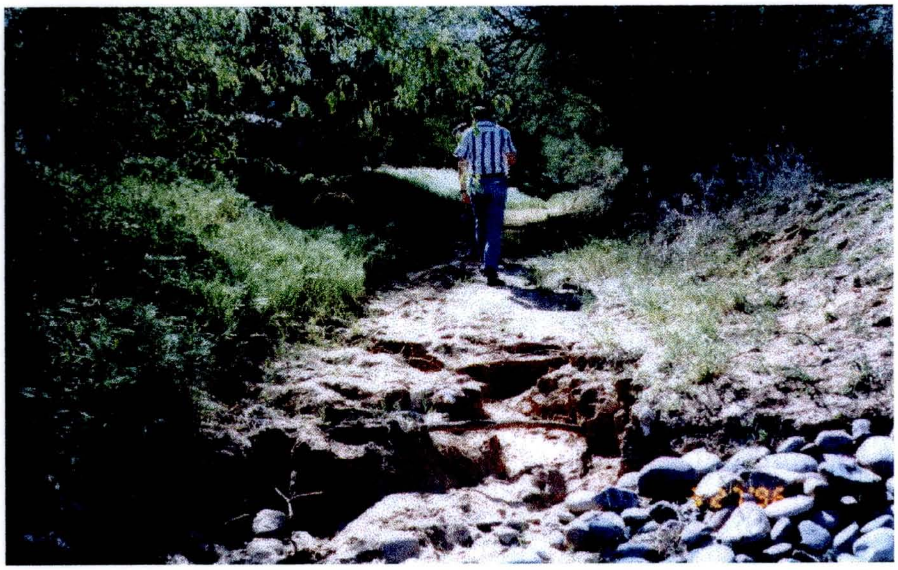
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
PHOENIX, ARIZONA

CHEROKEE WASH
HYDROLOGIC, HYDRAULIC AND SEDIMENTATION STUDY

FINAL REPORT



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JUNE 30, 1997

HYDROLOGIC, HYDRAULIC AND SEDIMENTATION STUDY

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Prepared for
THE FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY

FINAL REPORT
June 30, 1997

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EXECUTIVE SUMMARY

The Cherokee Wash study area is located within the Town of Paradise Valley near Phoenix, Arizona. Residents along Cherokee Wash have been flooded several times, including 1993 when access to a local elementary school was cut off for several hours. The Flood Control District of Maricopa County (FCD) authorized this study as part of the larger Doubletree Ranch Drainage Improvement Project to investigate flood control improvements for Cherokee Wash. A hydrologic, hydraulic and sedimentation analysis of Cherokee Wash was conducted by WEST Consultants, Inc. (WEST) for FCD under contract number FCD 95-37. The study was conducted to formulate and evaluate the hydrologic, hydraulic and sedimentation impacts on possible channelization and/or detention plans for Cherokee Wash.

Cherokee Wash in its existing condition cannot convey the flood event expected to occur, on the average, every two years (the 2-year event). Several options were considered to improve the capacity of the channel, from enlarged rectangular concrete-lined cross sections to selective widening of cross sections to convey the 2-year event. The latter option was chosen as the preferred alternative due to perceived costs, area geology, public reaction, and other factors.

Sedimentation and geomorphic studies were conducted for Cherokee Wash both in its existing condition and with the preferred alternative (with-project) condition. Results of the analyses indicate that Cherokee Wash is degrading. Long-term simulations of "average" conditions show that as much as six feet of degradation could be expected downstream of two road crossings at 56th Street and Mockingbird Lane. This scour could, however, be limited by bedrock, the depth of which is not known at these locations.

Recommendations are given herein for selective channel geometry modifications, grade control structures, channel vegetation maintenance and improved culverts at the 56th Street crossing. All existing dip road crossings along the study reach are not safe, based on estimated depths and velocities, for events greater than or equal to the 2-year event. Safety could be improved by installing culverts at the crossings and raising the roads, but this would reduce the already small channel conveyance.

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Appendix A: Cross Section Plots

Map Pocket: HEC-2/HEC-6 Cross Section Locations

1.0 INTRODUCTION

1.1 Purpose

Residents along Cherokee Wash have been flooded several times, including 1993 when access to the Cherokee Elementary School was cut off for several hours. The Flood Control District of Maricopa County (FCD) authorized this study as part of the larger Doubletree Ranch Drainage Improvement Project to investigate flood control improvements for Cherokee Wash. A hydrologic, hydraulic and sedimentation analysis of Cherokee Wash was conducted by WEST Consultants, Inc. (WEST) for FCD under contract number FCD 95-37. The study was conducted to formulate and evaluate the hydrologic, hydraulic and sedimentation impacts on possible channelization and/or detention plans (collectively called "improvements" herein) for Cherokee Wash.

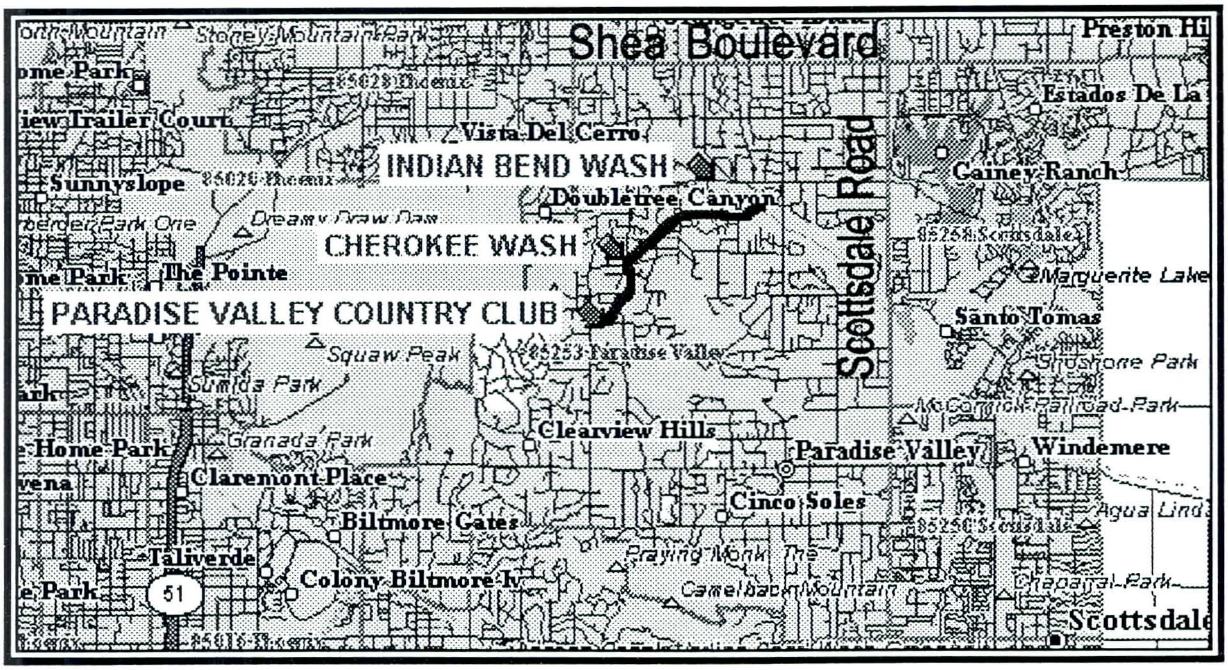
1.2 Study Area

Cherokee Wash is located in Paradise Valley and Phoenix, Arizona (Figure 1.1). The study area extends from the wash's headwaters in the Paradise Valley Country Club/Mummy Mountain to the its confluence with Indian Bend Wash. Cherokee Wash flows northeast into Indian Bend Wash at an elevation of 1319.3 feet. The drainage area is approximately two square miles and includes two major tributaries. A topographic map of the Cherokee Wash drainage basin is shown as Figure 1.2. Cherokee Wash has 12 road crossings, most of which are dip sections in residential streets. The reach is unimproved and varies greatly both in channel size and bed material.

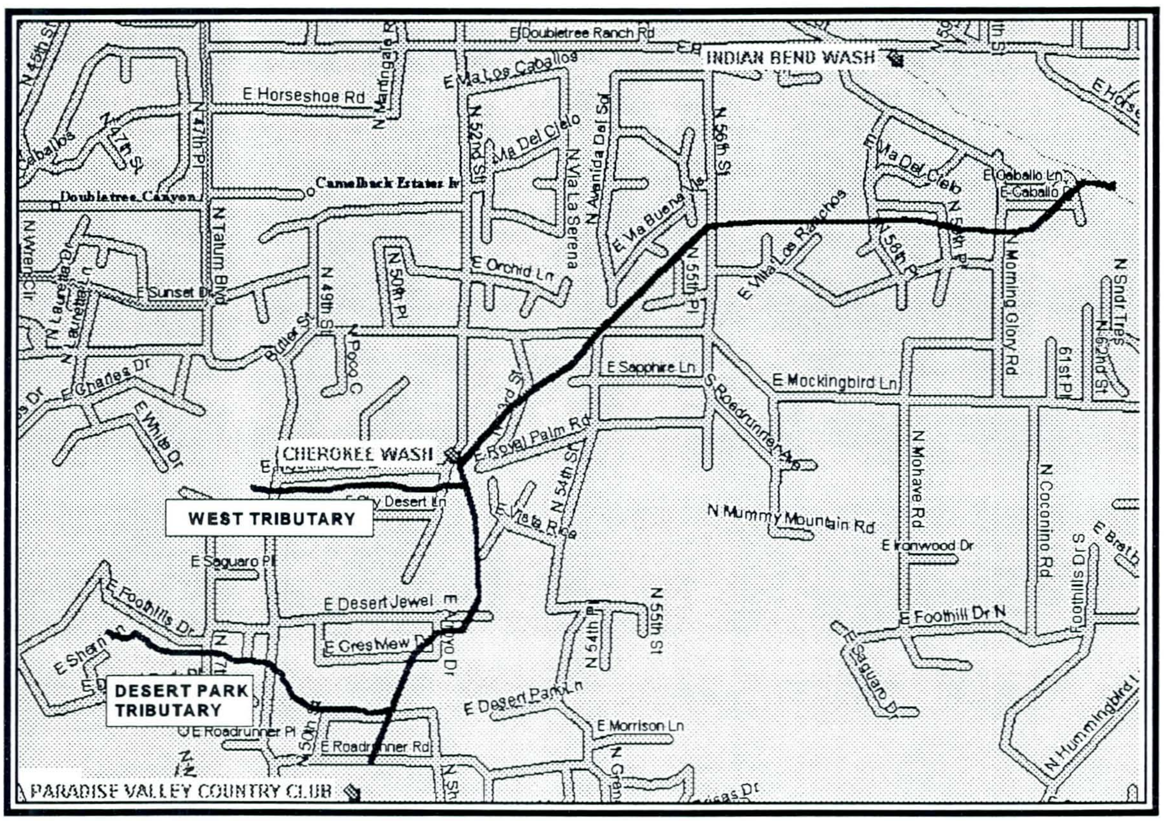
1.3 Acknowledgments

Mr. Martin Teal was the project manager for this study and was supported by Mssrs. Jeffrey Daniel and David Smith. Mr. Teal conducted the HEC-6 analysis and wrote a majority of this report. Dr. David Williams of WEST was the principal-in-charge for this study. He developed the technical approach used and provided quality assurance services. Dr. Williams and Mr. Teal conducted the field reconnaissance of the study area.

Ms. Marilyn DeRosa was Project Manager for FCD and provided invaluable assistance in conducting the study. Mr. Bruce Wolle of Law/Crandall coordinated the geotechnical investigation. Their contributions to this study are gratefully acknowledged.



(a)



(b)

Figure 1.1 - Location Maps: (a) General Location, (b) Close-up View

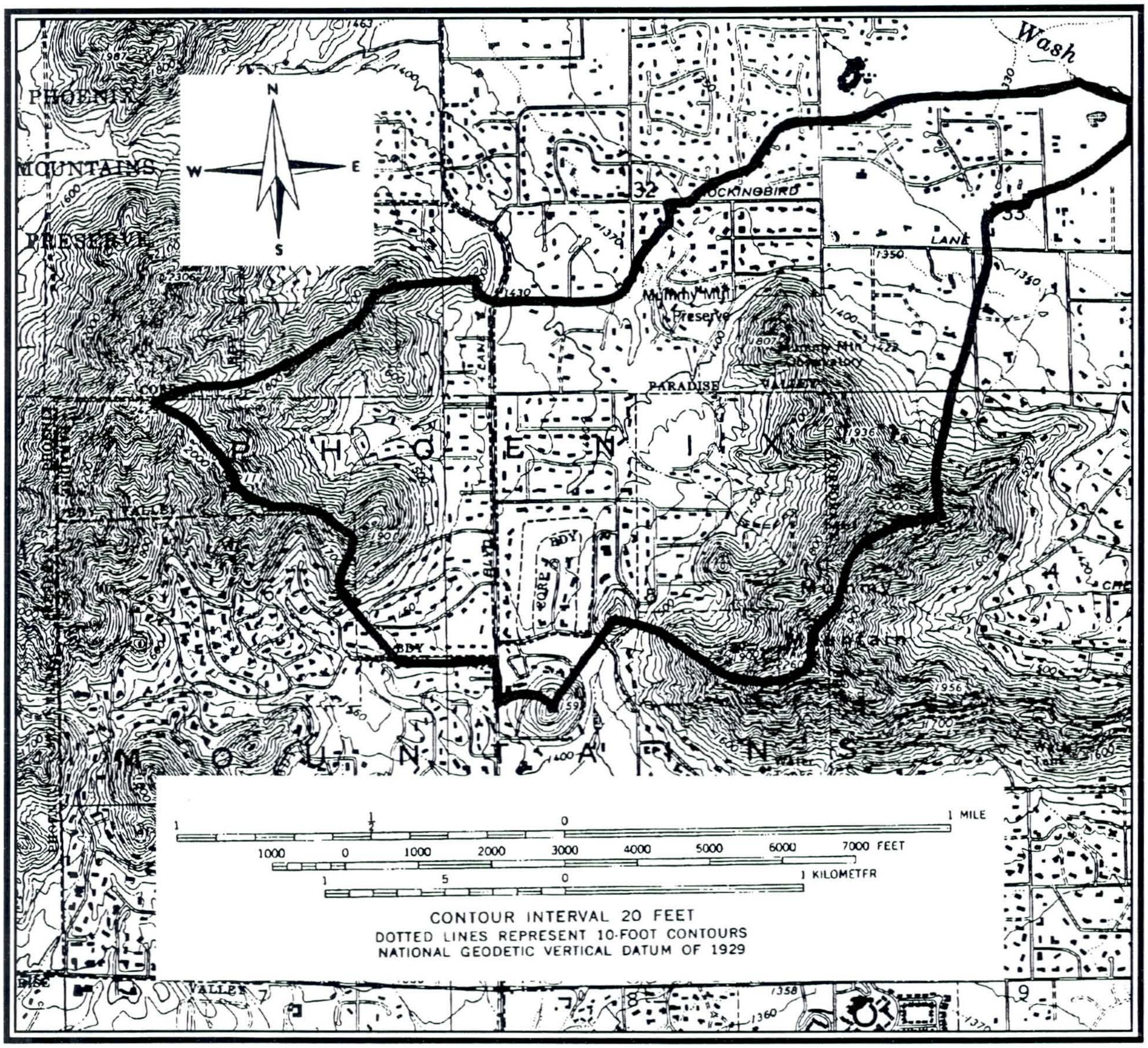


Figure 1.2 - Cherokee Wash Drainage Basin (USGS Topographic Map, 1965, Photorevised 1982)

2.0 HYDROLOGY

2.1 Introduction

The purpose of this section is to a) document WEST's review of the Doubletree Ranch Road Regional Drainage Study reports and Cherokee Wash HEC-1 models provided by the Maricopa County Flood Control District, and b) describe additional modeling efforts performed by WEST.

2.2 Doubletree Ranch Road Studies

The Doubletree Ranch Road Regional Drainage Study Existing Hydrology report was prepared by Kaminski-Hubbard Engineering (KHE) for Hook Engineering in November 1995 (KHE, 1995). This study covered a four square mile area which includes the Cherokee Wash drainage basin as well as other areas. Hydrologic models (HEC-1; USACE, 1990a) were prepared as part of this study. The report analyzed the existing condition hydrology for the 2-, 10-, 50-, and 100-year events for both the 6- and 24-hour duration storms. All models use the Clark Unit Hydrograph method, the Green-Ampt Loss Method, and generally follow procedures specified in the Maricopa County Drainage Design Manual, Volume I, Hydrology (FCDMC, 1995). A split flow analysis was included in the KHE report for water leaving Cherokee Wash at 56th Street.

This report was subsequently revised to reflect proposed flow diversions (KHE, 1996). An updated HEC-1 model was prepared by KHE for the 100-year, 6-hour duration event for "with-project" conditions. The project conditions include diversions at 52nd Street and 56th Street which intercept all flow up to the 100-year storm runoff.

2.2.1 HEC-1 Models

Nine HEC-1 models were provided to WEST by FCD in two subdirectories labeled DOUBLE and FCD. The DOUBLE subdirectory contained 8 files. Input files in the first group are named DTX6E.DAT for the 6-hour storm, existing conditions. The X in the file name takes on the value of 0 for the 2-year input file, 1 for the 10-year input file, 2 for the 50-year input file and 3 for the 100-year input file. Input files in the second group are named DTX24E.DAT for the 24-hour storm, existing conditions. The X values are the same as just described for the first group of files. These models were run and the output was found to correspond with results presented in Table 1 of the Doubletree Ranch Road existing conditions hydrology report (KHE, 1996).

Only one file was in the FCD subdirectory, DT36NME.DAT. This is a modified version of DT36E.DAT (100-year, 6-hour event) where all flows from Cherokee Wash at 56th Street are diverted to the Doubletree Ranch Road Project (file updated 2/27/96 by AA).

An additional HEC-1 data file, 298IMP10.DAT, was provided with the May 1996 KHE report. This model used the 100-year, 6-hour rainfall event and included flow diversions at 52nd Street and 56th Street (the diversions capture 100% of the incoming flow). When compared with the existing conditions model, several other changes were noted in the flow routing scheme as well.

2.3 Comparison of Existing Conditions HEC-1 Peak Flows with USGS Peak Flows

Results from KHE's 1996 report were compared with regression equation flows to check for reasonableness. Cherokee Wash lies in Central Arizona Region 12 (USGS, 1994). Discharge estimates for this region are a function of basin area and mean basin elevation (1.94 sq. miles and 1450 feet MSL, respectively). Flows calculated using USGS regression equations for the outlet of Cherokee Wash at Indian Bend Wash were much higher than the HEC-1 flows for the 100- and 50-year events, and were lower for the 10- and 2-year events as shown in Table 2.1. However, it is difficult to draw any conclusions about model accuracy from this comparison given the large standard errors associated with the regression equations (from 39 to 105 percent in Region 12's case).

Table 2.1 - Peak Flow Comparison

Event (6-hour rainfall)	HEC-1 Peak Flow[1] (cfs)	USGS Peak Flow[2] (cfs)
2-year	219	62
10-year	775	640
50-year	1490	2159
100-year	1790	3385

[1]KHE, 1996
[2]USGS, 1994

2.4 Storm Duration

Because the HEC-1 peak discharge results for both the 6-hour and 24-hour storms were nearly identical, the report recommends using the 6-hour duration storm to be consistent with earlier hydrology studies performed by FCD. However, the 24-hour storm does produce a slightly higher runoff volume which may be significant for detention basin design. Also, the FCD Hydrology Manual specifies the 100-year, 2-hour rainfall as the design event for on-site retention/detention facilities (page 2-3).

The existing conditions report compares its flow results with those obtained in an earlier study by Erie and Associates (1993). The 100-year, 6-hour results from the existing conditions report are lower than the 100-year, 2-hour results from the earlier study. The existing conditions report cites the fact that the earlier study did not reduce point rainfall values. However, the FCD Hydrology Manual explains that point rainfall reduction should not occur when using the 2-hour rainfall (page 2-20). Therefore, according to FCD criteria, the higher values should be used for detention basin design. For other design purposes (e.g., water surface profiles), flows resulting from the 100-year, 6-hour rainfall used in the existing conditions report should be sufficient.

2.5 WEST Hydrologic Models

The HEC-1 model from the May 1996 report (100-year, 6-hour rainfall) was deemed suitable for use by WEST. District procedures were followed in its preparation and node locations were suitable for later sediment analyses.

Although the model from the May 1996 KHE report was suitable, no models were prepared for the 2-, 10-, and 50-year events. Therefore, WEST used the 100-year model and revised the rainfall, unit hydrograph parameter and routing step parameters for each of the three other events. The 6-hour rainfall and unit hydrograph parameters from the existing conditions models were used. We calculated the number of routing steps for reaches where the routing had been changed from the existing conditions model.

Flow diversions from Cherokee Wash at 52nd and 56th streets, originally part of the base hydrology plan, were later eliminated. Therefore, it became necessary to modify the existing HEC-1 models to reflect the new flow conditions without diversions. The models discussed above were modified to eliminate the flow diversions. The split flow analysis performed by KHE for 56th Street appeared satisfactory and was included in the updated WEST models. The flows at key points along Cherokee Wash, both with and without diversions, are presented in Table 2.2.

Obviously, flows above the most upstream diversion (52nd Street) did not change with removal of the diversions. The changes were dramatic, however, for the larger flow events in the downstream reaches of the channel. Note that the model assumes that all flow calculated at a node is conveyed to the next node downstream, i.e., no water volume is being stored due to ponding in the overbanks and peak discharge is not affected. However, some flow will probably be stored in the overbank areas for the large flood events; therefore, the flow estimates are conservative. The new discharge estimates were input to the HEC-2 models to develop water surface profiles and HEC-6 models for sediment continuity analyses.

Table 2.2 - Selected Peak Flows (in cfs)

Location	HEC-1 ID	100-year		50-year		10-year		2-year	
		Q_{nodiv}	Q_{div}	Q_{nodiv}	Q_{div}	Q_{nodiv}	Q_{div}	Q_{nodiv}	Q_{div}
Roadrunner Rd.	HC335	425	425	364	364	218	218	69	69
Desert Jewel Dr.	HC365	1157	1157	964	964	514	514	160	160
52nd St.	HC385	1571	1571	1304	1304	689	689	213	213
Mockingbird Ln.	HC395	1631	70	1351	56	711	25	216	4
56th St.	HC405	1784	232	1475	189	773	90	230	19
Indian Bend Wash	HC435	1791	431	1491	349	776	166	219	37

2.6 With-Project Hydrology

Proposed improvements to Cherokee Wash in the form of channel geometry modifications and/or sedimentation basins are described in Section 3.4 of this report. The proposed channel widening at several cross sections will not significantly impact the flow routings in the HEC-1 models. Sedimentation basins, which could alter hydrology, are not recommended because the channel appears to be degrading. In any case, reductions in flow due to small sediment basins would be very small.

In the current hydrology models, a channel roughness coefficient of 0.03 is used for all routing reaches. To see the effects of using a smoother channel (e.g., concrete) this roughness was changed to 0.02 for a single run of the 100-year event. The biggest increase in flow was 14 cfs and occurred upstream of 56th Street. The increase at the confluence with Indian Bend Wash was 9 cfs, and all other increases throughout the basin were smaller. Increases for events more frequent than the 100-year flood would be smaller still. These increases in flow are within the amount of error that could be expected in a hydrologic model of this type. Therefore, the hydrology used in this study should not change significantly, even if smoother channels are constructed.

3.0 HYDRAULICS

3.1 Introduction

The purpose of this section is to document WEST's review of existing HEC-2 (USACE, 1990b) hydraulic models and development and analysis of new HEC-2 models for Cherokee Wash.

WEST was provided models by FCD based on previous work by Erie and Associates (1993). However, the model cross sections differed from field reconnaissance observations; in some cases the model cross sections were two or three times the actual channel width. FCD therefore authorized a new survey of Cherokee Wash in November, 1996 to obtain more accurate cross sections. The new surveys were performed by Morrison Maierle/CSSA under separate contract to FCD. All HEC-2 and HEC-6 models described in the following sections were constructed using these new cross sections. Cross section locations are shown on the map enclosed with Appendix A.

3.2 HEC-2 Model Development

FCD provided WEST with 69 surveyed cross-sections in HEC-2 digital format. A plan view was also provided in digital (DXF) and hard copy formats. WEST developed the HEC-2 model CHERORIG.DAT using the 46 main wash sections and the 23 south branch sections provided by FCD. The downstream model limit, cross section 0.0, is located at the confluence of Cherokee Wash and Indian Bend Wash. The upstream model limit, cross section 11035.0, is located on Road Runner Road. In creating the model, we:

- Determined Manning's n values from the FCD HEC-2 model and from two field visits performed by WEST (March 27, 1996, September 28, 1996). The Manning's n values are reported in Table 3.1.
- Implemented channel discharges obtained from the updated HEC-1 models for the 100-, 50-, 10-, and 2-year events. Discharge from in-line subbasins (i.e., the wash flows through the subbasin) was included in the discharge for the entire wash reach running through the particular subbasin. Flows from off-line subbasins were only included in the discharge downstream of their confluence with the main wash. Resulting flows input to HEC-2 for the different flow events are presented in Table 3.2.
- Determined starting water surface elevations (WSEL's) for the four flows by the slope-area method using the channel thalweg slope between 61st Place and Indian Bend Wash. The previous models for the 100-year event had used the 100-year WSEL in Indian Bend Wash as the starting WSEL. The slope-area method is a better approximation of the WSEL because the 100-year event will not necessarily occur in Indian Bend Wash at the same time the 100-year event occurs on Cherokee Wash. This method also provides a basis for starting WSEL's for events other than the 100-year flood.

Table 3.1 - Manning's n Values

LOCATION	SECNO	Manning's n Value		
		left	channel	right
Indian Bend	0.0	0.03	0.03	0.03
	300.0	0.03	0.03	0.03
d/s road	384.3	0.03	0.03	0.03
Caballo Ln.	409.3	0.02	0.02	0.02
u/s road	434.3	0.03	0.03	0.03
	600.0	0.03	0.03	0.03
	900.0	0.03	0.03	0.03
d/s road	1095.2	0.03	0.03	0.03
Morning Glory	1120.2	0.02	0.02	0.02
u/s road	1145.2	0.06	0.045	0.06
	1200.0	0.06	0.045	0.06
	1500.0	0.06	0.045	0.06
d/s road	1718.2	0.06	0.045	0.06
59th Place	1743.2	0.02	0.02	0.02
u/s road	1768.2	0.04	0.035	0.04
	1800.0	0.04	0.035	0.04
	2100.0	0.04	0.035	0.04
	2400.0	0.04	0.035	0.04
d/s road	2502.4	0.04	0.035	0.04
58th Place	2527.4	0.02	0.02	0.02
u/s road	2552.4	0.055	0.04	0.055
	2700.0	0.05	0.03	0.05
	3000.0	0.05	0.03	0.05
	3300.0	0.05	0.03	0.05
	3600.0	0.05	0.03	0.05
	3900.0	0.05	0.03	0.05
	4200.0	0.05	0.03	0.05
d/s road	4251.5	0.05	0.03	0.05
56th Street	4271.5	0.02	0.02	0.02
u/s road	4293.2	0.05	0.04	0.05
	4500.0	0.05	0.04	0.05
	4800.0	0.05	0.04	0.05
	5100.0	0.05	0.04	0.05
	5400.0	0.05	0.04	0.05
d/s road	5650.0	0.05	0.04	0.05
Mockingbird Ln.	5693.0	0.02	0.02	0.02
u/s road	5727.7	0.06	0.06	0.06
	6000.0	0.06	0.06	0.06
	6300.0	0.06	0.06	0.06
d/s road	6533.8	0.06	0.06	0.06
53rd Street	6558.8	0.02	0.02	0.02
u/s road	6600.0	0.05	0.04	0.05
	6900.0	0.05	0.04	0.05
	7200.0	0.05	0.04	0.05
	7500.0	0.05	0.04	0.05
52nd Street	7800.0	0.05	0.04	0.05
	8100.0	0.06	0.06	0.06
	8400.0	0.06	0.06	0.06
	8700.0	0.06	0.06	0.06
	9000.0	0.06	0.045	0.06
d/s road	9205.0	0.06	0.045	0.06
Desert Jewel Dr.	9230.0	0.02	0.02	0.02
u/s road	9255.0	0.06	0.045	0.06
	9300.0	0.06	0.045	0.06
d/s road	9448.0	0.06	0.045	0.06
Arroyo Dr.	9473.0	0.02	0.02	0.02
u/s road	9498.0	0.06	0.045	0.06
	9600.0	0.06	0.045	0.06
d/s road	9859.5	0.06	0.045	0.06
Crestview Dr.	9884.5	0.02	0.02	0.02
u/s road	9900.0	0.06	0.045	0.06
	10200.0	0.06	0.045	0.06
d/s road	10373.0	0.06	0.045	0.06
Desert Park Ln.	10398.0	0.02	0.02	0.02
u/s road	10423.0	0.05	0.045	0.05
	10500.0	0.05	0.045	0.05
	10800.0	0.05	0.045	0.05
d/s road	10999.0	0.05	0.045	0.05
Road Runner Rd.	11035.0	0.02	0.02	0.02

- Determined ineffective flow areas due to expansion/contraction effects. This consideration was especially important at dip road crossings and where flow was constricted by walls or buildings.

Table 3.2 - Flow Change Locations in HEC-2 Models

Reach Limits by Section Number	Q_{100} (cfs)	Q_{50} (cfs)	Q_{10} (cfs)	Q_2 (cfs)
0 - 4251.5	1791	1491	776	219
4283.2 - 5100	1784	1475	773	230
5400 - 7500	1631	1351	711	216
7800	1571	1304	689	213
8100 - 10200	1400	1162	619	194
10373 - 10500	822	691	383	109
10800 - 11035	425	364	218	69

3.3 HEC-2 Model Analysis

The model CHERORIG.DAT was run for the 100-, 50-, 10-, and 2-year events using discharges from the modified HEC-1 models. The results were used to perform specific analyses described below.

Output from CHERORIG.DAT contains no error messages. There are numerous warnings and cautions of "Conveyance Change Outside Acceptable Range", "Critical Depth Assumed", and "Minimum Specific Energy". The critical depth and minimum specific energy messages appear mostly at road crossings where the flow may become supercritical and at a few other locations. The conveyance warning message is caused by a large difference in conveyance between cross sections and is a common warning in HEC-2.

For plotting purposes the HEC-2 model CHERORIG.DAT was input into HEC-RAS (USACE, 1995a) and run using the HEC-2 conveyance option. Cross sections were plotted along with the water surfaces for the 100-year (Q100), 50-year (Q50), 10-year (Q10) and 2-year (Q2) events. These plots are presented in Appendix A. Ineffective flow areas are represented as solid areas in the cross section plots.

3.3.1 Identification of Hydraulically Similar Channel Reaches

The channel was divided into 10 hydraulically similar channel reaches based on the channel slope, velocity, depth and width. Channel thalweg slope (Figure 3.1) was plotted and compared to channel discharge (Table 3.2). The resulting 10 reach locations are shown in Figure 3.1 and Table 3.3.

3.3.2 Identification of Discharge Thresholds, Flood Breakout Points and Choke Points

The discharge threshold for a cross section is the amount of flow the section can convey with the water surface elevation at the defined channel "breakout" elevation. Therefore, any discharge exceeding the discharge threshold will exceed the channel capacity as defined by the channel breakout elevation. The breakout elevation often, but not always, matches one of the bank station elevations as defined in HEC-2. In instances where a low flow channel was within a larger channel, the larger channel was used for discharge threshold calculations. Discharge thresholds were determined as follows:

- Each cross section was analyzed using surveyed geometry provided by FCD and photos from site visits to determine the channel breakout points on each bank. Breakout elevations were often defined by obvious channel banks or the base of a fence or wall. In cases where no obvious feature defined the breakout point, or more than one feature was present, engineering judgement was applied. The elevation of the lowest of the two defined points for a given cross section was used as the threshold elevation (Table 3.3).

- Cross section geometry, threshold elevation, Manning's n values and average longitudinal reach slope were used as input to the WinXSPRO (WEST, 1996) computer program.

- Using normal depth equations, WinXSPRO computed a discharge for a water surface elevation equal to the threshold elevation. WinXSPRO computes discharge for a single cross-section only and does not account for any backwater effects; however, these effects are negligible for the purpose of this analysis.

- The discharge threshold for each cross section is reported in Table 3.3. Discharge thresholds were not computed for road crossings. The channel capacity is shown in Figure 3.2 along with return period discharges for comparison.

Although thresholds were not computed for the center-of-road cross sections, they were computed for the sections just upstream and downstream of the crossing. These cross sections are in a sense transition sections between the road crossing (wide dip section) and a normal channel section with defined banks. If a surveyed section resembles a dip section, the calculated capacity will be very high because the broad section would be able to convey much more water than a more defined "normal" wash section. This can be seen, for example, at the Caballo Lane and Morning Glory Road crossings

CHEROKEE WASH SEDIMENTATION STUDY

Figure 3.1 - Hydraulically Similar Channel Reaches

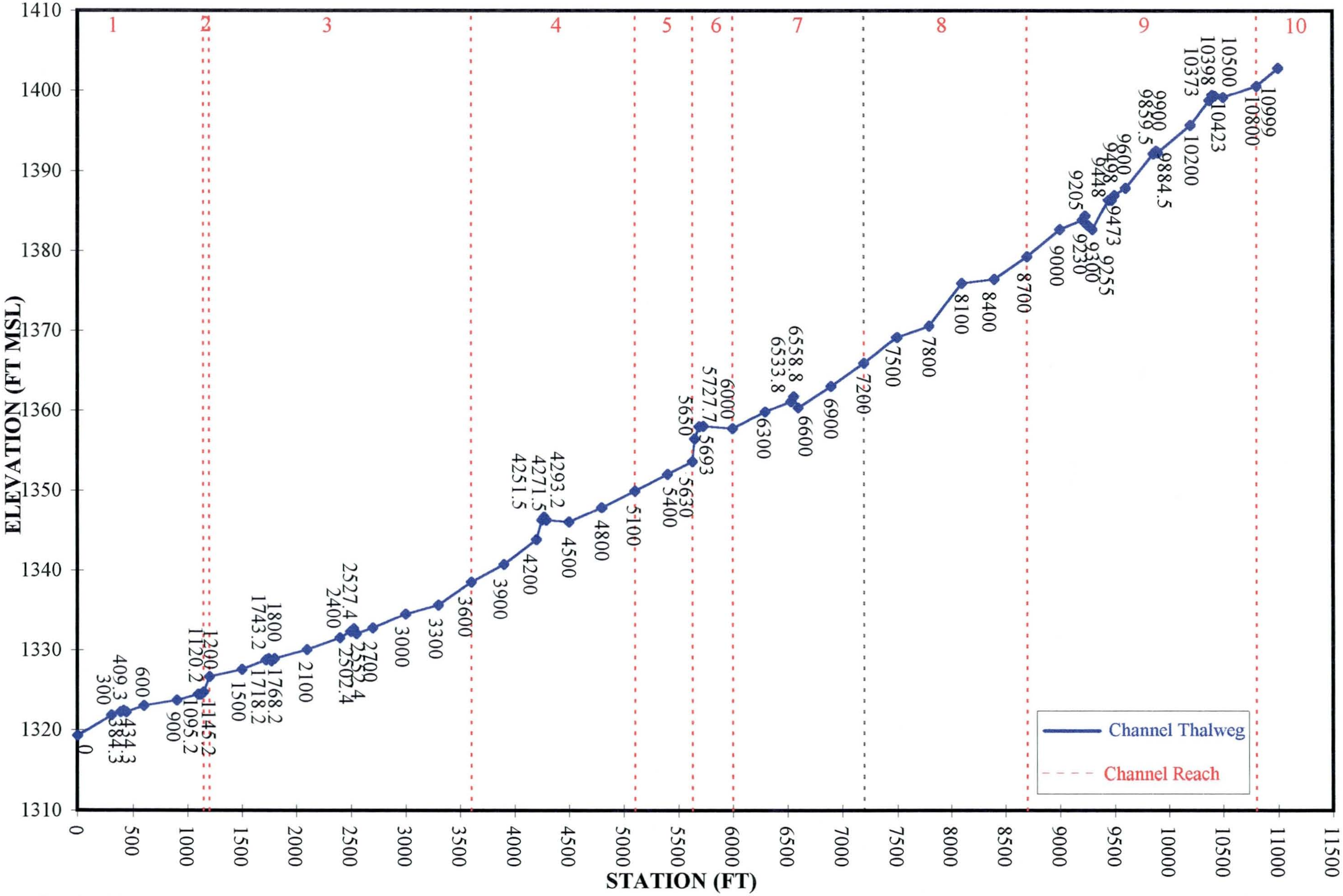


Table 3.3 - Channel Properties

Section Number	Channel Reach	Q_{100} (cfs)	Discharge Threshold (cfs)	Threshold Elevation (ft.)	Breakout Points	Choke Point	Road Crossings
0.0	1	1791	326	1321.6	X	X	Indian Bend Wash
300.0	1	1791	517	1326.0	X	X	
384.3	1	1791	989	1326.6	X		d/s road
409.3	1	1791					Caballo Lane
434.3	1	1791	991	1326.6	X		u/s road
600.0	1	1791	180	1325.8	X	X	
900.0	1	1791	375	1327.8	X	X	
1095.2	1	1791	975	1328.3	X		d/s road
1120.2	1	1791					Morning Glory Rd.
1145.2	2	1791	724	1328.8	X		u/s road
1200.0	2	1791	113	1329.3	X	X	
1500.0	3	1791	187	1331.3	X	X	
1718.2	3	1791	432	1333.4	X		d/s road
1743.2	3	1791					59th Place
1768.2	3	1791	220	1332.1	X		u/s road
1800.0	3	1791	151	1331.7	X		
2100.0	3	1791	308	1333.8	X	X	
2400.0	3	1791	472	1335.5	X		
2502.4	3	1791	270	1334.5	X	X	d/s road
2527.4	3	1791					58th Place
2552.4	3	1791	189	1335.2	X	X	u/s road
2700.0	3	1791	497	1337.9	X	X	
3000.0	3	1791	496	1339.5	X		
3300.0	3	1791	917	1341.7	X		
3600.0	3/4	1791	677	1343.8	X	X	
3900.0	4	1791	755	1344.9	X		
4200.0	4	1791	474	1347.3	X	X	
4251.5	4	1791			X		d/s road
4271.5	4	1784					56th Street
4293.2	4	1784			X		u/s road
4500.0	4	1784	491	1349.4	X		
4800.0	4	1784	281	1350.6	X		
5100.0	4/5	1784	317	1352.8	X	X	
5400.0	5	1631	560	1356.4	X		
5630.0	5/6	1631	560	1358.0	X	X	
5650.0	6	1631			X		d/s road
5693.0	6	1631					Mockingbird Lane
5727.7	6	1631	100	1359.5	X		u/s road
6000.0	6	1631	317	1360.8	X		

Table 3.3 - Channel Properties

Section Number	Channel Reach	Q_{100} (cfs)	Discharge Threshold (cfs)	Threshold Elevation (ft.)	Breakout Points	Choke Point	Road Crossings
6300.0		1631	263	1363.0	X		
6533.8		1631	418	1365.3	X	X	d/s road
6558.8	7	1631					53rd Street
6600.0		1631	214	1363.6	X	X	u/s road
6900.0		1631	340	1366.8	X	X	
7200.0		1631	1702	1372.2			
7500.0		1631	2519	1375.3			
7800.0		1571	1067	1376.0	X	X	
8100.0	8	1400	661	1378.5	X		
8400.0		1400	1342	1382.3	X	X	
8700.0		1400	982	1384.1	X		
9000.0		1400	2011	1387.1			
9205.0		1400	338	1386.9	X		d/s road
9230.0		1400					Desert Jewel Dr.
9255.0		1400	170	1385.8	X	X	u/s road
9300.0		1400	1715	1391.5			
9448.0		1400	1049	1390.4	X		d/s road
9473.0		1400					Arroyo Drive
9498.0		1400	924	1391.5	X	X	u/s road
9600.0	9	1400	2053	1397.7		X	
9859.5		1400	310	1395.2	X		d/s road
9884.5		1400					Crestview Drive
9900.0		1400	445	1395.8	X	X	u/s road
10200.0		1400	272	1398.5	X	X	
10373.0		822	857	1402.2			d/s road
10398.0		822					Desert Park Lane
10423.0		822	25	1400.2	X	X	u/s road
10500.0		822	1603	1404.4		X	
10800.0		425	302	1404.3	X	X	
10999.0	10	425	232	1406.1	X		d/s road
11035.0		425					Road Runner Road

in Figure 3.2.

Discharge thresholds were used to determine the channel breakout points. Each discharge threshold was compared to the 100-year event discharge. In cases where the channel could not contain the 100-year discharge, an X was entered in the Breakout Points column of Table 3.3. Note that historically water has left the wash and flowed down 56th Street to the north. This loss of flow is included in the model hydrology (see Section 2.5).

Choke points are sections of the wash where limited channel capacity restricts conveyance and causes backwater upstream. Choke points were determined by comparing the discharge threshold in one section to the discharge threshold in the next section downstream. A section was reported as a choke point in Table 3.3 when the downstream discharge was considerably lower than the upstream discharge. Critical Depth warning messages in the HEC-2 model output, indicators of large conveyance change locations were also considered to be possible choke points and were reported in Table 3.3.

3.3.3 Areas of Possible Channel Improvements to Increase Capacity

All cross sections listed as either breakout points or choke points in Table 3.3 represent sections that could be modified to increase conveyance and overall channel flow capacity. Figures 3.2a-3.2c display the capacity of the cross sections versus the flows for the different recurrence intervals. Selected channel improvements are described in Section 3.4 below.

3.3.4 Channel Location of Least Conveyance

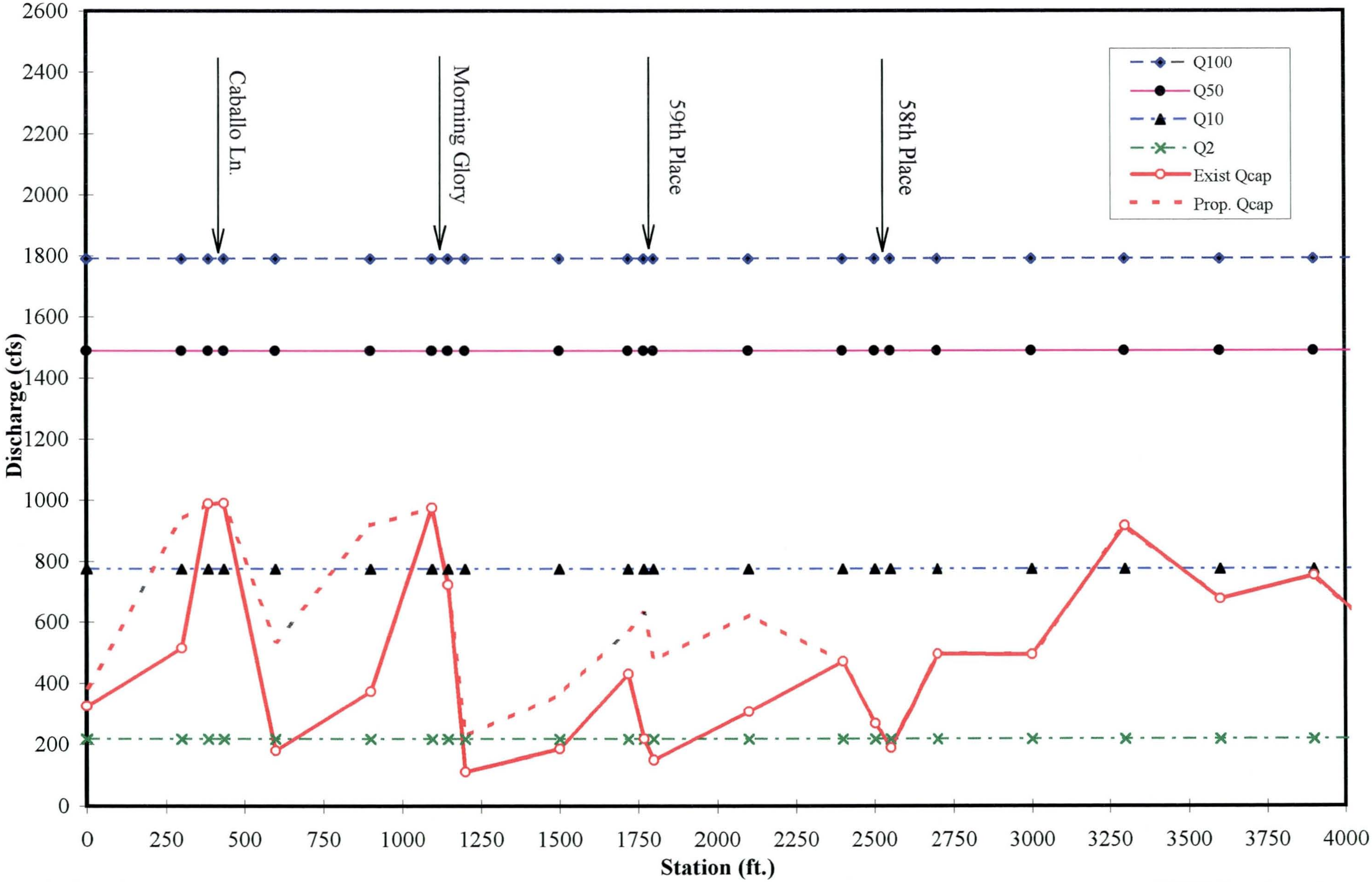
Not counting road sections (such as 4251.5 and 4293.2 at 56th Street and 5727.7 at Mockingbird Lane), the cross section with the least conveyance is at station 1200 (113 cfs). This section is about 50 feet upstream of Morning Glory Road. Cross sections 600, 1500, 1800, 2502.4, and 9255 also have low capacities, less than the 2-year flow event.

3.3.5 Flow Depths and Velocities at Road Crossings

The output from the HEC-2 model CHERORIG.DAT containing the 100-year discharge was analyzed to obtain the velocity and flow depth at 12 road crossings along the study reach. Results are summarized in Table 3.4. Ten of the crossings are paved dip crossings. The 56th Street crossing has very small culverts which were considered ineffective in the hydraulic analyses. Twin culverts under Road Runner Road were also considered ineffective in this analysis. At these crossings reported depths are distance of the water surface above the road (not above the culvert inlet). Flow depths and velocities exceed general safe pedestrian crossing standards (depth x velocity < 5) at all

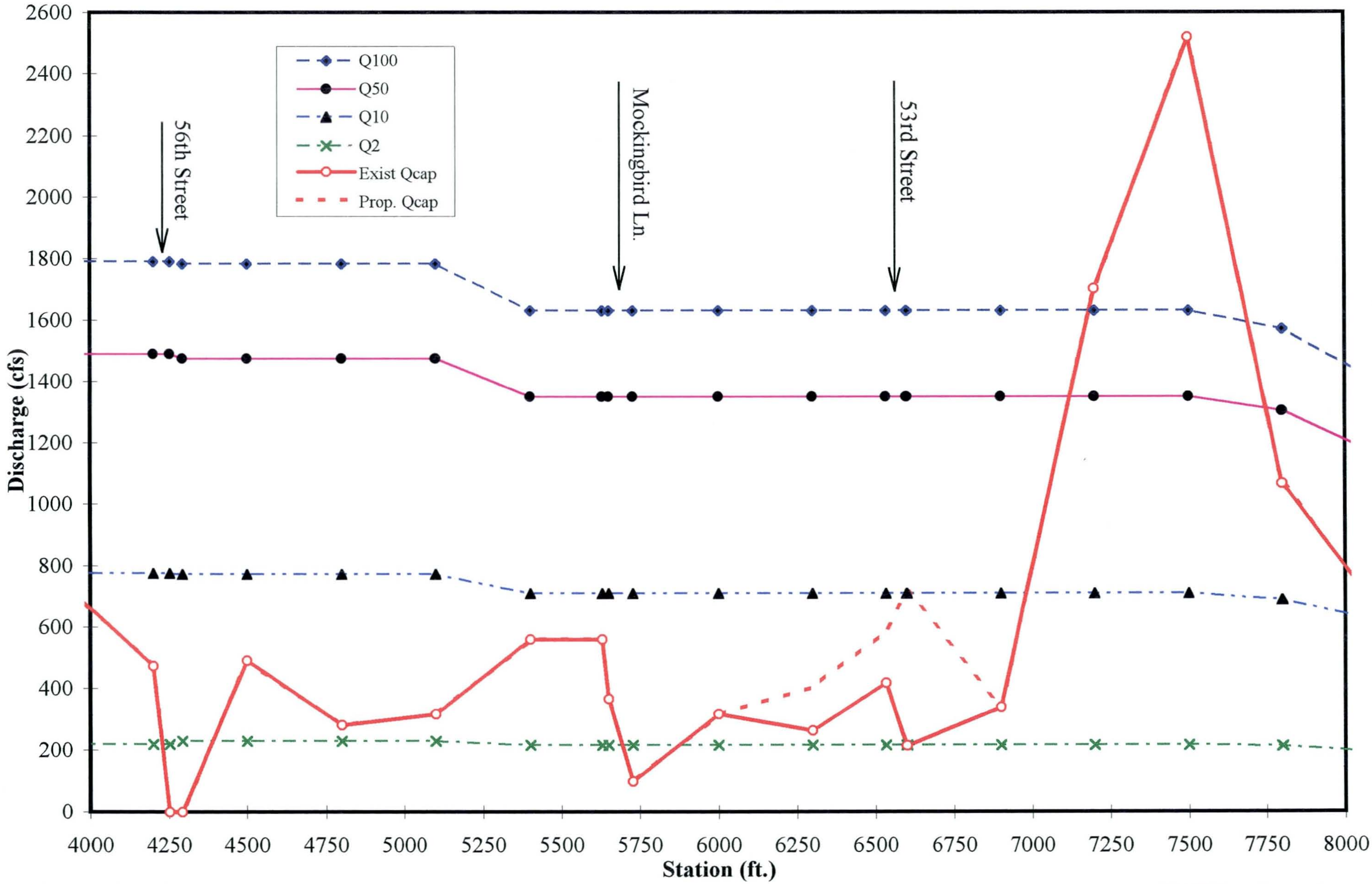
CHEROKEE WASH SEDIMENTATION STUDY

Figure 3.2a - Channel Capacity vs. Return Period



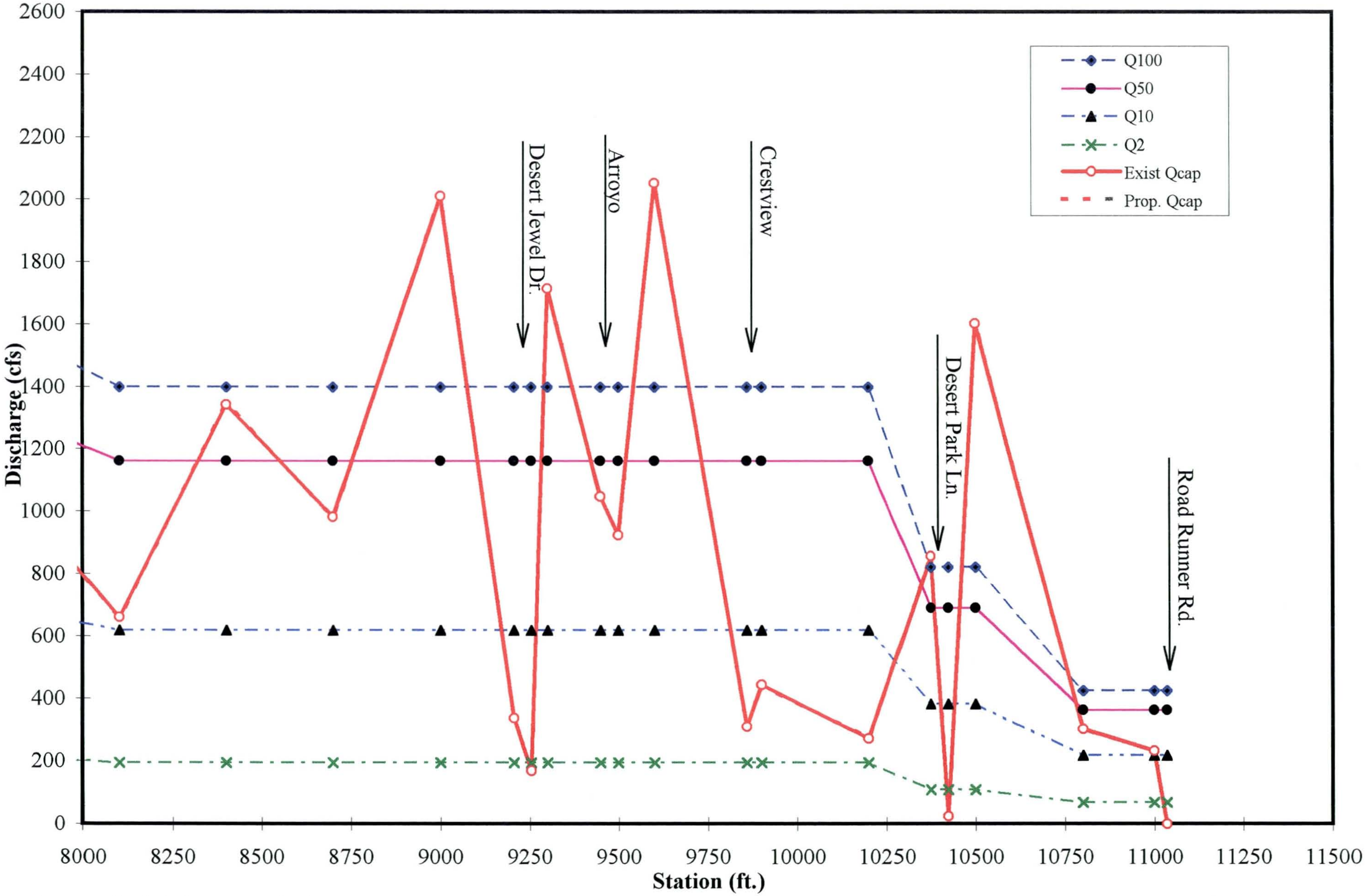
CHEROKEE WASH SEDIMENTATION STUDY

Figure 3.2b - Channel Capacity vs. Return Period



CHEROKEE WASH SEDIMENTATION STUDY

Figure 3.2c - Channel Capacity vs. Return Period



crossings except Road Runner Road for all events above the two year flow. All crossings except Caballo Lane are below this threshold for the 2-year event. Safe flow depth for vehicles was estimated as 0.5 feet per Maricopa County guidelines for new culvert construction at road crossings (FCDMC, 1996). This flow depth is equaled or exceeded at all crossings for all events with the exception of the 2-year flow at Road Runner Road. From a public safety standpoint, all of the crossings with the possible exception of Road Runner Road should be unsafe for flows equal to or greater than the two year event. Measures to improve the safety of the crossings could include automated barrier/gate systems or replacement of the dip crossings with culverts or a dip section with low flow culverts. If culverts are under consideration, studies should be performed to insure that residents upstream are not flooded more often due to backwater from the culverts.

Table 3.4 - Flow Depths (ft) and Velocities (ft/s) at Road Crossings by Event

Crossing	Cross Section	2-Year		10-Year		50-Year		100-Year	
		Depth	Vel.	Depth	Vel.	Depth	Vel.	Depth	Vel.
Caballo Lane	409.3	2.8	1.9	5.1	2.0	6.6	2.3	7.2	2.4
Morning Glory	1120.2	3.4	1.1	4.9	2.2	6.0	3.0	6.5	3.2
59th Place	1743.2	3.6	1.4	5.1	3.6	6.1	5.1	6.5	5.3
58th Place	2527.4	2.3	2.0	4.6	2.0	5.9	2.6	6.4	2.7
56th Street	4271.5	0.7	4.2	2.6	3.2	4.6	3.3	5.3	3.5
Mockingbird Ln.	5693	1.1	4.4	1.9	6.2	3.1	5.6	3.8	5.2
53rd Street	6558.8	2.7	1.5	4.6	1.7	5.8	2.1	6.2	2.3
Desert Jewel Dr.	9230	2.2	2.1	3.6	3.1	5.0	3.7	5.4	4.0
Arroyo Drive	9473	1.9	2.1	3.7	3.1	5.4	3.8	6.2	3.8
Crestview Drive	9884.5	2.3	1.5	4.4	1.7	5.7	2.2	6.2	2.3
Desert Park Ln.	10398	0.9	4.2	2.3	2.7	3.6	2.4	4.1	2.2
Road Runner Rd	11035	0.3	2.4	0.5	3.3	0.6	3.9	0.7	4.1

3.3.6 Channel Capacity Relative to Return Period

HEC-1 models were used to determine the 100-year, 50-year, 10-year and 2-year event flows. Flows for each event are plotted on Figures 3.2a-3.2c along with the discharge threshold flows from Table

3.3. All road crossings are also noted on these figures. Neglecting road crossings, most channel sections can convey the two year event but cannot pass the 10-year event. Note that exceedance of the discharge threshold for any given cross section does not mean that flood damages will necessarily occur, only that the water surface elevation is higher than the selected breakout elevation. To improve channel conveyance, several alternatives were considered as described in the next section.

3.4 Channel Improvement Options

Several reaches of Cherokee Wash will not be able to convey the 2-year event. Because of the lack of conveyance, several channel improvement options were considered:

1) Widen selected cross sections to a maximum top width of 40 feet with 2:1 (horizontal to vertical) side slopes.
2) Improve all sections of the channel to a 40 foot wide vertical side channel with gabion walls and a soft bottom.
3) Improve all sections of the channel to a 40 foot wide vertical side channel lined with concrete.
4) Perform in-channel vegetation maintenance.
5) Construct levees or flood walls along the wash.
6) Improve road crossings.
7) Construct detention basins.

Any option would be limited by several important factors. One of the most important to consider is the private ownership of most property through which the wash runs. This is closely followed by neighborhood reaction to improvements/construction on the wash. HEC-2 models were created for options one through four and are discussed in more detail below. Option 5 was not studied in detail for the following reasons:

- Many residents already have constructed walls along the wash to limit flooding
- Right of way is limited; a four foot high levee with 2:1 sideslopes needs a minimum base width of 10 feet
- Concrete flood walls are extremely expensive

A rough HEC-2 model was prepared for this option by adding encroachments to the base conditions model. Results showed levees would need to be 3 to 7 feet tall, plus any required freeboard. Most levees would be in the 2 to 4 foot range.

Option 6 was not considered in detail because improved road crossings, although resulting in greater public safety (see section 3.3.5), would actually decrease conveyance at the crossings. The exception to the last statement is the crossing at 56th Street (not a dip crossing like the others). Improving this crossing to at least match upstream and downstream conveyance would be beneficial. Although the present study assumed the small culverts to be ineffective, Kaminski-Hubbard Engineering (1995) estimated the capacity to be 64 cfs with water at the crown of the road. This is much less than our estimated channel capacity of about 500 cfs upstream and downstream.

Option 7 would reduce peak flows, but would have the effect of trapping sediment. This could cause further degradation of the channel, especially downstream of road crossings. For this reason we did not consider this option further.

3.4.1 Improved Channel HEC-2 Model Analysis

WEST created additional HEC-2 models for options 1,2, 3 and 4. In the option 1 model selected cross sections were widened to carry the 2-year event. The model for option 2 had a forty foot wide rectangular template cut into the original cross sections at the existing invert elevations The models for options 2 and 3 were identical except for roughness values assigned to the cross sections.. The option 4 model is identical to the base conditions model except that the maximum roughness value allowed in channel was 0.035. None of the channel invert elevations were modified for any of the models. Results of the analyses are as follows, comparing channel capacity at all sections not on/at road crossings:

- Option 1 (selective widening) would have the wash able to safely convey the 2-year event in all sections, the 10-year event in some sections, and flows up to the 100-year event in a few sections.

- Option 2 (40 foot rectangular channel with gabions and soft bottom) would contain the 2-year event in all sections, the 10-year event in all but 5 sections and the 100-year event in 21 of the 37 cross sections considered, most in the upstream end of the wash.

- Option 3 (40 foot rectangular channel, concrete lined) would contain the 2-year event in all sections, the 10-year event in all but 1 section and the 100-year event in 23 of the 37 cross sections considered, most in the upstream end of the wash. The model output indicates that the flow would be supercritical in some reaches for this option.

- Option 4 (vegetation maintenance) would improve conveyance at only a few sections in the upstream part of the wash. Cherokee wash would still not be able to pass the 2-year event or even the 10-year event at many cross sections.

Options 1 and 4 would require a small amount of work to implement compared to options 2 and 3 in which the entire wash would be modified. Construction costs for options 2 and 3 are expected to be high, especially considering the rock outcrops present in the upper reaches of the wash. However, even with the high costs of construction, these options would still not provide conveyance for the

100-year flood (unless a supercritical channel were designed). Based on these factors, we selected option 1 as the most feasible alternative for providing protection against the more frequent floods. This option will be called the "with-project" plan hereafter and is described in more detail in the following section.

3.4.2 With-Project HEC-2 Model Analysis

WEST created an HEC-2 model in which the existing condition channel geometry was widened in several locations along the Wash such that the 2-year flow would be conveyed without exceeding the threshold elevations (the "with-project" or "proposed condition" plan). Sections where the 2-year event would not pass (resulting water surface elevations were greater than the minimum bank station elevation) were modified by cutting a trapezoidal channel with 2:1 (horizontal to vertical) bank slopes, limiting the top width to the lesser of 40 feet or the existing top width (top width defined as the distance between defined bank stations). The sideslope and top width limitations for modified channel sections were agreed upon, after consultations with FCD, as reasonable limiting values. The channel invert elevations were not altered. Bank station locations were modified where necessary after consulting the photos, survey notes and site visit notes. A comment record was added before each X1 card describing the section modifications. The resulting HEC-2 model was named CHERMOD1.DAT. Cross sections whose geometry was modified are shown in Table 3.5. These cross sections correspond to reaches between Indian Bend Wash and 58th Place and just upstream and downstream of 53rd Place.

Note in Table 3.5 that in some cases the channel top width did not change, although conveyance was added to the cross section by increasing the channel bottom width and setting the side slopes to 2 horizontal to 1 vertical. The conveyance of each of the modified sections, based on normal depth calculations, is shown in Figures 3.2a through 3.2c as dashed lines. Table 3.6 compares the depths and velocities at road crossings for the 100-year flows for existing conditions and with-project conditions. Other comparisons between existing and with-project conditions are shown in the Geomorphic Analysis section of this report.

3.5 Summary

A base conditions hydraulic model of Cherokee Wash was prepared using recently surveyed cross sections and discharge values obtained from the hydrologic analysis. This model and hydraulic analyses using other methods revealed that the 2-year flood event could not be conveyed safely without the water surface exceeding selected break-out elevations. In addition, flow depths and velocities at the road crossings would be unsafe for most flood flow conditions. Several channel improvement options were considered. The option judged most feasible was selected widening of the channel to convey the 2-year event at all sections. However, the dip crossings would still be unsafe for most events. We also recommend that the culverts under 56th Street be improved to match upstream and downstream channel capacity (the existing small culverts were deemed ineffective in this study).

Table 3.5 - Channel Geometry Modifications for With-Project Conditions

Cross Section Number	Existing Top Width (ft)	Modified Top Width (ft)	Difference (ft)
0	84	84	0
300	39	39	0
600	31	39.2	8.2
900	59	59	0
1200	18.5	40	21.5
1500	21	40	19
1718.2	27	40	13
1788.2	23	40	17
1800	19	40	21
2100	27	40	13
6300	36.5	40	3.5
6533.8	38	40	2
6600	22	40	18

Table 3.6 - Comparison of Depths and Velocities at Road Crossings (100-year Event)

Crossing	Section	Existing Conditions		With-Project Conditions		Difference	
		Depth	Vel.	Depth	Vel.	Depth	Vel.
Caballo Lane	409.3	7.2	2.4	6.6	2.8	-0.6	0.4
Morning Glory Road	1120.2	6.5	3.2	5.9	3.4	-0.6	0.2
59th Place	1743.2	6.5	5.3	5.8	4.7	-0.7	-0.6
58th Place	2527.4	6.4	2.7	6.0	3.0	-0.4	0.3
56th Street	4271.5	5.3	3.5	5.3	3.5	0.0	0.0
Mocking-bird Lane	5693	3.8	5.2	3.8	5.2	0.0	0.0
53rd Street	6558.8	6.2	2.3	5.8	2.6	-0.4	0.3
Desert Jewel Dr.	9230	5.4	4.0	6.0	3.4	0.6	-0.6
Arroyo Drive	9473	6.2	3.8	6.3	4.4	0.1	0.6
Crestview Drive	9884.5	6.2	2.3	6.2	2.3	0.0	0.0
Desert Park Lane	10398	4.1	2.2	4.1	2.2	0.0	0.0
Road Runner Road	11035	0.7	4.1	0.7	4.1	0.0	0.0

4.0 GEOMORPHIC ANALYSIS

4.1 Introduction

This section summarizes the results of a geomorphic analysis of Cherokee Wash for the Flood Control District of Maricopa County (FCD). Included are a general description of the watershed and stream channel, an analysis of available historical data, and an evaluation of the relative impact of the proposed flood control project on the morphology of the watercourse. The purpose of this analysis is to characterize the stability of the Wash and the expected response to proposed flood control modifications.

4.2 Watershed Description

The Cherokee Wash watershed encompasses about 2 square miles in the Town of Paradise Valley, Maricopa County, Arizona. The watershed is bounded by the Phoenix Mountains Preserve on the west and Mummy Mountain to the south. The Wash is an ephemeral watercourse. It drains toward the northeast and is tributary to Indian Bend Wash. Elevations in the watershed range from about 1,320 feet mean sea level (ft msl) at the mouth of the Wash to over 2,000 ft msl along the watershed boundary. A topographic map of the basin is shown in Figure 1.2 in Chapter 1 of this report.

4.2.1 Historic Conditions

A single historic aerial photograph of the Cherokee Wash watershed provided by FCD, dated 1957, was available for review (Figure 4.1). No previous studies were available from the U.S. Natural Resources Conservation Agency (former Soil Conservation Service). As seen in the photograph, watershed development at the time was very limited. Mockingbird Lane is the only road crossing of the watercourse. At this time only a few single family residences are located in the watershed.

Generally, the location of Cherokee Wash in 1957 is similar to its current position. Two major tributaries enter the main wash from the west. The northernmost tributary in the photo (called the "west branch") also corresponds well to its present position. The other, southernmost, tributary appears to have been replaced by a series of culverts and ditches along Desert Park Lane. Watershed vegetation in the photo is limited to drainage ways. In the vicinity of Indian Bend Wash, the Cherokee Wash channel becomes less distinct, fanning out over the confluence area.

4.2.2 Existing Conditions

The existing Cherokee Wash watershed has been extensively developed for single family residential housing (Figure 4.2). Residential developments encroach upon the stream in many locations. Golf

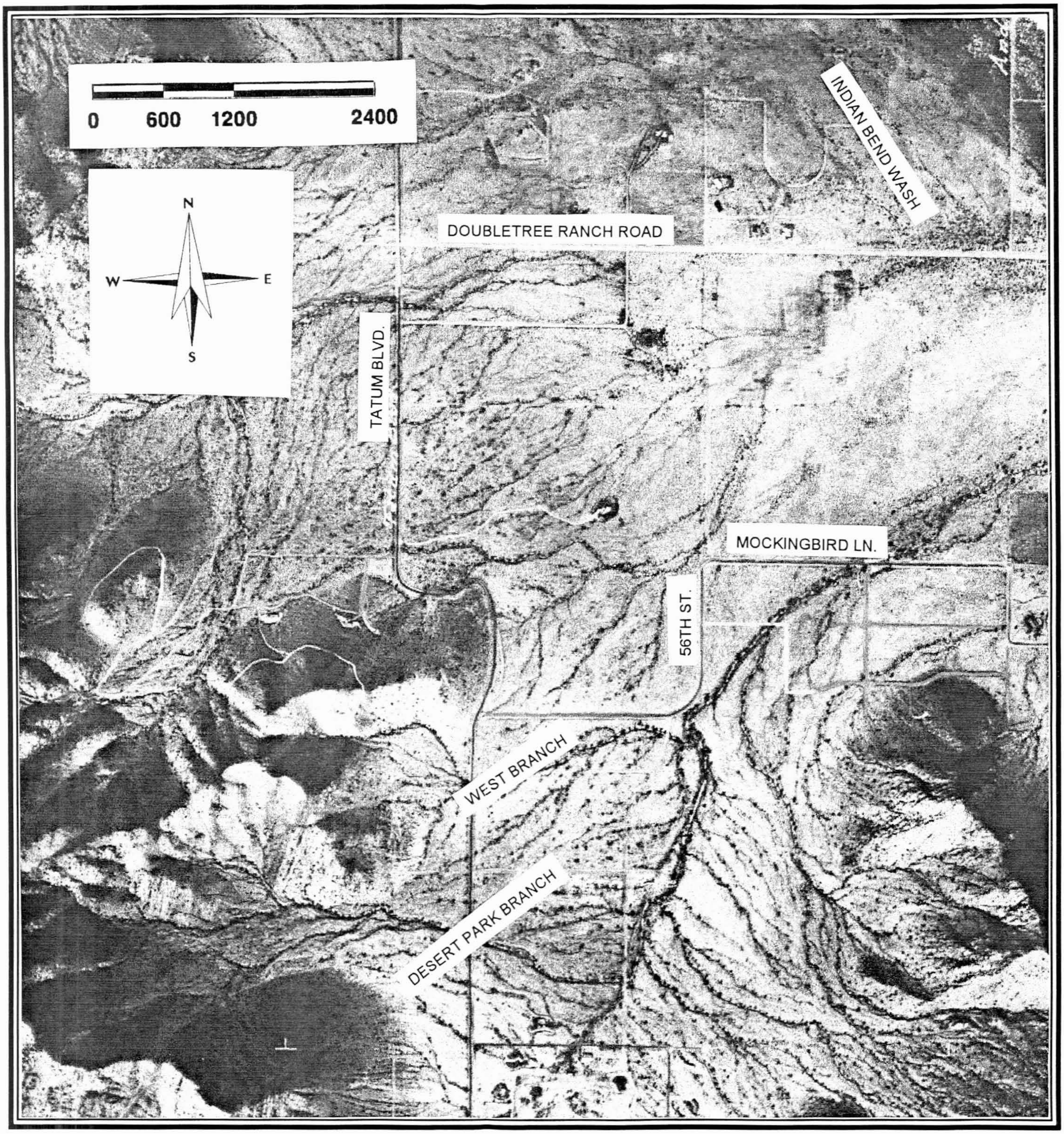


Figure 4.1. Cherokee Wash Study Area, 1957

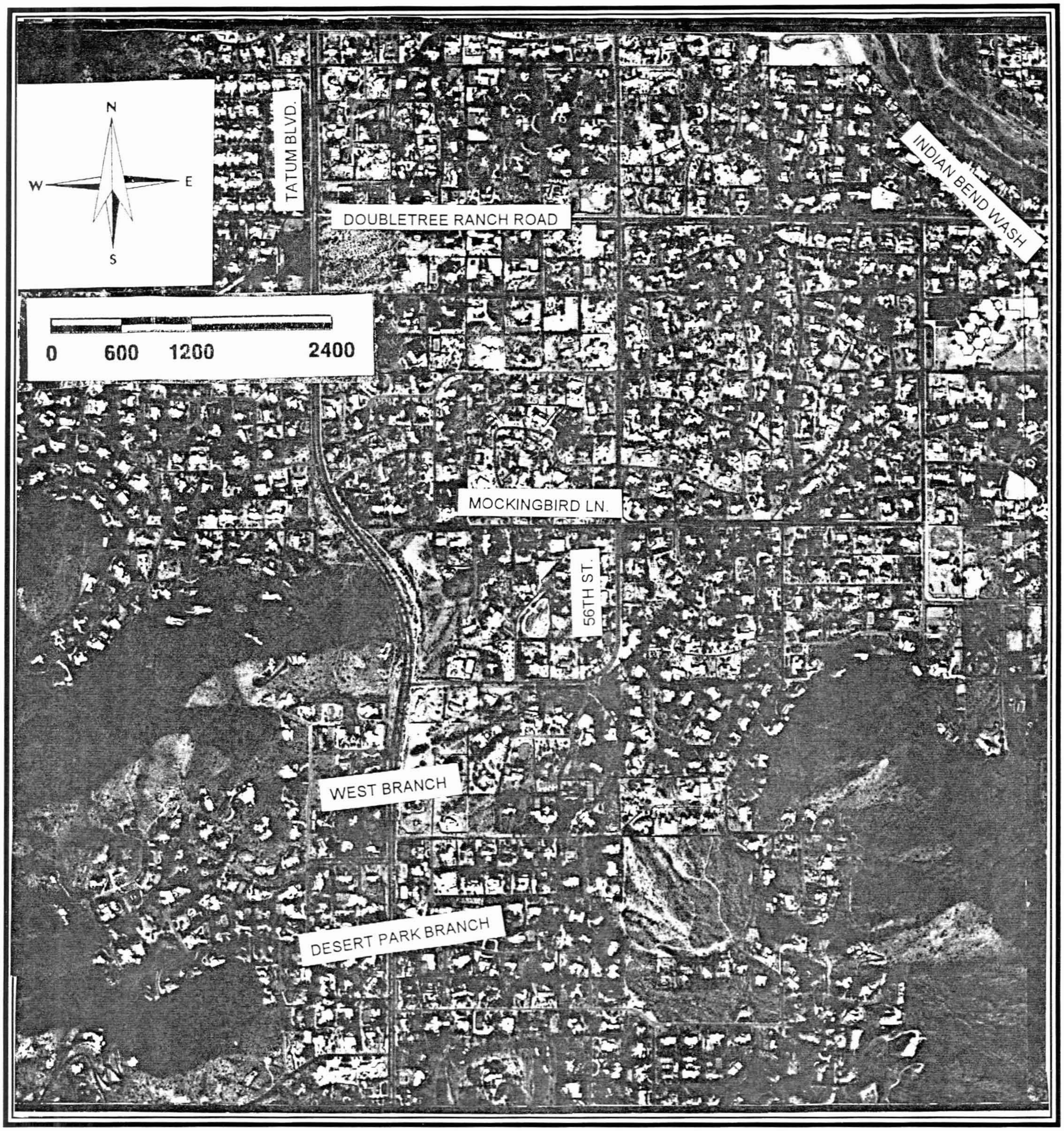


Figure 4.2. Cherokee Wash Study Area, 1997

courses are located both at the confluence with Indian Bend Wash and in the upper reaches of the wash. Increased impervious area and decreased time of concentration within the watershed caused by development has likely increased runoff volumes and peak discharges and decreased sediment yields compared to historic conditions.

The vegetation varies significantly with location along the channel, from non-existent to thick stands of trees and bushes. The trees, shrubs, and grasses along the channel present variable hydraulic roughness conditions. Clearing of channel vegetation and bulldozing of the channel was observed during one of the field visits; we could not determine which agency was responsible for these activities.

No significant bank erosion was observed along Cherokee Wash during the field reconnaissance activities with one exception. Some erosion was occurring at a tight bend about 500 feet upstream of 58th Street. However, banks were seen to be bare of vegetation in many locations with little or no erosion. Bank materials were observed to be comprised primarily of erodible sands and gravels. Localized bank erosion was noted at several locations where runoff from surrounding development overflowed unreveted bank materials.

The existing Cherokee Wash channel is crossed by roads at twelve locations within the study area. The road crossings include both culverts and dips. The road crossings may act as grade controls for any degradation that occurs along the stream. Observations of degradation were made at several locations along the stream during field reconnaissance. Localized erosion of the channel was seen upstream of Morning Glory Road. At the downstream side of Mockingbird Lane, a grouted riprap sill was found to be undermined. A head cut was also seen along Cherokee Wash between 58th Place and 56th Street. Subsequent channel maintenance, noted above, removed many of the degradational features notes during the initial field inspection.

4.2.3 With-Project Conditions

Several reaches of the channel would be modified under With-Project Conditions, described in Section 3.4. Channel widths for the existing and proposed conditions are shown in Table 3.5. Differences in other hydraulic parameters between existing conditions and modified channel conditions are described below.

4.3. Qualitative Geomorphic Assessment

In the following sections a qualitative geomorphic assessment is presented for Cherokee Wash. The evaluation includes an assessment of the geometry of the watercourse, examination of channel hydraulics for the dominant discharge, and characterization of channel bed materials. Both the existing channel geometry and proposed flood control geometry conditions for the Wash were examined as part of the hydraulic analysis. Comparisons of the two analysis conditions were made to assess the potential impact of the proposed flood control measures (localized channel widening) on channel stability.

4.3.1 Channel Reaches

To define sections of Cherokee Wash with similar hydraulic and sediment transport characteristics a series of reaches were defined. Note that the reaches defined here for the geomorphic analysis do **not** correspond to those previously defined for the HEC-2 analysis. The limits of these reaches were chosen to correspond with the location of road crossings along the Wash. These were chosen due to the significant number along the Wash and since they will probably act as grade controls for any channel degradation that is experienced. A schematic diagram of Cherokee Wash which delineates reach limits is shown in Figure 4.3.

4.3.2 Channel Profile

The existing profile of the Cherokee Wash thalweg was shown in Figure 3.1. As seen in the figure, the channel slope increases significantly in an upstream direction. Only minor breaks in the channel profile are noted along its length. The most significant breaks in slope are located at road crossings, including Morning Glory Road, Mockingbird Lane, and Arroyo Drive. A summary of slopes for the various analysis reaches along the wash is presented in Table 4.1. Slopes range from less than about 0.0029 (15 feet/mile) near Indian Bend Wash up to 0.0141 (74 feet/mile) upstream of the West Branch of the Wash.

Table 4.1 - Summary of existing channel slopes

REACH	U/S STA (feet)	U/S ELEV (feet)	D/S STA (feet)	D/S ELEV (feet)	SLOPE (ft/ft)	SLOPE (ft/mile)
1	409.3	1322.5	0	1319.3	0.00782	41
2	1095.2	1324.5	409.3	1322.5	0.00292	15
3	1718.2	1328.8	1095.2	1324.5	0.0069	36
4	2502.4	1332.4	1718.2	1328.8	0.00459	24
5	4251.5	1346.3	2502.4	1332.4	0.00795	42
6	5650	1356.5	4251.5	1346.3	0.00729	39
7	6533.8	1361.2	5650	1356.5	0.00532	28
8	7200	1366	6533.8	1361.2	0.00721	38
9	9205	1383.9	7200	1366	0.00893	47
10	9448	1386.4	9205	1383.9	0.01029	54
11	9859.5	1392.2	9448	1386.4	0.01409	74
12	10373	1398.8	9859.5	1392.2	0.01285	68
13	11035	1407.2	10373	1398.8	0.01269	67

REACH #	STATION
	0 + 00 - CONFLUENCE WITH INDIAN BEND WASH
1	
	4 + 09.3 - CABALLO LANE/61ST STREET
2	
	10 + 95.2 - MORNING GLORY ROAD
3	
	17 + 18.2 - 59TH PLACE
4	
	25 + 02.4 - 58TH PLACE
5	
	42 + 51.5 - 56TH STREET
6	
	56 + 50 - MOCKINGBIRD LANE
7	
	65 + 33.8 - 53RD STREET
8	
	72 + 00 - 52ND STREET
	80 + 00 - WEST BRANCH
9	
	92 + 05 - DESERT JEWEL DRIVE
10	
	94 + 48 - ARROYO DRIVE
11	
	98 + 59.5 - CRESTVIEW DRIVE
12	
	103 + 73 - DESERT PARK LANE
13	
	110 + 35 - ROAD RUNNER ROAD

Figure 4.3 - Cherokee Wash Schematic

4.3.3 Hydraulic Analysis

A hydraulic analysis of Cherokee Wash was performed utilizing the U.S. Army Corps of Engineers HEC-2 water surface profile program. Details of the hydraulic analysis are documented in Section 3 of this report. The purpose of the analysis was to define the hydraulic conditions along Cherokee Wash for the 2-, 10-, 50-, and 100-year flood under both existing and proposed channel geometry conditions.

For the purposes of this qualitative geomorphic assessment, the hydraulic conditions associated with the 10-year flood were evaluated. The 10-year flood was evaluated because it is assumed to be the dominant discharge, the discharge which has the most significant effect in forming the channel. A 10-year flood was assumed as the dominant discharge since it is large enough to affect and form the entire channel geometry and frequent enough to have the greatest effect on sediment transport. The 10-year discharge is most often used for the dominant discharge in arid regions (e.g. RCE, 1994; SLA, 1985; recent experimental work by Miller et al., 1997)

To evaluate the differences between existing and proposed geometry conditions, plots were developed for selected hydraulic parameters. Plots of reach-averaged values for main channel width/depth ratio, top width, and velocity for both existing and proposed channel geometry conditions are shown in Figures 4.4 through 4.6. Comparison of the existing and proposed values on the plots reveals only small differences in channel hydraulics and geometry between the alternative conditions.

Sediment transport rates are highly dependent on main channel velocity. Channel velocities for Cherokee Wash were found to display the same general trend and magnitude for both existing and proposed conditions. Reach-averaged channel velocities were found to range from 1 to 4 feet per second. The highest velocities are seen to occur in Reaches 10 through 12 in the upper reaches of the Wash and the lowest channel velocities in Reaches 6 through 8 in the middle section of the Wash.

In general, the expected channel velocities along Cherokee Wash are mild and are not expected to be highly erosive for a 10-year flood condition. The variation in velocity along the channel suggests that the greatest erosion potential exists in the steep reaches located in the headwaters of the stream. Due to reductions in channel velocity, aggradation may be expected in Reaches 6 through 8 in the middle portion of the Wash and along Reaches 2 and 3 near the mouth of the Wash. It is noted that hydraulic conditions at the mouth of the Wash may be affected by flow along Indian Bend Wash and the occurrence of aggradation or degradation along the channel may be affected by the numerous road crossings along the watercourse.

Comparison of plots of top width and width/depth ratio for existing and proposed geometry conditions show that the average top width is in the range of 30 to 60 feet. The narrowest channel sections are observed in Reach 8 under both analysis conditions. The width/depth ratio is seen to increase slightly for the proposed geometry condition due to widening of the channel for greater flood

FIGURE 4.4 - AVERAGE WIDTH/DEPTH RATIO BY REACH 10 YEAR FLOOD
EXISTING AND WITH-PROJECT CONDITIONS

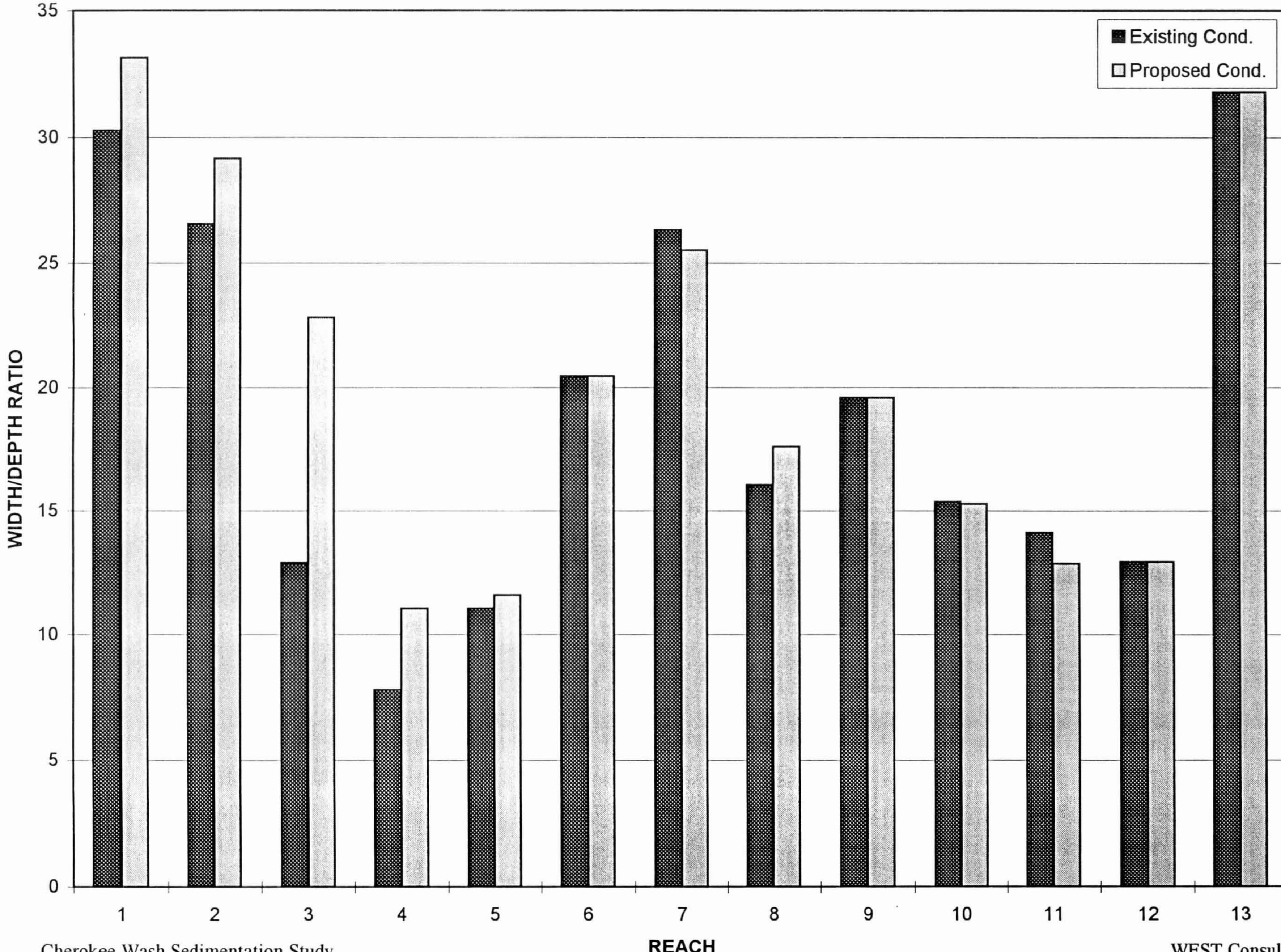


FIGURE 4.5 - AVERAGE TOP WIDTH BY REACH FOR 10 YEAR FLOOD
EXISTING AND WITH-PROJECT CONDITIONS

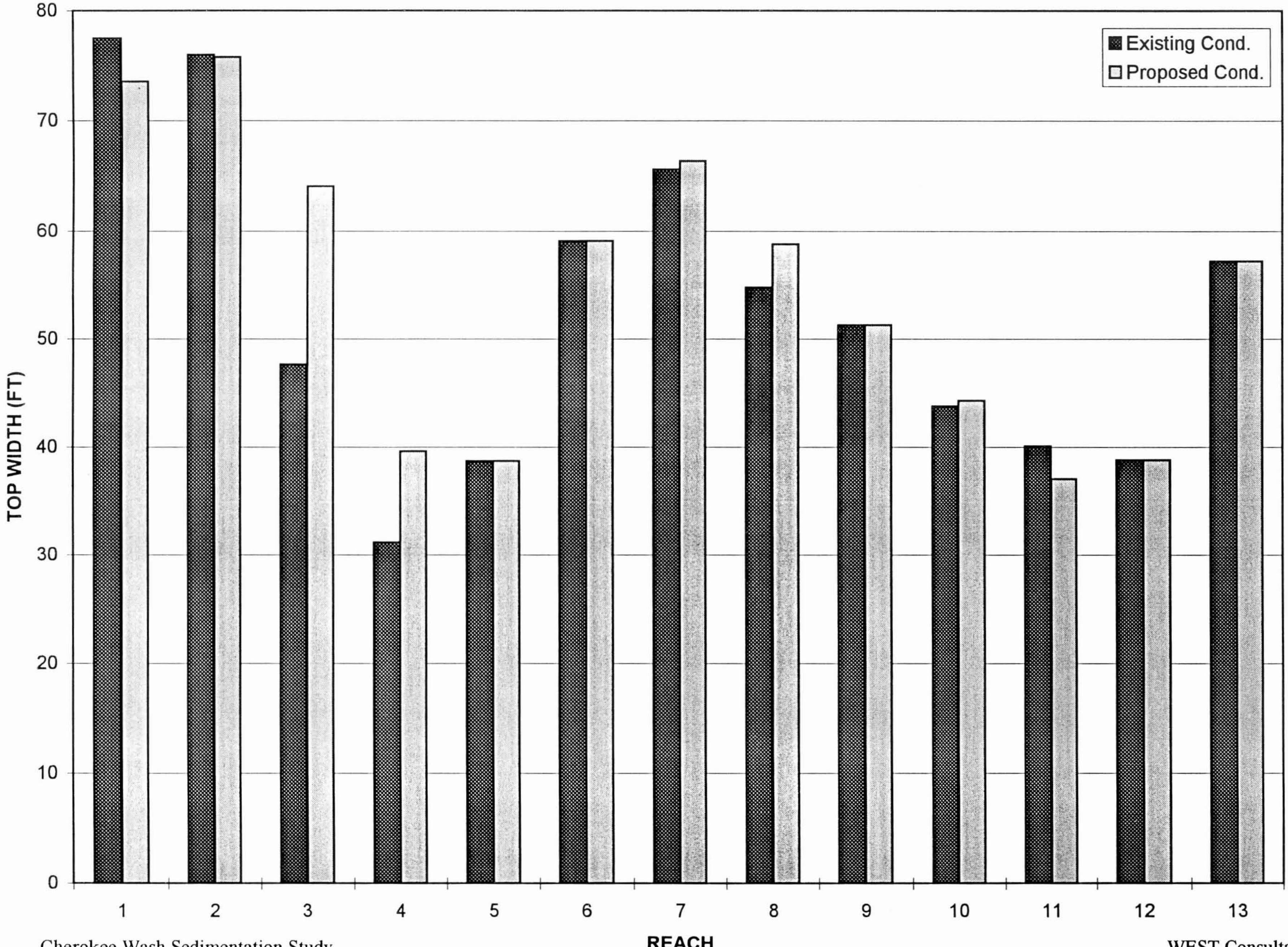
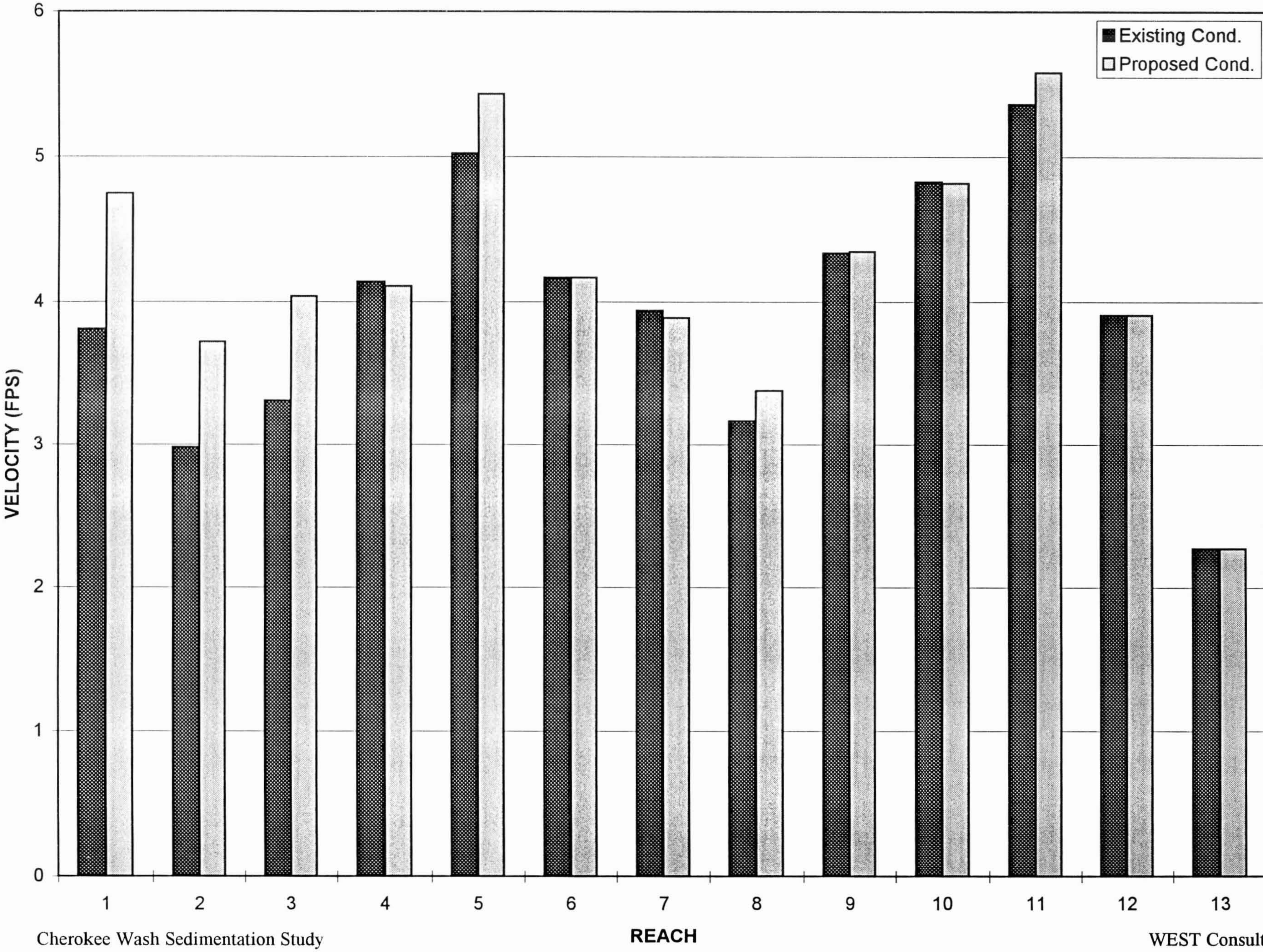


FIGURE 4.6 - AVERAGE VELOCITY BY REACH FOR 10 YEAR FLOOD
EXISTING AND WITH-PROJECT CONDITIONS



capacity. Overall, width changes are slight and do not significantly alter velocities along the Wash.

4.3.4 Bed Material

A comprehensive program of sampling bed materials along Cherokee Wash was conducted (Law/Crandall, 1996) according to a plan developed by WEST Consultants. The sampling program included both surface and subsurface channel bed material samples taken at locations throughout the extent of the Cherokee Wash. Sampling locations are shown in Figure 4.7.

Results of the sediment sampling are summarized in Figures 4.8 and 4.9. Figure 4.8 shows the sediment sizes for which 84, 50, and 16 percent of the sample is finer by weight for surface bed materials. Figure 4.9 shows similar information for subsurface bed materials. As seen from the figures, the median size of channel sediments changes significantly with location. From the confluence up to 58th Street, the median sediment size is silt-sized or finer for subsurface materials. For the surface bed material it is seen to be silt-sized or finer up to Morning Glory Road. Upstream, the median size for both surface and subsurface bed material samples was found to be significantly coarser, varying from sand- to gravel-sized material. Upstream of Mockingbird Lane, no sediment samples could be obtained for depths greater than 12 inches due to the presence of very large rocks and/or a conglomerated or cemented layer. Rock outcrops were noted in the area between Crestview Drive and Desert Park Lane. Armoring of the channel bed was observed at several locations, including at sample sites 9, 10, and 12.

A slight dip in the average sediment size for surface bed material can be seen in Figure 4.8 in the vicinity of Reaches 6 and 7. This slight reduction in average bed material size is attributed to the corresponding dip in average reach velocities previously noted on Figure 4.6.

4.4 Equilibrium Slope Analysis

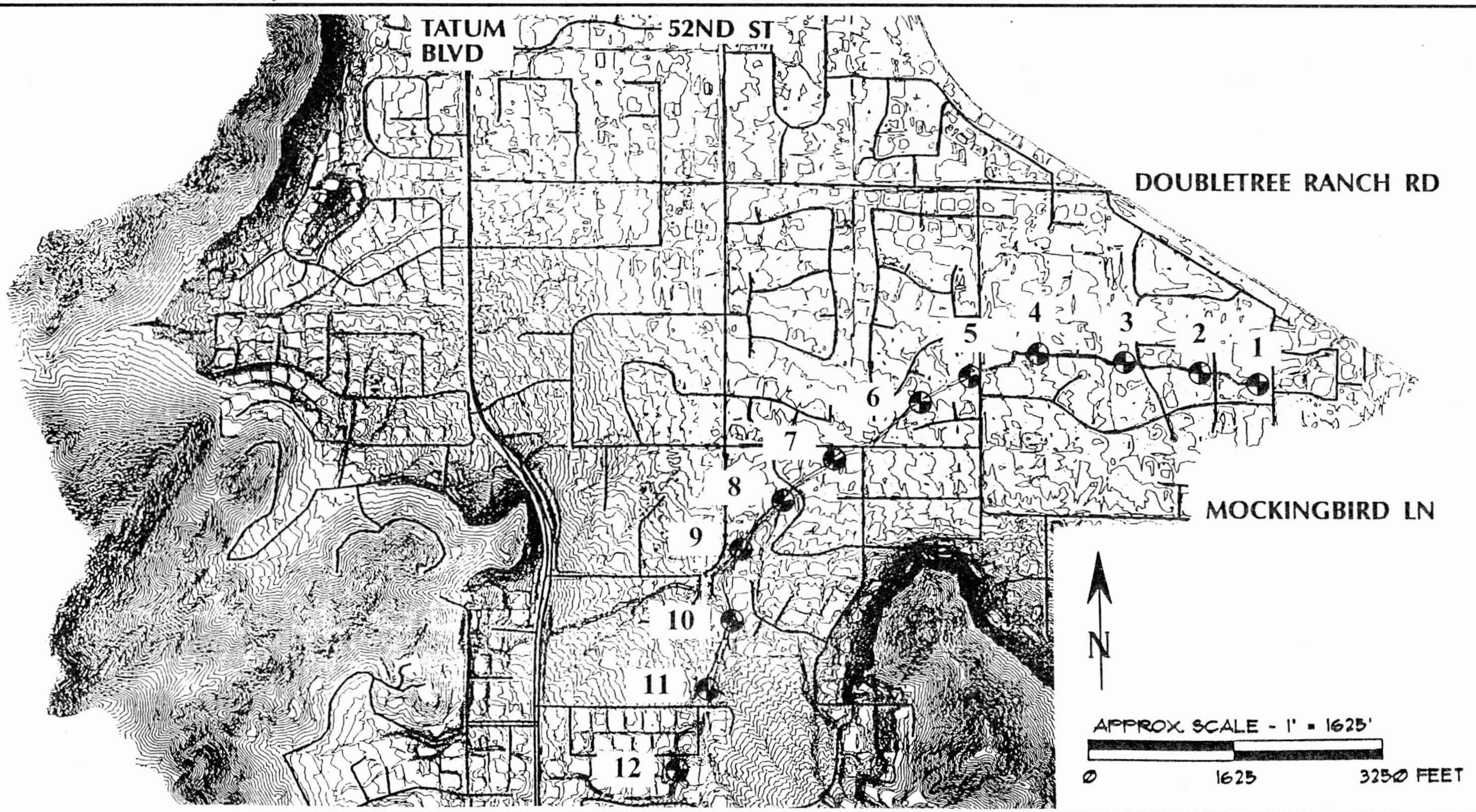
The slope at which Cherokee Wash would be expected to be stable was calculated for both existing and proposed geometry conditions. This was accomplished by application of the following four methods and dominant discharge hydraulic conditions (USBR, 1984):

- Schoklitsch Method
- Meyer-Peter, Muller Method
- Shields Diagram Method
- Lane's Tractive Force Method

It was assumed that the dominant discharge for Cherokee Wash is equal to the 10-year return period flood.

Figure 4.7 - Sediment Sampling Locations (after Law/Crandall, 1996)

CHECKED: P.M. ______ , PRINCIPAL MDD



REFERENCE		SAMPLING LOCATION PLAN		FIGURE 1	LAW/CRANDALL
ENTITLED:	DOUBLETREE RANCH DRAINAGE IMPROVEMENT PROJ.	LAW PROJECT NAME:	CHEROKEE WASH		A DIVISION OF LAW ENGINEERING AND ENVIRONMENTAL SERVICES INC.
		LAW PROJECT NO:	70241-50354.02		
PREPARED BY:	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY	DRAWN BY:	RAA		
		DATE:	8/96		

Figure 4.8 - Surface Bed Material Characteristics - 0" to 12" Depth

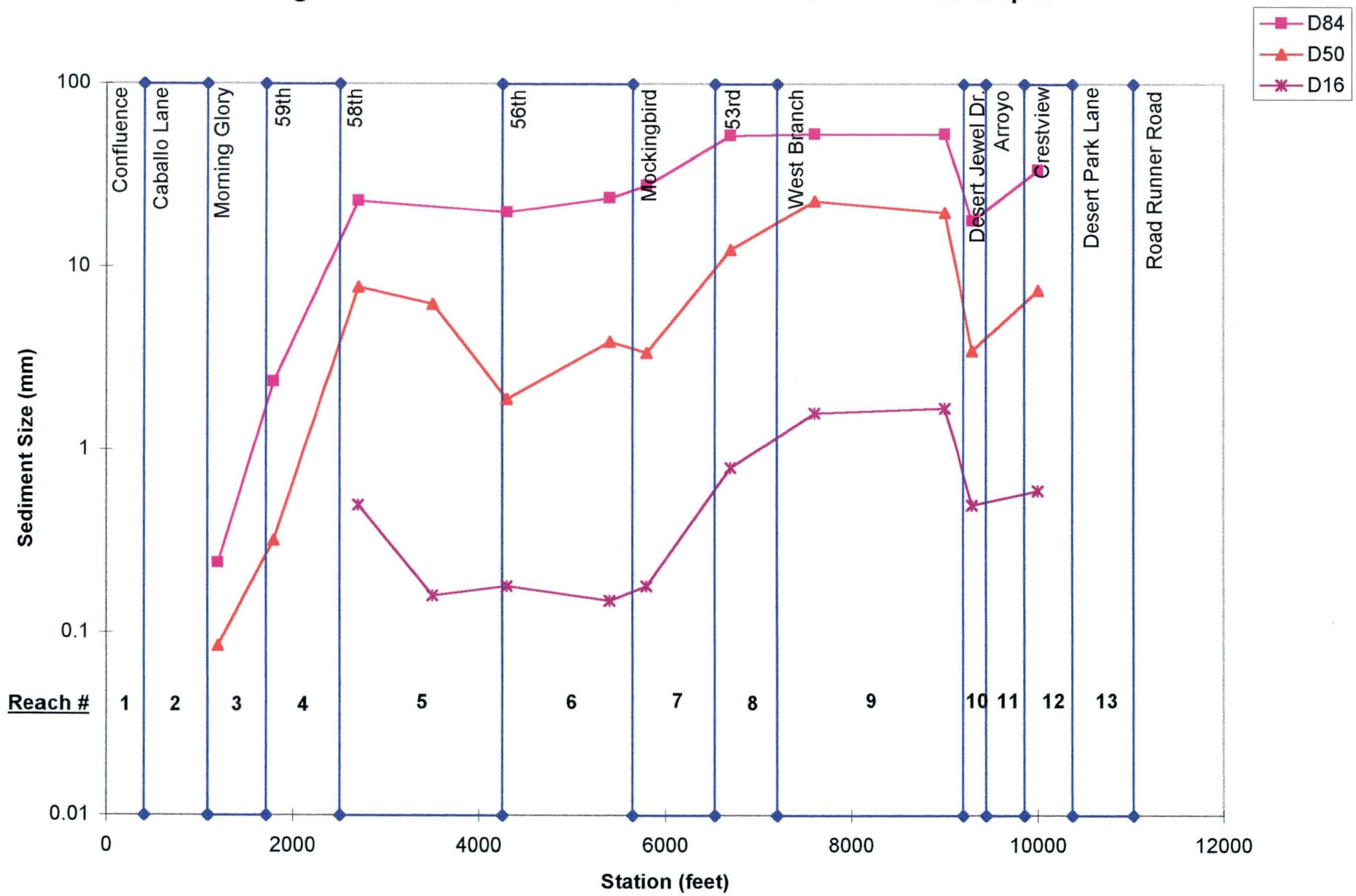
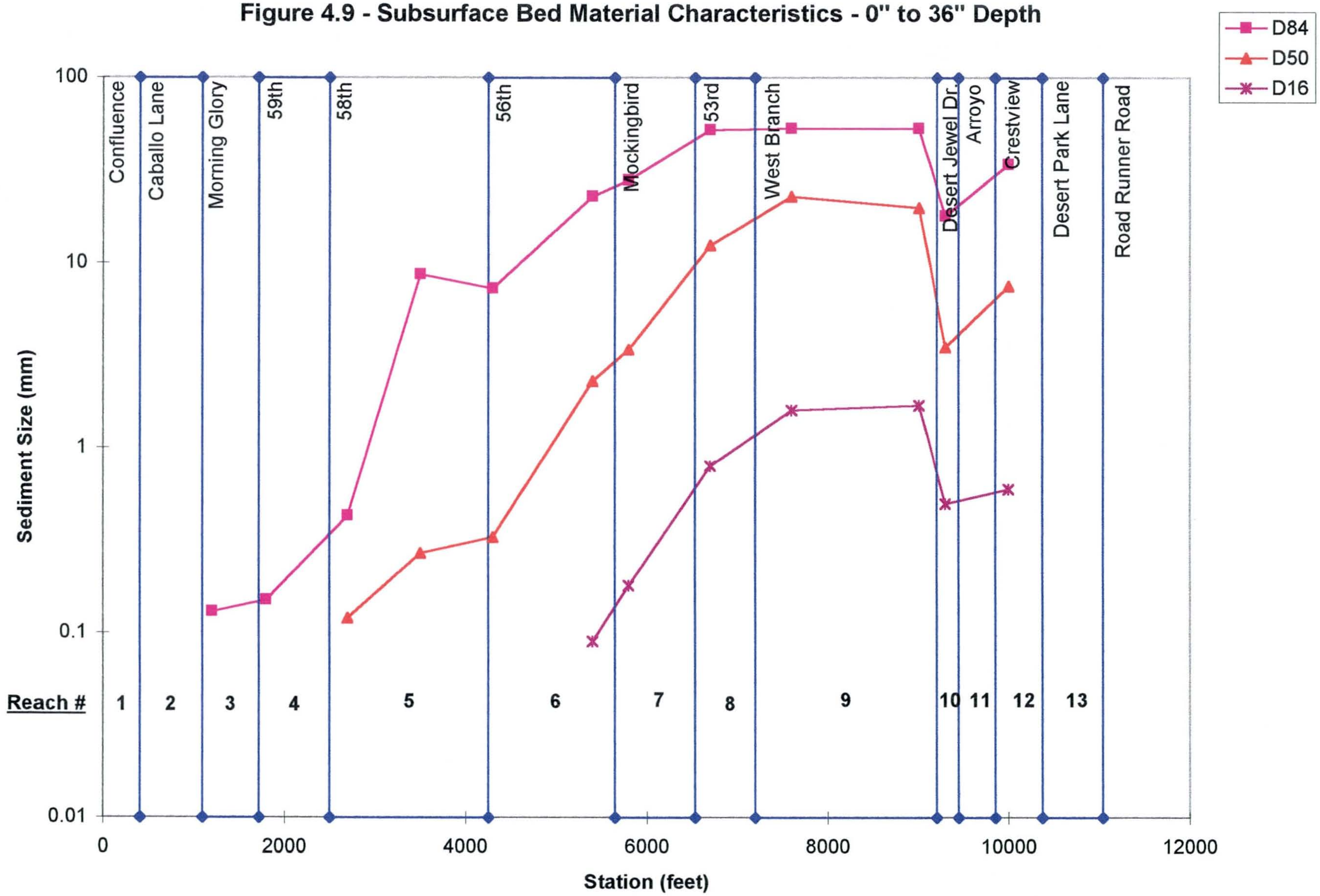


Figure 4.9 - Subsurface Bed Material Characteristics - 0" to 36" Depth



The stable slope was determined for each of the 13 reaches identified along Cherokee Wash. The reach limits generally correspond to the location of road crossings along the watercourse. If degradation along Cherokee Wash were to occur, it would to be limited to the reaches defined by the grade control provided by the various dip crossings and culverts along the Wash. Stable slopes would be expected to develop upstream of the control at each of the involved crossings.

Results of the equilibrium slope analysis for existing hydraulic conditions are summarized in Table 4.2. Results of the stable slope analysis for proposed conditions are shown in Table 4.3.

Table 4.2 - Summary of stable slope estimates for existing conditions

REACH	SCHOKLITSCH	MPM	SHIELDS	LANE	AVERAGE
1	0.00011	8.30E-05	1.20E-05	0.00012	8.10E-05
2	0.00012	7.60E-05	9.80E-06	0.0001	7.70E-05
3	0.0001	1.30E-04	7.60E-06	7.80E-05	7.90E-05
4	0.00007	6.30E-05	7.10E-06	7.30E-05	5.40E-05
5	0.00308	2.60E-03	1.90E-03	0.0006	0.00204
6	0.00261	1.50E-03	6.50E-04	0.00049	0.00132
7	0.01065	1.00E-02	3.90E-03	0.00125	0.0065
8	0.00945	6.70E-03	5.20E-03	0.0014	0.00568
9	0.00209	4.20E-03	1.90E-03	0.00053	0.00218
10	0.00087	9.90E-04	4.80E-04	0.00024	0.00065
11	0.00091	1.10E-03	5.50E-04	0.00028	0.00072
12	0.00148	1.90E-03	1.10E-03	0.00033	0.0012
13	0.00228	2.50E-03	1.50E-03	0.00046	0.00168

Table 4.3 - Summary of stable slope estimates for proposed conditions

REACH	SCHOKLITSCH	MPM	SHIELDS	LANE	AVERAGE
1	0.000105	8.32E-05	1.23E-05	0.000123	8.1E-05
2	0.000116	7.27E-05	9.7E-06	9.71E-05	7.38E-05
3	0.000096	0.000189	1.1E-05	0.000111	0.000102
4	0.000099	6.75E-05	7.8E-06	7.81E-05	6.3E-05
5	0.003202	0.002555	0.001913	0.000597	0.002067
6	0.002609	0.0015	0.000642	0.000485	0.001309
7	0.010663	0.010206	0.003906	0.001254	0.006507
8	0.009445	0.006694	0.005187	0.001399	0.005681
9	0.002235	0.004462	0.002005	0.000563	0.002316
10	0.000864	0.001093	0.000524	0.000267	0.000687
11	0.001007	0.001163	0.000558	0.000284	0.000753
12	0.001818	0.002544	0.001395	0.000435	0.001548
13	0.002835	0.002841	0.001686	0.000526	0.001972

As seen from comparison of Tables 4.2 and 4.3, the calculated stable slopes for both existing and proposed conditions are similar. Differences are attributable to slight changes in the hydraulic characteristics of the two analysis conditions. Note that the presented stable slopes are theoretical values. The actual slope that the channel will develop may be influenced by an excess or deficiency of sediment supplies from upstream reaches, limited by armoring during degradation, or man-made grade controls. The concept of armoring is discussed in the following section.

4.5 Armoring Potential

If the sediment within the stream channel is coarse, a potential for armoring of the stream channel exists. That is, degradation to reach an equilibrium slope may be limited due to large particle sizes on the bed surface shielding finer particles under them. To determine the armoring potential of Cherokee Wash for the channel forming discharge (Q_{10}) an incipient motion analysis was conducted for both existing and proposed geometry conditions. Three methods were utilized to determine the critical sediment size:

- Competent Bottom Velocity Method
- Lane's Tractive Force Theory
- Shield's Diagram

An average of the results from the three methods was taken to estimate the size of bed material required for armoring of the channel bed. Results for existing conditions are summarized in Table 4.4. The incipient motion sizes determined for proposed geometry conditions are given in Table 4.5.

Table 4.4 - Armor size for existing geometry conditions

REACH	COMPETENT VELOCITY (mm)	LANES (mm)	SHIELDS (mm)	AVG (mm)
1	16	11	8.3	11.8
2	7.6	3.5	3.9	5
3	7.4	8.5	7.1	7.7
4	13	9.1	7.9	10
5	14.2	9.2	8.6	10.7
6	4.1	3.6	4.7	4.1
7	2.2	6.5	5.2	4.6
8	5.3	6.6	5.2	5.7
9	29.3	52	40.8	40.7
10	25.7	27	21.4	24.7
11	30.1	33	26.4	29.8
12	21.6	23	18.2	20.9
13	10.3	12	8.7	10.3

Table 4.5 - Armor size for proposed geometry conditions

REACH	COMPETENT VELOCITY (mm)	LANES (mm)	SHIELDS (mm)	AVG (mm)
1	16	11	8.4	11.8
2	7.7	3.5	4.4	5.2
3	16.4	22	17.5	18.6
4	7.7	3.7	4.8	5.4
5	12.9	13	7.9	11.3
6	4.1	3.6	4.8	4.2
7	2.2	6.5	5.2	4.6
8	5.2	6.6	5.2	5.7
9	33.5	60	47.4	46.9
10	31.9	33	27.5	30.8
11	24	27	21.1	24
12	26.4	30	24.5	27
13	12.6	17	11.1	13.6

The probable vertical channel degradation necessary for development of an armor layer along Cherokee Wash was estimated assuming that the armor layer thickness equals three times the incipient motion sediment size for the 10-year flood and the following equation (USBR, 1984):

$$yd = ya\,(1/P - 1)$$

where:

yd = Depth from original stream bed to top of armor layer, ft.
ya = Thickness of armor layer (assumed to be 3 times incipient motion size), ft.
P = Decimal percentage of original bed material larger than the armor size.

A summary of the expected armor depths for existing geometry conditions is shown in Table 4.6. Included in the table for comparison is the potential scour depth associated with the stable (equilibrium) slope, presented previously.

A summary of the expected armor depths for proposed geometry conditions is shown in Table 4.7. Included in the table for comparison is the potential scour depth associated with the stable (equilibrium) slope, presented previously.

Table 4.6 - Armor depth for existing geometry conditions

REACH	EXISTING SLOPE	STABLE SLOPE	SLOPE DIFFERENCE	REACH LENGTH (feet)	SCOUR DEPTH (feet)	ARMOR DEPTH (feet)
1	0.00782	8.09E-05	0.00774	409.3	3.2	-
2	0.00292	7.65E-05	0.00284	685.9	1.9	-
3	0.0069	7.94E-05	0.00682	623	4.3	-
4	0.00459	5.36E-05	0.00454	784.2	3.6	-
5	0.00795	2.04E-03	0.00591	1749.1	10.3	0.15
6	0.00729	1.32E-03	0.00598	1398.5	8.4	0.07
7	0.00532	6.51E-03	-0.00119	883.8	-1.0	0.03
8	0.00721	5.68E-03	0.00152	666.2	1.0	0.03
9	0.00893	2.18E-03	0.00675	2005	13.5	1.20
10	0.01029	6.46E-04	0.00964	243	2.3	3.23
11	0.01410	7.19E-04	0.01338	411.5	5.5	7.04
12	0.01285	1.20E-03	0.01166	513.5	6	0.46
13	0.01269	1.68E-03	0.01101	662	7.3	0.12

Table 4.7 - Armor depth for proposed geometry conditions

REACH	EXISTING SLOPE	STABLE SLOPE	SLOPE DIFFERENCE	REACH LENGTH (feet)	SCOUR DEPTH (feet)	ARMOR DEPTH (feet)
1	0.00782	8.10E-05	0.00774	409.3	3.2	-
2	0.00292	7.38E-05	0.00284	685.9	1.9	-
3	0.0069	1.02E-04	0.00680	623	4.2	-
4	0.00459	6.30E-05	0.00453	784.2	3.6	-
5	0.00795	2.07E-03	0.00588	1749.1	10.3	0.15
6	0.00729	1.31E-03	0.00598	1398.5	8.4	0.07
7	0.00532	6.51E-03	-0.00119	883.8	-1.1	0.03
8	0.00721	5.68E-03	0.00152	666.2	1.0	0.03
9	0.00893	2.32E-03	0.00661	2005	13.3	1.55
10	0.01029	6.87E-04	0.0096	243	2.3	7.28
11	0.0141	7.53E-04	0.01334	411.5	5.5	2.39
12	0.01285	1.55E-03	0.0113	513.5	5.8	0.80
13	0.01269	1.97E-03	0.01072	662	7.1	0.18

Comparison of Tables 4.6 and 4.7 reveals similar armor depths for both existing and proposed geometry alternatives. In both cases it was found that due to the lack of coarse sediments, no armor depth should be expected to develop in Reaches 1 through 4. As discussed previously, these reaches may be influenced by backwater effects of Indian Bend Wash and may be aggradational. In any event, the maximum scour depth associated with development of stable (equilibrium) slopes for reaches 1 through 4 is less than 4.5 feet.

Along upstream reaches, armor depths were found to be relatively shallow, ranging from less than a tenth of a foot for most of the wash up to 7 feet in Reach 11. The value for Reach 11 is thought to be unrealistically high due to the sediment gradation characteristics chosen to represent the reach in the analysis (note that no sediment sample was taken in Reach 11).

A comparison was made of the computed armor depth to the potential scour depth associated with the estimated stable slopes for each reach. It can be concluded that armoring will effectively limit general degradation along the wash to depths of three feet or less.

Higher magnitude floods could have substantially greater scour potential with an armor layer forming at a greater scour depth. However, these effects are associated with local scour due to peak flows; the scoured portion of the bed would then fill on the receding limb of the hydrograph. The preceding discussions dealt with armoring potential for stable channels, calculated using the 10-year event as the channel forming discharge.

4.6 Summary and Conclusions

In the preceding sections an evaluation of the geomorphic conditions along Cherokee Wash was presented. The evaluation included a description of historic and existing conditions of the Cherokee Wash watershed, examination of the channel profile, analysis of hydraulic parameters for existing and proposed geometry conditions, and estimation of equilibrium slopes for each reach of the Wash for both analysis conditions. Finally, the armoring potential of the Wash was evaluated.

Conclusions of the analysis include the following:

- Comparison of historic and existing conditions revealed significant changes in the watershed of Cherokee Wash. No significant changes in the channel location was noted between historic and existing conditions. However, the watershed is now substantially developed for residential housing. Roads now cross the Wash at twelve locations. These crossings include both culverts and dips. The watershed development encroaches on the Wash along much of its length. The development has likely increased runoff volumes and peaks along the Wash.

- Channel slopes vary substantially along the Wash, from a minimum of 0.0029 (15 feet/mile) near its confluence with Indian Bend Wash to a maximum of 0.0141 (75 feet/mile) in Reach 11 (Arroyo Road to Crestview Road) along the upper extent of the Wash.

- Hydraulic analyses of existing and proposed geometry conditions for the wash revealed only minor differences. Velocities along the wash range from 1 to 4 feet/sec for the 10-year flood, which is assumed to be the dominant discharge. The highest velocities, and sediment transport potential, occurs in the steep upper reaches of the wash. Zones of lower velocity, and aggradational potential, were noted at the mouth of the wash (Reaches 2 and 3) and in the middle section of the wash (Reaches 6, 7 and 8). The highest width/depth ratios are also associated with Reaches 6 and 7 which may also indicate a zone of aggradation.

- Based on the comparison of hydraulic characteristics for the dominant discharge, siting of a sedimentation basin at 52nd Street, the upstream end of Reach 8, would intercept sediment production from the steep upper reaches of the wash and reduce supply to the aggradational Reaches 6 and 7. However, a sedimentation basin located at 56th Street, the upstream end of Reach 5, may not be effective since zones of lower velocity, and resulting aggradation, are located upstream in Reaches 6 and 7.

- Bed materials along the wash were evaluated. It was shown that surface and subsurface bed materials display similar size characteristics. Generally, bed materials are coarse, comprised primarily of sand- and gravel-sized sediments. Sediments sizes tend to become finer in a downstream direction. Sediment sizes in the lower reaches of Cherokee Wash may be influenced by backwater effects of Indian Bend Wash.

- Stable slopes for the various reaches of the Wash were estimated by four methods. An average value for each reach of the wash was determined for both existing geometry and proposed geometry analysis alternatives. The estimated stable slopes are theoretical values that would develop if the reach is not impacted by excess or deficiency of upstream sediment supplies, armoring of the channel bed material, or grade controls.

- The armoring potential of the wash was evaluated for both existing geometry and proposed geometry conditions. The critical sediment size for incipient motion was determined by three methods and an average value was chosen for each reach and analysis condition. The depth to which each reach would need to degrade to form an effective armor layer was calculated. It was found that due to the lack of coarse sediments, no armoring should be expected to develop in Reaches 1 through 4. Along upstream reaches armor depths were found to be relatively shallow, ranging from less than a tenth of a foot for most of the wash up to 7 feet in Reach 11. The armor depth value for Reach 11 is thought to be unrealistically high because of the sediment gradation characteristics chosen to represent the reach. It is noted that no sediment sample was taken in Reach 11.

- A comparison was made of the computed armor depth to the potential scour depth associated with the estimated stable slopes for each reach. It can be concluded that armoring will effectively limit general degradation along the wash to depths of three feet or less.

5.0 SEDIMENTATION ENGINEERING

5.1 Introduction

This section of the report documents the sediment analyses performed by WEST for Cherokee Wash including sediment sampling, sediment yield estimates, and numerical transport modeling. Several HEC-6 models were developed to analyze sediment continuity in Cherokee Wash for both existing and with-project conditions, for long-term and single event hydrology, and for sensitivity analyses.

5.2 Sediment Sampling

Sediment sampling and analysis was performed by Law/Crandall (LAW) under separate contract to FCD (FCD 95-40). Sediment gradations resulting from the sieve analyses were used in the geomorphic analysis and numerical modeling activities. WEST prepared a sediment sampling plan and accompanied LAW personnel to select sampling sites. Sediment was sampled at twelve locations along the wash and results were presented in LAW's report (LAW, 1996). Sampling locations were presented in Figure 4.7.

5.3 Sediment Yield Estimates

Estimates of the amount of sediment entering Cherokee Wash from its tributaries are necessary to correctly model the sediment continuity in the study reach. Sediment yield estimates were computed for two areas in the Phoenix Mountain Preserves considered to contribute the majority of the sediment supply to Cherokee Wash. The first area delivers its load via the West Branch of Cherokee Wash (or Western tributary), whose confluence is just south of the intersection of Mockingbird Lane and 52nd Street. The second area delivers its load via the tributary which runs along the northern edge of Desert Park Lane (herein called the Desert Park tributary). Several methods were used to estimate average annual sediment volume and sediment delivery volume for the 2-, 10-, 50- and 100-year events. All of the methods used required hydrologic parameters which were obtained from the WEST HEC-1 models. A summary of the various methods and comparison of the results from each follows.

5.3.1 Sediment Yield Equations

Modified Universal Soil Loss Equation (MUSLE) - Williams and Berndt (1972) developed the MUSLE equation to estimate the total sediment yield from a watershed for a single storm event. This method considers sheet and rill erosion only and does not account for gully erosion, channel bed and bank erosion or mass wasting. The Albuquerque Sediment and Erosion Design Guide (RCE,1994) recommends using MUSLE results for wash load only, and to add bed material load calculated by sediment transport calculations to obtain total yield.

Pacific Southwest Inter-Agency Committee (PSIAC) - The PSIAC method (PSIAC, 1968) computes sediment yield based upon nine factors. These are geology, soils, climate, runoff, topography, ground cover, land use, upland erosion, channel erosion, and sediment transport. Unlike the MUSLE method, PSIAC estimates total annual sediment yield rather than just sheet and rill erosion.

Renard - The Renard (1972) method predicts sediment yield from semiarid watersheds by simulating individual hydrographs and computing sediment yield for the estimated hydraulic conditions. Sediment yield is related to drainage area by the following equation:

$$Y = 0.001846 A^{0.1187}$$

where: Y = Average Annual Sediment Yield (acre*ft/acre/yr)
A = Drainage Area (acres)

USBR - The USBR method (Design of Small Dams; USBR, 1987) is based on sedimentation data from selected reservoirs in the southwestern U.S. This method also computes sediment yield from drainage area, using the following equation:

$$Q_s = 1.84 A^{-0.24}$$

where: Q_s = Average Annual Sediment Yield (acre*ft/mi^2/yr)
A = Drainage Area (mi^2)

Table 5.1 presents the estimated sediment yield in acre-ft/acre/year for each of the previously described methods for the two main tributaries of Cherokee Wash at the location where they enter the wash.

Table 5.1 - Sediment Yield Estimates (acre-ft/acre/year) for Cherokee Wash Tributaries

	MUSLE	PSIAC	Renard	USBR
Western Tributary	0.00681	0.00100	0.00108	0.00461
Desert Park Tributary	0.00651	0.00098	0.00099	0.00387

The Corps of Engineers (USACE, 1995b) reports values in Arizona and New Mexico ranging from 0.03 to 4.3 tons/acre/year (0.000014 to 0.002078 acre-ft/acre/year, assuming a sediment unit weight of 95 lb/ft^3). Fuller (1997) provides sediment yield estimates for detention basins in Arizona watersheds similar to that of Cherokee Wash ranging from 0.1 to 0.9 acre-ft/mi^2/year (0.00016 to

0.0014 acre-ft/acre/year). Alonso (1997) reports that yields from the Walnut Gulch Experimental Watershed (near Tombstone, Arizona) were between 100-300 metric tons/km^2/year (0.00019-0.00063 acre-ft/acre/year).

Based on the foregoing, we judged the Renard equation to be most applicable of the four methods presented. The MUSLE equation is too high and is not recommended for total load estimates by AMAFCA (RCE, 1994). The USBR equation also gives yields that are much higher than regional estimates. The PSIAC results appear to be valid, but are slightly lower than the Renard equation results.

5.3.1.1 Conversion of Average Annual Yield to Single Event Yield

A rough method of establishing single event yield based on average annual yield is presented in the Corps of Engineers Training Document No. 36 (USACE, 1995b):

$$\text{Yield}_i = \text{Yield}_{AvgAnn} * Q_i/Q_b$$

where Yield_i is the single event yield for an i^{th}-year storm, Q_i is the peak water discharge for the i^{th}-year event and Q_b is a reference event for which the yield is equivalent to the average annual yield. The Corps document suggests that the 2-year event produces an amount of sediment equal to the average annual yield (i.e., $Q_b = Q_2$). However, as mentioned in section 4.3.3 of this report, the 10-year event often correlates better to channel forming discharge in arid regions. Yields were therefore calculated for both tributaries for single events using Q_b equal to both the 2-year and the 10-year event. Results are presented in Table 5.2 for the yields at the confluence of each tributary with Cherokee Wash. Sediment flows (for use with HEC-6, described below) can now be calculated based on these yields. By multiplying the ratio of the sediment volume (obtained from the values in Table

Table 5.2 - Single Event Sediment Yields (in acre-ft) for Tributaries

Reference Event	Event	Western Tributary	Desert Park Tributary
$Q_b = Q_2$	100-year	1.32	0.88
	50-year	1.09	0.72
	10-year	0.56	0.37
	2-year	0.18	0.10
$Q_b = Q_{10}$	100-year	0.43	0.23
	50-year	0.35	0.19
	10-year	0.18	0.10
	2-year	0.06	0.02

5.2) to the water volume for an event (obtained from the HEC-1 results) and the peak flow for that event, sediment inflows are obtained. Results are presented graphically with equilibrium sediment transport results (see next section) in Figures 5.1 and 5.2 (figures are presented at the end of this chapter).

5.3.2 West Branch Equilibrium Sediment Transport

A second way to estimate sediment delivery is by assuming a tributary is in equilibrium and sediment delivered to the main wash equals the sediment transport capability of the tributary. Cross sections were available for the West Branch of Cherokee Wash such that this analysis could be performed. As no geometric data was available for the Desert Park Tributary, results had to be inferred from the West Branch analysis.

The West Branch cross sections were contained in one of the District's HEC-2 models (see Section 3). These were inserted into a new HEC-2 model, which used all original input parameters except for discharge (new inflows resulting from the HEC-1 models were used) and starting water surface elevations. For these, a rating curve was created from the final WEST model for unmodified geometry using the water surface elevation at the confluence of the West Branch with the main wash for each discharge event. Average velocities and depths for each flow were estimated for the West Branch for the reach downstream of Tatum Boulevard and upstream of the confluence. A computer package (described in Stevens and Yang, 1989) was used to calculate equilibrium sediment discharge for a series of flows which included the recurrence interval discharges. Results are presented in Figure 5.1 for transport methods judged to be most applicable, along with results from the Renard yield described in the preceding section.

For the Desert Park tributary no cross sections were available. An assumption was made that the slopes of the sediment rating curve determined for the West Branch by the Yang Sand and Gravel transport method would be applicable to the Desert Park tributary as well. The results from the Renard method yield and the Yang slope approximation are shown in Figure 5.2 (the Yang slope curve was applied assuming the 10-year sediment yield equal to the same yield from the Renard results).

5.3.3 Adopted Sediment Yield

The goal of the sediment yield analysis was to develop a good estimate of the inflowing sediment load from tributaries to the Cherokee Wash study reach being modeled. This inflowing load is a needed input to the HEC-6 sediment transport model described in the next section. Based on the foregoing discussion and results presented in the figures, we decided to use the sediment rating curve from the Yang Mixed (sand and gravel) transport equation as input to the HEC-6 model for the West Branch. For the Desert Park tributary, the curve developed with the Yang slope approximation was used. An inflowing sediment sensitivity analysis was performed as part of the sediment continuity analysis described below.

5.4 Sediment Continuity Analyses/HEC-6 Model Development

5.4.1 Description of HEC-6 Model

HEC-6 is a one-dimensional movable boundary, open channel flow model designed to simulate stream bed profile changes over fairly long time periods. Since its initial nationwide distributions by the Hydrologic Engineering Center (HEC) of the Corps of Engineers in 1973 and again in 1977, 1987 and 1991, it has been the most widely used one-dimensional sediment transport model in the United States, and particularly with the Corps of Engineers.

In general terms, the model first calculates the hydraulics of each discharge increment in a hydrograph to determine hydraulic parameters such as flow depth, water velocity, and effective flow width for each cross section. It then computes the sediment transport potential at each cross section using the hydraulics of the main channel. Sediment contribution at the upstream end of the reach being modeled is simulated by the use of a sediment vs. discharge relation and is specified by the user. This load is compared to the sediment transport potential of the cross section. If the inflowing load is larger than its transport potential, the difference is deposited in the cross section. If the inflowing load is less than the transport potential, it is picked up (scoured) from the bed, taking into account the availability of material in the bed (e.g., bedrock, armoring, etc.). The sediment load leaving the cross section then becomes the inflowing load to the next downstream cross section. This continues until the most downstream cross section is simulated. For the next discharge in the hydrograph, the hydraulics are again computed using the new cross sectional geometry formed by the previous discharge. The cycle is repeated until the entire hydrograph is simulated. Further details of the model are presented in the HEC-6 User's Manual (USACE, 1993) and MacArthur et al. (1990).

HEC-6 requires hydraulic analysis for the water discharge being simulated, the input of representative streambed material size distributions, the creation of an inflowing sediment rating curve, and the development of a design hydrograph containing the design event and a representative long term hydrograph. The procedures used in developing the HEC-6 inputs are described in the following sections.

5.4.2 Cross Section Geometry and Hydraulics

Cross sectional geometry of the streambed and overbanks is required for input into the sediment transport model. The geometry from the HEC-2 model CHERORIG.DAT was used in the base conditions model and the geometry from CHERMOD1.DAT was used in the improved channel model. Several cross sections, mostly near road crossings, were removed in the HEC-6 model to aid in model convergence.

5.4.3 Limits of Erodible Bed

From the field reconnaissance and plots of the cross sections, the lateral limits of scour and deposition were determined and input to the HEC-6 model. The model assumes that erosion is uniform between these limits but deposition can occur outside these limits but within the wetted portions of the channel. In general, the limits of scour are within what is termed the "active bed" and are often located just within the main channel limits. The vertical limit or "bottom elevation" of the erodible bed was set at ten feet for all sections other than road crossings. In the upstream reaches of the wash, armoring and cemented layer/rock were identified. However, by using the correct gradations in the model, armoring will be correctly simulated even with a large sediment reservoir. The rock outcrops observed were highly variable, and cannot be accounted for without data from detailed subsurface exploration or geophysical data. Therefore, the bottom limit was not fixed at zero anywhere other than road crossings.

5.4.4 Inflowing Sediment Load

Generally, no information was available on the sediment entering the stream crossings from upstream sources. We assumed the inflowing sediment at the upstream end of the model (Paradise Valley Country Club golf course) would be negligible because the relatively flat fairways would tend to trap sediment before it could reach the wash. The main sediment inflow to Cherokee Wash is assumed to come from the Phoenix Mountain Preserve. Two main inflow points were identified, coincident with two water inflow points. The Desert Park tributary delivers its load via a series of culverts along the northern edge of Desert Park Lane (cross section 10373). The West Branch of Cherokee Wash delivers water and sediment near the 52nd Street/Mockingbird Lane curve at cross section 7800. Sediment inflow from the Mummy Mountain area was assumed to be minor compared to the contribution from the Phoenix Mountain Preserve area. A sediment rating curve was entered into the models using the adopted data described in Section 5.3 (Sediment Yield Estimates).

5.4.5 Sediment Gradations

Estimates of bed material gradations are necessary input to HEC-6, both for the material in the bed and for any inflowing sediment loads.

5.4.5.1 Bed Material Gradations

Bed sediment samples and gradations within the study reach are a requirement for sediment transport modeling. The streambed gradations are input to the HEC-6 models. At cross sections that do not have sample gradations available, the upstream and downstream cross section gradations are linearly interpolated to produce a representative gradation. This interpolation is performed automatically in HEC-6. Representative sediment gradations were chosen from those prepared by LAW (1996). At many locations, several samples were collected at different depths. In general, the subsurface gradations were favored as they are more representative of the material actually transported.

5.4.5.2 Inflowing Load Gradations

The gradation of the total inflowing load was determined by two methods: 1) a weighted composite of the bed material and wash load gradation curves as described in the Corps of Engineers' Training Document No. 36 (USACE, 1995b), and 2) size fraction output from the SEDDISCH computer program (Stevens and Yang, 1989) when computing equilibrium sediment transport in the West Branch as described in Section 5.3.2.

In order to perform the weighted composite calculation, the relative amount of bed material load as opposed to wash load must be estimated. For the West Branch of Cherokee Wash values of 20% and 10% were used. Assuming $D_{10\ \text{bed load}} = D_{100\ \text{wash load}}$ (Einstein, 1950), a gradation curve was created for the wash load. Using both the computed wash load gradation curve and the bed load gradation curve from sediment sample 11A the percent of material in each size class was determined. The resulting distributions were assumed to apply to both tributaries. The distribution does not change with flow event.

The equilibrium sediment discharge analysis for the West Branch was described previously. Several of the transport functions evaluated compute sediment movement by size fraction. The resulting distributions for the 100-year discharge, along with the results for the weighted sediment calculations, are presented in Table 5.3. Gradations for the 2-year discharge were also evaluated but are not presented because, with the exception of the Yang Sand function, none of the values in Table 5.3 changed by more than 2 percent. The Yang Sand function for the 2-year discharge increased the percentage of very fine sand from 58 to 70 percent, bringing it more in line with the other functions' estimates. The gradation adopted for use in the HEC-6 models is also presented in Table 5.3.

Table 5.3 - Inflowing Sediment Load Gradation

	Sediment Transport Relation (100-year event)					USACE TD-36		
Size Class*	Yang Sand	Yang Gravel	Yang Mixed	Toffaleti	Schoklitsch	Wash - Bed Split 80-20	Wash - Bed Split 90-10	ADOPTED
VFS	58.35%	0.07%	79.32%	80.91%	28.98%	65.00%	73.00%	75.00%
FS	5.30%	0.07%	7.20%	10.24%	8.18%	17.00%	18.00%	12.00%
MS	2.87%	0.50%	3.90%	4.89%	8.66%	1.07%	0.53%	4.00%
CS	4.08%	3.36%	5.54%	3.24%	16.30%	2.33%	1.16%	5.00%
VCS	2.13%	5.00%	2.89%	0.50%	8.60%	2.81%	1.40%	3.00%
VFG	2.43%	10.22%	0.13%	0.15%	8.04%	2.11%	1.06%	0.10%
FG	2.96%	17.15%	0.22%	0.05%	6.97%	2.95%	1.48%	0.20%
MG	4.74%	19.22%	0.24%	0.02%	6.17%	3.21%	1.61%	0.20%
CG	6.36%	26.36%	0.33%	-	4.64%	3.00%	1.50%	0.30%
VCG	10.79%	18.08%	0.23%	-	3.46%	0.53%	0.26%	0.20%

*Size Class Code: V=very, F=fine, M=medium, C=coarse, S=sand, G=gravel

5.4.6 Sediment Transport Equation

Selection of an appropriate sediment transport relation, despite numerous studies that rank various equations, is still very much based on the judgement of the modeler. One must look at the data from which the functions were derived, what types of data they have been compared to (laboratory flume versus river measurements) and past usage. The user of HEC-6 is limited to 11 transport functions from which to choose. Based on the sediment sampling results and field reconnaissance, we judged Yang's stream power function to be the most applicable for Cherokee Wash. The function is a total load function (bed load plus suspended bed material load) which is often used for particle gradations with a D_{50} of 10mm or less (medium gravel). Although material in some reaches of the wash is coarser than 10mm, the great amount of material transported will be finer than this, and the larger material will still be present in calculations to see if armoring occurs. Bed load functions (e.g., Meyer-Peter Muller), which favor gravel transport, were not considered as they would not correctly estimate the sand sizes which constitute the majority of sediment transported. The sensitivity of the base model to transport function was tested and is discussed later in this chapter.

5.4.7 Single Event Hydrographs

A continuous hydrograph is modeled in HEC-6 as a series of steady flows. Hydrographs from the HEC-1 models for the 2-, 10-, 50- and 100-year events were subdivided into a series of short duration steady flows for input into HEC-6.

5.4.8 Average Annual Hydrograph

In order to use HEC-6 for long-term sedimentation analysis, an average annual flood hydrograph must be developed. We accomplished this by constructing a flow-duration curve for Cherokee Wash using flows downstream of 56th Street (Figure 5.3, at the end of the chapter). The four recurrence interval flows are plotted versus time exceeded; the four points were used to extrapolate a low flow of 150 cfs corresponding to approximately a 1-year flow event. From the recurrence interval of each of the flows, the number of days in an average year corresponding to that flow was calculated. Using the flow and duration of each event for an average year, the hydrograph was developed (Figure 5.4). The duration of every flow other than the 100-year event was split in two; half the flow duration was placed before the peak discharge, and half after, to create the shape seen in Figure 5.4. The average annual hydrograph was run twenty times, corresponding to a long term simulation of twenty years.

5.5 Existing Conditions Model

Changes in channel bed elevation were evaluated for both long-term simulations and for single events. HEC-6 results must be viewed with caution, however, for the single event results because the flashy response of the basin violates the model's assumption of gradually varying discharge.

5.5.1 Fixed-Bed Analysis

The existing conditions model was first executed with a fixed bed, steady discharges and no sediment inflow in order to compare the hydraulics to the HEC-2 model results. Most of the cross sections showed differences in elevation between the two models of less than 0.2 feet. However, at a few locations the water surfaces varied by as much as 0.6 feet, and for one location, 1.0 feet. This was principally due to the removal of HEC-2 cross sections in the HEC-6 model which had been at critical depth. This caused critical depth to move to a different section in the HEC-6 model. Overall we were confident that the hydraulics of the system were being correctly reproduced.

5.5.2 Long-term Simulation

The 20-year hydrology previously described was entered into the base model, which was then executed. The resulting average bed elevation profiles, at five-year intervals, are shown in Figure 5.5 at the end of the chapter. The most striking features of the plot are the areas of degradation downstream of the crossings at 56th Street and Mockingbird Lane. The results indicate degradation of the average bed elevation by 5.0 and 4.5 ft., respectively. It is important to note that depth to bedrock is not known at these locations and could limit the predicted scour. Another feature to note is degradation of the bed at the very upstream channel reach due to lack of sediment entering the system from the golf course. However, it is apparent that the resulting average bed elevations have stabilized after about 15 years due to armoring effects. Deposition of sediment is evident upstream from Mockingbird Lane (cross sections 5693 to 6900) as well as the reach just downstream of Desert Jewel Drive (cross sections 8400-9205) and just upstream of 58th Place (cross sections 2502 to 2700). There is a zone of degradation just downstream from the West tributary (cross sections 7200 to 7800) indicating that this tributary is perhaps sediment supply limited. There is also a zone of degradation at the very downstream end of the model, but this is thought to be created by backwater effects from the starting water surface elevations used and should be viewed with caution. Finally, there is a degradation zone between Morning Glory Road and 59th Place, probably due to the very narrow cross sections surveyed which produce higher channel velocities.

5.5.3 Single Event Simulations

Models were created for each of the frequency events: 100-year, 50-year, 10-year and 2-year. As previously mentioned, the flashy response of the watershed with its quickly rising and falling hydrographs (flash flood conditions) render some of the key assumptions in HEC-6 invalid; however, the general response and tendencies should be representative of prototype behavior. The analyses were run to see what change, if any, would occur in the channel average bed elevation profile and what sediment yield would be delivered to Indian Bend Wash. Results of the first part of the analysis are shown in Figures 5.6 through 5.9 at the end of the chapter. There is almost no change in the average bed profile for all events. The only noticeable features are degradation at the very downstream end of the model, which is suspect as previously described due to backwater effects, and some degradation just upstream of Morning Glory Road. Bed material delivery to Indian Bend Wash is described in section 5.8.

5.6 Sensitivity Analyses

Sensitivity analyses were performed on the base, long-term HEC-6 model to gauge the model's reaction to variations in inflowing sediment load, channel roughness, and sediment transport equation used.

5.6.1 Inflowing Sediment Load

The base model was run with the inflowing sediment load from both tributaries cut in half. Then the model was executed with inflowing loads increased 50% over the base values. The results of these runs are shown in Figure 5.10. It can be seen that the patterns of scour and deposition observed in the base case do not change when the inflowing sediment load is varied. There is more scour or less deposition when the load is cut in half and less scour or more deposition when the load is increased by 50 percent. Downstream of the crossings at 56th Street and Mockingbird Lane, the average bed elevations for half the inflowing load are about half a foot lower compared to the base case. Likewise, the increased inflowing sediment load results show about half a foot less scour compared to the base results.

5.6.2 Vegetation in the Channel (Roughness Values)

Two models were prepared to gage the sensitivity of the base model to changes in channel roughness. In the first model (high n), all channel roughness values upstream of Morning Glory Road (except street crossings) were increased to 0.06 to reflect growth of vegetation in an unmaintained channel. Cross sections downstream of Morning Glory Road are currently grass or dirt and are not expected to fill with heavy vegetation. In the second model (low n), all channel roughness values above 0.035 were reduced to 0.035 to reflect maintenance of vegetation (lesser values were not changed). Overbank roughness values were not changed in either model. The results of the modeling efforts are shown in Figure 5.11. The results show the same trends of scour and deposition as noted previously. However, they are unusual in that both the high n and low n models show less scour downstream of the road crossings at 56th Street and Mockingbird Lane than the base case. We believe the explanation for this behavior is as follows. For the low n model, almost all reductions in n value occur upstream of 56th Street. Therefore, increased velocities upstream are supplying more sediment to the downstream reaches, reducing the amount of scour downstream of the crossings and increasing the deposition upstream of Mockingbird Lane. Conversely, most of the changes in roughness in the high n model occur in the area downstream of Mockingbird Lane, reducing velocities and again reducing scour below the crossings. Note that the low n value model predicts greater scour for the wash upstream of where the West tributary enters the system (cross section 7800), while the high n model predicts less scour or even deposition compared to the base case results.

5.6.3 Sediment Transport Equation

As previously described, the Yang transport equation was judged most applicable for Cherokee Wash. Although typically used for sands, the Ackers-White formulation was also deemed sufficiently appropriate for the wash to be tested. The results of the comparison between the transport equations are shown in Figure 5.12. It can be seen that while producing almost exactly the same results as the Yang equation downstream of Mockingbird Lane, the Ackers-White equation predicts deposition from this crossing all the way upstream to Desert Jewel Drive. In fact, the Ackers-White model predicts an increase of over 5 feet in the average bed elevation at cross section 6000 (just upstream of Mockingbird Lane) while the base model predicted only about a foot. We believe this is due to the Ackers-White equation not being able to accurately model transport of the coarser material. In any case, changing the transport function in HEC-6 usually affects transport rates more than geometry changes (USACE, 1992). The fact that the same trends are shown with either transport equation lends confidence to the modeling process itself.

5.7 With-Project Model

The with-project HEC-2 model previously described in the Hydraulic Analysis section was modified for use with HEC-6. This model was first executed with a fixed bed, steady inflows, and no sediment inflow in order to compare the hydraulics to the HEC-2 model results. Most of the cross sections showed differences in elevation between the two models of less than 0.2 feet. However, at a few locations the water surfaces varied by as much as 0.6 feet, and for one location 1.9 feet. This was due to the removal of HEC-2 cross sections in the HEC-6 model which had been at critical depth. This caused critical depth to move to a different section in the HEC-6 model. Overall, we were confident that the hydraulics of the system were being correctly reproduced.

The model was then modified to include the long-term hydrology, moveable bed, and sediment inflows. The results of this model are shown in Figure 5.13 with the existing conditions results. It can be seen that the results are nearly identical. The only noticeable difference is an increase in degradation for the with-project conditions between 58th Place and 56th Street (cross sections 2502.4 to 4200). The with-project results indicate about half a foot more of degradation compared with the base condition results. There is also a noticeable increase in scour just downstream of 58th Place (cross section 2400). While shown to be relatively stable for base conditions, this location shows degradation of about a foot for with-project conditions. Overall, it appears that the effect of the with-project modifications will be minor.

5.7.1 Stable Channel Slopes

Stable channel slopes can be estimated for the with-project conditions based on the HEC-6 model results. Stable channel slopes per reach (using the geomorphic reach numbering shown in Figure 4.3) are given in Table 5.4 along with results from the geomorphic analysis for comparison. It can be seen that, except for reaches 7 and 8, the HEC-6 results are steeper than the geomorphic analysis results. This is probably a reflection of the limited sediment supply to the wash, as well as the rough nature of the slope estimates from the HEC-6 output (in some cases there were no more than two or three cross sections in a reach to define the slope).

Table 5.4 - Estimated Stable Slopes per Geomorphic Reach

REACH	HEC-6	Geomorphic
1	-0.00107	8.1E-05
2	0.00400	7.38E-05
3	0.00585	0.000102
4	0.003458	6.3E-05
5	0.003505	0.002067
6	0.003344	0.001309
7	0.005264	0.006507
8	0.003533	0.005681
9	0.010399	0.002316
10	0.029277	0.000687
11	0.016829	0.000753
12	0.012791	0.001548
13	0.005768	0.001972

5.8 Bed Material Delivery to Indian Bend Wash

The average bed material delivery rate to Indian Bend Wash, based on HEC-6 model results, is slightly higher at the beginning of the long-term simulation, about 8 acre-ft/year, than at the end when it drops to about 7.8 acre-ft/year. Results for sediment delivery for different conditions (sensitivity, with-project) are shown in Table 5.5. Compared to results obtained using annual sediment yield methods (Table 5.6), the HEC-6 results are much higher. This could be due to several factors including overestimation of flows (by hydrologic methods) or overestimation of sediment inflows. Note, however, that even if sediment inflows are overestimated that the system is still showing general degradation. Reduction of inflows, as shown in the sensitivity analysis, would result in only slightly more degradation in the Wash because of channel armoring. In any case, the sediment yield estimates described in Section 5.3 are site-specific whereas the annual yield methods are regional averages. Table 5.7 presents estimates of sediment delivered for the HEC-6 single event analyses. The ability of Indian Bend Wash to transport the delivered sediment can not be estimated without further study.

5.9 Summary

Due to the lack of measured data, no calibration of the models was possible. Assumptions were made to arrive at estimates of sediment yield from the tributaries to Cherokee Wash and inflowing sediment load gradations. Model geometry was developed from the HEC-2 models for both existing and with-project conditions. Models were run first in fixed bed mode with steady discharges to approximate HEC-2 results before continuing with mobile bed runs of the models. Hydrographs were developed for four frequency events and for a twenty year period of "average" flows. Sensitivity runs were performed to see how the model would react to changes in inflowing sediment load, channel roughness values, and sediment transport function used. The with-project model results were very similar to the existing condition model results for the long-term simulations. Bed material sediment delivery (does not include wash load) to Indian Bend Wash was estimated for several scenarios.

Table 5.5 - Average Annual Sediment Delivery to Indian Bend Wash from Cherokee Wash

Case	Base	Ackers-White	High n	Low n	Half Load	Increased Load	With-Project
Ave. Yield (ac-ft/year)	8.0	7.3	7.8	7.9	4.2	11.6	7.9

Table 5.6 - Average Annual Sediment Yield by Selected Methods

Method	Renard	USBR	PSIAC
Yield (ac-ft/year)	1.0	3.0	0.8

Table 5.7 - Event Sediment Delivery to Indian Bend Wash from Cherokee Wash

Event	2-year	10-year	50-year	100-year
Yield (ac-ft)	1.0	1.8	2.4	2.7

Figure 5.1 - West Branch Cherokee Wash at Cherokee Wash
Sediment Rating Curves

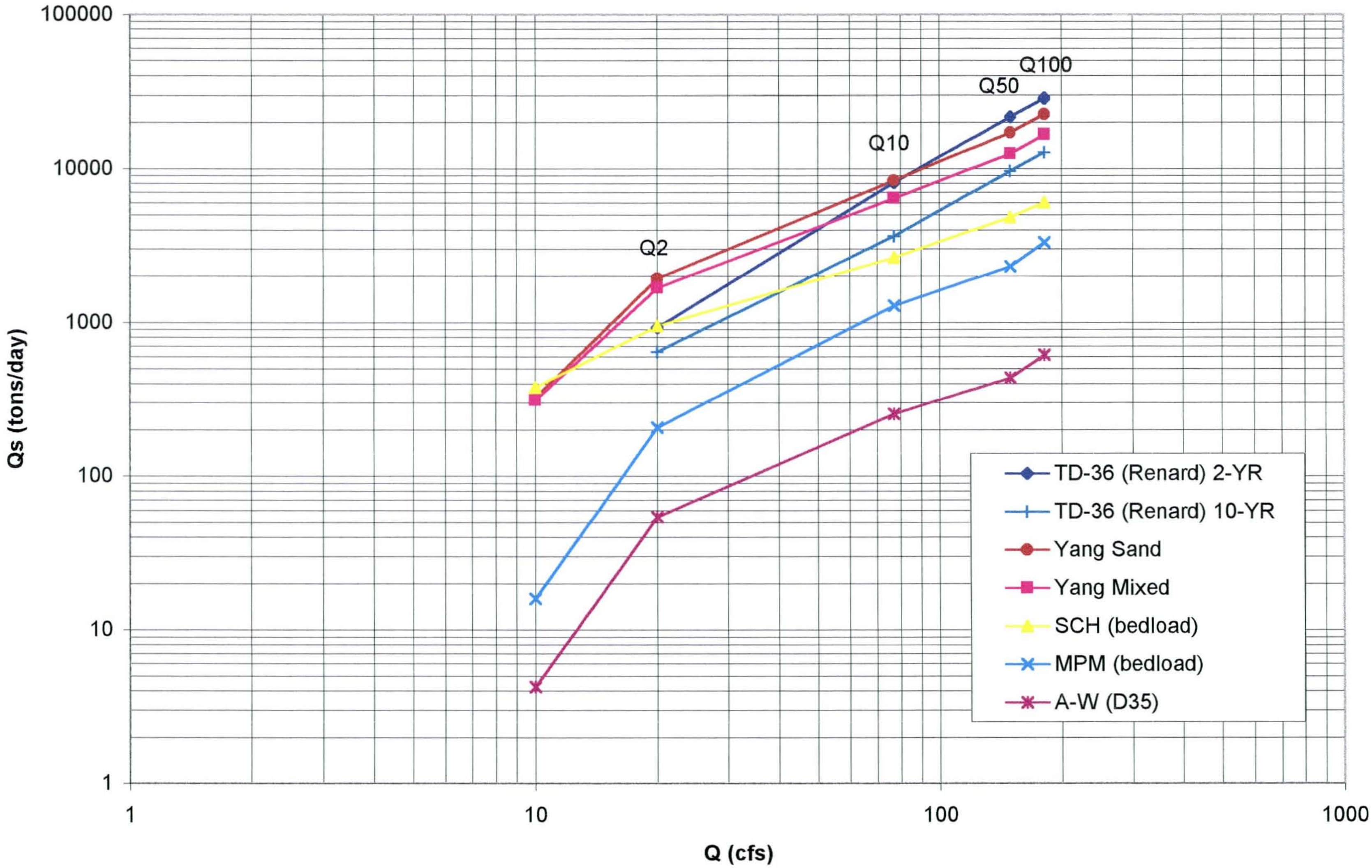


Figure 5.2 - Desert Park Tributary at Cherokee Wash
Sediment Rating Curves

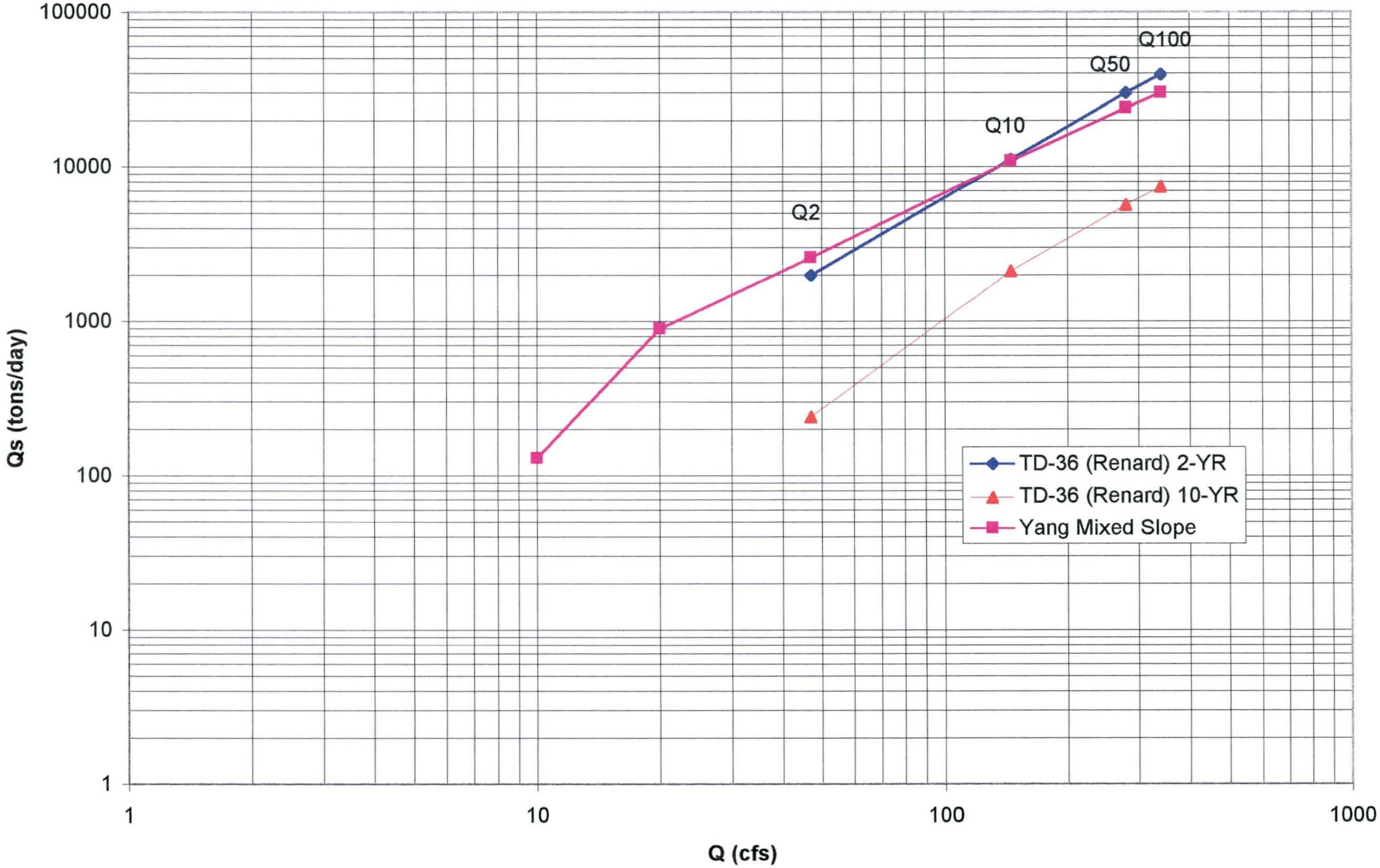


Figure 5.3 - Cherokee Wash Flow Duration

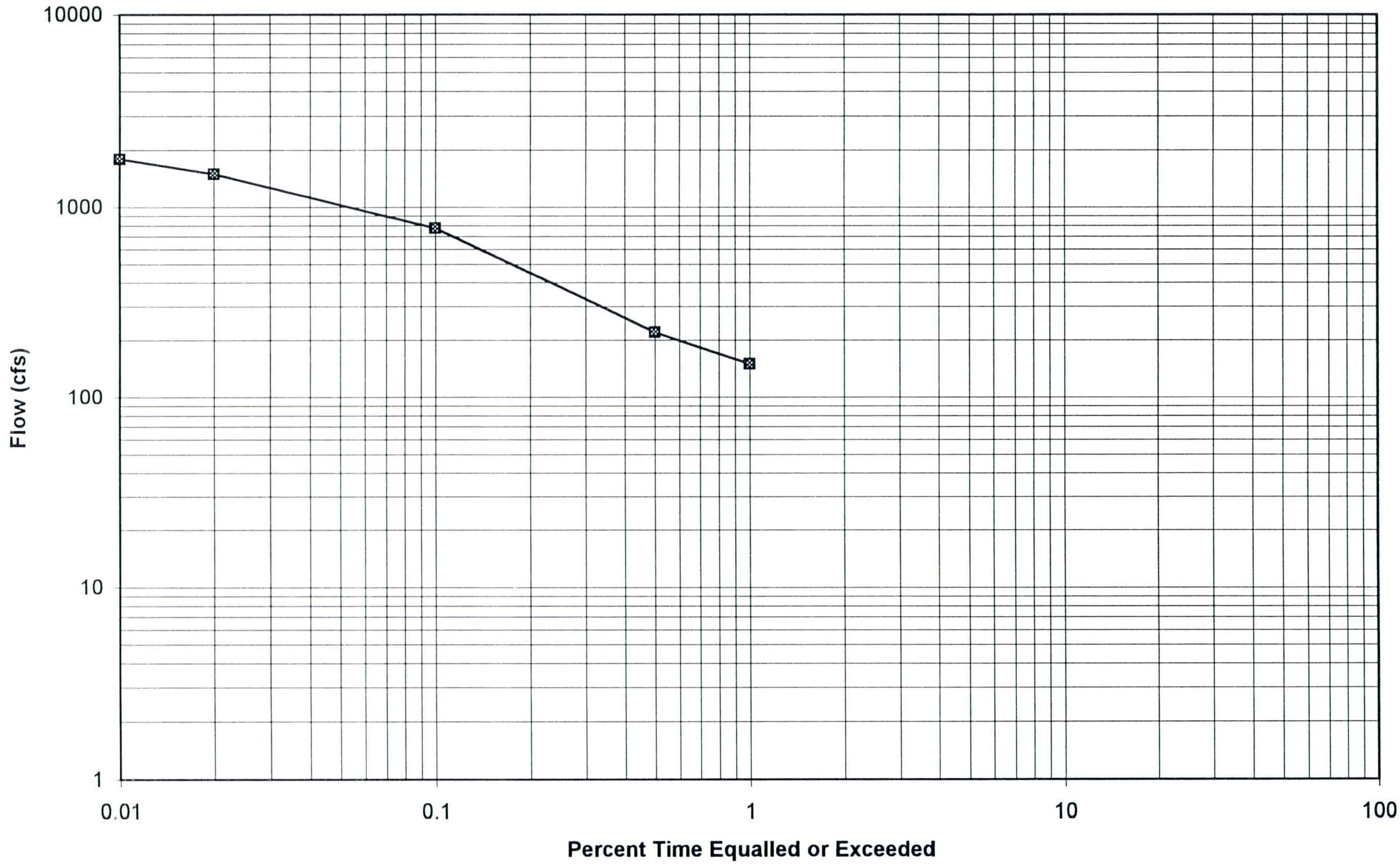


Figure 5.4 - Cherokee Average Annual Hydrograph

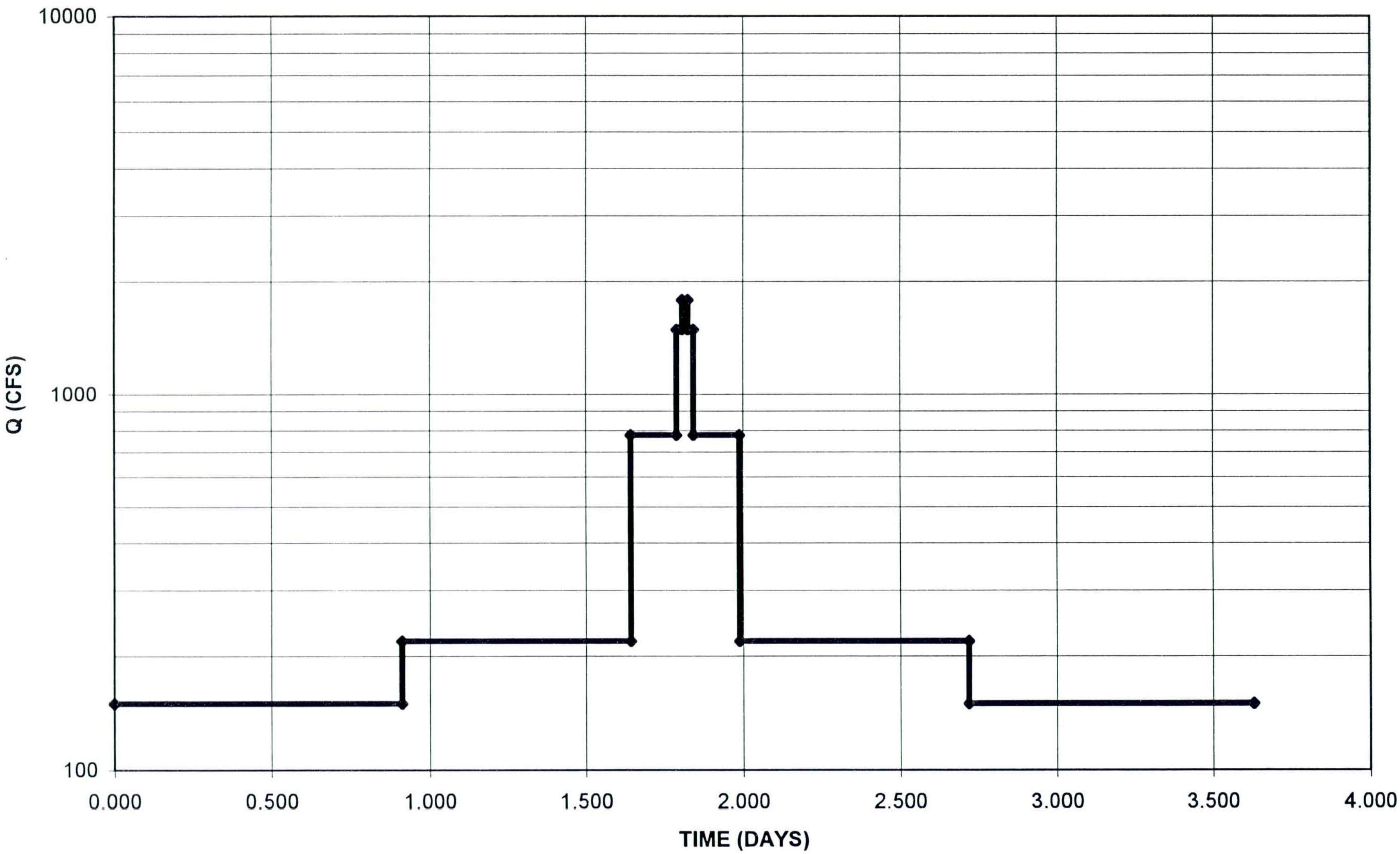


FIGURE 5.5 - CHEROKEE WASH
BASE CONDITIONS AVERAGE BED CHANGE

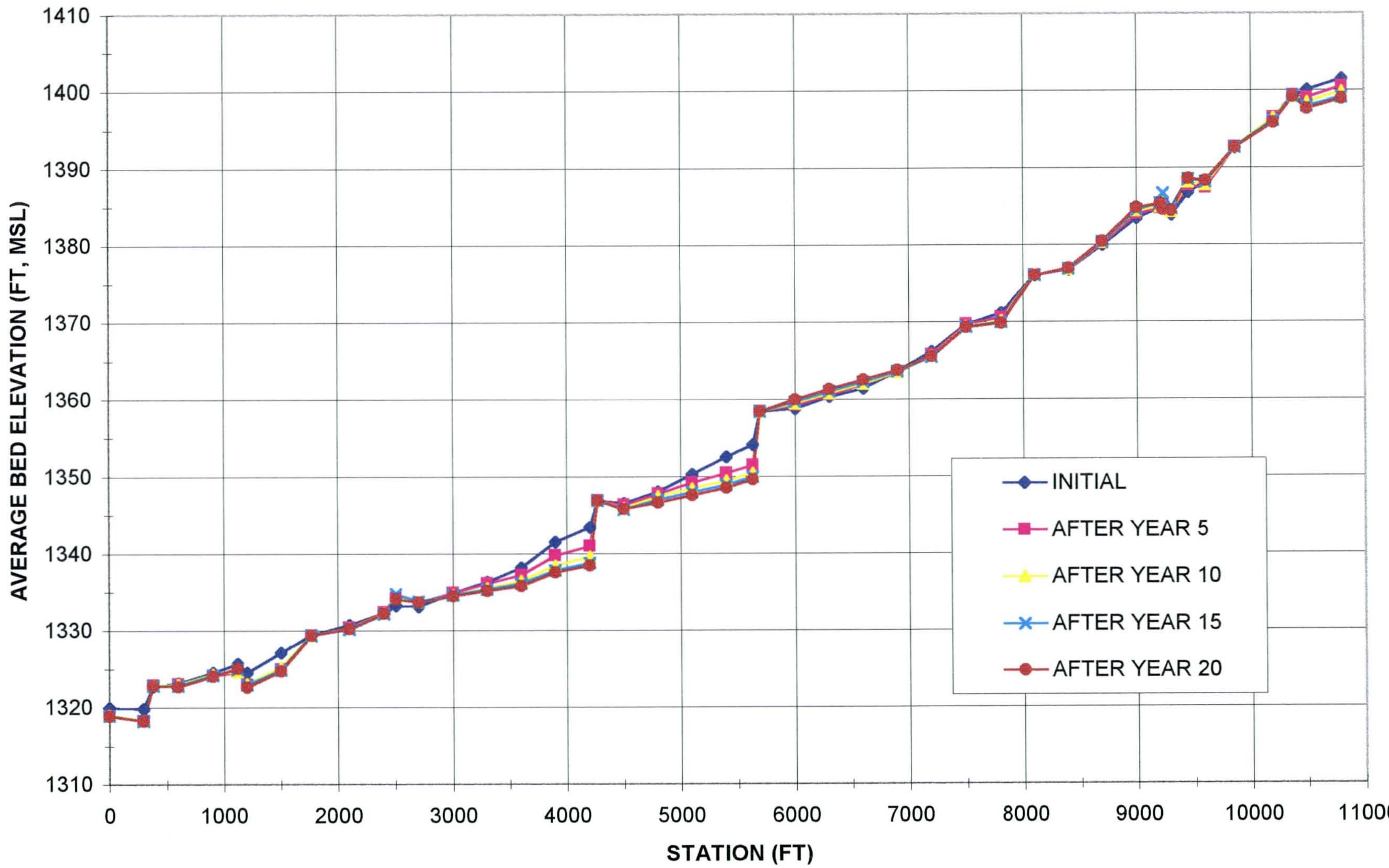
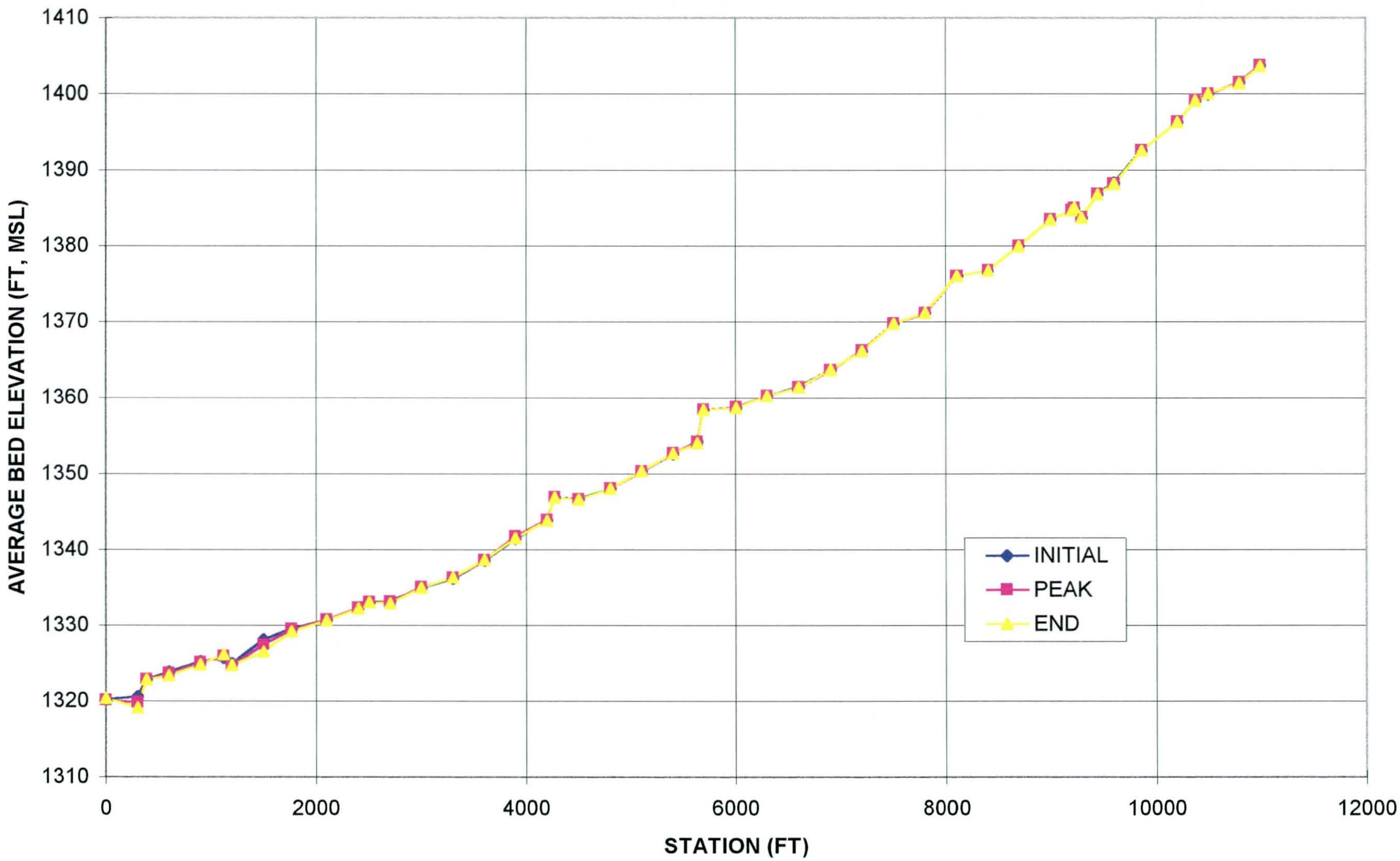


FIGURE 5.6 - CHEROKEE WASH
2-YEAR FLOOD EVENT



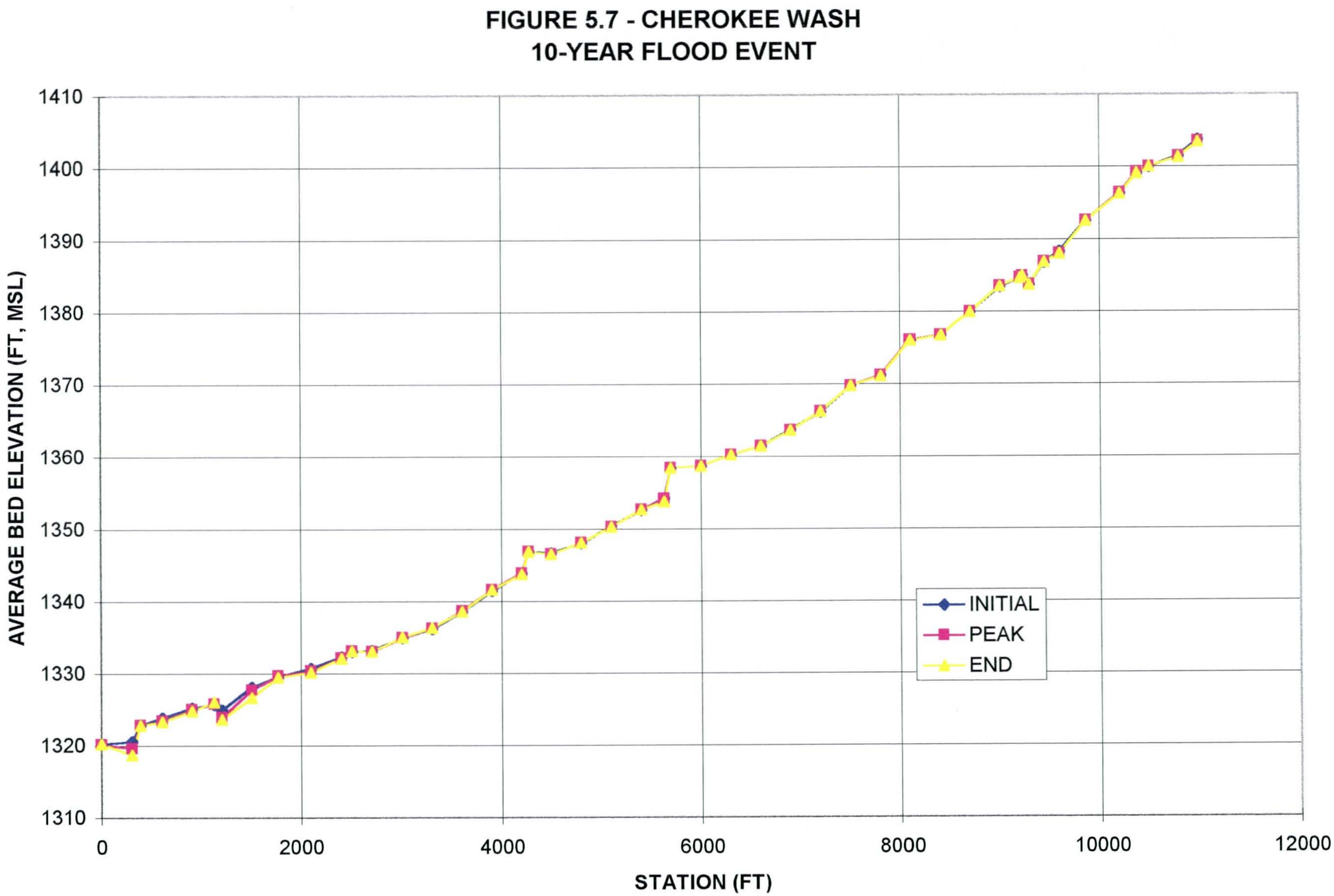


FIGURE 5.7 - CHEROKEE WASH
10-YEAR FLOOD EVENT
AVERAGE BED ELEVATION (FT, MSL)
1410
1400
1390
1380
1370
1360
1350
1340
1330
1320
1310
0
2000
4000
6000
8000
10000
12000
STATION (FT)
INITIAL
PEAK
END

FIGURE 5.8 - CHEROKEE WASH
50-YEAR FLOOD EVENT

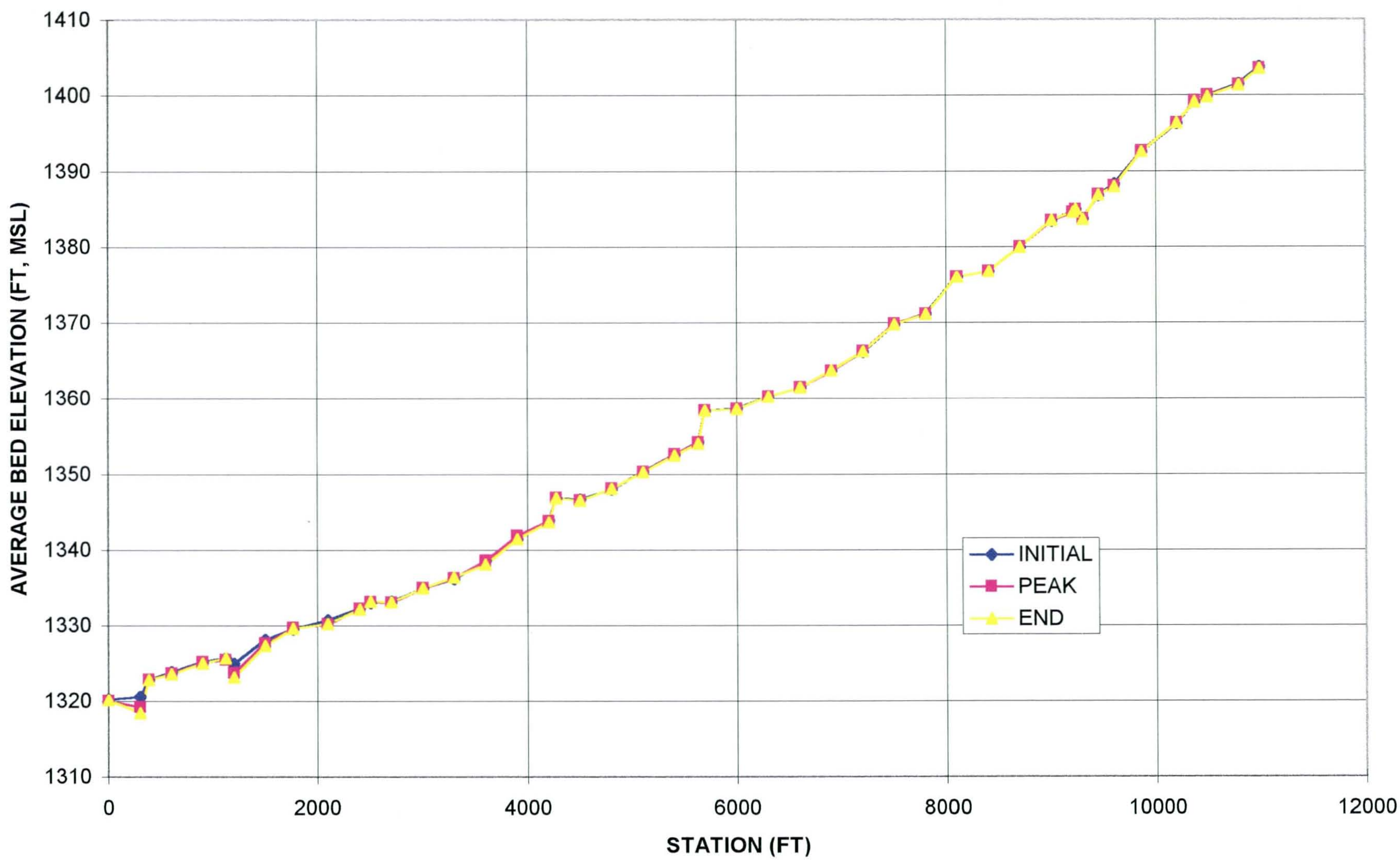


FIGURE 5.9 - CHEROKEE WASH
100-YEAR FLOOD EVENT

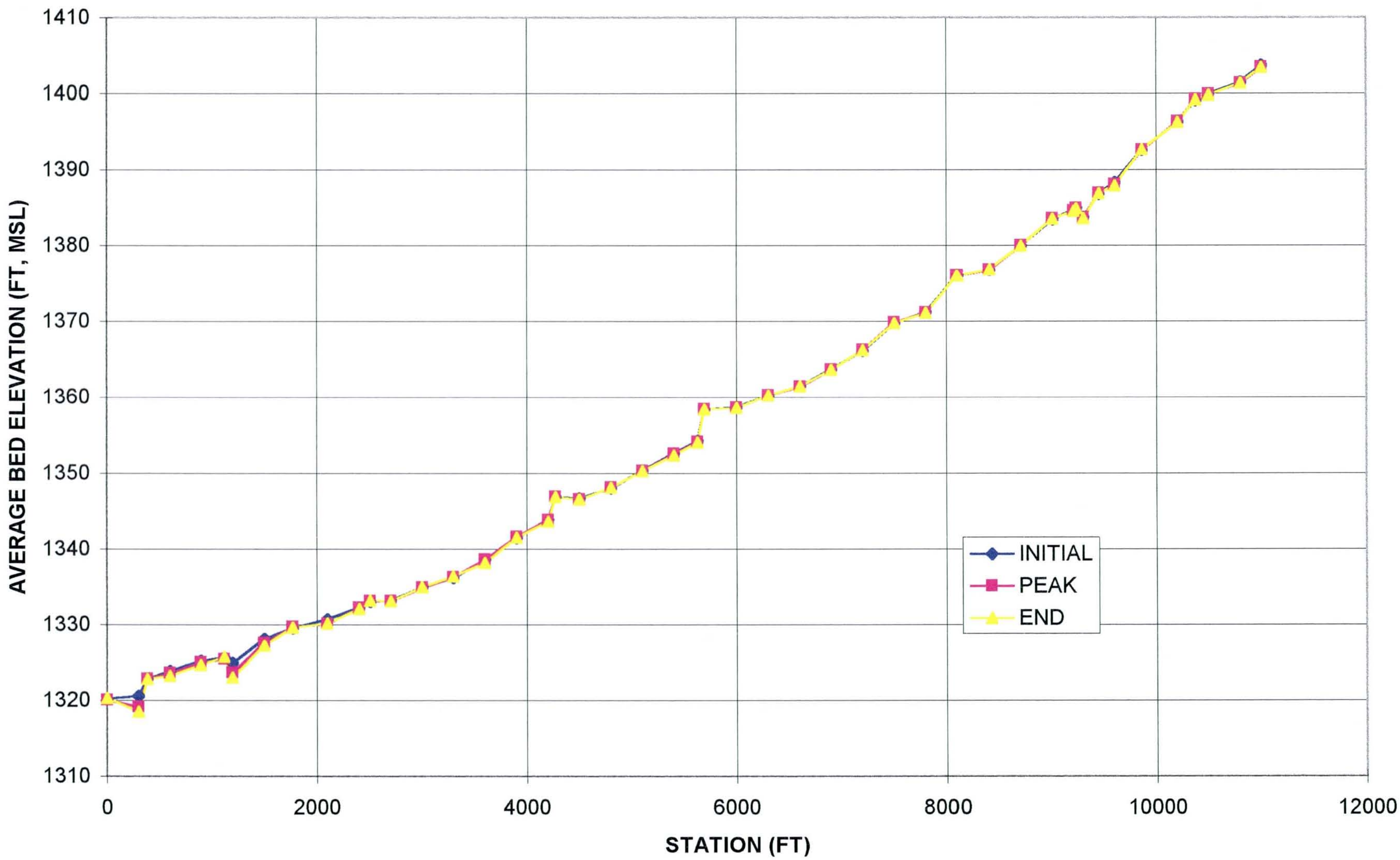
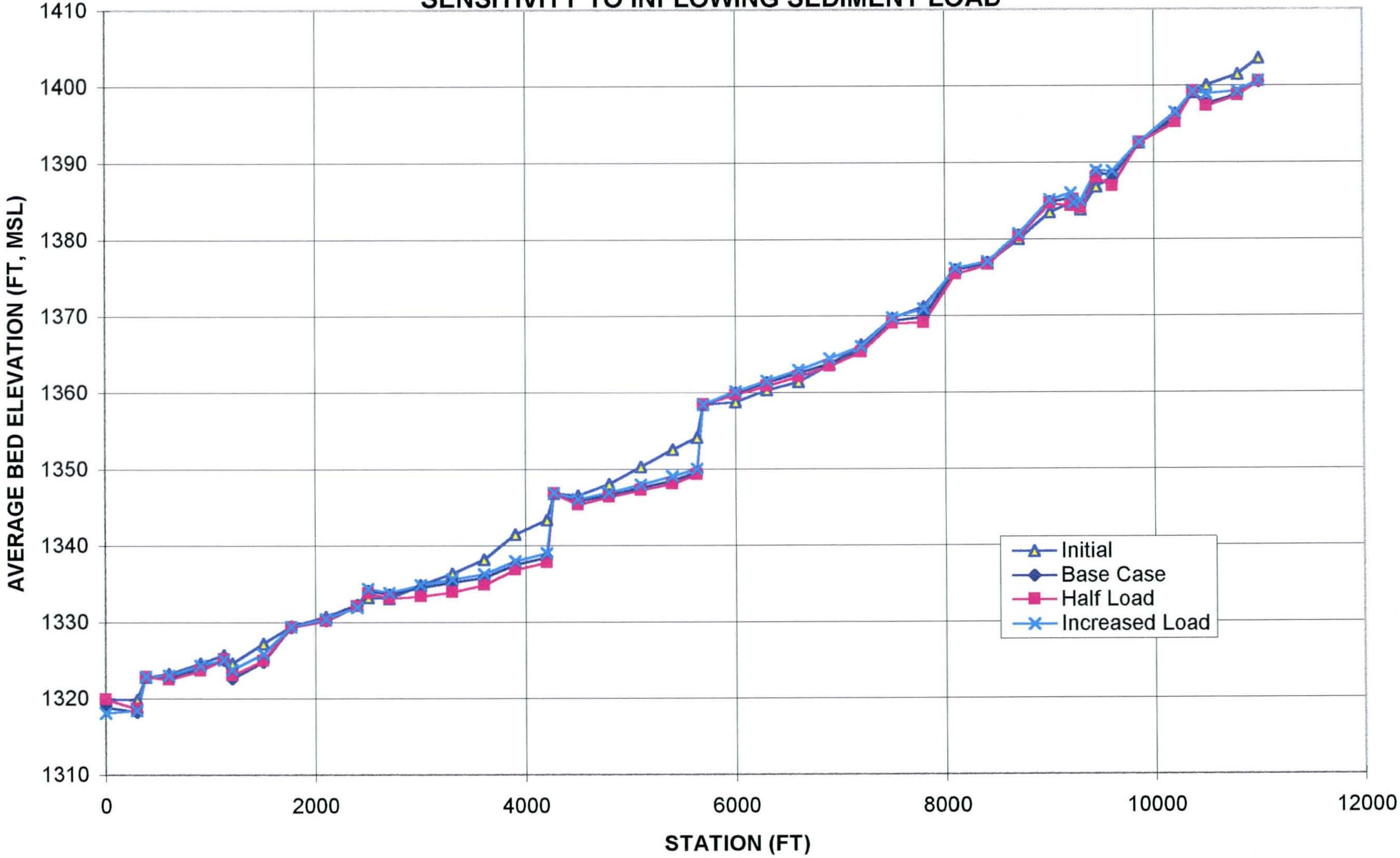


FIGURE 5.10 - CHEROKEE WASH
AVERAGE BED CHANGE AFTER 20 YEARS
SENSITIVITY TO INFLOWING SEDIMENT LOAD



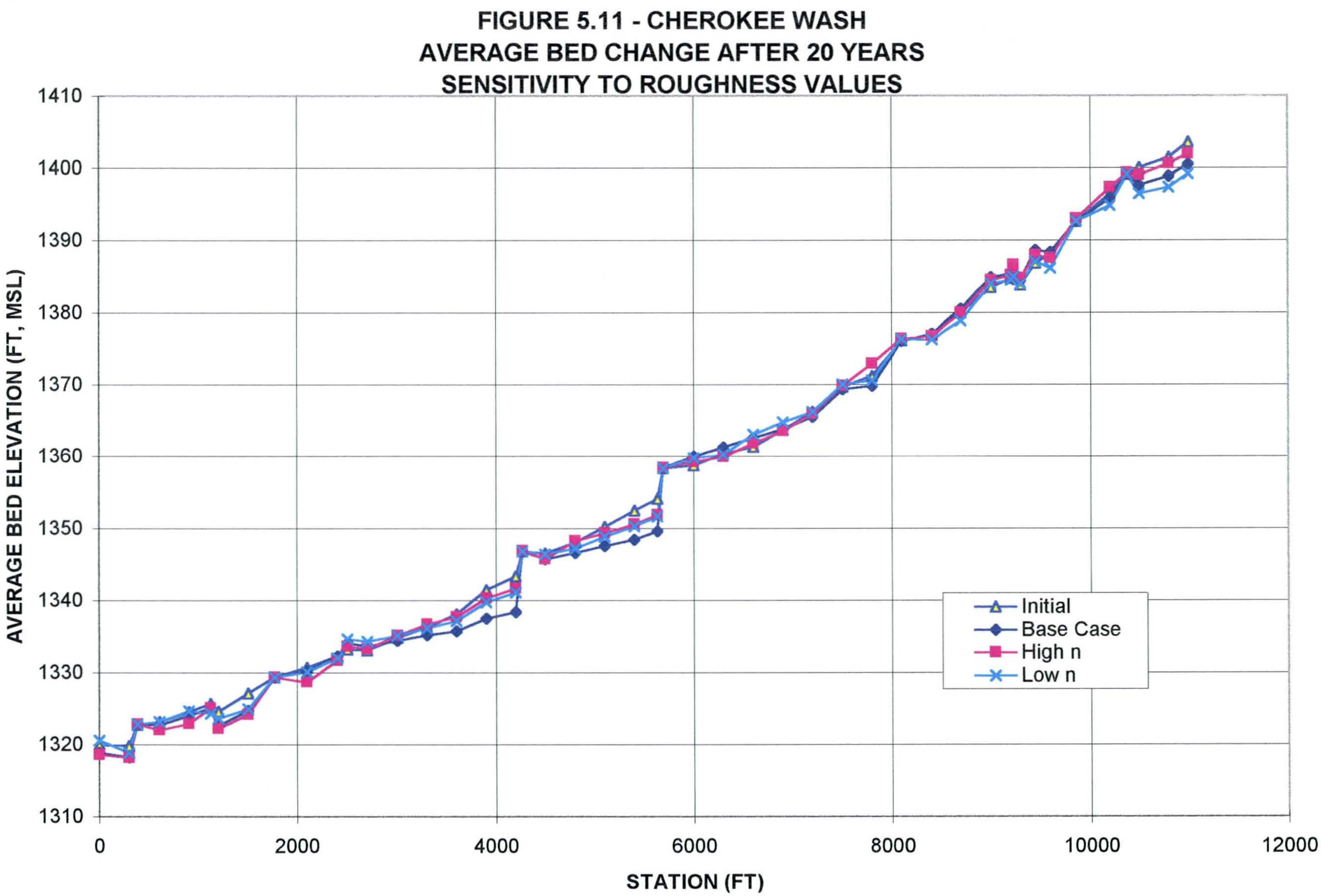


FIGURE 5.11 - CHEROKEE WASH
AVERAGE BED CHANGE AFTER 20 YEARS
SENSITIVITY TO ROUGHNESS VALUES
AVERAGE BED ELEVATION (FT, MSL)
1410
1400
1390
1380
1370
1360
1350
1340
1330
1320
1310
0
2000
4000
6000
8000
10000
12000
STATION (FT)
Initial
Base Case
High n
Low n

FIGURE 5.12 - CHEROKEE WASH
AVERAGE BED CHANGE AFTER 20 YEARS BY
TWO DIFFERENT TRANSPORT FUNCTIONS

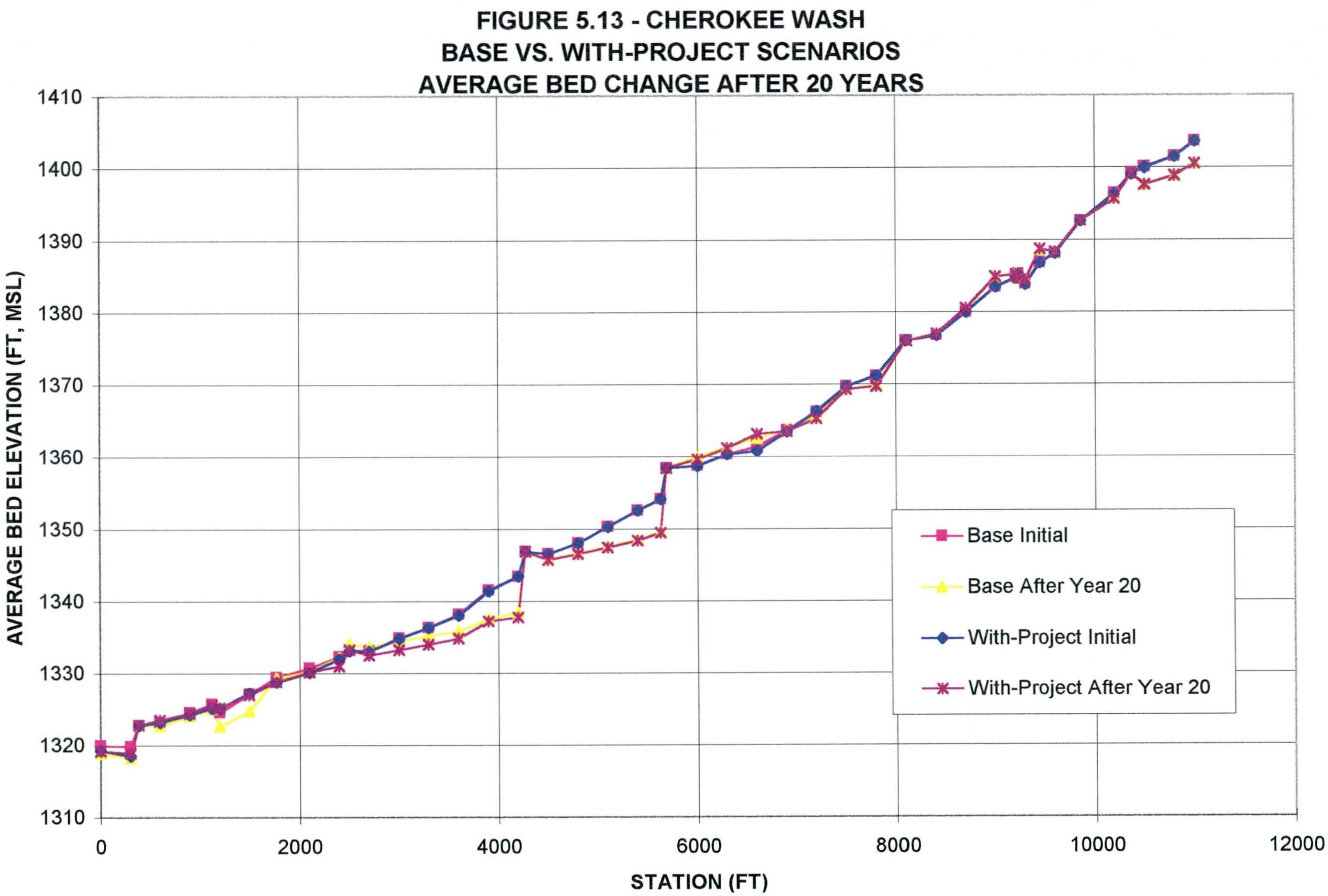


FIGURE 5.13 - CHEROKEE WASH
BASE VS. WITH-PROJECT SCENARIOS
AVERAGE BED CHANGE AFTER 20 YEARS
AVERAGE BED ELEVATION (FT, MSL)
1410
1400
1390
1380
1370
1360
1350
1340
1330
1320
1310
0
2000
4000
6000
8000
10000
12000
STATION (FT)
Base Initial
Base After Year 20
With-Project Initial
With-Project After Year 20

6.0 CHANNEL AND BASIN DESIGN RECOMMENDATIONS

6.1 Introduction

Based on the hydraulic, geomorphic, and sediment continuity analyses previously described, WEST assessed possible improvements to Cherokee Wash. The improvements considered include channel cross section modification, grade control, scour protection, and sedimentation basins and are described in the following sections. Implementation of any improvements will, of course, depend on political and economic factors which are outside the scope of this study.

6.2 Channel Modifications

Based on existing conditions hydraulic analyses, locations of reduced conveyance were located along the wash (see Section 3 - Hydraulics). As previously described, for the with-project scenario certain sections of the channel were widened to increase conveyance. Because the maximum top width was limited by a reasonable right-of-way estimate, only minor gains in conveyance were obtained. However, increasing the channel width in the indicated sections should improve the uniformity of conveyance along the channel and contain the 2-year flow event. As discussed in Chapter 3, more extensive (and expensive) channel modifications will not necessarily convey the 100-year flood discharge unless a supercritical flow channel is designed. The described with-project channel modifications are recommended as a way to increase conveyance to pass the most frequent events while limiting construction costs.

Installation of culverts to replace dip crossings is desirable from a public safety point of view as described in Chapter 3 but may locally reduce channel capacity. Installation of culverts at 56th Street is recommended such that culvert capacity will at least match channel capacity both up- and downstream of the crossing.

6.3 Grade Control and Scour Protection

These two items are joined in one section because they are both connected to hydraulic processes at the road crossings. The road crossings act essentially as grade control. However, as reported in Section 5, degradation downstream of the crossings may eventually threaten the integrity of the roads themselves. This lowering downstream of crossings can already be observed just downstream of Mockingbird Lane. Either low grade control structures and/or scour protection are advisable downstream of the crossings. Table 6.1 summarizes scour at key areas on Cherokee Wash (numbers in parentheses indicated deposition). Note that these results represent long-term degradation only and do not include any local scour due to concentration of flow or jets downstream of any drops. Minor degradation anticipated downstream of 58th Place may not warrant any protection at all.

Table 6.1- Average Bed Scour (in feet) at Key Areas on Cherokee Wash

Cross Section	2400	4200	5630	7800	10999
Area / Model:	Downstream of 58th Pl.	Downstream of 56th St.	Downstream of Mockingbird Ln	Downstream of W. Branch Confluence	Downstream of Roadrunner Road
Base	0.0	5.0	4.5	1.3	3.0
With-Project	0.9	5.6	4.7	1.5	3.1
Ackers-White	0.4	5.0	5.4	0.1	2.7
High n	1.0	2.6	2.4	(1.6)	1.9
Low n	0.3	2.3	2.3	0.7	4.1
Half Load	0.2	5.5	4.8	2.1	3.1
Increased Load	0.3	4.3	4.0	0.3	3.0

Depending on the type of drop structure selected, grade control downstream of 56th Street and Mockingbird Lane may best be handled by two separate small drops instead of one large drop. Two smaller structures are, however, usually more costly than one large one. Scour downstream of where the West Branch enters Cherokee Wash may be a concern as no road crossings are involved. Scour downstream of Road Runner Road could probably be controlled by a single drop.

6.3.1 Types of Structures

Given the urban setting of Cherokee Wash, vertical wall drop structures greater than 2 feet high are not recommended because of safety and liability concerns. Higher drops may be accomplished using sloping structures or stair-stepped structures. Examples of two types of structures used with drops greater than 3 feet in urban settings are shown in Figures 6.1 and 6.2.

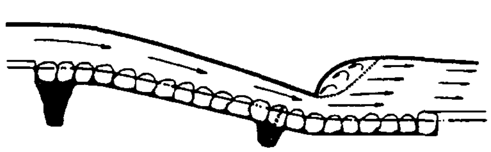


Figure 6.1 - Grouted Sloping Boulder Drop (after UDFCD, 1990)

6.3.2 Spacing of Structures

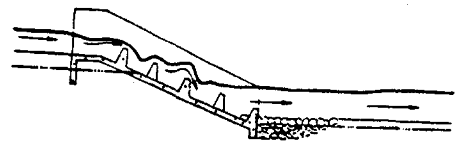


Figure 6.2 - Baffle Chute Drop (after UDFCD, 1990)

More than one grade control structure may be preferred to control long term degradation downstream of 56th Street and Mockingbird Lane. It should be noted however, that the drop at Mockingbird Lane already is about four feet. If, however, multiple drops are desired they must be spaced such that the equilibrium slope between the drops is maintained. Assuming a nominal drop of six feet for these crossings and the equilibrium slopes observed from HEC-6 results (given in Table 5.4), the number of drop structures needed is shown in Table 6.2.

Table 6.2 - Spacing and Number of Drop Structures

Maximum Drop Height (ft)	58th Place To 56th Street		56th St. to Mockingbird Lane	
	Number	Spacing (ft)	Number	Spacing (ft)
2	3	570	3	600
4	2	1140	2	1200
6	1	N/A	1	N/A

6.3.3 Scour Protection at Crossings

The maximum velocities expected at the road crossings during the 100-year event do not exceed 6 ft/s from HEC-2 model results. For these velocities, minimal stone, if any, is needed. A median stone size (D_{50}) of 1 foot would be sufficient protection for these velocities. Rock used downstream of drop structures should be larger; the size will depend on the type of structure and associated energy dissipation.

6.4 Sedimentation Basins

Because Cherokee Wash appears to be largely degradational, no sedimentation basins are recommended.

6.5 Project Operation and Maintenance

Because the Wash appears to be degrading in the majority of reaches studied, no sedimentation basins were recommended and thus have no maintenance requirements. Stone protection at the road crossings/stilling basins should be monitored and repaired, if necessary, after major events. Vegetation in the channel has offsetting effects as shown in the HEC-6 sensitivity analyses. Maintained vegetation will increase channel capacity but could also increase scour compared to an unmaintained conditions. If grade control is implemented on Cherokee Wash, in-channel vegetation maintenance is recommended to achieve higher levels of channel conveyance. Grade control structures should be founded a sufficient depth below the channel to account for possible increases in channel degradation resulting from the channel maintenance.

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APPENDIX A

CROSS SECTION PLOTS

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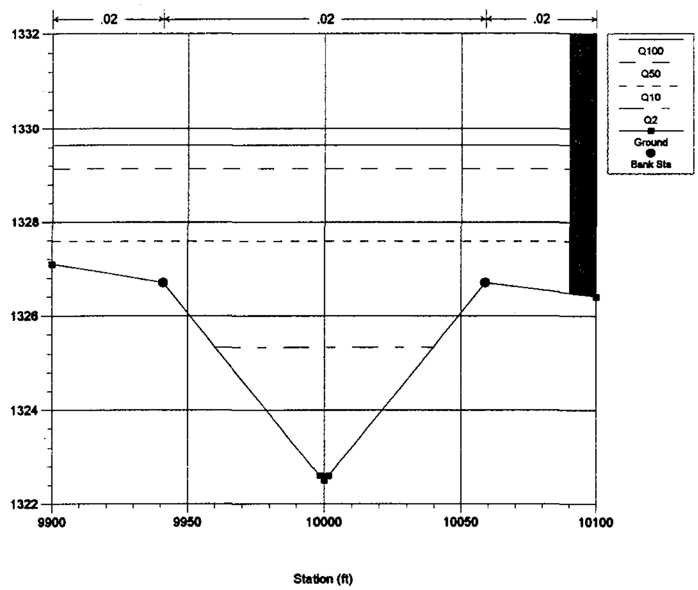

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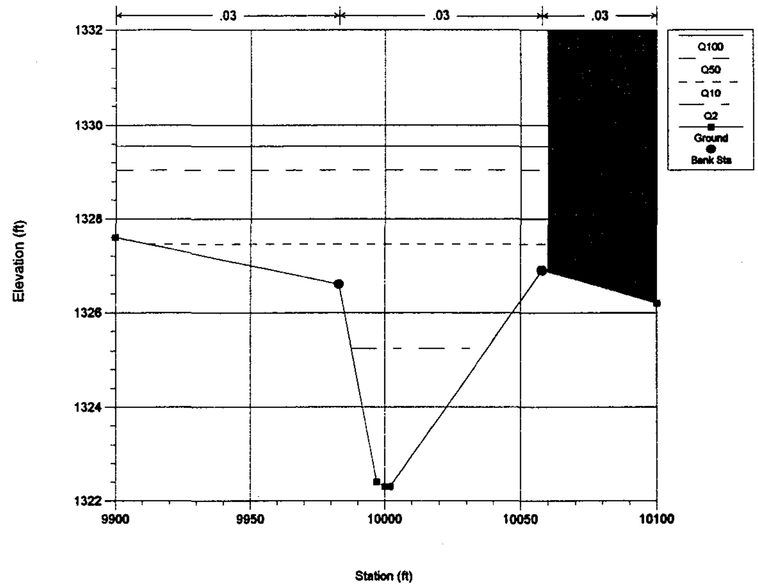

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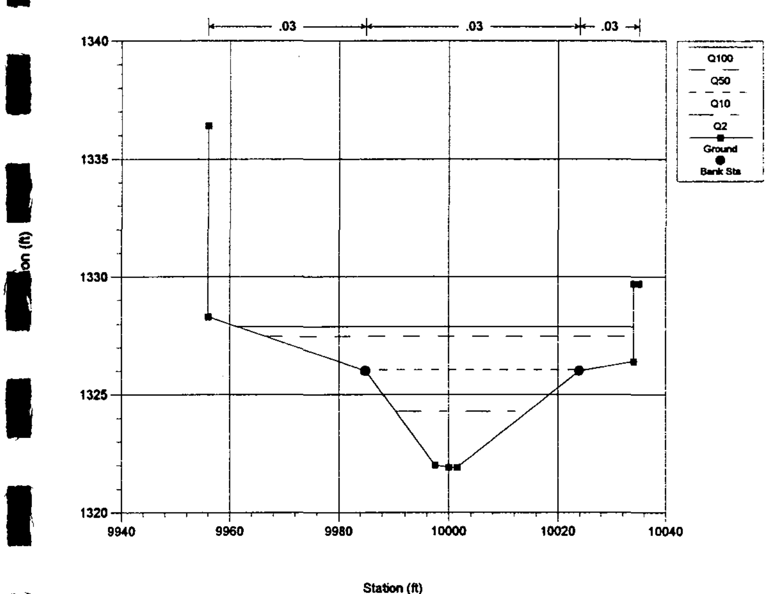

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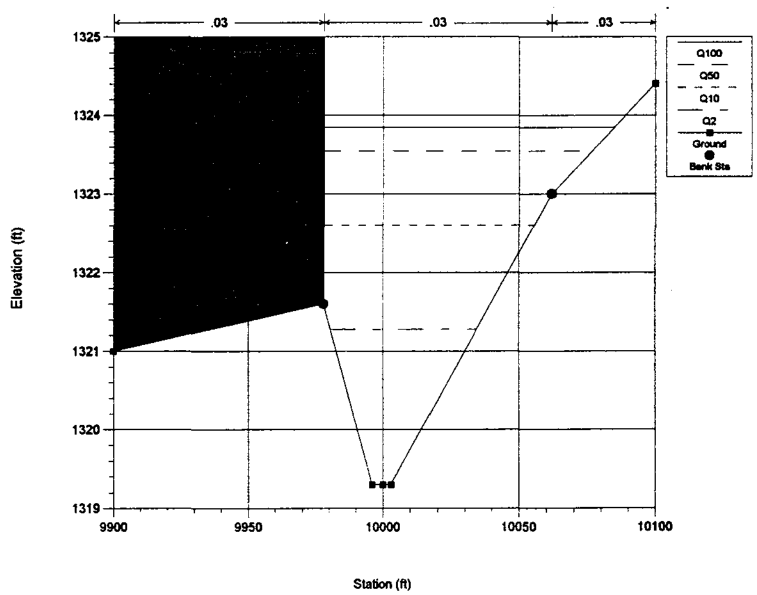

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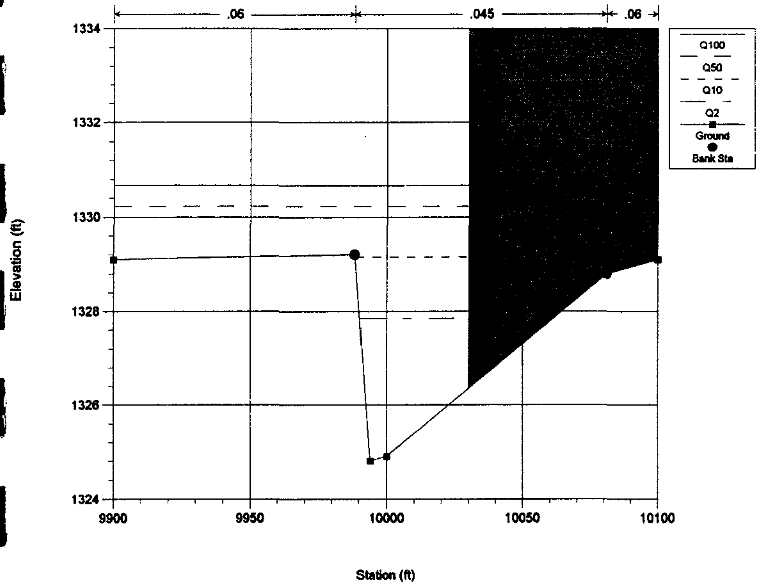

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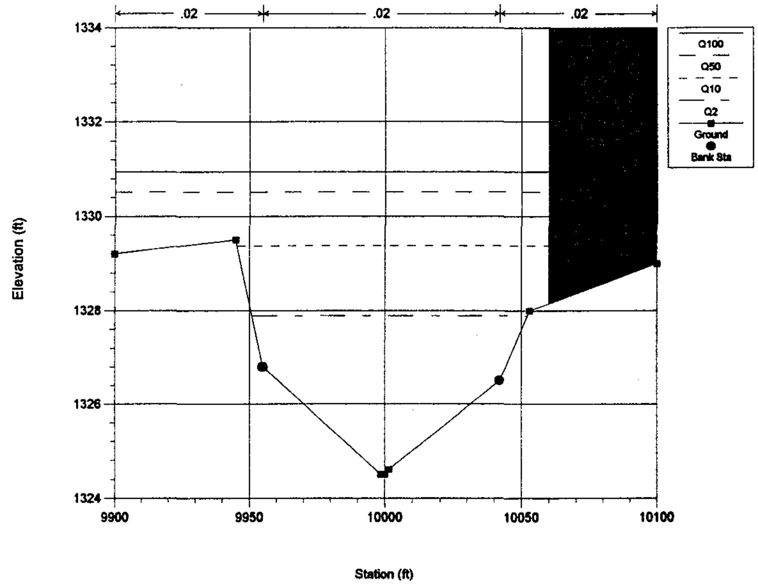

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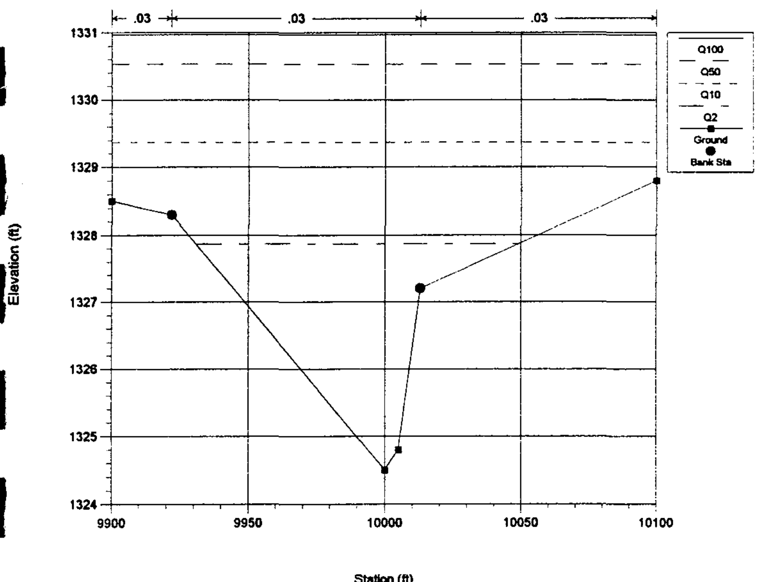

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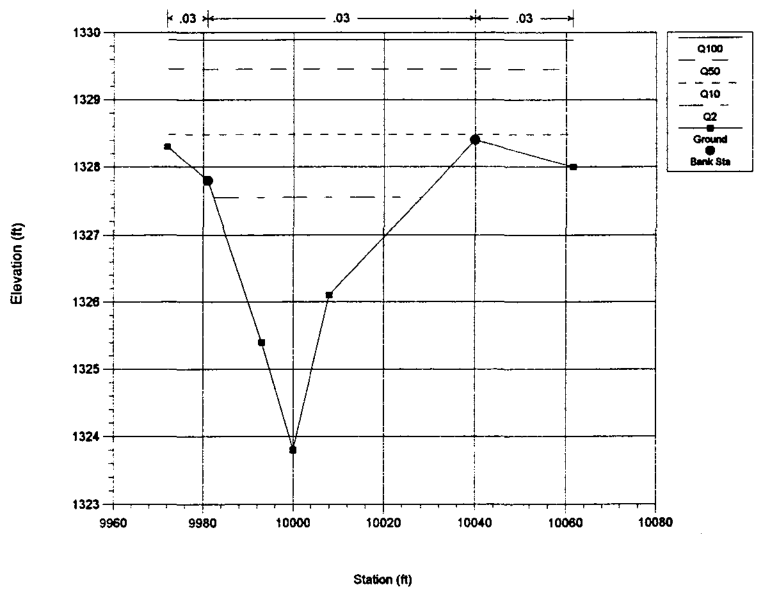

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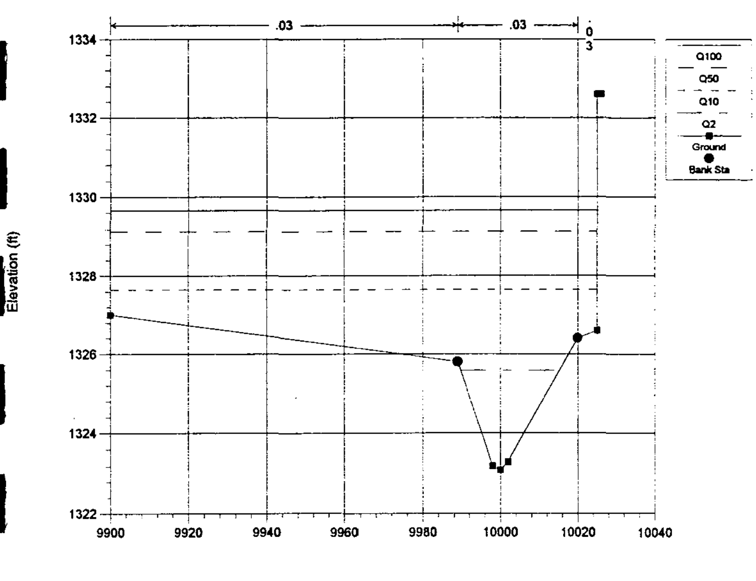

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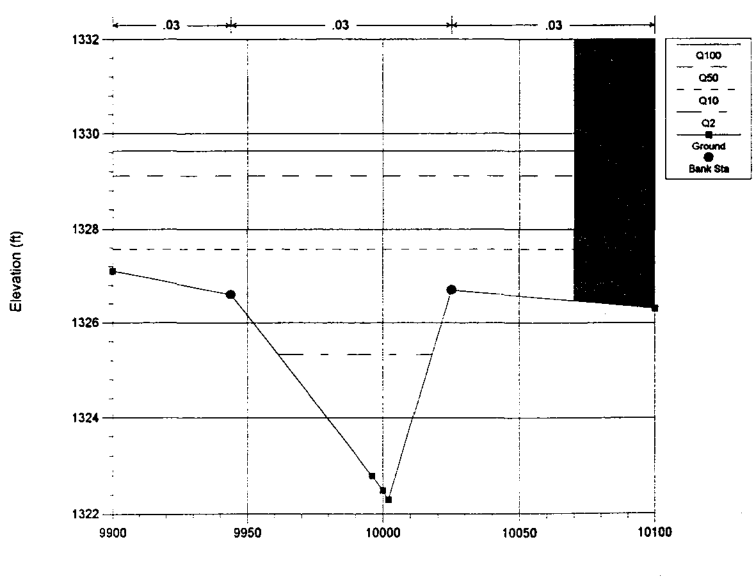

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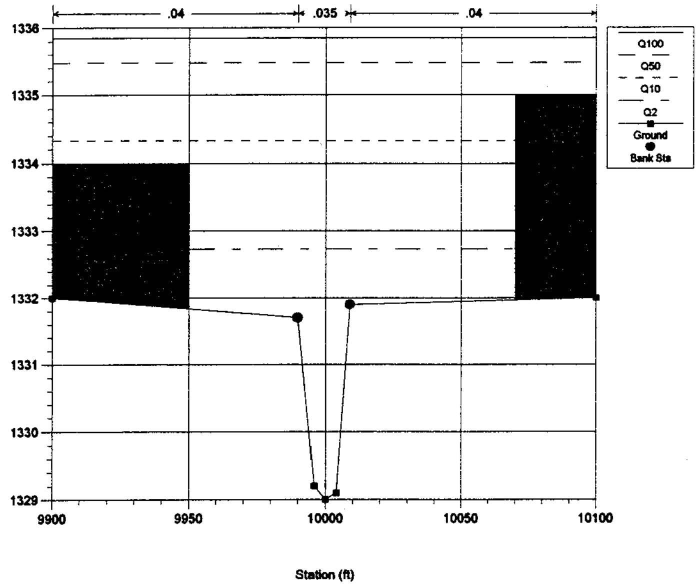

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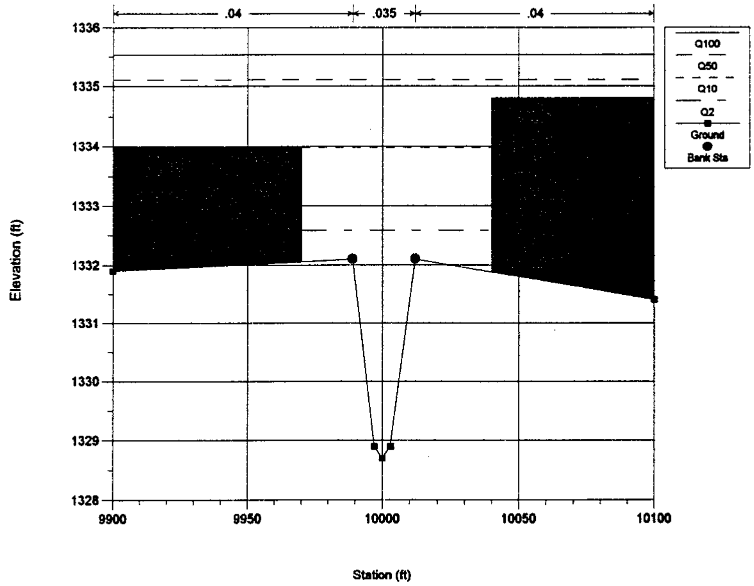

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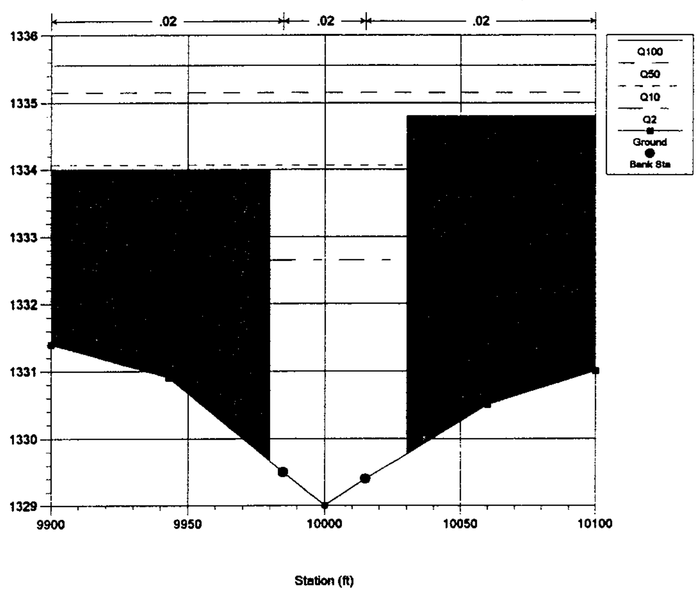

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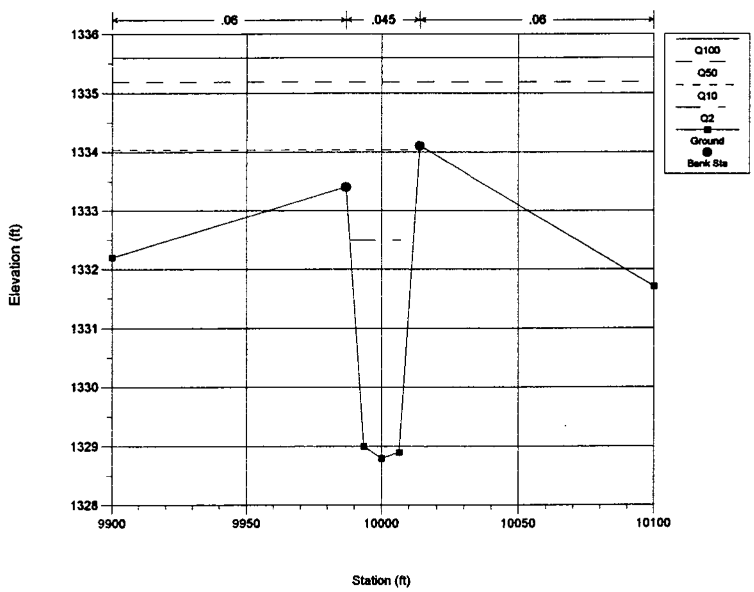

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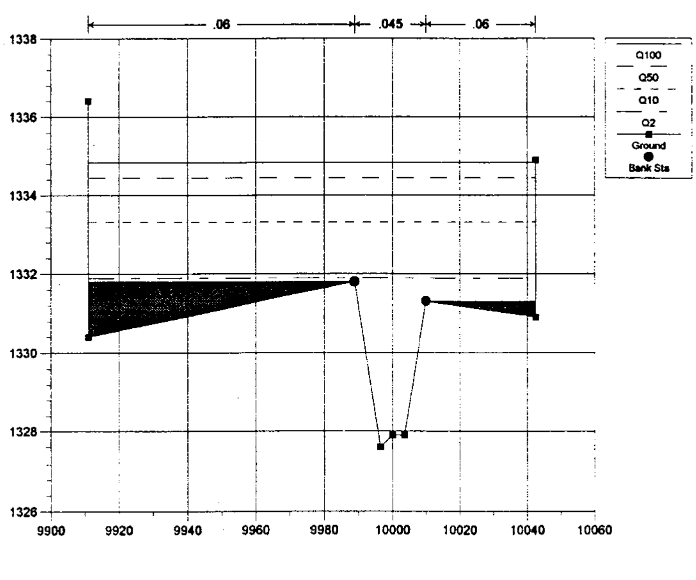

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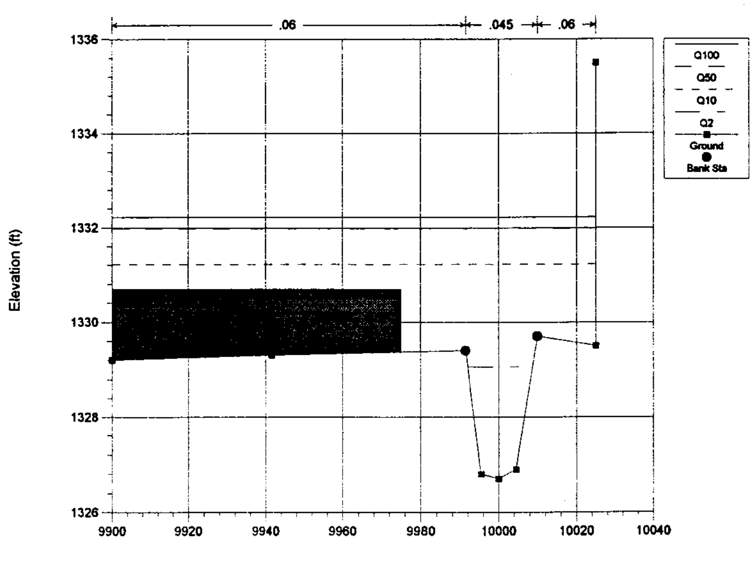

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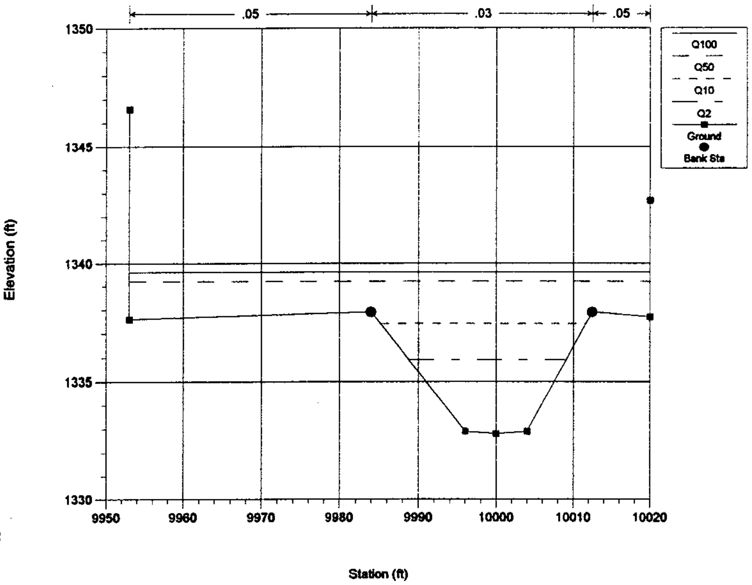

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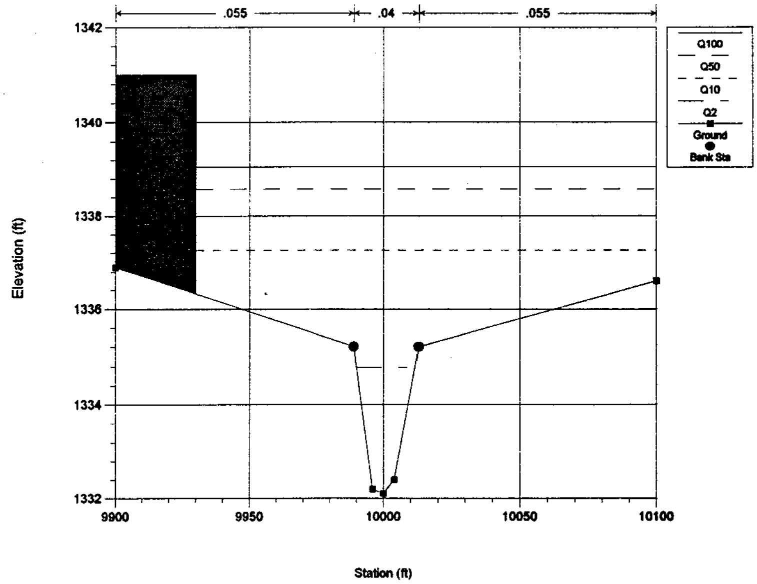

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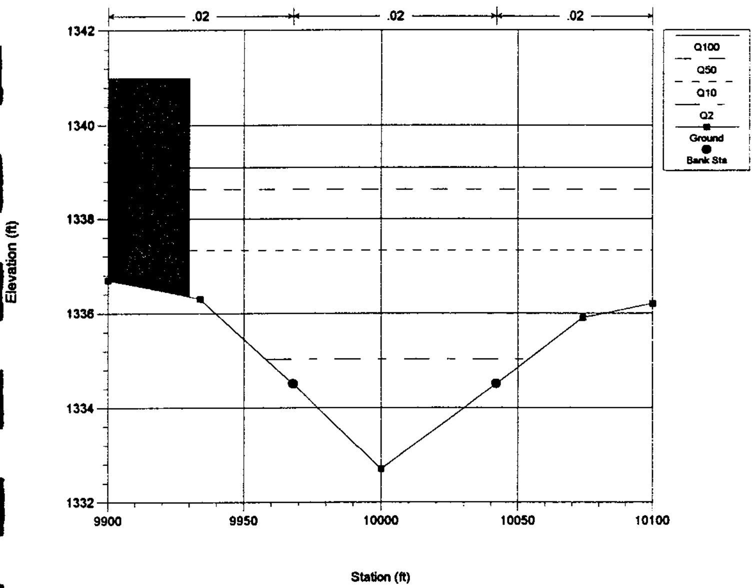

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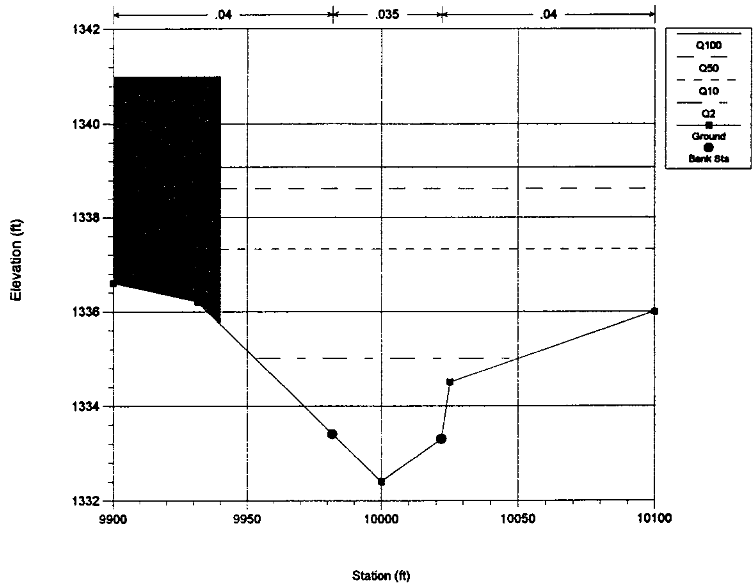

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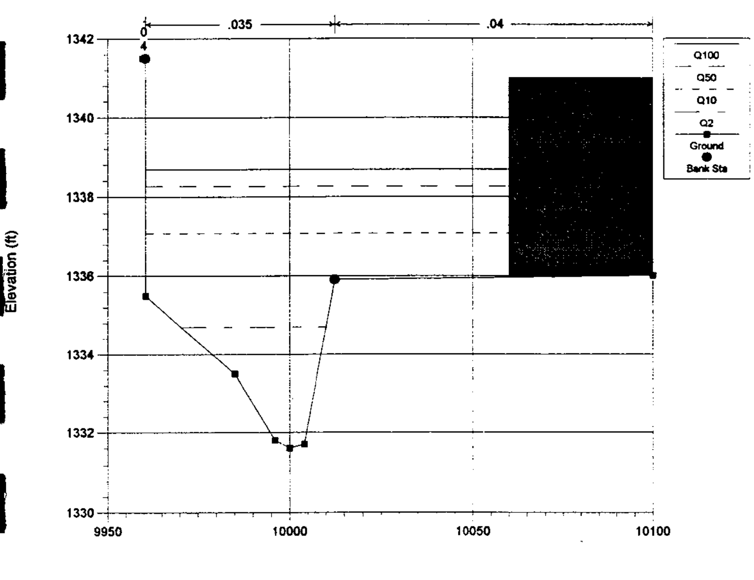

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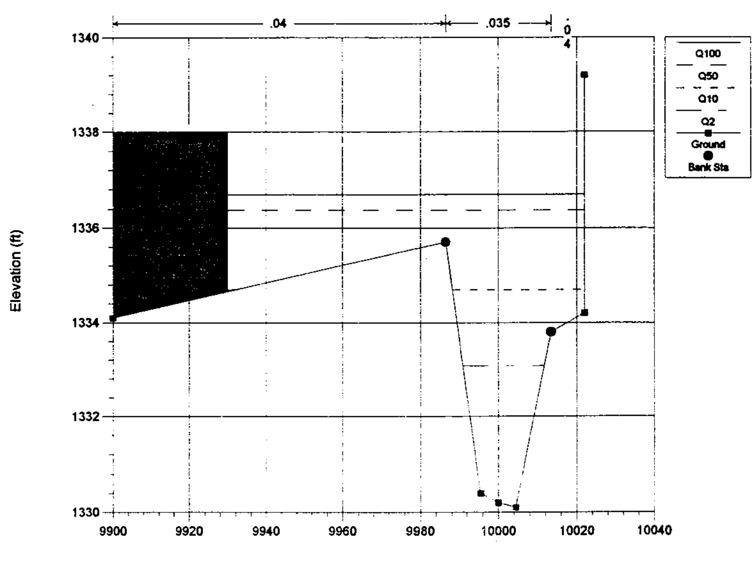

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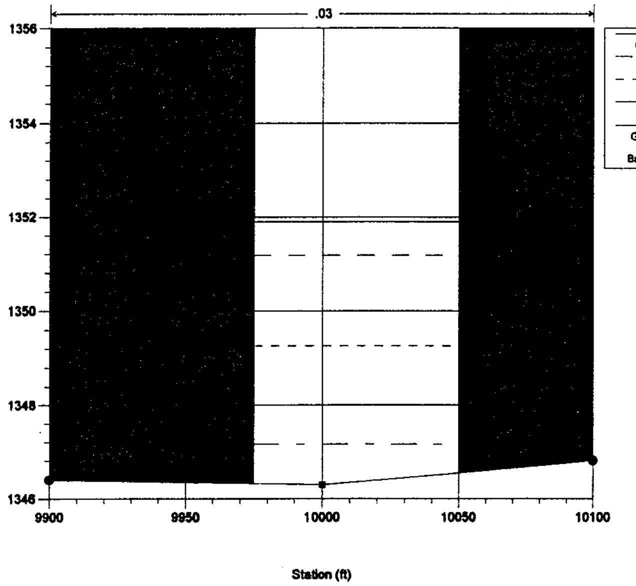

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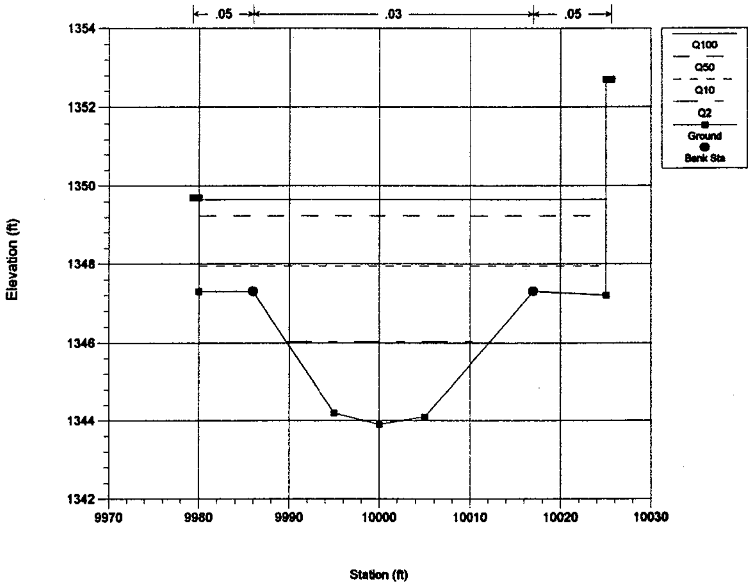

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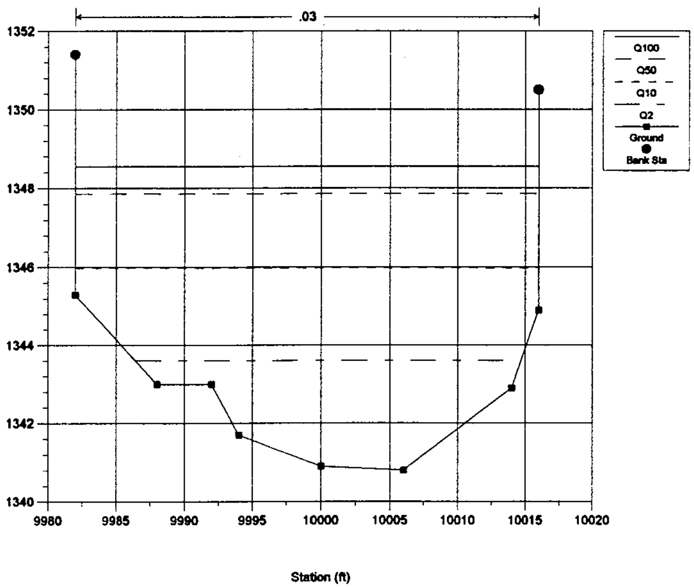

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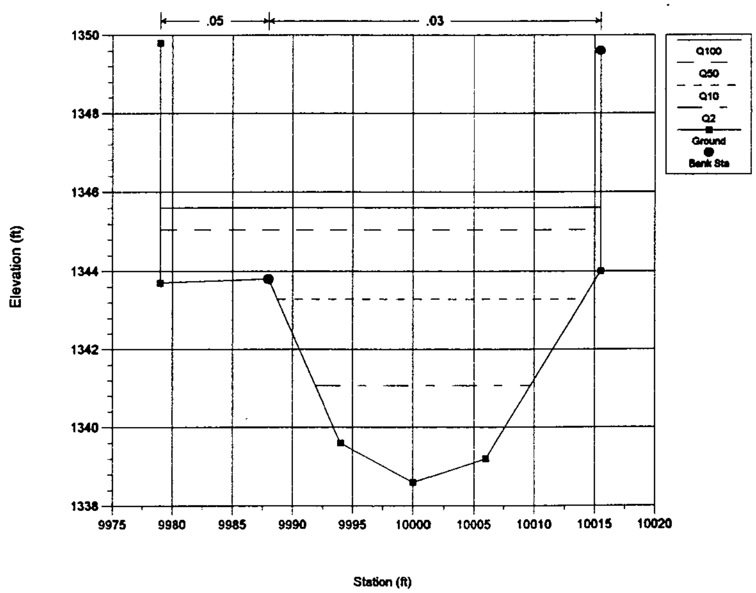

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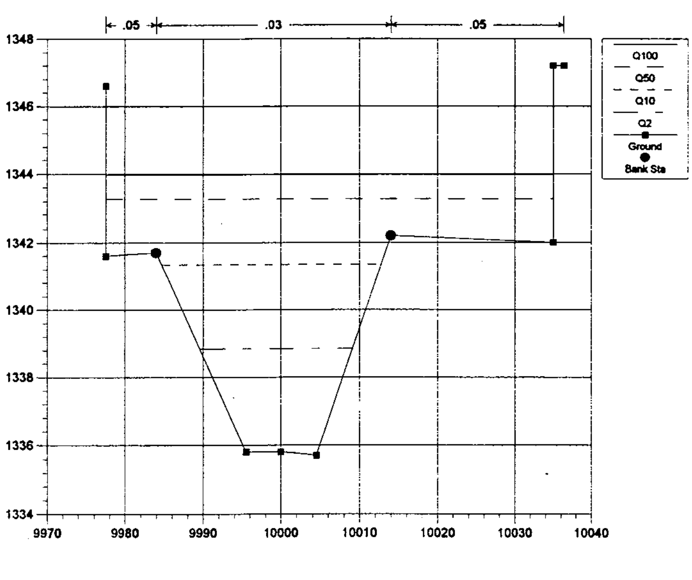

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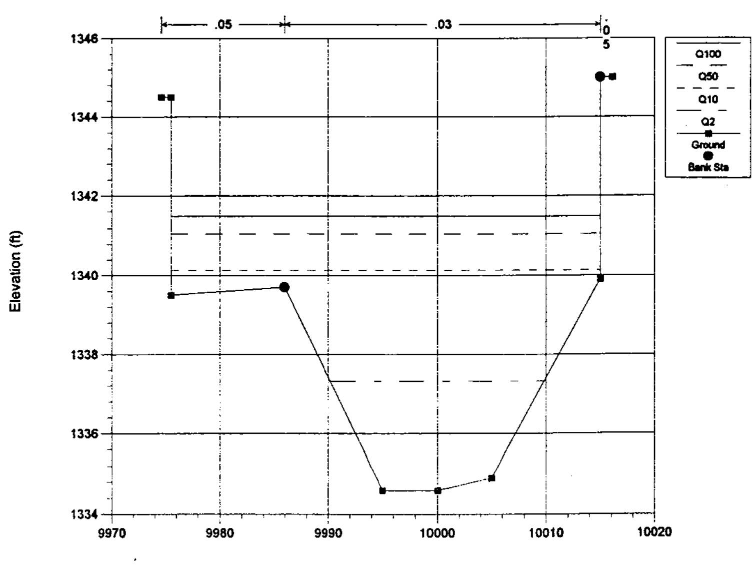

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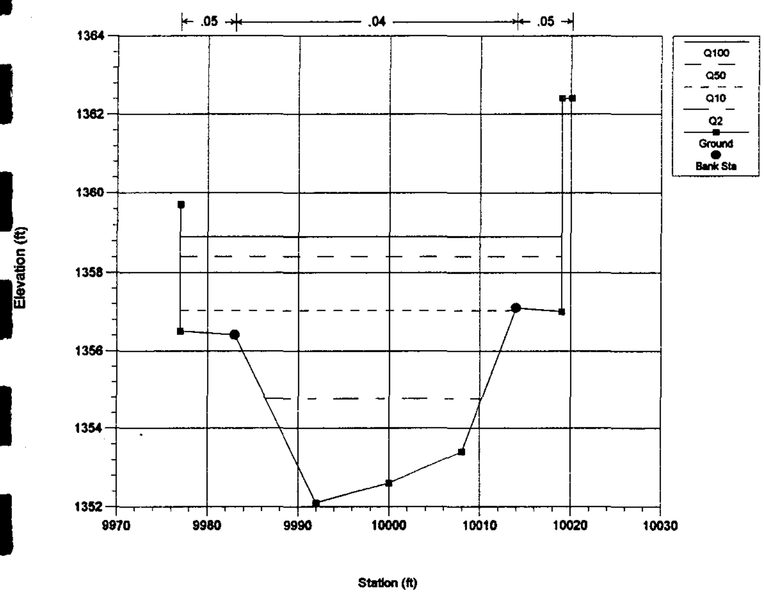

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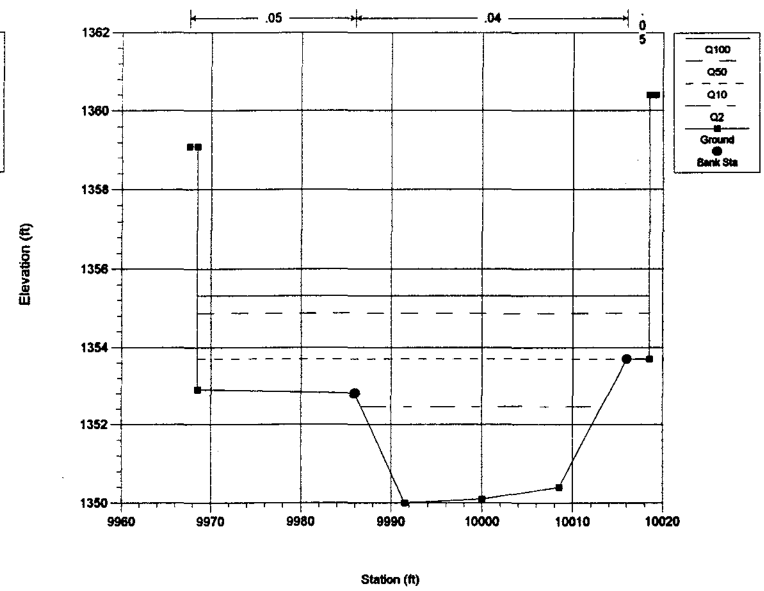

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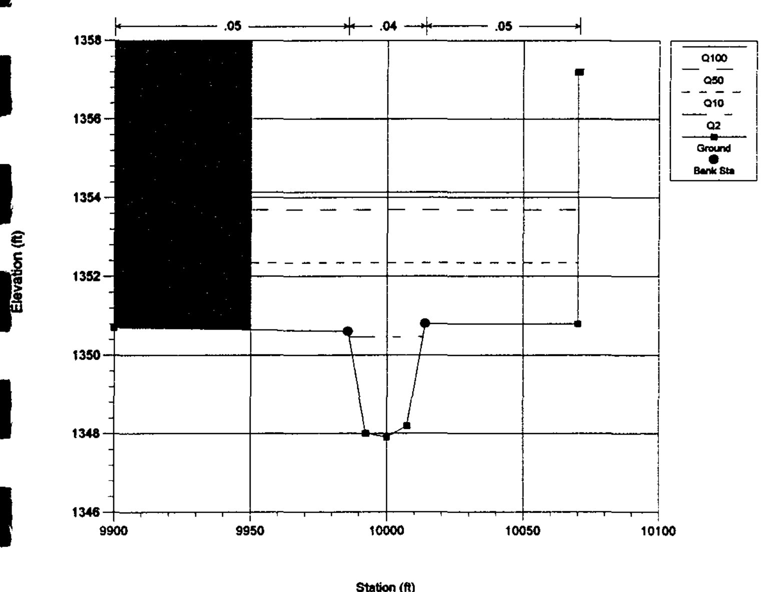

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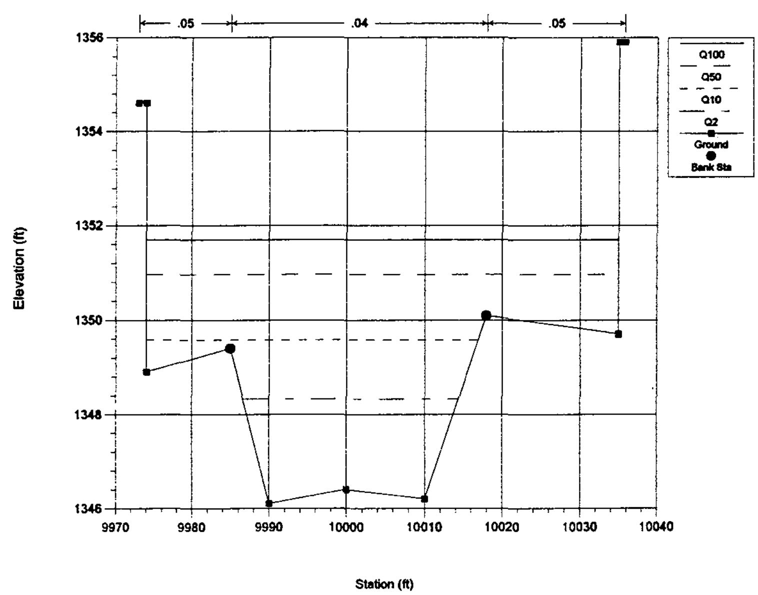

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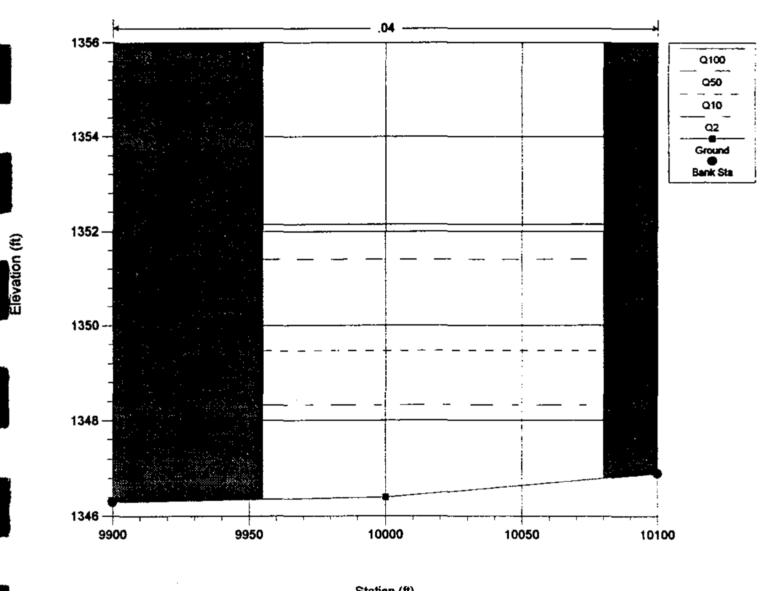

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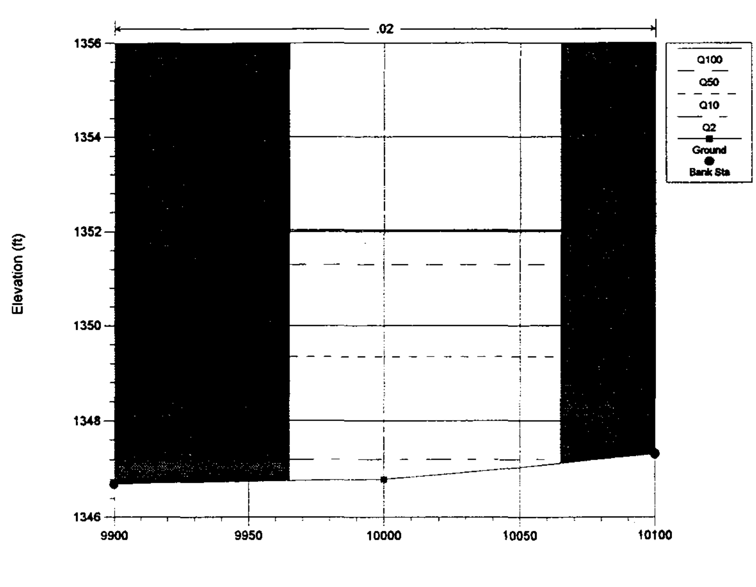

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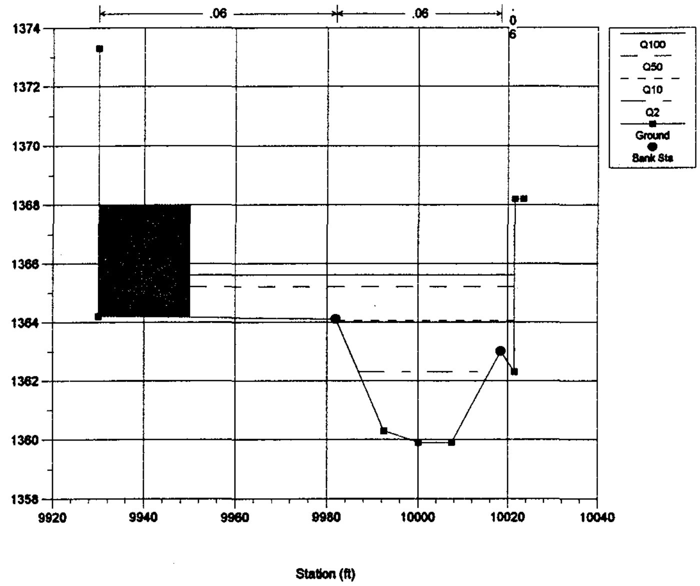

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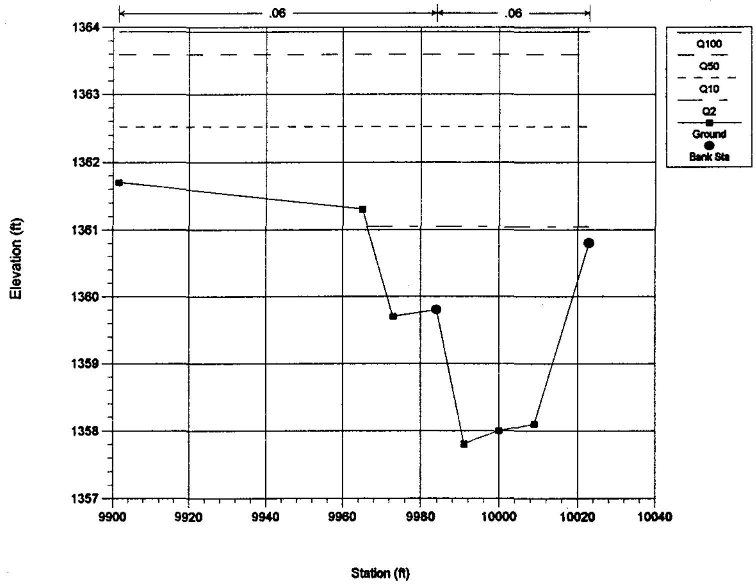

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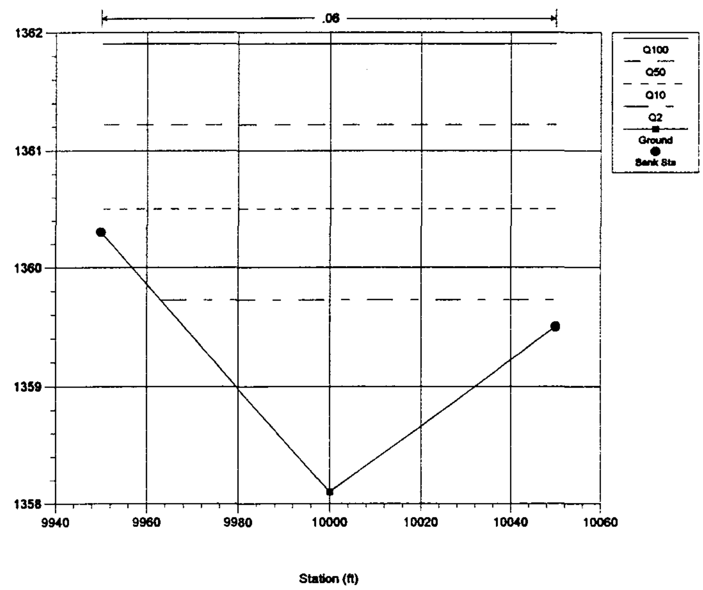

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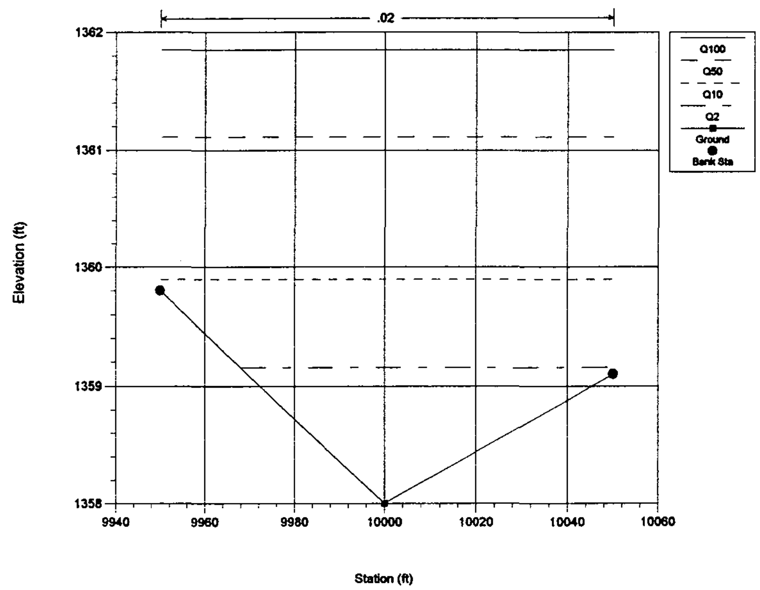

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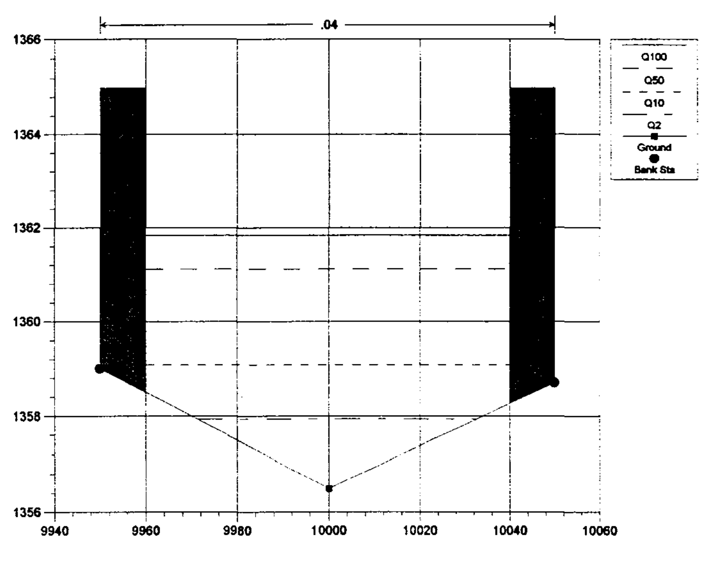

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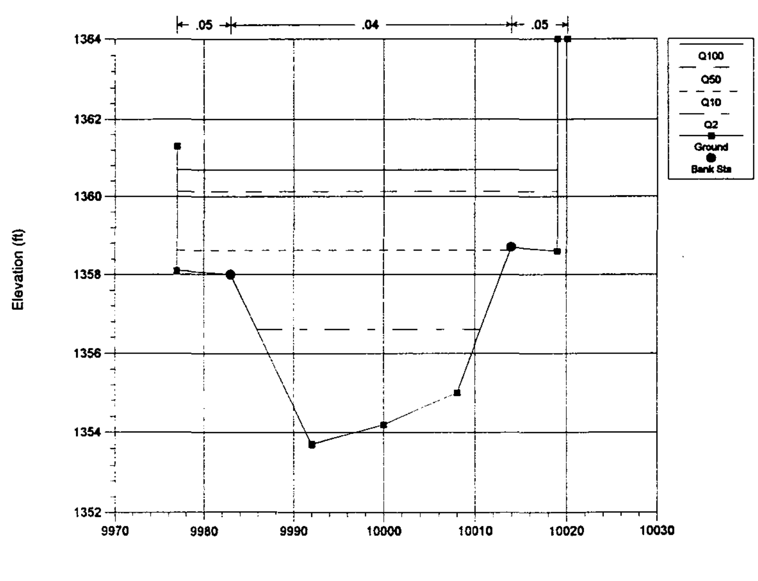

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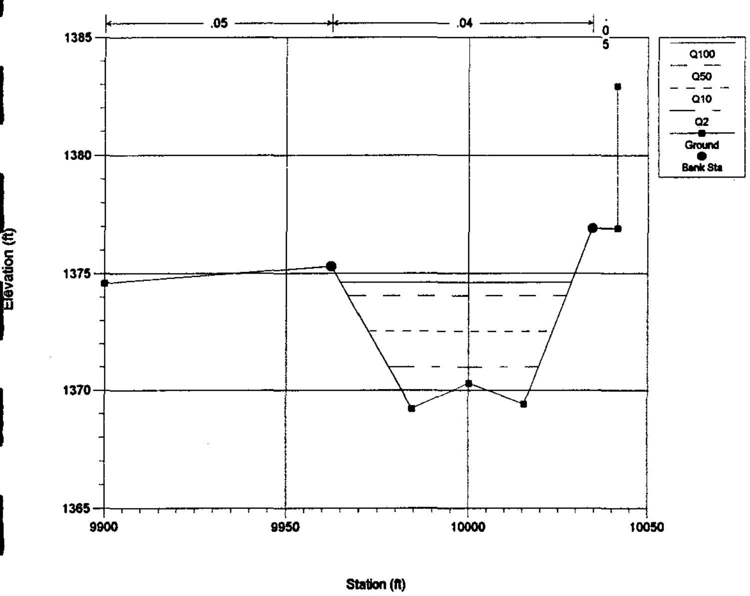
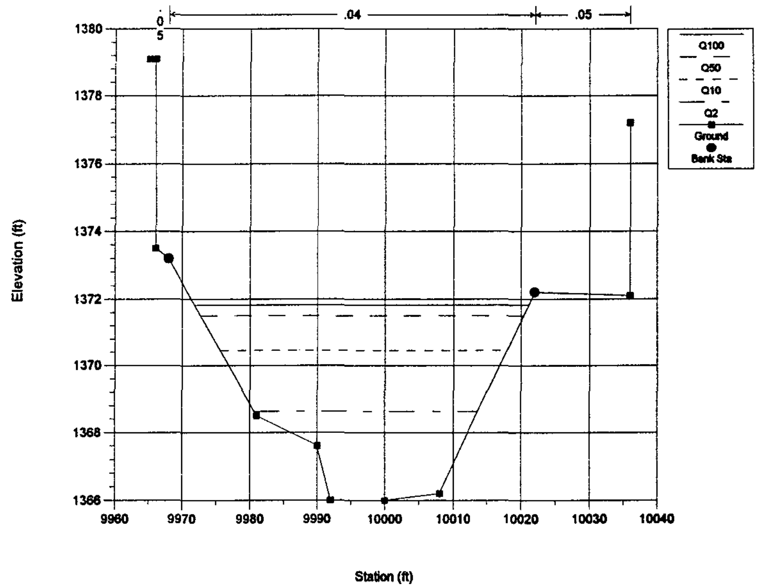
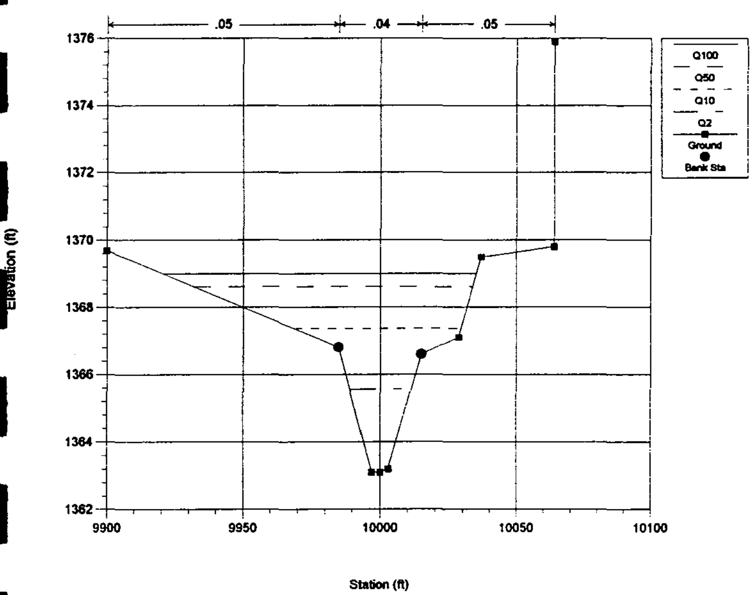


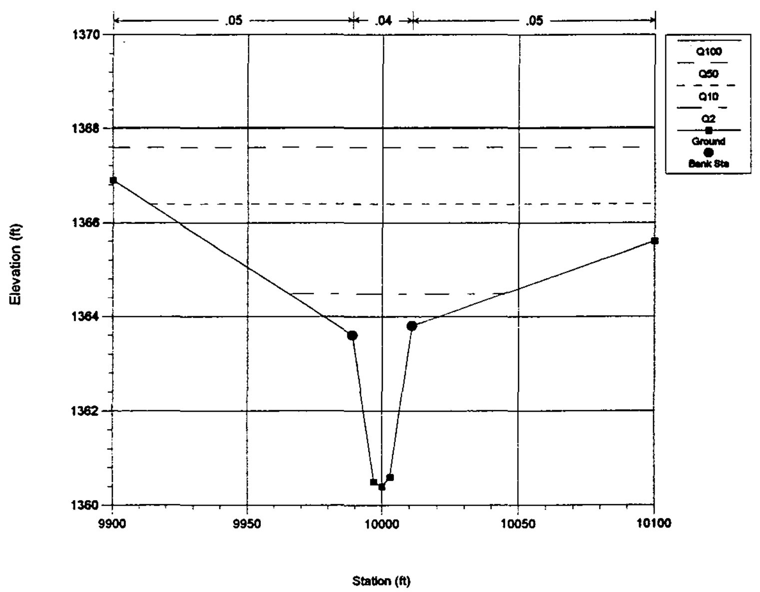
Riv Sta = 7200 CHEROKEE WASH



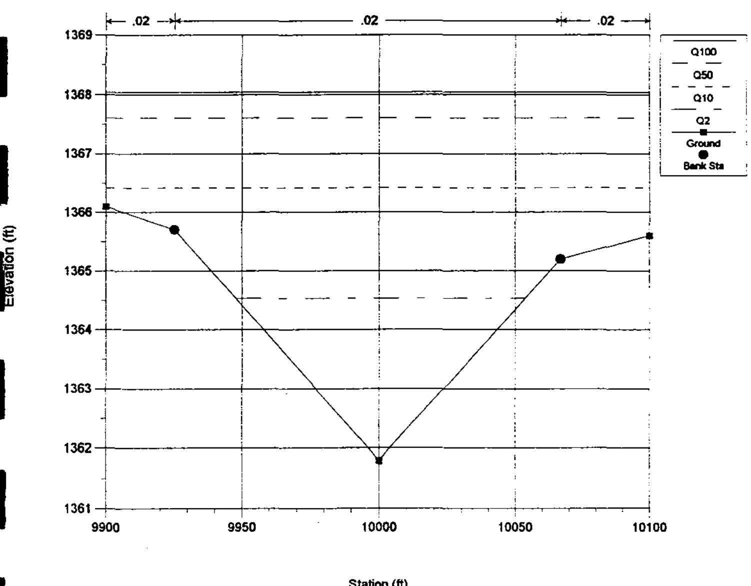
Riv Sta = 6900 CHEROKEE WASH



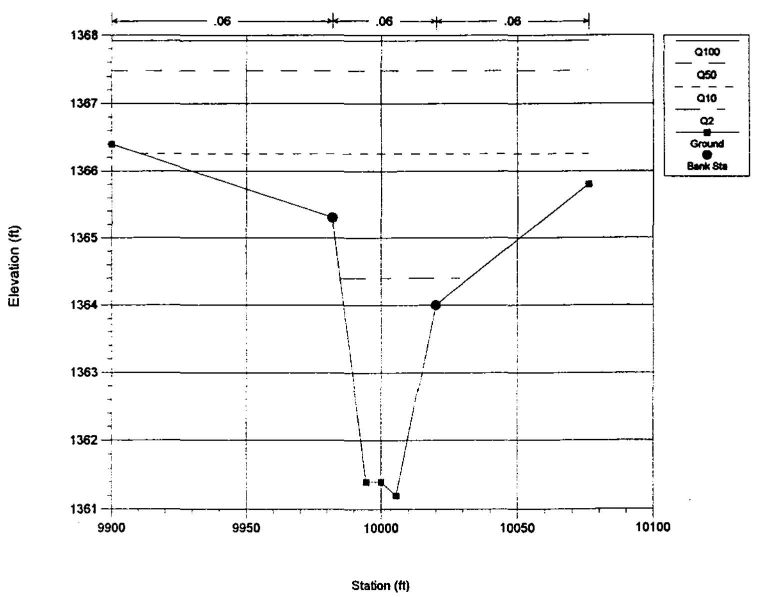
Riv Sta = 6600 CHEROKEE WASH



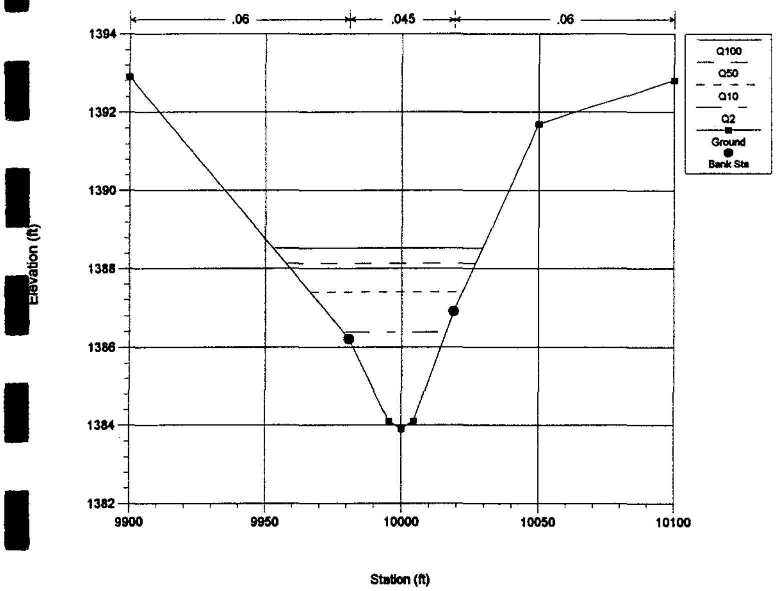
Riv Sta = 6558.8 CHEROKEE WASH



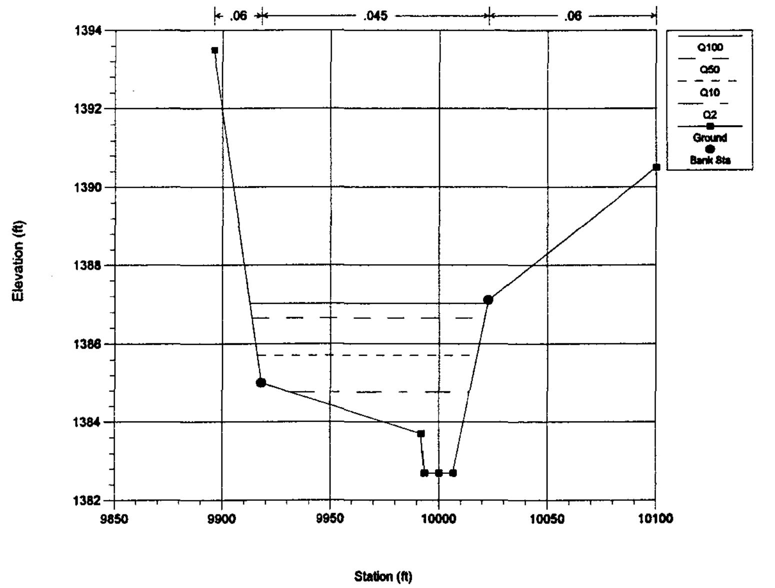
Riv Sta = 6533.8 CHEROKEE WASH



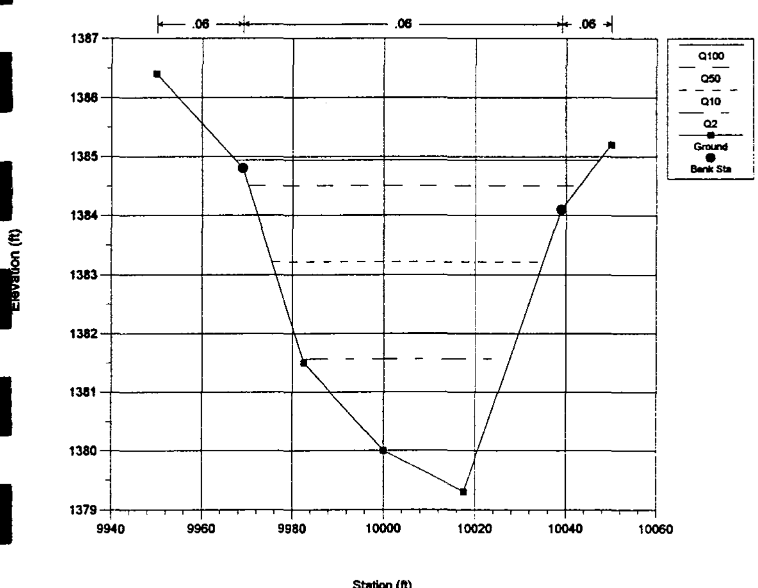
Riv Sta = 9205 CHEROKEE WASH



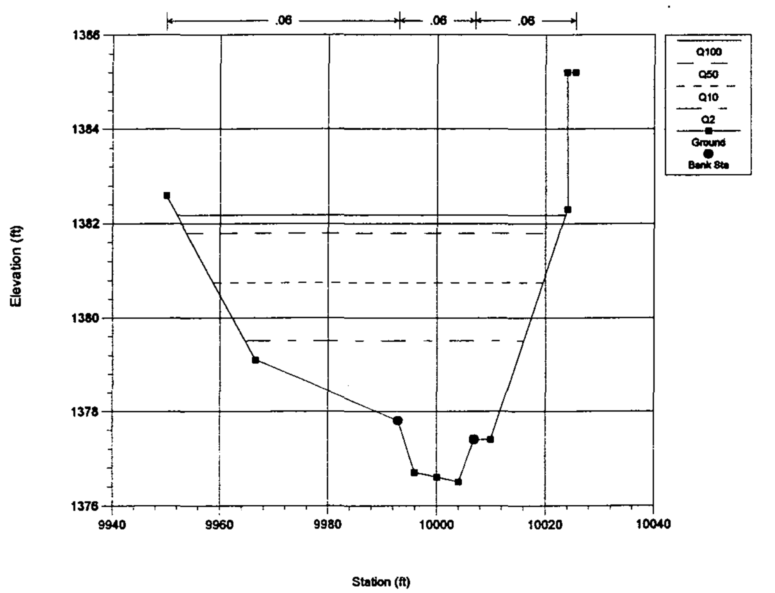
Riv Sta = 9000 CHEROKEE WASH



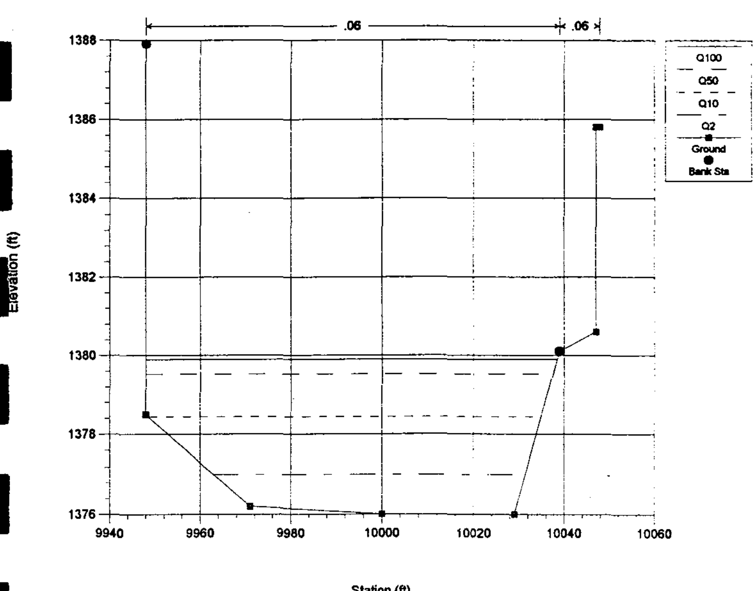
Riv Sta = 8700 CHEROKEE WASH



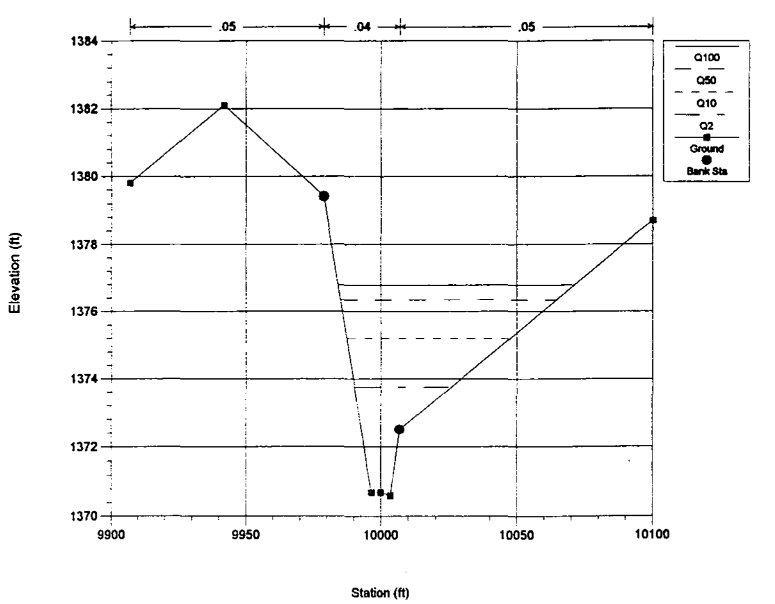
Riv Sta = 8400 CHEROKEE WASH

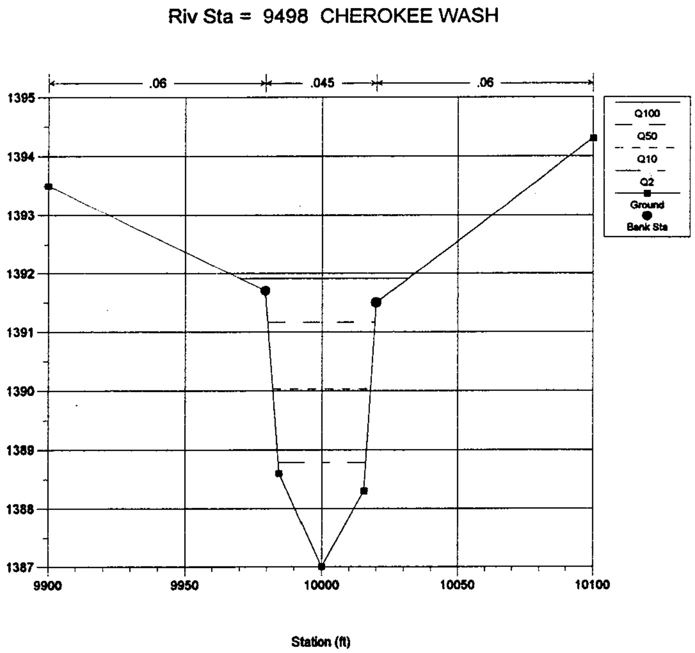


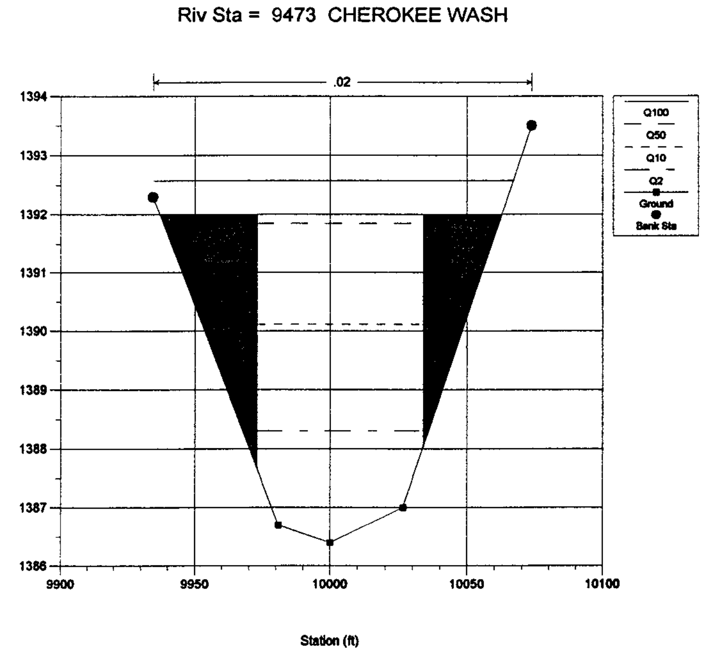
Riv Sta = 8100 CHEROKEE WASH

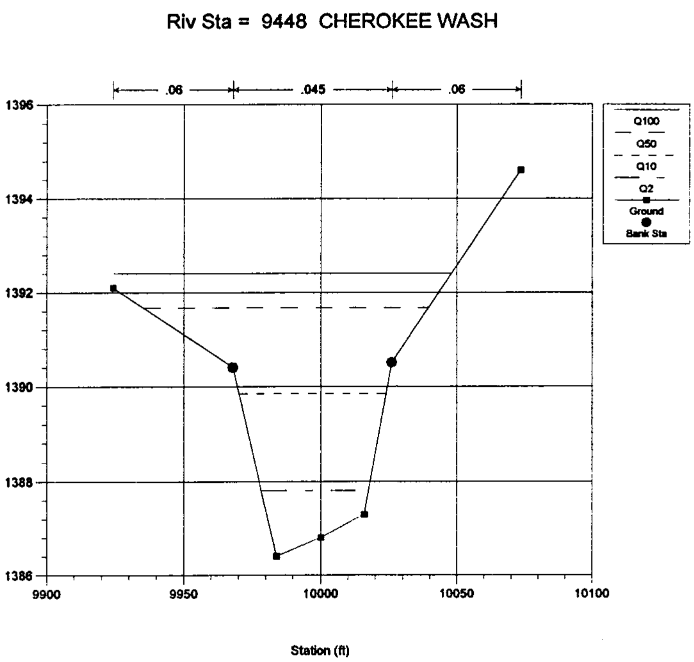


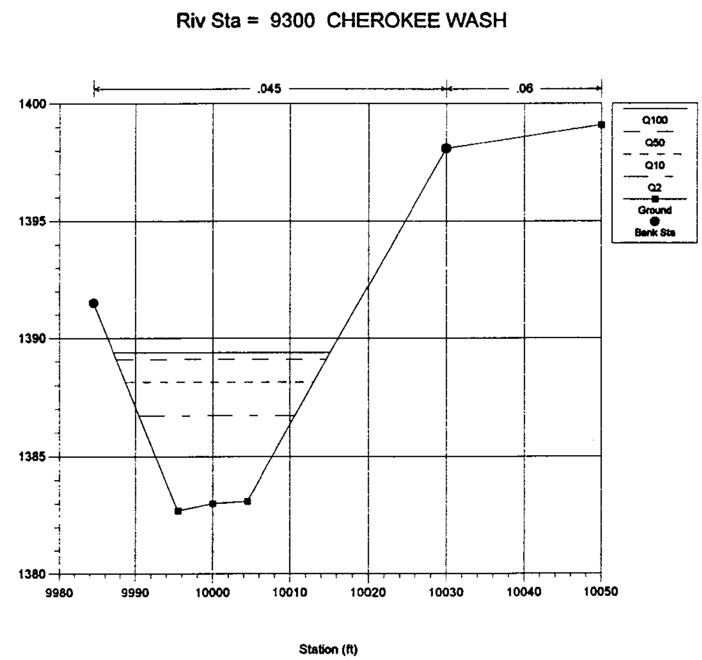
Riv Sta = 7800 CHEROKEE WASH

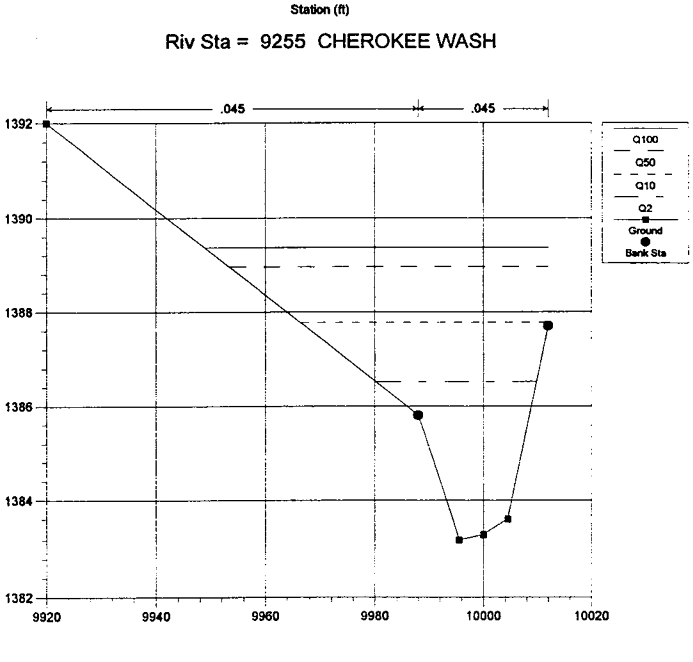


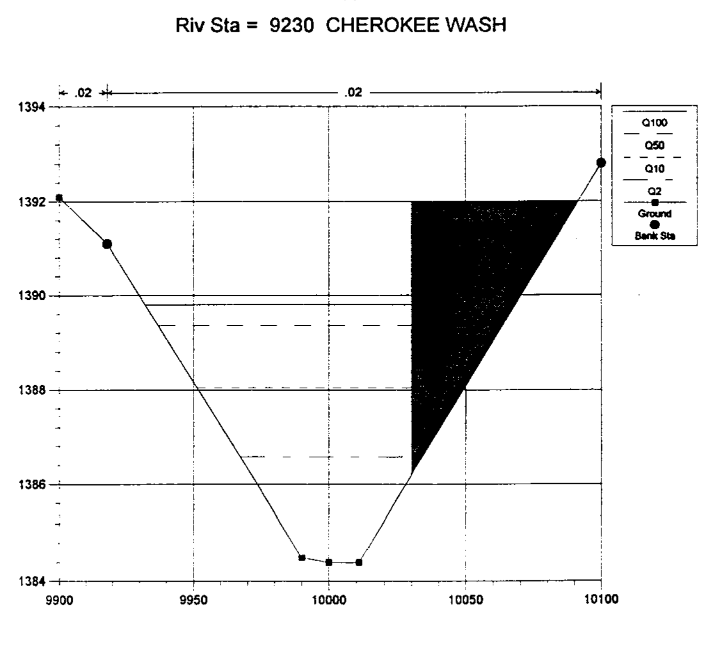

Riv Sta = 9498 CHEROKEE WASH
.06
.045
.06
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta


Riv Sta = 9473 CHEROKEE WASH
.02
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta

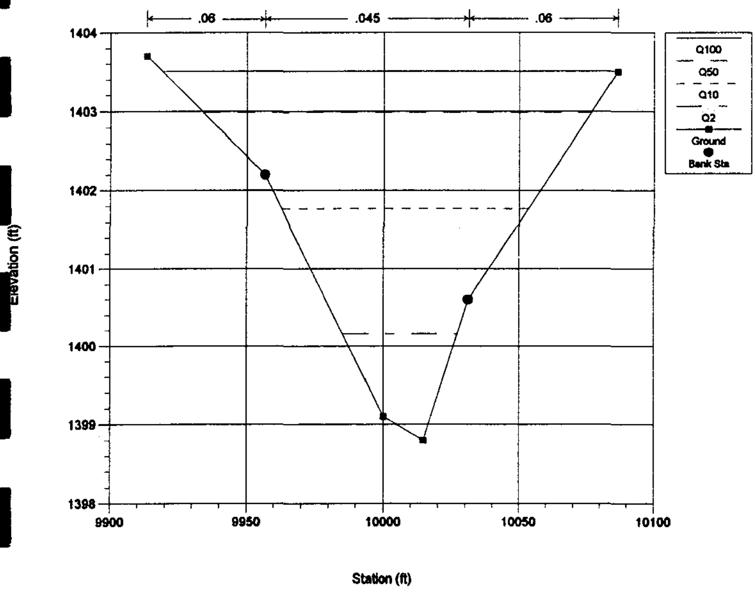

Riv Sta = 9448 CHEROKEE WASH
.06
.045
.06
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta


Riv Sta = 9300 CHEROKEE WASH
.045
.06
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta

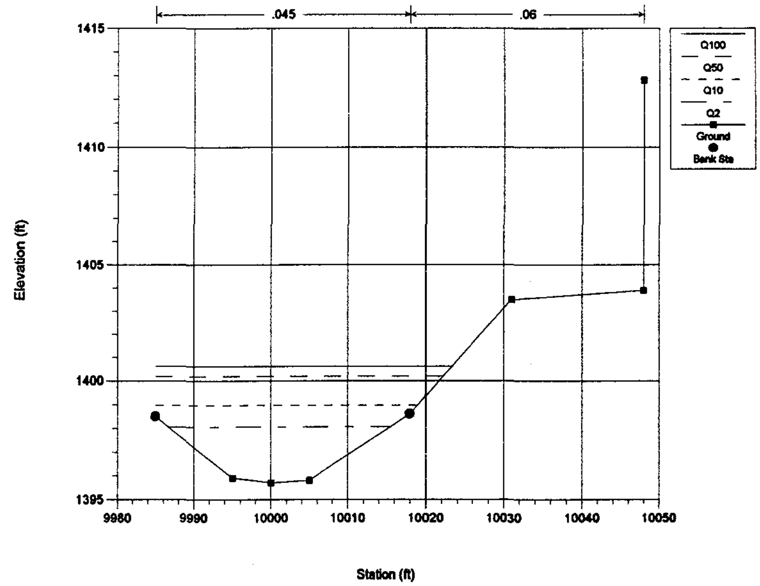

Riv Sta = 9255 CHEROKEE WASH
.045
.045
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta


Riv Sta = 9230 CHEROKEE WASH
.02
.02
Elevation (ft)
Station (ft)
Q100
Q50
Q10
Q2
Ground
Bank Sta

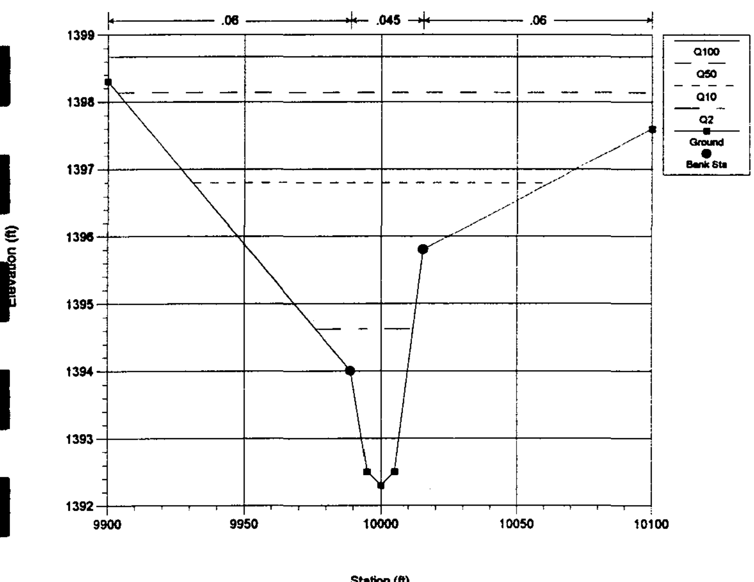
Riv Sta = 10373 CHEROKEE WASH



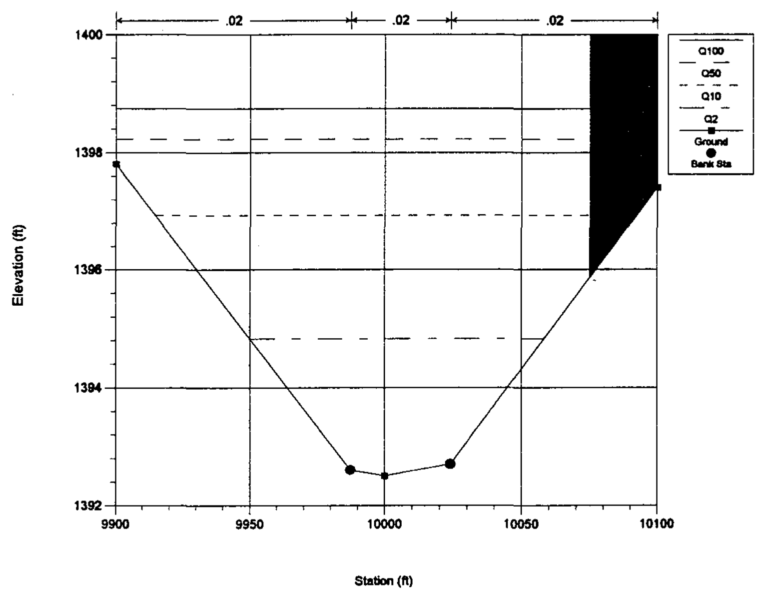
Riv Sta = 10200 CHEROKEE WASH



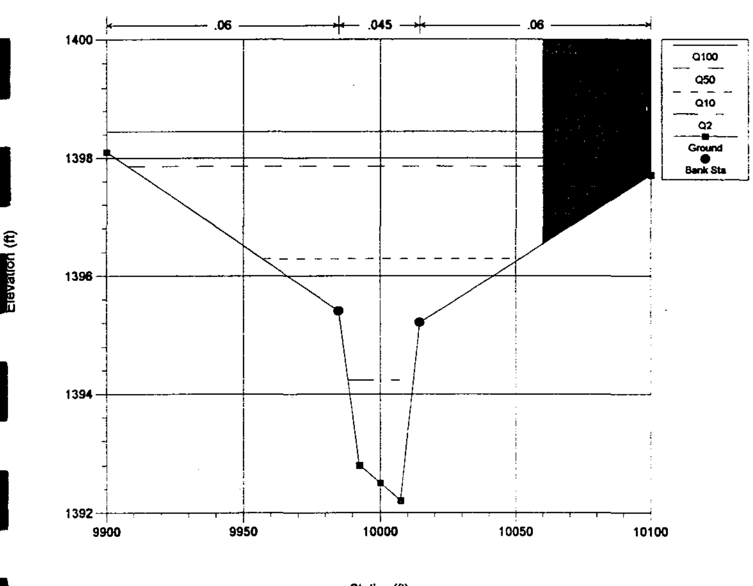
Riv Sta = 9900 CHEROKEE WASH



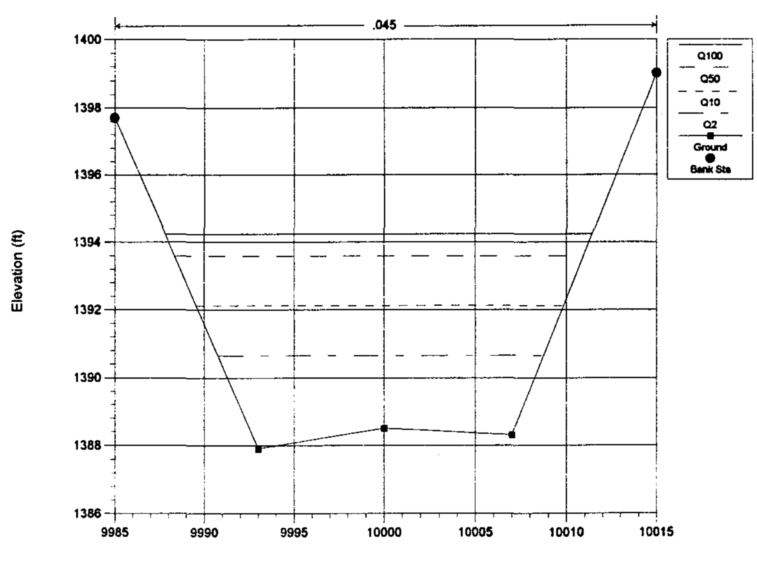
Riv Sta = 9884.5 CHEROKEE WASH



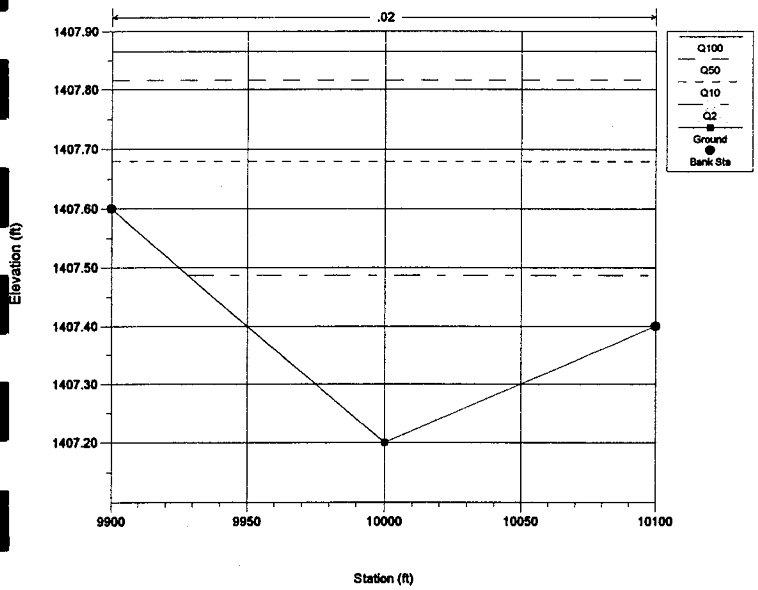
Riv Sta = 9859.5 CHEROKEE WASH



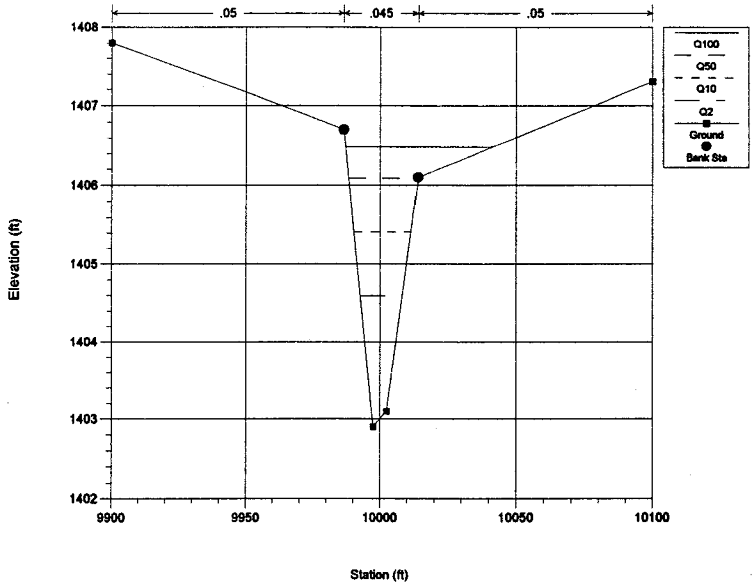
Riv Sta = 9600 CHEROKEE WASH



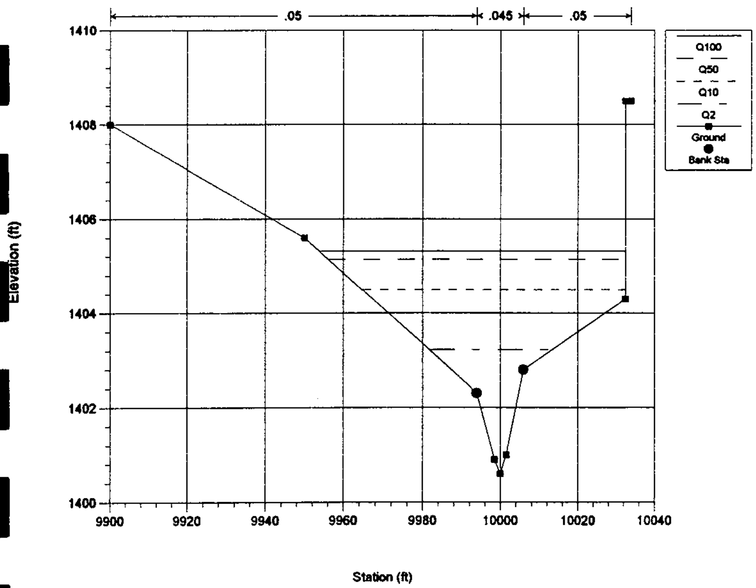
Riv Sta = 11035 CHEROKEE WASH



Riv Sta = 10999 CHEROKEE WASH



Riv Sta = 10800 CHEROKEE WASH



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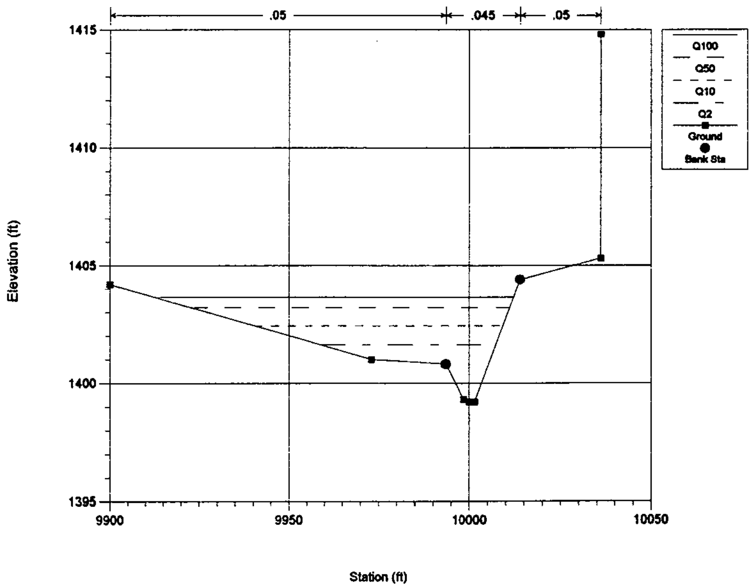
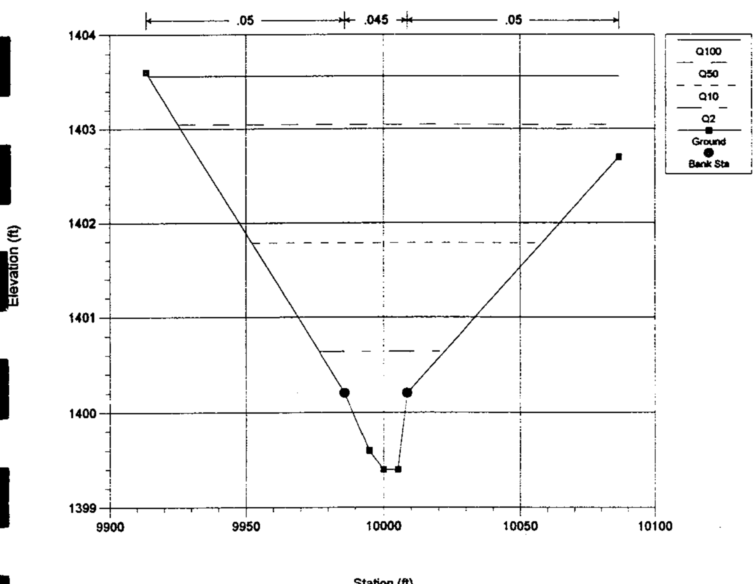
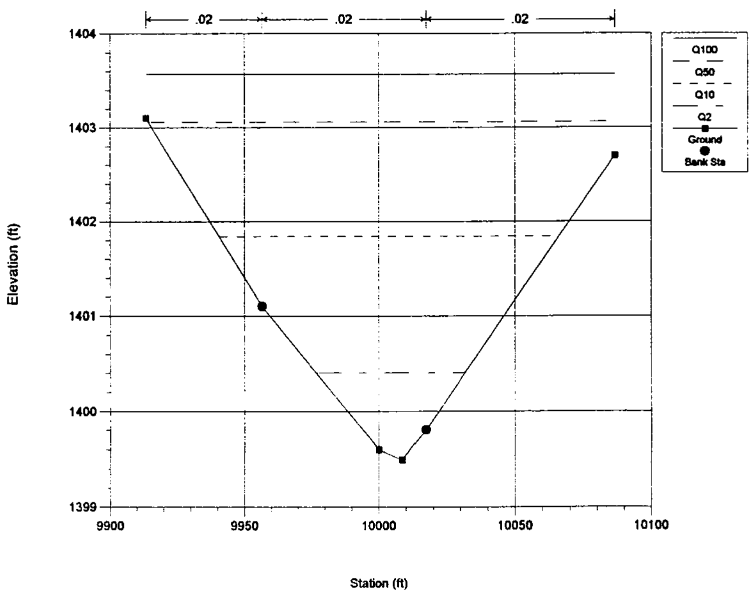
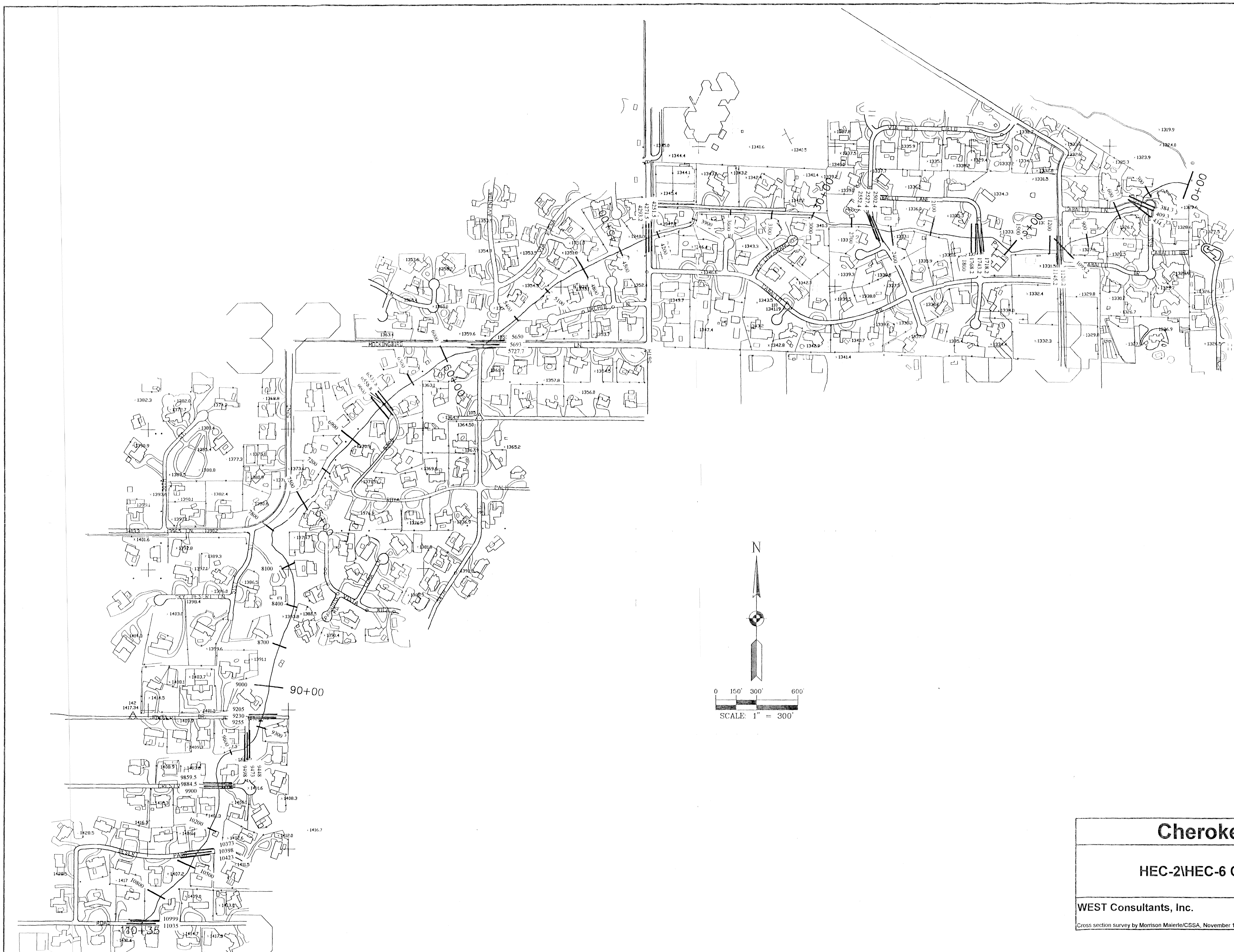


Riv Sta = 10423 CHEROKEE WASH



Riv Sta = 10398 CHEROKEE WASH





Cherokee Wash

HEC-2\HEC-6 Cross Sections

WEST Consultants, Inc. June 1997

Cross section survey by Morrison Maierle/CSSA, November 1996