during floods or in the channels between floods. However, to be conservative, the low-flow thalweg component of scour was assumed to be one foot for the purposes of the scour analysis.

Bend scour, Z_{bs}, occurs on the outside of bends in a stream channel, and is caused by spiral transverse currents. Bend scour was estimated using the following equation:

$$Z_{bs} = 0.0685\, Y_{max}\, V_m^{0.8}\, Y_h^{-0.4}\, S_e^{-0.3}\, \{2.1\, [\sin^2(\alpha/2)/\cos \alpha]^{0.2} - 1\}$$

where:

Z_{bs} = Bend-scour component of total scour depth (ft), and
= 0 when $r_c/T > 10.0$, or $\alpha < 17.8^{\circ}$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8^{\circ} < \alpha < 60^{\circ}$
= computed value when $\alpha = 60^{\circ}$ when $r_c/T < 0.5$, or $\alpha > 60^{\circ}$

Y_{max} = Maximum depth of flow immediately upstream of the bend (ft)
V_m = Average velocity of flow immediately upstream of the bend (ft/sec)
Y_h = Hydraulic depth of flow immediately upstream of the bend (ft)
S_e = Energy slope immediately upstream of the bend (ft/ft)
α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel (degrees)
r_c = radius of curvature along centerline of channel (ft)
T = channel topwidth (ft)

The reach-averaged bend angle was computed from the arccosine of the reciprocal of the sinuosity.

Local scour, Z_{ls}, occurs where there is an abrupt change in the direction of flow caused by obstructions such as bridge piers, abutments, or other structures. Local scour will occur at the Carefree Highway and New River Road bridges, as well as at new bridge crossings currently planned but not constructed south of the Carefree Highway. However, since local scour at these structures will be limited to the bridge section itself, the local scour component was not included in the estimate of total scour for the entire study reach. Local scour may also occur along the margin of the floodplain bank protection proposed for the Tramonto Subdivision which is currently under construction.

Long-term scour, or aggradation and degradation, is best evaluated from historical evidence and field data. Historical evidence of long-term changes in channel bed elevation was discussed in Chapters 3 and 4 of this report. Depending on the time scale considered, long-term scour can be the largest component of scour. For example, if sufficient time is allowed for the channel to achieve its equilibrium slope or to become armored, the long-term scour component could more than double the scour estimate. A practical rule of thumb for determining a reasonable maximum long-term estimate for undisturbed watersheds is to use the height of the floodplain above the channel bottom or the bank height (Table 5-5).

Results. Scour estimates for Skunk Creek and Sonoran Wash obtained from the City of Tucson scour equations are summarized in Tables 5-16 and 5-17. In general, the largest component of scour other than long-term scour is the bend scour. Given that the bed scour is limited to the outside of channel bends, the scour estimates listed in the first columns of Tables 5-21 to 5-24 are conservative when applied to an entire reach. However, given the potential for future channel movement within the stream corridor, consideration of bend scour at any point within the reach is prudent for design of any structure with an extended design life. In every reach within the study area, general scour was calculated as a negative value, which the COT Manual dictates should be interpreted as a zero depth of scour. Local scour was estimated as zero for the study, since reach-averaged values for a local condition could not be justified. Thalweg scour was also estimated as zero because a low flow thalweg was not observed in the study reaches.

Skunk Creek. Scour estimates for Skunk Creek are shown in Table 5-16. Neglecting the bend scour component, the total scour along Skunk Creek is less than one foot for the 2-year event, less than three feet for the 10-year event, and two to four feet for the 100-year event. In sinuous reaches, the bend scour component increases the total scour estimate by a factor of four to five. The 2- and 10-year scour depths[7] are most similar to the depth of the observed clay-rich layer in the excavated channel soil pits, indicating either that the channel has not experienced a recent extreme flood or that the 100-year scour depths are overestimated. The total scour depths shown in the first column of Table 5-16 are primarily due to the bend scour component, which itself is driven by the channel sinuosity. Therefore, the more sinuous reaches have the greatest total scour estimates. The similarity of the 10- and 100-year total scour estimates is due to marginal increase in channel discharge for flows greater than the 10-year event. That is, flows greater than the 10-year peak discharge tend to inundate the floodplain, and do not significantly increase the depth and velocity of flow in the main channel.

[7] Bend scour is neglected for this comparison since channel pits were excavated in straight reaches at mid-channel.

Table 5-14. Skunk Creek Watercourse Master Plan Armoring Analysis Results – Skunk Creek								
Reach	Methodology – Critical Armor Diameter (mm)				Average Critical Diam. (mm)	Field D50 (mm)	Armor Layer Likely?	Depth to Armor (ft.)
	MPM	CBV	Yang	Shield				
100-Year Flood								
SR	112	180	192	175	165	9	Yes	2.6
NR	46	93	100	94	83	15	Yes	1.6
6	84	168	179	155	146	16	No	5.8
5	99	221	236	200	189	9	No	15.0
4	94	217	232	175	180	9	No	12.7
3	115	266	284	181	211	6	No	7.9
CFR	111	245	262	169	197	6	No	26.3
2	119	279	298	190	222	6	No	37.1
1	71	181	193	103	137	5	No	18.6
10-Year Flood								
SR	126	191	204	198	180	9	Yes	3.0
NR	30	56	60	60	51	15	Yes	1.0
6	62	115	123	115	104	16	Yes	3.9
5	57	116	124	113	102	9	No	5.3
4	81	176	188	147	148	9	No	8.8
3	87	180	192	137	149	6	No	6.0
CFR	86	164	175	131	139	6	No	5.3
2	93	201	215	148	164	6	No	15.2
1	58	137	147	87	107	5	No	7.1
2-Year Flood								
SR	42	42	45	66	49	9	Yes	1.2
NR	11	14	15	22	15	15	Yes	0.3
6	23	28	30	42	31	16	Yes	0.7
5	26	35	37	51	37	9	Yes	1.2
4	28	39	42	49	39	9	Yes	1.0
3	22	29	31	35	29	6	Yes	0.6
CFR	17	18	20	25	20	6	Yes	0.4
2	24	32	34	38	32	6	Yes	0.7
1	18	26	28	28	25	5	Yes	0.5
MPM = Meyer-Peter, Muller Yang = Yang's Incipient Motion			CBV = Competent Bottom Velocity Shield = Shield's Method					

Sonoran Wash. As shown in Table 5-15, an armor layer forms at a relatively shallow depth on the bed of Sonoran Wash during the 2-year flood. However, armor layers are unlikely to form during floods larger than the 2-year event. For the 10- and 100-year events the depth of scour and duration of flow required to form an armor layer is too great to be effective at limiting scour. However, field evidence suggests that some of the boulder riffles in Reaches 5 and 6 are coarser than the reach-averaged sediment distribution and will be armored, at least for the 10-year flood. Field evidence also indicates that much of the coarsest sediment observed on the bed of Sonoran Wash has been transported during past floods.

Table 5-15. Skunk Creek Watercourse Master Plan Armoring Analysis Results – Sonoran Wash								
Reach	Methodology – Critical Armor Diameter (mm)				Average Critical Diam. (mm)	Field D50 (mm)	Armor Layer Likely?	Depth to Armor (ft.)
	MPM	CBV	Yang	Shield				
100-Year Flood								
6	73	152	163	141	132	19	No	14.2
5	81	177	189	157	151	19	No	11.9
4	77	160	171	149	139	19	No	17.7
3	72	157	167	134	132	19	No	11.1
2	63	148	159	112	120	5	No	9.7
1	30	118	126	59	83	4	No	39.5
10-Year Flood								
6	66	133	142	129	118	19	No	7.7
5	71	150	161	139	130	19	No	8.0
4	69	138	148	134	122	19	No	10.5
3	65	138	147	123	118	19	No	5.9
2	54	122	131	96	101	5	No	15.2
1	29	105	112	58	76	4	No	42.1
2-Year Flood								
6	17	19	20	33	22	19	Yes	0.4
5	17	21	23	34	24	19	Yes	0.4
4	20	25	27	40	28	19	Yes	0.5
3	19	24	26	35	26	19	Yes	0.4
2	16	24	25	29	24	5	Yes	0.7
1	9	19	20	18	16	4	No	5.2

MPM = Meyer-Peter, Muller CBV = Competent Bottom Velocity
Yang = Yang's Incipient Motion Shield = Shield's Method

Conclusions. The following conclusions can be drawn from the armoring analysis results summarized in Tables 5-14 and 5-15:

- The channel bed scour depth is probably limited by armoring during frequent flows and small floods, but the average bed material is too small to prevent scour during large flood events.
- The channel bed material is mobile, and will be transported during moderate to large flood events. Cobble and boulder transport should be considered in the sediment routing analysis.
- The depth of the inactive clay-rich layer of alluvium observed in the channel soil pits is generally shallower than the depth required to form an armor layer for the 10- and 100-year events. Therefore, scour is probably limited by factors other than formation of an armor layer.
- Soil profiles observed in the channel pits were not significantly more coarse-grained than the material exposed on the surface, although the finest grain sizes generally were not exposed directly on the surface. That is, effective armor layers were not observed in the field at the soil pits.

Table 5-19. Skunk Creek Watercourse Master Plan Comparison of Armoring, Scour, and Equilibrium Slope Estimates – Skunk Creek								
1	2	3	4		5		6	7
Reach	Depth to Armor (ft)	Scour Depth (ft)	Stable Slope		Slope Adjustments		Armor v. Scour	Armor v. Slope
			Regime	Actual	1000 ft.	5000 ft.		
Q100								
SR	2.6	13.3	0.0097	0.0130	-3.3	-16.3	Yes	Yes
NR	1.6	11.7	0.0104	0.0062	4.2	20.8	Yes	N/A
6	5.8	12.1	0.0104	0.0093	1.1	5.6	Yes	N/A
5	15.0	13.7	0.0069	0.0097	-2.8	-13.8	No	No
4	12.7	12.7	0.0066	0.0085	-2.4	-12	No	No
3	7.9	2.3	0.0056	0.0082	-2.6	-12.9	No	No
CFR	26.3	2.9	0.0053	0.0054	-0.1	-0.6	No	No
2	37.1	2.5	0.0055	0.0080	-2.5	-12.6	No	No
1	18.6	13.4	0.0047	0.0057	-1	-4.8	No	No
Q10								
SR	3.0	11.8	0.0107	0.0130	-2.3	-11.3	Yes	Yes
NR	1.0	8.8	0.0105	0.0062	4.3	21.3	Yes	N/A
6	3.9	9.6	0.0106	0.0093	1.3	6.5	Yes	N/A
5	5.3	10.8	0.0067	0.0100	-3.3	-16.5	Yes	N/A
4	8.8	10.2	0.0067	0.0085	-1.8	-9.2	Yes	No
3	6.0	1.7	0.0058	0.0082	-2.4	-11.9	No	No
CFR	5.3	1.6	0.0059	0.0054	0.5	2.3	No	N/A
2	15.2	1.9	0.0056	0.0080	-2.3	-11.7	No	No
1	7.1	11.3	0.0048	0.0057	-0.9	-4.3	Yes	No
Q2								
SR	1.2	3.2	0.0172	0.0130	4.3	21.3	Yes	N/A
NR	0.3	4.4	0.0131	0.0062	6.9	34.7	Yes	N/A
6	0.7	3.2	0.0138	0.0093	4.5	22.4	Yes	N/A
5	1.2	4.0	0.0095	0.0100	-0.5	-2.6	Yes	Yes
4	1.0	3.0	0.0090	0.0085	0.5	2.6	Yes	N/A
3	0.6	0.4	0.0085	0.0082	0.4	1.8	No	N/A
CFR	0.4	0.3	0.0087	0.0054	3.3	16.6	No	N/A
2	0.7	0.5	0.0079	0.0080	0.0	-0.1	No	N/A
1	0.5	3.2	0.0078	0.0057	2.2	10.9	Yes	N/A

Notes:

1. No = scour not limited by armoring
2. Yes = scour limited by formation of armor layer
3. N/A = not applicable, aggradation is predicted (no long-term scour)
4. Armor v. Scour: compare column 3 to 1, i.e. will scour will be limited by armoring?
5. Armor v. Slope: compare column 5 to 1, i.e. will long-term scour be limited by armoring?

Sonoran Wash. As shown in Table 5-20, armoring would have no impact on long-term slope adjustments since aggradation (last column) is predicted for most of Sonoran Wash. In reach 2, where some long-term degradation is predicted for the 10- and 100-year events, armoring would not prevent the possible long-term bed elevation change. Short term scour will be prevented by armoring in reaches 2 to 6 during a 2-year event, in reaches 3, 5 and 6 in the 10-year event, and in reaches 2, 3 and 5 during a 100-year event. The reaches of Sonoran Wash that cannot limit scour by armoring will be most susceptible to erosion caused by bed scour during floods.

Table 5-20. Skunk Creek Watercourse Master Plan
Comparison of Armoring, Scour, and Equilibrium Slope Estimates – Sonoran Wash

1	2	3	4		5		6	7
Reach	Depth to Armor (ft)	Scour Depth (ft)	Stable Slope		Slope Adjustments		Armor v. Scour	Armor v. Slope
			Regime	Actual	1000 ft.	5000 ft.		
Q100								
6	14.2	10.4	0.0127	0.0084	4.3	21.6	No	N/A
5	11.9	12.0	0.0124	0.0069	5.6	27.8	Yes	N/A
4	17.7	11.4	0.0116	0.0074	4.2	20.9	No	N/A
3	11.1	12.5	0.0105	0.0060	4.5	22.5	Yes	N/A
2	9.7	11.4	0.0058	0.0060	-0.2	-1.0	Yes	No
1	39.5	12.8	0.0041	0.0032	0.8	4.1	No	N/A
Q10								
6	7.7	9.3	0.0129	0.0084	4.5	22.4	Yes	N/A
5	8.0	10.8	0.0125	0.0069	5.6	28.2	Yes	N/A
4	10.5	10.1	0.0117	0.0074	4.3	21.6	No	N/A
3	5.9	11.3	0.0106	0.0060	4.6	23.2	Yes	N/A
2	15.2	10.3	0.0058	0.0060	-0.2	-1.0	No	No
1	42.1	10.7	0.0042	0.0032	1.0	4.8	No	N/A
Q2								
6	0.4	2.3	0.0184	0.0084	10.0	50.1	Yes	N/A
5	0.4	2.2	0.0171	0.0069	10.2	51.1	Yes	N/A
4	0.5	2.3	0.0165	0.0074	9.1	45.6	Yes	N/A
3	0.4	2.7	0.0146	0.0060	8.5	42.7	Yes	N/A
2	0.7	2.6	0.0080	0.0060	2.0	10.2	Yes	N/A
1	5.2	2.5	0.0056	0.0032	2.3	11.7	No	N/A

Notes:
1. No = scour not limited by armoring
2. Yes = scour limited by formation of armor layer
3. N/A = not applicable, aggradation is predicted (no long-term scour)
4. Armor v. Scour: compare column 3 to 1, i.e. will scour will be limited by armoring?
5. Armor v. Slope: compare column 5 to 1, i.e. will long-term scour be limited by armoring?

Conclusion. The engineering analyses described in the preceding sections predict mixed trends of aggradation and degradation for Skunk Creek and Sonoran Wash. These mixed trends indicate that the streams are subject to erosive conditions during floods, and will experience scour and slope adjustments best depicted by the type of erosion and deposition documented in the recent historical record.

- Tributary confluences
- HEC-1 model concentration points
- Bridge or culvert crossings
- Areas of change in channel planimetric form

Defining the stream segments based on these geographic features seemed to incorporate the more subtle variations in geomorphic parameters such as bank height, channel pattern, floodplain width, and bank materials. Reaches near bridge and culvert crossings were considered as separate reaches to distinguish the hydraulic impacts of upstream flow contraction, acceleration through the structures, and downstream expansion from the natural characteristics of the less disturbed adjacent reaches. A supply reach was defined for each stream to account for the effects of upstream hydraulics and geomorphology on the study area.

The reaches defined for the Skunk Creek Watercourse Master Plan are listed in Table 2-15 and illustrated in Figure 2-16 and Exhibit 1. Field photographs showing typical conditions in each of the study reaches are shown in Figures 2-17 to 2-32.

Table 2-15. Skunk Creek Watercourse Master Plan
Stream Reach Designation

Reach Code	HEC-RAS Section		Description	Comment	
	D/S End	U/S End			
Skunk Creek					
Supply Reach	25.83	26.17	Upstream of New River Road	Upstream supply reach	
NR Bridge	25.63	25.78	New River Road Bridge	"New River" Reach	
6	23.87	25.56	New River Road to Cline Creek	"New River" Reach	
5	22.15*	23.55	Cline Creek to Rodger Creek	Q100	"Cline Creek"
	21.49	23.55		Q10 & Q2	"Rodger Creek"
4	18.74	22.08*	Rodger Creek to Skunk Tank	Q100	"Rodger Creek"
	18.74	21.41		Q10 & Q2	"Skunk Tank"
3	16.96	18.57	Skunk Tank to Carefree Highway	"Skunk Tank/Carefree" Reach	
CFH Bridge	16.86	16.87	Carefree Highway Bridge		
2	14.89	16.68	Sec. 14 to Carefree Highway	"Cutbank/Knoll" Reach	
1	13.00	14.74	CAP Canal to Sec.14	"Braided/Greasewood" Reach	
Sonoran Wash					
6	3.61	3.84	Upstream end of study reach	"Hackberry" Reach	
5	2.93	3.54	Tributary to double tributary	"Hackberry" Reach	
4	2.35	2.88	19th Ave to tributary	"Ironwood" Reach	
3	1.72	2.28	¼ section to 19th Ave	"Ironwood" Reach	
2	1.15	1.65	Dixileta Dr. to ¼ section	"Main Stem" Reach	
1	0.52	1.09	CAP to Dixileta Dr.	"Sandy" Reach	

Crossing #4 is in Reach 2

* FEMA FIS HEC-2 model shows addition of discharge from Rodger Creek at RM 22.79. The Reach 5–Reach 4 boundary differs for the 100-yr and the 2-yr & 10-yr models due to intermingling of flood waters from Rodger Creek further upstream in the 100-yr flood than for the 2- or 10-yr floods.

Ref. 7

APPENDIX F. METHODOLOGY SUPPORTING DOCUMENTATION

- Reproduced Portions of: *State Standards SSA 5-96* **(Reference 6)**
- Reproduced Portions of: *AFMA Scour Short Course* **(Reference 10)**
- Reproduced Portions of *Draft Hydrology Manual* **(Reference 5)**

Excerpts Reproduced From:
State Standards Attachment 5-96 (Reference 6)

Description:
The process described in the following pages was used as a guide to help determine the scour depths at the wash crossings.

Number of Pages: 13

GUIDELINE 2

Channel Degradation Estimation for Alluvial Channels in Arizona

Ref. 6

TABLE OF CONTENTS

Introduction

Channel degradation occurs within watercourses composed of erodible material, where local or general differentials in sediment transport capacity exist. Numerous factors control the short and long term degradation potential of channel reaches, including the size and cohesiveness of the material of which the channel is composed, the vegetation type and density in the channel, the hydraulic characteristics generated within the channel under flood events, and the existence of flow redirection or concentration structures within the channel. A key factor, however, is the amount of variation in channel properties from reach to reach. A channel reach attempts to adjust to conditions imposed on it by factors occurring up- and downstream; thus, the more uniform the channel is along the system under study, the less the potential exists for channel degradation to be a significant factor. Natural and man-made discontinuities along the system can create local increases in sediment transport potential, which often result in local degradation of the channel. System-wide disturbances, such as those associated with urbanization of the watershed or dam construction, have more far reaching impact, as the entire channel is forced to adjust to a change in sediment supply.

This document presents procedures that may be used for estimation of channel degradation in unlined watercourses within Arizona. Three levels of procedures are provided, with data requirements, procedural complexity, and accuracy of results all increasing as the analysis level is incremented. The Level I approach provides an initial estimate of local channel degradation potential for generally stable, natural channel conditions. The resulting initial estimate may be reduced through use of the more rigorous Level II methodologies. Level III procedures are outlined for situations that warrant more detailed channel degradation determination.

Procedure

General

Three levels of procedures for estimation of channel degradation depth are described in the following paragraphs. The first level of analysis provides an initial estimate of the potential scour depth to consider for design of structures to be placed near a streambed or along the banks of a channel. This first level of analysis is recommended only for channel reaches that are expected to be in general balance with the surrounding system -- i.e. no major disturbances (dams, bridges, encroachments, etc..) are evident in the site vicinity -- and where the desire is to establish a "safe" scour depth to allow for the concentration of flows that can naturally occur within channels composed of erodible material. The Level II procedures provided are methods for demonstrating the site specific limits to erosion potential, involving computations which require local hydraulic information and sediment size distributions, or historical evidence of channel performance. The third level of procedures outlined will provide more definitive determination of channel stability in the reaches under study. This level of analysis is recommended in areas where local flow characteristics are complex, where the channel has been redirected or otherwise modified by acts of man, or where the safety of local paralleling or crossing structures is of high concern.

Level I

This level of analysis requires the following information :

> Peak discharge associated with the 100-year flood (Q_{100}). May be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-96), USGS regression equations, or other appropriate local or more detailed methods.

The total scour depth, d_s, is the combination of general degradation and long term degradation and can be computed as follows:

$$d_s = d_{gs} + d_{lts}$$

where:

d_s = Total scour depth, in feet
d_{gs} = General degradation, in feet
d_{lts} = Long term degradation, in feet

General degradation can be computed as follows:

$$d_{gs} = 0.157(Q_{100})^{0.4} \quad \text{for straight channel reaches.}$$

and

$$d_{gs} = 0.219(Q_{100})^{0.4} \quad \text{for channel reaches with curvature.}$$

The second equation will give the worst-case scour for channel curvature, and is not recommended unless significant curvature is evident along the channel reach.

Long term degradation can be computed as follows:

$$d_{lts} = 0.02(Q_{100})^{0.6}$$

This equation for long term degradation should only be used when no downstream controls exist within the channel system.

The total scour depth, d_s, should be applied to the lowest point in the local cross section for determination of the elevation to which scour will occur.

For Level I, the minimum total scour depth, d_s, shall be 3 feet.

Level II

The Level II approaches presented below may be used to demonstrate the ability of the existing channel system to resist degradation, and to justify a lesser burial requirement than that computed using the Level I equations.

Erodibility evaluation

Three procedures for determination of the erodibility of local channel material under computed hydraulic conditions are presented in the ADWR's State Standard for Lateral Migration Setback Allowance for Riverine Floodplains in Arizona. These procedures are: (1) the allowable velocity approach; (2) the tractive stress approach; and, (3) the tractive power approach. One or more of these procedures can be used to demonstrate the adequacy of the material of which the channel is composed to resist the erosive action of the flow under 100 year flow conditions.

Armoring potential evaluation.

An evaluation of relative channel stability can be made by evaluating incipient motion parameters and determining armoring potential. The definition of incipient motion is based on the critical or threshold condition where hydrodynamic forces acting on a grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. For given hydrodynamic forces, or equivalently for a given discharge, incipient motion conditions will exist for a single particle size. Particles smaller than this will be transported downstream and particles equal to or larger than this will remain in place.

The Shields diagram (Figure 1) may be used to evaluate the particle size at incipient motion for a given discharge. The Shields diagram was developed through measurements of bed-load transport for various values of the Shields parameter (y axis of Figure 1) at least twice as large as the critical value, and extrapolated to the point of vanishing bed load. In the turbulent range, where most flows of practical engineering interest occur, this diagram suggests that the Shields parameter is independent of flow conditions and the following relationship is established:

$$D_c = \tau_p / [0.047 (\gamma_s - \gamma)]$$

where D_c is the diameter of the sediment particle for conditions of incipient motion, τ_p is the boundary shear stress acting on the particle, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient. Any consistent set of units may be used with this equation. Typical values for γ_s and γ in English units are 165 lb/ft^3 and 62.4 lb/ft^3, respectively.

For computation of shear stress on the boundary particles, the following relations are recommended:

$$\tau_p = \tfrac{1}{8} f \rho V^2$$

$$f = 116.5\, n^2 / R^{1/3}$$

$$n = D_{90}^{1/6} / 26$$

where
- f = friction factor (dimensionless)
- ρ = density of the water
- V = flow velocity
- n = Manning resistance value
- R = hydraulic radius of the channel
- D_{90} = particle size which is larger than 90 percent of all sizes

The units of the above are as follows: τ is in lb/ft^2; ρ is in slugs/ft^3 (typically 1.94 slugs/ft^3); V is in feet per second; and R is in feet. The relation presented above relating the Manning n value to the D_{90} of the local bed material yields the resistance factor associated with the particle roughness only, and assumes D_{90} is in meters.

The shear stress computed from the above equation should be increased in areas of channel curvature using Figure 2.

The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, and increasing number of non-moving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor," the entire bed surface. When fines can no longer be leached from the underlying bed, degradation is arrested.

Potential for development of an armor layer can be assessed using Shields' criteria for incipient motion and a representative bed-material composition. In this case a representative bed material composition is that which is typical of the depth of anticipated degradation. Using the equation presented above, the incipient-motion particle size can be computed for a given set of hydraulic conditions. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur. Armoring is probable when the particle size computed from the above equation is equal to or smaller than the D_{90} size.

After determination of the percentage of the bed material equal to or larger than the armor particle size (D_c), the depth of scour necessary to establish an armor layer (ΔZ_a) can be calculated from the following equation:

$$\Delta Z_a = y_a \, [(1/P_c) - 1]$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_c), depending on the value of D_c. Field observations suggest that a relatively stable armoring conditions requires a minimum of two layers of armoring particles.

Channel profile history comparison

This procedure, applicable where sufficient data is available, relies on the historical record for indication of the degradation potential of the local channel reach. This procedure should be used to demonstrate the stable or aggrading tendency of the reach in question, rather than to estimate potential degradation depths. Given a reach of channel with successive record of channel profile changes, associated with hydrologic information for the events occurring between surveys, the reviewer can determine the trend of the channel changes and assess the likelihood of trend continuation for the future. Where the stable or aggradational trend is obvious, and no changes are anticipated in the channel system to alter the on-going trend, a lesser degradation allowance than that provided under the Level I guidelines would be reasonable.

Grade stabilization measures adequacy analysis

Grade stabilization measures of some form may be proposed or already in place which may act to limit the degradation potential of the watercourse of concern. In some areas within Arizona, procedures are in place for assessment of the adequacy of channel stabilization measures. For areas without standardized procedures, two references are recommended which detail evaluation procedures:

> Design Manual for Engineering Analysis of Fluvial Systems, Arizona Department of Water Resources, 1985.
>
> Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson Department of Transportation, Engineering Division, 1989.

Level III

This level of analysis involves modeling the hydraulic and sediment transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area of concern. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for establishment of degradation potential be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

Works Cited

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-96", July 1996.

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Lateral Migration Setback Allowance for Riverine Floodplains in Arizona - State Standard 5-96", September 1996.

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Blench, T., "Mobile-Bed Fluviology", Edmonton: University of Alberta Press, 1969.

City of Tucson Department of Transportation, Engineering Division. "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona". December 1989.

Lacey, G., "Stable Channels in Alluvium," in Proceedings of the Institution of Civil Engineers, Vol. 229, 1930.

Example Application

Example 1: Proposed Syphon Crossing of an Earthen Channel

- **Problem Statement.** A natural earthen channel traverses a site where an irrigation channel is being constructed. The watershed contributing to the earthen channel upstream of the site is 700 acres in area. A syphon is proposed to convey irrigation water across the channel.

- **Objective.** Determine the burial depth for the proposed syphon.

Level I Analysis

A 100-year peak discharge value of 530 cfs was determined from local hydrology methodology. The channel in the site vicinity has 2:1 side slopes and a bottom width of 15 feet. The proposed crossing site is at a mild bend in the channel. A sieve analysis of the local bed material yields a median grain size D_{50} = 1.0 mm = 0.0033 feet.

Calculations:

General degradation, $d_{gs} = 0.157(530)^{0.4} = 1.93$ feet

Long term degradation, $d_{lts} = 0.02(530)^{0.6} = 0.86$ feet

Total scour, d_s = 1.93 feet + 0.86 feet = 2.79 feet

Since the total scour calculated is less than the recommended minimum of 3 feet, use a total scour depth of 3.0 feet.

Level II Analysis

Further evaluation is desired to investigate the potential for reducing the burial depth indicated through application of the Level I procedure. Although no historical data is available for determination of the local aggradation/degradation trends of the earthen channel, the erodibility and armoring potential of the existing channel material can be checked using the recommended Level II procedures. The site specific hydraulic and grain size information is collected to check if erosion of the channel would be naturally limited. The channel slope in the site vicinity is estimated from USGS quadrangle maps at 0.010 feet/foot, and the Manning n value for total channel resistance is estimated at 0.030.

Using normal depth procedures, the hydraulic characteristics of the local channel under 100-year flood conditions are determined:

Flow Depth = 3.0 feet
Flow Velocity = 8.4 feet/second

The sieve analysis of the local channel material sample yields the following information:

D_{90} = 55 mm = 0.180 ft = 0.217 inches
D_{75} = 4 mm = 0.013 ft = 0.16 inches
D_{65} = 1.9 mm = 0.0062 ft = 0.07 inches

Calculations:

Erodibility Evaluation (using procedures and figures provided in Attachment 1 to this State Standard)

(1) Allowable velocity approach, assuming sediment laden flow

Entering Figure 1 with D_{75} = 4 mm yields a basic velocity of 4.0 ft/sec.

In this case, we are concerned with erosion of the channel invert in a reach containing only a mild bend, so the correction factors for channel curvature reduces to 1.0. The correction factor for side slope, which must be considered for evaluating the erodibility of the channel banks, is not applied in this case.

Entering Figure 4 with Depth = 3.0 feet yields C_c = 1.01

Maximum allowable velocity = (4.0)(1.0)(1.01) = 4.0 ft/sec

Since the computed velocity of 8.4 ft/sec exceeds the maximum allowable velocity, erosion may be expected to occur.

(2) Tractive stress approach

Since D_{75} is less than 0.25 inches, the reference tractive stress method is used;

Assuming a water temperature of 60° F, the kinematic viscosity (ν) = 0.0000121 ft²/sec, and the density (ρ) = 1.94 slugs/ft³

Compute $V^3/(g\nu S_e) = 1.52 \times 10^8$

Compute $V/(gD_{65}S_e)^{1/2} = 188$

From Figure 9, $V/(\tau/\rho)^{1/2} = 18.2$

Solving the above equation yields $\tau = 0.41$ lb/ft².

No correction factor for side slope is applied, and the correction factor for channel curvature reduces to 1.0 for a mild bend.

From Figure 12, Curve 1 (for high sediment content), the allowable tractive force is 0.09 lb/ft². Since 0.09 is less than 0.41, the channel is erosive.

(3) Tractive power approach

An unconfined compressive strength (UCS) test of the saturated channel soils is performed, yielding a strength of 800 lb/ft³.

Assuming half of this strength for design purposes, UCS_{design} = 400 lb/ft³.

Compute tractive power = $V\tau_s = 3.44$

From Figure 13, the condition falls above the S-Line, indicating that the channel is erosive.

Armoring potential evaluation

D_{90}

Manning's n related to particle roughness = $[55/1000]^{1/6} / 26 = 0.024$

Channel flow area = [15+2(3.0)](3.0) = 63.0 square feet

Channel wetted perimeter = $15 + 2(3.0)(5)^{1/2} = 28.4$ feet

Hydraulic Radius = 63.0/28.4 = 2.22 feet

Friction factor = $f = 116.5\ (0.024)^2 / (2.22)^{1/3} = 0.051$

ρ Vel

Particle shear stress = $\tau_p = \frac{1}{8}(0.051)(1.94)(8.4)^2 = 0.87$ lb/ft²

Critical particle size = $D_c = .87/[0.047(165\text{-}62.4)] = 0.18$ feet = 54.9 mm

Since the critical particle size is essentially equal to D_{50}, armoring is a possibility.

Therefore, the percent of material greater than D_c = 54.9 mm is 10%

Armor thickness = $y_a = 2D_c = 0.36$ feet

Depth of degradation required for armoring to form:

$$\Delta Z_a = y_a\,[(1/P_c) - 1] = 0.36[(1/0.10) - 1] = 3.24 \text{ feet}$$

Since the depth required for armoring to occur exceeds the Level I burial depth, armoring will not control, and the recommended burial depth is the minimum allowable value of 3.0 feet.

Level III Analysis

The conclusions derived from the Level II analysis and the nature of the problem indicate that the Level III analysis would probably not be applied in this case. However, should the designer wish to proceed with the degradation investigation, a registered engineer with experience in sediment transport modeling could be employed for this purpose. The engineer would be expected to collect available historic information, document the historic planform changes to the watercourse under events of varying frequency, apply steady state hydraulic and sediment transport calculation procedures to determine the erosion/sedimentation characteristics of the local reach of channel, and, potentially apply a moveable boundary river simulation model to quantify the changes likely along the study reach under design event conditions.

46%

Excerpts Reproduced From:
AFMA Scour Short Course (Reference 10)

Description:
The process described in the following pages was used to estimate the general scour at the Emergency Spillway Wash (Crossing No. 2a).

Number of Pages: 7

- Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation in addition to the live-bed equation, and that the smaller calculated scour depth be used.

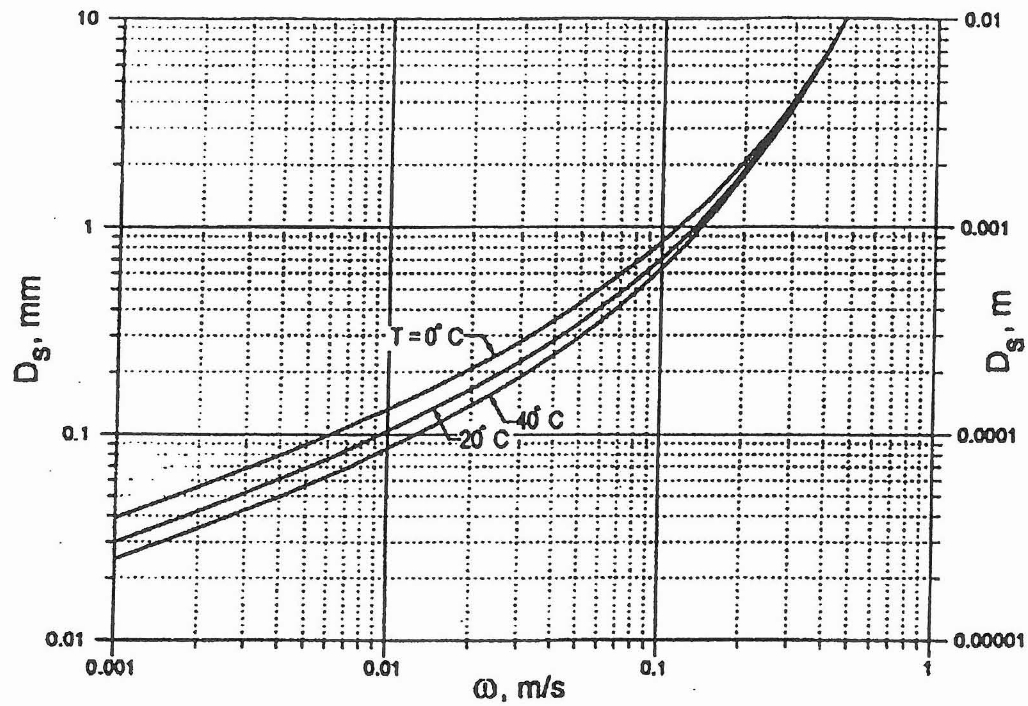


Figure 5.8 Fall Velocity of Sand-Sized Particles with Specific Gravity of 2.65 in Metric Units (HEC 18, 2001)

General Scour

Equations used by U.S. Bureau of Reclamation
[*Reference:* Pemberton and Lara, 1984]

Channel scour during peak flood flows (general scour)

- Category 1: Natural channel scour
- Category 2: Scour induced by structures in or adjacent to the channel

Classification of scour equation for various structure designs

- Type A: Natural channel for restrictions and bends
  a. Siphon crossing or any buried pipeline

b. Stability study of natural bank

c. Waterway for one-span bridge

- Type B: Bankline structuresAbutments to bridge or siphon crossing

    a. Bank slope protection such as riprap, etc.

    b. Spur dikes, groins, etc.

    c. Pumping plants and canal headworks

- Type C: Mid-channel structures

    a. Piling for bridge

    b. Piers for flume over river

    c. Power line footings

    d. River bed water intake structure

- Type D: Hydraulic structures across channel

    a. Dams and diversion dams

    b. Erosion controls

    c. Rock cascade drops, gabion controls, and concrete drops

Notes:

- U.S. Bureau of Reclamation practice is to compute scour by several methods, use judgement in averaging results or selecting appropriate method.

- Four methods for estimating general scour at constricted waterways adapted from Neill are considered proper approach for either design.

Field Measurements of Scour (Envelope Curve)

Method consists of observing or measuring actual scoured depths either at the river under investigation or a similar type river. Measurements are taken during high flow.

$$y_s = K\,(q)^{0.24}$$

where:

y_s = depth of scour below streambed, (ft)
K = 2.45 inch-pound units
q = unit water discharge, ($ft^3/s/ft$)

Qs going through channel (Bank to Bank) divided by Top width

Notes:

- Ephemeral, relatively steep, wide sand bed streams in southwestern U.S.
- D_{50} from 0.5 to 0.7 mm (coarse sand)
- Slopes form 0.004 to 0.008 ft

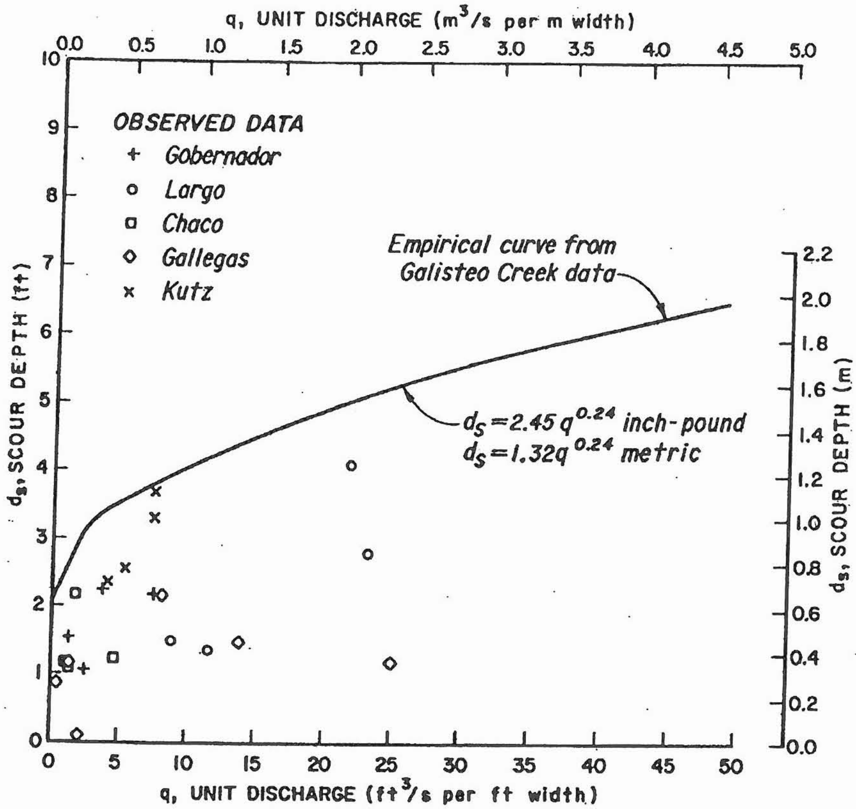


Figure 8. Navajo Indian Irrigation Project – Scour versus Unit Discharge (Pemberton and Lara, 1984)

Regime Equations – Neill's Approach (1973)

Based on field measurements in an incised reach:

$$y_f = y_i \; (q_f / q_i)^m$$

where:

y_f = scoured depth (general scour) below design floodwater level, (ft)
y_i = average depth at bankfull discharge in incised reach, (ft)
q_f = design flood discharge per unit width, ($ft^3/s/ft$)
q_i = bankfull discharge in incised reach per unit width, ($ft^3/s/ft$)
m = exponent varying from 0.67 for sand to 0.85 for coarse gravel

Regime Equations – Blench Equation (1969)

$$y_{f0} = q_f^{2/3} / F_{b0}^{1/3}$$

where:

y_{f0} = water depth for zero bed sediment transport, (ft)
q_f = design discharge per unit width, ($ft^3/s/ft$)
F_{b0} = Blench's "zero bed factor" in ft/s^2 from Figure 9

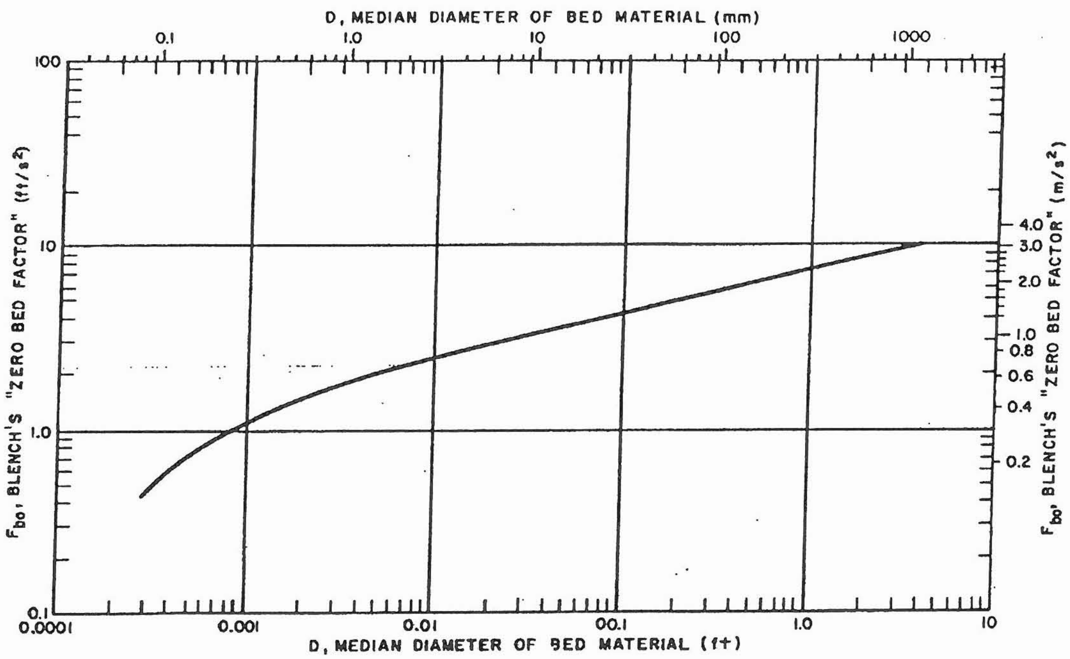


Figure 9. Chart for Estimating F_{b0} – After Blench (Pemberton and Lara, 1984)

Regime Equations – Lacey's Equation (1930)

$$y_m = 0.47\,(Q / f)^{1/3}$$

where:

y_m = mean water depth ~~at design~~ discharge, (ft)
Q = design discharge, (ft^3/s)
f = Lacey's silt factor = 1.76 $(D_m)^{1/2}$
D_m = mean grain size of bed material, (mm)

Regime Equations – Calculating Scour Depth

Calculating scour depth with the regime equations

- Accounts for probable concentration of flood flows in some portion of the natural channel

- Depth of scour below streambed (Figure VII-14) [general scour plus bend scour and thalweg formation]:

$y_s = Z\, y_f$ Neill

$y_s = Z\, y_{f0}$ Blench

$y_s = Z\, y_m$ Lacey

where:

y_s = depth of scour below low point in existing stream bed, in units of y_f, y_m, and y_{f0}

Z = multiplying factor from Table 7

Table 7. Multiplying Factors, Z, for Use in Scour Depths by Regime Equation (Pemberton and Lara, 1984)

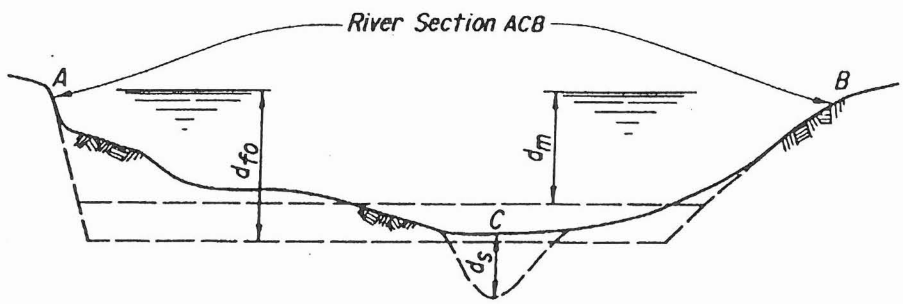
Radius of curvature

R/W = 10 - 30

>10

Top width.

Condition	Value of Z		
	Neill $d_s = Z\, d_f$	Lacey $d_s = Z\, d_m$	Blench $d_s = Z\, d_{fo}$
Equation Types A and B			
Straight reach	0.5	0.25	
Moderate bend	0.6	0.5	1/ 0.6
Severe bend	0.7	0.75	
Right angle bends		1.0	1.25
Vertical rock bank or wall		1.25	



NOTE: $d_{fo} > d_f > d_m$. Point C is low point of natural section.

Figure 10. Sketch of Natural Channel Scour by Regime Method (Pemberton and Lara, 1984)

Mean Velocity from Field Measurements

Procedure:

- Obtain at least 4 surveyed cross sections
- Obtain y_m from water surface profile
- $y_s = Z\, y_m$ using Lacy Z values (see Table 7) for general and bend scour

Competent or Limiting Velocity

Assumes scour will occur until mean velocity is less than velocity for significant bed material movement (general scour)

Empirical curves , Figure 12, derived by Neill for competent velocity with sand or coarser bed material (>0.30 mm) represent a combining of regime criteria, Shields criterion for material >1.0 mm, and a mean velocity formula relating mean velocity to the shear velocity.

Competent velocities for erosion of cohesive materials recommended by Neill are given in Table 8.

$$y_s = y_m\,(V_m / V_c - 1)$$

where:

y_s = scour depth below streambed, (ft)
y_m = mean depth, (ft)
V_c = competent mean velocity, (ft/s)
V_m = mean velocity, (ft/s)

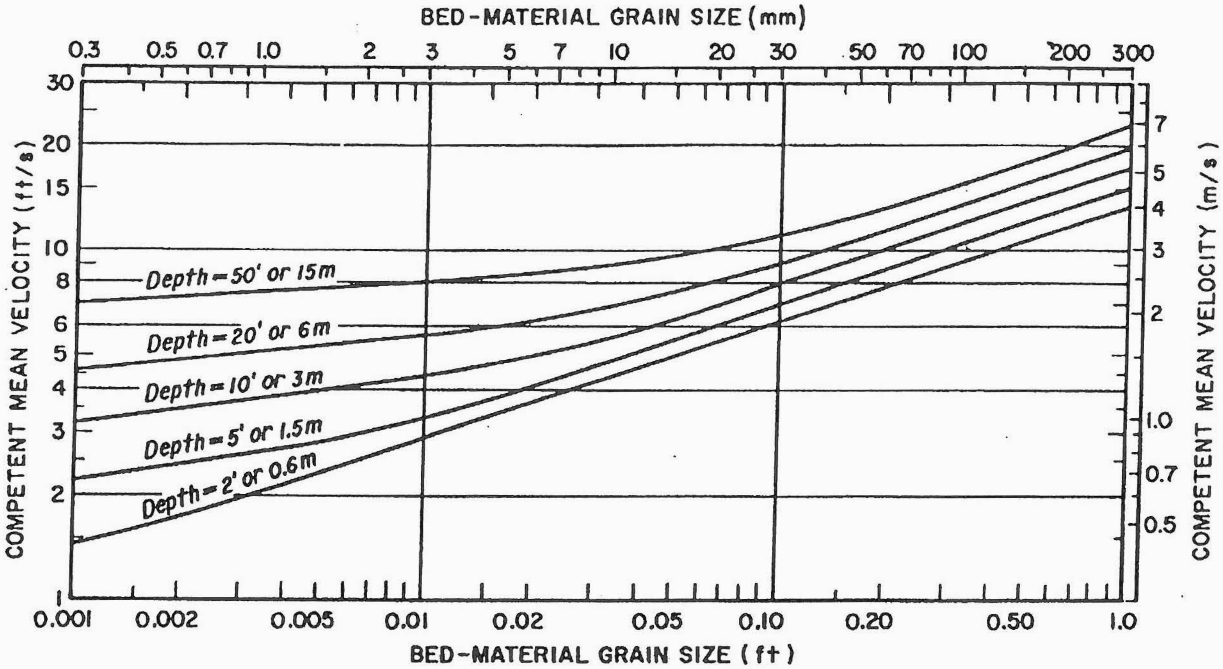


Figure 12. Suggested Competent Mean Velocities for Significant Bed Movement of Cohesionless Materials, in Terms of Grain Size and Depth of Flow – Neill, 1973 (Pemberton and Lara, 1984)

Table 8. Tentative Guide to Competent Velocities for Erosion of Cohesive Materials* (Pemberton and Lara, 1984)

Depth of flow		Competent mean velocity					
		Low values - easily erodible material		Average values		High values - resistant material	
ft	m	ft/s	m/s	ft/s	m/s	ft/s	m/s
5	1.5	1.9	0.6	3.4	1.0	5.9	1.8
10	3	2.1	0.65	3.9	1.2	6.6	2.0
20	6	2.3	0.7	4.3	1.3	7.4	2.3
50	15	2.7	0.8	5.0	1.5	8.6	2.6

* Notes: (1) This table is to be regarded as a rough guide only, in the absence of data based on local experience. Account must be taken of the expected condition of the material after exposure to weathering and saturation. (2) It is not considered advisable to relate the suggested low, average, and high values to soil shear strength or other conventional indices, because of the predominating effects of weathering and saturation on the erodibility of many cohesive soils.

Excerpts Reproduced From:
Draft Hydrology Manual (Reference 5)

Description:
The process described in the following pages was used to estimate the 100-year peak flow at the Emergency Spillway Wash (Crossing No. 2a).

Number of Pages: 10

3 RATIONAL METHOD

TABLE OF CONTENTS

3.1 GENERAL

The Rational Method was originally developed to estimate runoff from small areas and its use should be generally limited to those conditions. For the purposes of this manual, its use should be limited to areas of up to 160 acres. In such cases, the peak discharge and the volume of run-off from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development are to be retained. This is the required criteria for unin-corporated areas of Maricopa County. For incorporated areas, the 100-year, 2-hour duration storm is the minimum recommended criteria, however the Policies and Standards manual for the jurisdictional entity should be referenced for any variations. If the development involves channel routing, the procedures given in Chapters 4 through 6 should be used, since the peak generated by the Rational Method cannot be directly routed.

3.2 RATIONAL EQUATION

The Rational Equation relates rainfall intensity, a runoff coefficient and the watershed size to the generated peak discharge. The following shows this relationship:

$$Q = CiA \qquad (3.1)$$

where:

Q = the peak discharge, in cfs, from a given area.

C = a coefficient relating the runoff to rainfall.

i = average rainfall intensity, in inches/hour, lasting for a T_c.

T_c = the time of concentration, in hours.

A = drainage area, in acres.

The Rational Equation is based on the concept that the application of a steady, uniform rainfall intensity will produce a peak discharge at such a time when all points of the watershed are contributing to the outflow at the point of design. Such a condition is met when the elapsed time is equal to the time of concentration, T_c, which is defined to be the floodwave travel time from the most remote part of the watershed to the point of design. The time of concentration should be computed by applying the following equation developed by Papadakis and Kazan (1987):

$$T_c = 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38} \tag{3.2}$$

where:

T_c = time of concentration, in hours.

L = length of the longest flow path, in miles.

K_b = watershed resistance coefficient (see Figure 3.1, or Table 3.1).

S = watercourse slope, in feet/mile.

i = rainfall intensity, in inches/hour.*

*It should be noted that i is the "rainfall excess intensity" as originally developed. However, when used in the Rational Equation, rainfall intensity and rainfall excess intensity provide similar values because of the hydrologic characteristics of small, urban watersheds which result in minimal rainfall loss. This is because of the extent of imperviousness associated with urban watersheds and the fact that the time of concentration is usually very short.

Rational Method runoff coefficients for various natural conditions and land uses are provided in Table 3.2.

Table 3.1
EQUATION FOR ESTIMATING K_B IN THE T_C EQUATION

$$K_b = m \log A + b$$

Where A is drainage area, in acres

Type	Description	Typical Applications	Equation Parameters	
			m	b
A	Minimal roughness: Relatively smooth and/or well graded and uniform land surfaces. Surfaces runoff is sheet flow.	Commercial/industrial areas Residential area Parks and golf courses	-0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	-0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths.	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	-0.025	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	-0.030	0.20

Figure 3.1
RESISTANCE COEFFICIENT K_b
AS A FUNCTION OF WATERSHED SIZE AND SURFACE ROUGHNESS CHARACTERISTICS

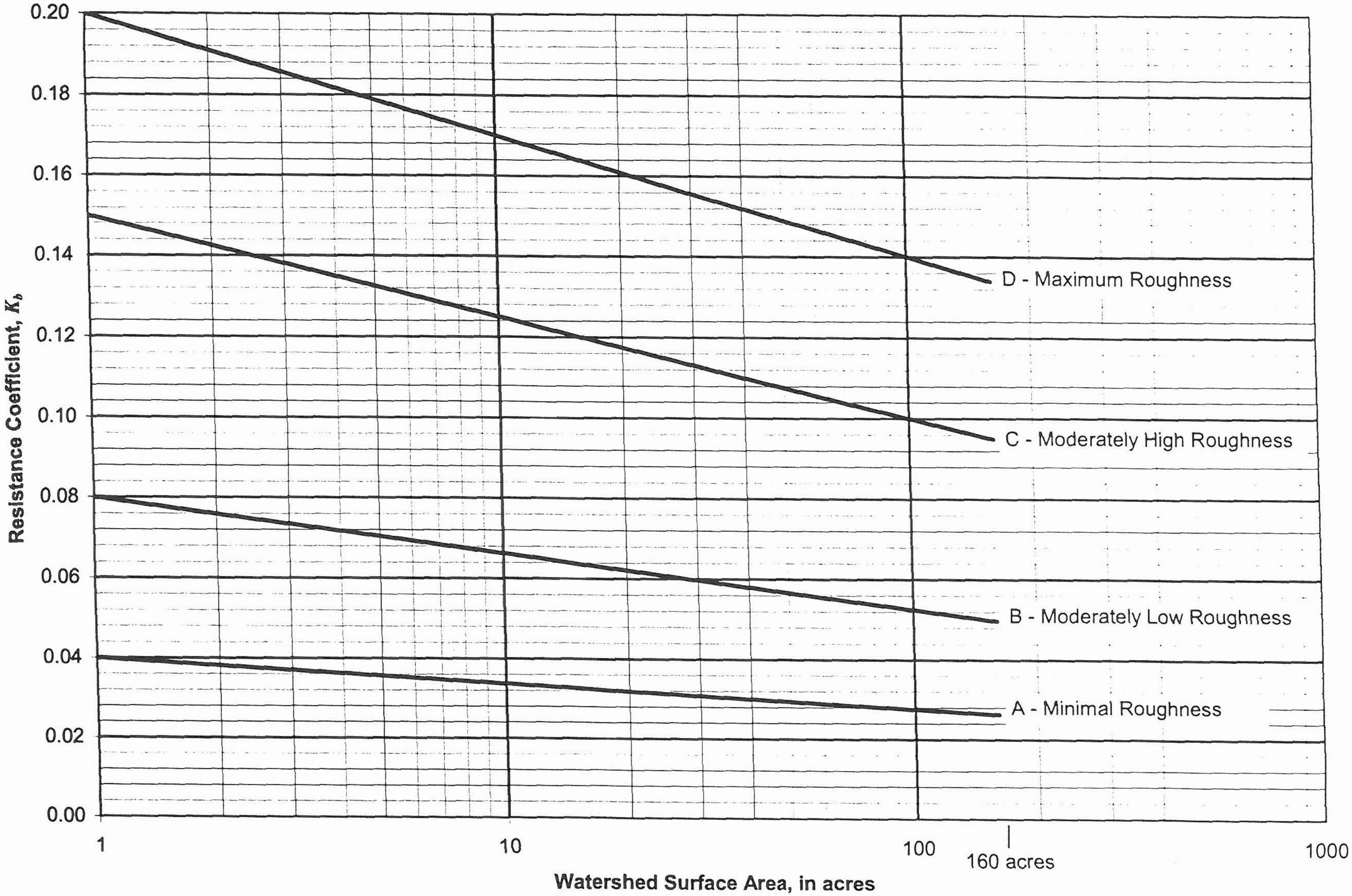


Table 3.2
RUNOFF COEFFICIENTS FOR MARICOPA COUNTY

Land Use Code	Land Use Category	Runoff Coefficients by Storm Frequency[1, 2]							
		2-10 Year		25 Year		50 Year		100 Year	
		min	max	min	max	min	max	min	max
VLDR	Very Low Density Residential[3]	0.33	0.42	0.36	0.46	0.40	0.50	0.41	0.53
LDR	Low Density Residential[3]	0.42	0.48	0.46	0.53	0.50	0.58	0.53	0.60
MDR	Medium Density Residential[3]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.82
MFR	Multiple Family Residential[3]	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
I1	Industrial 1[3]	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
I2	Industrial 2[3]	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
C1	Commercial 1[3]	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
C2	Commercial 2[3]	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
P	Pavement and Rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
GR	Gravel Roadways & Shoulders	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
AG	Agricultural	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
LPC	Lawns/Parks/Cemeteries	0.10	0.25	0.11	0.28	0.12	0.30	0.13	0.31
DL1	Desert Landscaping 1	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
DL2	Desert Landscaping 2	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NDR	Undeveloped Desert Rangeland	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NHS	Hillslopes, Sonoran Desert	0.40	0.55	0.44	0.61	0.48	0.66	0.50	0.69
NMT	Mountain Terrain	0.60	0.80	0.66	0.88	0.72	0.95	0.75	0.95

Notes:

1. Runoff coefficients for 25-, 50- and 100-Year storm frequencies were derived using adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the 2-10 Year values with an upper limit of 0.95.
2. The ranges of runoff coefficients shown for urban land uses were derived from lot coverage standards specified in the zoning ordinances for Maricopa County.
3. Runoff coefficients for urban land uses are for lot coverage only and do not include the adjacent street and right-of-way, or alleys.

Table 3.3
RUNOFF COEFFICIENT DESCRIPTIONS FOR MARICOPA COUNTY

Land Use Code	Land Use Category Description
VLDR	40,000 sq. feet and greater lot size
LDR	12,000 – 40,000 sq. feet lot size
MDR	6,000 – 12,000 sq. feet lot size
MFR	1,000 – 6,000 sq. feet lot size
I1	Light and General
I2	General and Heavy
C1	Light, Neighborhood, Residential
C2	Central, General, Office, Intermediate
P	Asphalt and Concrete, Sloped Rooftops
GR	Graded and Compacted, Treated and Untreated
AG	Tilled Fields, Irrigated Pastures, slopes < 1%
LPC	Over 80% maintained lawn
DL1	Landscaping with impervious under treatment
DL2	Landscaping without impervious under treatment
NDR	Little topographic relief, slopes < 5%
NHS	Moderate topographic relief, slopes > 5%
NMT	High topographic relief, slopes > 10%

3.3 ASSUMPTIONS

Application of the Rational Equation requires consideration of the following:

1. The peak discharge rate corresponding to a given intensity would occur only if the rainfall duration is at least equal to the time of concentration.

2. The calculated runoff is directly proportional to the rainfall intensity.

3. The frequency of occurrence for the peak discharge is the same as the frequency for the rainfall producing that event.

4. The runoff coefficient increases as storm frequency decreases.

3.4 VOLUME CALCULATIONS

Volume calculations should be done by applying the following equation:

$$V = C\left(\frac{P}{12}\right)A \tag{3.3}$$

where:

V = calculated volume, in acre-feet.

C = runoff coefficient from Table 3.2.

P = rainfall depth, in inches.

A = drainage area, in acres.

In the case of volume calculations for stormwater storage facility design, P equals the 100-year, 2-hour depth, in inches, as discussed in Section 2.2, and is determined from Figure A.2 of Appendix A, Section 1.

3.5 LIMITATIONS

Application of the Rational Method is appropriate for watersheds less than 160 acres in size. This is based on the assumption that the rainfall intensity is to be uniformly distributed over the drainage area at a uniform rate lasting for the duration of the storm. The Maricopa County Unit Hydrograph Procedure described in Chapter 5 may also be used for areas less then 160 acres where hydrograph routing is desired, or, in cases where the Rational Method assumptions do not apply.

3.6 APPLICATION

The Rational Method can be used to calculate the generated peak discharge from drainage areas less than 160 acres. Procedures for calculating peak discharge are provided in the following sections. Notes and general guidance in the application of these procedures along with a detailed example are provided in Section 9.2.

3.6.1 Peak Discharge Calculation

1. Determine the area within the development boundaries.

2. Select the Runoff Coefficient, C from Table 3.2. If the drainage area contains subareas of different runoff characteristics, and thus different C coefficients, arithmetically area-weight the values of C.

3. Compute the depth-duration-frequency (D-D-F) statistics for the project site using the PREFRE program (see Section 2.2). Alternatively, if the project site lies within the Phoenix Metro area, then the I-D-F graph in Appendix B can be used to compute intensity.

4. Calculate the time of concentration. This is to be done as an iterative process.

   a. Determine the K_b parameter from Figure 3.1 or Table 3.1. If the drainage area contains subareas of different K_b values, arithmetically area-weight the values of K_b.

   b. Make an initial estimate of the duration and compute the intensity from the PREFRE output for the desired frequency. If the project site is within the Phoenix metro area, the I-D-F graph provided in Appendix B can be used as an alternative.

   c. Compute an estimated T_c using Equation (3.2). If the computed T_c is reasonably close to the estimated duration, then proceed to Step 5, otherwise repeat this step with a new estimate of the duration. The minimum T_c should not be less than 10-minutes.

5. Determine peak discharge Q by using the above value of i in Equation (3.1).

6. As an alternative to the above procedure, the DDMSW program may be used to calculate peak discharges.

3.6.2 Multiple Basin Approach

The Rational Method can be used to compute peak discharges at intermediate locations within a drainage area less than 160 acres in size. A typical application of this approach is a local storm drain system where multiple subbasins are necessary to compute a peak discharge at each proposed inlet location. Consider the schematic example watershed shown in Figure 3.2. A peak discharge is needed for all three individual subareas, subareas A and B combined at Concentration Point 1 and subareas A, B and C combined at Concentration Point 2.

1. Compute the peak discharge for each individual subarea using steps 1 through 5 from Section 3.6.1.

2. Compute the arithmetically area-weighted value of C for subareas A and B.

3. Follow step 4 from Section 3.6.1 to calculate the T_c for the combined area of subareas A and B at Concentration Point 1.

4. Compare the T_c values from subareas A and B to the T_c value for the combined area at Concentration Point 1. Compute the peak discharge at Concentration Point 1 using the *i* for the longest T_c from step 3. If the combined peak discharge is less than the discharges for the individual subareas, use the largest discharge as the peak discharge at Concentration Point 1. The design discharge SHOULD NOT INCREASE going downstream in a conveyance system unless storage facilities are used to attenuate peak flows.

5. Compute the arithmetically area-weighted value of C for subareas A, B and C.

6. Calculate the T_c for the combined area at Concentration Point 2 using the following two methods:

   Method 1 - Follow step 4 from Section 3.6.1 to calculate the T_c for the single basin composed of all three subareas.

   Method 2 - Compute the travel time from Concentration Point 1 to Concentration Point 2 using the continuity equation or other appropriate technique and hydraulic parameters for the conveyance path. Add the computed travel time for the conveyance path to the T_c from Concentration Point 1.

7. Compare the T_c values from Methods 1 and 2 as well as the T_c from subarea C and calculate the peak discharge at Concentration Point 2 as follows:

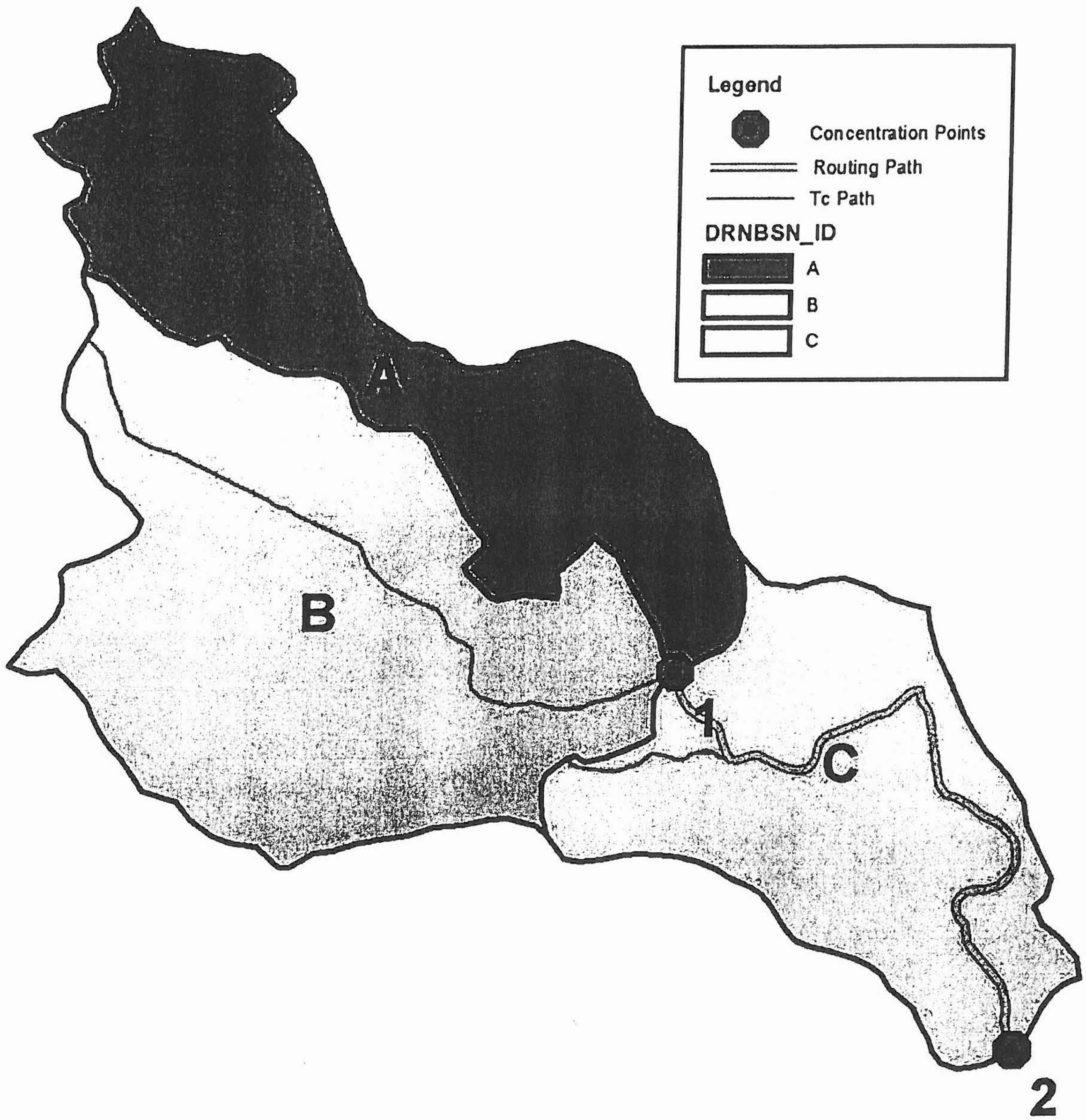
   a. If the T_c value from Method 1 is the longest, compute the total peak discharge using the Method 1 intensity, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   b. If the T_c value from Method 2 is the longest, determine i directly from the D-D-F statistics from step 3 of Section 3.6.1. Compute the total peak discharge at Concentration Point 2 using the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   c. If the T_c from subarea C is the longest, compute the total peak discharge using the i for subarea C, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

8. As an alternative to the above procedure, the DDMSW program may be used to calculate the peak discharge at intermediate locations.

Figure 3.2
SCHEMATIC EXAMPLE WATERSHED



Excerpts Reproduced From:
Draft Hydrology Manual (Reference 5)

Description:
The process described in the following pages was used as a guide to help determine the scour depths at the wash crossings.

Number of Pages: 16

room (1981), Rosgen (1996), Schumm (1961, 1971 and 1977), Hjalmarson (1998), and Thorn (1998).

10.13 ESTIMATION OF SCOUR

10.13.1 General

Definition of Scour

Scour, for the intent of this discussion, is the lowering of the bed elevation of a watercourse, either locally or over some defined reach length of watercourse, due to the hydraulics of flowing water. Scour is estimated as the sum of independent scour components that are due to factors along a defined reach of a watercourse plus scour at a specific location in a watercourse.

Purpose of Scour Estimates

Scour estimates are often needed for the following drainage and flood related purposes:

a. Estimation of the response of a watercourse due to altered management in the watershed. For example, scour in a natural watercourse may need to be evaluated due to urbanization that would alter the natural flood magnitude-frequency relations.

b. Estimation of the response of a watercourse due to alterations of the hydraulic conditions in the watercourse. Examples in this regard include floodplain encroachment, flood control modifications such as bank protection, and instream mining of sand and gravel.

c. Estimation of depth of toe-down for structural bank lining.

d. Estimation of depth of scour immediately at or downstream of hydraulic structures.

e. Estimation of potential scour depth for buried utility crossings of watercourses.

Applications and Limitations

The estimation of scour is an engineering application that requires both specific expertise and experience. Every application of scour technology is unique because of the wide variability of hydrologic, hydraulic and geologic/geomorphic factors. It is not possible to compile a comprehensive methodology in a drainage design manual that would be adequate to address all aspects of scour estimation. In addition, the knowledge of erosion and sedimentation is continually expanding because of the need to provide better technology in this field of engineering. Often, newer methodologies are presented in the engineering literature that should be considered and used, if appropriate. Therefore, the following are general guidelines for estimating scour along with currently used references that are considered applicable in Maricopa County.

Ref.
5

10.13.2 Total Scour

Total scour, for a given application, should consider the following components of scour:

a. Long-term degradation of the bed of the watercourse.

b. General scour through a specific reach of the watercourse.

c. Local scour.

d. Scour induced due to a bend in the watercourse.

e. Scour associated with bedform movement through the watercourse.

f. Scour due to low-flow incisement.

Total scour (Z_t) is the sum of each of these individual components (Z_i) of scour. Total scour can be expressed as:

$$Z_t = FS\,(Z_{long\text{-}term} + Z_{general} + Z_{local} + Z_{bend} + Z_{bedform} + Z_{low\text{-}flow}) \qquad (10.9)$$

A multiplying factor (FS) is used depending upon the purposes of the total scour estimation. For example, an FS equal to 1.0 may be appropriate when estimating total scour due to altered conditions in a watershed. However, in that case it would be advisable to estimate maximum and minimums of each individual component of scour and to estimate the range of total scour that can be expected. An FS of 1.3 is often used for the design of toe-down for bank protection. The use of higher FS, such as 1.5, may be justified where underestimation of scour would cause catastrophic failure that may result in loss of life or unacceptable economic consequences.

The following is a discussion of each component of scour that should normally be considered when estimating total scour.

Long-Term Degradation

Long-term degradation can be estimated by the following methods:

a. A trend analysis of historic bed elevation data.

b. Simulation by use of sediment transport modeling such as HEC-6 (USACE, 1991).

c. Application of equilibrium slope analyses.

A trend analysis of historic bed elevation data is limited by the availability of adequate, long-term data for the watercourse. Therefore, such an analysis may be possible only for some of the major watercourses in Maricopa County. In addition, factors such as instream gravel mining and channelization of the watercourse may complicate such historic analyses.

Simulation modeling may provide useful results; however, that method is dependent upon appropriate hydraulic data for the watercourse (hydraulic geometry and sediment characteristics). Furthermore, the results are highly sensitive to hydrologic input (flood magnitude-frequency relations, flow duration, shape of hydrograph, etc.). Simulation modeling may only be appropriate for regional studies of major watercourses, especially those for which structural flood control alternatives are being considered.

Equilibrium slope is a method that can often be applied to estimate long-term degradation without extensive data or modeling effort. The application of this method does require the identification of a downstream bed elevation control (pivot point) at which the bed elevation is not expected to change. Such a control can be bedrock, a reach of armored channel bed, or a constructed facility such as a diversion dam, roadway crossing, and so forth.

Long-term degradation using equilibrium slope analysis is estimated by

$$Z_{long\text{-}term} = L_w \, \Delta S \tag{10.10}$$

where $Z_{long\text{-}term}$ is the bed elevation change, in feet, at a distance, L_w, in feet, upstream of the pivot point and ΔS is the decrease in bed slope, in ft/ft from the existing slope. Equilibrium slope analysis resulting in an increase in bed slope upstream from the pivot point would indicate an aggradational zone rather than long-term degradation.

Application of Equation 10.10 is illustrated by the following:

> A natural watercourse has a slope of 22 feet per mile (0.0042 ft/ft). Proposed channelization of the watercourse will increase the unit discharge and the equilibrium slope is estimated to decrease to 15 feet per mile (0.0028 ft/ft). Grade control structures are to be constructed within the channelized reach at a distance of 2,000 feet between structures. The long-term degradation at the toe of each drop structure is estimated by:
>
> $$Z_{long\text{-}term} = (2000\,ft)(0.0042\text{-}0.0028\,ft/ft)$$
>
> $$= 2.8 \text{ feet}$$

Several methods are recommended by Pemberton and Lara (1984) for performing equilibrium slope analyses; the Schoklitsch bedload equation (Shulits, 1935), the Meyer-Peter, and Muller (1948) bedload equation, the Shields (1936) diagram, and Lane's (1952) relation for critical tractive force. The limitations and assumptions of each method should be carefully evaluated when making the selection of a preferred method. Often, more than one method can be used and the results compared. Corroborating results by two or more methods would increase reliance on those results. However, there often is considerable deviation in results by the various methods. In which case, independent data, regional experience and/or engineering judgement must be used in selecting the equilibrium slope.

General Scour

General scour is that component of total scour that would occur during the passage of a design flood. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. The scour is caused by increased velocities and shear stresses dictated by the local area geometry (such as at constrictions) and water surface controls. For major watercourses, general scour would often be estimated by a sediment transport model study, such as the use of HEC-6 (USACE, 1991). General scour in minor watercourses can be estimated by the following equation (Zeller, 1981):

$$Z_{general} = Y_{max}\left[\frac{0.0685 v^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1\right] \tag{10.11}$$

where: $Z_{general}$ is the general scour depth, in feet,

Y_{max} is maximum depth of flow, in feet,

Y_h is the hydraulic depth, in feet,

v is the average velocity of flow, in ft/sec, and

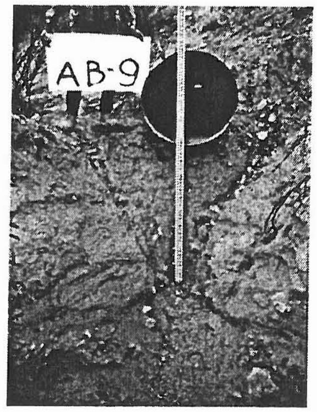
S_e is the energy slope (or bed slope if uniform flow is assumed), in ft/ft.

The reference by Zeller (1981) should be consulted prior to applying this equation. If Equation 10.11 yields negative results, a value of zero is to be used for general scour.

Local Scour

Local scour is that component of total scour that is caused by flow irregularities. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth of scour is increased, the strength of the vortex or vortices is reduced, the transport rate is reduced and equilibrium is reestablished and scouring ceases.

Flow irregularities can occur in natural watercourses due to bends or restrictions along the banks. Flow irregularities also occur due to constructed facilities such as bank lining, bank protection works (such as groins), hydraulic structures across the watercourse (such as diversion dams or grade control structures), and structures in the watercourse (such as bridges or culverts). Bridge scour, including the local component of bridge scour, is discussed in Section 10.13.4.

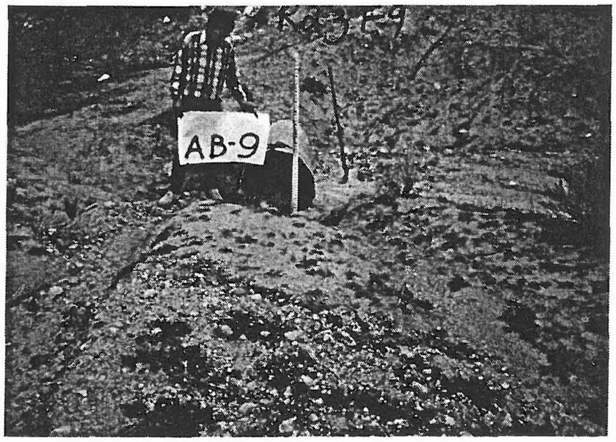


Local scour at unprotected culvert outlet.

Generally, local scour depths are much larger than long-term degradation or general scour. But, if there are major changes in watercourse conditions, such as a water storage facility built upstream or downstream or severe straightening of the watercourse, long term bed elevation changes can be the larger element in the total scour.

Five methods for estimating local scour due to natural restrictions and bends, or bank lining are presented by Pemberton and Lara (1984). The USBR Method I is for wide, sand bed watercourses with d_{50} ranging from 0.5 to 0.7 mm, and slopes from 0.004 to 0.008 ft/ft. That method probably has limited application in Maricopa County. USBR Method II is recommended for subcritical flow and includes consideration of watercourse curvature. The Lacey (1930) equation is for subcritical flow, includes consideration of watercourse curvature, and requires the use of bed material size and a "silt factor." The Blench (1969) equation is a function of unit discharge (q in cfs/foot width), Blench's "zero bed factor" and a factor for watercourse curvature. The Neill (1973) equation is based on flow depth and velocity and the estimation of "competent velocity." These equations can be used to estimate the local scour due to bank lining or similar applications. The report by Pemberton and Lara (1984) or the individual references should be consulted prior to application of any method. Often, more than one method can be applied and the results compared. Engineering judgment and experience are needed when selecting the value for local scour.

Local scour downstream of a hydraulic structure can be estimated by empirical equations. Flow over the structure can be either submerged or free falling depending on tailwater conditions. For free falling conditions, three local scour equations are available. The Schoklitsch (1932) equation requires hydraulic parameters including the effective drop height and the bed material particle size. The Veronese (1937) equation requires hydraulic parameters including effective drop height, but is independent of bed material grain size and may overestimate local scour for some watercourses in Maricopa County. The Zimmerman and Maniak (1967) equation is a function of the d_{85} bed material particle size, but is independent of many hydraulic parameters and does not consider the drop height. Therefore, that equation should only be used for relatively low (possibly not greater than half of the approach flow depth) drop heights. The Pemberton and Lara (1984) or the original references should be consulted when selecting or applying any of these equations.



Culvert causes backwater resulting in upstream aggradation.

For a submerged structure, the local scour depth can be estimated by the Simons, Li & Associates (1986) equation. The equation is a function of drop height and other hydraulic parameters, but is independent of bed material grain size. It may overestimate scour depth for coarse bed material watercourses. That reference should be consulted when using that equation.

Bend Scour

Bend scour may need to be estimated if not included as a component of local scour (see above). For sand-bed watercourses, Zeller (1981) presents a bend scour equation. That reference should be consulted in its use and application.

Bedform Trough Depth

Bedforms develop in alluvial channels in response to the hydraulics of the flowing water and they are part of the mechanics of sediment transport. Bedforms are of various configurations and typically they consist of alternating "mounds" and "troughs," and being mobile, they move longitudinally along the bed of the watercourse. A bedform trough is a component of total scour and should be accounted for under appropriate conditions. The component of scour that is associated with bedforms is equal to one-half of the bedform amplitude (vertical distance from top of mound to bottom of trough) as shown in the following equation.

$$Z_{bedform} = 0.5 d_h \tag{10.12}$$

Bedform trough depth should be estimated for dunes that occur during lower regime flow, and antidunes that occur during upper regime flow. Simons and Senturk (1992) provide dune height equations. Dune height is estimated by:

$$d_h = 0.066 Y_h^{1.21} \tag{10.13}$$

where: d_h is dune height, in feet, and

y is hydraulic depth of flow, in feet.

Antidune height is estimated by:

$$d_h = 0.28\pi Y_h F_r^2 \tag{10.14}$$

where: d_h = antidune height, in feet,

Y_h is hydraulic depth of flow, in feet, and

F_r is Froude Number.

Dunes form during lower regime flow, typically at F_r less than about 0.7, and antidunes form during the upper regime flow and may form during the transition from lower to upper regime flows. Therefore, antidunes can be expected for F_r greater than about 0.7. Antidune height will usually be greater than dune height. In the transition region, about 0.7 to 1.0 F_r, the larger of either dune or antidune height should be used.

Low-Flow Incisement

The normal irregularities in the bed of a watercourse (both natural and man-made) result in a low-flow channel. That channel is formed by the predominance of a low-flow condition or due too low-flows that persist after a flood. The magnitude of low-flow incisement may best be estimated by representative field assessment. In the absence of field data, or for planning and design purposes, low-flow incisement should be estimated as no less than 1 foot and possibly in excess of 2 feet. A lower value can be used for small and minor watercourses and a higher value should be used for regional watercourses.

10.13.3 Limits to Scour from Armoring

Armoring is the process in an alluvial watercourse wherein sediment transport removes bed material smaller than a certain size thus leaving a bed that is armored by the larger bed particle material. All alluvial channels experience the mechanics of armoring through the selective transport of finer bed material and leaving the coarser bed material. However, watercourses that continually receive the inflow of bed material load in excess of transport capacity or those watercourses for which the bed material does not contain adequate quantities of the larger, armoring-size bed material, will not experience armoring. Also, armoring is flood magnitude dependent; that is, an armoring layer can develop over time due to a sequence of flood events, but a flood event sufficiently larger than those that formed the armor layer can penetrate the armor layer resulting in additional scour depth.

Armoring can be a limiting agent to scour, and, in fact, the placement of riprap as a watercourse liner or around hydraulic structures is an "engineered" armoring. Therefore, when considering scour, particularly long-term and general scour, the potential for armoring should be considered.

The above photo indicates failed riprap from road overtopping and high exit velocities from culverts. Note head cutting.

Several methods are available for evaluating the potential for armoring. The incipient motion method (see Section 10.11.2) is commonly used and easily applied. Other methods include use of the Meyer-Peter, Muller equation (Sheppard, 1960), the competent bottom velocity method (Mavis and Laushey, 1948), Lane's tractive force method (Lane, 1952), and Yang's incipient motion relation (Yang, 1973). The user should consult those references when making application of those methods to evaluate armor potential.

10.13.4 Bridge Scour

Total Scour at Bridges

Scour at bridges must consider all reasonable components of scour that can apply to detrimentally impact a bridge pier or abutment. The total scour (Z_t) at a bridge is typically expressed as:

$$Z_t = FS\,(Z_{long\text{-}term} + Z_{local} + Z_{contraction}) \qquad (10.15)$$

where FS is a factor of safety which is set at 1.0 for most conditions, but under certain conditions of hazard, including potential economic loss or uncertainty in analyses, could be set higher than 1.0.

The component of long-term scour ($Z_{long\text{-}term}$) can be estimated by procedures discussed in Section 10.13.2. The potential for armoring (Section 10.13.3) may be considered, but should be used cautiously to limit scour depth.

The procedure in *Evaluating Scour at Bridges*, HEC-18 (USDOT, 2001b) should be consulted when estimating scour at bridges. Usually the largest component of scour is from local scour at the pier or abutment. Certain scour equations include the angle of attack of the flow, and therefore, bend scour is not normally added because it can be accounted for in the local scour.

Contraction scour occurs when the flow area of the watercourse is reduced because of natural conditions or because of the bridge approaches encroaching into the watercourse. Two equations are provided in HEC-18 (USDOT, 2001b) for contraction scour. One is for live bed conditions; that is, when there is bed material transport from upstream of the bridge. For that condition, a modified version of Laursen's live-bed contraction scour equation (Laursen, 1960) is used. The second is for clear water conditions; that is, when there is little or no sediment transport from upstream of the bridge. For that condition, Laursen's clear-water contraction scour equation (Laursen, 1963) is used. The HEC-18 publication (USDOT, 2001b) should be consulted when estimating contraction scour.

Pier Scour

The commonly used pier scour equations are the Colorado State University equation (Richardson and others, 2001) and Froehlich (1988). Both of those equations are considered in the HEC-RAS program for bridge pier scour (USACE 2001 and 2001b); however, only the Colorado State University equation is recommended in HEC-18 (USDOT, 2001b). The Froehlich equation has been shown to compare well with observed data. Those references should be consulted when estimating pier scour.

Abutment Scour

The commonly used abutment scour equations are the HIRE equation (Richardson and others, 2001) and Froehlich (1989). Those equations are provided both in HEC-18 (USDOT, 2001b) and the HEC-RAS program (USACE 2001a and 2001b). Those references should be consulted when estimating abutment scour.

Watercourse Stability at Highways

The stability of the watercourse at and near highway structures should be considered if channel instability is suspected. Procedures to investigate watercourse stability are provided in HEC-20 (USDOT 2001c).

Bridge Scour Countermeasures

Procedures to provide bridge scour countermeasures are provided in USDOT (2001a).

(Alonso, 1997), reports sediment yield of 0.12 to 0.4 acre-feet per square mile per year for the Walnut Gulch Experimental Watershed near Tombstone, Arizona.

The wide range of sediment yield is explained by soil conditions, precipitation, and watercourse conditions among other things. For example, the relatively small yield of 0.08 acre-feet per square mile per year from the 30 square mile basin above Saddleback Flood Retarding Structure in Maricopa County, Arizona, is due to the well-developed soil covered with desert pavement. The differences of sediment yield are also related to climate differences. For example, certain watersheds in San Diego County, CA reflect yields of only 0.07 and 0.13 acre-feet per square mile per year due to the low annual precipitation of only 3 inches. Some sites with a large sediment yield such as Davis Tank, AZ are known to have watercourse bed and bank erosion. Lastly, other sites with relatively high yield such as Black Hills Tank, AZ may have experienced a large flood during a short period of data collection.

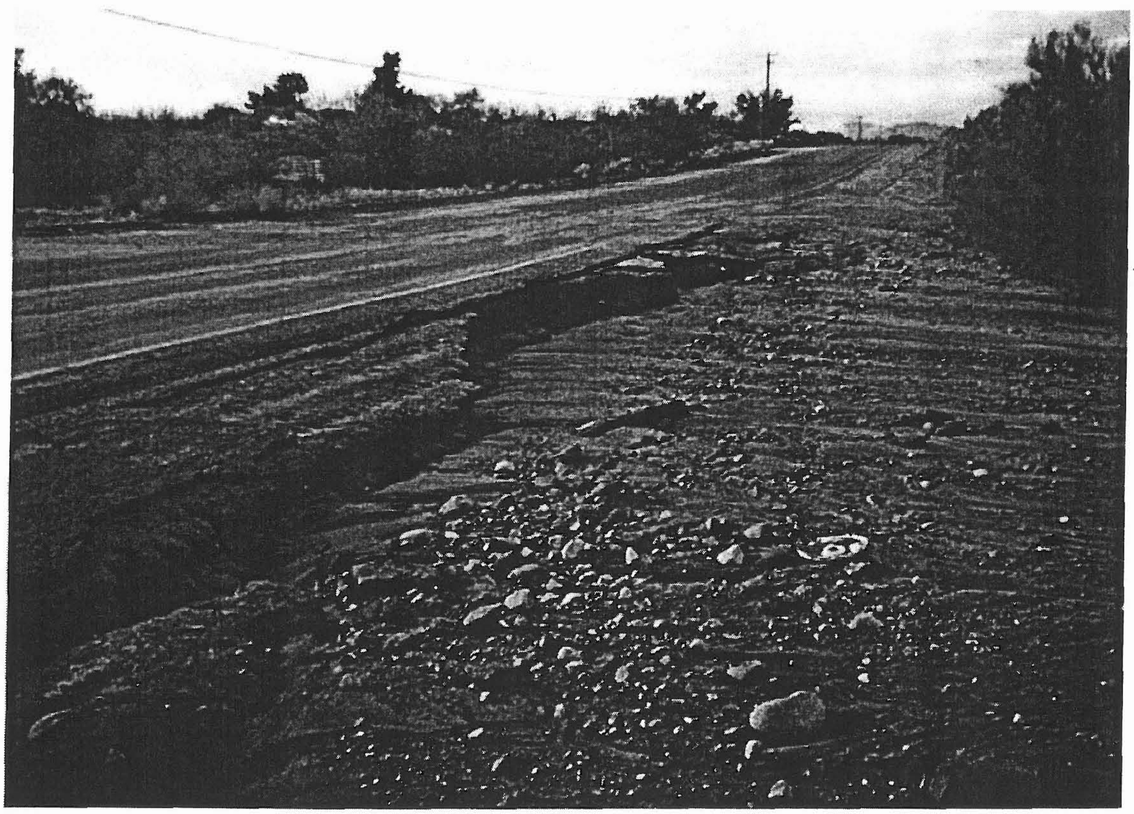
Runoff and sediment yield data were collected at the Black Hills Tank, near Cave Creek, Arizona, from 1945 to 1948 (Langbein and others, 1951, and Peterson, 1962). The precise location of the site is uncertain but it was near the northern end of the McDowell Mountains on a granite pediment at an elevation of about 2,600 feet. Vegetation was mountain-brush type consisting mainly of snakeweed, yucca, creosote bush, and cactus, with small palo verde and mesquite trees along the channels. According to Langbein and others (1951), the approximately 2.5 mile long drainage basin was 1.56 square miles in area, headed at 3,200 feet elevation, and was drained by a network of 0.5 to 2 feet deep watercourses at a slope of about 2 percent. The granitic rock is capped with a thin veneer of coarse residual soil. The watershed sediment yield was 0.9 acre-feet per year or 0.58 acre-feet per square mile per year based on capacity surveys at the beginning and end of the data collection. A field examination of the 1948 flood reportedly showed coarse sediment with uprooted mesquite trees deposited in a fan at the entrance to the tank. There was no spill during the period. According to Peterson (1962) the drainage basin is only 1.14 square miles and the watershed sediment yield is 0.78 acre-feet per year or 0.68 acre-feet per square mile per year. The difference in reported sediment yield for the same watershed is not significant. However, the reported large flood in 1948 is significant because unusually large amounts of sediment were deposited in the tank. The reported average annual sediment yield in Table 10.3 for Black Hills Tank for the 4-year period probably is too high because of the 1948 flood. However, that data does indicate the magnitude of sediment that can be produced from a single intense runoff event.

10.11 SEDIMENT TRANSPORT

10.11.1 General

The magnitude of sediment transport is dependent upon the ability of the flowing water to transport incoming sediment and/or to erode the material making up the bed and/or banks of the watercourse. Watercourses composed predominately of sand-sized material will respond to virtually the entire range of flows to which it is subjected. However, watercourses composed of significant quantities of coarser (gravel, cobble and boulder) material will be limited to adjustments only during large flow events.

A basic understanding of sediment transport mechanics is fundamental in qualitative and quantitative sediment transport analyses. Inherent in that understanding are the concepts of incipient motion and armoring. Incipient motion analysis provides a means to estimate the largest size of sediment particle that can be transported during a given flow event. In cases where there is a sufficient quantity of coarse sediment, an armor layer may form that can act as a complete or partial control to sediment transport. The application and limitation of the numerous sediment transport equations must be understood and appreciated when performing sediment transport analyses and quantitative studies.



Shallow flow over roadway initially causes headcutting into road subgrade and pavement

10.11.2 Bed Form

Sediment transport is highly dependent upon the resistance to flow, and resistance to flow in an alluvial channel is strongly related to the physical shape of the bed. The physical elements that comprise the shape of the bed are called bed form. For a more thorough discussion of bed form and its impact on flow resistance see Simons and Senturk (1992). Those bed forms in common occurrence in alluvial channels are briefly described:

Plane bed - A flat or nearly-flat and smooth surface of the bed.

Ripples - Small bed forms that are typically less than a foot long and less than 1 1/2 inches high. They occur in lower regime flow.

Bars - Large bed forms that have lengths of the same order as channel width and heights about the same as flow depth. There are several kinds of bars, such as point bars, alternate bars, tributary bars and middle bars.

Dunes - Bed forms that are larger than ripples and smaller than bars. Size is a function of the geometry of the watercourse. It indicates higher transport rates than ripples.

Antidunes - Bed forms in upper regime flow that are often in trains. They are often called standing waves. They exhibit surface waves that are in phase with the antidunes.

Chutes and Pools - Bed forms of large elongated chutes of high slope and high velocity flow separated by low velocity pools. These represent very high sediment transport rates.

Watercourse exhibiting potential for large bed load discharge

Bed form is often associated with regime of flow. (Note: This is a different concept than the regime of Section 10.6.2.). Plane bed, ripples and dunes are typically in lower flow regime where the Froude number is usually less than 0.4. The transition to washed-out dunes and a return to plane bed (with high bed load transport) represent the transition regime where the Froude number is typically between about 0.4 to 0.7. Antidunes with standing waves or with violent breaking waves and chute and pool are in upper flow regime where the Froude number is typically greater than 0.7 (Guy, 1970).

10.11.3 Incipient Motion

Incipient motion occurs when the hydrodynamic forces acting on a grain of sediment of given size is equal to the forces resisting movement. Incipient motion is often analyzed using the Shields relation:

$$d_c = \frac{\tau_o}{F_*(\gamma_s - \gamma)} \qquad (10.6)$$

where: d_c is the sediment diameter at incipient motion in feet,

τ_o is the bed shear stress in pounds per square foot,

γ_s is the sediment specific weight, typically 165 pounds per cubic foot,

γ is the water specific weight, 62.4 pounds per cubic foot,

and F_* is the dimensionless shear stress, often referred to as the Shields parameter. F_* ranges from 0.03 to 0.06 and a value of 0.047 is often used (AMAFCA, 1994).

The bed shear stress in pounds per square foot, is calculated by

$$\tau_o = \gamma R S \tag{10.7}$$

where R is the hydraulic radius, in feet, and S is the channel friction slope, in ft/ft.

Incipient analysis, as presented herein, does not cover all aspects of incipient motion. For a discussion of applications, limitations and modifications see AMAFCA (1994), ASCE (1975), Richardson and others (2001), Simons and Senturk (1992), Yang (1973), ADWR (1985), Chang (1988), and Shen (1971, 1972 and 1973).

Application of incipient motion analysis may provide information on the magnitude of discharge required to move the particles lining the watercourse bed and/or banks. These analyses are generally most reliable and useful for gravel or cobble-bed watercourses. When applied to sand-bed systems, incipient motion results usually show that the sediment particles are in motion, even at small discharges.

10.11.4 Armoring

Armoring occurs when material finer than the incipient motion size is eroded and transported away leaving a layer of coarser, immobile (for a given discharge) material on the surface. If the watercourse is in a degradational mode, this process can continue over a range of discharge events, each larger event removing the increasing larger particle sizes. Armoring is effective only to a given magnitude of flood event, flows exceeding that magnitude may disrupt the armor layer causing bed scour and degradation.

Armoring analysis normally requires the application of incipient motion analysis and data on bed material size gradation within the anticipated depth of scour. In application, the d_{95} particle size is considered to be the maximum size for armor formation. Therefore, armoring (for a given discharge) can be expected when the computed incipient motion size is equal to or smaller than the d_{95} size of the bed material.

The depth of scour (Y_s) necessary to establish an armor layer can be estimated by Pemberton and Lara (1984).

$$Y_s = Y_a\left(\frac{1}{P_c} - 1\right) \tag{10.8}$$

where Y_a is the desired thickness of the armor layer (normally assumed to be 2 to 3 times the critical particle size, d_c, and P_c is the decimal fraction of bed material coarser than the armoring size.

10.11.5 Sediment Transport Methods

The planning and design of drainage and flood control facilities often requires the analysis of sediment transport. Often those analyses are performed using sediment transport methods. Those methods may be mathematical or graphical and can be theoretically or empirically based. Often the method is some combination of all of the above. Some of the more popular sediment transport methods are the Einstein bed load function, the Meyer-Peter, Muller equation, the Yang unit stream power concept and the Colby relations. However, there are virtually dozens of sediment transport relations in the literature. A problem for the engineer is to select one or more of these relations for use in solving a particular problem. When selecting a sediment transport method, the data base (sediment size, flow condition, mode of transport process, etc.) used to develop each method must be understood. The selection, however, is not straightforward and often it is not possible to determine which one is best for a particular application. Often the selection process indicates that no one method is best and two or more methods may need to be used and the respective results evaluated. The results by different methods often differ drastically. It is absolutely imperative that the application and limitation of the various methods be understood when using those to estimate sediment transport. The engineer must use experience and judgement in both the selection of the sediment transport method and in the interpretation of the results. See AMAFCA (1994), ASCE (1975), Yang (1973), ADWR (1985), Chang (1988), Richardson and others (2001), Shen (1971, 1972, 1973), Sheppard (1960), and Simons and Senturk (1992) for further discussions of sediment transport methods.

10.12 BANK EROSION AND LATERAL MIGRATION

10.12.1 Bank Erosion

Bank erosion and widening of watercourses occurs from two primary mechanisms; grain-by-grain erosion and bank failure. Commonly, grain-by-grain erosion and bank failure act together; fluvial erosion scours the toe of the bank, and failure follows. Removal of the failed bank material occurs through fluvial erosion and the process is repeated.

The bank erosion process can result from watercourse incision (degradation), flow around bends, flow deflection due to local deposition or obstructions, aggradation, or a combination of the above. For the case of an incising watercourse, exceedence of the maximum stable bank height will lead to mass failure and bankline retreat. Flow around a bend can cause erosion at the toe of the bank and subsequent bank failure due to increased shear stress on the outside of the

APPENDIX G. EMERGENCY SPILLWAY WASH, CROSSING NO. 2A

- Rational Method Determination of Flow
- Level 2 Scour Analysis
- Tables and Figures Supporting Hydrology
- Cross Section Development from *Cave Creek FDS* Mapping
- Table Supporting Scour Analysis
- Hydraulic Calculations for Scour Analysis
- Grain-size Distribution of Cave Creek Wash
- Drainage Area Delineation

Rational Method for Determining Peak Flow at Emergency Spillway Wash #1 (ESW1)
This spreadsheet follows the procedures outlined in the *Draft Hydrology Manual for Maricopa County* **(Reference 5)**.

Drainage Basin Information (Input Data)

	A =	140.00	(drainage area, in acres, estimated using USGS Quads)
	C =	0.70	(coeffecient related to rainfall runoff, from **Table 3.2** on **Sheet 4**)
	L =	4872.00	(length of longest flow path, in feet, estimated using USGS Quads)
	Ue =	1840.00	(upstream watercourse elevation, feet, estimated using USGS Quads)
	De =	1570.00	(downstream watercourse elevation, feet, estimated using USGS Quads)
$S = (Ue - De) / L$	**S =**	292.61	(watercourse slope, in feet/mile)
	m =	-0.03	(K_b equation parameter, from **Table 3.1** on **Sheet 4**)
	b =	0.15	(K_b equation parameter, from **Table 3.1** on **Sheet 4**)
$K_b = m \log A + b$	**K_b =**	0.10	(watershed resistance coefficient)

Duration and Frequency (Iterative Data)

Note: The storm duration is adjusted until it is equal to the time of concentration.	**D =**	0.30	(storm duration, in hours)
	***i* =**	5.00	(rainfall intensity, in inches/ hour, from **Table 1** on **Sheet 3**)

Time of Concentration Equation (Output Data)

$T_C = 11.4L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}$	**T_C =**	0.30	(time of concentration, in hours)

Rational Equation (Output Data)

$Q = C\, i\, A$	**Q =**	**490**	**(peak discharge, in cfs)**

Determination of General Scour Depth at Emergency Spillway Wash #1 (ESW1)
This spreadsheet follows the procedures outlined in the *AFMA Scour Analysis Short Course* **(Reference 10)**.

General Information (Input Data)

	$Q =$	490.00	(discharge, in cfs, from **Sheet 1**)
	$V =$	8.58	(flow velocity, in feet/ second, from **Sheet 7**)
	$y =$	1.65	(depth in feet, from **Sheet 7**)
	$A =$	56.35	(area, in square feet, from **Sheet 7**)
	$P =$	72.44	(wetted perimeter, in feet, from **Sheet 7**)
$R = A / P$	$R =$	0.78	(hydraulic radius of channel, in feet)
	$W_{top} =$	72.35	(top width of flow, in feet, from **Sheet 7**)
$D_h = A / Wtop$	$D_h =$	0.78	(hydraulic depth, in feet)
	$D_{90} =$	0.24	(particle size which is larger than 90% of all sizes, in meters, from **Sheet 8**)
	$D_{90} =$	0.79	(particle size which is larger than 90% of all sizes, in feet, from **Sheet 8**)
	$D_m =$	110.00	(mean grain size of bed material assumed to be D50, in milimeters, from **Sheet 8**)
	$\gamma =$	62.40	(specific weight of water, in pounds/ cubic feet)
	$\rho =$	1.94	(density of the water, in slugs/ cubic feet)
	$S_e =$	0.06	(energy slope, in feet/feet, from **Sheet 1**)

Incipient Motion Analysis

$\boxed{\tau_\rho = 1/8 \times f \times \rho \times V^2}$	$\tau_\rho =$	2.08	(boundary shear stress, in pound per square feet)
$\boxed{D_c = \tau_\rho / (S(\gamma_s - \gamma))}$	$D_c =$	0.67	(diameter of the sediment particle at incipient motion conditions, in feet)
	$D_c =$	205.73	(diameter of the sediment particle at incipient motion conditions, in milimeters)
	$\gamma_s =$	165.00	(specific weight of sediment, in pounds per cubic feet)
	$S =$	0.03	(shields parameter)
$f = 116.5\ n^2 / R^{1/3}$	$f =$	0.12	(friction factor)
$n = D_{90}^{1/6} / 26$	$n =$	0.03	(Manning resistance value)

Shield's Method for Determining Armoring Depth

	$y_a =$	2.02	(thickness of the armor layer, in feet, varies from 1 - 3 times the critical particle size, 3 used to be conservative)
	$P_c =$	0.43	(percent of material coarser than the critical particle size expressed as a decimal fraction, from **Sheet 8**)
$y_s = y_a\ [(1 / P_c) - 1]$	$y_s =$	**2.68**	**(depth of degradation or scour required to form armor layer, in feet)**

General Scour Equations and Results - Lacey's Equation

$Lf = 1.76(D_m)^{1/2}$	$Lf =$	18.46	(Lacey's silt factor)
$Ym = 0.47\ (Q/ Lf)^{1/3}$	$Ym =$	1.40	(mean water depth at design discharge, feet)
	$Z =$	0.25	(multiplying factor from **Table 7** on **Sheet 6**)
$y_{gs} = Z(Y_m)$	$y_{gs} =$	**0.35**	**(general scour depth using Lacey's Equation, in feet)**

General Scour Equations and Results - Blench Equation

$y_{fo} = q_f^{2/3} / F_{bo}^{1/3}$	$y_{fo} =$	1.97	(water depth for zero bed sediment transport, in feet)
$q_f = Q / W_{top}$	$q_f =$	6.77	(design discharge per unit width, in cubic feet per second per foot)
	$F_{bo} =$	6.00	(Blench's "zero bed factor" in feet per second, from **Figure 9** on **Sheet 6**)
	$Z =$	0.60	(multiplying factor from **Table 7** on **Sheet 6**)
$y_{gs} = Z(Y_{fo})$	$y_{gs} =$	**1.50**	**(general scour depth using Blench's Equation, in feet)**

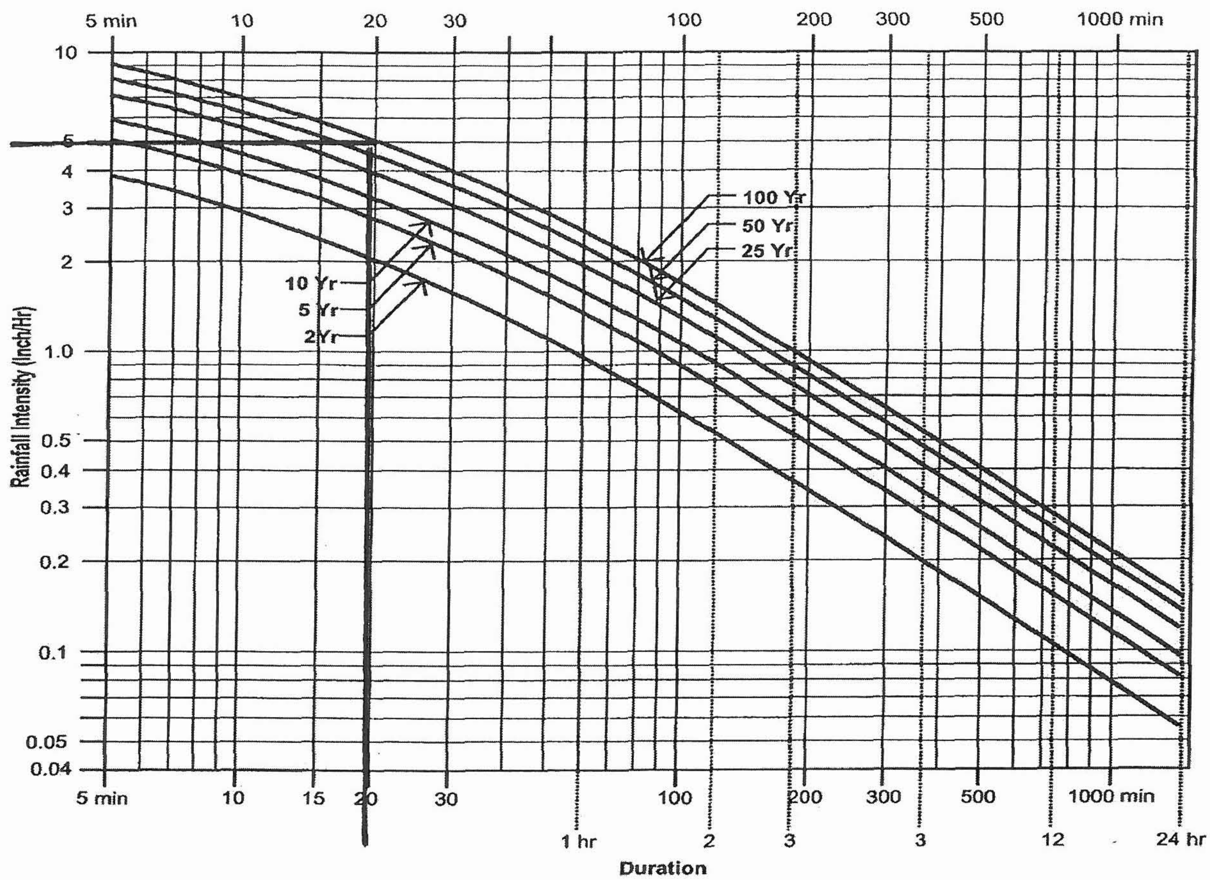
The Blench Equation yields a more conservative estimate of the general scour, and does not exceed the armoring depth. The estimate was rounded up to the nearest 1/2 foot. **Using the assumptions listed throughout these calculation sheets, the general scour has been estimated to be 1.5 feet.**

Entellus Inc.
CLIENT: DSWA
JOB: North Gateway Force Main - Scour Analysis

JOB # 910005a
SHEET 3 OF 9

Supporting Figures and Tables for Determining Peak Flow at Emergency Spillway Wash #1 (ESW1)
This sheet contains tables and formulas from the *Draft Hydrology Manual for Maricopa County* **(Reference 5)**.

TABLE 1



Supporting Figures and Tables for Determining Peak Flow at Emergency Spillway Wash #1 (ESW1)
This sheet contains tables and formulas from the *Draft Hydrology Manual for Maricopa County* **(Reference 5)**.

Table 3.1
EQUATION FOR ESTIMATING K_B IN THE T_C EQUATION

$$K_b = m \log A + b$$

Where A is drainage area, in acres

Type	Description	Typical Applications	Equation Parameters	
			m	b
A	Minimal roughness: Relatively smooth and/or well graded and uniform land surfaces. Surfaces runoff is sheet flow.	Commercial/industrial areas Residential area Parks and golf courses	-0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	-0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths.	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	-0.025	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	-0.030	0.20

Table 3.2
RUNOFF COEFFICIENTS FOR MARICOPA COUNTY

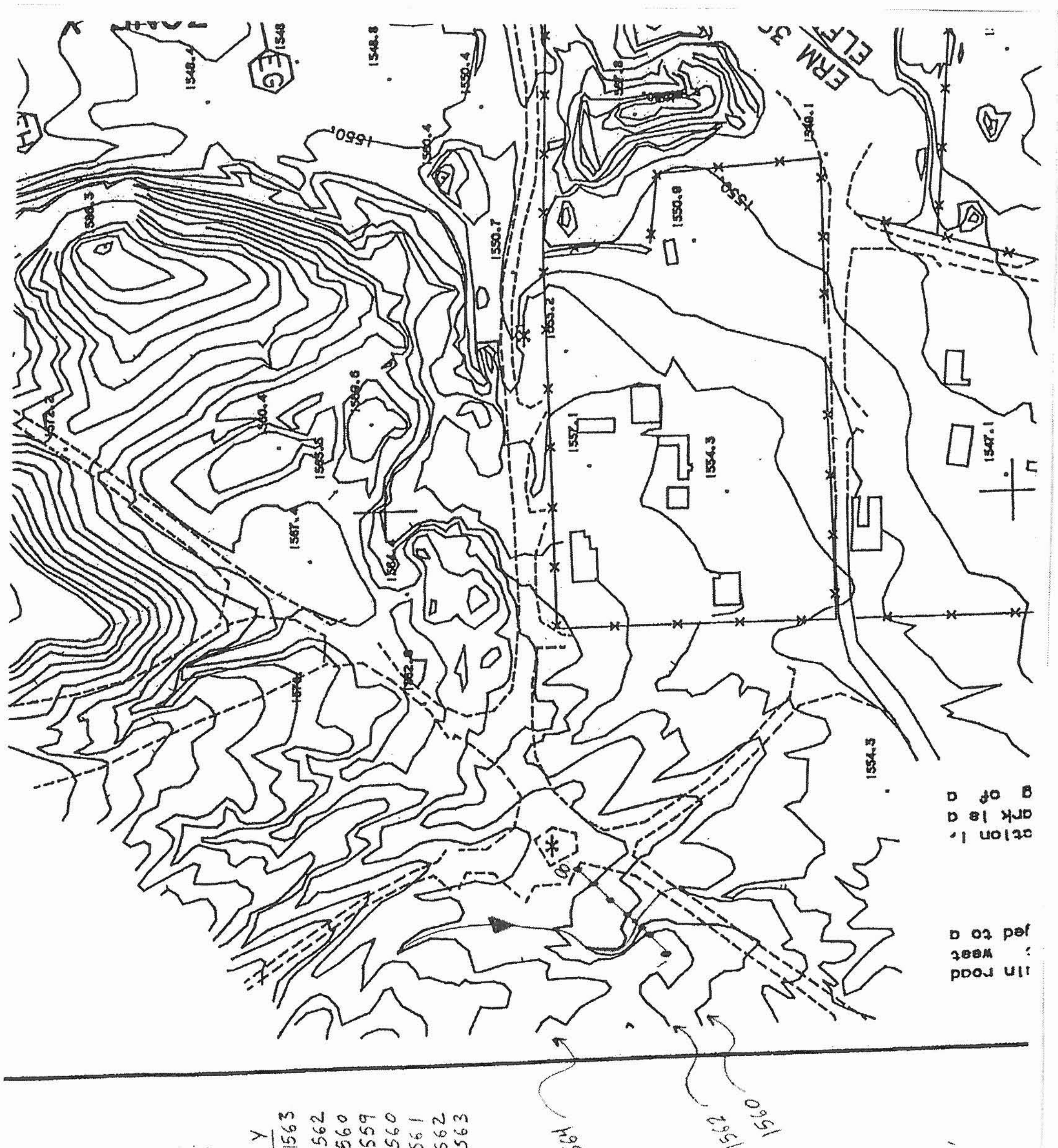
Land Use Code	Land Use Category	Runoff Coefficients by Storm Frequency[1, 2]							
		2-10 Year		25 Year		50 Year		100 Year	
		min	max	min	max	min	max	min	max
VLDR	Very Low Density Residential[3]	0.33	0.42	0.36	0.46	0.40	0.50	0.41	0.53
LDR	Low Density Residential[3]	0.42	0.48	0.46	0.53	0.50	0.58	0.53	0.60
MDR	Medium Density Residential[3]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.82
MFR	Multiple Family Residential[3]	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
I1	Industrial 1[3]	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
I2	Industrial 2[3]	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
C1	Commercial 1[3]	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
C2	Commercial 2[3]	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
P	Pavement and Rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
GR	Gravel Roadways & Shoulders	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
AG	Agricultural	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
LPC	Lawns/Parks/Cemeteries	0.10	0.25	0.11	0.28	0.12	0.30	0.13	0.31
DL1	Desert Landscaping 1	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
DL2	Desert Landscaping 2	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NDR	Undeveloped Desert Rangeland	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NHS	Hillslopes, Sonoran Desert	0.40	0.55	0.44	0.61	0.48	0.66	0.50	0.69 -.70
NMT	Mountain Terrain	0.60	0.80	0.66	0.88	0.72	0.95	0.75	0.95

Notes:

1. Runoff coefficients for 25-, 50- and 100-Year storm frequencies were derived using adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the 2-10 Year values with an upper limit of 0.95.
2. The ranges of runoff coefficients shown for urban land uses were derived from lot coverage standards specified in the zoning ordinances for Maricopa County.
3. Runoff coefficients for urban land uses are for lot coverage only and do not include the adjacent street and right-of-way, or alleys.

Supporting Figures and Tables for Determining Scour Depth at Emergency Spillway Wash #1 (ESW1)

This sheet contains a reproduced portion of Sheet 11 from the *1991 Middle Cave Creek Floodplain Delineation Map* **(Reference 2)**.
The contours were used to estimate the wash geometry at the crossing.
The wash geometry was used to estimate the hydraulic parameters.



Pt#	X	Y
1	0	1563
2	40	1562
3	60	1560
4	80	1559
5	100	1560
6	140	1561
7	180	1562
8	200	1563

NOTE: MAP IS NOT TO SCALE

Tables and Figures Supporting Scour Analysis From AFMA Scour Course (Reference 10)

Table 7. Multiplying Factors, Z, for Use in Scour Depths by Regime Equation (Pemberton and Lara, 1984)

Condition	Value of Z: Neill $d_s = Z\ d_f$	Value of Z: Lacey $d_s = Z\ d_m$	Value of Z: Blench $d_s = Z\ d_{fo}$
Equation Types A and B			
Straight reach	0.5	0.25	1/ 0.6
Moderate bend	0.6	0.5	1/ 0.6
Severe bend	0.7	0.75	1/ 0.6
Right angle bends		1.0	1.25
Vertical rock bank or wall		1.25	

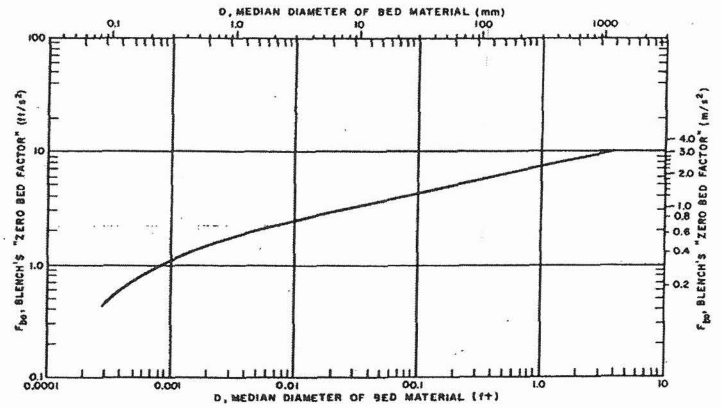
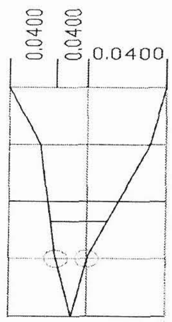


Figure 9. Chart for Estimating F_{bo} – After Blench (Pemberton and Lara, 1984)

Cross-Section Geometry with 100 Year Water Surface



Normal Depth Results

Cross-Section:	5	
Elevation:	1560.650	ft MSL
Depth:	1.650	ft
Discharge:	490.000	cfs
Energy Gradient:	0.0554	ft/ft
Froude Number:	1.1946	
Flow Regime:	Supercritical	
Flow Area:	56.350	sq ft
Average Velocity:	8.580	ft/s
Maximum Velocity:	9.600	ft/s
Composite n:	0.04	
Hydraulic Radius:	0.78	ft
Wetted Perimeter:	72.440	ft
Wetted Top Width:	72.350	ft
Critical Slope:	0.0533	ft/ft

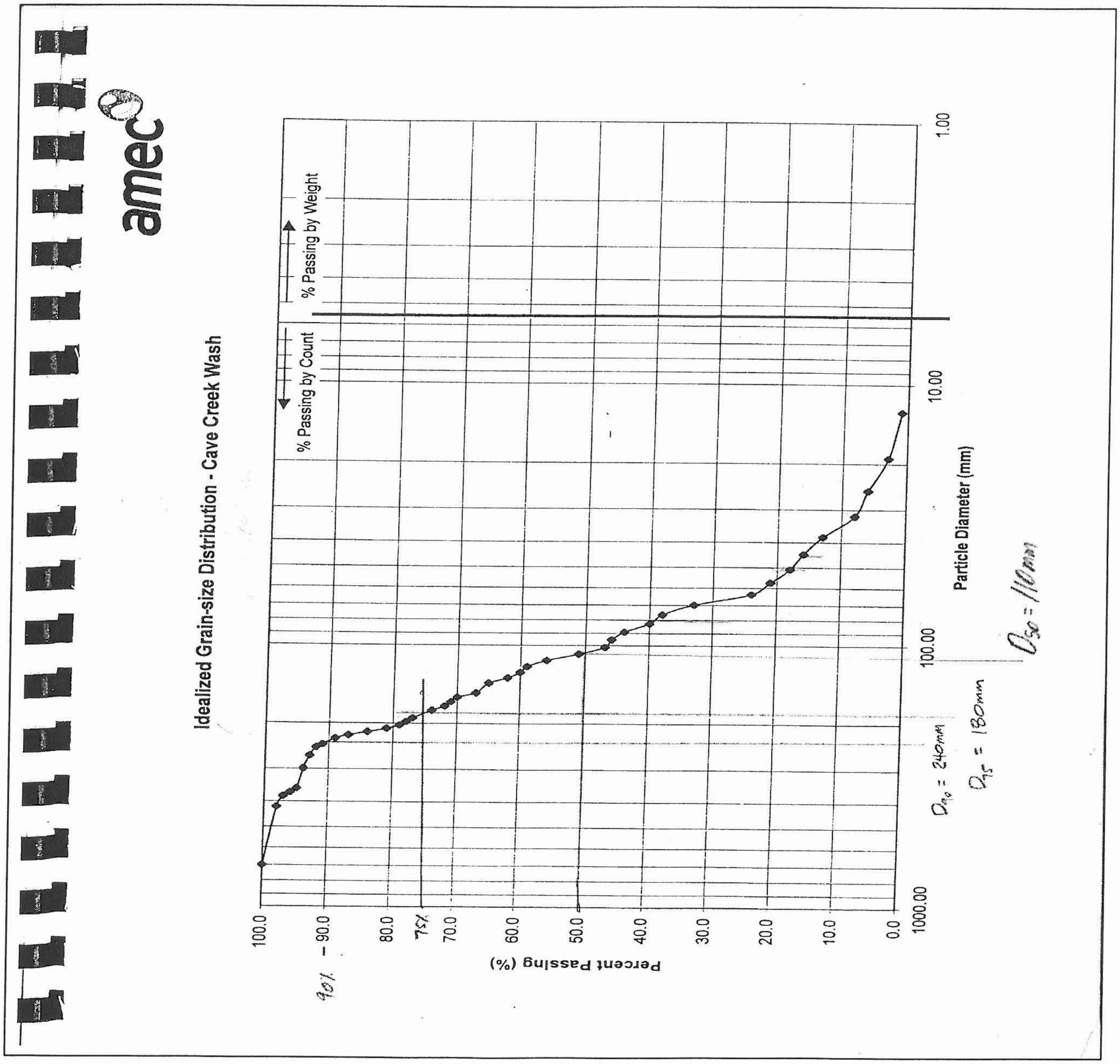
(Output from HEC-RAS)

Station* (feet)	Elevation* (feet)	n value**
0	1563	0.04
40	1562	0.04
60	1560	0.04
80	1559	0.04
100	1560	0.04
140	1561	0.04
180	1562	0.04
200	1563	0.04

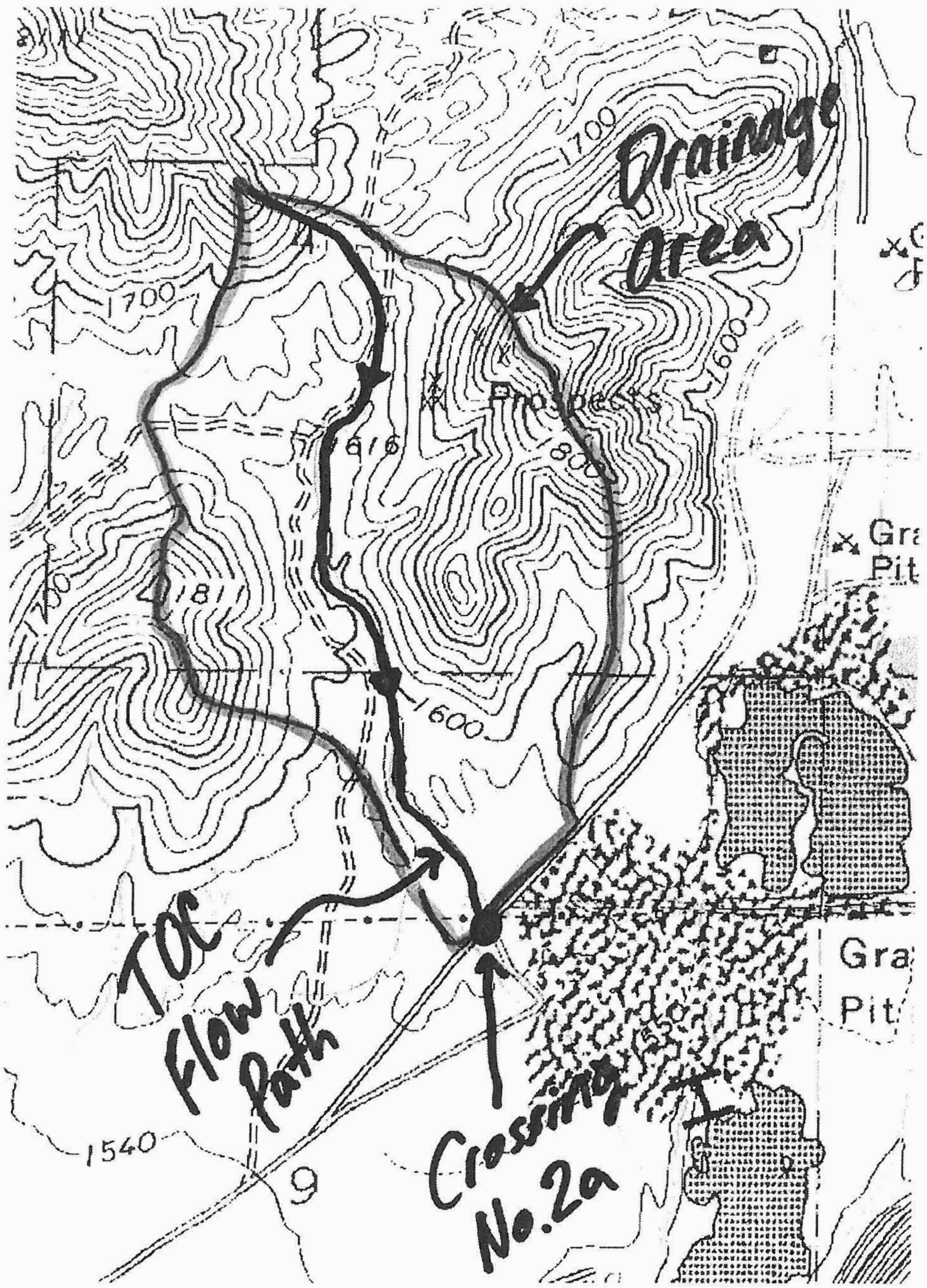
*(from Sheet 5)
**(The n values were estimated using District aerial photography)

Gradation Curve at Cave Creek Wash from *Geotechnical Report* (Reference 9)

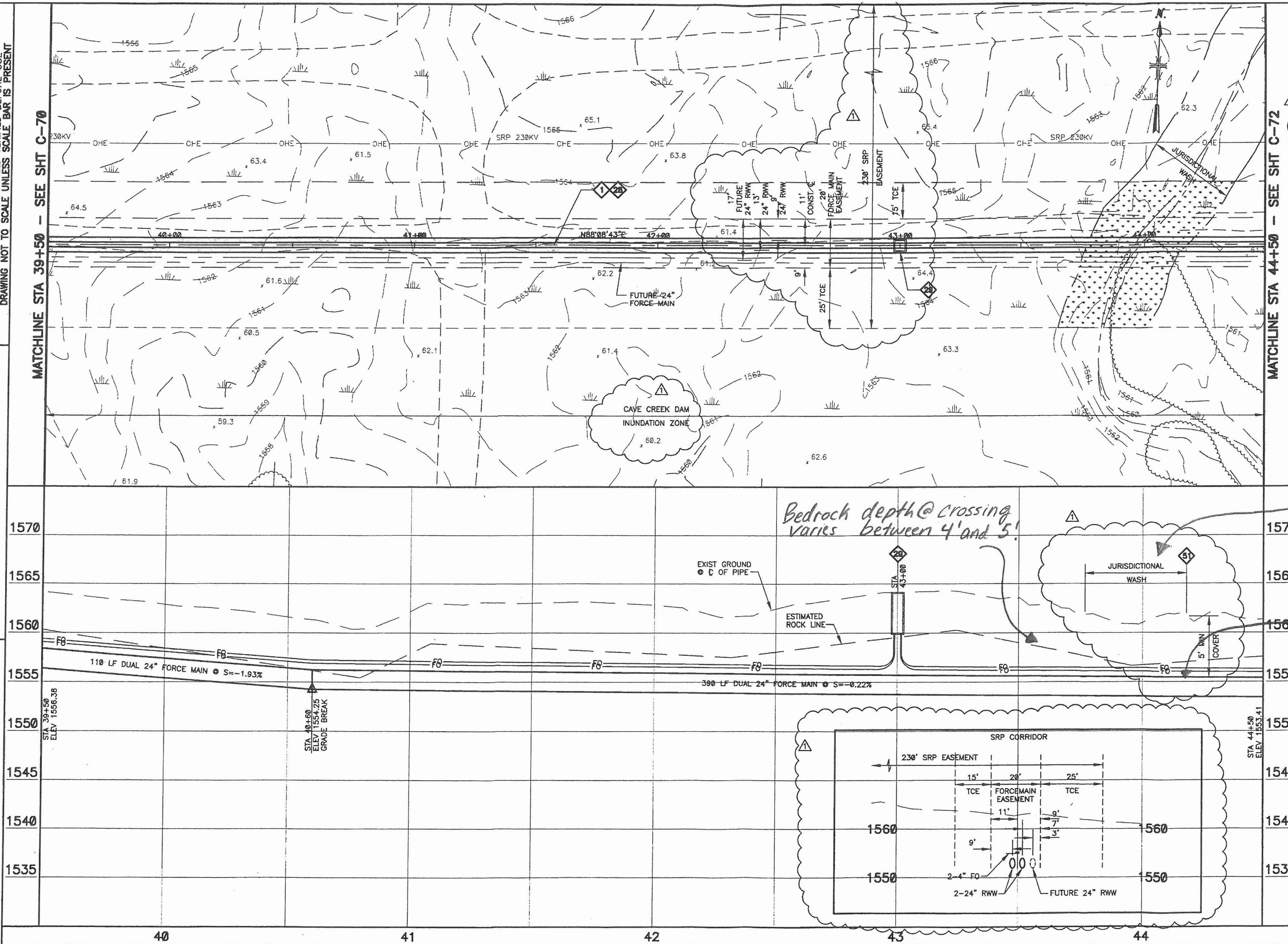
This sheet contains a reproduction of the soil gradation curve given to Entellus by DSWA.
This soil data was collected as a "surface sample" from Cave Creek Wash at Happy Valley and 19th Avenue
The actual soil gradation at the ESW#1 could be different, and should be verified.



Drainage Area Delineation Using USGS 7.5 Quadrangle Map: Union Hills



THIS DRAWING MUST BE FIELD VERIFIED BEFORE USE
DRAWING NOT TO SCALE UNLESS SCALE BAR IS PRESENT



Crossing No. 2a at wash below emergency spillway

24" Force main depth @ crossing varies between 5' and 7', which is below the bedrock depth

KEY NOTES

1. 24" FORCE MAIN — 1000 LF
28. 4" FO CONDUIT IN ACCORDANCE WITH INDUSTRY AND MFR GUIDE LINES AND APPLICABLE CODES. — 1000 LF
29. ~~4'x4'x4' FO HAND HOLE~~ — 1 EA
51. INSTALL 4" FO BELOW SCOUR DEPTH. PROVIDE MIN BURIAL DEPTH AS SHOWN.

△ BRASS CAP IN HAND HOLE AT INTERSECTION OF CENTRAL AVE AND HAPPY VALLEY ROAD
STA:88+56.77, 21' L
NORTH:987126.52
EAST:652217.73
ELEV:1538.66

SCALE IN FEET 1"=5' VERT

SCALE IN FEET 1"=20' HORIZ

FACILITY DRAWINGS
This drawing was supplied by a Consultant Engineer from a past construction project. The original construction drawing was modified based on information provided by the Contractor to provide the Record Drawing. The City does not warrant this drawing to be complete and accurate portrayal of facilities as they exist in the field.

CITY OF PHOENIX ORDINANCE G-4396, THESE PLANS ARE FOR OFFICIAL USE ONLY AND MAY NOT BE SHARED WITH OTHERS EXCEPT AS REQUIRED TO FULFILL THE OBLIGATIONS OF YOUR CONTRACT WITH THE CITY OF PHOENIX.

Ref. 11

DSWA DAMON S. WILLIAMS ASSOCIATES, L.L.C.

Registered Professional Engineer (Civil) CERTIFICATE NO. 30012 JOHN H. MATTA Date Signed 7-1-04 ARIZONA, U.S.A.

NO.	BY	DATE	CKD	REMARKS
△	PGH	06/04	RJK	REVISED PER ADDENDUM 1

DES DCM | DWN SAR/PGH | CKD RJK

City of Phoenix

CITY OF PHOENIX WATER SERVICES DEPARTMENT
NORTH GATEWAY WATER RECLAMATION PLANT PHASE 1 RESIDUALS PUMP STATION AND FORCE MAINS

CIVIL
SEGMENT 5
SALT RIVER PROJECT EASEMENT CORRIDOR PLAN & PROFILE
STA 39+50 TO 44+50

COPYRIGHT © 2004
CITY PROJECT NO. WS85090003
DATE: MAY 2004
DWG NO. C-71
SHEET NO. 107 OF 159
CAD FILE: C-NGFM-PP72.dwg

FOR CITY OF PHOENIX USE ONLY – REFERENCE CID NUMBER: REV. | ENG. (CO.) | PROJ. NO. | PROJ. NAME | DATE | CAD FILE NAME | DWG NUMBER | REMARKS

Dwg File: U:\Cadd\830378\dwg\830372\C-NGFM-PP72.dwg Modified: Jul 02, 2004 – 11:09am Plotted: Jul 08, 2004 – 10:54am

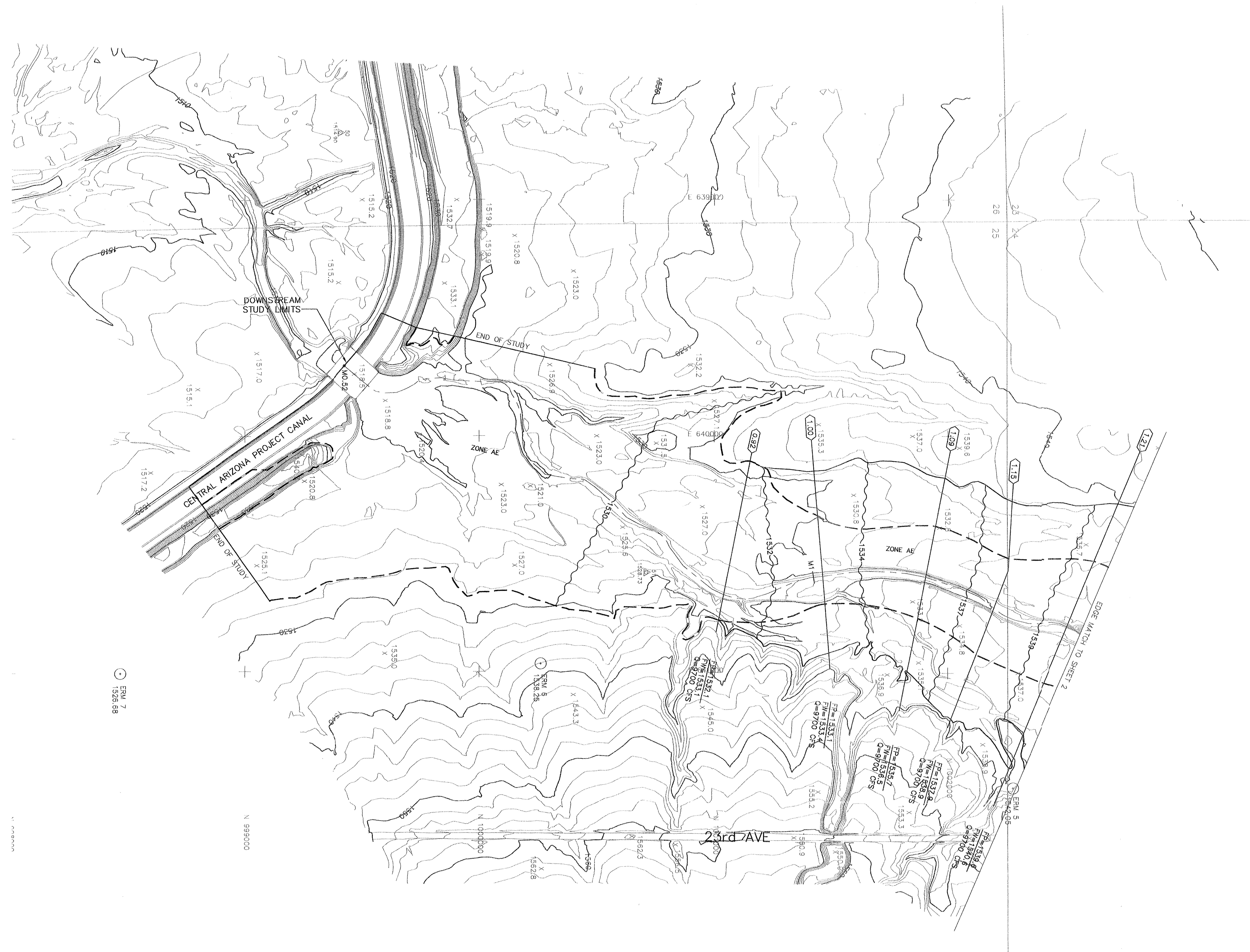
DSWA DAMON S. WILLIAMS ASSOCIATES, L.L.C.
Consulting Civil and Environmental Engineers

City of Phoenix
Water Services Department

North Gateway Water Reclamation Plant
Phase 1 – Residuals Pump Station and Force Main
Project No WS85090003

FIGURE 1

FORCE MAIN ALIGNMENT



200' 0' 200' 400'

SCALE: 1"= 200 FEET
CONTOUR INTERVAL = 2 FEET
FLIGHT DATE: JULY 20, 1999

LEGEND

100-YR FLOODPLAIN BOUNDARY	
FLOODWAY BOUNDARY	
HYDRAULIC BASE LINE WITH RIVER MILE	M12.0 M13.0
CROSS SECTION (FEMA CROSS SECTION DESIGNATION; CROSS SECTION RIVER MILE DESIGNATION)	A 0.10 FP= 1888.74 (FLOODPLAIN WATER SURFACE ELEVATION) FW= 1889.54 (ENCROACHED WATER SURFACE ELEVATION) Q = CFS (100 YEAR DISCHARGE)
ELEVATION REFERENCE MARK	ERM XXX
BASE FLOOD ELEVATIONS	1221
ZONE DESIGNATIONS	ZONE AE
SECTION LINES	
CORPORATE BOUNDARIES	

ELEVATION REFERENCE MARKS

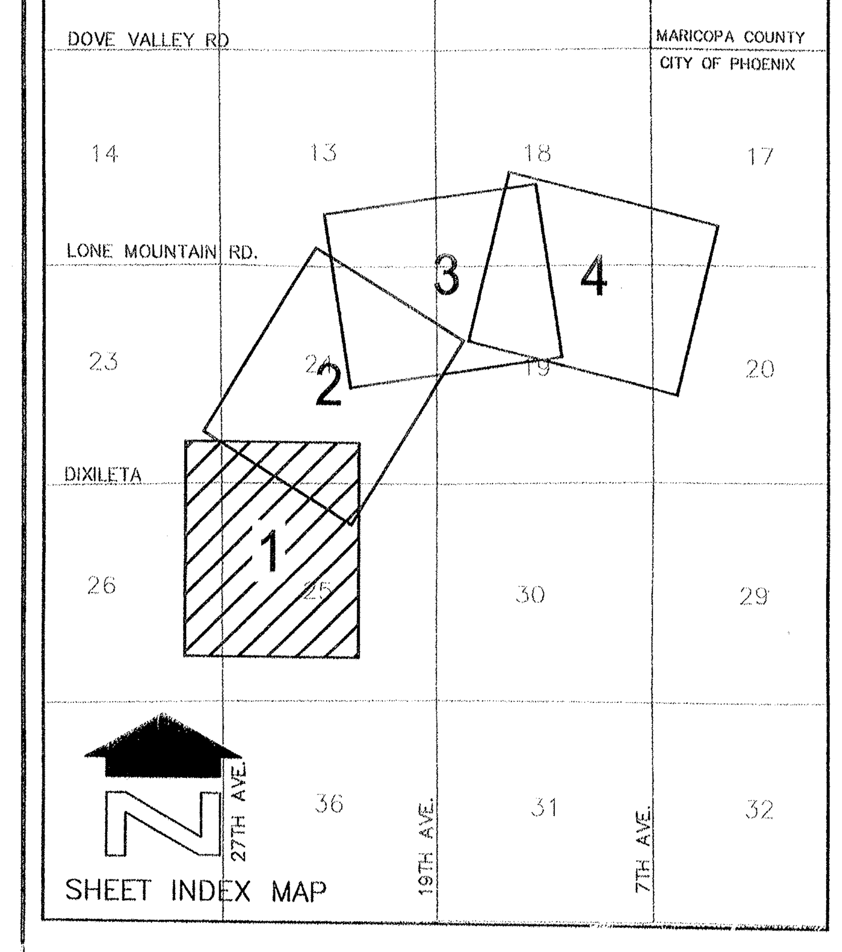
NOTE: ALL ELEVATIONS ARE BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929.

I.D. NUMBER	ELEVATION (FT)	DESCRIPTION/LOCATION
ERM 5	1542.05	3" BRASS CAP, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, SET IN CONCRETE. 1.2 FT NORTH OF A STEEL "T" POST, 19 FT SOUTH OF THE ℄ OF AN 8 FT WIDE E/W DIRT ROAD, 56 FT NORTH OF THE ℄ OF A 12 FT WIDE E/W DIRT ROAD. N: 1002279.603536 E: 641483.706425
ERM 6	1538.25	3" BRASS CAP, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, SET IN CONCRETE. 1.4 FT NORTH OF A STEEL "T" POST AND 33 FT NORTHEAST OF A MULTI-ARMED SAGUARO CACTUS. N: 1000261.013195 E: 640965.392187
ERM 7	1526.68	3" BRASS CAP, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, SET IN CONCRETE. 2.2 FT NORTH OF A STEEL "T" POST, 22 FT SOUTH OF THE ℄ OF AN 8 FT WIDE SE/NW DIRT ROAD, 53 FT WEST OF A N/S FENCE LINE (T POSTS NO WIRE) AND, 275± FT EAST OF THE EASTERN TOP EDGE OF THE CAP CANAL LEVEE. N: 998471.031574 E: 641005.539727

NOTES

1– THE HYDRAULIC BASE LINE IS CROSS SECTION STATION 10,000 UNLESS NOTED OTHERWISE

2– COORDINATES ARE IN NAD 1983 HORIZONTAL AND NGVD 1929 VERTICAL. COORDINATES ARE GRID AND IN INTERNATIONAL FEET.



SHEET INDEX MAP

NO.	REVISION	BY	DATE
2			
1			

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PLATE 3
SONORAN WASH
FLOODPLAIN WORK MAPS
F.C.D. CONTRACT NO. 99–23

Registered Professional Engineer (Civil) CERTIFICATE NO. 31680 PATRICK J. ELLISON Date Signed 8/29/01 ARIZONA U.S.A.

Stantec

	BY	DATE
DESIGN	–	–
DESIGN CHK.	–	–
PLANS	NKD	8/01
PLANS CHK.	PJE	8/01

SHEET 1 OF 4

THIS MAP WAS PREPARED BY PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1"= 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

AERIAL MAPPING COMPANY PHOTOGRAMMETRY JULY 20, 1999.

GROUND CONTROL SURVEY DATA PROVIDED BY TETRA TECH INC.

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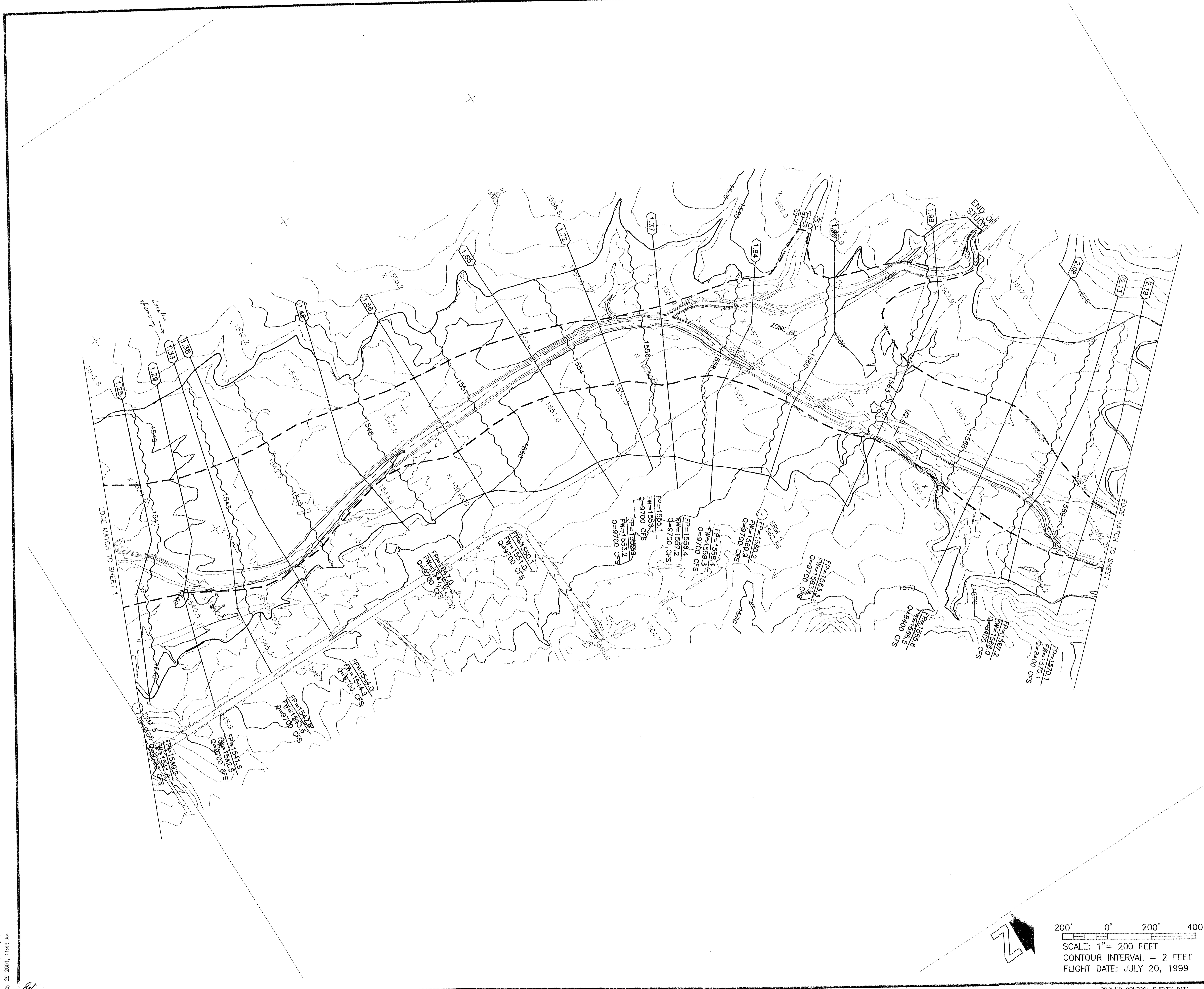
June 21 2001, 08:50 AM



200' 0' 200' 400'
SCALE: 1"= 200 FEET
CONTOUR INTERVAL = 2 FEET
FLIGHT DATE: JULY 20, 1999

LEGEND

100-YR FLOODPLAIN BOUNDARY
FLOODWAY BOUNDARY
HYDRAULIC BASE LINE WITH RIVER MILE — M12.0 M13.0
CROSS SECTION — (A) 0.10 FP= 1888.74 FW= 1889.54 Q = CFS — FEMA CROSS SECTION DESIGNATION; CROSS SECTION RIVER MILE DESIGNATION; FLOODPLAIN WATER SURFACE ELEVATION; ENCROACHED WATER SURFACE ELEVATION; 100 YEAR DISCHARGE
ELEVATION REFERENCE MARK — ERM XXX
BASE FLOOD ELEVATIONS — 1221
ZONE DESIGNATIONS — ZONE AE
SECTION LINES
CORPORATE BOUNDARIES

ELEVATION REFERENCE MARKS

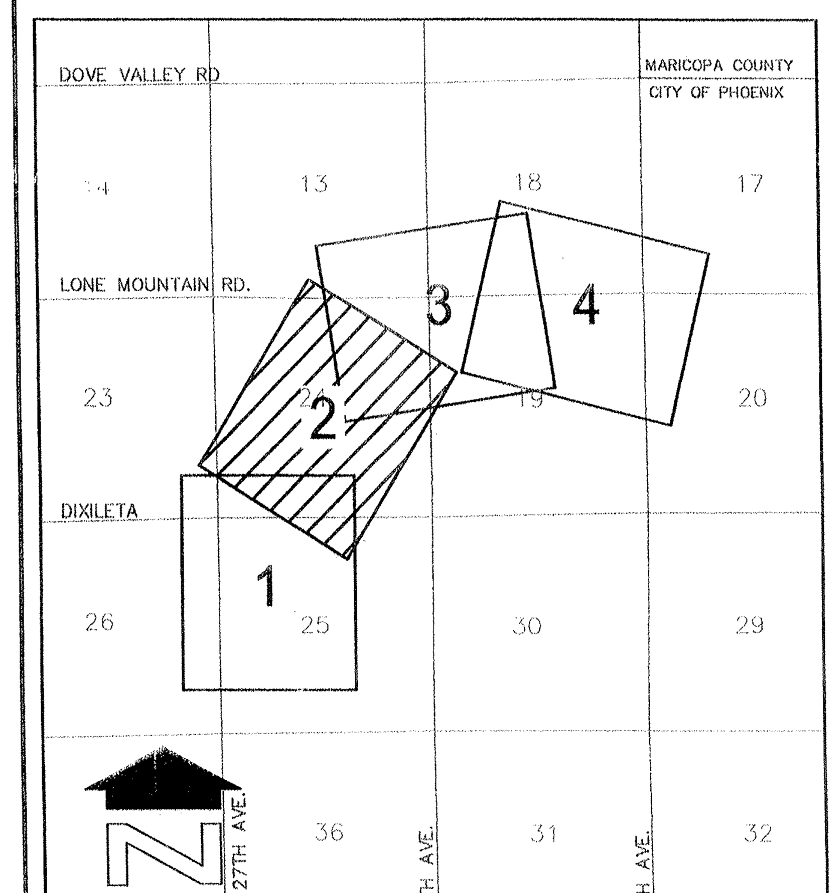
NOTE: ALL ELEVATIONS ARE BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929.

I.D. NUMBER	ELEVATION (FT)	DESCRIPTION/LOCATION
ERM 4	1562.36	3" BRASS CAP, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, SET IN CONCRETE. 1.6 FT NORTH OF A STEEL "T" POST, 42 FT EAST OF THE ℄ OF A 12 FT WIDE N/S DIRT ROAD AND 335± FT NNW OF THE NORTHWEST CORNER OF THE LANDSCAPE MATERIALS GRAVEL PIT. N: 1005114.697250 E: 642265.756249
ERM 5	1542.05	3" BRASS CAP, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, SET IN CONCRETE. 1.2 FT NORTH OF A STEEL "T" POST, 19 FT SOUTH OF THE ℄ OF AN 8 FT WIDE E/W DIRT ROAD, 56 FT NORTH OF THE ℄ OF A 12 FT WIDE E/W DIRT ROAD. N: 1002279.603536 E: 641483.706425

NOTES

1– THE HYDRAULIC BASE LINE IS CROSS SECTION STATION 10,000 UNLESS NOTED OTHERWISE

2– COORDINATES ARE IN NAD 1983 HORIZONTAL AND NGVD 1929 VERTICAL. COORDINATES ARE GRID AND IN INTERNATIONAL FEET.



SHEET INDEX MAP

NO.	REVISION	BY	DATE
2			
1			

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PLATE 3
SONORAN WASH
FLOODPLAIN WORK MAPS
F.C.D. CONTRACT NO. 99–23

Stantec

	BY	DATE
DESIGN	–	–
DESIGN CHK.	–	–
PLANS	NKD	8/01
PLANS CHK.	PJE	8/01

SHEET 2 OF 4

THIS MAP WAS PREPARED BY PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1"= 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

AERIAL MAPPING COMPANY PHOTOGRAMMETRY JULY 20, 1999.

GROUND CONTROL SURVEY DATA PROVIDED BY TETRA TECH INC.

20. ERM EL. = 1679.52

This station is located on the top of Cave Butte Dam just east of a concrete structure. The mark is an aluminum cap in a concrete monument marked " CB-3 1980 ".

21. ERM EL. = 1535.25

This station is located approximately on the section line of section 9 and 10, T4N, R3E 60 feet +/- south of the southeast corner of the Knochel Brothers lease land. The mark is a brass cap epoxyed to a large boulder, marked "MCFCD 1535.25 CAVE CREEK".

22. ERM EL. = 1562.57

This station is located on the east side of the east main road from Cave Butte Dam, 0.15 miles south of an east west transmission line. The mark is an aluminum cap epoxyed to a headwall, marked " MCFCD 1562.57 CAVE CREEK ".

39. ERM EL. = 1553.88

This station is located just east of the west line of Section 1, T4N, R3E, on the east - west transmission line. The mark is a chiseled "+" on the concrete base for the southeast leg of a transmission tower.

Summary of Discharges

Flooding Source and Location	D.A. (S.M.)	10-YR (cfs)	50-YR (cfs)	100-YR (cfs)
Cave Creek Wash				
Above the Central Arizona Project Canal	2.5	1,700	2,500	2,900
Below the Central Arizona Project Canal	4.0	2,700	4,000	4,600
Below Deer Valley Drive	5.0	3,000	4,500	5,200
Above the East Fork Cave Creek	8.6	3,300	4,900	5,700
East Fork of Cave Creek				
Near Paradise Lane & Central Avenue	13.4	5,300	7,900	9,100
Near Coral Gables & 7th Avenue	14.1	5,500	8,200	9,400
Mouth (Confluence with Cave Creek)	14.6	5,500	8,300	9,500
Cave Creek Wash				
Below the East Fork Cave Creek	23.5	8,400	12,700	14,600
Below 19th Avenue	25.5	8,900	13,400	15,400
Above Moon Valley Wash	26.5	8,900	13,400	15,400
Below Moon Valley Wash	33.1	10,100	15,300	17,500
At the Confluence with the Arizona Canal Diversion Channel	34.7	10,300	16,100	18,500

CONTOUR INTERVAL = 2 FEET

NO.	REVISIONS	DATE	BY	CHK.

Burgess & Niple, Inc.
Engineers and Architects
Akron, OH • Cincinnati, OH • Columbus, OH • Crestview Hills, KY
Painesville, OH • Parkersburg, WV • Payson, AZ • Phoenix, AZ

MIDDLE CAVE CREEK FLOODPLAIN DELINEATION STUDY
FCD 88-56
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

JOB NO. 7833
DESIGNED BY: JEM
DRAWN BY: CEU / AK
CHECKED BY: JEM
APPROVED BY:
DATE: MARCH-91

FLOOD BOUNDARY AND FLOODWAY MAP

SCALE: 0 100 200 SCALE IN FEET

SHEET NO. 11 OF 11

Ref. 2