FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FINAL DESIGN REPORT
SKUNK CREEK CHANNEL IMPROVEMENTS

Submitted to:

Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, AZ

Submitted by:

Simons, Li & Associates, Inc.
4600 South Mill Avenue, Suite 200
Tempe, AZ 85282

June, 1998

sla Simons, Li & Associates, Inc.

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DENNIS L.
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June, 1998

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I. INTRODUCTION

1.1 Authorization and Purpose

The Flood Control District of Maricopa County (District) contracted with Simons, Li & Associates, Inc. (SLA) to complete the final design for Skunk Creek Channel improvements between approximately 74th Avenue and 51st Avenue. The purpose of the project is to improve the capacity of Skunk Creek to contain the 100-year flood within a stable channel and provide information needed by the Federal Emergency Management Agency (FEMA) to reduce the floodplain shown on the Flood Insurance Rate Maps to an area within the channel banks. The contract includes the preparation of final design plans, construction documents, and engineers estimate. This report summarizes the methodology and procedures used and the final design developed for the Skunk Creek channel improvement project.

1.2 Project Description

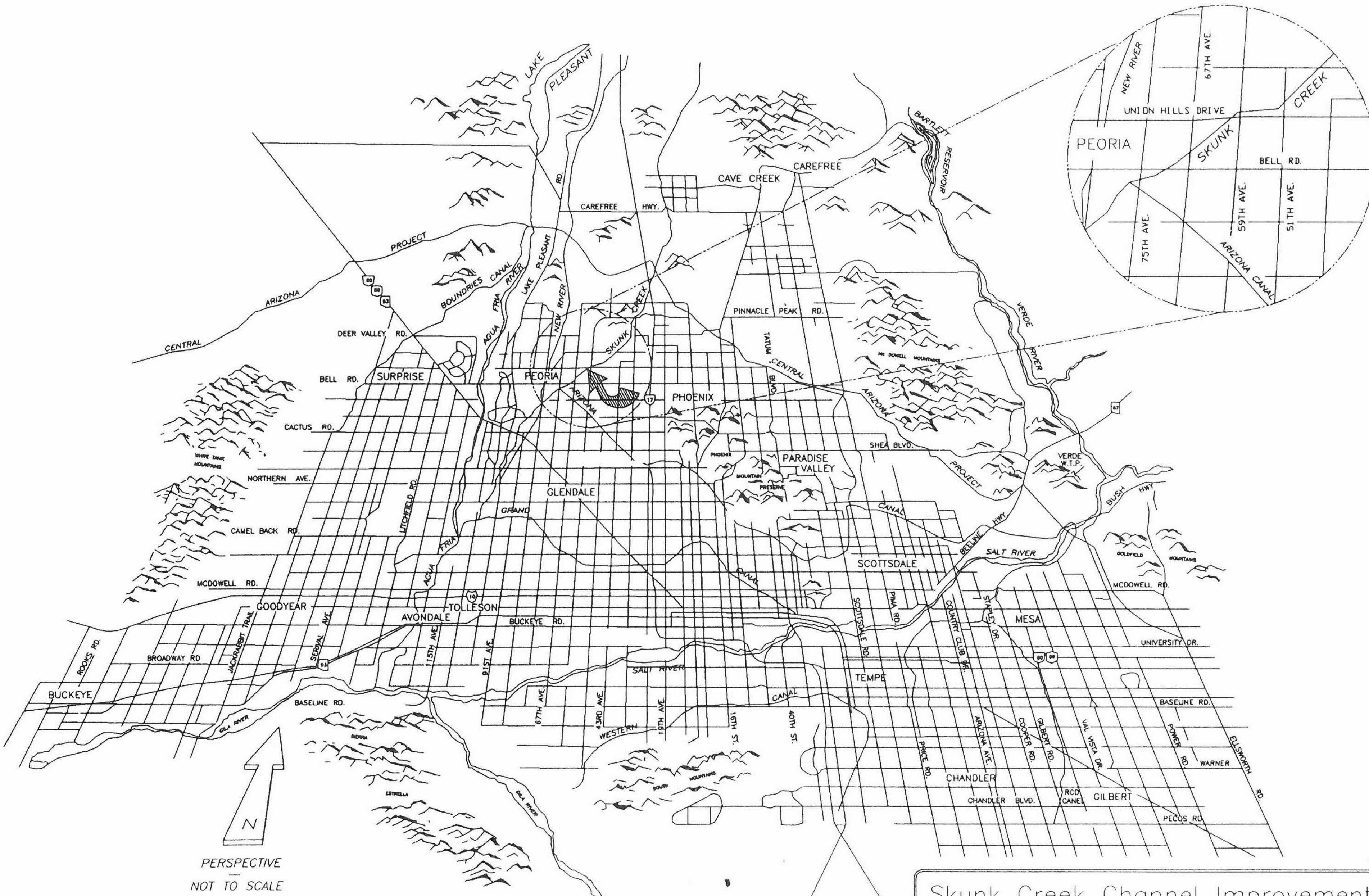
As shown on Figure 1.1, the project location is within the Cities of Glendale and Peoria, Arizona. The study reach extends from the confluence with the Arizona Canal Diversion Channel (ACDC) to the Adobe Dam. The project reach extends from approximately the 74th Avenue alignment upstream to 51st Avenue. A Master Plan (1) was prepared for the Skunk Creek channel between the ACDC and Adobe Dam by Sverdrup Civil, Inc. in August 1995. The study established preliminary right-of-way requirements, 100-year capacity channel designs, grade control structure locations, and bank protection in unlined reaches of the channel. As a result of public participation, the project was designed to accommodate multiple use trails for pedestrians, cyclists, and equestrians. A channel maintenance plan, which includes vegetation management, has been developed by the District (2).

1.3 Scope of Work

The scope of this project includes the completion of final design plans, construction documents, specifications and cost estimate required to provide a stable channel capable of conveying the 100-year FEMA discharge within its banks. The final design is based on the Skunk Creek Master Plan, Final Report and consists of a soft bottom channel with grade control structures as required. The banks are armored primarily with gabions buried under one foot of soil to support vegetation for aesthetics. A minimum amount of revegetation may be required to replace any mature trees lost during construction. The final design includes rough grading, where necessary, to accommodate trails and ramps for pedestrians, equestrians, and maintenance vehicles. Coordination with the Cities of Glendale and Peoria, as well as the District, was conducted throughout the project in this regard.

The analyses required for the final design of the project includes a qualitative and quantitative evaluation of the hydraulic, sediment transport, and scour characteristics of the channel, for both existing and proposed conditions, to determine toe-down and top-of-bank requirements for bank protection and grade control. The existing and future sediment supply from both Scatter Wash and Adobe Dam releases was estimated and an evaluation of alternative drop structure types, heights, hydraulic characteristics, costs, and aesthetics was conducted.

VICINITY MAP



Skunk Creek Channel Improvements

sla

SIMONS, LI & ASSOCIATES, INC.
4600 S. MILL AVENUE, SUITE 200
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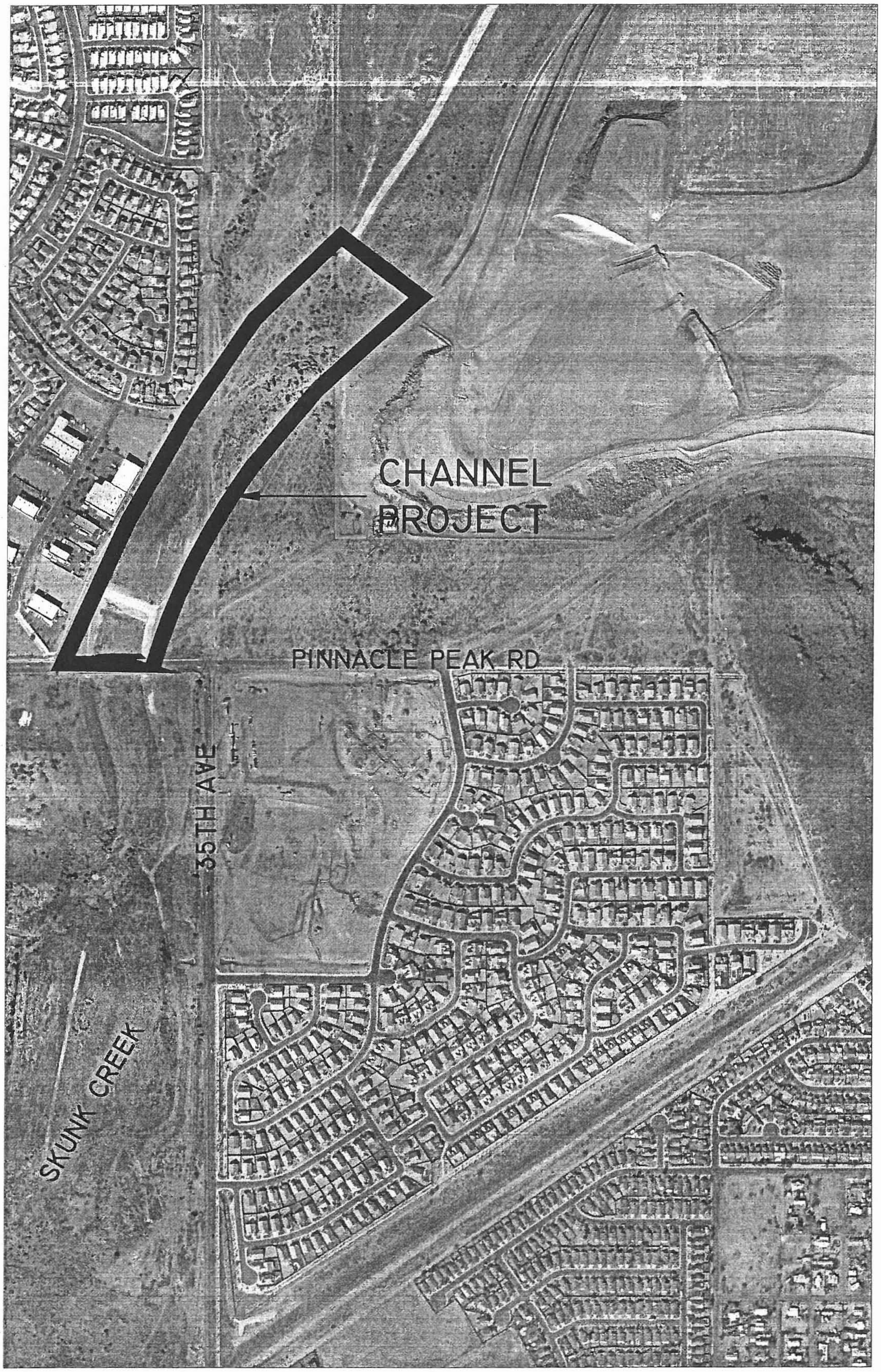
(602) 491-1391

FIGURE 1.1

The project design was developed in two phases with two independent contract packages. The first phase extends from just downstream of Union Hills Drive to the upstream project limit near 51st Avenue. The second phase extends from the downstream project limit, near the 74th Avenue alignment, to just downstream of Union Hills Drive.



KUNK CREEK CHANNEL PROJECT
UPSTREAM OF PINNACLE PEAK RD

QS #45-21

N

SCALE: NOT TO SCALE

II. DATA BASE

2.1 Data Collection Summary

A large number of as-built and design plans for residential developments, Skunk Creek bank protection, roads and bridges, storm drains, sewer lines, water lines, natural gas lines, and electric utilities were obtained from various sources, including the District and the Cities of Glendale and Peoria, and reviewed for conflicts with the proposed final design. Table 2.1 is a summary listing of this information. Sewer, water, and gas lines that crossed Skunk Creek were pot-holed to determine their exact location. The project design was reviewed by and coordinated with Southwest Gas Company and Arizona Public Service. The right-of-way for the final design was provided by the District.

2.2 Hydrology

The current FEMA 100-year (3) flow magnitudes were used as design discharges. These discharges are consistent with those developed by the U.S. Army Corps of Engineers for Adobe Dam and those used in the Master Plan. The Master Plan identified the Arrowhead Drain at the 55th Avenue alignment as the location of a change in a discharge rate. This same location was used as the point of changing discharge in the final design.

A summary of peak-flow values and associated recurrence intervals used in the analysis and design is shown in Table 2.2:

Table 2.2 Hydrology for Skunk Creek

Return Period (years)	Peak Discharge Above Arrowhead Drain (cfs)	Peak Discharge Below Arrowhead Drain (cfs)
10	2270	2970
50	5500	6700
100	8400	11000
500	22000	33000

The sediment routing analysis requires a flood hydrograph as input. The hydrograph used for Skunk Creek was taken from the report, "Final Sediment Transport Report for the New River and Skunk Creek," prepared by SLA in 1985 (4). The discretized hydrograph is reproduced here as Figure 2.1.

Table 2.1 Skunk Creek Data Collection Summary

No.	Location/Type of Data	Source	No. of Sheets	Consultant	Project No.	Date
1	Copper Crest	FCD	2	Clouse Engineering, Inc.	921,202	12/9/96
2	North Creek Grading/Paving	FCD	9	Hendrich Eberhart & Associates	93,151	5/94
3	Arrowhead Valley Unit Three	FCD	6	Randy Delbridge Surveying Inc.	920,723	1/1/93
4	Casa Campana Unit Three - Gabions West Bank	FCD	1	Associated Engineers	470-7	7/18/96
5	Crystal Creek	FCD	17	Age Engineering	1,005,093	4/94
6	Carmel Park/Skunk Creek - Gabions West Bank	FCD	4	Coe & Van Loo	920090 B.M. 1279.90	9/94
7	Arrowhead Ranch from 55th to 59th Avenues	FCD	6	Lowry & Associates		5/85
8	Arrowhead Valley Unit 3	FCD	17	Randy Delbridge Surveying Inc.	920,723	11/19/92
9	Chelsea Village - Gabions East Bank	FCD	3 (6)	Clouse Engineering Inc.	930804 B.M. 1283.82	6/17/94
10	Del Webb Coventry Homes	FCD	3	American Engineering Co.	93,151	7/94
11	Crystal Creek	FCD	3	Age Engineering	1,005,093	4/94
12	75th to 73rd Avenues - Bell Road to ACDC Channel Paradise Lane Bridge - 75th Avenue Bridge	FCD	59	Dibble & Associates Hoffman-Miller	9,303	4/94 50%
12A	77th to 73rd Avenue Bell Road to CDC Channel Paradise Lane Bridge - 75th Avenue Bridge	FCD	102	Dibble & Associates Hoffman-Miller	9,303	3/95 100%
13	Arrowhead Ranch Infrastructure: Bridge Channel Improvements (Soils Test Pits Gabions) Union Hills Bridge	FCD	42	FC Civil Engineering Co. HNTB - 59th & Union Hills/BM. ½' Rebar Pothole	A834042 1267	4/86
14	Arrowhead Ranch - Bridge and Channel Improvements (Soils Test Pits Gabions) 59th Avenue Bridge	FCD	24	International Engineering Dooley Jones Associates John Carollo Engineering Arizona Public Service	845,015	5/86
15	71st Avenue Drainage Channel Improvements	FCD	13	City of Glendale	901,037	5/91
16	Aerial Photos - ACDC Area Drainage Mater Study - Phase I (1" = 400')	FCD	6	Kenney Aerial - Kaminski Hubbard	146	11/28/90
17	Chelsea Village - Preliminary Plat	FCD	1	Clouse Engineering Inc.	930,804	3/8/94
18	Aerial Photos - Quarter Sections	City of Glendale	9	Pictorial Sciences of Arizona		10/10/94

Table 2.1 Skunk Creek Data Collection Summary - Continued

No.	Location/Type of Data	Source	No. of Sheets	Consultant	Project No.	Date
19	Coventry Estates - Tract G and H	City of Glendale	7	American Engineering Co.	93,151	2/21/95
20	Copper Crest - Tract A	City of Glendale	8	Clouse Engineering Inc.	921,202	7/7/95
21	Chelsea Village - Tract A and B	City of Glendale	4	Clouse Engineering Inc.	921,202	8/25/94
22	Sunset Vista Unit 6 - Tracts A and B	City of Glendale	1	Brown Engineering		6/2/92
23	Drafted Quarter Section - City of Glendale	City of Glendale	9	City of Glendale		12/16/96
24	Carmel Park at 51st Avenue - Tract B	City of Glendale	5	Coe & Van Loo	1782-84-01	6/10/92
25	Arrowhead Valley Unit 3 - Tract B	City of Glendale	6	Randy Delbridge Surveying Inc.	920,723	1/1/93
26	Sunset Vista Unit 5 - Tracts A B and C	City of Glendale	1	Brown Engineering		6/2/92
27	Aerial Photos - 83rd Avenue to Deer Valley	City of Glendale	6	Kaminski Hubbard	146	11/28/96
28	Quarter Section Maps - Water/Sewer	City of Glendale			36-11 37-11 12 13 38-13 14 39-14 15 16 40-16	
29	Development at 73rd Avenue and Bell Road	City of Peoria		Hook Engineering	3,033	12/96
30	Bridge at Paradise Lane on Skunk Creek	City of Peoria		Hoffman-Miller	9,303	3/96
31	R.O.W. Acquisition Maps - Phase I and Phase II	FCD		FCD - 8 ½" x 11"		3/13/97
32	Skunk Creek Channel - 77th Avenue to 73rd Avenue (Bell Road to ACDC Channel) See full-sized set #12A/102 sheets	City of Peoria	12	Dibble & Associates (Attached to 11 x 17 R.A.M. report)	9,303	4/19/95
33	Bell Road Bridge Widening over Skunk Creek Channel - Foundation Data		8	Maricopa County Highway Department (½ size plans)	MCHD 43000	6/75
34	Bell Road Bridge Widening over Skunk Creek - Foundation Data/Channel As-Builts		5	Maricopa C.H.D. - Full-sized as-built plans City of Phoenix - Street Transportation (½ size plans)	As-Built	11/77
35	51st Avenue Bridge Over Skunk Creek		10 of 15	Mathews Kessler & Associates City of Phoenix - Street Transportation (½ size plans)	BR- 885806	8/92

Table 2.1 Skunk Creek Data Collection Summary - Continued

No.	Location/Type of Data	Source	No. of Sheets	Consultant	Project No.	Date
36	69th Avenue Dip Crossing		2	American Engineering Co. - 6th Avenue & Grovers - Improvement plans (Sheets #22 and #37)		2/95
37	71st Avenue Drainage Channel Improvements		1	City of Glendale - Public Works/Engineering	901,037	5/91
38	67th Avenue at Skunk Creek - Bridge Plans		23	Inca Engineering (½ size plans)	55	11/95
39	Bell Road Improvements - Outer loop to 67th Avenue - Water/Sewer		4	City of Glendale - Public Works/Engineering		1/92
40	Skunk Creek Channel Improvements at Bell Road - Gabions, North Bank		4	Val Tec, Inc.		5/95
41	ALTA Survey - 54th Avenue		1	Landmark Engineering		7/97
42	Arrowhead Crossing - Phase II - 75th Avenue and Skunk Creek to 77th Avenue (Grading and Drainage)		15	CMX Group		
43	Arrowhead Ranch Channelization of Skunk Creek - 55th Avenue to 57th Avenue		9	Lowry & Associates		10/8/85

Scatter Wash is a significant tributary to Skunk Creek that enters the waterway approximately 3120 feet upstream of 51st Avenue. To evaluate the significance of sediment supplied from this watershed, discharges were needed for the 2-year through 100-year events. The recently completed and approved FIS study conducted on Scatter Wash by Kaminski-Hubbard in 1995 (5) included an updated HEC-1 hydrologic study. This study provided discharges for the 10-, 50-, 100-, and 500-year floods, as well as a hydrograph configuration. The 2-, 5-, and 25-year floods were obtained by extrapolation and interpolation. In addition, the previous FIS hydrology was included in the evaluation on request of the District (3). The flood magnitudes were established as previously described. Table 2.3 summarizes the peak discharges used for Scatter Wash. The discharge-frequency curves are included in Appendix G.

Table 2.3 Hydrology for Scatter Wash

Frequency (years)	2	5	10	20	50	100
New FIS Discharge (cfs)	1075	1290	1540	2000	2545	2760
Old FIS Discharge (cfs)	200	370	580	1565	3500	6100

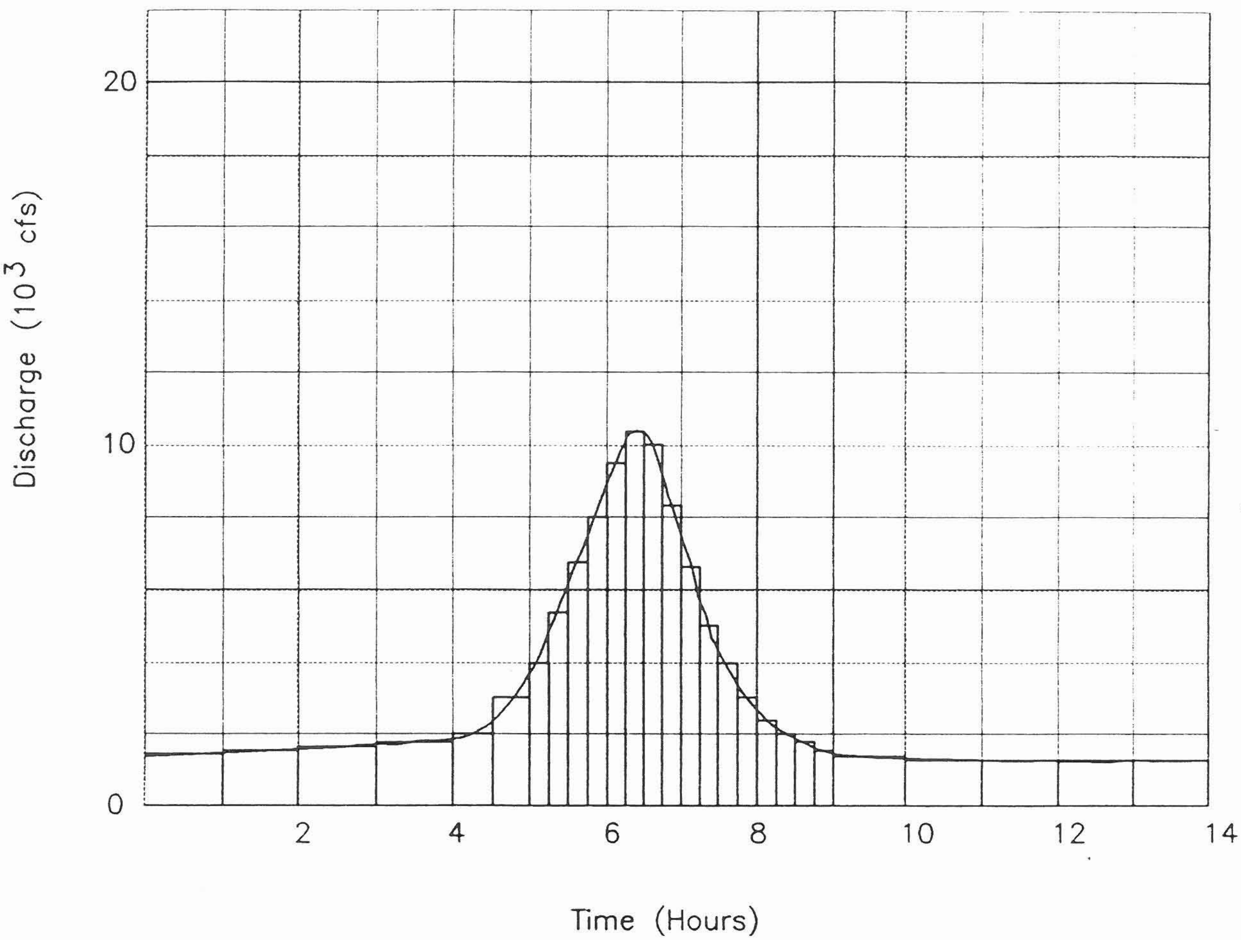


Figure 2.1 Skunk Creek 100-Year Hydrograph

2.3 Sediment Characteristics

A geotechnical investigation of the channel sediment was conducted by Ricker, Atkinson, McBee & Associates, Inc. (RAM). The results of the investigation are summarized in a project report dated May 8, 1997 (6), which is included as Appendix E. The investigation included sampling the bed and bank sediments and performing a sieve analysis and Atterberg Limits tests. The bed sediment samples were taken at approximately 500-foot intervals along the project reach and consisted of a surface sample, typically zero to four-foot deep, and a subsurface sample from four to ten-feet deep. Where needed, samples were also taken to the depth of anticipated local drop-scour, downstream of grade control structures. The bank samples were taken approximately every 1000 feet on alternating banks with the same tests being performed. In addition, a pH and minimum resistivity test was conducted on the bank sediment samples to evaluate the corrosion potential of the gabion wire.

Results of the sieve analyses for the bed samples were averaged to develop the characteristic sediment gradation curve shown in Figure 2.2 which was used in the long-term armoring analysis. The individual sample gradations were used as input in the sediment transport capacity and routing analyses conducted for the general scour and sediment supply estimates. These gradations are shown in Figure 2.3.

The pH and resistivity test results indicate that a high corrosion potential exists at sample locations 14A and 32A. Location 14A is on the north channel bank approximately 700 feet downstream of 67th Avenue. Location 32A is on the south bank approximately 200 feet downstream of 57th Avenue. PVC coated wire can be used for the gabion baskets in these areas to reduce the corrosion potential.

Four bed samples were taken along Scatter Wash and gradation data provided by RAM for the evaluation of sediment supply to Skunk Creek. The sediment gradation data reported in the Master Plan for Skunk Creek, between the confluence with Scatter Wash and 51st Avenue, were used to extend the sediment transport information to the project reach.

2.4 Land Use

The existing land use along Skunk Creek is a mixture of undeveloped land and residential and commercial properties. The majority of undeveloped land is privately owned and zoned for residential use. There are two exceptions. One area within the City of Peoria, bounded by the ACDC, Skunk Creek, and the Glendale City limits, is zoned for light industrial and general agricultural use. Also, an area within the City of Glendale, located on both sides of the Creek, between 57th Avenue and 53rd Avenue, is zoned for agriculture. The land adjacent to Skunk Creek within the Cities of Glendale and Peoria is expected to be fully developed within the near future.

All areas of available land within the entire Skunk Creek watershed are being developed rapidly. As the land within a watershed develops, the amount of runoff can be expected to increase, while the amount of sediment supplied to the channel can be expected to decrease. The net result will typically be general degradation of the channel bottom with eventual instability of the banks.

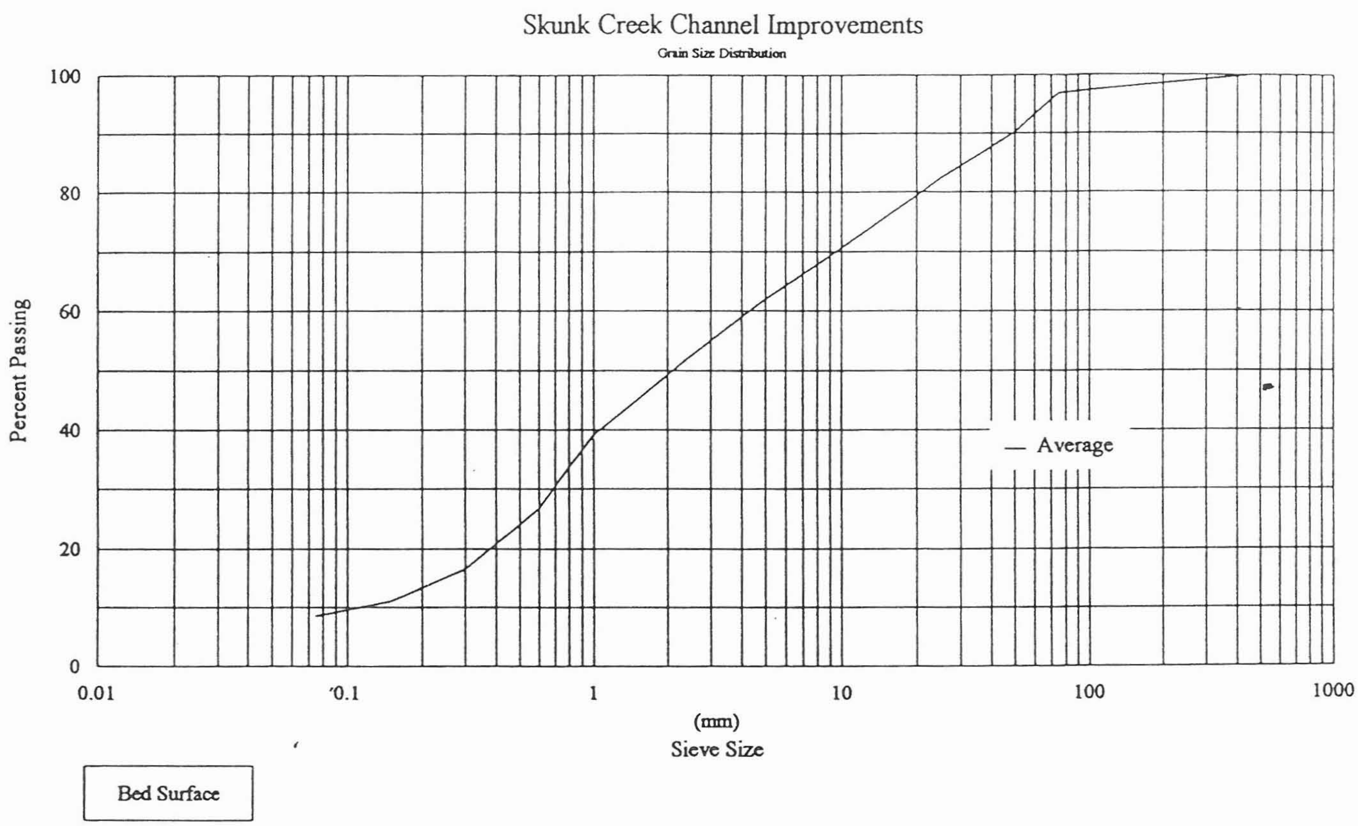
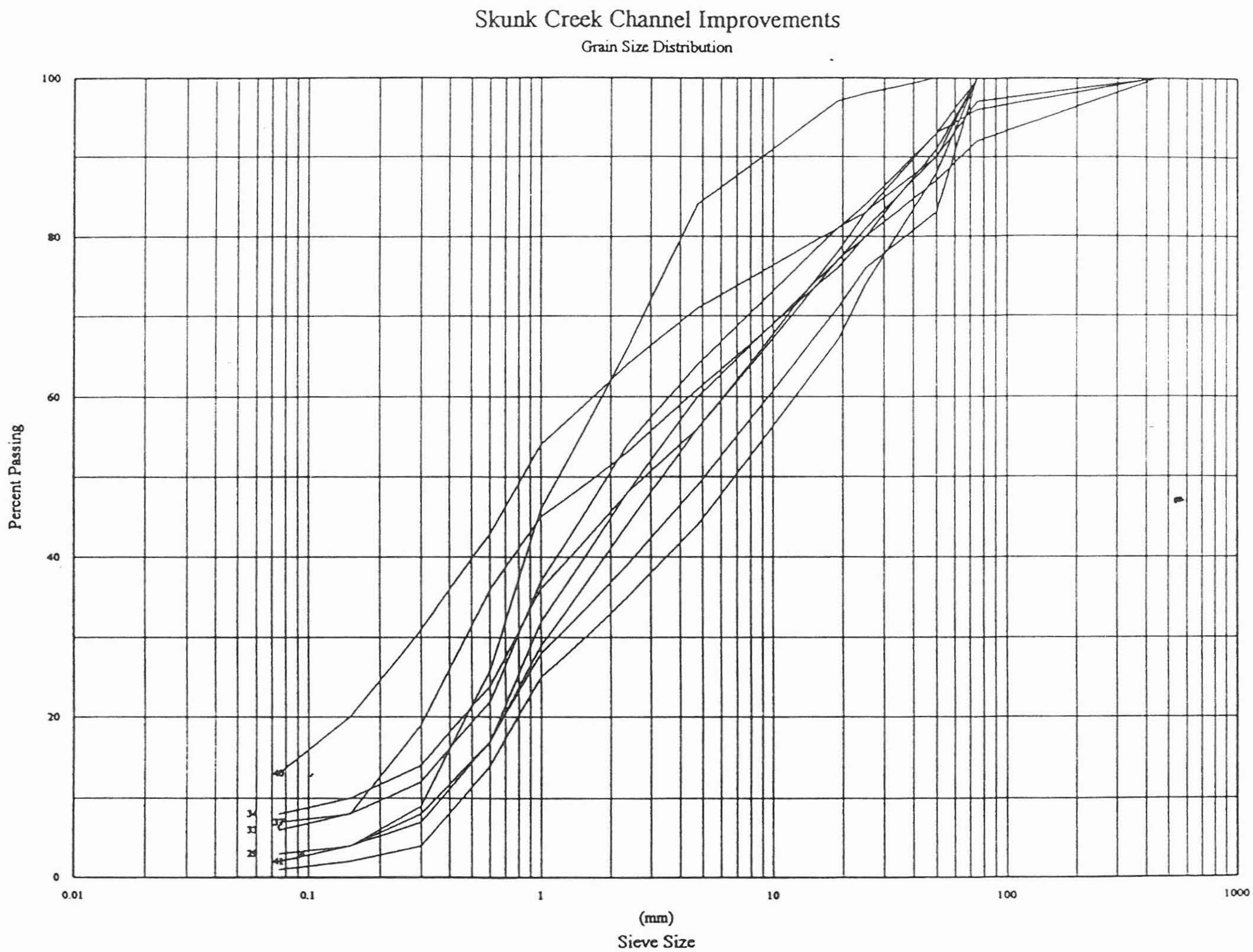


Figure 2.2 Characteristic Sediment Gradation



Bed Surface

Figure 2.3 Sediment Sample Gradations

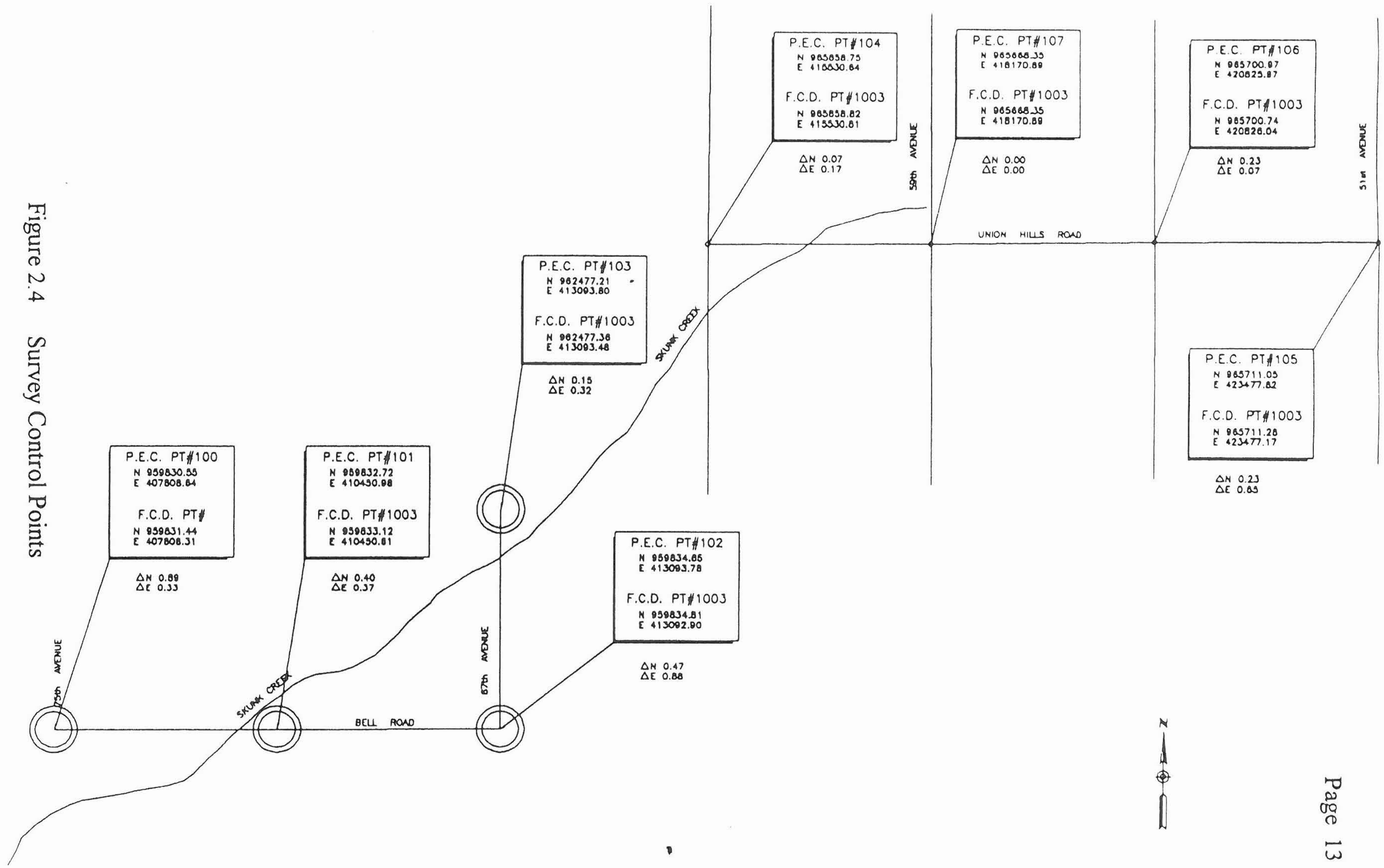
2.5 Topographic Mapping and Survey Control

The topographic mapping used in the analyses and design of the Skunk Creek channel improvements was prepared in April 1997, by Kenney Aerial Mapping, Inc. (KAM). The aerial photography was taken in January 1997, and extended approximately 300 feet on either side of the channel centerline within the project limits. A digital terrain model (DTM) with one-foot contour intervals was provided by KAM in AutoCAD, Release 13 format.

Survey controls for the mapping and project design were provided by the District. The final coordinate locations of the control points used are shown on Figure 2.4. The control and project survey was conducted by Project Engineering Consultants, Ltd. (PEC).

The mapping for the effective flood insurance study was used for channel and floodplain geometry information on Scatter Wash (5). Information contained in the Master Plan was used to define the channel geometry between 51st Avenue and the confluence of Skunk Creek and Scatter Wash.

Figure 2.4 Survey Control Points



Simons, Li & Associates, Inc.
Water Resources & Civil Engineering Consultants

III. ANALYSES

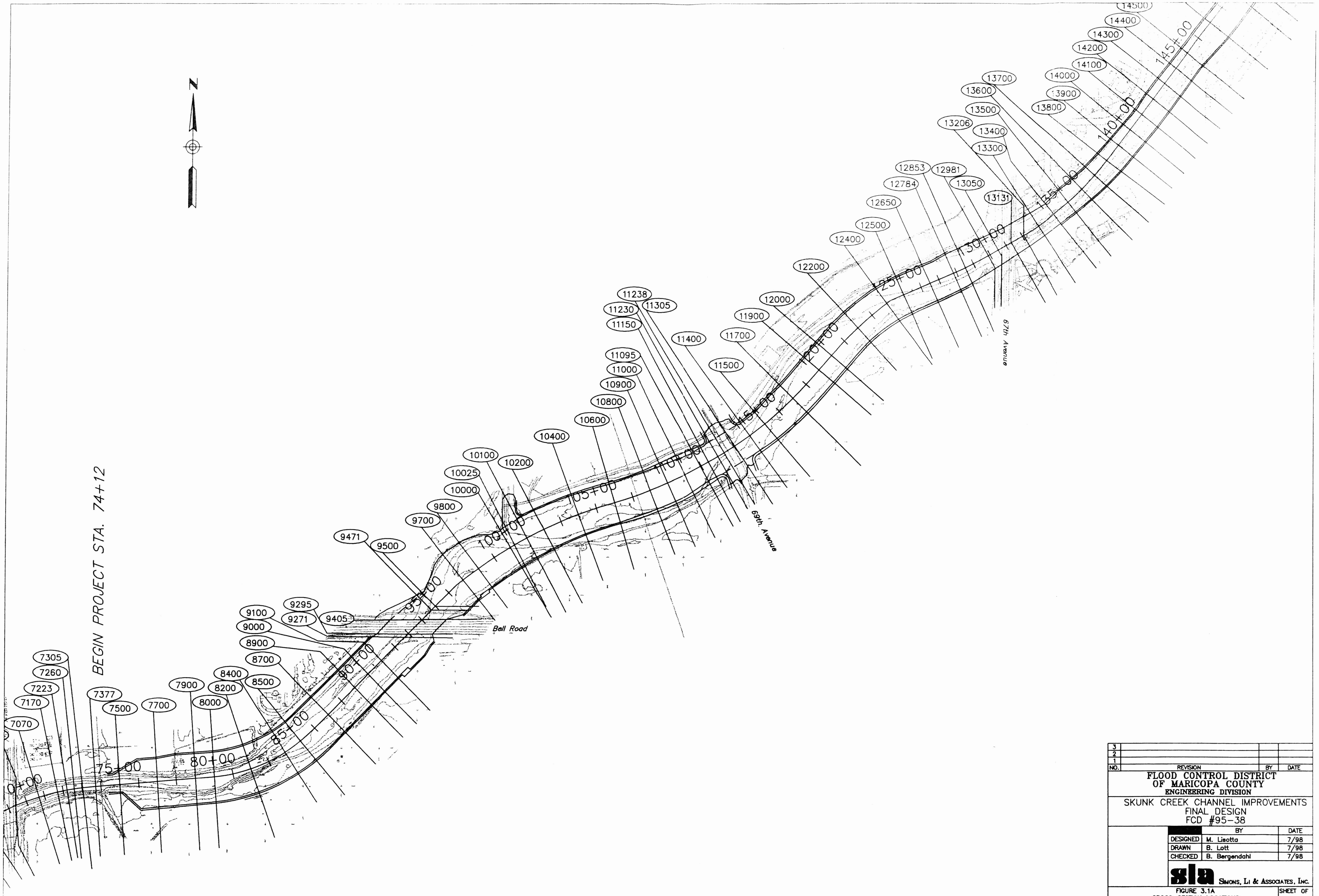
3.1 Hydraulic Analysis

The U.S. Army Corps of Engineers computer program "HEC-2 Water Surface Profiles" (7) was used to calculate the hydraulic parameters for Skunk Creek through the project reach for both existing and proposed conditions. All cross-sections were selected and encroachment stations inserted to accommodate HEC-2's one dimensional flow limitations and maintain reasonable section-to-section conveyance continuity. Cross-section locations are illustrated in Figure 3.1. The cross-section numbers coincide with the design control-line stationing. Ineffective flow areas were eliminated through encroachments using a 1:1 transition rate at contractions and a 4:1 transition rate at expansions, where appropriate. Because of the channelized nature of Skunk Creek the need for encroachments was limited. General expansion and contraction coefficients were set at 0.3 and 0.1, respectively, in the model. The expansion and contraction coefficients were modified to 0.5 and 0.3, respectively, through the various bridge crossings in the study reach. These values are consistent with recommendations provided in the HEC-2 Users Manual. The starting water-surface elevations were computed by using the slope-area method. The energy grade-line slopes were taken from the effective FIS for Skunk Creek at the confluence with the ACDC.

Structures modeled within the project reach included the existing bridges at Bell Road, Union Hills Drive, 59th Avenue, and 51st Avenue; and the 67th Avenue bridge which was under construction at the beginning of this study. Proposed structures modeled included bridges at Paradise Lane and 75th Avenues, the drop structure at the downstream end of the project, designed by Dibble and Associates, and all grade control structures. All bridges were modeled using the HEC-2 special bridge routine. The hydraulic structures modeled for existing and proposed conditions are summarized in Table 3.1.

Table 3.1 Hydraulic Structure Summary

Existing Conditions	Proposed Conditions
75th Avenue Dip Crossing	Paradise Lane Bridge 75th Avenue Bridge Dibble Drop Structure
Bell Road Bridge	Bell Road Bridge/Grade Control Structure
69th Avenue Dip Crossing 67th Avenue Dip Crossing	69th Avenue Dip Crossing/Grade Control Structure 67th Avenue Bridge/Grade Control Structure Station 155 Grade Control Structure
Union Hills Bridge	Union Hills Bridge/Grade Control Structure
59th Avenue Bridge	59th Avenue Bridge/Grade Control Structure
57th Avenue Dip Crossing	57th Avenue Dip Crossing/Grade Control Structure
54th Avenue Dip Crossing	54th Avenue Dip Crossing/Grade Control Structure Station 245 Grade Control Structure
51st Avenue Bridge	Station 256 Grade Control Structure 51st Avenue Bridge



NO.	REVISION	BY	DATE
3			
2			
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FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
ENGINEERING DIVISION

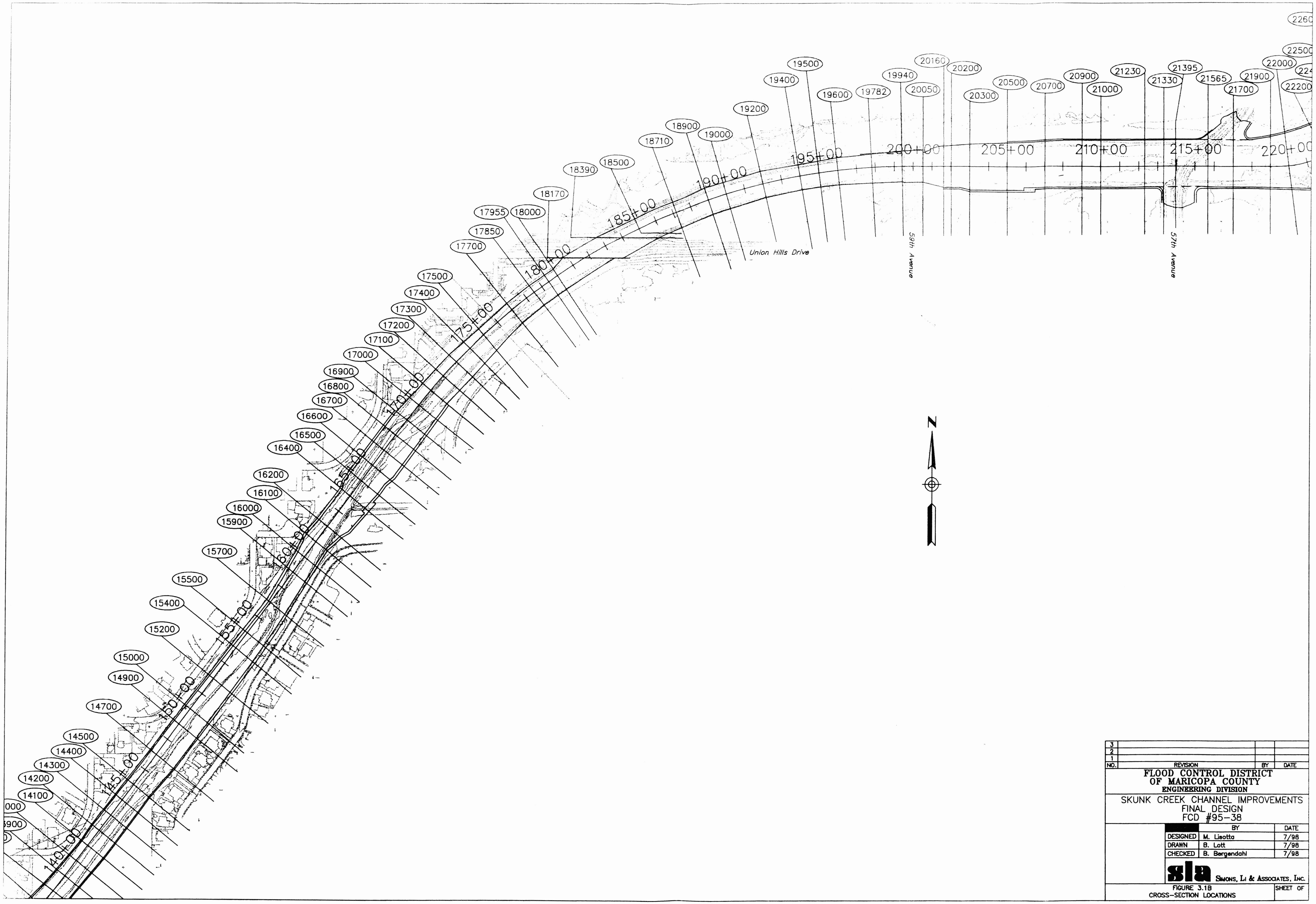
SKUNK CREEK CHANNEL IMPROVEMENTS
FINAL DESIGN
FCD #95-38

	BY	DATE
DESIGNED	M. Lisotta	7/98
DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

SIMONS, LI & ASSOCIATES, INC.

FIGURE 3.1A
CROSS-SECTION LOCATIONS

SHEET OF


Union Hills Drive
59th Avenue
57th Avenue
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NO.
REVISION
BY
DATE
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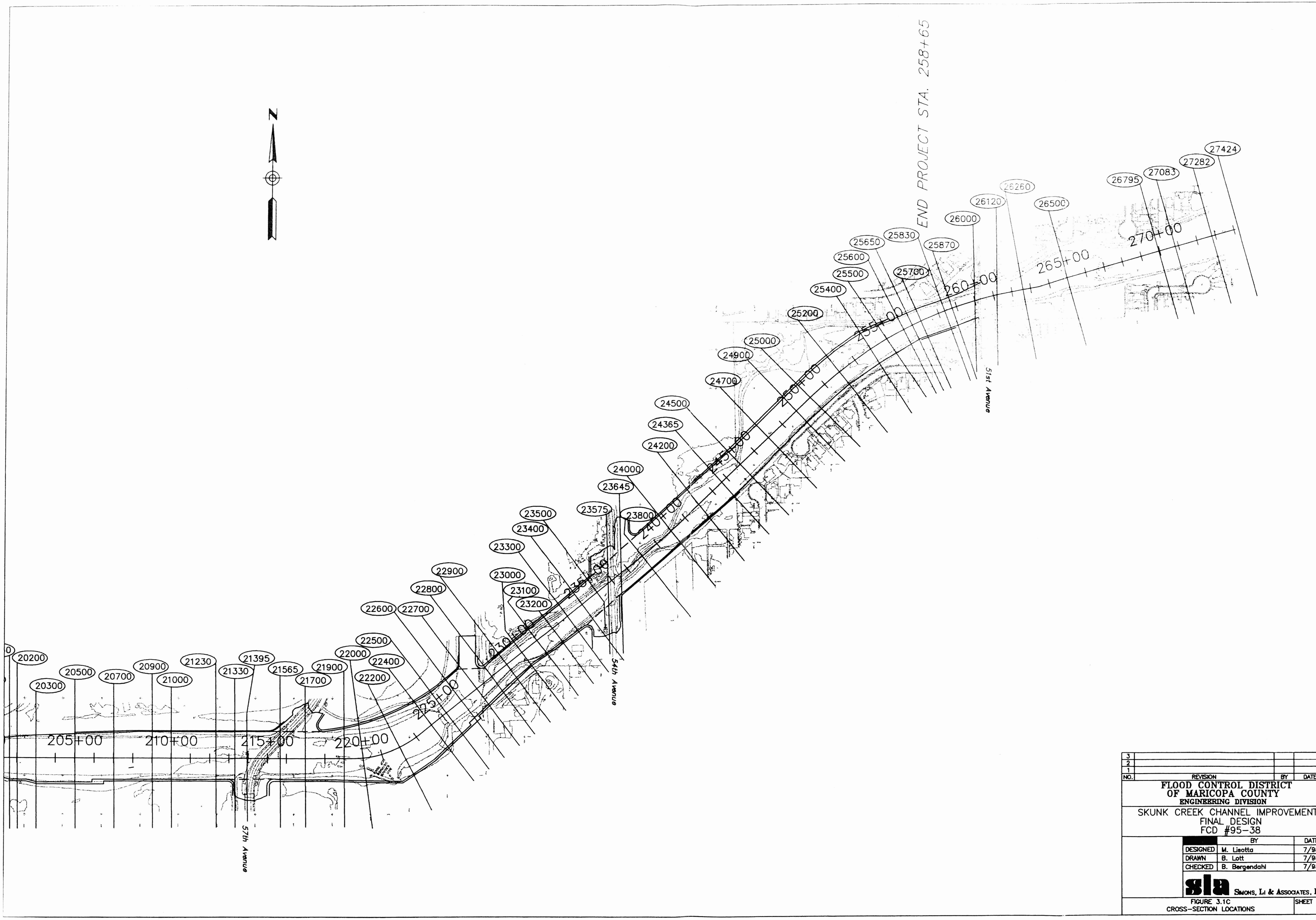
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
ENGINEERING DIVISION
SKUNK CREEK CHANNEL IMPROVEMENTS
FINAL DESIGN
FCD #95-38
BY
DATE
DESIGNED M. Lisotta 7/98
DRAWN B. Lott 7/98
CHECKED B. Bergendahl 7/98
sla Simons, Li & Associates, Inc.
FIGURE 3.1B
CROSS-SECTION LOCATIONS
SHEET OF



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FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
ENGINEERING DIVISION

SKUNK CREEK CHANNEL IMPROVEMENTS
FINAL DESIGN
FCD #95-38

	BY	DATE
DESIGNED	M. Lisotta	7/98
DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

sla SIMONS, LI & ASSOCIATES, INC.

FIGURE 3.1C
CROSS-SECTION LOCATIONS

SHEET OF

The existing conditions HEC-2 hydraulic model was developed to establish a baseline to measure the impacts of various design alternatives. The model output is provided in Appendix A of this report. A Manning's roughness coefficient of 0.035 was used for the channel. Table 3.2 provides a summary of the key hydraulic parameters for the existing conditions, 100-year flow in the project reach.

The proposed conditions HEC-2 model output is provided in Appendix B of this report. The Manning's roughness (n-value) was varied from 0.025 to 0.040 depending on the purpose of the analysis. To account for variable vegetation density, a Manning's n-value of 0.04 was selected for a more conservative estimate of water surface elevations. In contrast, an n-value of 0.025 was selected for sediment transport analysis since lower roughness produces higher velocities and, therefore, more conservative sediment transport estimates. Table 3.3 provides a summary of the key hydraulic parameters for the proposed conditions (n=0.04), 100-year flow in the project reach.

3.2 Qualitative Channel Stability Analysis (Level I)

The first level of stability analysis conducted for Skunk Creek was qualitative and is based on interpretation of field observations of the channel and watershed, soils data, historic mapping and photos, geomorphology principles, and general hydraulic relationships.

Figure 3.2 graphically illustrates the geomorphic classifications defined by Lane, and Leopold and Wolman (8). These relationships illustrate the long-term tendencies of natural alluvial channel systems as a function of discharge and slope. Using the dominant 10-year discharge of 2970 cfs and the average channel slope of approximately 0.5 percent, these relationships show that Skunk Creek has the natural tendency to be a braided stream. This fact is confirmed by inspecting old USGS mapping and aerial photographs, and supports the potential for low-flow incisement channels developing between the proposed stabilized banks.

Figure 3.3 compares profile plots of the Skunk Creek thalweg within the study reach for 1957 and 1997. The plot reflects the general tendency for channels to degrade due to the reduced sediment supply and increased runoff associated with development and urbanization. Because of the continued urbanization of the watershed and the relatively recent completion of the Adobe Dam and the CAP canal, this degradation pattern is expected to continue.

3.3 Sediment Transport Capacity Analysis (Level II)

The second level of stability analysis conducted was quantitative and included the evaluation of sediment transport capacities for proposed conditions based on the application of sediment transport equations and steady-state hydraulics. For this analysis, the study reach is divided into 79 subreaches. Delineation of the channel into subreaches is based on consideration of: 1) physical characteristics of the channel, such as top width and slope; 2) hydraulic parameters, particularly velocity, 3) sediment characteristics; 4) areas of interest, (i.e., bridges, dip crossings, and grade control structures); and 5) the desire to maintain subreach lengths as uniform as possible throughout the project. The velocity and top-width profiles for the 100-year flow are presented in Figures 3.4. and 3.5, respectively.

Table 3.2 Existing Conditions 100-Year Hydraulics

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top-Width (ft)
5724	11000	1194.7	9.4	11.7	128
5838	11000	1195.0	9.6	13.4	112
5994	11000	1195.4	9.4	13.6	113
6094	11000	1195.7	9.3	13.6	113
6194	11000	1195.9	9.3	13.7	113
6260	11000	1196.1	9.1	13.7	116
6394	11000	1196.4	9.2	13.7	115
6594	11000	1196.8	9.3	13.6	113
6694	11000	1197.0	9.4	13.6	113
6876	11000	1197.5	9.1	13.7	117
6964	11000	1197.7	9.1	13.7	117
7094	11000	1198.0	9.1	13.7	117
7194	11000	1198.2	9.1	13.9	114
7247	11000	1198.4	9.0	14.0	115
7284	11000	1199.1	6.5	14.7	138
7329	11000	1199.1	6.5	14.7	138
7334	11000	1199.0	7.5	12.9	133
7354	11000	1199.6	14.2	7.1	124
7363	11000	1202.1	14.6	9.6	115
7500	11000	1204.8	15.9	10.8	89
8000	11000	1209.3	9.7	12.3	121
8500	11000	1212.9	6.0	11.9	240
9000	11000	1213.3	8.1	10.3	201
9295	11000	1214.2	8.0	10.2	203
9405	11000	1214.4	10.1	8.4	212
9539	11000	1215.7	8.0	9.7	210
9725	11000	1216.5	6.3	9.5	282
9960	11000	1217.1	5.5	6.1	391
10025	11000	1216.7	9.8	6.7	248
10125	11000	1217.2	11.1	6.2	227
10500	11000	1220.2	6.9	8.2	291
11000	11000	1221.8	8.5	9.8	320
11238	11000	1223.0	10.0	7.0	240
11305	11000	1224.1	7.4	7.1	300
11500	11000	1224.5	10.3	6.5	239
12000	11000	1227.3	8.0	8.3	245
12500	11000	1229.5	8.2	7.5	217
12776	11000	1230.5	7.8	9.5	223
12822	11000	1230.7	7.6	9.7	224
12952	11000	1230.2	12.5	7.2	162
13050	11000	1232.2	8.4	9.2	513
13131	11000	1231.8	13.0	6.8	166
13206	11000	1232.8	11.8	7.8	152
13400	11000	1234.3	10.8	9.3	270
13500	11000	1235.0	10.0	10.0	148
14000	11000	1236.8	10.6	10.8	153
14500	11000	1238.9	10.0	10.9	136
15000	11000	1240.6	8.9	9.6	212

Table 3.2 Existing Conditions 100-Year Hydraulics (continued)

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top-Width (ft)
15500	11000	1242.5	8.2	10.5	324
16000	11000	1244.4	12.1	8.4	130
16200	11000	1245.2	15.0	9.2	106
16500	11000	1248.7	11.2	10.7	348
16900	11000	1250.9	7.6	12.9	357
17000	11000	1251.1	7.7	12.1	289
17500	11000	1251.9	11.2	7.9	198
17850	11000	1253.8	10.9	8.8	304
18170	11000	1256.1	6.2	9.1	229
18390	11000	1256.2	6.2	8.2	250
18710	11000	1257.1	12.2	7.1	267
19000	11000	1259.1	11.4	8.1	138
19500	11000	1261.5	12.1	7.5	133
19782	11000	1263.7	9.7	8.7	150
19950	11000	1264.5	9.0	9.5	155
20050	11000	1264.6	9.0	8.6	156
20220	11000	1265.9	5.2	9.9	242
20235	11000	1266.0	5.1	10.0	261
20300	11000	1266.1	4.9	10.1	378
20500	11000	1266.2	5.0	9.2	289
21000	11000	1266.5	7.7	8.5	251
21330	11000	1267.7	8.6	7.7	266
21395	11000	1268.0	9.0	8.0	248
21565	11000	1269.3	6.5	8.3	366
21700	11000	1269.2	10.6	7.2	225
22000	11000	1271.6	7.4	6.6	276
22200	11000	1272.2	7.7	6.2	282
22500	11000	1276.1	11.3	8.1	270
22625	11000	1278.1	7.5	8.1	318
22877	8400	1278.6	9.6	10.6	108
23000	8400	1278.9	10.2	9.9	102
23500	8400	1280.7	9.0	8.7	125
23575	8400	1281.4	6.9	9.4	220
23645	8400	1283.5	11.5	11.5	264
24000	8400	1285.5	5.0	11.5	197
24365	8400	1285.9	6.2	8.9	188
24500	8400	1285.9	8.0	8.9	167
25000	8400	1287.2	2.6	9.2	378
25500	8400	1286.8	12.0	5.8	160
25600	8400	1288.6	8.9	7.6	161
25870	8400	1289.6	11.0	5.6	153
26000	8400	1290.5	12.6	5.5	136
26020	8400	1292.0	9.7	7.0	142
26120	8400	1293.8	8.4	7.8	151
26260	8400	1294.7	6.7	8.7	274
26500	8400	1295.3	5.9	6.3	253
26700	8400	1295.6	7.3	5.6	225
27000	8400	1296.7	8.5	5.7	215

Table 3.3 Proposed Conditions 100-Year Hydraulics

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top Width (ft)
5724	11000	1194.6	8.7	13.6	127
5814	11000	1194.7	9.6	13.3	112
5970	11000	1195.3	9.3	13.7	113
6070	11000	1195.6	9.2	13.8	114
6170	11000	1195.9	9.0	13.9	115
6236	11000	1196.1	8.9	13.9	117
6370	11000	1196.5	8.9	14.0	116
6570	11000	1197.0	9.0	14.0	115
6670	11000	1197.3	8.9	14.1	115
6852	11000	1197.8	8.6	14.3	119
6940	11000	1198.0	8.7	14.2	119
7070	11000	1198.3	8.6	14.3	119
7170	11000	1198.6	8.6	14.5	116
7223	11000	1198.7	8.5	14.6	117
7260	11000	1199.4	6.2	15.2	140
7305	11000	1199.4	6.2	15.2	140
7310	11000	1199.3	7.2	13.4	135
7330	11000	1199.3	14.3	7.1	124
7339	11000	1201.9	14.6	9.6	115
7340	11000	1202.0	14.6	7.2	116
7350	11000	1204.0	10.0	9.2	138
7377	11000	1204.9	7.2	10.1	181
7500	11000	1205.6	4.5	10.2	271
7700	11000	1205.7	5.0	9.3	266
7900	11000	1205.9	5.5	8.5	261
8000	11000	1206.0	5.8	8.1	261
8200	11000	1206.3	6.4	7.4	255
8400	11000	1206.8	6.9	6.9	251
8500	11000	1207.1	7.2	6.7	250
8700	11000	1207.7	7.6	6.3	248
8900	11000	1208.5	7.9	6.1	247
9000	11000	1209.0	7.9	6.1	248
9100	11000	1209.4	8.0	6.0	246
9271	11000	1210.2	8.1	6.0	246
9295	11000	1210.2	8.9	5.9	210
9405	11000	1210.6	9.2	5.7	210
9471	11000	1211.4	7.8	6.2	247
9500	11000	1211.6	7.8	6.2	247
9700	11000	1212.4	8.3	6.0	237
9800	11000	1213.0	8.2	6.1	238
10000	11000	1213.9	8.5	6.1	231
10025	11000	1214.2	8.0	6.3	239
10100	11000	1214.5	8.7	6.1	226
10200	11000	1215.0	8.8	6.1	223
10400	11000	1216.1	9.1	6.2	213
10600	11000	1217.3	9.1	6.4	208
10800	11000	1218.5	8.9	6.6	207
10900	11000	1219.0	8.9	6.6	206

Table 3.3 Proposed Conditions 100-Year Hydraulics (continued)

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top-Width (ft)
11000	11000	1219.5	9.5	6.6	196
11095	11000	1220.1	9.5	6.7	194
11100	11000	1222.1	12.4	5.2	187
11150	11000	1223.5	10.1	6.6	185
11230	11000	1224.2	9.5	7.1	184
11238	11000	1224.3	9.4	7.2	184
11305	11000	1224.9	8.6	7.8	188
11310	11000	1224.8	8.8	7.6	187
11400	11000	1225.4	8.0	8.3	191
11500	11000	1225.8	8.1	8.1	194
11700	11000	1226.5	7.7	8.4	195
11900	11000	1227.2	7.4	8.7	197
12000	11000	1227.5	7.4	8.8	197
12200	11000	1228.0	7.3	8.8	198
12400	11000	1228.5	7.2	8.9	199
12500	11000	1228.8	7.1	9.0	199
12645	11000	1229.2	7.1	9.1	199
12650	11000	1229.0	8.2	8.0	193
12660	11000	1229.0	8.1	8.0	193
12784	11000	1229.7	7.3	8.5	209
12822	11000	1229.8	7.1	8.5	213
12853	11000	1229.7	8.1	8.3	188
12884	11000	1229.7	8.9	8.3	194
12981	11000	1229.8	11.3	8.2	160
13050	11000	1231.2	8.7	9.4	144
13131	11000	1231.6	8.4	9.7	144
13206	11000	1231.6	9.8	9.5	153
13300	11000	1232.1	10.2	9.9	151
13400	11000	1232.5	10.9	10.1	125
13500	11000	1233.8	8.0	11.2	146
13600	11000	1234.0	7.9	11.2	146
13700	11000	1234.3	7.9	11.3	147
13800	11000	1234.5	7.9	11.3	146
13900	11000	1234.8	7.8	11.4	147
14000	11000	1235.0	7.8	11.4	147
14100	11000	1235.2	7.7	11.4	148
14200	11000	1235.4	7.8	11.4	147
14300	11000	1235.7	7.7	11.5	148
14400	11000	1235.9	7.7	11.5	148
14500	11000	1236.1	7.8	11.5	146
14700	11000	1236.5	8.3	11.6	149
14900	11000	1237.0	9.0	11.7	140
15000	11000	1237.3	9.3	11.8	136
15200	11000	1238.0	9.2	12.1	135
15400	11000	1238.7	9.7	12.4	129
15495	11000	1239.1	9.4	12.6	131
15500	11000	1237.9	15.0	9.4	106
15510	11000	1239.0	12.8	10.5	113

Table 3.3 Proposed Conditions 100-Year Hydraulics (continued)

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top-Width (ft)
15700	11000	1240.5	13.6	11.1	106
15900	11000	1242.4	13.3	12.0	105
16000	11000	1244.0	11.0	13.1	116
16100	11000	1245.1	9.0	13.7	130
16200	11000	1245.7	7.9	13.9	142
16400	11000	1246.3	6.6	13.5	163
16500	11000	1246.6	6.3	13.3	173
16600	11000	1246.7	6.5	12.9	170
16700	11000	1246.8	6.8	12.5	168
16800	11000	1246.9	6.9	12.2	166
16900	11000	1247.1	7.2	11.9	164
17000	11000	1247.2	8.0	11.5	147
17100	11000	1247.4	8.2	11.2	147
17200	11000	1247.5	9.1	10.9	131
17300	11000	1247.8	9.2	10.7	129
17400	11000	1248.1	9.6	10.5	129
17500	11000	1248.4	9.9	10.3	126
17700	11000	1249.3	10.0	10.2	130
17850	11000	1249.8	10.4	10.1	123
17950	11000	1250.4	10.1	10.1	125
17955	11000	1252.3	13.9	6.5	132
17960	11000	1252.4	13.9	6.5	133
18000	11000	1254.0	11.1	8.0	138
18170	11000	1255.1	12.0	8.1	126
18390	11000	1257.0	10.8	9.0	131
18500	11000	1257.6	11.3	9.1	126
18710	11000	1259.3	9.2	9.8	140
18900	11000	1260.0	9.8	9.0	140
19000	11000	1260.4	10.0	8.9	141
19200	11000	1261.5	9.3	9.5	145
19400	11000	1262.3	9.4	9.3	144
19500	11000	1262.6	10.1	8.8	139
19600	11000	1263.1	10.0	8.9	138
19782	11000	1264.3	8.2	9.8	154
19940	11000	1264.8	8.2	9.5	158
20050	11000	1264.8	8.4	9.4	154
20160	11000	1265.7	6.0	10.3	200
20200	11000	1266.0	5.1	10.5	231
20300	11000	1266.1	5.3	10.1	234
20500	11000	1266.3	5.8	9.4	226
20700	11000	1266.6	6.0	8.7	262
20900	11000	1266.9	6.4	8.1	260
21000	11000	1267.2	6.5	7.9	265
21230	11000	1267.8	7.0	7.4	255
21240	11000	1267.8	7.1	7.3	256
21330	11000	1268.1	7.1	7.2	263
21395	11000	1268.3	10.9	4.3	273
21565	11000	1270.8	6.4	6.8	334

Table 3.3 Proposed Conditions 100-Year Hydraulics (continued)

Section Number	Discharge (cfs)	Computed WSEL (ft)	Velocity (ft/s)	Depth (ft)	Top-Width (ft)
21700	11000	1271.2	6.1	8.5	238
21900	11000	1271.5	6.1	7.9	253
22000	11000	1271.9	4.9	7.8	310
22200	11000	1272.2	5.0	7.1	332
22400	11000	1272.3	7.0	6.3	268
22500	11000	1272.6	7.9	6.1	247
22600	11000	1272.8	10.5	5.8	198
22700	11000	1273.9	10.1	6.4	182
22800	11000	1274.4	11.4	6.5	161
22900	8400	1276.4	7.4	8.0	158
23000	8400	1276.7	7.3	7.8	163
23100	8400	1277.0	7.7	7.6	160
23200	8400	1277.3	7.6	7.5	171
23300	8400	1277.7	7.7	7.3	170
23310	8400	1277.7	7.7	7.4	170
23400	8400	1278.0	8.0	7.2	167
23500	8400	1278.4	8.4	7.1	162
23575	8400	1278.6	9.3	6.9	151
23645	8400	1279.2	8.4	7.6	155
23800	8400	1280.0	7.7	7.3	172
24000	8400	1280.8	7.9	7.1	172
24200	8400	1281.6	7.8	6.9	177
24365	8400	1282.2	8.2	6.8	168
24500	8400	1282.9	7.8	6.8	178
24510	8400	1283.5	11.5	4.4	179
24700	8400	1286.0	7.5	6.9	180
24900	8400	1286.8	7.0	7.7	176
25000	8400	1287.2	6.3	8.1	186
25200	8400	1287.7	6.0	8.3	191
25400	8400	1288.0	7.0	7.4	183
25500	8400	1288.2	7.6	7.0	176
25600	8400	1288.5	8.4	6.7	167
25650	8400	1288.7	8.6	6.6	166
25660	8400	1288.1	11.5	5.0	160
25700	8400	1289.2	9.5	6.1	167
25827	8400	1290.4	7.8	7.3	175
25830	8400	1290.4	7.9	7.2	174
25870	8400	1290.4	8.7	7.0	157
26000	8400	1291.0	9.4	6.8	152
26120	8400	1291.3	10.7	6.3	141
26260	8400	1293.1	8.5	5.6	195
26500	8400	1294.7	6.6	5.7	242
26795	8400	1295.8	7.3	5.8	231
27083	8400	1297.3	8.4	4.3	249
27282	8400	1299.2	10.0	3.2	275
27424	8400	1301.1	8.7	3.6	288
27741	8400	1303.0	6.6	5.0	274
27952	8400	1303.7	6.4	5.5	240

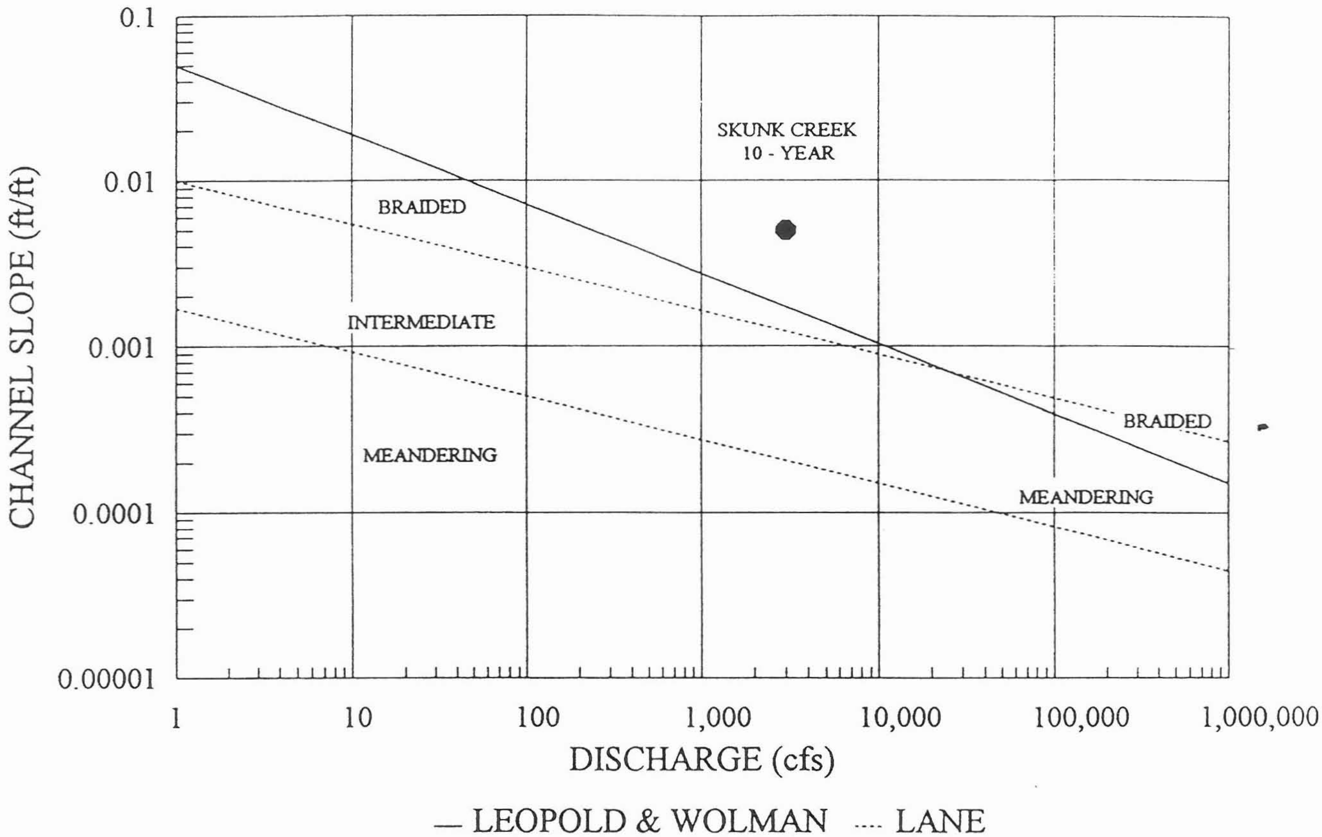


Figure 3.2 Skunk Creek Geomorphic Classification

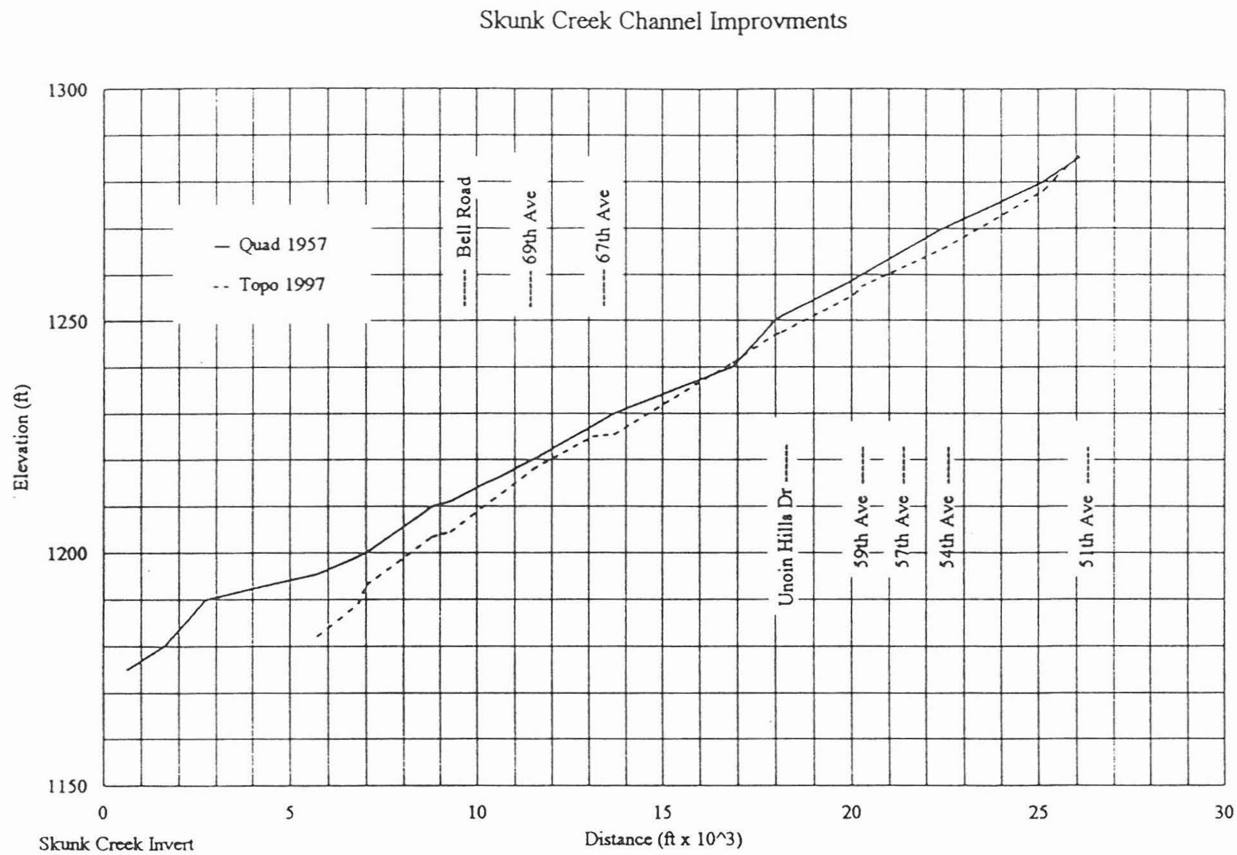


Figure 3.3 Skunk Creek Thalweg Profiles

Simons, Li & Associates, Inc.
Water Resources & Civil Engineering Consultants

Figure 3.4 Velocity Profile for Propsed Conditions 100-Year Discharge

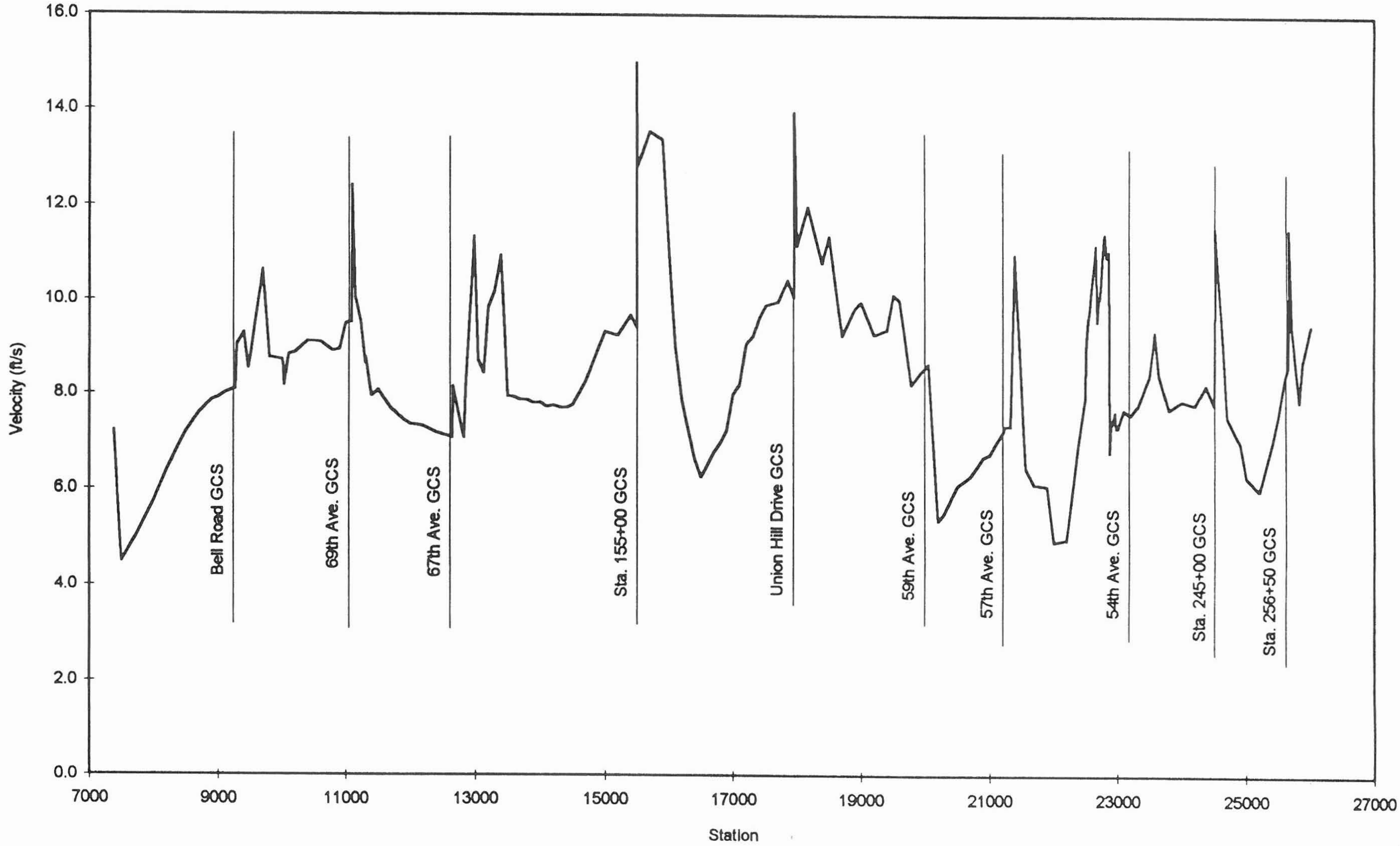
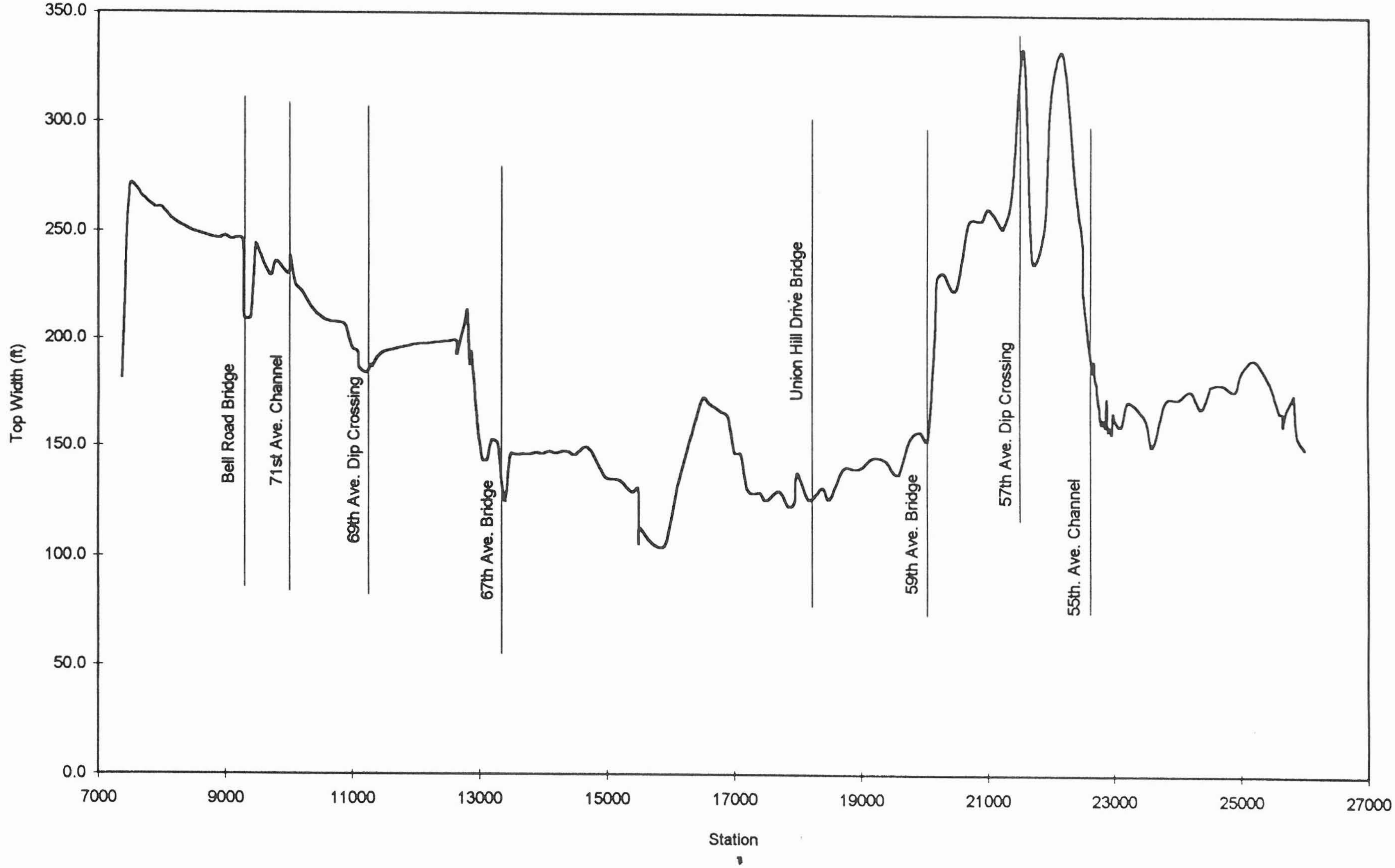


Figure 3.5 Top Width Profile for Proposed Conditions 100-Year Discharge



The sediment transport equations used in the Level II analysis were Meyer-Peter, Muller (MPM) function for bed load and the modified Einstein procedure for suspended bed-material load. Combined, these two functions determine the total sediment load. The MPM and Einstein procedures have been used successfully on rivers with similar channel bed characteristics and are appropriate for this study.

For the short-term river bed response, the degradation and depositional tendencies within the project reach are estimated using the transport capacity of each subreach and the continuity concept for various peak discharges. The sediment outflow from an upstream subreach acts as the inflow to the next downstream subreach. Degradation can be expected in the subreaches where the transport capacity exceeds the upstream supply. Conversely, aggradation can be expected in subreaches where the transport capacity is less than the upstream supply.

Table 3.4 contains the average flow velocities, effective widths, and depths for the proposed conditions 100-year flood for each of the subreaches. Figure 3.6 shows the sediment transport capacities of each subreach for the 100-year flood. Figure 3.7 shows the results of the short-term sediment continuity analysis based on transport capacity and sediment inflow from Scatter Wash with existing sediment supply. The trend for short-term aggradation and degradation, subreach to subreach, is summarized in Table 3.5.

Over long time periods, a river system will adjust to meet the sediment supply provided by upstream reaches. Therefore, in the analysis of long-term bed response the sediment transport capacity of all downstream reaches is compared with the upstream sediment supply reach rather than the subreach immediately upstream. The long-term sediment continuity analysis based on transport capacity and the sediment inflow from the Scatter Wash for existing and future conditions is presented in Figures 3.8 and 3.9, respectively. The same procedures were used for the 10-, 25-, and 50-year floods to observe the channel response to various flood levels. Figures 3.10 and 3.11 present the results.

The Level II long-term analysis indicates that Skunk Creek will tend to degrade for large floods with existing sediment supply and for medium to large floods with the reduced sediment supplies expected in the future.

3.4 Sediment Supply Analysis

Sediment supply from upstream tributaries can have a significant impact on the aggradation and degradation characteristics of alluvial channels. Scatter Wash is the only upstream tributary considered to have the potential of providing a significant sediment supply. Due to the presence of Adobe Dam, it was assumed that the upstream sediment supply from Skunk Creek above the confluence with Scatter Wash will be negligible. Also, the sediment supply from the Arrowhead Drain (north bank, 55th Avenue alignment) will be negligible due to the lake approximately 4500 feet upstream of the Skunk Creek confluence.

The sediment supply to the project reach was estimated using the mapping, hydraulics, and soils data described in Sections 2.1, 2.2, and 2.3 of this report. The Level II analysis, described in Section 3.4, was conducted over a range of discharges to determine the sediment transport characteristics of Scatter Wash. Sediment discharge versus water discharge relationships (Q_s vs. Q_w) were derived for discharges ranging from the 2 to the 100-year flood and are shown as Figure 3.12.

Table 3.4 100-Year Subreach Hydraulics

Subreach Number	Average Depth (ft)	Average Velocity (ft/s)	Effective Width (ft)
1	8.4	12.7	103
2	9.0	11.5	107
3	9.0	11.7	105
4	9.1	11.1	109
5	9.3	11.0	108
6	11.2	7.4	134
7	6.4	14.4	119
8	8.1	8.7	160
9	8.5	4.9	267
10	7.1	6.0	258
11	5.5	8.1	246
12	4.4	10.6	238
13	4.3	10.9	238
14	5.1	10.1	217
15	4.8	9.7	236
16	4.7	10.4	226
17	4.4	11.9	210
18	4.6	11.9	200
19	4.7	12.3	190
20	5.0	12.1	184
21	5.7	10.7	180
22	6.2	9.6	186
23	6.3	9.4	187
24	6.3	9.4	187
25	6.3	9.3	188
26	5.2	11.8	180
27	6.0	9.4	196
28	6.1	12.2	152
29	7.2	10.9	142
30	8.4	8.1	162
31	8.9	6.4	194
32	8.7	6.3	202
33	8.0	8.1	171
34	7.3	10.9	138
35	7.0	13.0	122
36	7.3	13.1	116
37	7.2	14.2	109
38	7.4	14.8	101
39	9.1	9.3	131
40	9.4	7.2	163
41	8.5	8.5	154
42	7.7	11.3	127
43	7.0	13.3	119
44	6.6	14.4	117

Table 3.4 100-Year Subreach Hydraulics (continued)

Subreach Number	Average Depth (ft)	Average Velocity (ft/s)	Effective Width (ft)
45	6.3	13.6	129
46	6.5	13.2	129
47	7.5	11.6	128
48	7.1	11.6	135
49	6.6	12.4	136
50	6.3	13.1	134
51	7.1	11.1	141
52	7.6	9.2	162
53	8.3	5.9	227
54	6.9	7.1	225
55	5.5	8.5	236
56	4.9	10.1	221
57	4.2	9.6	281
58	6.6	6.9	240
59	6.0	5.8	315
60	4.3	10.8	238
61	5.2	12.1	177
62	6.4	8.2	160
63	5.5	9.4	163
64	5.0	11.2	152
65	5.4	9.7	161
66	4.7	10.9	164
67	4.5	10.8	175
68	5.7	8.6	173
69	6.3	7.3	183
70	4.8	10.5	167
71	4.4	12.0	158
72	5.2	10.4	157
73	4.4	10.1	194
74	3.5	10.2	234
75	3.0	9.9	280
76	4.8	6.4	274
77	7.1	5.1	239
78	6.9	4.1	304
79	6.6	3.2	490

Figure 3.6 100-Year Subreach Sediment Transport Capacity

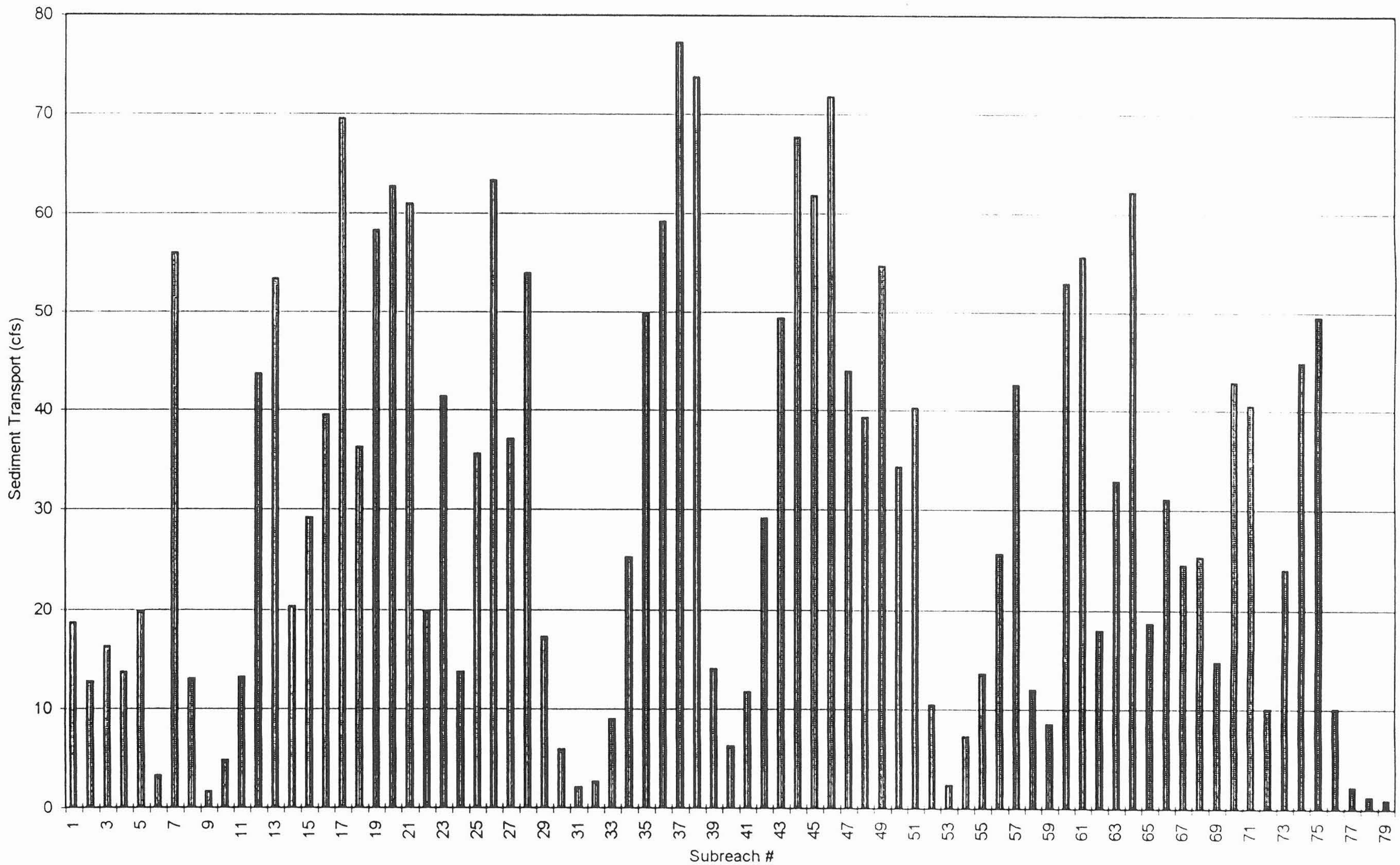


Figure 3.7 100-Year Short-Term Aggradation/Degradation Rate
(Existing Sediment Supply)

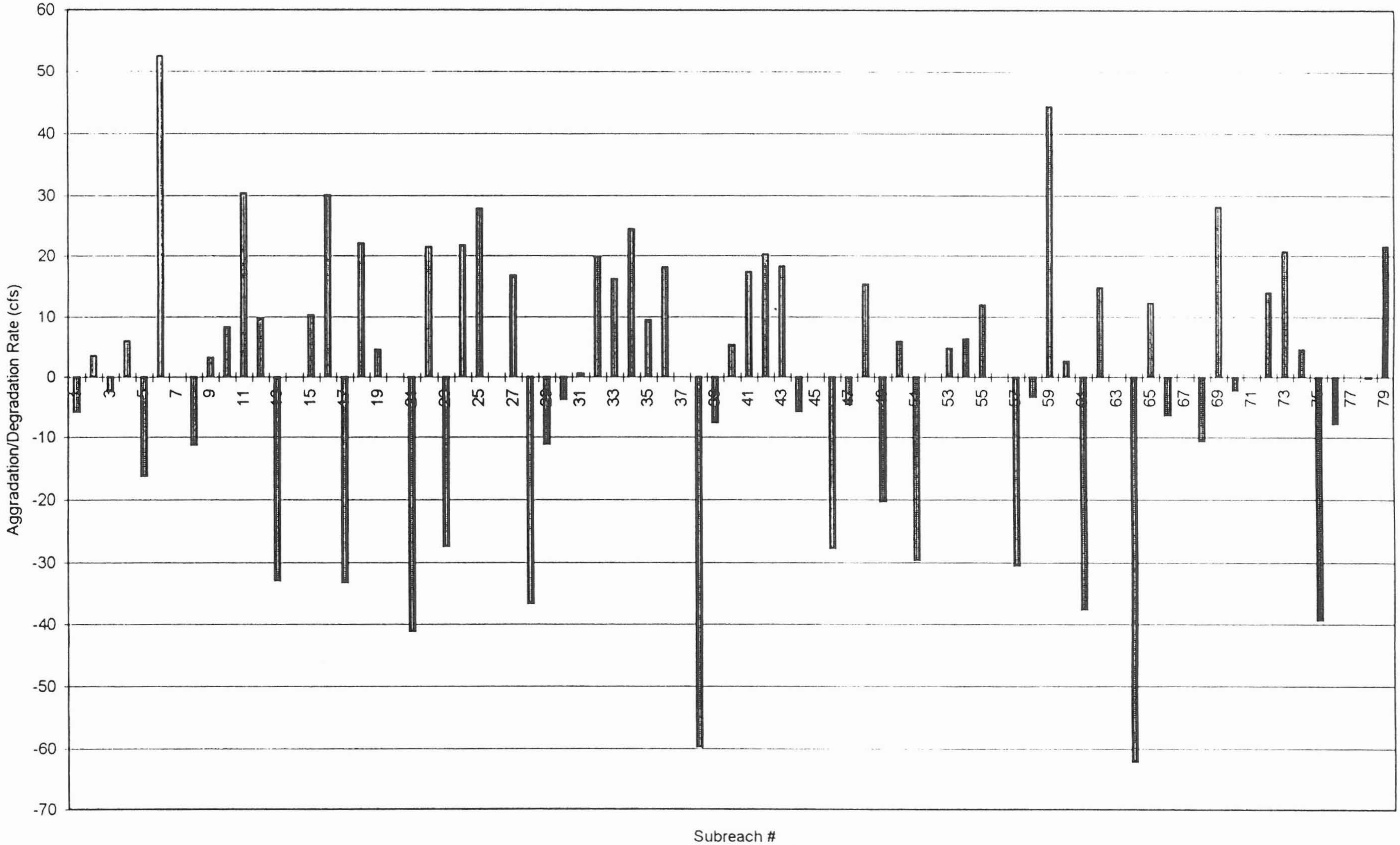


Table 3.5 Estimated Short-Term Aggradation/Degradation Trend

Subreach Number	Station From	Station To	Sediment Transport (cfs)	Aggradation/Degradation Rate (cfs)	Aggradation/Degradation Trend
1	5724	6080	18.7	-5.9	Degradation
2	6080	6380	12.7	3.6	Aggradation
3	6380	6680	16.3	-2.6	Degradation
4	6680	6950	13.8	6.0	Aggradation
5	6950	7250	19.8	-16.4	Degradation
6	7250	7330	3.4	9.7	Aggradation
7	7330	7340	13.1	0.0	Grade Control Structure
8	7340	7490	13.1	-11.4	Degradation
9	7490	7890	1.7	3.2	Aggradation
10	7890	8190	4.9	8.3	Aggradation
11	8190	8550	13.3	30.4	Aggradation
12	8550	8950	43.7	9.7	Aggradation
13	8950	9271	53.4	-33.0	Degradation
14	9271	9471	20.4	8.8	Grade Control Structure
15	9471	9810	29.2	10.3	Aggradation
16	9810	10150	39.5	30.1	Aggradation
17	10150	10500	69.5	-33.3	Degradation
18	10500	10850	36.2	22.0	Aggradation
19	10850	11100	58.2	2.8	Aggradation
20	11100	11150	61.0	0.0	Grade Control Structure
21	11150	11320	61.0	-41.2	Degradation
22	11320	11710	19.8	21.5	Aggradation
23	11710	12010	41.3	-27.5	Degradation
24	12010	12410	13.8	21.8	Aggradation
25	12410	12650	35.5	1.5	Aggradation
26	12650	12660	37.0	0.0	Grade Control Structure
27	12660	12860	37.0	16.9	Aggradation
28	12860	13060	53.9	-36.7	Degradation
29	13060	13410	17.2	-11.2	Degradation
30	13410	13650	6.0	-3.8	Degradation
31	13650	13950	2.2	0.5	Aggradation
32	13950	14250	2.7	6.2	Aggradation
33	14250	14450	9.0	16.3	Aggradation
34	14450	14750	25.3	24.4	Aggradation
35	14750	15100	49.7	9.5	Aggradation
36	15100	15500	59.2	14.6	Aggradation
37	15500	15510	73.8	0.0	Grade Control Structure
38	16010	15510	73.8	-59.7	Degradation
39	16010	16310	14.1	-7.7	Degradation
40	16310	16610	6.4	5.4	Aggradation
41	16610	17010	11.7	17.4	Aggradation
42	17010	17350	29.1	20.3	Aggradation
43	17350	17650	49.4	18.3	Aggradation
44	17650	17950	67.7	-67.7	Degradation

Table 3.5 Estimated Short-Term Aggradation/Degradation Trend (continued)

Subreach Number	Station From	Station To	Sediment Transport (cfs)	Aggradation/Degradation Rate (cfs)	Aggradation/Degradation Trend
45	17950	17960	71.8	0.0	Grade Control Structure
46	17960	18200	71.8	-27.8	Degradation
47	18200	18600	44.0	-4.7	Degradation
48	18600	18940	39.2	15.4	Aggradation
49	18940	19240	54.7	-20.4	Degradation
50	19240	19540	34.2	6.0	Aggradation
51	19540	19940	40.2	-37.7	Degradation
52	19940	20160	2.5	0.0	Grade Control Structure
53	20160	20490	2.5	4.8	Aggradation
54	20490	20890	7.3	6.3	Aggradation
55	20890	21230	13.6	29.0	Aggradation
56	21230	21240	42.6	0.0	Grade Control Structure
57	21240	21570	42.6	-30.6	Degradation
58	21570	21910	12.0	-3.4	Degradation
59	21910	22250	8.5	44.4	Aggradation
60	22250	22550	52.9	2.7	Aggradation
61	22550	22875	55.6	-37.7	Degradation
62	22875	23300	17.9	44.3	Aggradation
63	23300	23310	62.2	0.0	Grade Control Structure
64	23310	23610	62.2	-43.6	Degradation
65	23610	24010	18.6	12.4	Aggradation
66	24010	24500	31.0	-5.6	Degradation
67	24500	24510	25.4	0.0	Grade Control Structure
68	24510	24910	25.4	-10.7	Degradation
69	24910	25300	14.7	28.2	Aggradation
70	25300	25650	42.9	-32.8	Degradation
71	25650	25660	10.1	0.0	Grade Control Structure
72	25660	26100	10.1	14.0	Aggradation
73	26100	26650	24.1	20.7	Aggradation
74	26650	27150	44.9	4.7	Aggradation
75	27150	27550	49.5	-39.4	Degradation
76	27550	27952	10.1	-7.8	Degradation
77	27952	27969	2.3	-0.9	Grade Control Structure
78	27969	28270	1.3	-0.3	Degradation
79	28270	29023	1.0	21.6	Aggradation

Figure 3.8 100-Year Long-Term Aggradation/Degradation Rate
(Existing Sediment Supply)

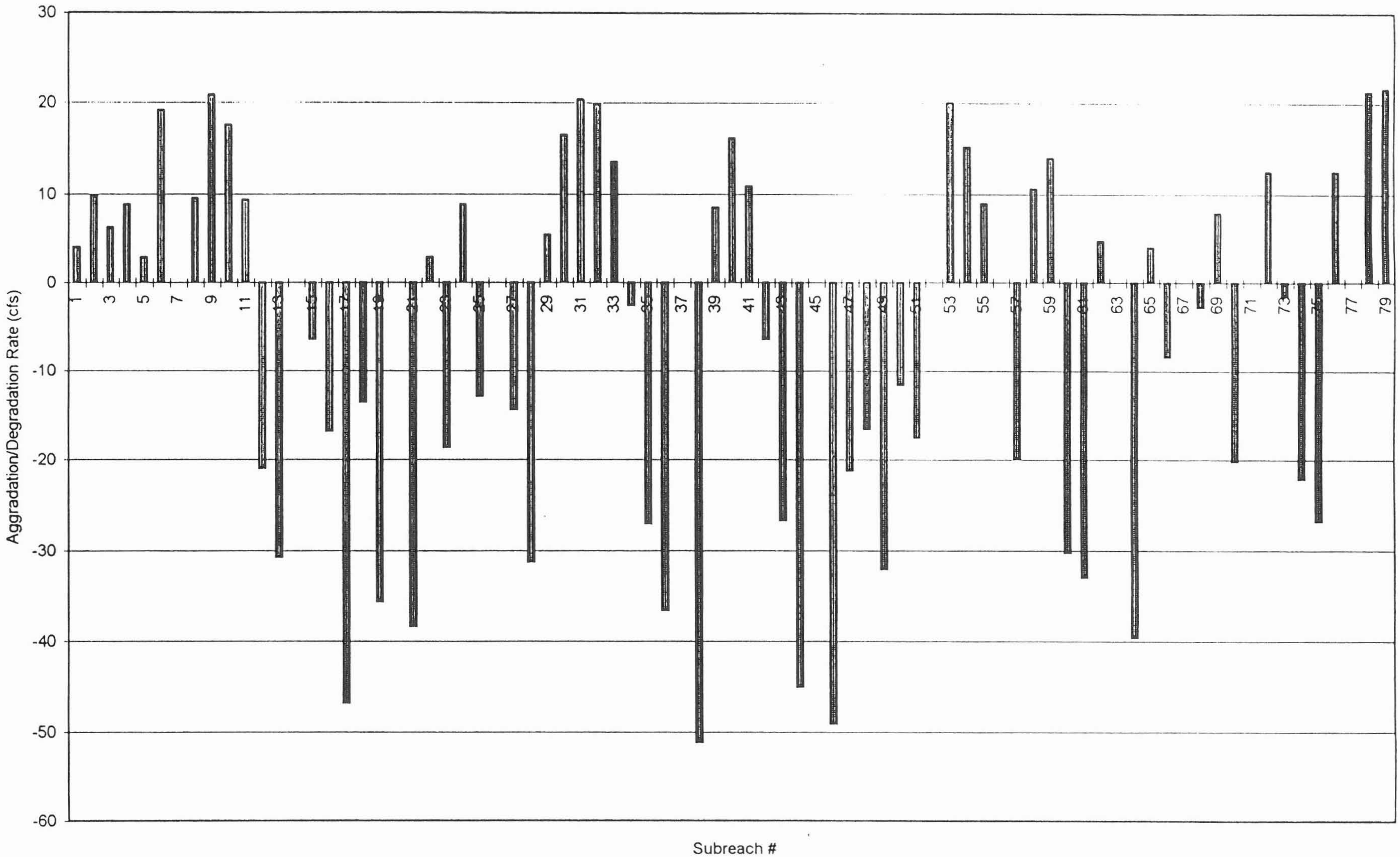


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Figure 3.9 100-Year Long-Term Aggradataion/Degradation Rate
(Reduced Sediment Supply)

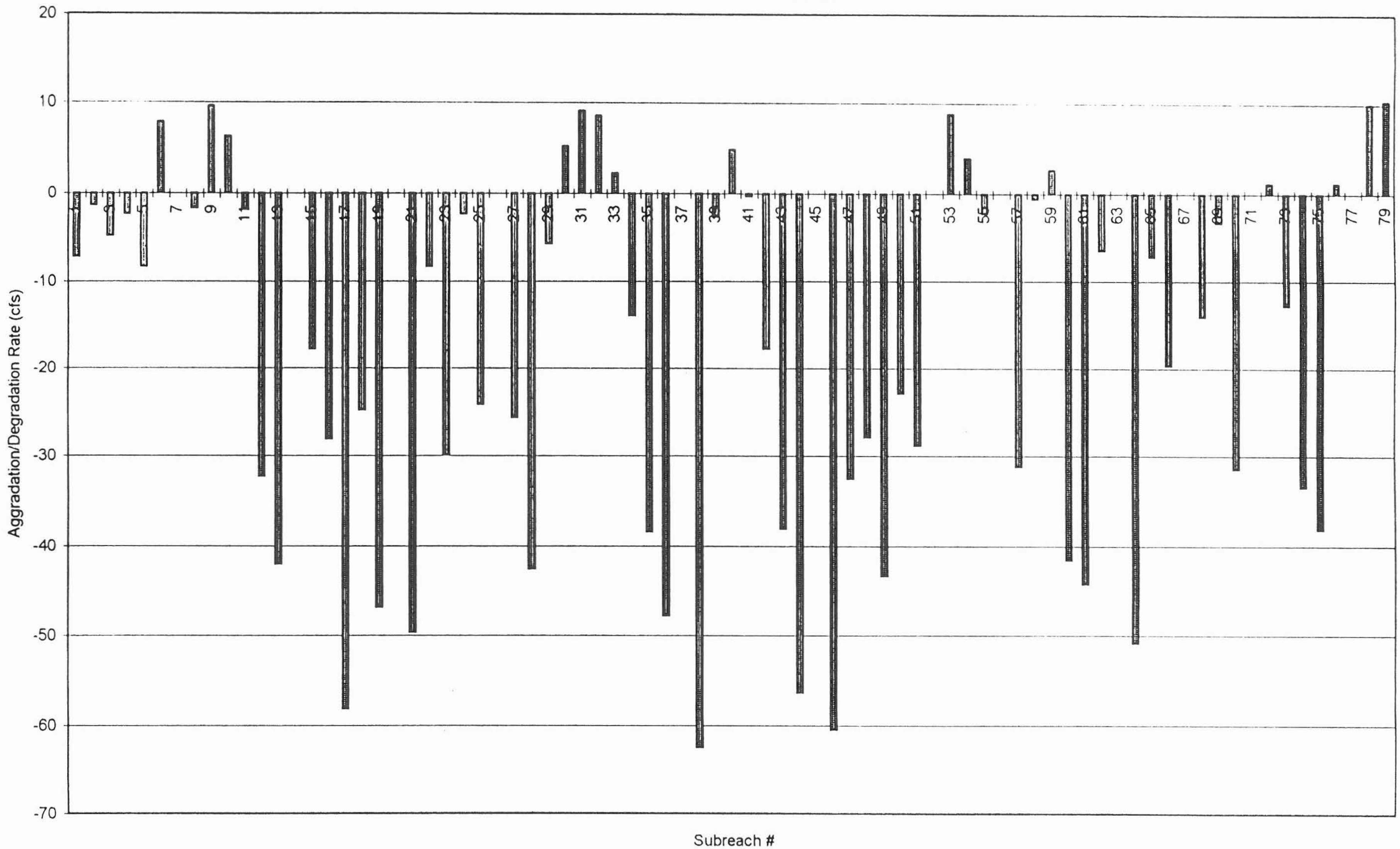

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Figure 3.10 Long-Term Aggradation/Degradation Rates for Peak Discharges
(Existing Sediment Supply)

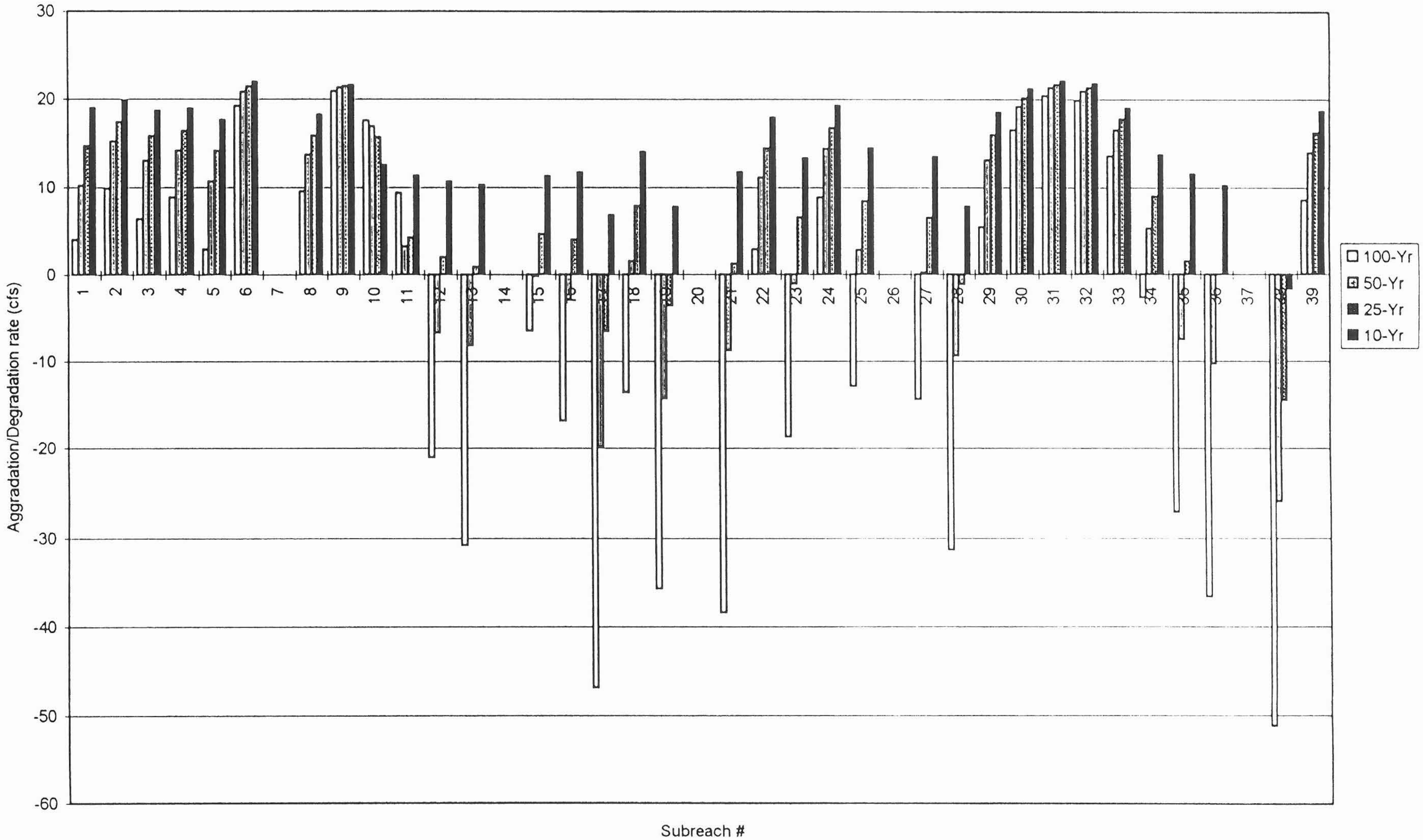

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Figure 3.10 Long-Term Aggradation/Degradation Rates for Peak Discharges
(Existing Sediment Supply)

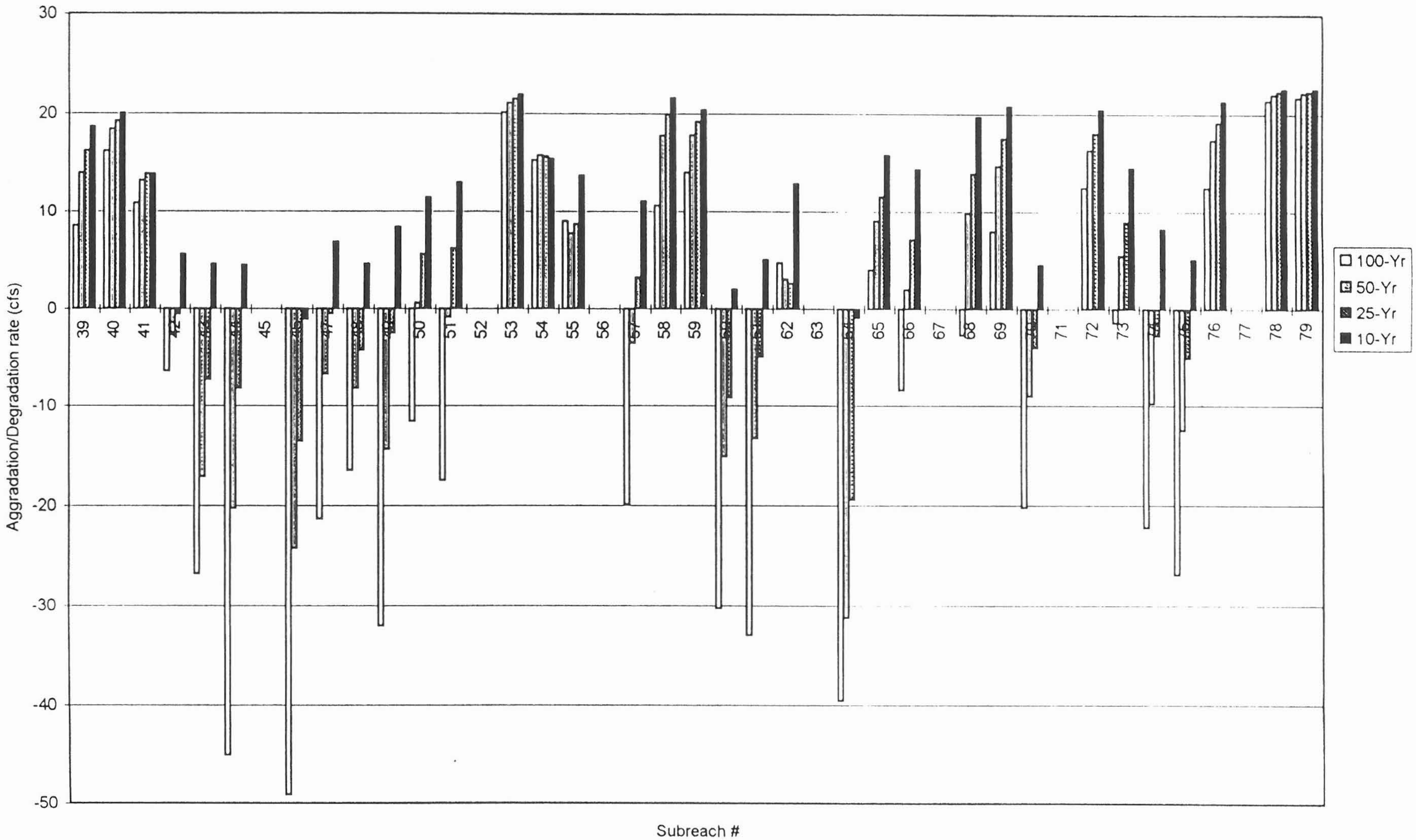

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Figure 3.11 Long-Term Aggradation/Degradation Rates for Peak Discharges
(Reduced Sediment Supply)

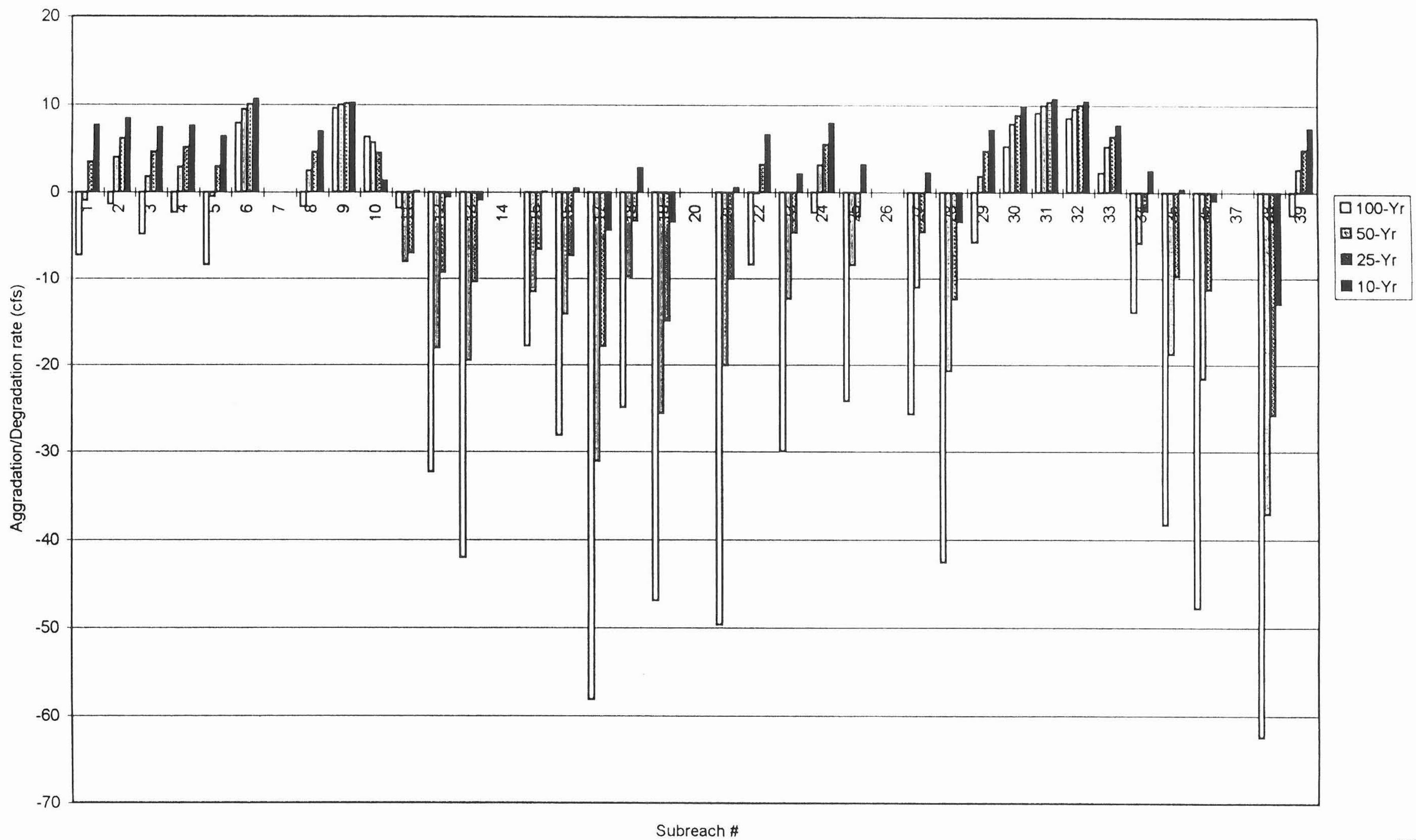


Figure 3.11 Long-Term Aggradation/Degradation Rates for Peak Discharges
(Reduced Sediment Supply)

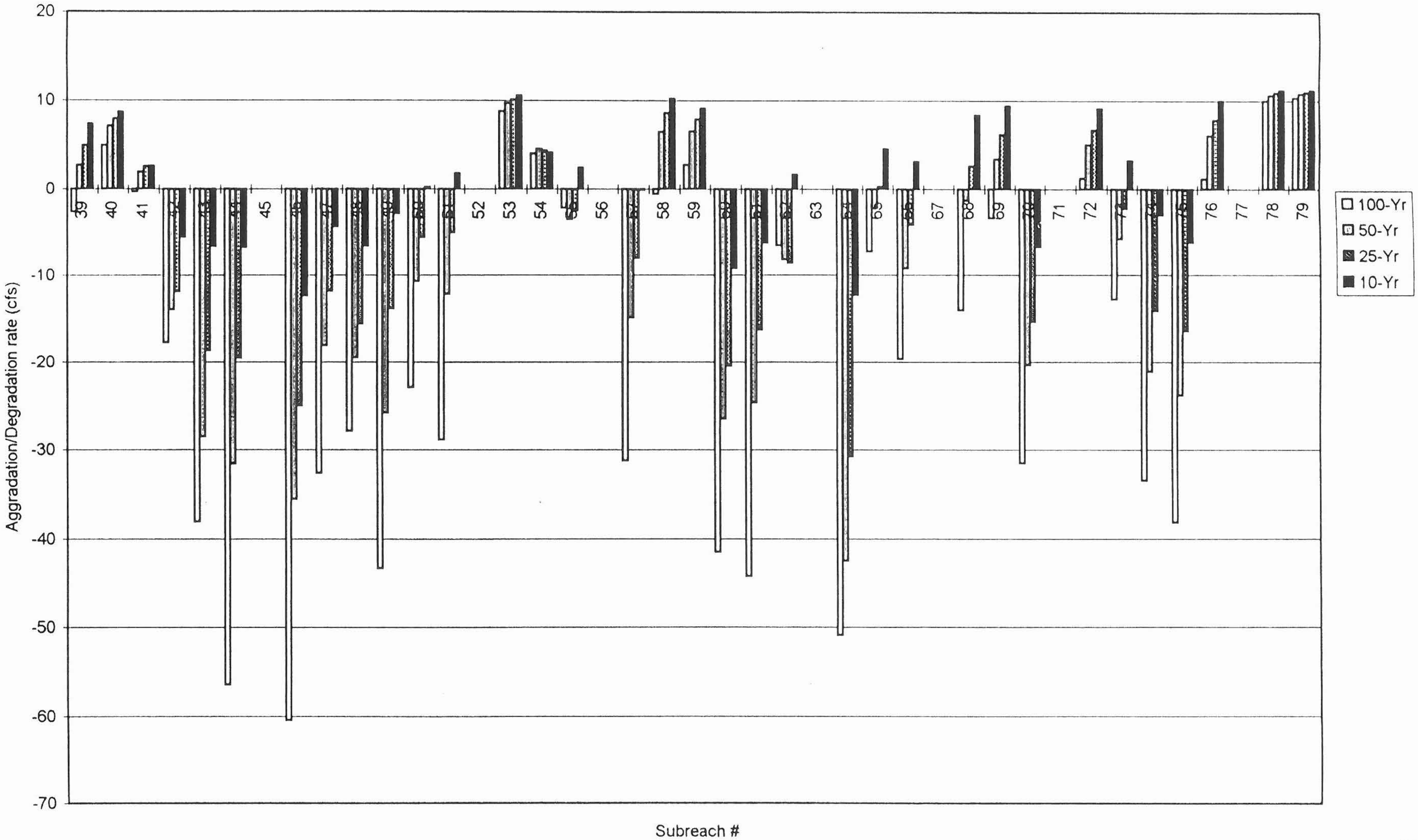
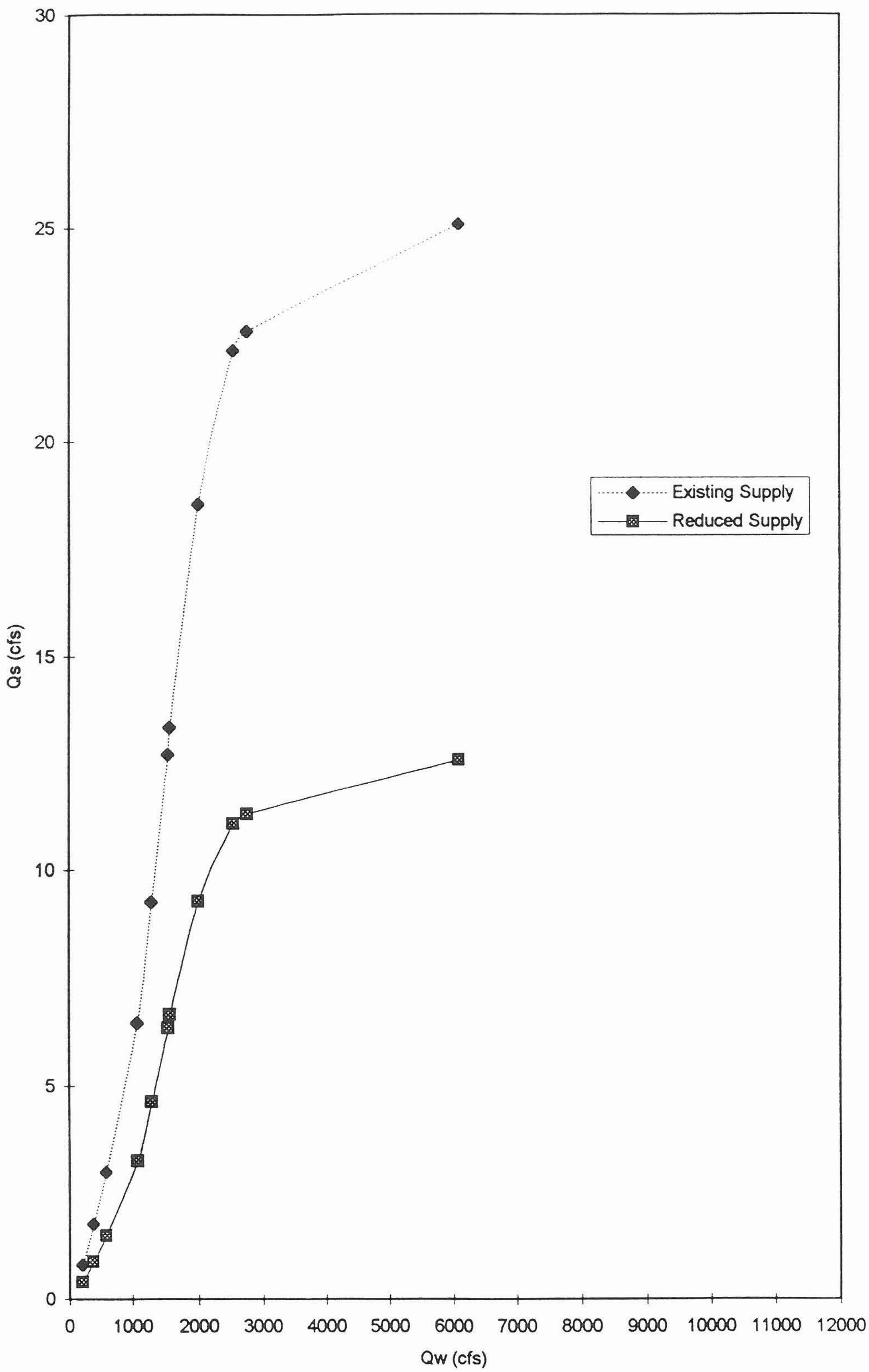


Figure 3.12 Sediment Discharge vs. Water Discharge for Scatter Wash



As shown in the figure, a 50% decrease in sediment supply was evaluated for the future as a worst-case for potential general scour. A regression equation in the form of $Q_s=a*Q_w^{\ b}$ was developed to define these curves for input to the level II and level III analyses for the main channel of Skunk Creek. The supply produced by the previous FIS, 100-year discharge was also estimated, as requested by the District.

3.5 Moveable Bed Modeling (Level III)

The QUASED (QUAsi-dynamic hydraulic and SEDiment routing) computer program developed by SLA was applied to the study reach to quantify the local imbalance between sediment supply and transport capacity along the project reach. The QUASED sediment transport calculations were performed using the hydrograph presented in Figure 2.1 and the characteristic grain size distributions presented in Figure 2.3.

The sediment transport calculations were again performed using the Meyer-Peter, Muller bedload equation and Einstein's procedure for integration of the suspended load to determine the bed material sediment transport capacity. However, in this analysis the computations are done by size-fractions in each gradation, summed to account for the total sample, and compared to the sediment supply. Also, during the QUASED analysis, the channel geometry is adjusted, and the hydraulics and bed material transport are updated after each time step of the discretized 100-year flood hydrograph, shown in Figure 2.1.

The maximum scour occurring at each cross-section during the passage of the flood event was used as the estimated general scour component of the total potential scour. Also, the maximum water-surface elevation occurring during the flood within the QUASED model was compared to the fixed-bed HEC-2 model to establish the maximum water surface for freeboard requirements. Table 3.6 provides a cross-section by cross-section summary of the minimum invert elevations, maximum scour depth, maximum water surface elevation, and maximum depth from the QUASED model.

The model was run for both the estimated existing sediment supply from Scatter Wash and the reduced future sediment supply, as described in Section 3.4 of this report, as well as for the supply generated by the previous FIS, 100-year discharge. The results indicate the magnitude of general scour in the project reach is not significantly affected by a change in the sediment supply from Scatter Wash.

3.6 Scour Components

This section of the report presents the procedures, methodology, assumptions, and results of the scour analyses for the proposed bank protection design. The procedures, methodology, and assumptions used are consistent with those prescribed by the District's Channel Design Criteria for Major Watercourses, February 1994 (9). Several scour components were considered in determining the total scour potential. These are described below. With the exception of long-term degradation, the scour depths used to establish the design toe-down elevations were estimated for the 100-year design flood. Example hand calculations for the scour components are contained in Appendix C of this report.

Table 3.6 QUASED Minimum Invert and Maximum Water-Surface Elevation

Section Number	Minimum Invert Elevation (ft)	Maximum Scour Depth (ft)	Maximum Water-Surface Elevation (ft)	Maximum Water-Surface Depth (ft)
5724	1181.0	0.0	1195.6	14.6
5814	1181.5	0.0	1195.8	14.3
5970	1181.7	0.0	1196.2	14.5
6070	1181.9	0.0	1196.5	14.6
6170	1182.1	0.0	1196.7	14.6
6236	1182.3	0.0	1196.8	14.5
6370	1182.6	0.0	1197.1	14.5
6570	1183.1	0.0	1197.6	14.5
6670	1183.3	0.0	1197.8	14.5
6852	1183.3	0.3	1198.3	15.0
6940	1183.6	0.2	1198.5	14.9
7070	1183.8	0.3	1198.7	14.9
7170	1183.7	0.4	1199.0	15.3
7223	1183.7	0.4	1199.1	15.4
7260	1184.1	0.1	1199.7	15.6
7305	1184.0	0.1	1199.8	15.8
7310	1185.7	0.1	1199.6	13.9
7330	1192.3	0.0	1199.4	7.1
7339	1192.3	0.0	1201.9	9.6
7340	1194.8	0.0	1202.0	7.2
7350	1195.1	0.0	1204.1	9.0
7377	1195.0	0.0	1204.9	9.9
7500	1195.6	0.0	1205.6	10.0
7700	1196.6	0.0	1205.7	9.1
7900	1197.4	0.0	1205.9	8.5
8000	1197.9	0.0	1206.1	8.2
8200	1198.9	0.0	1206.5	7.6
8400	1199.9	0.0	1207.0	7.1
8500	1200.4	0.0	1207.3	6.9
8700	1201.4	0.0	1208.0	6.6
8900	1202.4	0.0	1208.7	6.3
9000	1203.0	0.0	1209.1	6.1
9100	1203.5	0.0	1209.6	6.1
9271	1204.3	0.0	1210.5	6.2
9295	1204.4	0.0	1210.5	6.1
9405	1204.9	0.0	1210.8	5.9
9471	1205.2	0.0	1211.6	6.4
9500	1205.3	0.1	1211.7	6.4
9700	1206.3	0.1	1212.5	6.2
9800	1206.8	0.1	1213.0	6.2
10000	1207.8	0.1	1214.0	6.2
10025	1207.8	0.1	1214.2	6.4
10100	1208.3	0.1	1214.5	6.2
10200	1208.4	0.5	1215.0	6.6
10400	1209.4	0.5	1216.1	6.7

Table 3.6 QUASED Minimum Invert and Maximum Water-Surface Elevation (continued)

Section Number	Minimum Invert Elevation (ft)	Maximum Scour Depth (ft)	Maximum Water-Surface Elevation (ft)	Maximum Water-Surface Depth (ft)
10600	1209.9	0.8	1217.3	7.4
10800	1210.9	0.9	1218.5	7.6
10900	1211.3	1.1	1219.2	7.9
11000	1211.5	1.4	1219.5	8.0
11095	1212.0	1.4	1219.9	7.9
11100	1216.9	0.0	1222.1	5.2
11150	1216.9	0.0	1223.5	6.6
11230	1216.9	0.2	1224.3	7.4
11238	1216.9	0.2	1224.4	7.5
11305	1216.9	0.2	1224.8	7.9
11310	1217.0	0.2	1224.8	7.8
11400	1217.1	0.0	1225.3	8.2
11500	1217.7	0.0	1225.6	7.9
11700	1218.1	0.0	1226.4	8.3
11900	1218.5	0.0	1227.1	8.6
12000	1218.7	0.0	1227.4	8.7
12200	1219.2	0.0	1228.0	8.8
12400	1219.6	0.0	1228.5	8.9
12500	1219.7	0.1	1228.7	9.0
12645	1220.0	0.1	1229.2	9.2
12650	1221.0	0.0	1229.0	8.0
12660	1221.0	0.0	1229.1	8.1
12784	1221.1	0.1	1229.7	8.6
12822	1221.2	0.1	1229.8	8.6
12853	1221.3	0.1	1229.7	8.4
12884	1220.7	0.7	1229.8	9.1
12981	1220.5	1.1	1229.9	9.4
13050	1220.9	0.9	1231.0	10.1
13131	1221.1	0.8	1231.4	10.3
13206	1221.2	0.9	1231.4	10.2
13300	1221.3	0.9	1231.8	10.5
13400	1221.3	1.1	1232.2	10.9
13500	1222.3	0.3	1233.2	10.9
13600	1222.6	0.2	1233.8	11.2
13700	1223.1	0.0	1234.0	10.9
13800	1223.3	0.0	1234.4	11.1
13900	1223.5	0.0	1234.6	11.1
14000	1223.9	0.0	1234.8	10.9
14100	1224.1	0.0	1234.9	10.8
14200	1224.3	0.0	1235.0	10.7
14300	1224.4	0.0	1235.3	10.8
14400	1224.6	0.0	1235.4	10.8
14500	1224.7	0.0	1235.5	10.8
14700	1225.0	0.0	1236.2	11.2
14900	1225.3	0.0	1236.8	11.5
15000	1225.5	0.0	1237.1	11.6

Table 3.6 QUASED Minimum Invert and Maximum Water-Surface Elevation (continued)

Section Number	Minimum Invert Elevation (ft)	Maximum Scour Depth (ft)	Maximum Water-Surface Elevation (ft)	Maximum Water-Surface Depth (ft)
15200	1225.7	0.2	1237.9	12.2
15400	1226.2	0.1	1238.7	12.5
15495	1226.4	0.1	1239.2	12.8
15500	1228.5	0.0	1237.9	9.4
15510	1228.5	0.0	1239.0	10.5
15700	1227.7	1.7	1240.7	13.0
15900	1228.6	1.8	1241.9	13.3
16000	1229.3	1.6	1243.3	14.0
16100	1231.3	0.2	1244.2	12.9
16200	1231.7	0.1	1245.0	13.3
16400	1233.6	0.0	1245.7	12.1
16500	1234.0	0.0	1246.1	12.1
16600	1234.5	0.0	1246.2	11.7
16700	1234.4	0.0	1246.4	12.0
16800	1234.8	0.0	1246.6	11.8
16900	1235.3	0.0	1246.8	11.5
17000	1235.8	0.0	1246.9	11.1
17100	1236.2	0.0	1247.2	11.0
17200	1236.7	0.0	1247.3	10.6
17300	1237.1	0.0	1247.7	10.6
17400	1237.3	0.3	1248.0	10.7
17500	1237.8	0.3	1248.4	10.6
17700	1238.2	0.9	1249.2	11.0
17850	1238.9	0.9	1249.8	10.9
17950	1240.3	0.0	1250.6	10.3
17955	1245.8	0.0	1252.3	6.5
17960	1245.9	0.0	1252.4	6.5
18000	1245.1	0.9	1254.3	9.2
18170	1246.1	0.9	1255.1	9.0
18390	1247.6	0.4	1255.5	7.9
18500	1248.1	0.4	1256.6	8.5
18710	1248.9	0.6	1258.6	9.7
18900	1250.4	0.6	1259.4	9.0
19000	1251.3	0.2	1259.8	8.5
19200	1251.8	0.2	1261.2	9.4
19400	1252.3	0.7	1262.0	9.7
19500	1253.1	0.7	1262.3	9.2
19600	1253.3	0.9	1262.9	9.6
19782	1253.8	0.7	1264.0	10.2
19940	1255.3	0.0	1264.5	9.2
20050	1255.4	0.0	1264.6	9.2
20160	1255.4	0.0	1265.6	10.2
20200	1255.6	0.0	1265.8	10.2
20300	1256.1	0.0	1265.9	9.8
20500	1257.0	0.0	1266.1	9.1
20700	1257.9	0.0	1266.5	8.6

Table 3.6 QUASED Minimum Invert and Maximum Water-Surface Elevation (continued)

Section Number	Minimum Invert Elevation (ft)	Maximum Scour Depth (ft)	Maximum Water-Surface Elevation (ft)	Maximum Water-Surface Depth (ft)
20900	1258.3	0.5	1267.0	8.7
21000	1258.8	0.5	1267.2	8.4
21230	1260.4	0.0	1267.8	7.4
21240	1260.5	0.0	1267.8	7.3
21330	1260.6	0.3	1268.2	7.6
21395	1263.8	0.2	1268.3	4.5
21565	1263.8	0.2	1270.6	6.8
21700	1263.0	0.0	1271.0	8.0
21900	1263.8	0.0	1271.5	7.7
22000	1264.4	0.0	1271.8	7.4
22200	1265.4	0.0	1272.2	6.8
22400	1265.8	0.2	1272.6	6.8
22500	1266.3	0.2	1272.8	6.5
22600	1266.1	0.9	1273.3	7.2
22700	1266.5	1.0	1273.7	7.2
22800	1266.7	1.2	1274.1	7.4
22900	1268.1	0.3	1275.4	7.3
23000	1268.7	0.2	1276.0	7.3
23100	1269.2	0.2	1276.3	7.1
23200	1269.6	0.2	1276.9	7.3
23300	1270.3	0.0	1277.3	7.0
23310	1270.3	0.0	1277.4	7.0
23400	1270.3	0.5	1277.7	7.4
23500	1270.8	0.5	1278.1	7.3
23575	1271.1	0.6	1278.3	7.2
23645	1271.3	0.4	1279.0	7.7
23800	1272.3	0.4	1279.8	7.5
24000	1273.3	0.4	1280.6	7.3
24200	1274.3	0.4	1281.4	7.1
24365	1275.1	0.3	1282.2	7.1
24500	1276.1	0.0	1282.9	6.8
24510	1279.1	0.0	1283.5	4.4
24700	1279.2	0.0	1285.9	6.7
24900	1279.2	0.0	1286.8	7.6
25000	1279.7	0.0	1287.0	7.3
25200	1280.0	0.0	1287.9	7.9
25400	1280.5	0.1	1288.5	8.0
25500	1281.1	0.1	1288.7	7.6
25600	1281.7	0.1	1289.0	7.3
25650	1282.1	0.0	1289.5	7.4
25660	1283.1	0.0	1289.3	6.2
25700	1282.8	0.3	1289.7	6.9
25827	1282.8	0.3	1290.5	7.7
25830	1282.9	0.3	1290.5	7.6
25870	1283.1	0.3	1290.5	7.4
26000	1283.8	0.4	1291.1	7.3

3.6.1 General Scour

General scour refers to the vertical lowering of the entire channel bed over relatively short time periods, typically during the passage of a single flood event. General scour occurs because an increase in slope or decrease in channel width causes the average velocity and bed shear stress to increase. This produces an increase in stream power (τV), therefore, more bed material is transported through the section than is transported into it. As the bed level is lowered, velocity decreases, and shear stress decreases and equilibrium is restored when the transport rate through the section is equal to the incoming rate. The maximum scour from the QUASED output for the 100-year event, described in Section 3.5, was used as the general scour component at each cross-section.

3.6.2 Long-Term Degradation

The procedures described in the Bureau of Reclamation publication, "Computing Degradation and Local Scour" (11) were used to quantify the potential long-term degradation component of total scour. Long-term degradation was computed using the concepts of equilibrium slope and stream-bed armoring with the lesser of the two governing.

The dominant discharge was used for the long-term degradation analysis. The dominant discharge is defined as the discharge which, if allowed to flow constantly, would have the same overall channel shaping effect as the natural fluctuating discharges. The dominant discharge is typically between a 5-year and 10-year event for ephemeral channels of the Southwest. The average hydraulics of the 10-year event were used to determine the long-term degradation response for the project reach of Skunk Creek. All proposed and existing grade control structures were used as pivot points for the equilibrium slope analysis.

3.6.3 Low-Flow Incisement

The natural braiding tendencies of the design project reach of Skunk Creek, described in Section 3.2 of this report, require that consideration be given to the likely development of a low-flow channel or channels after construction. There are no rigorous methodologies for the prediction of low-flow channel incisement. However, a review of existing field conditions and experience from previous projects in Maricopa County indicate a low-flow incisement channel depth of 1.5 feet is appropriate for the project reach.

3.6.4 Bed-Form Scour

The bed-form scour component was estimated to be one-half of the dune or antidune heights. The dune height was calculated using a relationship developed by Allen (12). The antidune height was calculated using relationships developed by Kennedy (13). The actual type of bed form present in the project reach is a function of the flow regime. Since the flow regime will change with the fluctuating discharges of the flood hydrograph, both bed forms could occur during a single flood event. The maximum scour depth calculated from the above two relationships was used as the bed-form scour component of the total potential scour.

3.6.5 Factor of Safety

A factor of safety was included to account for non-uniform flow distributions typical of alluvial channels. This factor of safety is calculated as 30 percent of the sum of the general scour, long-term scour, and bed-form scour as directed by the District's channel design criteria (9).

3.6.6 Standard Toe-Down Depth

The standard toe-down depth is the sum of the long-term degradation, general scour, low-flow incisement, bed form components, and the factor of safety. This calculated depth is adjusted by contraction, local, and bend scour concerns, when applicable, to generate the final calculated toe-down depth. Table 3.7 provides a cross-section by cross-section accounting of the standard depths used to establish the bank protection toe-down elevations.

3.6.7 Local Scour

In this reach of Skunk Creek, two types of local scour were investigated. Local scour at bridge piers and downstream of drop structures. Local scour at bridge piers is due to the acceleration of flow and the development of local flow vortices around the pier obstruction. Local scour over drops, such as grade control structures, is caused by the falling flow jet impinging on the channel bottom. Local scour can be significant and may control the depth of bank protection toe-downs because of the large zone of influence that may be created. The local scour due to bridge piers was computed using the method described in FHWA Manual HEC-18 (10). Two feet was added to the effective pier widths to account for potential debris accumulation. Local scour due to drops over grade control structures was computed using the results of a physical model by SLA (13).

3.6.8 Bend Scour

In sufficiently long and sharp channel bends, secondary currents will develop due to the super-elevation of the flow at the outside of the bend. These currents result in additional scour of the channel bottom at the base of the outer bank. Bend scour was computed using the equation developed by SLA (8).

3.6.9 Toe-Down Adjustments

Table 3.8 provides a summary of the adjustments to the standard toe-down depths used to establish the bank protection toe-down design elevations. These adjustments are necessary at cross-sections where pier scour, drop scour, or bend scour are a concern.

Table 3.7 Standard Toe-Down Depth Summary

Section Number	General Scour (feet)	LONG-TERM SCOUR quilibriu (feet)	LONG-TERM SCOUR Armoring (feet)	Low Flow Scour (feet)	BEDFORM SCOUR Dune (feet)	BEDFORM SCOUR Antidune (feet)	Factor of Safety (feet)	Standard Toe-Down Depth (feet)	Remarks
5724	0.0	12.9	0.0	1.5	0.6	1.9	0.6	4.0	
5814	0	12.9	0.4	1.5	0.5	2.5	0.9	5.2	
5970	0.0	12.9	0.6	1.5	0.6	2.1	0.8	4.9	
6070	0.0	12.9	0.7	1.5	0.6	2.0	0.8	5.1	
6170	0.0	9.1	0.9	1.5	0.6	1.9	0.9	5.2	
6236	0.0	9.1	1.1	1.5	0.6	1.7	0.8	5.1	
6370	0.0	9.1	1.4	1.5	0.6	1.8	0.9	5.6	
6570	0.0	15.4	1.8	1.5	0.6	1.9	1.1	6.3	
6670	0.0	15.4	2.0	1.5	0.6	1.8	1.2	6.5	
6852	0.3	10.6	2.4	1.5	0.6	1.7	1.3	7.2	
6940	0.2	10.6	2.6	1.5	0.7	1.6	1.3	7.2	
7070	0.3	7.6	2.8	1.5	0.6	1.6	1.4	7.6	
7170	0.4	7.6	2.8	1.5	0.7	1.6	1.5	7.8	
7223	0.4	7.6	2.9	1.5	0.7	1.6	1.5	7.8	
7260	0.1	1.2	2.9	1.5	0.7	0.7	0.6	4.2	
7305	0.1	1.2	2.9	1.5	0.7	0.7	0.6	4.2	
7310	0.1	1.2	4.5	1.5	0.6	1.0	0.7	4.5	
7330									Dibble GCS
7339									
7350	0.0	2.3	0.0	1.5	0.5	1.4	0.4	3.3	
7377	0.0	2.3	0.0	1.5	0.5	0.7	0.2	2.4	
7500	0.0	0.4	0.6	1.5	0.5	0.3	0.3	2.7	
7700	0.0	0.4	1.5	1.5	0.5	0.4	0.3	2.6	
7900	0.0	2.6	2.4	1.5	0.4	0.5	0.9	5.2	
8000	0.0	2.6	2.9	1.5	0.4	0.5	0.9	5.6	
8200	0.0	4.6	3.8	1.5	0.3	0.7	1.3	7.3	
8400	0.0	4.6	4.7	1.5	0.3	1.0	1.7	8.7	
8500	0.0	4.6	5.2	1.5	0.3	1.1	1.7	8.9	
8700	0.0	4.7	6.1	1.5	0.2	1.4	1.8	9.5	
8900	0.0	4.7	7.0	1.5	0.2	1.6	1.9	9.7	
9000	0.0	4.4	7.5	1.5	0.2	1.6	1.8	9.3	
9100	0.0	4.4	7.9	1.5	0.2	1.6	1.8	9.3	
9271									Bell GCS
9295									
9500	0.1	4.2	0.1	1.5	0.3	1.0	0.4	3.0	
9700	0.1	4.2	1.1	1.5	0.2	1.4	0.8	4.9	
9800	0.1	4.2	1.5	1.5	0.2	1.5	0.9	5.6	
10000	0.1	5.3	2.4	1.5	0.2	1.8	1.3	7.2	
10025	0.1	5.3	2.4	1.5	0.3	1.2	1.1	6.4	
10100	0.1	5.3	2.9	1.5	0.2	1.7	1.4	7.6	
10200	0.5	9.1	3.4	1.5	0.2	1.8	1.7	8.9	
10400	0.5	9.1	4.3	1.5	0.2	2.0	2.0	10.3	
10600	0.8	9.7	5.2	1.5	0.2	2.0	2.4	11.9	
10800	0.9	9.7	6.1	1.5	0.2	1.8	2.7	13.0	
10900	1.1	16.6	6.6	1.5	0.2	1.9	2.9	14.0	

Table 3.7 Standard Toe-Down Depth Summary (continued)

Section Number	General Scour (feet)	LONG-TERM SCOUR		Low Flow Scour (feet)	BEDFORM SCOUR		Factor of Safety (feet)	Standard Toe-Down Depth (feet)	Remarks
		quilibriu (feet)	Armoring (feet)		Dune (feet)	Antidune (feet)			
11000	1.4	16.6	7.1	1.5	0.2	2.1	3.2	15.2	
11095	1.4	16.0	7.5	1.5	0.2	2.1	3.3	15.8	
11100									69th GCS
11150									
11230	0.2	3.0	0.2	1.5	0.3	1.8	0.7	4.4	
11238	0.2	3.0	0.2	1.5	0.3	1.7	0.6	4.2	
11305	0.2	3.0	0.2	1.5	0.3	1.4	0.5	3.7	
11310	0.2	3.0	0.3	1.5	0.3	1.4	0.6	4.0	
11400	0.0	1.9	0.1	1.5	0.4	1.1	0.4	3.2	
11500	0.0	1.9	0.7	1.5	0.3	1.3	0.6	4.1	
11700	0.0	1.9	1.0	1.5	0.3	1.2	0.7	4.4	
11900	0.0	1.9	1.4	1.5	0.3	1.2	0.8	4.8	
12000	0.0	1.9	1.5	1.5	0.3	1.2	0.8	5.0	
12200	0.0	2.0	2.0	1.5	0.3	1.2	1.0	5.6	
12400	0.0	2.0	2.3	1.5	0.3	1.2	1.0	5.6	
12500	0.1	1.9	2.5	1.5	0.4	1.2	1.0	5.6	
12645	0.1	1.9	2.7	1.5	0.4	1.2	0.9	5.6	
12650									67th GCS
12660									
12784	0.1	2.3	0.2	1.5	0.3	1.2	0.4	3.3	
12822	0.1	2.3	0.3	1.5	0.3	1.1	0.4	3.4	
12853	0.1	2.3	0.4	1.5	0.3	1.5	0.6	4.1	
12884	0.7	16.8	0.3	1.5	0.3	2.2	1.0	5.7	
12981	1.1	16.8	0.5	1.5	0.4	2.4	1.2	6.7	
13050	0.9	16.8	0.7	1.5	0.5	1.1	0.8	5.0	
13131	0.8	5.4	0.8	1.5	0.5	1.0	0.8	4.8	
13206	0.9	5.4	1.0	1.5	0.5	1.5	1.0	5.8	
13300	0.9	5.4	1.0	1.5	0.5	1.7	1.1	6.3	
13400	1.1	5.4	1.2	1.5	0.5	2.2	1.4	7.4	
13500	0.3	1.3	1.4	1.5	0.6	1.0	0.8	4.9	
13600	0.2	1.3	1.6	1.5	0.6	0.8	0.7	4.4	
13700	0.0	0.6	1.7	1.5	0.6	0.6	0.4	3.1	
13800	0.0	0.6	1.9	1.5	0.6	0.6	0.4	3.0	
13900	0.0	0.6	2.1	1.5	0.6	0.5	0.3	3.0	
14000	0.0	0.7	2.3	1.5	0.6	0.5	0.4	3.1	
14100	0.0	0.7	2.4	1.5	0.5	0.5	0.4	3.1	
14200	0.0	0.7	2.6	1.5	0.5	0.6	0.4	3.2	
14300	0.0	2.5	2.8	1.5	0.5	0.8	1.0	5.7	
14400	0.0	2.5	3.0	1.5	0.5	1.0	1.1	6.1	
14500	0.0	19.9	3.1	1.5	0.5	1.5	1.4	7.6	
14700	0.0	19.9	3.4	1.5	0.5	1.7	1.5	8.1	
14900	0.0	24.5	3.7	1.5	0.5	2.2	1.8	9.2	
15000	0.0	24.5	3.9	1.5	0.5	2.4	1.9	9.7	
15200	0.2	21.0	4.3	1.5	0.5	2.1	2.0	10.1	
15400	0.1	21.0	4.6	1.5	0.5	2.5	2.2	10.8	

Table 3.7 Standard Toe-Down Depth Summary (continued)

Section Number	General Scour (feet)	LONG-TERM SCOUR quilibriu (feet)	LONG-TERM SCOUR Armoring (feet)	Low Flow Scour (feet)	BEDFORM SCOUR Dune (feet)	BEDFORM SCOUR Antidune (feet)	Factor of Safety (feet)	Standard Toe-Down Depth (feet)	Remarks
15500									Sta. 155 GCS
15510									
15700	1.7	47.7	0.9	1.5	0.5	3.1	1.7	8.9	
15900	1.8	47.7	1.8	1.5	0.6	3.2	2.0	10.4	
16000	1.6	47.7	2.3	1.5	0.7	2.0	1.8	9.1	
16100	0.2	3.2	2.8	1.5	0.7	1.3	1.3	7.1	
16200	0.1	3.2	3.1	1.5	0.7	1.0	1.3	7.0	
16400	0.0	1.4	4.1	1.5	0.7	0.7	0.6	4.3	
16500	0.0	1.4	4.6	1.5	0.7	0.7	0.6	4.2	
16600	0.0	1.4	5.0	1.5	0.6	0.7	0.6	4.3	
16700	0.0	8.2	5.5	1.5	0.6	0.8	1.9	9.7	
16800	0.0	8.2	5.9	1.5	0.6	0.9	2.0	10.3	
16900	0.0	8.2	6.4	1.5	0.6	1.0	2.2	11.1	
17000	0.0	8.2	6.8	1.5	0.5	1.3	2.4	12.0	
17100	0.0	23.8	7.3	1.5	0.5	1.4	2.6	12.8	
17200	0.0	23.8	7.8	1.5	0.5	1.8	2.9	13.9	
17300	0.0	23.8	8.2	1.5	0.4	1.9	3.0	14.7	
17400	0.3	28.4	8.7	1.5	0.4	2.3	3.4	16.1	
17500	0.3	28.4	9.1	1.5	0.4	2.5	3.6	17.0	
17700	0.9	28.1	10.0	1.5	0.4	2.7	4.1	19.2	
17850	0.9	28.1	10.7		0.4	2.9	4.4	18.9	
17950									Union GCS
17955									
18000	0.9	23.9	0.1	1.5	0.4	2.0	0.9	5.4	
18170	0.9	23.9	1.0	1.5	0.3	2.8	1.4	7.6	
18390	0.4	18.0	2.0	1.5	0.5	1.6	1.2	6.6	
18500	0.4	18.0	2.5	1.5	0.4	2.1	1.5	7.9	
18710	0.6	21.4	3.4	1.5	0.4	1.5	1.7	8.7	
18900	0.6	21.4	4.8	1.5	0.4	2.1	2.3	11.3	
19000	0.2	17.0	5.3	1.5	0.3	2.3	2.4	11.7	
19200	0.2	17.0	5.8	1.5	0.4	1.8	2.3	11.6	
19400	0.7	24.2	6.7	1.5	0.4	2.0	2.8	13.7	
19500	0.7	24.2	7.5	1.5	0.3	2.6	3.2	15.6	
19600	0.9	5.3	7.9	1.5	0.4	2.2	2.5	12.4	
19782	0.7	5.3	8.1	1.5	0.4	1.2	2.2	10.9	
19940									59th GCS
20050									
20200	0.0	0.5	0.0	1.5	0.5	0.4	0.2	2.2	
20300	0.0	0.5	0.5	1.5	0.5	0.5	0.3	2.7	
20500	0.0	2.5	1.4	1.5	0.4	0.6	0.6	4.1	
20700	0	2.5	2.2	1.5	0.4	0.7	0.9	5.4	
20900	0.5	5.2	3.1	1.5	0.3	0.9	1.4	7.5	
21000	0.5	5.2	3.6	1.5	0.3	1.0	1.5	8.1	
21230									57th GCS
21240									

Table 3.7 Standard Toe-Down Depth Summary (continued)

Section Number	General Scour (feet)	LONG-TERM SCOUR quilibriu (feet)	LONG-TERM SCOUR Armoring (feet)	Low Flow Scour (feet)	BEDFORM SCOUR Dune (feet)	BEDFORM SCOUR Antidune (feet)	Factor of Safety (feet)	Standard Toe-Down Depth (feet)	Remarks
21330	0.3	3.4	0.3	1.5	0.2	1.4	0.6	4.2	
21395	0.2	3.4	3.4	1.5	0.2	1.6	1.6	8.3	
21565	0.2	3.4	3.4	1.5	0.3	0.8	1.3	7.2	
21700	0.0	0.3	2.0	1.5	0.4	0.6	0.3	2.7	
21900	0.0	0.3	2.9	1.5	0.3	0.7	0.3	2.8	
22000	0.0	0.3	3.3	1.5	0.3	0.4	0.2	2.5	
22200	0.0	0.3	4.2	1.5	0.3	0.5	0.2	2.5	
22400	0.2	16.7	5.1	1.5	0.2	1.2	1.9	9.9	
22500	0.2	16.7	5.5	1.5	0.2	1.8	2.2	11.2	
22600	0.9	18.2	6.0	1.5	0.2	2.0	2.7	13.1	
22700	1.0	18.2	6.4	1.5	0.3	1.6	2.7	13.2	
22800	1.2	18.2	6.8	1.5	0.3	2.3	3.1	14.9	
22900	0.3	1.9	7.3	1.5	0.4	0.8	2.5	12.4	
23000	0.2	1.9	7.7	1.5	0.4	0.8	2.6	12.9	
23100	0.2	1.9	8.2	1.5	0.3	1.0	2.8	13.7	
23200	0.2	1.9	8.6	1.5	0.3	1.0	3.0	14.3	
23300									54th GCS
23310									
23400	0.5	17.3	0.4	1.5	0.3	1.4	0.7	4.5	
23500	0.5	17.3	0.9	1.5	0.3	1.7	0.9	5.5	
23575	0.6	17.3	1.2	1.5	0.2	2.1	1.2	6.6	
23645	0.4	4.0	1.2	1.5	0.3	1.3	0.9	5.2	
23800	0.4	4.0	2.2	1.5	0.3	1.2	1.1	6.4	
24000	0.4	4.0	3.1	1.5	0.3	1.4	1.5	7.9	
24200	0.4	5.7	4.0	1.5	0.2	1.5	1.8	9.1	
24365	0.3	5.7	4.7	1.5	0.2	1.7	2.0	10.3	
24500									Sta. 245 GCS
24510									
24700	0.0	0.9	0.0	1.5	0.3	1.1	0.3	2.9	
24900	0.0	0.9	0.0	1.5	0.3	0.9	0.3	2.7	
25000	0.0	0.5	0.0	1.5	0.3	0.7	0.2	2.5	
25200	0.0	0.5	0.1	1.5	0.3	0.7	0.2	2.5	
25400	0.1	9.9	1.2	1.5	0.3	1.1	0.7	4.6	
25500	0.1	9.9	1.7	1.5	0.2	1.5	1.0	5.8	
25600	0.1	9.9	2.3	1.5	0.2	2.0	1.3	7.2	
25650									Sta. 256 GCS
25660									
25700	0.3	3.2	0.0	1.5	0.3	1.5	0.5	3.8	
25827	0.3	3.2	0.0	1.5	0.3	1.1	0.4	3.3	
25830	0.3	3.2	0.0	1.5	0.3	1.1	0.4	3.4	
25870	0.3	3.2	0.2	1.5	0.3	1.4	0.6	4.0	
26000	0.4	3.2	0.9	1.5	0.3	2.1	1.0	5.9	

Table 3.8 Toe-Down Adjustment Summary

Section Number	LOCAL SCOUR Bridge Pier (feet)	LOCAL SCOUR Grade Control (feet)	Bend Scour (feet)	Standard Toe-Down Depth (feet)	Adjusted Toe-Down Depth (feet)	Remarks
8000			0.4	5.6	6.0	
8500			0.4	8.9	9.3	
9280		13.1			13.1	Bell Road GCS
9350	10.2					Bell Road Bridge
9700			1.2	4.9	6.1	
9800			1.2	5.6	6.8	
10000			1.2	7.2	8.4	
10100			1.2	7.6	8.8	
10200			1.2	8.9	10.1	
10400			1.2	10.3	11.5	
10600			1.2	11.9	13.1	
11100		21.2			21.2	69th Ave. Crossing GCS
12000			1.6	5.0	6.6	
12500			1.6	5.6	7.2	
12650		13.8			13.8	67th Ave. GCS
13050			0.4	5.0	5.4	
13100	14.1					67th Ave. Bridge
13206			0.4	5.8	6.2	
13400			0.4	7.4	7.8	
13500			0.4	4.9	5.3	
13800			0.4	3.0	3.4	
15500		30.8			30.8	155 GCS (Closed)
17950		24.3			24.3	Union Hills GCS (Closed)
18170			0.2	7.6	7.8	
18300	14.5					Union Hills Bridge
18390			0.2	6.6	6.8	
18710			0.2	8.7	8.9	
19000			0.2	11.7	11.9	
19940		22.4			22.4	59th Ave. GCS (Closed)
20050	14.5					59th Ave. Bridge
21230		9.5			9.5	57th Ave. Crossing GCS
22000			2.3	2.5	4.8	
22200			2.3	2.5	4.8	
22400			2.3	9.9	12.2	

Section Number	LOCAL SCOUR Bridge Pier (feet)	Grade Control (feet)	Bend Scour (feet)	Standard Toe-Down Depth (feet)	Adjusted Toe-Down Depth (feet)	Remarks
23280		16.3			16.3	54th Ave. Crossing GCS
24530		13.8			13.8	245 GCS
25000			0.3	2.5	2.8	
25500			0.3	5.8	6.1	
25600			0.3	7.2	7.5	
25650		11.6			11.6	256 GCS
25870			0.3	9.7	10.0	
26100	14.3					51st Av Bridge

IV. FINAL DESIGN

4.1 Criteria and Constraints

The design criteria and constraints for the Skunk Creek project can be categorized into three groups. Those specified in the scope of work and Master Plan; those identified during the course of project development, and those developed for final design. Those specified in the scope of work are general in nature and include the criteria of the Master Plan Final Report. The major general criteria and constraints include the following:

- Design a soft bottom channel.
- Maximize the use of existing bank protection.
- Use gabions covered by one foot of soil in areas without existing bank protection.
- Stay within the available right-of-way, if possible.
- Protect existing utilities, as required.
- Accommodate future equestrian and multi-purpose pedestrian trails.
- Retain existing storm drain outfalls.
- Provide for maintenance access to all portions of the channel.

A number of additional design criteria and constraints were identified during the course of project development. These included the following:

- Protect Bell Road Bridge pier foundations.
- Accommodate equestrian trails within the channel.
- Accommodate multi-purpose pedestrian trails on the channel banks.
- Provide cross-over ramp grading, as specified by City of Glendale.
- Use Americans with Disabilities Act criteria for multi-purpose pedestrian ramp grading.
- Use 10 percent as maximum grade for equestrian ramps.

Criteria used for final design include the following:

- Protect existing roadway dip crossings and utilities with grade control structures
- Use a combination excavation and horizontal aprons to provide scour protection
- Use estimated long-term degradation plus one foot to determine apron elevations
- Use six feet as a minimum total scour
- Use four feet as a minimum apron depth.

4.2 Channel Geometry

The approach used to develop the general channel geometry was to maximize the channel width, thereby minimizing the potential general scour, while matching existing bank protection and bridge openings. Whenever possible, at least a 22-foot width between the right-of-way and the top of channel bank was provided and 3:1 side slopes were used, as recommended by the Master Plan. Bottoms widths varied accordingly. The design invert for the channel was established by smoothing the existing profile, while meeting the fixed invert elevations at roadway dip crossings, side channels, and storm drain outfall structures.

Since the bank protection is provided by gabion mattresses, the design toe-down depth was achieved through a combination of excavation and horizontal apron. This minimized the cost of excavation for gabion installation. To accommodate Glendale's desire for the gabions to remain covered with one foot of soil, the horizontal apron was buried a minimum of one foot below the estimated long-term channel degradation or 4 feet, whichever was greater. Example calculations and a summary table describing the required thickness of the gabion mattresses used are provided in Appendix F. Figure 4.1 depicts the typical channel section with the proposed toe-down configuration for the gabion mattresses.

4.3 Ramp Configuration and Criteria

Ramps are required on the project for three purposes: 1) channel crossings for access to parks and residential developments; 2) bridge under-crossings to avoid crossing busy streets at grade; and 3) maintenance access to the channel. Figure 4.2 depicts the typical ramp section used for rough grading pedestrian and equestrian ramps. Equestrian ramps will have a maximum 10 percent grade, while pedestrian ramps will comply with the criteria of the Americans with Disabilities Act (ADA). Figure 4.3 illustrates the location of each proposed ramp.

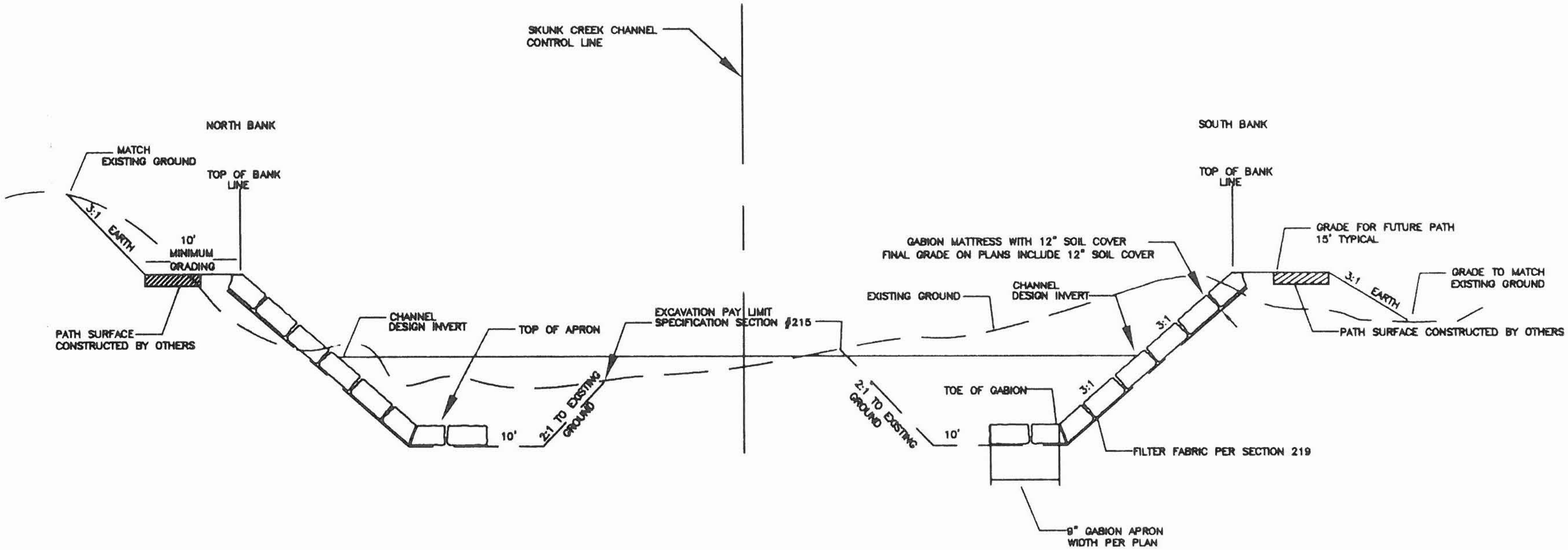
4.4 Grade Control Structures

Because of the importance of existing roadway dip crossings and the utilities under them or nearby, grade control structures were located just downstream of these crossings to minimize long-term degradation of the channel. Dip crossings are located at 69th Avenue, 57th Avenue, and 54th Avenue.

The final sediment transport and scour analyses indicated grade control structures were required to protect bridge foundations and existing bank protection from potential undermining. Grade control structures were included at the Bell Road and 59th Avenue Bridges to protect the foundations from scour. The existing utilities near these bridges were incorporated into the grade control structures to ensure protection.

Grade control structures were located downstream of 67th Avenue, Union Hills Drive, and at Stations 245 and 256 to prevent existing bank protection from being undermined. A final grade control structure was located at Station 155 to prevent excessive long-term degradation. Figure 4.4 shows the locations of these grade control structures.

The configuration used for the typical grade control structure is presented in longitudinal section in Figure 4.5. This type is simple to construct, yet very effective, and it allows a direct comparison of costs for different armor materials. In addition, the hydraulic performance is virtually identical regardless of armor material. Upstream armor extends below the design invert a distance equal to the standard toe-down depth for the bank protection, while the downstream armor extends down an additional distance equal to the local scour potential due to the grade control drop. The depth assumes long-term degradation has occurred. Armor extends laterally and ties into the bank protection to provide continuous protection and prevent undermining.



TYPICAL CHANNEL SECTION
LOOKING UPSTREAM

NO.	REVISION	BY	DATE
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FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
ENGINEERING DIVISION

SKUNK CREEK CHANNEL IMPROVEMENTS
FINAL DESIGN
FCD #95-38

	BY	DATE
DESIGNED	M. Lisotta	7/98
DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

sla SIMONS, LI & ASSOCIATES, INC.

FIGURE 4.1
DESIGN CHANNEL TYPICAL SECTION

SHEET OF

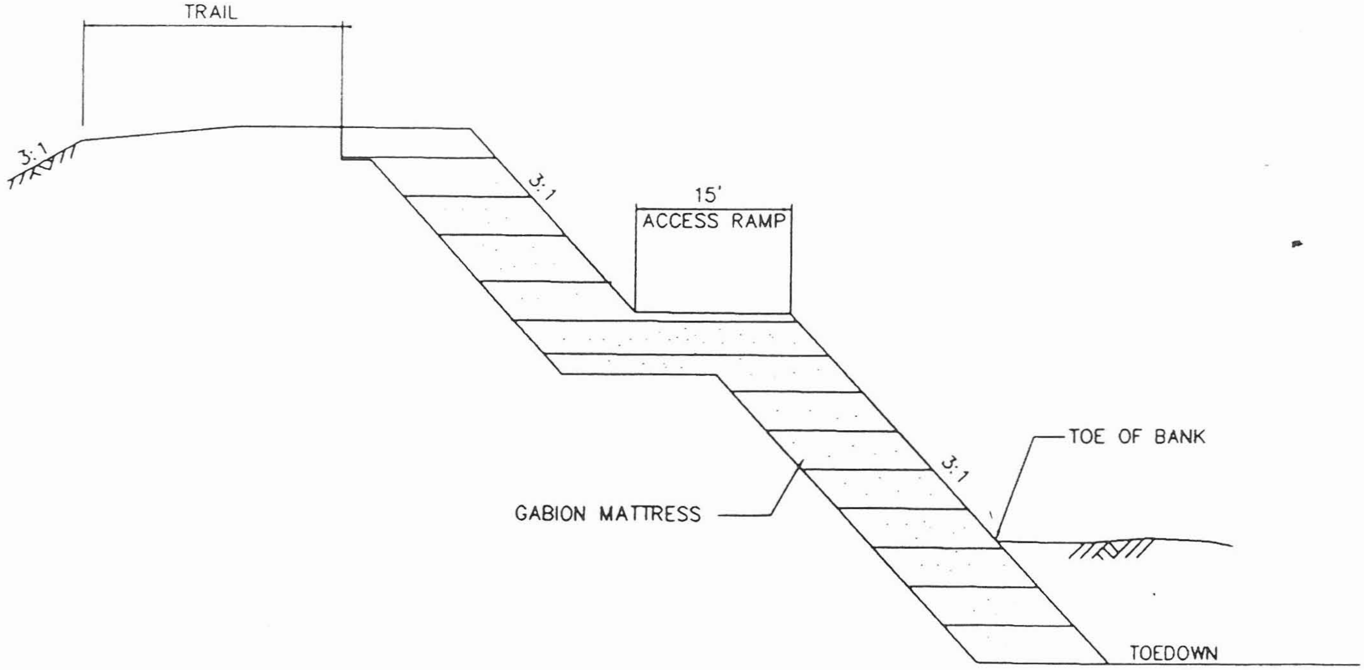
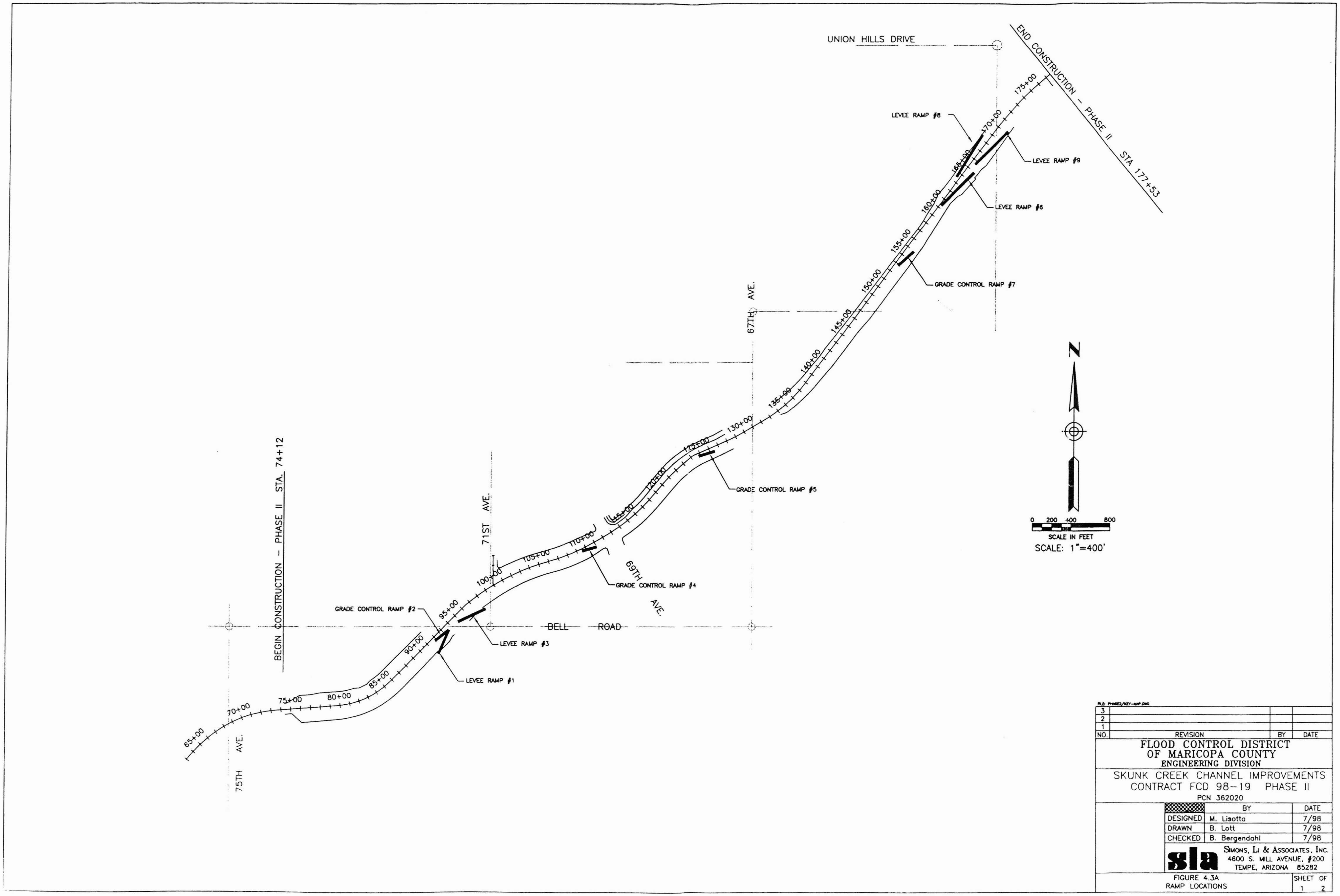


Figure 4.2 Typical Ramp Section (nts)


UNION HILLS DRIVE
END CONSTRUCTION - PHASE II STA 177+53
LEVEE RAMP #8
LEVEE RAMP #9
LEVEE RAMP #6
GRADE CONTROL RAMP #7
67TH AVE.
GRADE CONTROL RAMP #5
GRADE CONTROL RAMP #4
69TH AVE.
71ST AVE.
BEGIN CONSTRUCTION - PHASE II STA. 74+12
GRADE CONTROL RAMP #2
LEVEE RAMP #3
LEVEE RAMP #1
BELL ROAD
75TH AVE.
65+00
70+00
75+00
80+00
85+00
90+00
95+00
100+00
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110+00
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SCALE IN FEET
SCALE: 1"=400'
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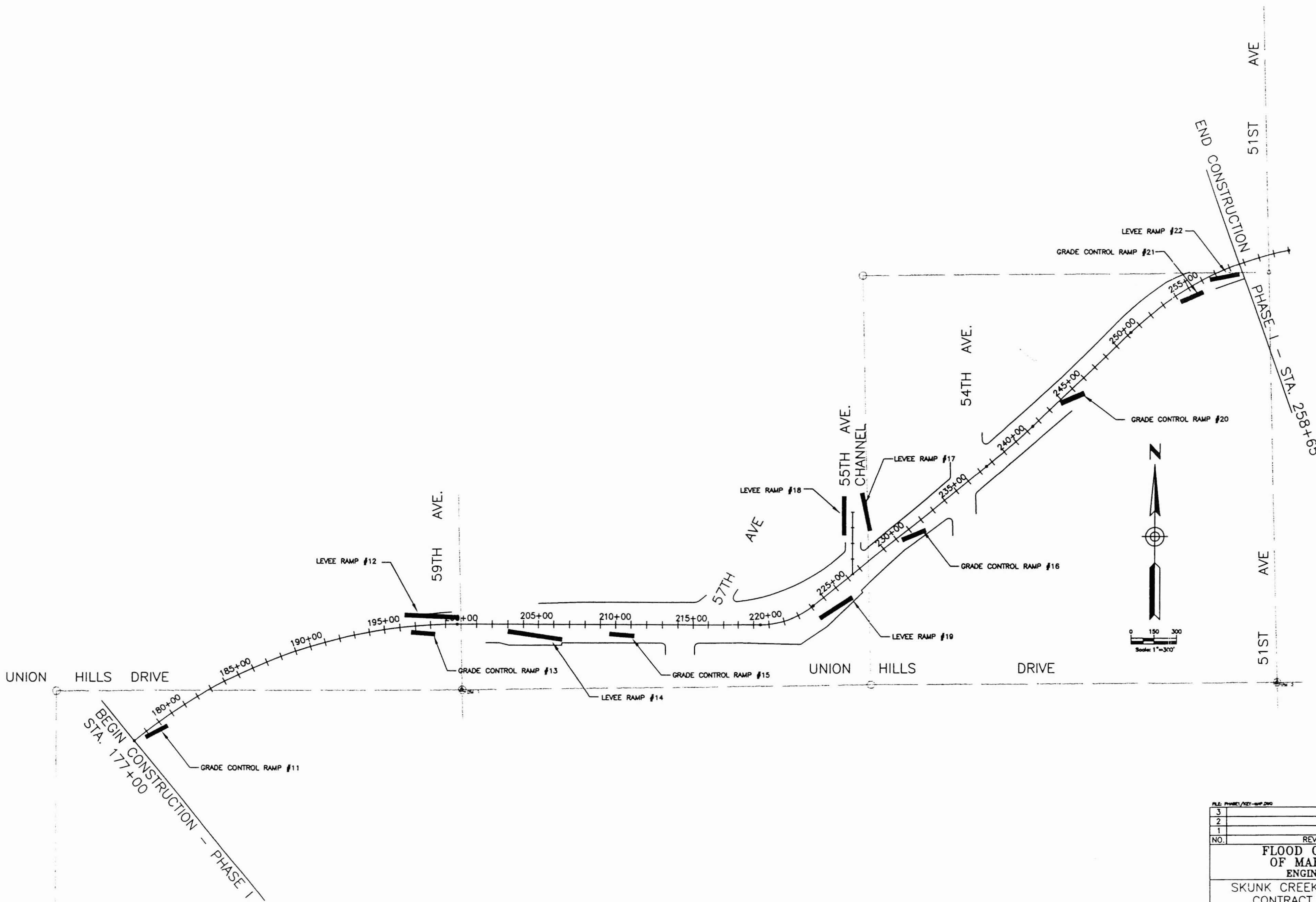
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
ENGINEERING DIVISION
SKUNK CREEK CHANNEL IMPROVEMENTS
CONTRACT FCD 98-19 PHASE II
PCN 362020
BY DATE
DESIGNED M. Lisotta 7/98
DRAWN B. Lott 7/98
CHECKED B. Bergendahl 7/98
sla
SIMONS, LI & ASSOCIATES, INC.
4600 S. MILL AVENUE, #200
TEMPE, ARIZONA 85282
FIGURE 4.3A
RAMP LOCATIONS
SHEET 1 OF 2



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FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
ENGINEERING DIVISION

SKUNK CREEK CHANNEL IMPROVEMENTS
CONTRACT FCD 98-06 PHASE I
PCN 362010

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DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

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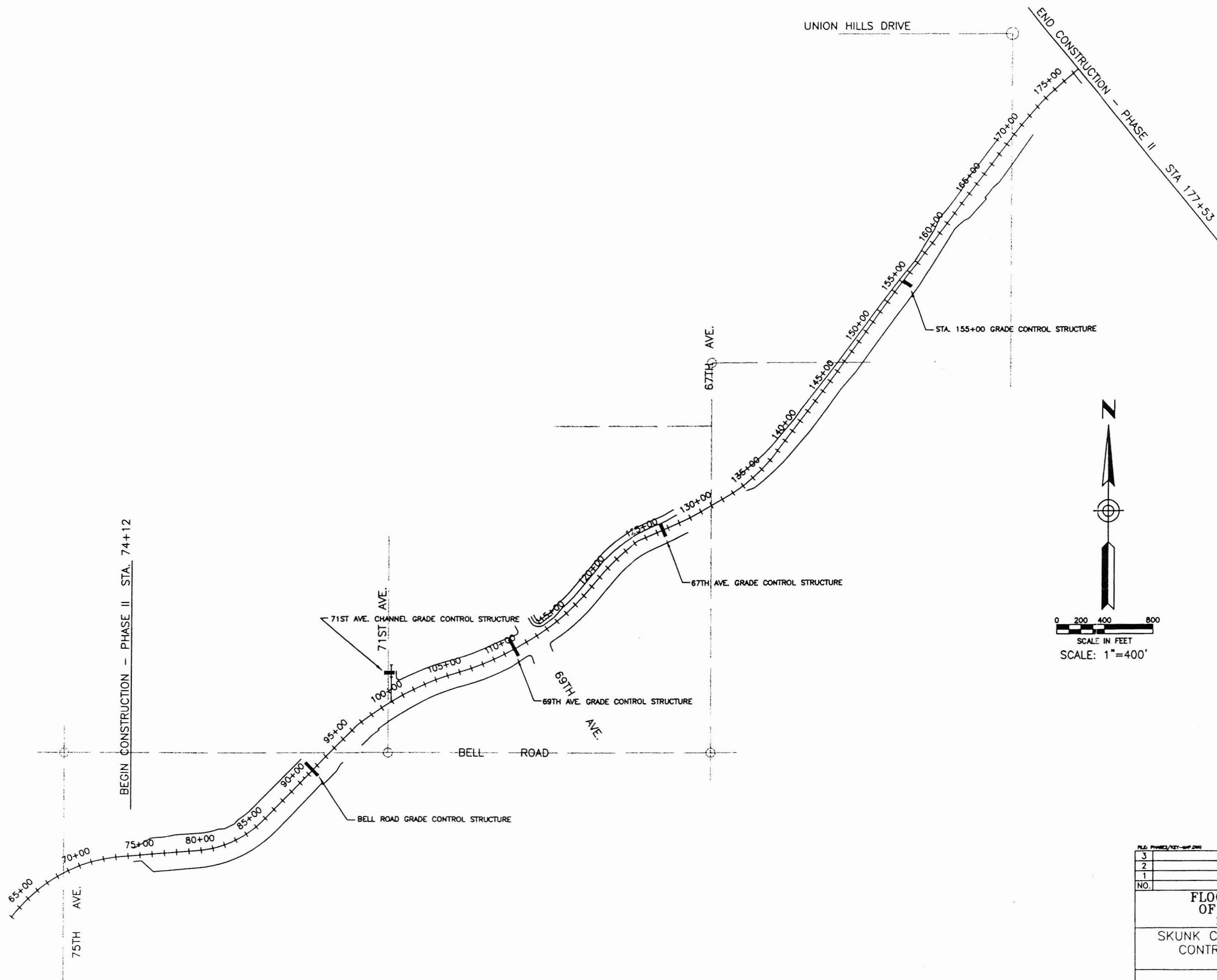
TEMPE, ARIZONA 85282

FIGURE 4.3B
RAMP LOCATIONS

SHEET 2 OF 2



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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
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SKUNK CREEK CHANNEL IMPROVEMENTS
CONTRACT FCD 98-19 PHASE II
PCN 362020

	BY	DATE
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DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

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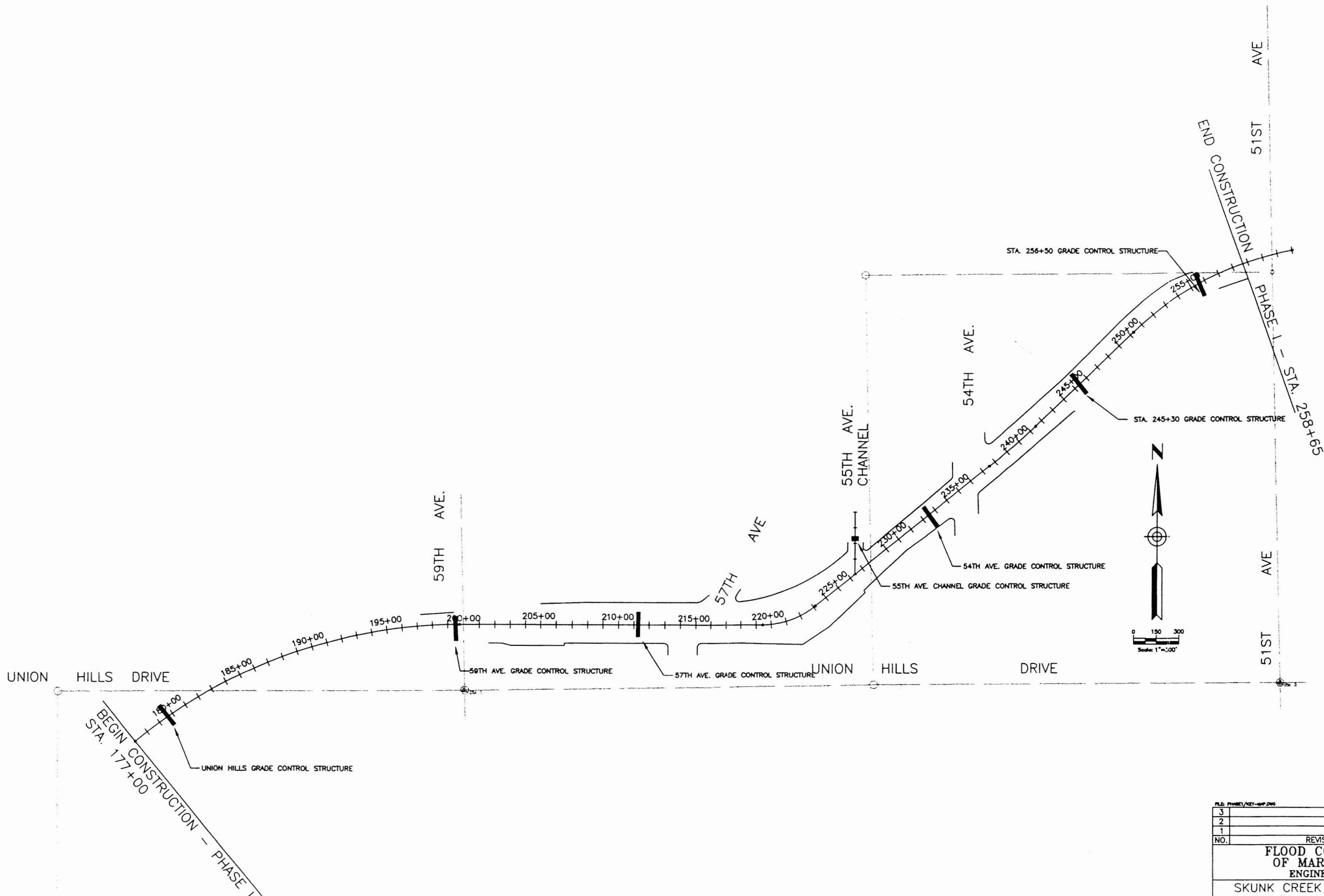
FIGURE 4.4A
GRADE CONTROL STRUCTURE LOCATIONS

SHEET 1 OF 2



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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
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SKUNK CREEK CHANNEL IMPROVEMENTS
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PCN 362010

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CHECKED	B. Bergendahl	7/98

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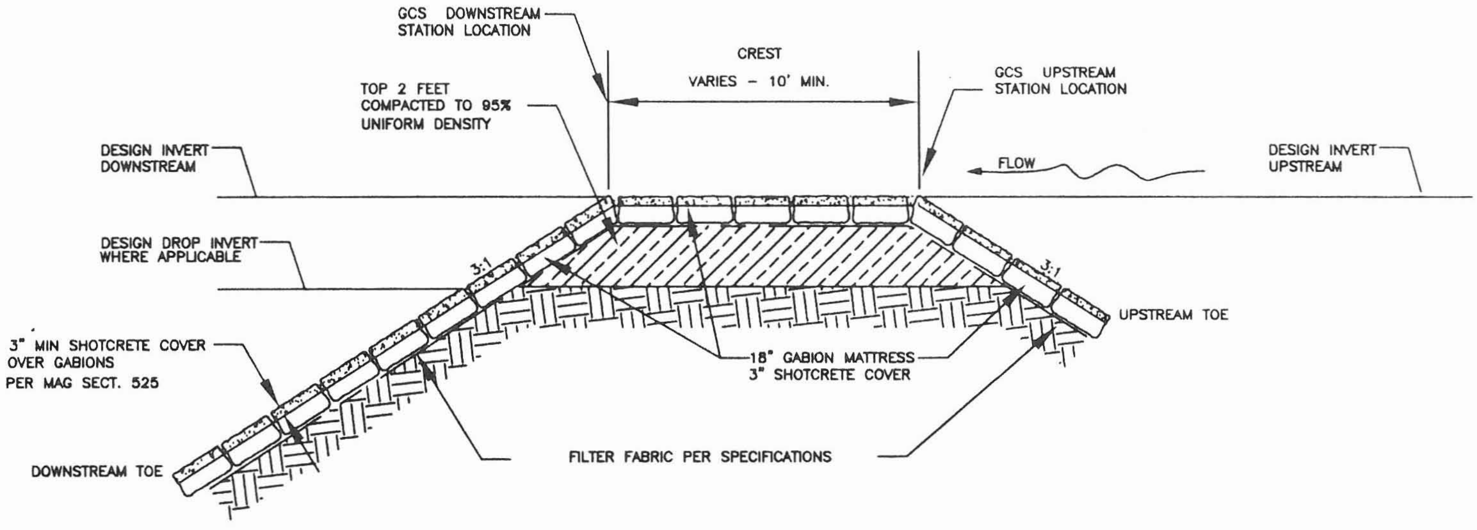
FIGURE 4.4B
GRADE CONTROL STRUCTURE LOCATIONS

SHEET 2 OF 2



TYPICAL GRADE CONTROL STRUCTURE

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1			

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
ENGINEERING DIVISION

SKUNK CREEK CHANNEL IMPROVEMENTS
FINAL DESIGN
FCD #95–38

	BY	DATE
DESIGNED	M. Lisotta	7/98
DRAWN	B. Lott	7/98
CHECKED	B. Bergendahl	7/98

sla Simons, Li & Associates, Inc.

FIGURE 4.5
GRADE CONTROL STRUCTURE TYPICAL SECTION

SHEET OF

A geotechnical stability analysis was conducted on the design grade control structure to determine the safety factors against failure by piping, overturning, uplift, and sliding. The analysis was conducted for a minimum crest length of ten feet and a worst-case head differential across the structure. The results indicated the proposed configuration meets or exceeds a safety factor of 1.5 for all modes of failure analyzed. A copy of the final geotechnical analysis report is contained in Appendix E.

Using the typical section shown in Figure 4.5, costs per unit width of grade control were estimated for four types of armor material: 1) cast-in-place, reinforced concrete (3000 psi); 2) articulating concrete block revetment; 3) gabion mattress; and, 4) gabion mattress with a shotcrete cover for abrasion and corrosion resistance. The comparison is summarized in Table 4.1. Because the costs are based on both square foot and cubic yard measurements, a slope length unit of 27 feet was used for the comparison. Because of the relatively small armor quantities, cement stabilized alluvium and soil cement were not considered.

Table 4.1 Unit Width Cost of Grade Control Armor

Armor Type	Unit Cost	Thickness	Unit Slope Length Cost
Reinforced Concrete (3000 psi)	$160/cu.yd.	1 foot	$160
Articulating Concrete Block	$6.50/sq.ft.	N/A	$175
Gabion Mattress	$75/cu.yd.	1.5 foot	$112.50
Gabion Mattress with Shotcrete	$75/cu.yd. $13/sq.yd.	1.5 foot 0.25 feet	$151.50

The cost comparison shows that gabions are the least expensive of the alternatives considered, even with the shotcrete cover. The unit cost reflects the large quantities of gabions which will be used as bank protection. Building all required grade control structures at this time will take advantage of this low unit cost. "Closed bottoms" were provided on the downstream side of grade control structures which could not be constructed to the estimated full-scour depth, due to narrowness of the channel and the 3:1 side slopes.

The grade control structures are to buried to the design invert level following construction. At structures without a design drop, the crest elevation will be flush with both the upstream and downstream channel invert. As long-term degradation and local scour occur, the downstream face of the grade control structures will begin to be exposed. Therefore, aesthetics was a consideration in selecting the final design armor type. Of the alternatives considered, gabions would provide the most natural appearing armor. However, due to higher hydraulic forces, the higher potential for impact forces from cobbles and boulders, and the higher potential for abrasion from moving sediment, a three-inch layer of shotcrete was provided to protect the wire. The shotcrete will also provide corrosion protection for the wire.

4.5 Toe-Down Elevations

By subtracting the minimum toe-down depth from the design channel invert elevation, the minimum toe-down elevation is obtained. A minimum toe-down depth of six feet was used for design. The maximum design toe-down depths for each cross-section are summarized in Table 4.2.

Table 4.2 Design Toe-Down Depth and Apron Width Summary

Station Number	Design Invert Elevation (feet)	Standard Toe-Down Depth (feet)	Minimum Design Toe-Down Depth (feet)	Design Apron Depth (feet)	Design Apron Width (feet)	Remarks
74+12	1194.9	2.4	6.0	4.1	5	
75+00	1195.4	2.7	6.0	4.2	5	
77+00	1196.4	2.6	6.0	4.5	5	
79+00	1197.4	5.2	6.0	4.8	5	
80+00	1197.9	5.6	6.0	4.9	5	
82+00	1198.9	7.3	7.3	5.2	9	
84+00	1199.9	8.7	8.7	5.5	9	
85+00	1200.4	8.9	8.9	5.7	9	
87+00	1201.4	9.5	9.5	6.0	9	
89+00	1202.4	9.7	9.7	5.9	9	
90+00	1202.9	9.3	9.3	5.8	9	
91+00	1203.4	9.3	9.3	5.8	9	
						Bell Road GCS
95+00	1205.4	3.0	6.0	4.0	5	South Bank
97+00	1206.4	4.9	6.0	4.0	5	South Bank
98+00	1206.9	5.6	6.0	4.0	5	South Bank
100+00	1207.9	7.2	7.2	4.0	9	South Bank
100+25	1208.0	6.4	6.4	4.0	9	
101+00	1208.4	7.6	7.6	4.0	9	
102+00	1208.9	8.9	8.9	4.5	14	
104+00	1209.9	10.3	10.3	5.5	14	
106+00	1210.9	11.9	11.9	6.5	14	
108+00	1211.9	13.0	13.0	7.5	14	
109+00	1212.4	14.0	14.0	17.4	17	
110+00	1212.9	15.2	15.2	19.7	17	
						69th Ave. GCS
112+30	1217.0	4.4	6.0	4.4	5	
112+38	1217.0	4.2	6.0	4.4	5	
113+05	1217.1	3.7	6.0	4.3	5	
113+10	1217.1	4.0	6.0	4.3	5	
114+00	1217.3	3.2	6.0	4.3	5	
115+00	1217.5	4.1	6.0	4.3	5	
117+00	1218.0	4.4	6.0	4.2	5	
119+00	1218.4	4.8	6.0	4.1	5	
120+00	1218.6	5.0	6.0	4.1	5	
122+00	1219.1	5.6	6.0	4.1	5	
124+00	1219.5	5.6	6.0	4.0	5	
125+00	1219.7	5.6	6.0	4.0	5	
126+45	1220.1	5.6	6.0	3.9	5	
						67th Ave. GCS
127+84	1221.3	3.3	6.0	4.0	5	
128+22	1221.3	3.4	6.0	4.0	5	
						Existing Bank Protection
135+00	1222.7	4.9	6.0	4.0	5	
136+00	1222.9	4.4	6.0	4.0	5	
137+00	1223.1	3.1	6.0	4.0	5	
138+00	1223.3	3.0	6.0	4.0	5	

Table 4.2 Design Toe-Down Depth and Apron Width Summary (Continued)

Station Number	Design Invert Elevation (feet)	Standard Toe-Down Depth (feet)	Minimum Design Toe-Down Depth (feet)	Design Apron Depth (feet)	Design Apron Width (feet)	Remarks
139+00	1223.5	3.0	6.0	4.0	5	
140+00	1223.7	3.1	6.0	4.0	5	
141+00	1223.9	3.1	6.0	4.0	5	
142+00	1224.1	3.2	6.0	4.0	5	
143+00	1224.3	5.7	6.0	4.0	5	
144+00	1224.5	6.1	6.1	4.0	9	
145+00	1224.6	7.6	7.6	4.0	9	
147+00	1225.1	8.1	8.1	4.5	9	
149+00	1225.5	9.2	9.2	4.9	12	
150+00	1225.7	9.7	9.7	5.0	12	
152+00	1226.1	10.1	10.1	5.4	12	
154+00	1226.5	10.8	10.8	5.8	12	Closed Bottom
154+95	1226.7	10.6	10.6	6.0	12	Closed Bottom
						Sta. 155+00 GCS
157+00	1229.5	8.9	8.9	4.2	16	Closed Bottom
159+00	1230.4	10.4	10.4	4.1	16	Closed Bottom
160+00	1230.9	9.1	9.1	4.1	16	Closed Bottom
161+00	1231.4	7.1	7.1	4.1	16	Closed Bottom
162+00	1231.9	7.0	7.0	4.3	10	Closed Bottom
164+00	1232.8	4.3	6.0	5.2	10	
165+00	1233.3	4.2	6.0	5.6	10	
166+00	1233.8	4.3	6.0	6.1	10	
167+00	1234.3	9.7	9.7	6.5	10	
168+00	1234.7	10.3	10.3	7.0	10	
169+00	1235.2	11.1	11.1	7.4	10	
170+00	1235.7	12.0	12.0	7.9	10	
171+00	1236.2	12.8	12.8	8.3	10	
172+00	1236.7	13.9	13.9	8.8	15	
173+00	1237.1	14.7	14.7	9.2	15	Closed Bottom
174+00	1237.6	16.1	16.1	9.7	15	Closed Bottom
175+00	1238.1	17.0	17.0	10.1	19	Closed Bottom
177+00	1239.1	19.2	19.2	11.0	19	Closed Bottom
178+50	1239.5	18.9	18.9	11.0	19	Closed Bottom
						Union Hill Dr. GCS
180+00	1246.0	5.4	6.0	6.0	12	Existing Apron
181+70	1246.6	7.6	7.6	0.0	18	Apron Extension
183+90	1248.0	6.6	6.6	0.0	18	Apron Extension
185+00	1248.5	7.9	7.9	0.0	22	Apron Extension
187+10	1249.6	8.7	8.7	0.0	22	Apron Extension
189+00	1251.0	11.3	11.3	0.0	28	Apron Extension
190+00	1251.5	11.7	11.7	0.0	28	Apron Extension
192+00	1252.0	11.6	11.6	0.0	28	Apron Extension
194+00	1253.0	13.7	13.7	0.0	36	Apron Extension
195+00	1253.8	15.6	15.6	0.0	36	Apron Extension
196+00	1254.2	12.4	12.4	0.0	30	Apron Extension
						59th Ave. GCS
202+00	1255.5	2.2	6.0	4.0	5	South Bank

Table 4.2 Design Toe-Down Depth and Apron Width Summary (Continued)

Station Number	Design Invert Elevation (feet)	Standard Toe-Down Depth (feet)	Minimum Design Toe-Down Depth (feet)	Design Apron Depth (feet)	Design Apron Width (feet)	Remarks
203+00	1255.9	2.7	6.0	4.0	5	South Bank
205+00	1256.9	4.1	6.0	4.2	5	South Bank
207+00	1257.9	5.4	6.0	4.3	5	South Bank
209+00	1258.8	7.5	7.5	4.5	9	South Bank
210+00	1259.3	8.1	8.1	4.6	9	South Bank
						57th Ave. GCS
213+30	1262.0	4.2	6.0	5.7	9	
213+95	1263.2	8.3	8.3	6.7	9	
215+65	1264.0	7.2	7.2	6.4	9	
217+00	1262.7	2.7	6.0	4.2	5	
219+00	1263.6	2.8	6.0	4.1	5	
220+00	1264.1	2.5	6.0	4.1	5	
222+00	1265.1	2.5	6.0	4.0	5	
224+00	1266.0	9.9	9.9	6.2	14	
225+00	1266.5	11.2	11.2	6.7	14	
226+00	1267.0	13.1	13.1	7.2	14	
227+00	1267.5	13.2	13.2	7.7	14	
228+00	1267.9	14.9	14.9	8.1	14	
229+00	1268.4	12.4	12.4	8.6	14	
230+00	1268.9	12.9	12.9	9.1	14	
231+00	1269.4	13.7	13.7	9.6	14	
232+00	1269.9	14.3	14.3	10.1	14	
						54th Ave. GCS
234+00	1270.8	4.5	6.0	4.3	5	
235+00	1271.3	5.5	6.0	4.5	5	
235+75	1271.6	6.6	6.6	4.7	7	
236+45	1271.9	5.2	6.0	4.7	7	
238+00	1272.7	6.4	6.4	5.1	7	
240+00	1273.7	7.9	7.9	5.4	12	
242+00	1274.7	9.1	9.1	5.8	12	
243+65	1275.4	10.3	10.3	6.1	12	
						Sta. 245+00 GCS
247+00	1279.1	2.9	6.0	7.3	5	North Bank
249+00	1279.1	2.7	6.0	5.5	5	North Bank
250+00	1279.1	2.5	6.0	4.9	5	North Bank
252+00	1279.4	2.5	6.0	4.1	5	North Bank
254+00	1280.6	4.6	6.0	4.0	5	North Bank
255+00	1281.2	5.8	6.0	4.0	5	North Bank
256+00	1281.8	7.2	7.2	4.0	9	North Bank
						Sta. 256+00 GCS
257+00	1283.1	3.8	6.0	6.0	0	
258+27	1283.3	3.3	6.0	6.0	0	
258+30	1283.3	3.4	6.0	6.0	0	
258+70	1283.4	4.0	6.0	6.0	0	
260+00	1284.2	5.9	6.0	6.0	0	

To minimize the cost of installation, the final design met the minimum toe-down elevations through a combination of depth and horizontal aprons. Theoretically, the gabion mattress aprons launch themselves downward as the scour occurs below them, preserving the stability of the bank. The depth of the horizontal aprons was computed as the depth of long-term degradation plus one foot or a minimum of four feet, whichever was greater. This would theoretically keep the aprons covered with at least one foot of sediment, thereby reducing the potential hazards for future equestrian traffic, and eliminate the need to excavate to the full toe-down elevation. The final depths to apron and apron widths, summarized in Table 4.2, were rounded to provide smooth transitions between cross-sections, and allow for efficient construction.

4.6 Top-of-Bank Elevations

The design top-of-bank elevations were set above the maximum water-surface elevation for the 100-year design flood in accordance with the freeboard criteria of the "Drainage Design Manual for Maricopa County, Volume II, Hydraulics," (15). The maximum 100-year water surface included the elevations from the QUASED model, where higher than the HEC-2 results, and super-elevations at the outside of bends. A minimum of one foot of freeboard was provided in areas with existing bank protection and a minimum of three feet of freeboard was provided for the proposed north-bank levee, just downstream of 51st Avenue, as specified by the District's channel design criteria (9). The top-of-bank elevations were also increased slightly at various locations to achieve a smooth, easily constructable design. The final top-of-bank elevations and the resulting freeboard are summarized in Table 4.3.

4.7 Local Drainage Structures

Local storm drain outlets were adjusted using the same pipe size and material as the existing. A prefabricated end section with riprap protection was provided at each outlet. Where required, new culverts were of corrugated metal pipe with prefabricated end-sections at the inlets and outlets. Exceptions were the five apron drains for the Sunset Vista residential area. These drains were continued through the new bank protection using drop inlets and concrete headwalls at the inlets, and prefabricated end sections with riprap protection at the outlets. Culverts were designed to pass the 100-year runoff, as specified in the local drainage reports, with a 10-year tailwater level in Skunk Creek.

The Skunk Creek channel was designed to retain the 100-year water surface within its banks. This was achieved primarily by lowering the existing invert and providing grade control structures. Consequently, the design avoids the need for levees and backflow prevention devices, such as flapgates. An exception is at the north bank just downstream of 51st Avenue. A levee was required to keep the channel within the available right-of-way, and a back flow prevention device was provided within the culvert that drains the area behind the levee.

The information used to design the culverts for local drainage is contained in Appendix G. The minimum size culvert used was an 18-inch diameter, as requested by the District.

Table 4.3 Design Top of Bank Elevations and Freeboard

Station Number	Max Water-Surface. Elevation (ft)	Required Freeboard (N/S) (ft)	Super-Elevation (ft)	Design T.O.N.B. (ft)	Design T.O.S.B. (ft)	Design N.B. Freeboard (ft)	Design S.B. Freeboard (ft)
74+12	1205.1	2.7		1207.8	1207.8	2.7	2.7
75+00	1205.6	2.6		1208.2	1208.2	2.6	2.6
77+00	1205.7	2.4		1208.5	1208.5	2.8	2.8
79+00	1205.9	2.2		1208.8	1208.8	2.9	2.9
80+00	*1206.1	2.2	0.5(S)	1209.0	1209.0	2.9	2.4
82+00	*1206.5	2.0	0.5(S)	1209.3	1209.3	2.8	2.3
84+00	*1207.0	1.9	0.5(S)	1209.6	1209.6	2.6	2.1
85+00	*1207.3	1.9	0.5(S)	1209.7	1209.7	2.5	2.0
87+00	*1208.0	1.8		1210.0	1210.0	2.0	2.0
89+00	*1208.7	1.8		1211.0	1211.0	2.3	2.3
90+00	*1209.1	1.8		1211.5	1211.5	2.4	2.4
91+00	*1209.6	1.8		1212.0	1212.0	2.4	2.4
92+71	*1210.5	1.7		1212.9	1212.9	2.3	2.3
92+95	*1210.5	1.8		1213.0	1213.0	2.5	2.5
94+05	*1210.8	1.0/1.8		1217.0	1213.5	6.2	2.7
94+71	*1211.6	1.0/1.8		1216.8	1213.9	5.2	2.3
95+00	*1211.7	1.0/1.8		1216.9	1214.0	5.2	2.4
97+00	*1212.5	1.0/1.8	0.2(N)	1217.4	1215.0	4.7	2.5
98+00	1213.0	1.0/1.8	0.2(N)	1217.7	1215.5	4.5	2.5
100+00	1214.0	1.0/1.8	0.2(N)	1218.2	1216.5	4.1	2.5
100+25	1214.2	1.0/1.8	0.2(N)	1218.3	1216.6	3.9	2.4
101+00	1214.5	1.8	0.2(N)	1217.0	1217.0	2.3	2.5
102+00	1215.0	1.8	0.2(N)	1217.5	1217.5	2.3	2.5
104+00	1216.1	1.9	0.2(N)	1218.5	1218.5	2.2	2.4
106+00	1217.3	1.9	0.2(N)	1219.5	1220.9	2.0	3.6
108+00	1218.5	2.0	0.4(S)	1220.5	1223.3	2.0	4.4
109+00	*1219.2	2.0	0.4(S)	1222.1	1224.5	2.9	4.9
110+00	1219.5	2.0	0.4(S)	1223.6	1225.7	4.1	5.8
110+95	1220.1	2.0	0.4(S)	1225.1	1226.9	5.0	6.4
111+00	1222.1	1.9	0.4(S)	1225.2	1226.9	3.0	4.4
111+50	1223.5	2.0	0.5(S)	1225.9	1227.5	2.4	3.5
112+30	*1224.3	2.1	0.5(S)	1227.2	1228.5	2.9	3.7
112+38	*1224.4	2.1	0.5(S)	1227.3	1228.6	2.9	3.7
113+05	1224.9	2.2	0.5(S)	69th Ave	69th Ave		
113+10	1224.8	2.2	0.5(S)				
114+00	1225.4	2.3	0.5(S)	1228.6	1229.6	3.1	3.6
115+00	1225.8	2.3	0.5(S)	1228.9	1229.8	3.1	3.6
117+00	1226.5	2.3	0.5(S)	1229.5	1230.3	3.0	3.3
119+00	1227.2	2.4	0.5(S)	1230.1	1230.8	2.9	3.2
120+00	1227.5	2.4	0.2(N)	1230.4	1231.1	2.7	3.6
122+00	1228.0	2.4	0.2(N)	1231.0	1231.6	2.8	3.6

* QUASED Model
Shaded Areas Represent Existing Bank Protection
N-North Bank
S-South Bank

Table 4.3 Design Top of Bank Elevations and Freeboard (Continued)

Station Number	Max Water-Surface. Elevation (ft)	Required Freeboard (N/S) (ft)	Super-Elevation (ft)	Design T.O.N.B. (ft)	Design T.O.S.B. (ft)	Design N.B. Freeboard (ft)	Design S.B. Freeboard (ft)
124+00	1228.5	2.4	0.2(N)	1231.6	1232.1	2.8	3.5
125+00	1228.8	2.4	0.2(N)	1231.9	1232.3	2.9	3.5
126+45	1229.2	2.5		1232.3	1232.7	3.1	3.5
126+50	1229.0	2.3	0.4(S)	1232.3	1232.7	3.3	3.3
126+60	1229.1	2.3	0.4(S)	1232.3	1232.7	3.3	3.2
127+84	1229.7	2.3	0.4(S)	1232.7	1233.0	3.0	3.0
128+22	1229.8	1.0	0.4(S)	1231.5	1231.5	1.7	1.3
128+53	1229.7	1.0	0.4(S)	1231.7	1231.4	2.0	1.3
128+84	1229.8	1.0	0.4(S)	1231.9	1231.3	2.1	1.1
129+81	1229.9	1.0	0.4(S)	1231.7	1232.0	1.8	1.7
130+50	1231.2	1.0	0.4(S)	1233.5	1233.5	2.3	1.9
131+31	1231.6	1.0		1233.5	1235.5	1.9	3.9 (wall)
132+06	1231.6	1.0	0.3(S)	1234.7	1233.7	3.1	1.8 (wall)
133+00	1232.1	1.0	0.3(S)	1234.7	1234.7	2.6	2.3 (wall)
134+00	1232.5	1.0	0.3(S)	1234.2	1235.8	1.7	3.0
135+00	1233.8	3.0	0.3(S)	1237.0	1237.5	3.2	3.4
136+00	1234.0	3.1	0.3(S)	1237.6	1237.7	3.6	3.4
137+00	1234.3	3.0	0.3(S)	1237.9	1237.9	3.6	3.3
138+00	1234.5	3.0	0.3(S)	1238.1	1238.1	3.6	3.3
139+00	1234.8	3.0		1238.4	1238.3	3.6	3.6
140+00	1235.0	3.0		1238.6	1238.5	3.6	3.5
141+00	1235.2	3.0		1238.9	1238.7	3.6	3.5
142+00	1235.4	3.0		1239.1	1238.9	3.7	3.5
143+00	1235.7	2.9		1239.4	1239.1	3.7	3.4
144+00	1235.9	3.0		1239.6	1239.3	3.7	3.4
145+00	1236.1	3.0		1239.9	1239.5	3.8	3.4
147+00	1236.5	3.1		1240.4	1239.9	3.9	3.4
149+00	1237.0	3.2		1240.9	1240.3	3.9	3.3
150+00	1237.3	3.2		1241.1	1240.5	3.9	3.3
152+00	1238.0	3.3		1241.6	1242.0	3.6	4.0
154+00	1238.7	3.4		1243.2	1243.5	4.5	4.8
154+95	*1239.2	3.5		1244.0	1244.2	4.8	5.1
155+00	1237.9	3.2		1244.0	1244.3	6.1	6.4
155+10	1239.0	3.3		1244.1	1244.3	5.1	5.3
157+00	1240.7	3.5		1245.6	1245.8	4.9	5.1
159+00	1242.4	3.7		1247.2	1247.3	4.8	4.9
160+00	1244.0	3.7		1248.0	1248.0	4.0	4.0
161+00	1245.1	3.7		1248.8	1248.8	3.7	3.7
162+00	1245.7	3.7		1249.6	1249.5	3.9	3.8
164+00	1246.3	3.6		1249.9	1250.0	3.6	3.7
165+00	1246.6	3.5		1250.1	1250.3	3.5	3.7

* QUASED Model
Shaded Areas Represent Existing Bank Protection
N-North Bank
S-South Bank

Table 4.3 Design Top of Bank Elevations and Freeboard (Continued)

Station Number	Max Water-Surface. Elevation (ft)	Required Freeboard (N/S) (ft)	Super-Elevation (ft)	Design T.O.N.B. (ft)	Design T.O.S.B. (ft)	Design N.B. Freeboard (ft)	Design S.B. Freeboard (ft)
166+00	1246.7	3.4		1250.3	1250.5	3.6	3.9
167+00	1246.8	3.3		1250.4	1250.8	3.6	4.0
168+00	1246.9	3.2		1250.6	1251.0	3.6	4.1
169+00	1247.1	3.2	0.1(N)	1250.7	1251.3	3.6	4.2
170+00	1247.2	3.1	0.1(N)	1251.2	1251.5	3.9	4.4
171+00	1247.4	1.0/3.1	0.1(N)	1252.5	1251.8	5.0	4.4
172+00	1247.5	1.0/3.0	0.1(N)	1252.7	1251.9	5.1	4.4
173+00	1247.8	3.0	0.1(N)	1253.2	1252.7	5.3	4.9
174+00	1248.1	3.0	0.1(N)	1253.7	1253.2	5.5	5.1
175+00	1248.4	3.0	0.1(N)	1254.0	1253.7	5.5	5.3
177+00	1249.3	2.9	0.1(N)	1252.7	1255.0	3.4	5.8
178+50	1249.8	2.9		1255.8	1256.0	6.0	6.2
179+50	1250.6	2.9	0.2(N)	1256.5	1256.7	5.8	6.2
179+55	1252.3	2.4	0.2(N)	1256.6	1256.7	4.1	4.4
179+60	1252.4	2.4	0.2(N)	1256.6	1257.0	4.0	4.6
180+00	1254.3	2.5	0.2(N)	1259.4	1257.0	5.0	2.7
181+70	1255.1	2.6	0.2(N)	1260.4	1258.7	5.1	3.6
183+90	1257.0	3.0/1.0	0.2(N)	1263.2	1260.9	6.0	3.9
185+00	1257.6	3.0/1.0	0.2(N)	1263.8	1262.0	6.0	4.5
187+10	1259.3	2.3/1.0	0.2(N)	1261.8	1262.3	2.3	3.0
189+00	1260.0	2.2/1.0	0.2(N)	1262.4	1262.6	2.2	2.6
190+00	1260.4	2.1/1.0	0.2(N)	1262.7	1263.0	2.1	2.6
192+00	1261.5	2.1/1.0	0.2(N)	1263.8	1263.2	2.1	1.7
194+00	1262.3	2.2/1.0	0.2(N)	1264.7	1264.2	2.2	1.9
195+00	1262.6	1.6/1.0	0.2(N)	1264.3	1264.8	1.6	2.3
196+00	1263.1	2.5/1.0	0.2(N)	1265.8	1265.3	2.5	2.2
197+82	1264.3	1.7/1.0	0.2(N)	1266.2	1266.0	1.7	1.7
199+40	1264.8	2.7/1.0		1267.8	1269.0	3.0	4.3
200+50	1264.8	1.0/2.7		1269.0	1268.4	4.2	3.6
201+60	1265.7	1.0/2.7		1268.3	1268.6	2.6	2.9
202+00	1266.0	2.3/2.7		1268.3	1268.7	2.3	2.7
203+00	1266.1	2.1/2.6		1268.2	1268.9	2.1	2.8
205+00	1266.3	2.5		1268.7	1269.3	2.5	3.0
207+00	1266.6	2.3		1269.3	1269.7	2.7	3.1
209+00	1267.0	2.2		1269.9	1270.1	3.0	3.1
210+00	1267.2	2.1		1270.2	1270.3	3.0	3.1
212+30	1267.8	2.1		1270.9	1270.8	3.1	3.0
212+40	1267.8	1.6		1270.9	1270.8	3.1	3.0
213+30	*1268.2	1.8		1271.2	1271.0	3.0	2.8
213+95	1268.3	1.8		1271.4	57th ave.	3.0	
215+65	1270.8	1.8		57th ave.	1273.9		3.2

* QUASED Model
Shaded Areas Represent Existing Bank Protection
N-North Bank
S-South Bank

Table 4.3 Design Top of Bank Elevations and Freeboard (Continued)

Station Number	Max Water-Surface. Elevation (ft)	Required Freeboard (N/S) (ft)	Super-Elevation (ft)	Design T.O.N.B. (ft)	Design T.O.S.B. (ft)	Design N.B. Freeboard (ft)	Design S.B. Freeboard (ft)
217+00	1271.2	2.3		1274.1	1274.2	2.9	3.0
219+00	1271.5	2.1	0.7(S)	1274.5	1274.6	3.0	2.4
220+00	1271.9	2.0	0.7(S)	1274.7	1274.8	2.8	2.2
222+00	1272.2	1.9	0.7(S)	1275.1	1275.2	2.9	2.3
224+00	*1272.6	1.8	0.7(S)	1275.5	1275.6	2.9	2.3
225+00	*1272.8	1.8		1275.7	1275.8	2.9	3.0
226+00	*1273.3	1.9		1275.9	1276.0	2.7	2.8
227+00	1273.9	2.0			1277.2		3.2
228+00	1274.4	2.1		1278.5	1278.3	4.1	3.9
229+00	1276.4	2.2		1278.9	1278.7	2.5	2.3
230+00	1276.7	2.2		1279.3	1279.1	2.6	2.4
231+00	1277.0	2.1		1279.7	1279.5	2.7	2.5
232+00	1277.3	2.1		1280.1	1279.9	2.8	2.6
233+00	1277.7	2.1		1280.5	1280.3	2.8	2.6
233+10	1277.7	2.0		1280.5	1280.3	2.8	2.6
234+00	1278.0	2.0		1280.9	55th Ave.	2.9	
235+00	1278.4	2.0		1281.3		2.9	
235+75	1278.6	2.1			1281.5		2.9
236+45	1279.2	2.1		55th Ave.	1281.9		2.7
238+00	1280.0	2.1	0.1(S)		1282.3		2.2
240+00	1280.8	2.0	0.1(S)	1283.3	1283.1	2.5	2.2
42+00	1281.6	2.0	0.1(S)	1284.1	1283.9	2.5	2.2
243+65	1282.2	2.0		1285.6	1284.7	3.4	2.5
245+00	1282.9	1.9		1286.6	1285.3	3.7	2.4
245+10	1283.5	1.6		1286.8	1285.4	3.3	1.9
247+00	1286.0	2.0/1.0		1290.0	1288.8	4.0	2.8
249+00	1286.8	2.1/1.0		1290.6	1289.5	3.8	2.7
250+00	1287.2	2.2/1.0	0.3(N)	1290.9	1289.8	3.4	2.6
252+00	*1287.9	2.2/1.0	0.3(N)	1291.5	1290.4	3.3	2.5
254+00	*1288.5	2.0/1.0	0.3(N)	1292.1	1290.9	3.3	2.4
255+00	*1288.7	2.0/1.0	0.3(N)	1292.4	1291.4	3.4	2.7
256+00	*1289.0	1.0	0.3(N)	1292.7	1291.9	3.4	2.9
256+50	*1289.5	1.0	0.3(N)	1292.9	1292.2	3.1	2.7
256+60	*1289.3	1.01.8	0.3(N)	1292.9	1293.1	3.3	3.8
257+00	*1289.7	1.0/1.9	0.3(N)	1293.1	1292.4	3.1	2.7
258+27	*1290.5	1.0/2.1	0.3(N)	1293.2	1292.9	2.4	2.4
258+30	*1290.5	1.0/2.0	0.3(N)	1293.2	1293.0	2.4	2.5
258+70	*1290.5	1.0/2.0	0.3(N)	1293.4	1293.2	2.6	2.7
260+00	*1291.1	1.0	0.3(N)	1293.5	1293.9	2.1	2.8

* QUASED Model
Shaded Areas Represent Existing Bank Protection
N-North Bank
S-South Bank

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