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Sediment Transport Analysis - Queen Creek & Sanokai Wash Hydraulic Master Plan East Maricopa Floodway Capacity Mitigation Study

Project No. 05-0949-01

WEST CONSULTANTS, INC.

Water • **E**nvironmental • **S**edimentation • **T**echnology

Hydraulics · Hydrology · Sedimentation · Water Quality · Erosion Control · Environmental Services

Final Report

Sediment Transport Analysis - Queen Creek & Sanokai Wash Hydraulic Master Plan East Maricopa Floodway Capacity Mitigation Study

Project No. 05-0949-01

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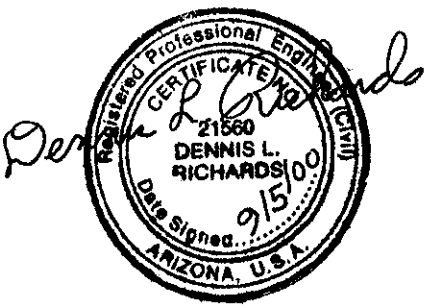

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1 INTRODUCTION

1.1 General

Queen Creek, Sanokai Wash, and the East Maricopa Floodway (EMF) are located within unincorporated areas of Maricopa County and Pinal County, as shown in Figure 1-1. The headwaters of Queen Creek are in the hills of western Pinal County. From there it flows to the west into Whitlow Reservoir before reaching the Bureau of Reclamation Sanoqui Flood Retarding Structure immediately upstream of the Central Arizona Project (CAP) Aqueduct. Four 72-inch steel pipes convey Queen Creek flow across the aqueduct, which is the upstream limit of the Queen Creek study area. The Queen Creek channel discharges into a sedimentation basin located upstream of the confluence with the East Maricopa County Floodway (EMF), at the downstream limit of the study area.

The limits of the Masterplan/Feasibility Study for Queen Creek are from the CAP aqueduct in the east, the Maricopa/Pinal County boundary to the south and the East Maricopa Floodway (EMF) in the west. Also included in the study is Sanokai Wash, a tributary that joins Queen Creek about 1.5 miles upstream of the confluence of Queen Creek with the EMF. From its confluence with Queen Creek, the EMF continues in a southerly direction and empties into the Gila River. All the stream channels are ephemeral, in that they are typically dry except during and after significant rainfall events.

In the upper portion of the study area, the Queen Creek channel is in a natural state except for a few locations that are influenced by gravel mining operations. Queen Creek is primarily a leveed channel in the middle and lower portions of the study area where it passes through farmland and developed areas.

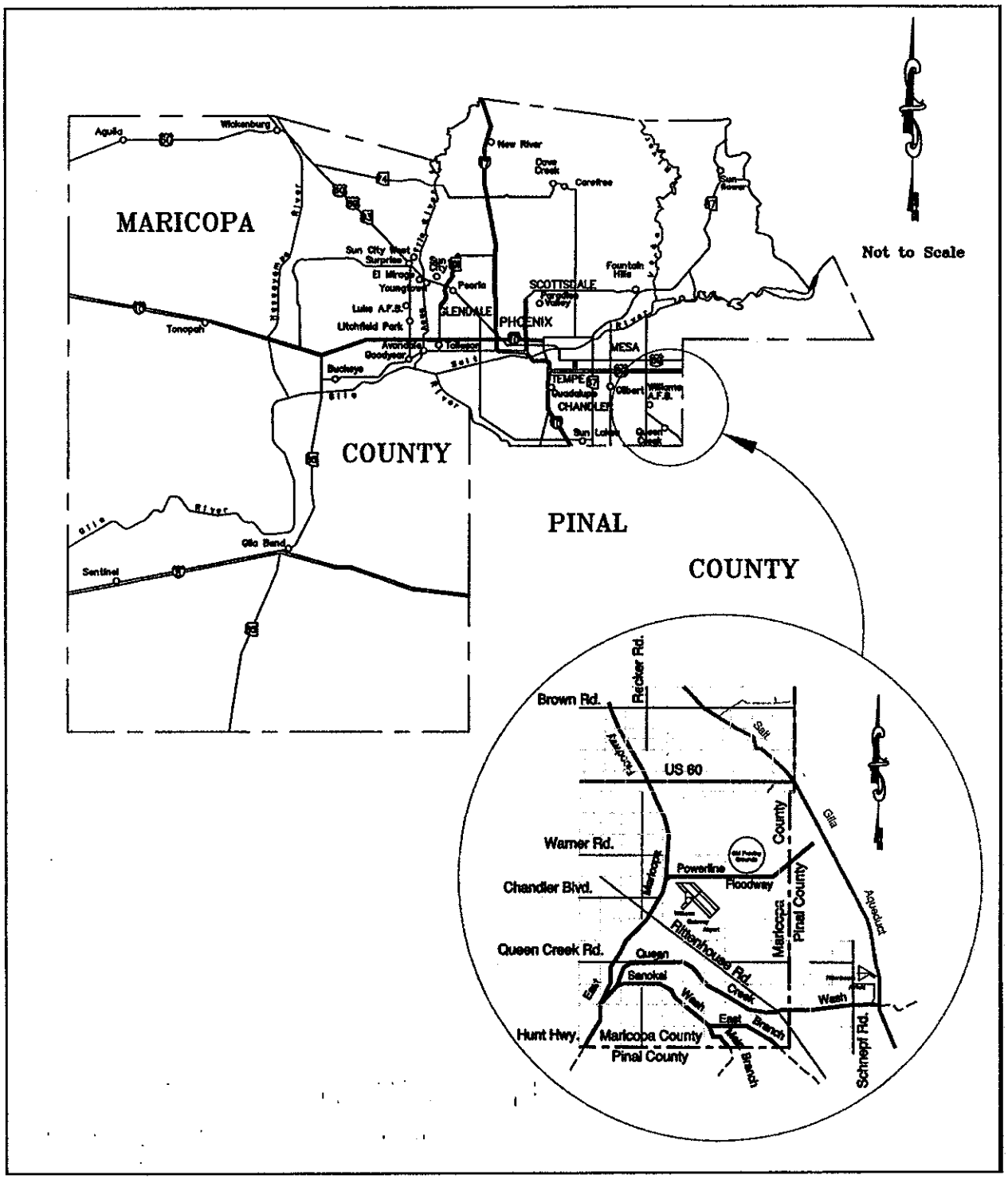

Figure 1-1. Location and vicinity map

1.2 Study Purpose

Huitt~Zollars, in cooperation with the Flood Control District of Maricopa County is developing a Hydraulic Master Plan (HMP) to determine engineering guidelines for management of the 100-year recurrence interval storm flows along Queen Creek and Sanokai Wash. The objective of this effort is to reduce the risk of flood damage in the Queen Creek and Sanokai Wash basins. The purpose of this study by WEST Consultants, Inc. is to perform sediment transport analyses as defined in Arizona Department of Water Resources State (ADWR) Standard 5-96 the State Standard for Watercourse System Sediment Balance, dated September 1996.

1.3 Authorization

WEST Consultants, Inc. (WEST) was contracted to conduct this study by Huitt~Zollars, Inc. under Contract Number FCD 98-26 with the Flood Control District of Maricopa County.

1.4 Scope of Work

This report presents a qualitative and quantitative sediment transport analysis of Queen Creek and Sanokai Wash for the preferred alternative channel configurations developed by Huitt~Zollars, Inc. The preferred alternative channel configurations considered are:

Queen Creek

Incise Queen Creek from the EMF to Recker Road matching the established channel invert at Recker Road. Incise Queen Creek from Power Road to Hawes Road. Construct a new Queen Creek channel along the south side of Queen Creek Road from Higley Road to a new confluence with the EMF

Sanokai Wash

Channelize Sanokai Wash from the EMF to Riggs Road and the Main Branch of Sanokai Wash from its confluence with the East Branch to Hunt Highway (County Line). Create a new channel for the East Branch of Sanokai Wash along the south side of Riggs Road from its confluence with the Main Branch to Crismon Road. Divert peak flows from the East Branch of Sanokai Wash into a detention basin located near Signal Butte Road (Signal Butte Basin). Divert peak flows from the East Branch of Sanokai Wash into a detention basin located south of Riggs Road and ~0.5 miles east of Hawes Road (Riggs Basin). Divert peak flows from Sanokai Wash into a detention basin located at the northeast corner of Sossaman Road and Chandler Heights Road (Sossaman Basin). Construct a new Sanokai Wash channel along the Ocotillo Road alignment from Higley Road to the EMF.

EMF Confluence

Construct a new Queen Creek channel along the south side of Queen Creek Road from Higley Road to a new confluence with the EMF plus a new Sanokai Wash channel along the Ocotillo Road alignment from Higley Road to the EMF.

1.4.1 Sediment Transport

Sediment transport evaluations for the preferred alternative were performed as defined in the Arizona Department of Water Resources State (ADWR) Standard 5-96 the State Standard for Watercourse System Sediment Balance, dated September 1996.

This included sediment sampling and testing of the existing channel bed and banks throughout the study reach and upstream sediment source areas. A sediment-sampling plan was developed based on site visit observations. Approximately twenty-five (25) samples were obtained in accordance with the sampling plan, including surface and subsurface samples.

1.4.2 Lateral Migration

The potential lateral migration of each watercourse was determined using methodologies defined in the ADWR State Standard 5-96 and by analyzing historical information gathered during the data collection phase, and by geological investigations. Activities such as sand and gravel mining were identified, and their impacts analyzed in accordance with State of Practice Report, Evaluation of River Stability Impacts Associated with Sand and Gravel.

2 DATA

A significant amount of data and project information has been obtained as a result of WEST Consultants' data collection efforts. Data obtained from the Flood Control District of Maricopa County and Huitt~Zollars, Inc. and used in the hydrologic, hydraulic and sediment transport analyses included the following:

2.1 Hydrology

- Precipitation values for 2-, 5-, 10-, 25-, 50-, and 100-year return period events.
- Updated HEC-1 model for 100-year 24-hour storm duration.
- Topographic maps and historical aerial photos of the project area.
- Existing hydrologic analysis reports.

2.2 Hydraulics

- HEC-RAS models of preferred alternatives.
- Existing hydraulic analysis reports.

2.3 Sediment Transport

- Cross-section information.
- Aerial photos of the project area.
- Bed- and bank sediment size distribution information.
- Existing sediment transport analysis reports.
- Flood history

A field reconnaissance of the study area was conducted to collect data and observe the physical characteristics of the system. Observations were made of the watershed, bridge crossings, existing bank-protection measures, channel characteristics and locations of significant erosion and sedimentation problems. A sediment-sampling plan was developed and implemented based on the site visit observations.

3 HYDROLOGY

The Corps of Engineers HEC-1 flood hydrograph package (U.S. Army Corps of Engineers, 1990) was used by Huitt~Zollars to develop discharge estimates of 100-year, 24-hr future conditions event for Queen Creek and Sanokai Wash. This model was developed from previous study models and updated by Huitt~Zollars for new future hydrologic conditions as part of the Queen Creek - Sanokai Wash Hydraulic Master Plan (HMP) & EMF Capacity Mitigation Study. Previous studies include the Queen Creek Area Drainage Master Study (ADMS) (Wood & Associates, 1991) and the Sanokai Wash Flood Insurance Study (FIS) (FEMA, 1991). This updated HEC-1 model was developed for the 100-year, 24-hr future conditions event and included the watershed areas tributary to Queen Creek and Sanokai Wash and the watershed areas tributary to the EMF south of Rittenhouse Road and north of Hunt Highway (County Line). The various flood events (2-, 5-, 10-, 25-, and 50-year) required for modeling sediment transport were developed from this updated HEC-1 model by WEST Consultants, Inc.

3.1 Hydrology for Queen Creek and Sanokai Wash

In order to develop the future condition hydrology, future land-uses for the sub-basins were estimated from Maricopa County, Pinal County and local municipality General Plans. For sub-basins located within Maricopa County, the 100-yr, 2-hr retention volume was estimated for each sub-basin under future land-use conditions. Pinal County requires that post-development flows should not exceed pre-development flows for any new development. To simulate this requirement, the existing land use (along with the existing land-use sub-basin parameters) was used for all sub-basins located in Pinal County to model the future development conditions.

In addition to the 100-year return period precipitation event developed by Huitt~Zollars (Huitt~Zollars, 2000), the 2-, 5-, 10-, 25-, and 50-year return period precipitation events were developed. Precipitation values for the 24-hour events for each return period were taken from NOAA (National Oceanic and Atmospheric Administration) isopluvial maps for Maricopa County, Arizona. The precipitation values were reduced by the depth-area reduction factor (Table 2.1a of Maricopa Drainage Design Hydrology Manual, 1995) for the same size areas as in the model for future conditions. Storms tend to be most intense over a small area. The depth-area reduction factor adjusts the storm intensity – most intense for small basin areas, and progressively less intense for larger basins. As specified for general use in Maricopa County the same Soil Conservation Service (SCS) Type II distribution curve (Wood & Associates, 1991) used for the future conditions 100-year return period precipitation event was used for the 2-, 5-, 10-, 25-, and 50-year return period precipitation events. The precipitation and depth-area reduction factors for the events analyzed are presented in Table 3-1.

Table 3-1. Precipitation and depth-area reduction factors for various events.

Area (sq miles)	Ration to point rainfall	2-yr precip (in)	5-yr precip (in)	10-yr precip (in)	25-yr precip (in)	50-yr precip (in)
0.01	1	1.35	1.90	2.30	2.85	3.20
1	0.994	1.34	1.89	2.29	2.83	3.18
5	0.969	1.31	1.84	2.23	2.76	3.10
10	0.94	1.27	1.79	2.16	2.68	3.01
30	0.9	1.22	1.71	2.07	2.57	2.88
60	0.86	1.16	1.63	1.98	2.45	2.75
90	0.846	1.14	1.61	1.95	2.41	2.71
120	0.834	1.13	1.58	1.92	2.38	2.67
150	0.825	1.11	1.57	1.90	2.35	2.64

The adjusted precipitation values were run for each return interval in the future conditions model in HEC-1. The results of the model are shown in Figure 3-1 and Figure 3-2.

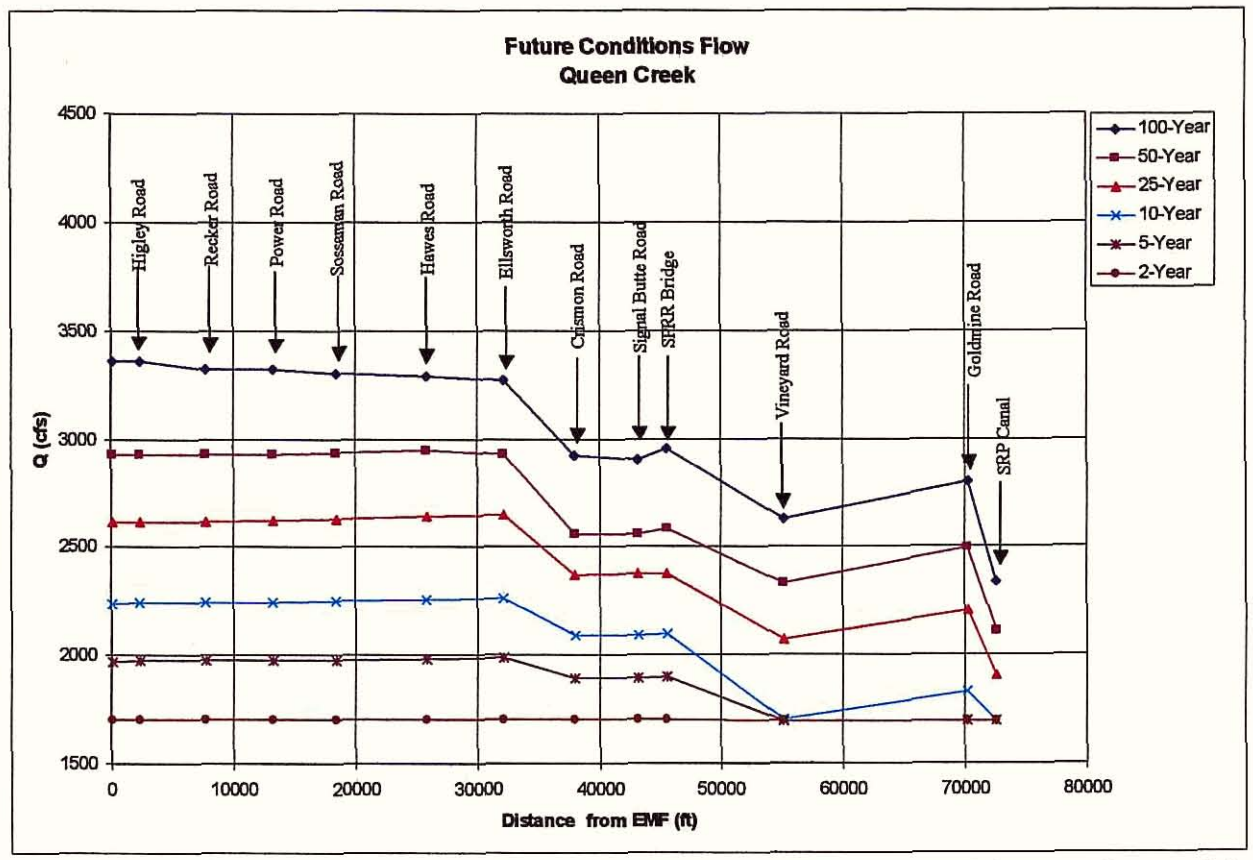

Figure 3-1. Future conditions flow along Queen Creek for Preferred Alternative model

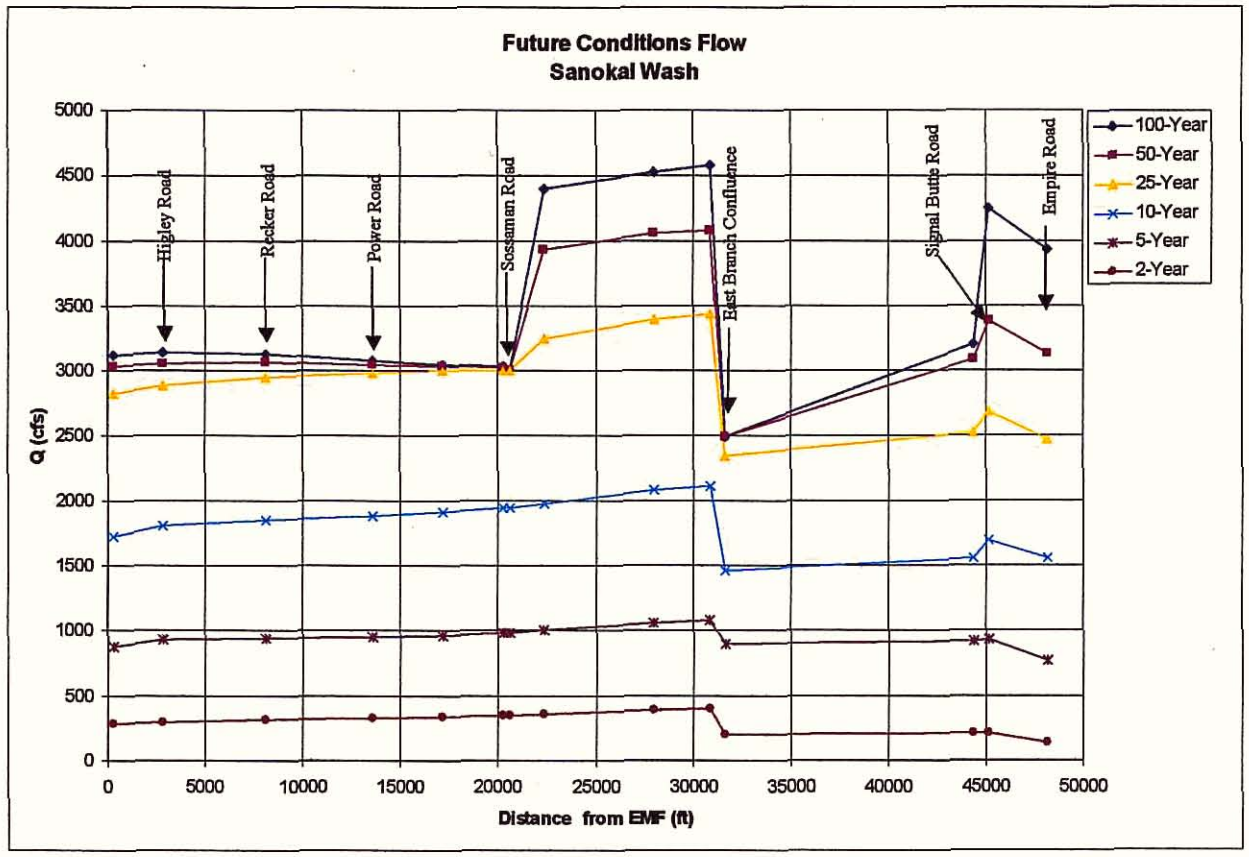

Figure 3-2. Future conditions flow along Sanokai Wash Main and East Branch for Preferred Alternative

The hydrographs at the downstream section (prior to EMF confluence) for Queen Creek and Sanokai Wash are shown respectively in Figure 3-3 and Figure 3-4. For Queen Creek the peak discharges are 3,300 cfs and 1,900 cfs for the 100-year and 5-year flow, respectively. Time to peak is approximately 12 hours and the duration is approximately 1-day. Sanokai Wash has peak discharges of 3,100 cfs and 900 cfs for the 100-year and 5-year flow, respectively. Time to peak is approximately 12 hours and the duration of the hydrographs is approximately 1-day. The 1,500 cfs base flow at the tail end of the Queen Creek hydrographs is the result of the upstream input hydrograph from the outlet of the Sanoqui Flood Retarding Structure (SFRS).

Figure 3-3. Hydrographs for 5- and 100-Year flows at Queen Creek

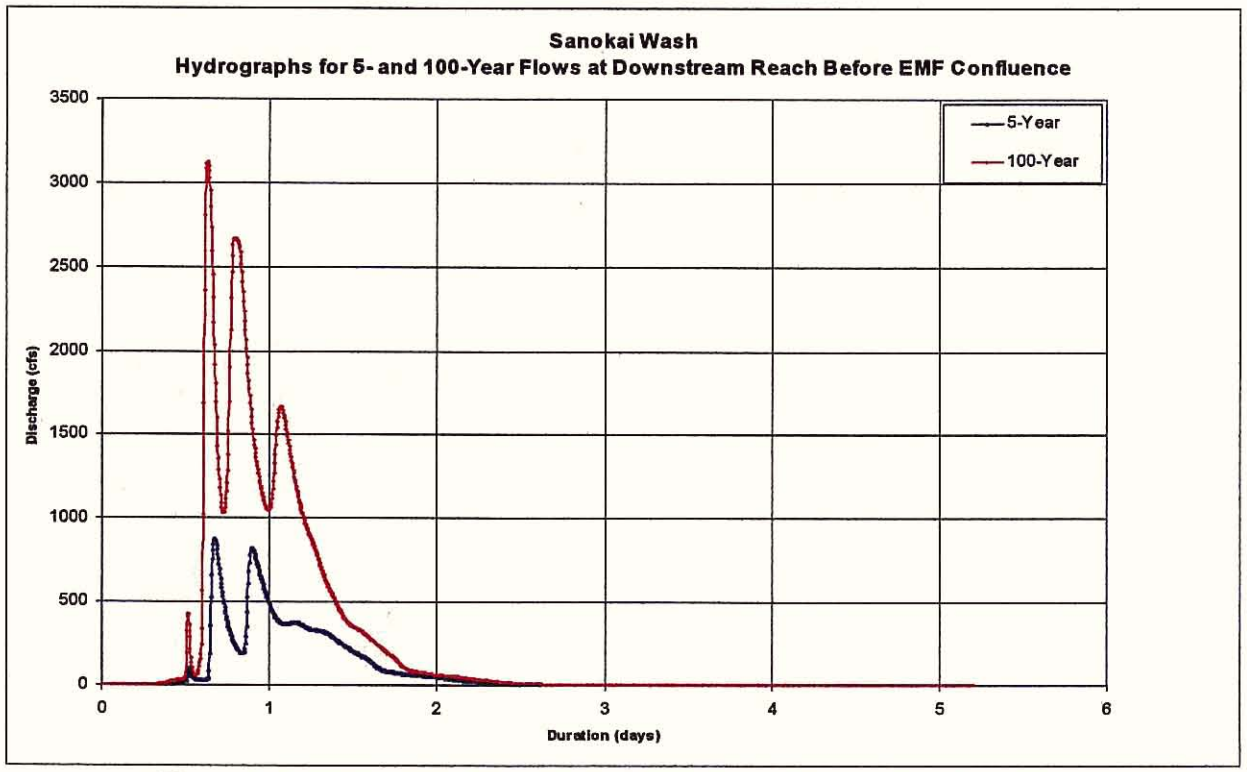

Figure 3-4. Hydrographs for 5- and 100-Year flows at Sanokai Wash

3.1.1 Summary

Hydrologic analyses using HEC-1 were conducted for Queen Creek and Sanokai Wash to estimate the various flood events (5-, 10-, 25-, and 50-year), required for the hydraulic and sediment transport analyses. Precipitation values for the 24-hour events for each return period were taken from NOAA (National Oceanic and Atmospheric Administration) isopluvial maps for Maricopa County, Arizona. The precipitation values were reduced by the depth-area reduction factor. As specified for general use in Maricopa County the same Soil Conservation Service (SCS) Type II distribution curve (Wood & Associates, 1991) used for the future conditions 100-year return period precipitation event was used for the 2-, 5-, 10-, 25-, and 50-year return period precipitation events. Additionally, the duration of the hydrographs were used to estimate the sediment volume from Queen Creek and Sanokai Wash to the EMF.

4 HYDRAULIC ANALYSIS

An analysis of hydraulic conditions along Queen Creek and Sanokai Wash was conducted for discharges corresponding to 2-, 5-, 10-, 25-, 50- and 100-year return period discharge events. The objective of the analysis was to define the hydraulic impacts of the preferred alternative. In order to define the impacts, reaches of similar hydraulic characteristics were defined based on hydraulic characteristics such as velocity, slope, top width and hydraulic depth; and hydraulic controls such as bridges, culverts and dip road crossing. The reaches are shown in Figure 4-1.

4.1 HEC-RAS Analysis

Huitt~Zollars used the Corps of Engineers River Analysis System standard-step backwater computer program (U.S. Army Corps of Engineers, 1997) to develop hydraulic models for Queen Creek and Sanokai Wash. Two separate hydraulic models, one for Queen Creek and one for Sanokai Wash Main and East Branch, for existing conditions and preferred alternative for the 100-year return period flow, were obtained from Huitt~Zollars. The details of the updated HEC-RAS models are provided below:

<u>Hydraulic model of Queen Creek from the EMF to the CAP Aqueduct</u>

This is a combined model of Queen Creek from the EMF to the Southern Pacific Railroad Bridge (SPRR). It utilizes the following hydraulic models from downstream to upstream.

1. Wood and Associates' QC ADMS model.
2. Coe and Van Loo's Power Ranch (Meadow Brook Village) model.
3. Maricopa County Flood Control Districts' Power Road to Hawes Road model.
4. Collins-Piña model from Hawes Road to the SPRR, Letter of Map Revision (LOMR).
5. Huitt~Zollars model from the SPRR to CAP aqueduct.

<u>Hydraulic model of Sanokai Wash from Higley Road to Hunt Highway/Ellsworth Road</u>

This is the base model for the Sanokai Wash channel. The base model was developed by Entellus for the Sanokai Wash Flood Insurance Study and was provided by the Flood Control District of Maricopa County for use in the Queen Creek / Sanokai Wash HMP.

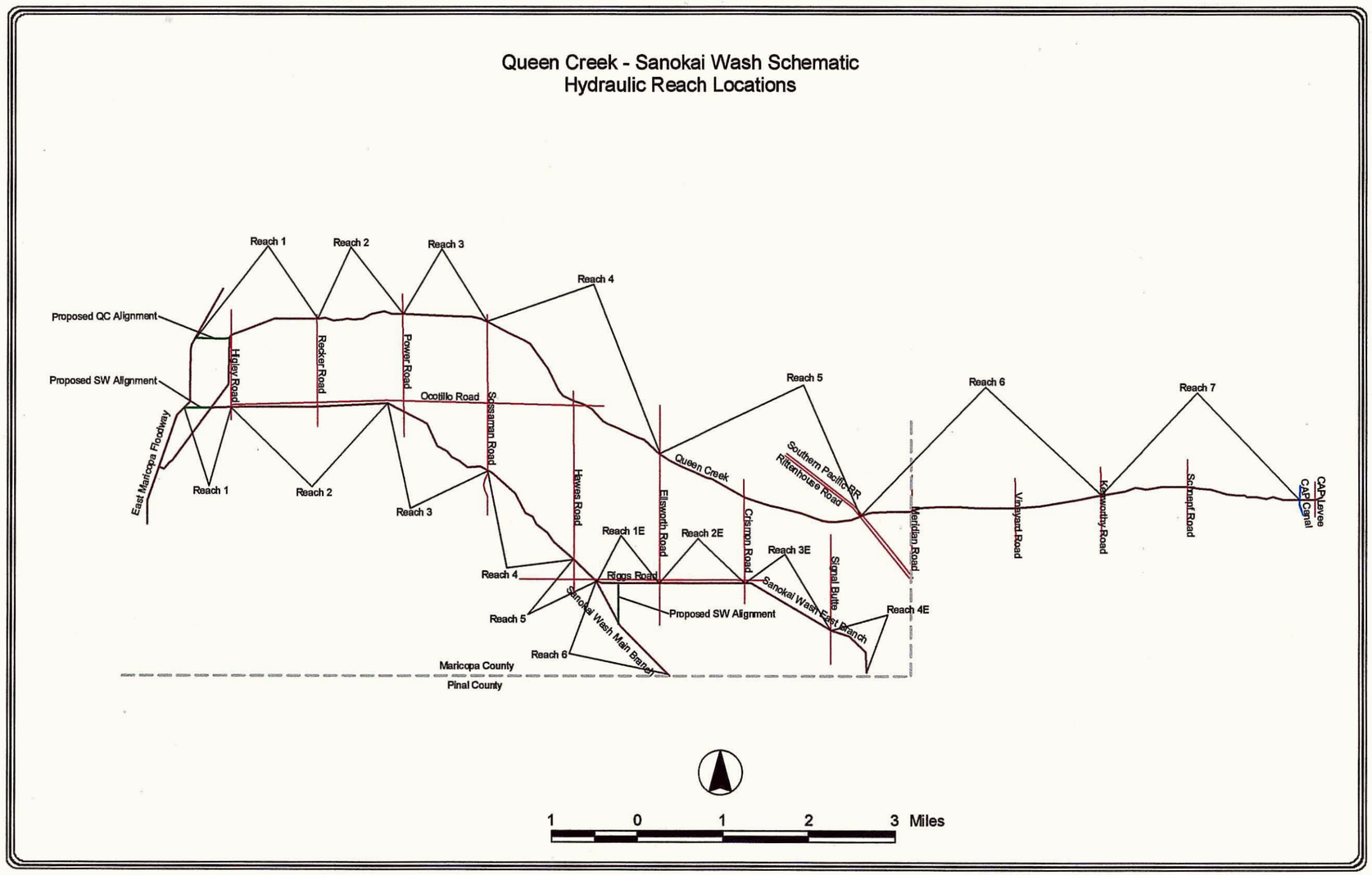

Figure 4-1. Queen Creek – Sanokai Wash schematic showing hydraulic reach locations

4.2 Hydraulic Analysis for Queen Creek

Plots of the channel profile and hydraulic parameters of main channel velocity, main channel top width and hydraulic depth were developed for 2-, 5-, 10-, 25-, 50- and 100-year return period events from the results of the hydraulic models of existing conditions and the preferred alternative. These plots for the existing conditions and preferred alternative for Queen Creek, for the 100-year return period events are presented as Figures 4-2 through 4-5.

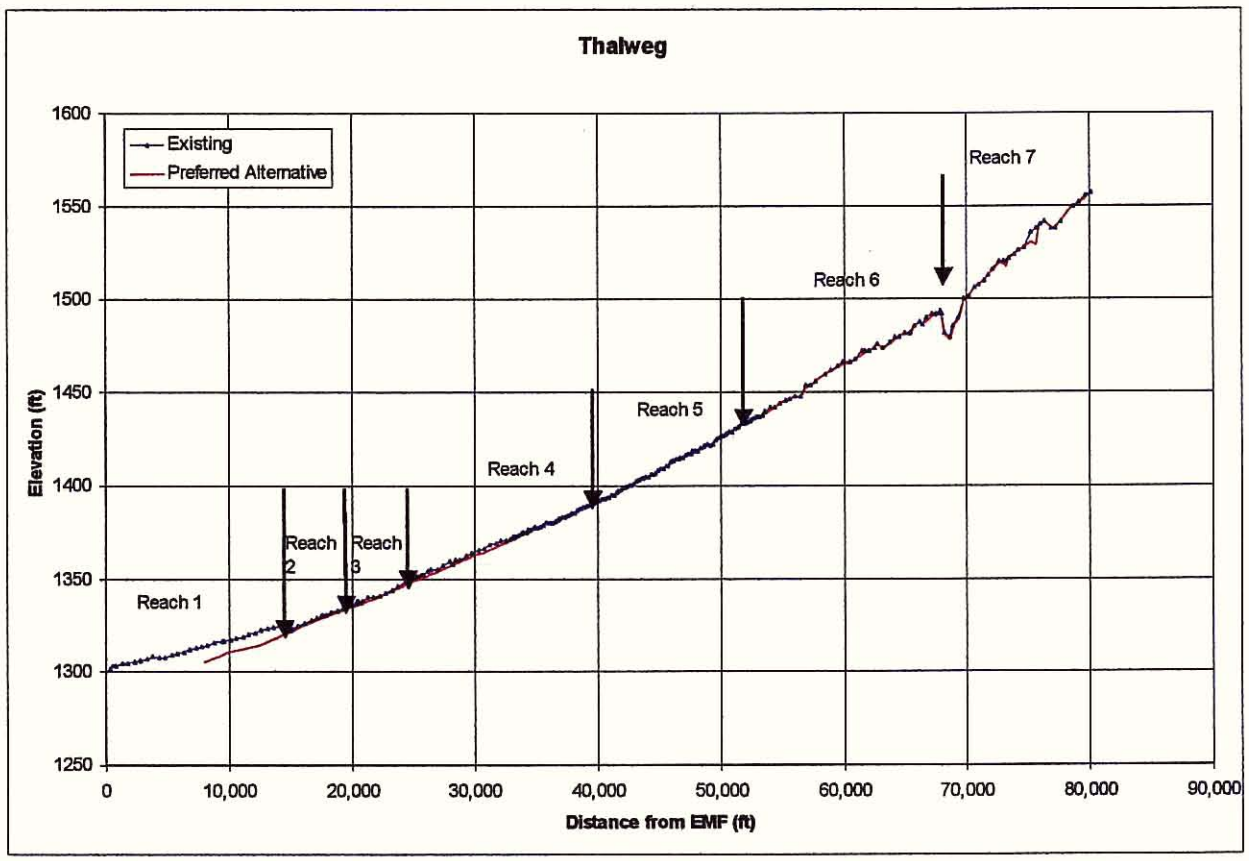

Figure 4-2. Queen Creek channel thalweg profile for Existing Conditions and Alternative

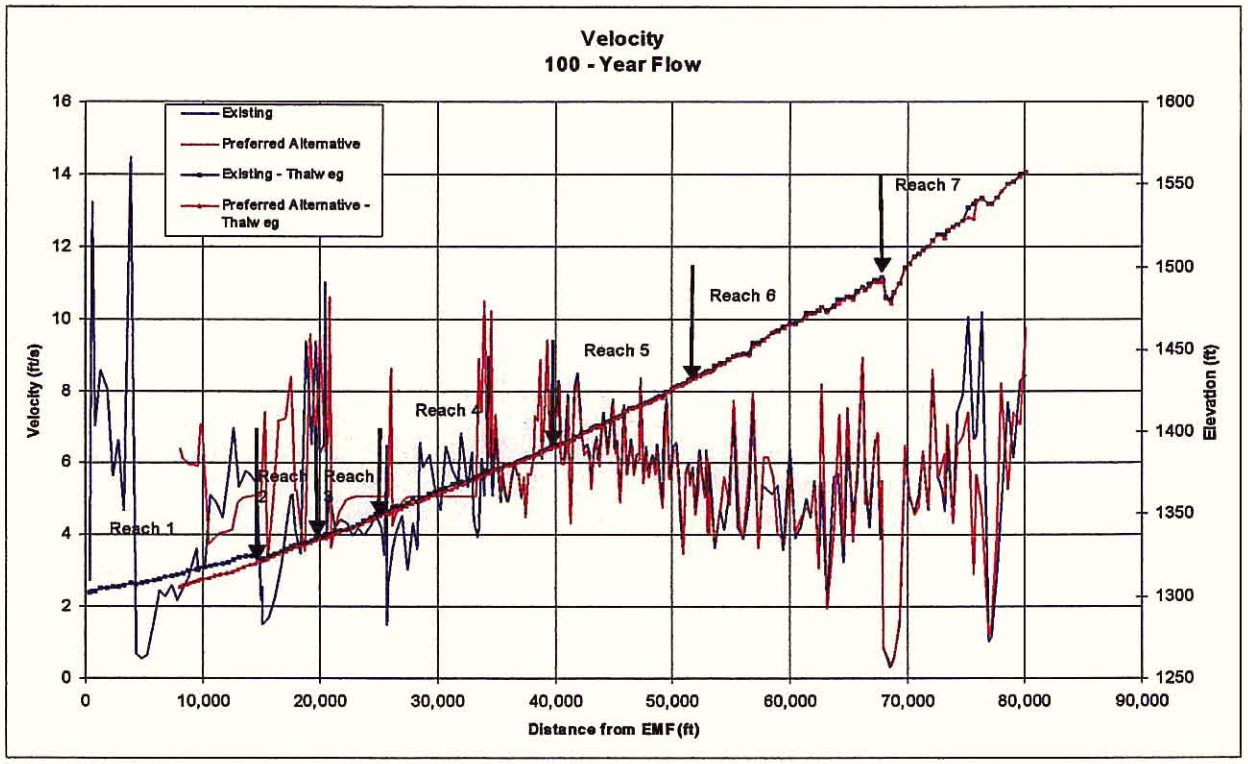

Figure 4-3. Queen Creek main channel velocity profile for Existing Conditions and Alternative at 100-year flow

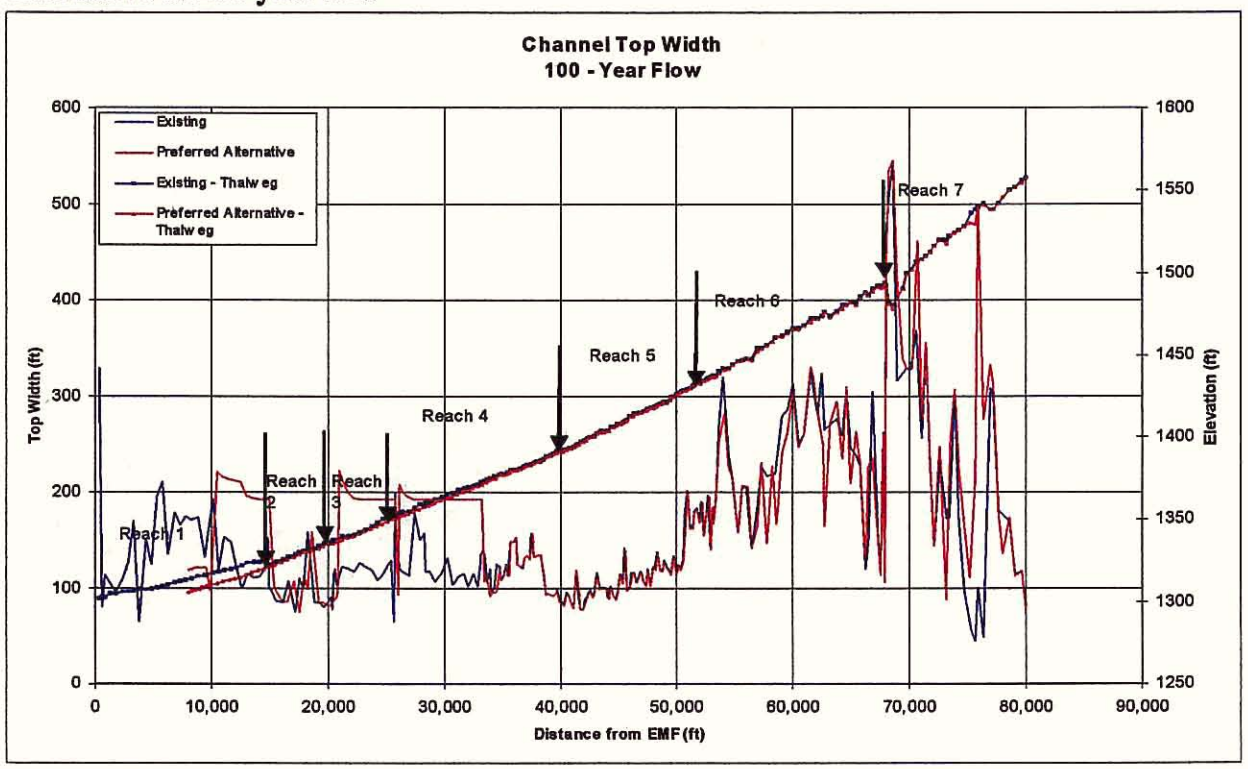

Figure 4-4. Queen Creek main channel top width for Existing Conditions and Alternative at 100-year flow

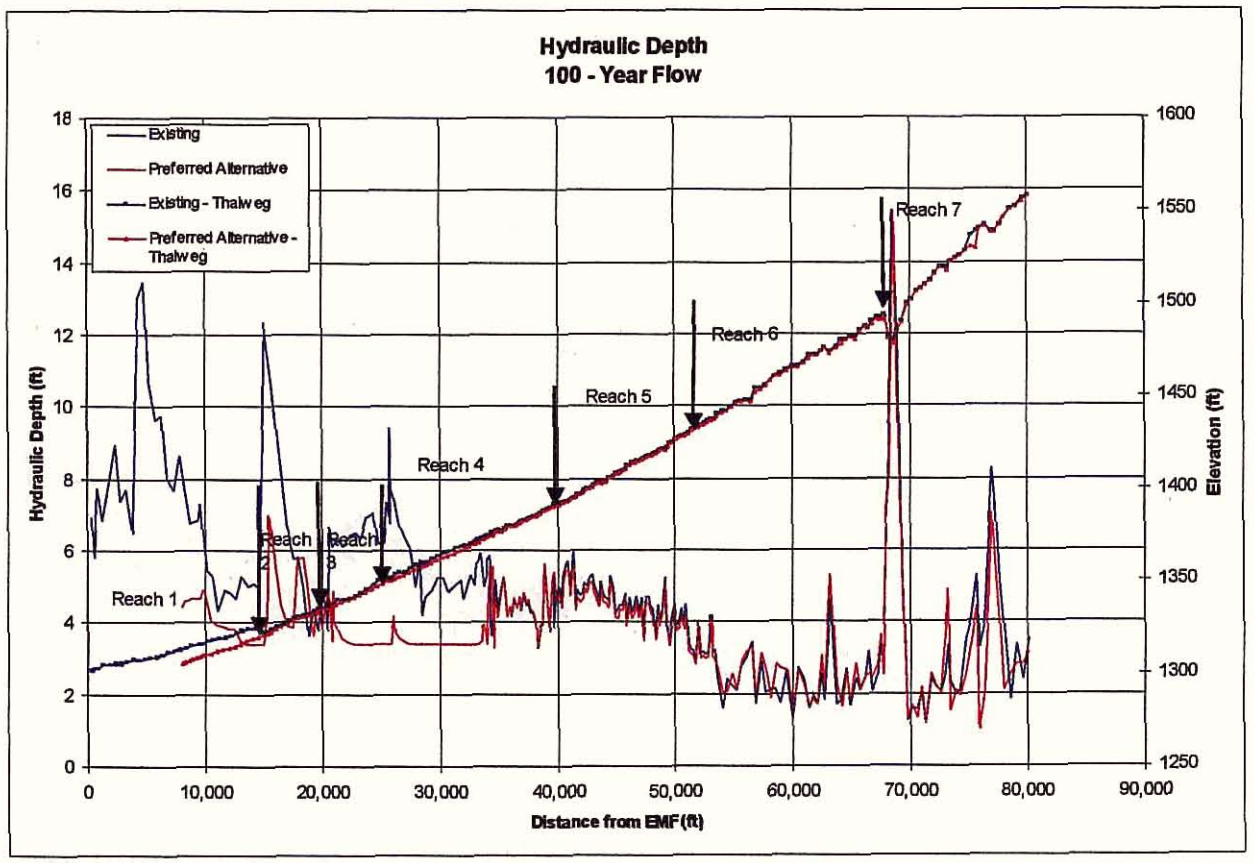

Figure 4-5. Queen Creek main channel hydraulic depth for Existing Conditions and Alternative at 100-year flow

4.2.1 Reach Delineation

Based on the results of the hydraulic analysis, reaches of similar hydraulic characteristics were delineated by identifying significant changes in the hydraulic parameter profiles, with the most emphasis placed on changes in the main channel velocity profile. The hydraulic reach limits identified are delineated in Table 4-1. Figure 4-1 shows the schematic of various reaches for Queen Creek. Having identified reaches of similar hydraulic characteristics, average values for the channel velocity, main channel top width and hydraulic depth were developed for each reach and each discharge. The values are shown in Table 5-2.

Table 4-1. Queen Creek Reaches

Reach	Downstream Location	Upstream Location	U/S Cross Section	Length (feet)
1	EMF	Recker Road	7,666	7,490
2	Recker Road	Power Road	13,159	13,005
3	Power Road	Sossaman Road	18,366	18,298
4	Sossaman Road	Ellsworth Road	32,084	31,762
5	Ellsworth Road	Rittenhouse Road	45,410	45,092
6	Rittenhouse Road	Kenworthy Road	59,007	58,650
7	Kenworthy Road	CAP Canal	72,630	72,253

Along Queen Creek, the proposed changes by Huitt~Zollars to channel geometry were considered for reaches 3, 2 and 1. These proposed changes included channel widening (flatter side slope) and reduction of hydraulic roughness. Additionally, the thalweg elevation in Reach 1 will be lowered and a new Queen Creek channel will be constructed along the south side from Higley Road to a new confluence with the EMF (Figure 4-1).

4.2.2 Results of Hydraulic Analysis for Queen Creek

The following sections describe the results of the hydraulic modeling conducted for Queen Creek for the 100-year return period discharge event by reach, for Existing Conditions and Preferred Alternative.

4.2.2.1 Reach 7

The average main channel velocity in Reach 7 is 5.7 fps for Existing Conditions and 5.4 fps for the Preferred Alternative. The average main channel hydraulic depth is 3.1 feet for Existing Conditions and 2.7 feet for the Preferred Alternative. At the downstream end of Reach 7 there is a large gravel pit. The local increase in channel gradient, due to the gravel pit, could result in a headcut upstream. Gravel pits tend to act as sediment traps that could cause a reduction in sediment supply to downstream reaches, potentially leading to increased channel degradation and/or bank erosion. The model defaults to critical depth in several places in this reach, which could indicate a need for additional cross sections.

4.2.2.2 Reach 6

The average main channel velocity in Reach 6 is 4.9 fps for Existing Conditions and 5.1 fps for the Preferred Alternative. The average main channel hydraulic depth is 2.4 feet for Existing Conditions and 2.6 feet for the Preferred Alternative. Velocity reduces steadily in Reach 6. This may be due to the presence of gravel pits upstream of the reach. Gravel pits could lead to potential problems elsewhere in reaches 7 and 6. The stability of these reaches requires further detailed analysis.

4.2.2.3 Reach 5

The average main channel velocity in Reach 5 is 6.3 fps for Existing Conditions and 6.2 fps for the Preferred Alternative. The average main channel hydraulic depth is 4.5 feet for Existing conditions and 4.3 feet for the Preferred Alternative. The velocity for Existing Conditions and Alternatives increases steadily in the downstream direction and then drops at the upstream end of Reach 4 due to increase in channel width. This could indicate a potential site of degradation at the downstream end of Reach 5 and sediment deposition at the upstream end of Reach 4.

4.2.2.4 Reach 4

The average main channel velocity in Reach 4 is 5.5 fps for Existing Conditions and 5.6 fps for the Preferred Alternative. The average main channel hydraulic depth is 5 feet for Existing Conditions and 3.9 feet for the Preferred Alternative.

4.2.2.5 Reach 3

The average main channel velocity in Reach 3 is 4.3 fps for Existing Conditions and 5.1 fps for the Preferred Alternative. The average main channel hydraulic depth is 6.4 feet for Existing Conditions and approximately 3.4 feet for the Preferred Alternative. The reduction in the channel

roughness value in Reach 3 for the Preferred Alternative results in the increase in main channel velocity. The channel widening at the Preferred Alternative decreases the hydraulic depth. Power Road Bridge, at the downstream end of the reach, restricts flow and creates backwater. This location may be prone to sediment deposition. The upstream end of the reach could be subject to erosion as the velocities are high.

4.2.2.6 Reach 2

The average main channel velocity in Reach 2 is 5.1 fps for Existing Conditions and 6.6 fps for the Preferred Alternative. The average main channel hydraulic depth is 5.8 feet for Existing Conditions and around 3.5 feet for the Preferred Alternative. The reduction in channel roughness values for the Preferred Alternative at the upstream reach (Reach 3) increases the average main channel velocity. The channel widening for the Preferred Alternative decreases the hydraulic depth.

4.2.2.7 Reach 1

The average main channel velocity in Reach 1 is 4.6 fps for Existing Conditions and around 5.5 fps for the Preferred Alternative. The average hydraulic depth is 6.9 feet for Existing Conditions and around 4.0 feet for the Preferred Alternative. The reduction in the channel roughness value in Reach 1 for the Preferred Alternative and the channel modifications results in the increase in main channel velocity. The widening of channel for the Preferred Alternative decreases the hydraulic depth. Sedimentation in Queen Creek will be influenced by the downstream boundary condition. The downstream boundary condition will be controlled by the water surface elevation of the East Maricopa Floodway. As it is currently modeled, the downstream boundary condition does not seem to have much influence on the channel hydraulics.

4.3 Hydraulic Analysis for Sanokai Wash Main Branch

Plots of the channel profile and hydraulic parameters of main channel velocity, main channel top width and hydraulic depth were developed for 2-, 5-, 10-, 25-, 50- and 100-year return period events from the results of the hydraulic models of Existing Conditions and Preferred Alternative. These plots for the Existing Conditions and the Preferred Alternative for Sanokai Wash Main Branch, for the 100-year return period events are presented as Figures 4-6 through 4-13.

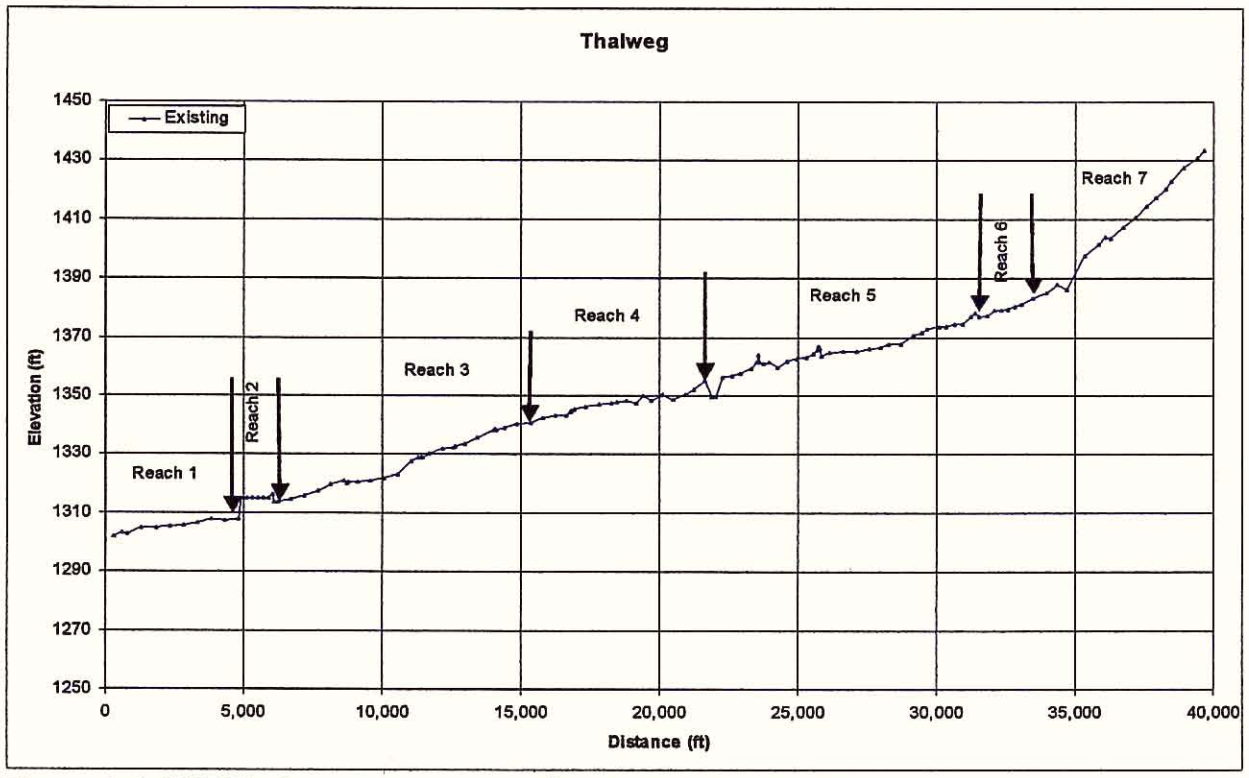

Figure 4-6. SWMB channel thalweg profile for Existing Condition

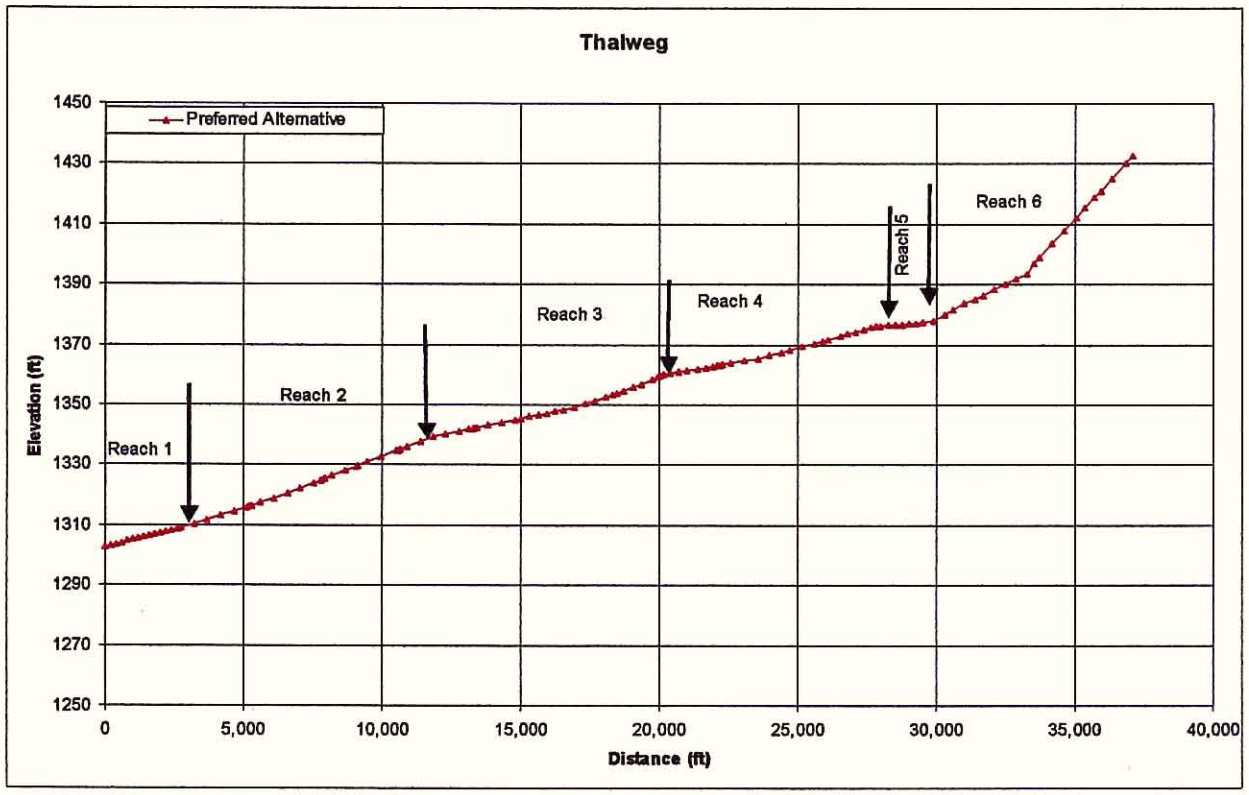

Figure 4-7. SWMB channel thalweg profile for Preferred Alternative

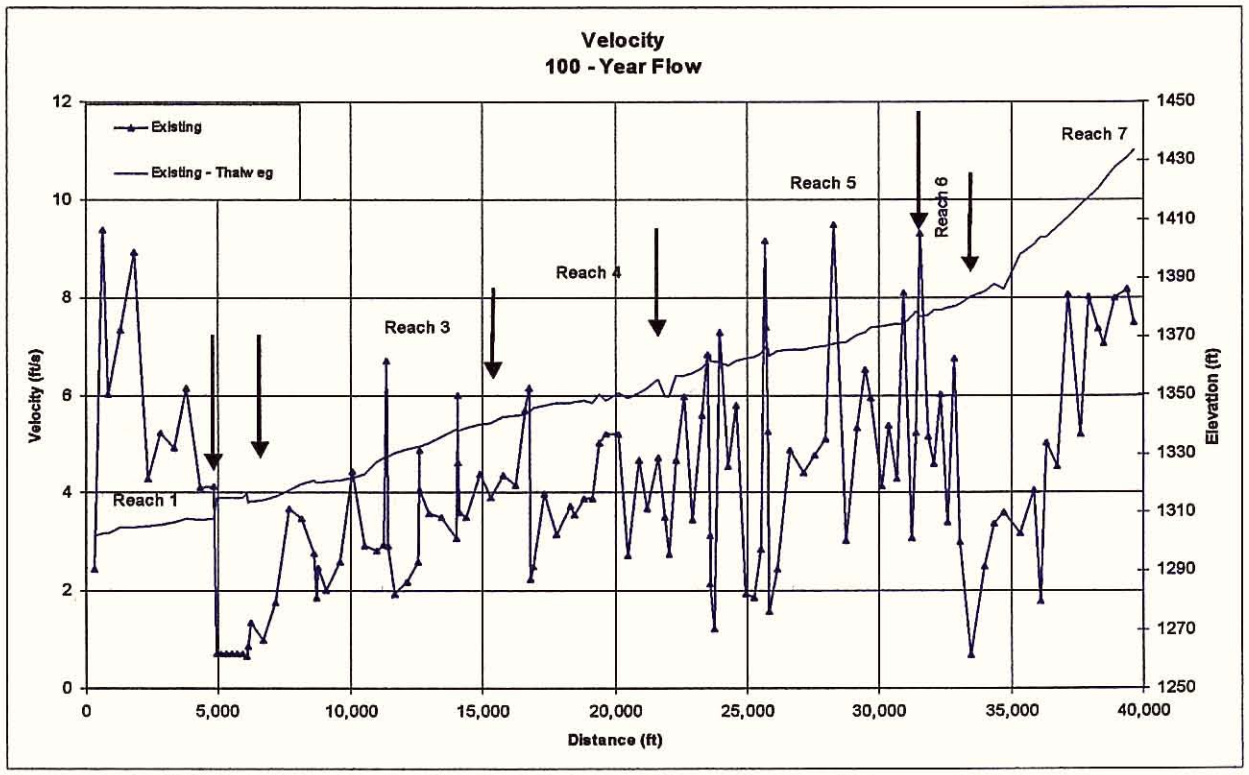

Figure 4-8. SWMB velocity profile for Existing Condition at 100-year flow

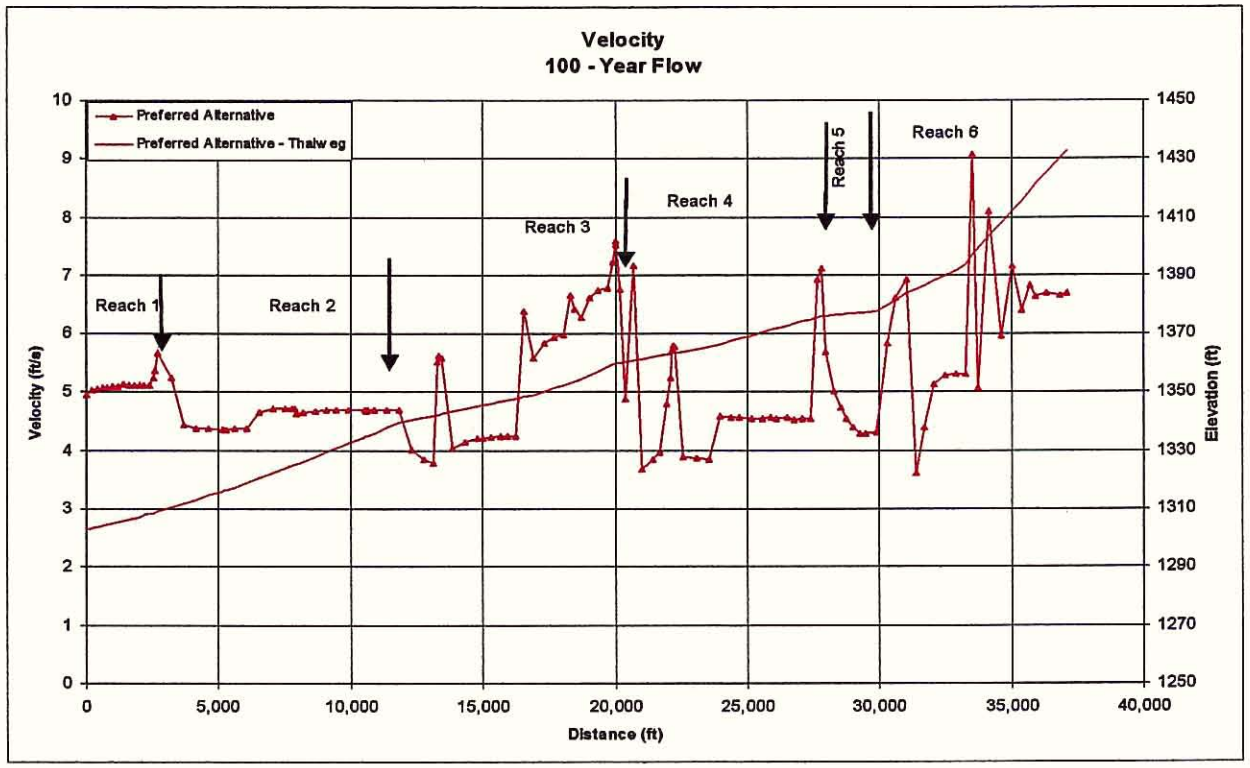

Figure 4-9. SWMB velocity profile for Preferred Alternative at 100-year flow

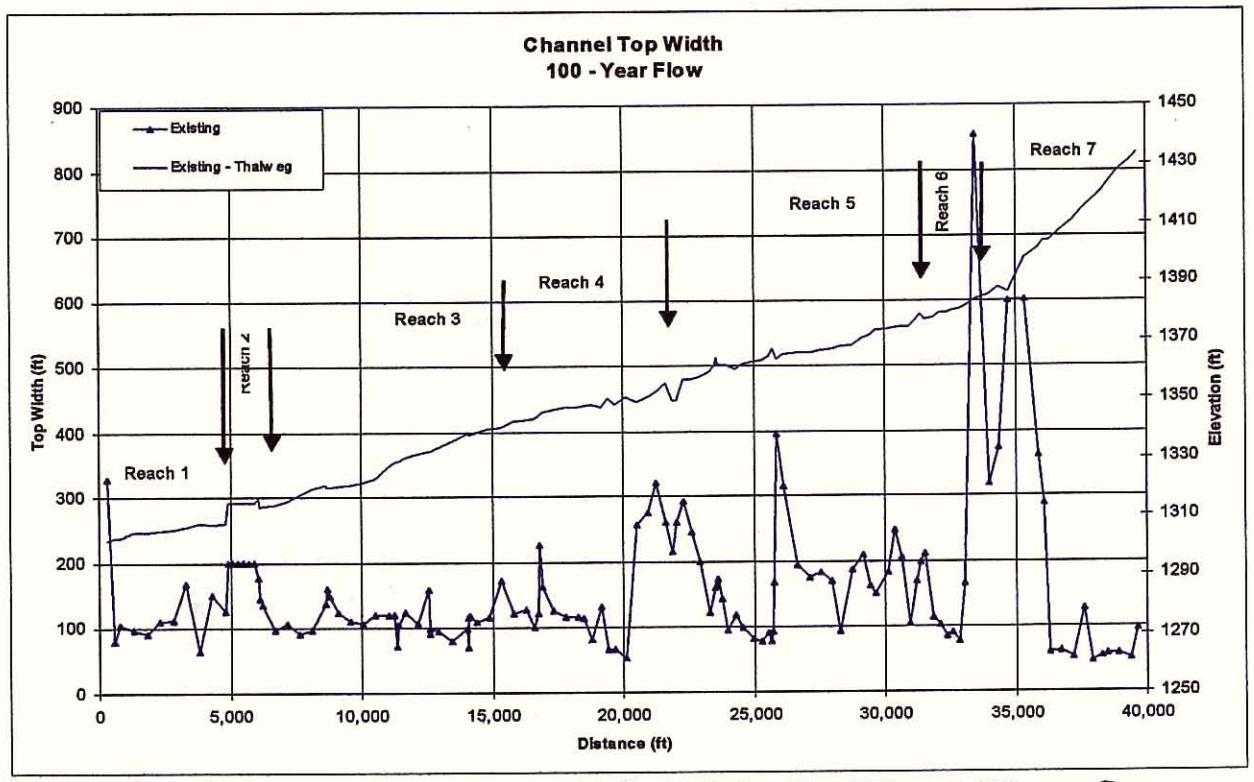

Figure 4-10. SWMB main channel top width for Existing Condition at 100-year flow

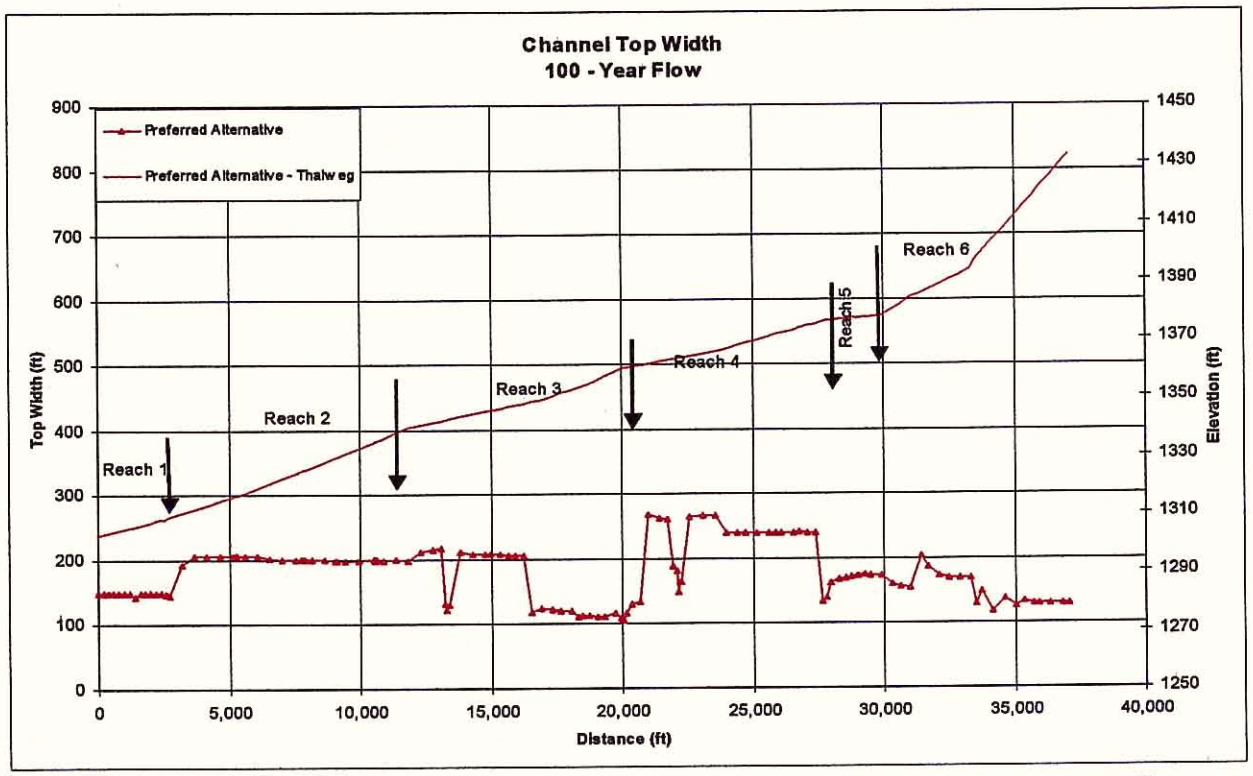

Figure 4-11. SWMB main channel top width for Preferred Alternative at 100-year flow

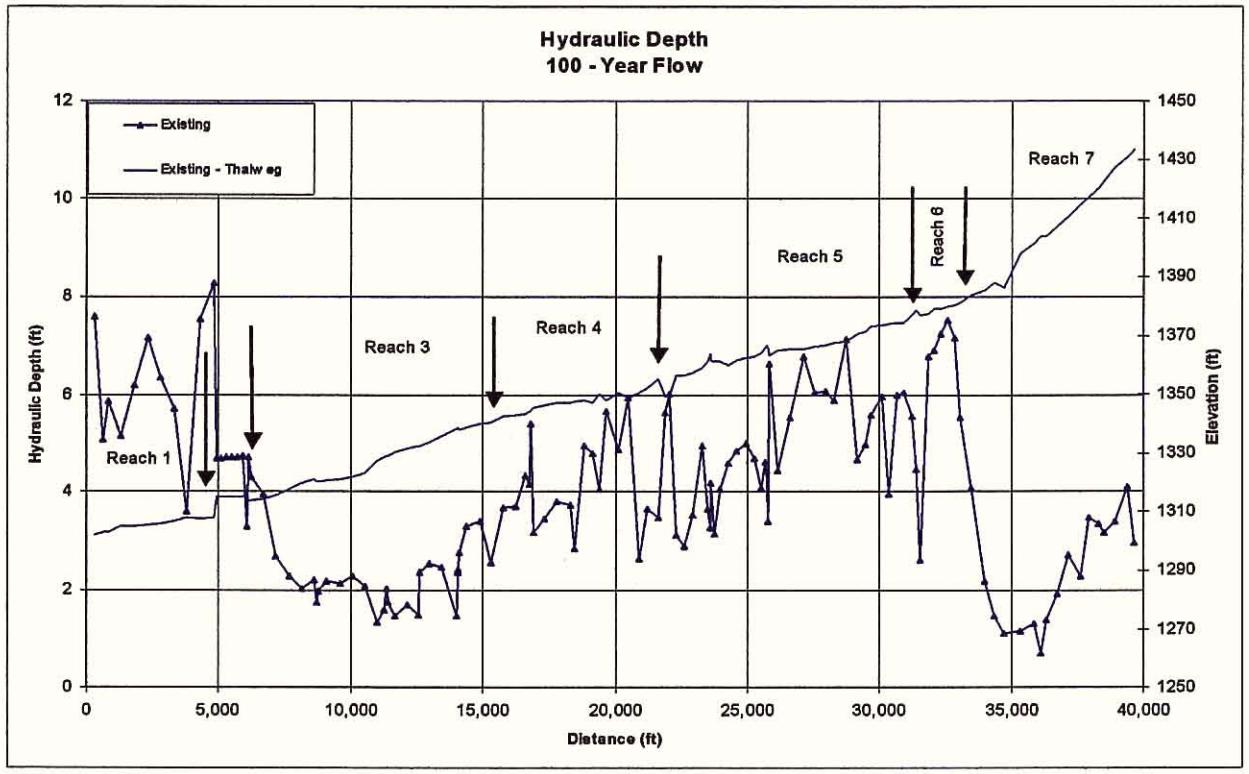

Figure 4-12. SWMB channel hydraulic depth for Existing Condition at 100-year flow

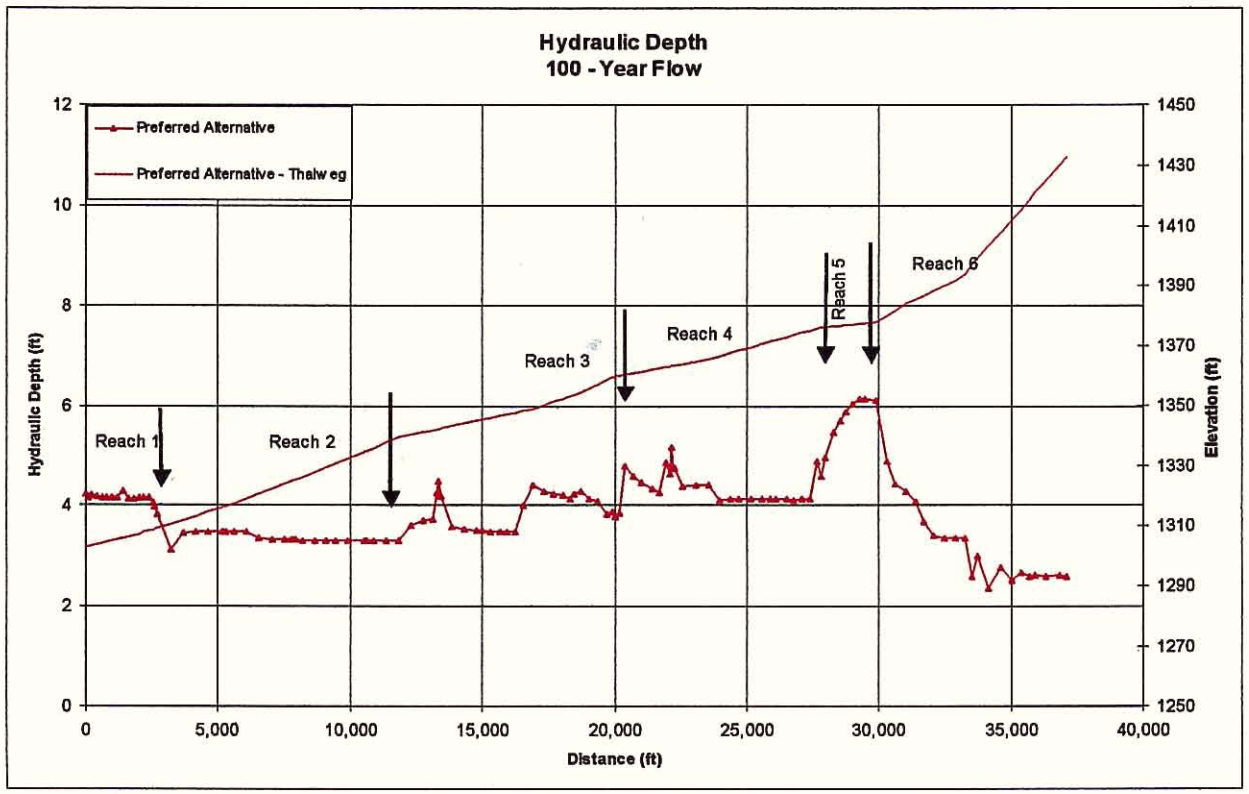

Figure 4-13. SWMB channel hydraulic depth for Preferred Alternative at 100-year flow

4.3.1 Reach Delineation

Based on the results of the hydraulic analysis, reaches of similar hydraulic characteristics were delineated by identifying significant changes in the hydraulic parameter profiles, with the most emphasis placed on changes in the main channel velocity profile. The hydraulic reach limits identified are delineated in Table 4-2. Figure 4-1 shows the schematic of various reaches at Sanokai Wash Main Branch. Having identified reaches of similar hydraulic characteristics, average values for the channel velocity, main channel top width and hydraulic depth for each discharge were developed for each reach and each discharge. These values are shown in Table 5-3.

Table 4-2. Sanokai Wash Main Branch Reaches

Reach	Downstream Location	Upstream Location	U/S Cross Section	Length (feet)
1	New Sanokai Wash Confluence	Higley Road	2,784	6222
2	Higley Road	Bend d/s of Power Road	11,602	15329
3	Bend D/s of Power Road	Sossaman Road	20,242	21620
4	Sossaman Road	Hawes Road	28,000	31530
5	Hawes Road	Riggs Road	29,692	33461
6	Riggs Road	County Line	6,142	39643

Along Sanokai Wash Main Branch, the proposed changes by Huitt~Zollars to channel geometry were considered. The proposed changes included channelizing Sanokai Wash from the EMF to Riggs Road and the Main Branch of Sanokai Wash from its confluence with the East Branch to Hunt Highway (County Line). The proposed changes also included constructing a new Sanokai Wash channel along the Ocotillo Road alignment from Higley Road to the EMF.

4.3.2 Results of Hydraulic Analysis for Sanokai Wash Main Branch

The following sections describe the results of the hydraulic modeling conducted for Sanokai Wash Main Branch for the 100-year return period discharge event by reach, for Existing Conditions and the Preferred Alternative.

4.3.2.1 Reach 6

The average main channel velocity in Reach 6 is 4.7 fps for Existing Conditions and Preferred Alternative. The average main channel hydraulic depth is 1.5 feet for Existing Conditions and 1.7 feet for the Preferred Alternative. This reach has the greatest slope and highest channel velocity within the Sanokai Wash Main Branch. This reach will likely transport all incoming sediment and may have some degradation.

4.3.2.2 Reach 5

The average main channel velocity in Reach 5 is 3.8 fps for Existing Conditions and around 2.7 fps for the Preferred Alternative. The velocity decrease associated with the Preferred Alternative is attributed to the widening of the channel. The average main channel hydraulic depth is 3.5 feet for Existing Conditions and 2.8 feet for the Preferred Alternative. The velocity for this reach is considerably lower than in Reach 7 and the slope is considerably flatter. This will create a potential for sediment deposition. It is reasonable to expect that this potential would exist for lower flows as well.

4.3.2.3 Reach 4

The average main channel velocity in Reach 4 is 3.0 fps for Existing Conditions and for the Preferred Alternative. The average main channel hydraulic depth is 2.6 feet for Existing Conditions and approximately 2.2 feet for the Preferred Alternative. The high velocity upstream of this reach will increase the potential for erosion.

4.3.2.4 Reach 3

The average main channel velocity in Reach 3 is 3.1 fps for Existing Conditions and around 3.5 fps for the Preferred Alternative. The velocity increase associated with the Preferred Alternative is due to the decrease in the channel roughness value. The average main channel hydraulic depth is 2.9 feet for existing conditions and around 2.3 feet for the alternatives. At the upstream end of this reach there is a large depressed area (due to excavation). The local increase in channel gradient could result in a headcut upstream. This depression in the channel will act as a sediment trap that could also cause a reduction in sediment supply to downstream reaches, potentially leading to increased channel degradation and/or bank erosion.

4.3.2.5 Reach 2

The average main channel velocity in Reach 2 is 2.4 fps for Existing Conditions and around 3.3 fps for the Preferred Alternative. The velocity increase associated with the alternatives is due to the proposed channel modifications. The average main channel hydraulic depth is 1.9 feet for Existing Conditions and around 1.5 feet for the Preferred Alternative. This reach is progressively steeper with relatively uniform velocities so sedimentation/erosion problems are unlikely. The top width and the hydraulic depth are fairly uniform throughout this reach.

4.3.2.6 Reach 1

The average main channel velocity in Reach 1 is 3.3 fps for Existing Conditions and the Preferred Alternative. The average main channel hydraulic depth is 2.7 feet for Existing Conditions and around 2.1 feet for the Preferred Alternative. This reach is progressively steeper with relatively uniform velocities so sedimentation/erosion problems are unlikely. The top width and the hydraulic depth are fairly uniform throughout these reach.

4.4 Hydraulic Analysis for Sanokai Wash East Branch

Plots of the channel profile and hydraulic parameters of main channel velocity, main channel top width and hydraulic depth were developed for 2-, 5-, 10-, 25-, 50- and 100-year return period events from the results of the hydraulic models of Existing Conditions and the Preferred Alternative. These plots for the existing conditions and the Preferred Alternative for Sanokai Wash East Branch, for the 100-year return period events are presented as Figures 4-14 through 4-17. The difference in reach lengths in these figures between the existing conditions and the Preferred Alternative is due to the proposed alignment changes from Higley Road to the EMF.

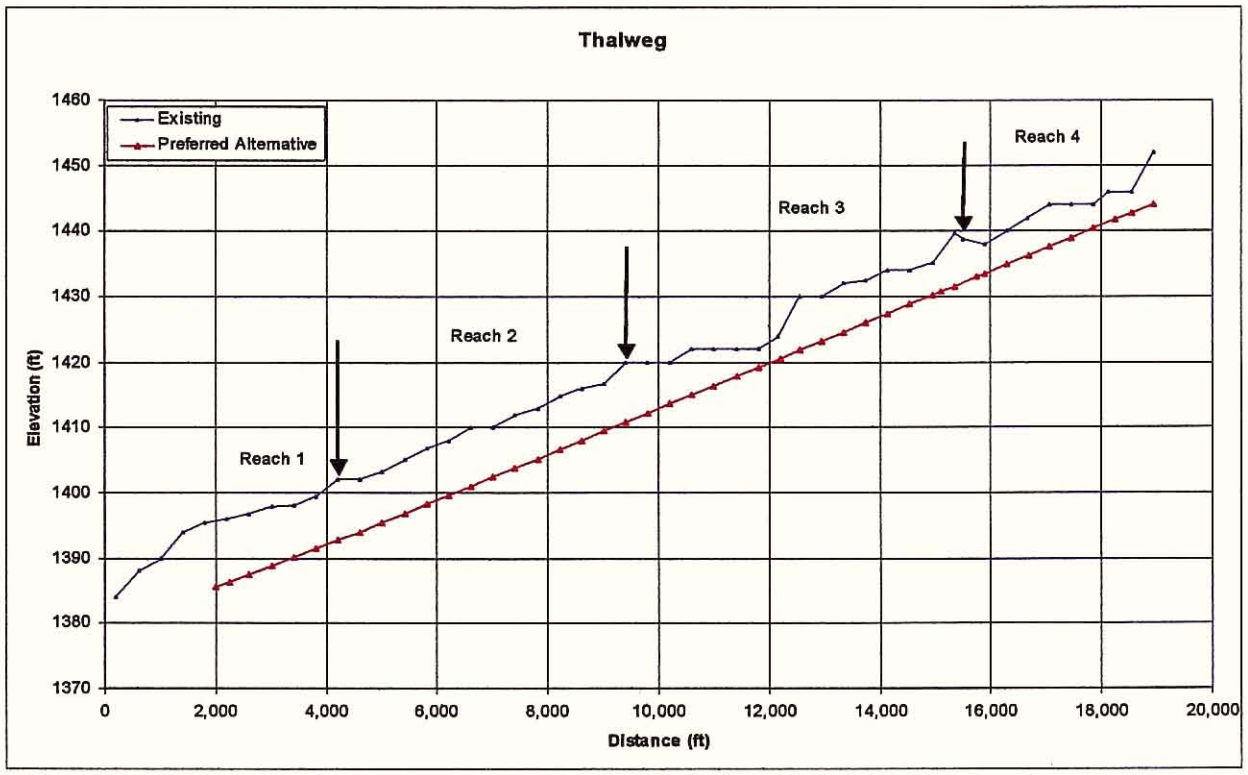

Figure 4-14. SWEB channel thalweg profile for Existing Condition and Preferred Alternative

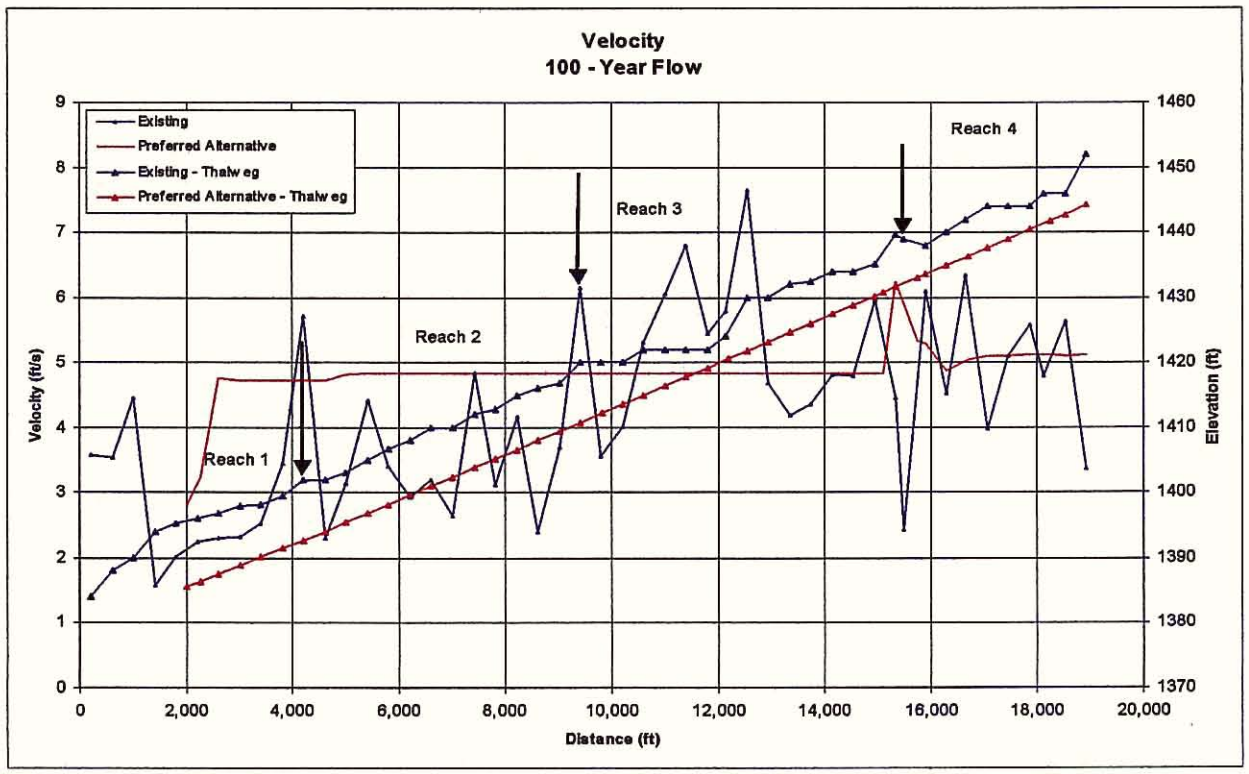

Figure 4-15. SWEB main channel velocity profile for Existing Condition and Preferred Alternative at 100-year flow

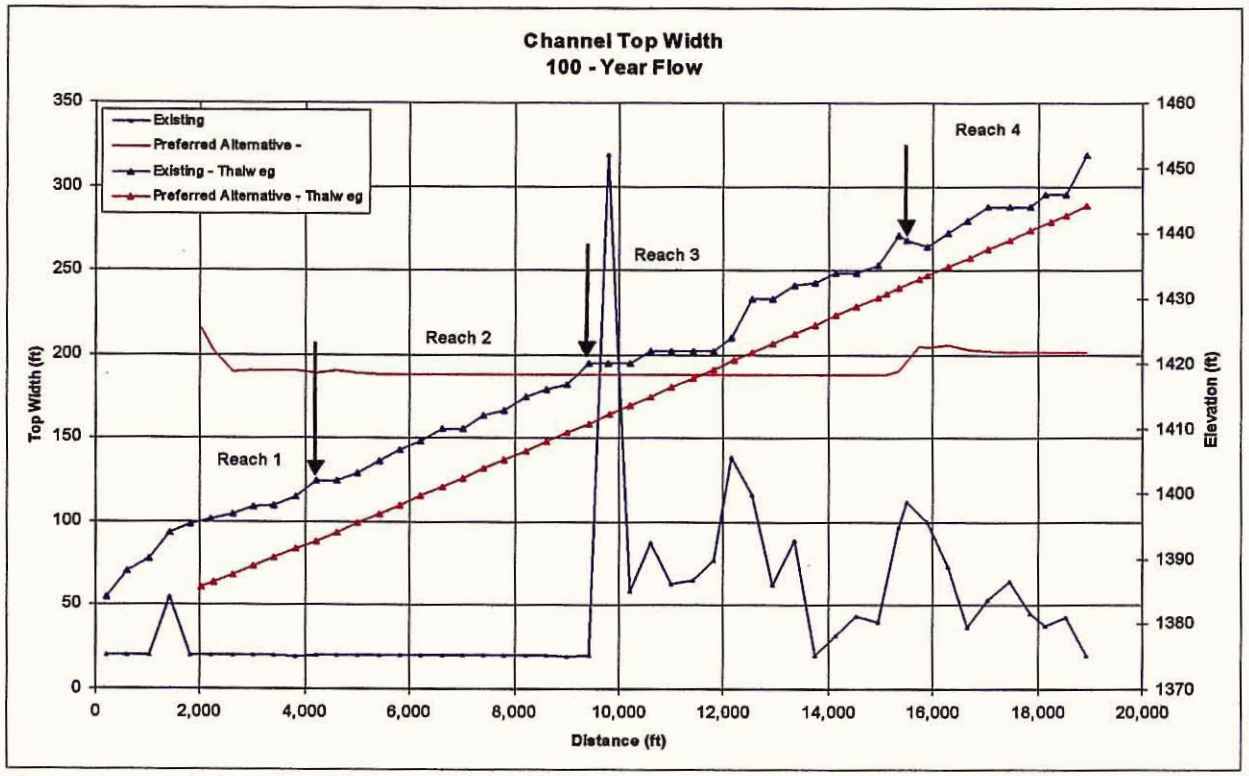

Figure 4-16. SWEB main channel top width for Existing Condition and Preferred Alternative at 100-year flow

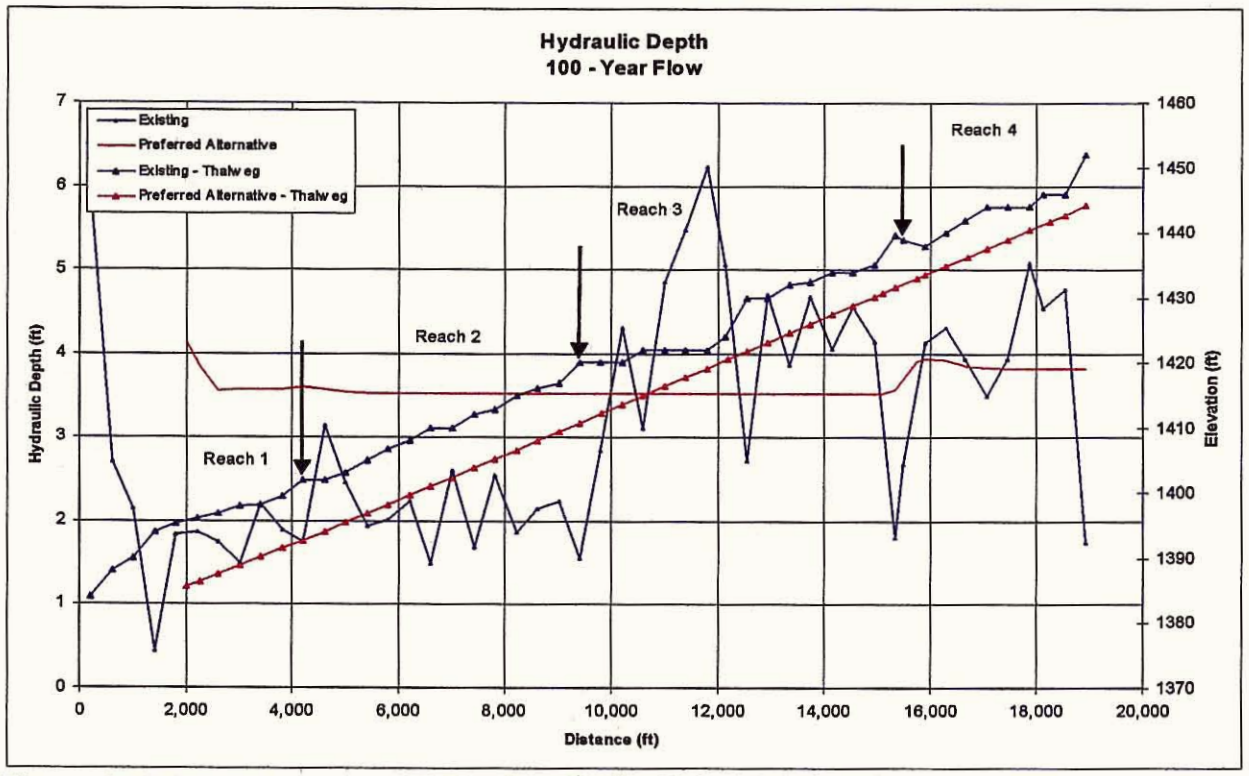

Figure 4-17. SWEB main channel hydraulic depth for Existing Condition and Preferred Alternative at 100-year flow

4.4.1 Reach Delineation

Based on the results of the hydraulic analysis, reaches of similar hydraulic characteristics were delineated by identifying significant changes in the hydraulic parameter profiles, with the most emphasis placed on changes in the main channel velocity profile. The hydraulic reach limits identified are delineated in Table 4-3. Figure 4-1 shows the schematic of various reaches at Sanokai Wash East Branch. Having identified reaches of similar hydraulic characteristics, average values for the channel velocity, main channel top width and hydraulic depth were developed for each reach and each discharge. These are shown in Table 5-4.

Table 4-3. Sanokai Wash East Branch Reaches

Reach	Downstream Location	Upstream Location	U/S Cross Section	Length (feet)
1E	Riggs Road	Ellsworth Road	33,900	4210
2E	Ellsworth Road	Crismon Road	39,474	9411
3E	Crismon Road	Signal Butte	45,015	15492
4E	Signal Butte	County Line	48,164	18930

Along Sanokai Wash East Branch, the proposed changes by Huitt~Zollars to channel geometry were considered. The proposed changes included channelizing Sanokai Wash from its confluence with the East Branch to Hunt Highway (County Line). Also proposed changes included creating a new channel for the East Branch of Sanokai Wash along the south side of Riggs Road from its confluence with the Main Branch to Crismon Road.

4.4.2 Results of Hydraulic Analysis for Sanokai Wash East Branch

The following sections describe the results of the hydraulic modeling conducted for Sanokai Wash East Branch for the 100-year return period discharge event by reach, for Existing Conditions and the Preferred Alternative.

4.4.2.1 Reaches 4E through 2E

The average main channel velocity for the Existing Conditions and all Alternatives do not change significantly. The average main channel hydraulic depths are also similar. No obvious problems with regard to sedimentation/erosion were detected with the current boundary condition configuration.

4.4.2.2 Reach 1E

The average main channel velocity in Reach 1E is approximately 2.8 fps for Existing Conditions and 4.7 fps for the Preferred Alternative. The velocity increase associated with the Preferred Alternative is due to the proposed channel modifications. The average hydraulic depth is approximately 1.9 feet for Existing Conditions and 3.6 feet for the Preferred Alternative.

4.4.3 Summary

A hydraulic analysis of Queen Creek and Sanokai Wash was conducted for 2-, 5-, 10-, 25-, 50- and 100-year return period discharge events, to define the hydraulics of the Preferred Alternative. Reaches of similar hydraulic characteristics were defined based on hydraulic

characteristics such as velocity, slope, top width and hydraulic depth, and hydraulic controls such as bridges, culverts and dip road crossing. The reaches are shown in Figure 4-1. The 100-year flow reach averaged channel velocities range from 5.1 to 6.6 fps for Queen Creek, 4.6 to 6.4 fps for Sanokai Wash Main Branch, and 4.7 to 5.1 fps for Sanokai Wash East Branch.

It should be noted that the original hydraulic model developed by Coe and Van Loo for Power Ranch (Meadow Brook Village) located in Reach 2 may not represent the actual channel geometry as channel modifications were made after their study was conducted. Additional hydraulic analysis using the actual channel geometry in Reach 2 should be conducted during the design phase.

5 SEDIMENT TRANSPORT

5.1 Qualitative Analysis

The following sections provide a qualitative analysis of sediment transport. The analysis includes observations regarding sediment supply and deposition from field reconnaissance, sediment size characteristics of bed and bank material, and potential impacts of gravel mining.

5.1.1 Field Reconnaissance

Observations were made of the watershed, bridge crossings, existing bank-protection measures, channel characteristics and locations of significant erosion and sedimentation problems. A sediment-sampling plan was developed and implemented based on the site visit observations. Locations of sediment sampling sites are shown in Figure 5-1. Field observations of existing channel conditions are given in the following sections.

5.1.1.1 Queen Creek Field Observations

The Queen Creek channel above the Sanoqui Flood Retarding Structure (SFRS) is depositional. Cobble, gravel and sand deposits are very abundant. As Queen Creek enters the SFRS culverts, it makes a sharp bend to the right of about 45 degrees. The inside of the culverts has approximately 6 to 8 inches of sediment deposition consisting mostly of medium to coarse sand and some fine to medium gravels. Deposition is most prevalent at the upstream end along the right side of the culverts as the flow is forced to make the bend from the channel into the culverts. The entrance of the culverts is partially blocked by a headwall, which likely restricts flows during large runoff events. This likely induces additional deposition by increasing backwater. Sediment coarser than sand and fine gravel is not likely to be transported beyond the SFRS culverts.

Below the outlets and energy dissipator for the SFRS culverts, the Queen Creek channel is depositional and consists primarily of medium to coarse sand. The channel is approximately 800 feet long between the SFRS culverts and the Central Arizona Project (CAP) canal culverts. No bed material was present within the CAP culverts, however fine to medium sand was present within the open spaces in the riprap stilling basin below culvert outlets. Below the CAP canal culverts the channel consists of cobble, gravel and sand. The cobble and gravel portion of the bed material is likely the armor layer that has developed due to the reduced supply of sediment from upstream and was likely transported to its current location prior to the existence of the SFRS and CAP canal.

Approximately 1.8 miles below the CAP canal, between Schnepf Road and Kenworthy Road, a portion of the channel has been excavated as part of a gravel mining operation. The gravel pits are approximately 30 to 40 feet below the existing channel. No significant flows have occurred in Queen Creek since the excavation began. It is expected that during the next significant flow event, head cutting will occur upstream of the pit. This may extend far enough upstream to impact the Schnepf Road crossing. It could possibly extend further upstream to the CAP canal culvert outlets. Additional analysis should be made to assess the potential impacts of head cutting associated with the gravel pits at these locations. The gravel pits will also trap a large amount of bed material. This would starve the downstream of sediment, potentially creating erosion problems downstream of the pits. Additional active and abandoned gravel pits exist

along the margins of Queen Creek between Rittenhouse Road and Schnepf Road. These are separated from the channel by levees composed of extracted pit material. The levees are not engineered to resist erosion by the creek during flood events so are expected to have the same erosion potential as the native bank material. Channel avulsion and headcutting should be expected at these locations if mitigation measures are not employed. Active gravel mining operations end at the Vineyard Road dip crossing. Downstream of Vineyard Road, the channel has a braided planform within the confines of levees on both sides of the channel suggesting that this reach remains depositional.

Between Rittenhouse Road and Power Road the channel is relatively confined between levees. There were no signs of significant deposition or erosion suggesting this is a transport reach. The abutment of Rittenhouse Road bridge showed signs of scour, however this may be due to local hydraulic conditions associated with the bridge and not general degradation within the reach.

Below Power Road, the Queen Creek channel consists primarily of sand and has been altered as part of a housing development. Channel alterations include increased channel width, flatter bank slopes and a steeper channel slope. It should be noted that no significant flows have occurred in Queen Creek since the channel alteration occurred, thus the response of the channel to the alterations is unknown. This reach may be a potential site for sedimentation due to the increased channel width and the existence of two bridges (multiple arch and Recker Road) that will likely create backwater during large flows. However, the increased slope associated with the channel alterations may cause erosion in this reach. The downstream end of the channel alterations is at the Recker Road bridge. From this point the channel is at an adverse slope for several hundred yards downstream due to the upstream channel alterations. Sedimentation is likely to occur at this location.

Queen Creek makes a sharp left turn to the south at Higley Road. The bed consists mostly of medium sand. From Higley Road down to the confluence with the EMF, the channel appears to be slightly depositional. Just above the confluence of Queen Creek with the EMF is an inline sedimentation basin. No significant deposition was apparent, however, according to the Flood Control District significant sedimentation did occur during an event in 1991-92 due to a breakout of the EMF. However, it is unknown if the EMF or Queen Creek was the main source of the sediment (personal communication with Huitt~Zollars, 2000).

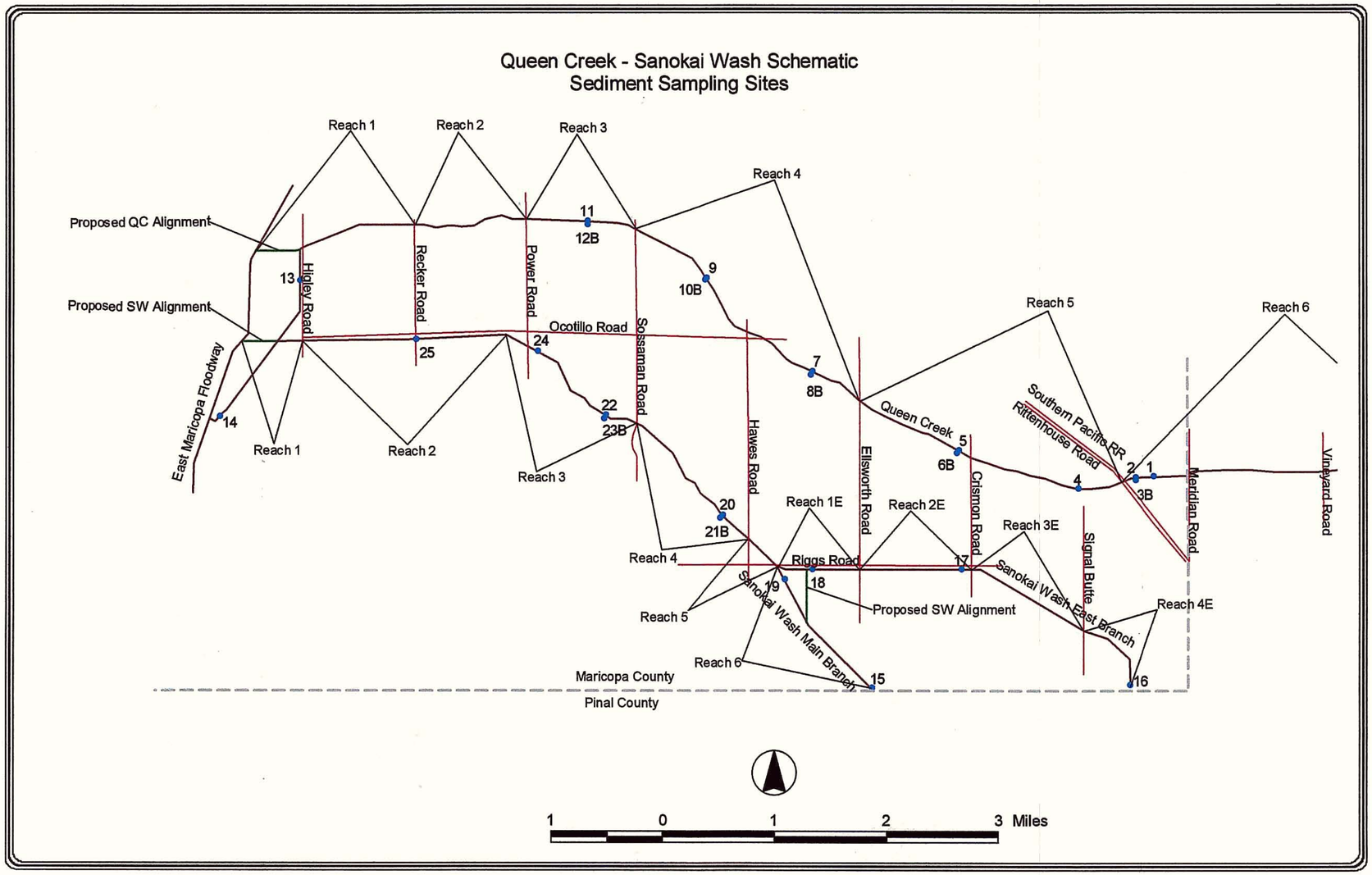

Figure 5-1. Schematic of Queen Creek and Sanokai Wash show sediment sampling sites.

5.1.1.2 Sanokai Wash Field Observations

The entire area surrounding the upper extent of the Sanokai Wash Study area appears to be an alluvial fan. The ground slopes gently downward to the north with a poorly defined system of small washes. Sanokai Wash Main Branch enters the study area through culverts under Ellsworth Road. These culverts and road crossing may induce sedimentation upstream of this point as they likely create a backwater effect at this location during large flow events. However, significant sedimentation was not apparent from field observations.

From Ellsworth Road to San Tan Road, the channel is well defined by levees forming a relatively small trapezoidal shaped channel approximately 150 feet wide at the top and approximately 30 feet wide along the bottom with a bed that consists primarily of sand. This channel appears to have been constructed with the purpose of concentrating flow that would otherwise be unconfined. There did not appear to be any significant sedimentation or erosion problems within this section of the Sanokai Wash channel. The channel ends at San Tan Road. From this point to Riggs Road water must travel overland. Sediment carried down the channel upstream of San Tan Road is likely deposited at and below San Tan Road as the flow is dispersed by the San Tan Road crossing. The dispersion of flow causes it to become less concentrated and therefore have a reduced sediment transport capacity. It is also likely that a large portion of the flow infiltrates into the existing soils, further reducing the sediment transport capacity and causing sediment deposition.

Below Riggs Road the channel is fairly well defined as it passes the landfill and Hawes Road. The right bank of the channel in confined by the edge of the landfill while the left bank is less confined. There appears to be an abundance of sediment available within the channel in this section suggesting this to be a potential area of deposition. Above Sossaman Road the channel remains somewhat confined, but shows signs of past migration as the area surrounding the channel has fairly wide and somewhat irregular topography. There is also an abundance of small rills and gullies creating a local supply of sediment to the main channel.

Below Sossaman Road the channel is confined by housing developments on both sides down to the dip crossing at Via del Jardin Road. This location did not appear to be a significant area of erosion or deposition. Downstream of Via del Jardin Road, the channel narrows to a well-defined trapezoidal channel, which is confined by levees and surrounded by farm fields. Just above Higley Road, the Sanokai Wash channel changes from a well-defined trapezoidal shaped channel to a poorly defined irregular channel. Culverts carry low flows under Higley Road, while high flows are likely dispersed over Higley Road causing overland flow and sedimentation to occur at and downstream of the road crossing. Downstream of Higley Road, the Sanokai Wash channel becomes more defined but still irregular as it approaches Queen Creek. Sanokai Wash and Queen Creek join at a low spot in the left bank levee along Queen Creek. There did not appear to be any signs of significant sediment inputs from Sanokai Wash into Queen Creek at the confluence location. It is likely that the majority of the sediment carried to the downstream end of Sanokai Wash deposits in the poorly defined sections in the vicinity of Higley Road.

5.1.2 Sediment Size Characteristics

Twenty-five (25) sediment samples were collected consisting of 18 bed and 7 bank samples. Laboratory analysis consisted of dry sieve analysis. Sediment sizes smaller than a No. 200 sieve

(0.0625 mm) were not analyzed. Particle size distributions were developed for the samples and are presented in the following sections.

5.1.2.1 Queen Creek Bed Material

Bed material size distributions for various locations along Queen Creek are shown in Figure 5-2. Bed material size distributions for locations near Rittenhouse Road range from coarse gravel to fine sand with a D_{50} of about 1.5 mm. As seen in Figure 5-3 the size of bed material below Rittenhouse Road decreases significantly. The D_{84} changes from approximately 6 mm down to approximately 1 mm. However, the D_{50} only drops to approximately 0.5 mm. The coarsest material deposits upstream of Rittenhouse Road as the transport capacity of the channel is significantly reduced.

5.1.2.2 Queen Creek Bank Material

Bank material size distributions for Queen Creek are shown in Figure 5-4. They are slightly finer than the bed material samples except in the reaches above Rittenhouse Road. At this location, the channel material appears to be much coarser due to armoring. Bank material ranges from medium gravel to very fine sand with a D_{50} of approximately 0.1 mm. Some bank material samples contained greater than 50% fines (less than 0.0625mm), which were not analyzed. The D_{84} ranged from 0.2 to 0.4 mm, however no apparent longitudinal pattern in material size was found. This is likely due to disturbances such as gravel mining and levee construction. The size distributions and lack of cohesiveness observed in the field would suggest that the bank material of Queen Creek is easily eroded by significant flows.

5.1.2.3 Sanokai Wash Bed Material

Bed material size distributions for various locations along the Main Branch of Sanokai Wash are shown in Figure 5-5. Bed material size distributions for locations near the Maricopa-Pinal County line range from coarse gravel to very fine sand with a D_{50} of about 1.5 mm. As seen in Figure 5-6 the size of bed material below the county line decreases significantly. The D_{84} changes from approximately 4 mm to approximately 0.3 mm and the D_{50} is less than 0.0625 mm in the vicinity of the confluence with the East Branch. The coarsest material deposits are upstream of Ellsworth Road as the transport capacity of the channel is significantly reduced. This may be due to the flow restriction and backwater created by the culverts at Ellsworth Road. The sediment transport capacity is further reduced in the vicinity of Riggs Road as the size distribution shows an abundance of silt size or smaller material.

Bed material size distributions for various locations along the East Branch of Sanokai Wash are shown in Figure 5-8. Bed material size distributions for locations near the Maricopa-Pinal County line range from medium gravel to very fine sand with a D_{50} of about 1.2 mm. Below this location at Riggs Road, the D_{84} changes from approximately 2 mm to approximately 0.1 mm and the D_{50} is less than 0.0625 mm. This would suggest that deposition in silt size or smaller material occurs in this reach.

5.1.2.4 Sanokai Wash Bank Material

Bank material samples were only collected for the Main Branch of Sanokai Wash. The size distributions for these samples are shown in Figure 5-7. Bank material sizes are slightly finer than the bed material and range from coarse sand to silt with a D_{50} of about 0.08 mm. The size

distributions and lack of cohesiveness observed in the field would suggest that the bank material on Sanokai Wash is easily eroded by significant flows.

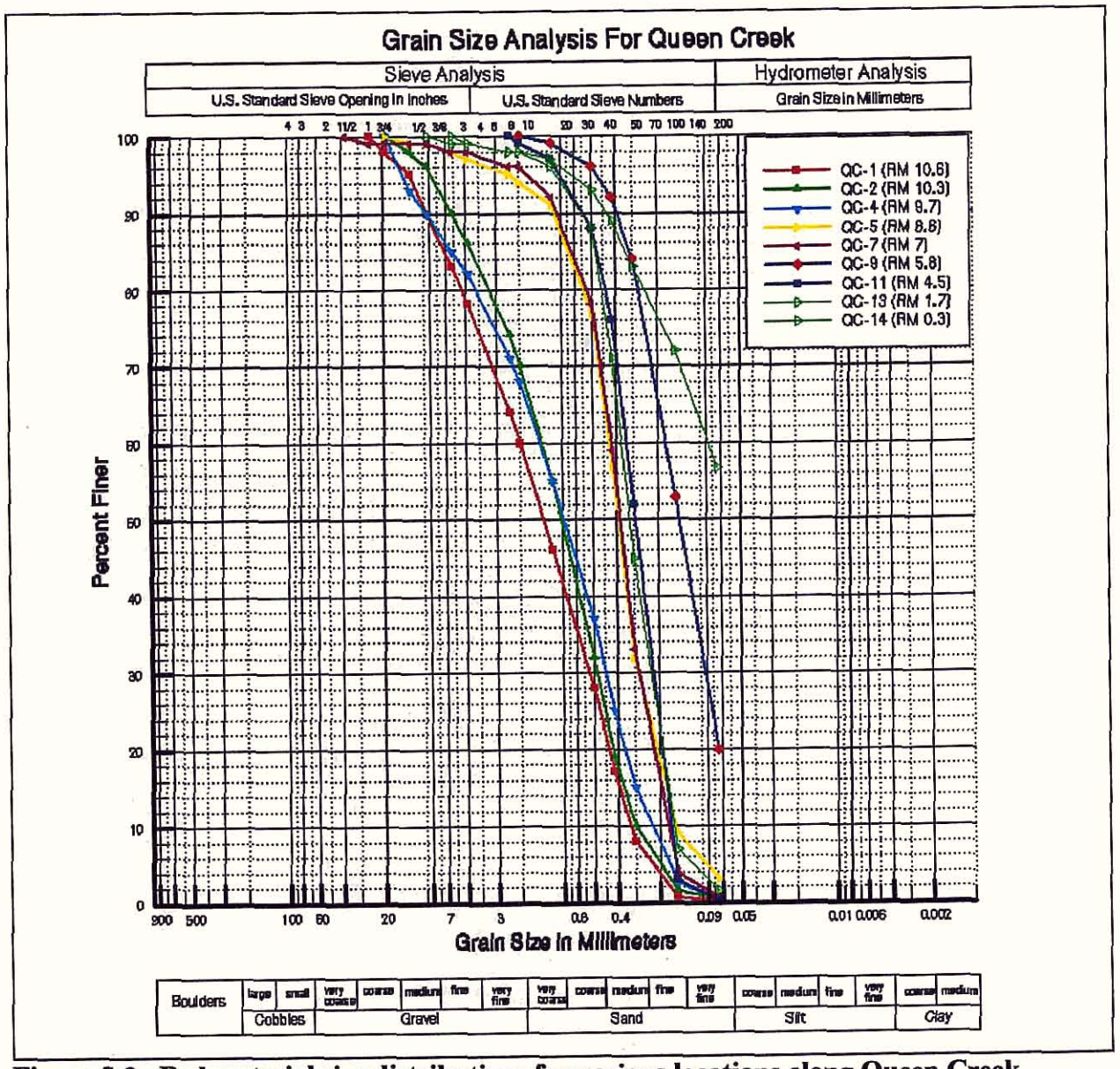

Figure 5-2. Bed material size distributions for various locations along Queen Creek

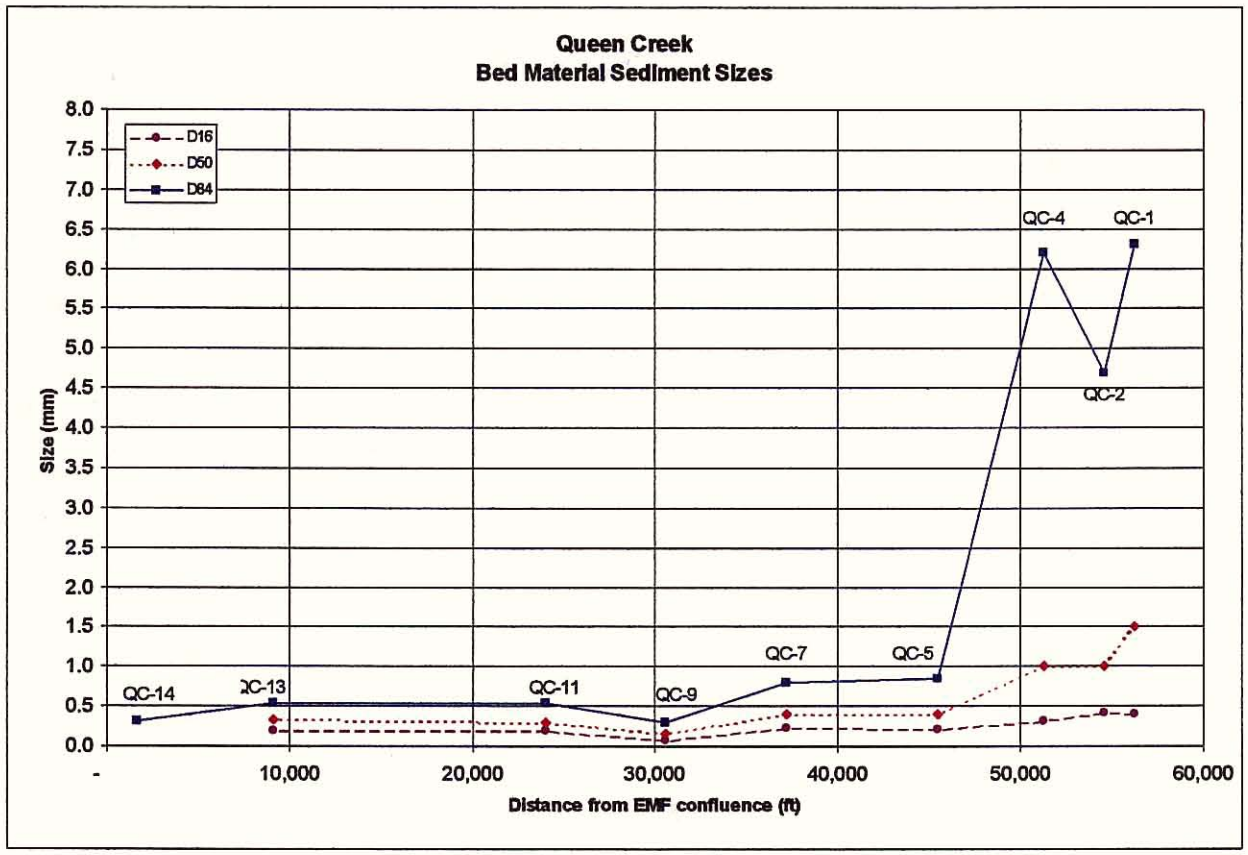

Figure 5-3. Longitudinal variation in bed material sizes for Queen Creek.

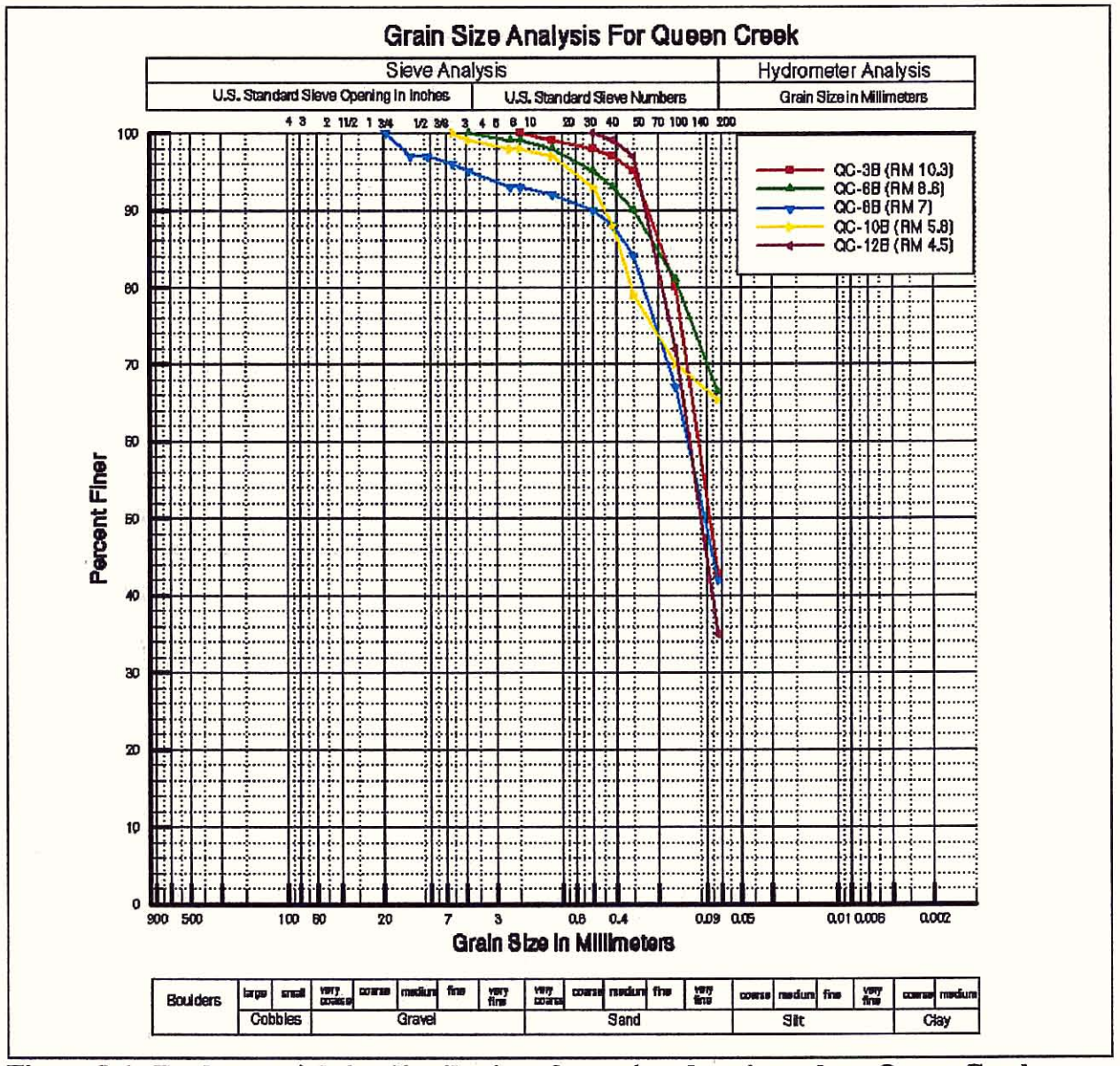

Figure 5-4. Bank material size distributions for various locations along Queen Creek

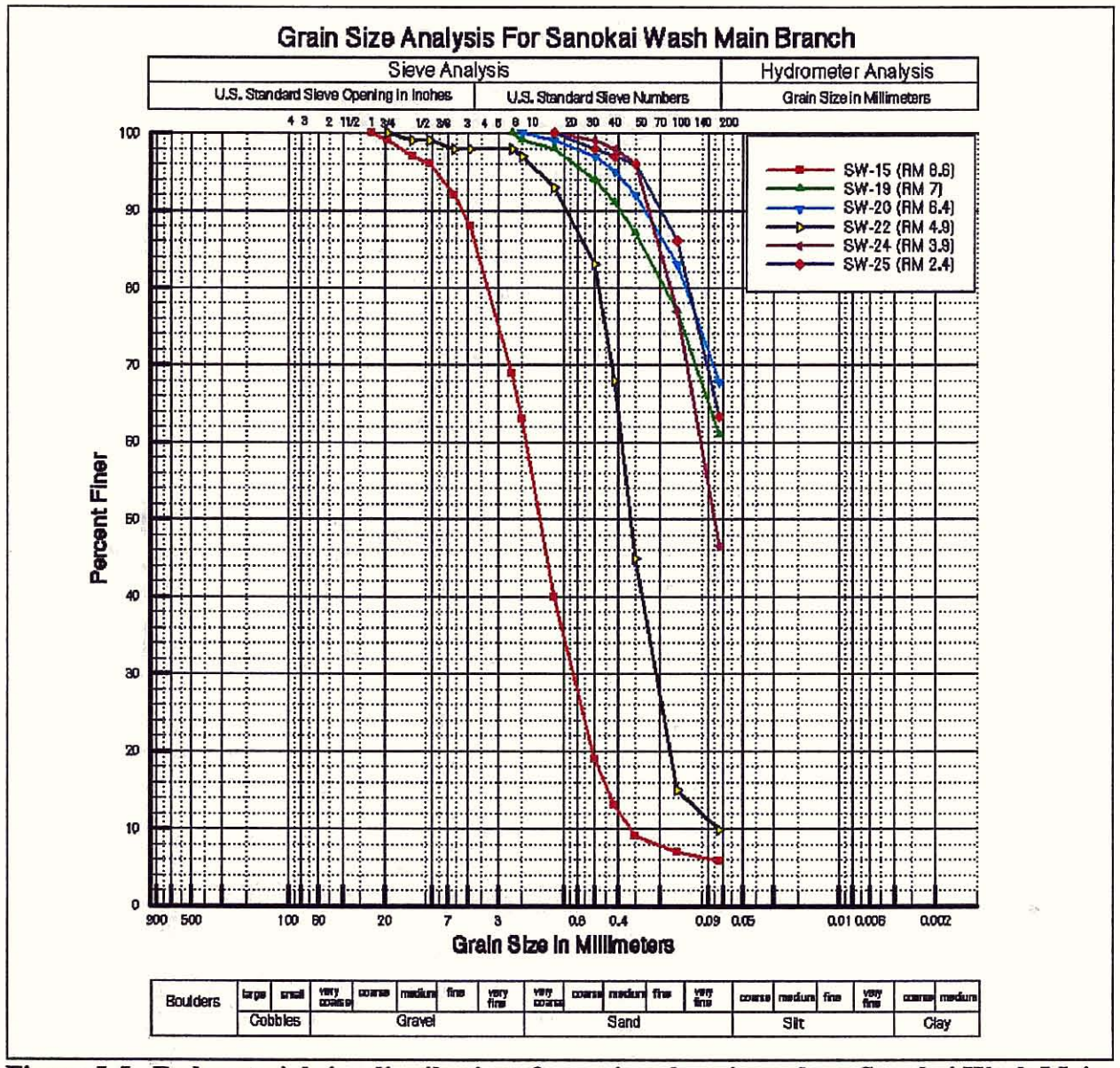

Figure 5-5. Bed material size distributions for various locations along Sanokai Wash Main Branch

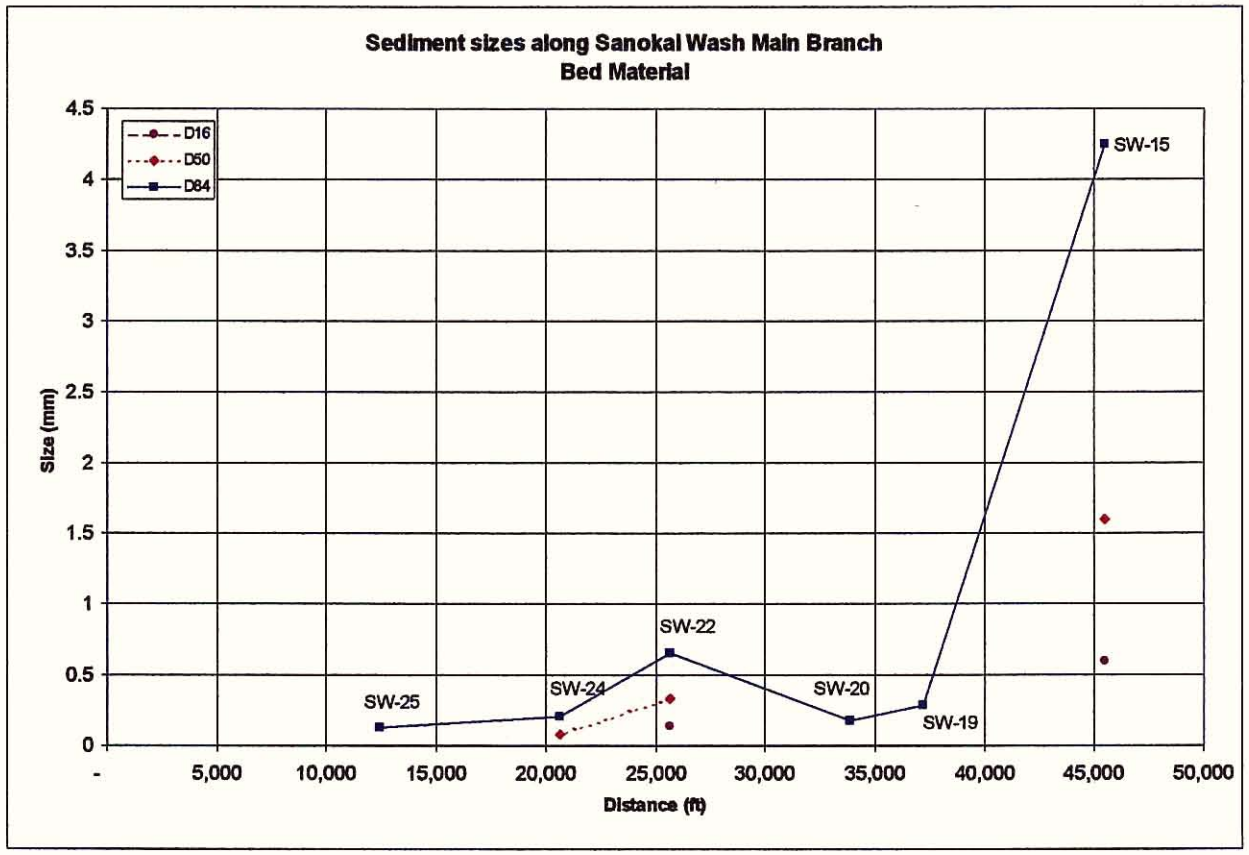

Figure 5-6. Longitudinal variation in bed material size distribution for Sanokai Wash Main Branch.

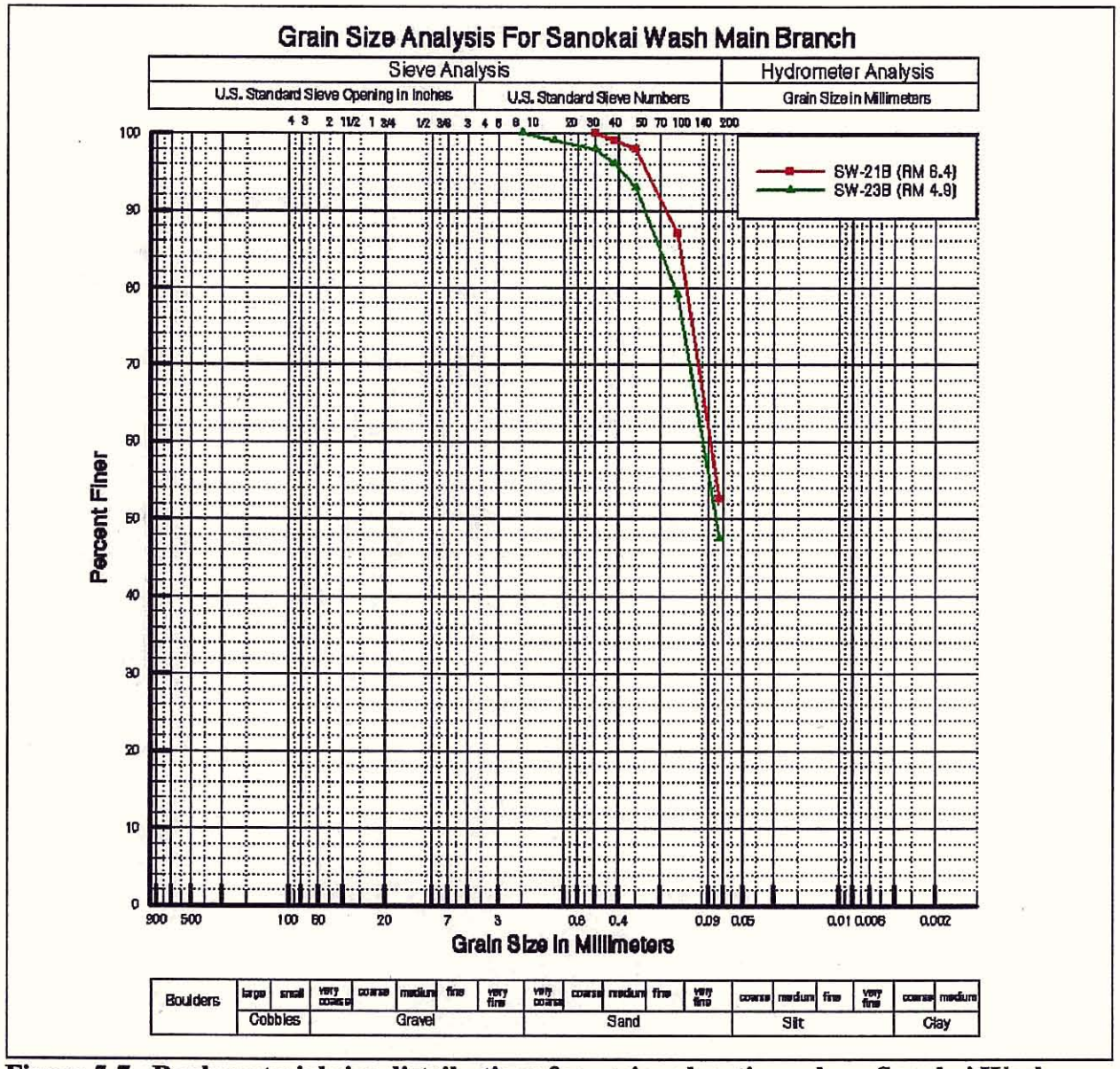

Figure 5-7. Bank material size distributions for various locations along Sanokai Wash Main Branch

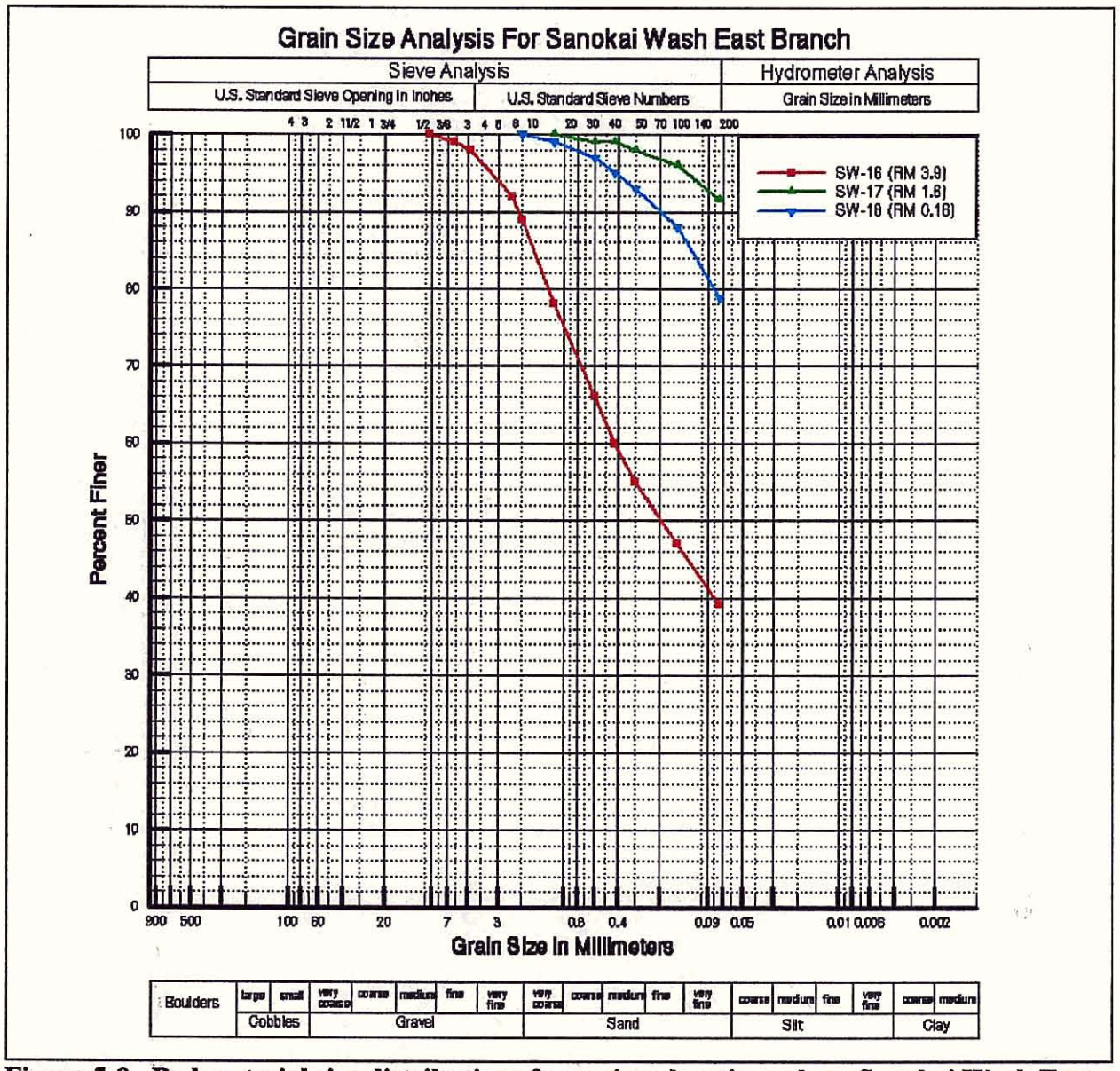

Figure 5-8. Bed material size distributions for various locations along Sanokai Wash East Branch

5.1.3 Gravel Mining Impacts

The upper reaches of Queen Creek have historically been the location of gravel mining operations. Currently, several abandoned off-channel pits exist near the edge of the channel. There is also a large active gravel mining operation located approximately 0.5 miles downstream of Schnepf Road. This operation is removing material from the active channel of Queen Creek. There is a 30 to 40 foot deep pit in the center of the Queen Creek channel. The potential for channel avulsion at this location is very high. Additionally, several other abandoned pits located along the edge of the channel have a high potential for avulsion as levees created to protect the pits from the channel are created from existing channel material that is not engineered to protect against large flow events. Avulsions caused by breaches into gravel pits can create may impacts

that require further characterization. The following sections describe the potential impacts associated with avulsions into gravel pits.

5.1.3.1 Characterization of Impacts from Avulsion into Gravel Pits

Impacts from the avulsion of the creek into a floodplain gravel pit can be characterized as short-term or long-term. Short-term impacts are those changes to the morphology of the channel that take place during and shortly after the avulsion. Long-term impacts are those that continue to affect the morphology of the channel well into the future. Additionally, these impacts can be described by their location in relation to the avulsion site. Table 5-1 summarizes the impacts from avulsion described in this section.

Upstream Impacts

Short-term impacts upstream of an avulsion into a gravel pit include head cutting, which causes degradation of the bed and increased channel slope, channel armoring, and/or an increase in the channel armor size (bed coarsening). When a gravel pit is breached, a localized difference occurs in the energy between the higher elevation flow in the channel and the lower elevation water (if present) in the pit causing a steep energy gradient to form. The increased energy gradient will increase the sediment transport capacity of the channel, creating a demand for sediment. If the material forming the armor layer on the channel bed is too small to resist the forces created by the energy imbalance, the channel bed material will erode and be transported downstream. This erosion will then propagate (head cut) upstream until the channel bed has formed a stable slope and armor layer that will resist the forces of the flow. The upstream extent of head cutting is controlled by the size characteristics of the bed sediment, the hydraulics associated with the flow, and the existence of any channel grade controls such as a geologic outcrop or man-made structure.

Long-term impacts include continued bed coarsening, channel incision, bank failure due to increased bank heights and slopes caused by the incision, and reduced sediment deposition due to the increased channel slope. During subsequent high flow events, the channel bed may continue to adjust to the changes in hydraulics. Higher flow events could cause additional disruption of the armor layer, increasing degradation and coarsening the bed. The down cutting of the bed could cause an increase in channel bank height and degradation along tributaries. As the channel erodes the higher banks, an increase in the amount of material input to the stream will occur for the same amount of lateral erosion. This will help satisfy the transport capacity of the channel and cause a reduction in the rate of lateral migration. At the same time, excessive bank heights can cause instability and increase the chance of slope failure. The increased slope associated with head cutting will increase the sediment transport capacity of the channel and reduce the amount of material that would otherwise deposit in the degraded channel reach. Upstream channel degradation can also affect the stability of hydraulic structures such as levees or bridges by undermining support structures (Collins and Dunne, 1990). These structures include Schnepf Road Bridge and the CAP canal.

Local Impacts

An avulsion into a gravel pit has many potential localized impacts. The specific impacts are dependent on the characteristics of the channel and gravel pit at the avulsion site. Typically, short-term impacts in the immediate vicinity of an avulsion can include an immediate change in

hydraulic conditions from a high velocity shallow channel to a low velocity deep and wide lake-like system. A delta will develop at the entrance to the pits formed from material that composed the high ground that formerly divided it from the channel and from material removed from the upstream channel by head cutting. Typically, the former gravel pit will act as a deposition zone for sediment, holding a large portion of the sediment load that might otherwise been deposited within or have been transported through the reach. Additionally, a section of channel will be abandoned as the stream changes course and flows through the gravel pits. During higher flows, the stream may use the abandoned channel as a secondary conveyance. This channel may act as a deposition zone for finer material such as sands and silts that are carried as suspended load during high flows.

In the long-term, the former gravel pit will continue to be a wide and deep channel having very low velocities until substantial filling with sediment has occurred. As the delta continues to form and grow at the entrance to the pits, flow conveyance and sediment transport into the pit will decrease. Velocities will increase and depth will decrease at the entrance to the pit while further downstream, the velocities continue to be slow in the wide and deep channel. Additionally, the gravel pits can act as flood storage during high flows, which may reduce downstream flood levels. However, this will decrease over time as the pits fill with sediment.

Downstream Impacts

As the former gravel pit traps sediment, the supply of sediment to the downstream channel is curtailed. Until the sediment transport conditions in the section of the channel within the pits return to pre-avulsion conditions, bed degradation, bed coarsening, and increased bank erosion along the downstream channel may occur. With a reduced supply of sediment to the downstream reaches, the sediment transport capacity will not be fulfilled. This may cause erosion of the channel bed and/or banks. The stream will transport the finer sediments downstream leaving behind the coarser material, causing the bed material to coarsen or armor, protecting against subsequent high flow events. Reduced upstream sediment supply may cause the channel bed elevation to lower until it becomes controlled by armoring. To accommodate the sediment supply deficit, bank erosion may occur resulting in channel widening.

Another possible impact to reaches located downstream of the avulsed pit is reduced flood levels. The increased width and depth associated with the geometry of the gravel pit creates additional flood storage. The amount of reduction in flood levels provided by the changed geometry is related to the volume of additional storage and the magnitude and duration of the flood event.

Table 5-1. Summary of the possible effects of a stream avulsing into a gravel pit.

Element of Avulsion	Nature of Impact		
	Upstream	**Local**	**Downstream**
Geomorphic Characteristics	• Incision of channel • Increased gradient • Coarsening of bed • Undercutting and erosion of banks • +/- lateral migration rates	• Alluvial fan development • Reshaping of pits • Abandonment of former channel • Loss of natural channel geometry	• Increased lateral migration • Increased channel width
Sediment Transport	• Increased sediment transport capacity • Reduction in bed load deposition	• Deposition of sediment in pits • Erosion of gravel pit banks	• Reduced sediment supply • Erosion of bed • Coarsening of bed • Increased bank erosion
Hydraulics	• Increased slope • Increased velocities • Decreased normal depth • Increased bed roughness	• Decreased slope • Increased channel depth • Increased channel width • Reduced bed roughness	• Increased bed roughness
Hydrology		• Increased flood storage	• Reduction of flood levels • Attenuation of flood peaks

5.1.4 Summary

Sediment size distributions for Queen Creek and Sanokai Wash suggest that the bed material generally consists of fines in the downstream direction as the sediment transport capacity of the channel is reduced. Significant deposition of coarse material occurs above the Sanoqui Flood Retarding Structure (SFRS) upstream of the study area. Additionally, coarse materials such as cobbles and gravels do not appear to be present with the study limits for Sanokai Wash, suggesting that deposition of these materials occurs further upstream. However, cobble and gravel sized materials are present within the upper reaches of Queen Creek. This material was likely transported to this area prior to the placement of the SFRS and the CAP canal. The major sediment size fractions transported through the study reaches of Queen Creek and Sanokai Wash are sands, silts and clays.

Bed material sizes range from very coarse gravel to silts and finer in Queen Creek while bed material sizes for Sanokai Wash were slightly finer ranging from coarse gravel to silts and finer. Bank material is generally finer than bed material and is non-cohesive. This would suggest that high flows with sufficient velocities could erode the banks along both Queen Creek and Sanokai Wash.

The large amount of fine material forming the bed of Sanokai Wash in the vicinity of Riggs Road suggests that this area is depositional. The break in slope at this location reduces the channel velocities thus reducing the sediment transport capacity. The reduced transport capacity causes sediment to deposit at this location. This area may require occasional excavation to maintain flood conveyance.

Gravel mining in and along Queen Creek has the potential for impacting the channel not only at the locations of the pits themselves but also at locations upstream and downstream of the site. Further analysis of the potential impacts due to breaching of existing off-channel pits as well as in-channel pits should be conducted. Infrastructure such at the upstream road crossing at Schnepf Road and the culverts through the CAP canal may be impacted by headcutting associated with the channel avulsing into the pits. Channel instability both upstream and downstream of the pits should be expected.

5.2 Quantative Analysis

The following section provide a quantitative analysis of sediment transport conducted to define the armoring potential, sediment transport capacity, sediment continuity, local scour potential, and bank erosion potential for Queen Creek and Sanokai Wash.

5.2.1 Armoring Potential Evaluation

An evaluation of relative channel stability can be estimated based upon incipient motion analysis and armoring potential. The incipient motion particle size was estimated for various reaches using the Shield's parameter, as defined in Arizona Department of Water Resources State (ADWR) Standard 5-96, State Standard for Watercourse System Sediment Balance, dated September 1996.

Incipient motion particle sizes for the Preferred Alternative were estimated for the 5- and 100-year return period discharge events for Queen Creek and Sanokai Wash Main and East Branches. The incipient motion particle size results were used to estimate the degradation depth required to form an armor layer. The depth required to form an armor layer (Y_d) is estimated from the following equation.

$$Y_d=Ya[(1/P_c)-1]$$

where Ya is the thickness of the armor layer and P_c is the decimal fraction of material coarser than the material forming the armoring layer. The thickness of the armor layer (Y_a) typically ranges from one to three times the armor particle diameter (D_c). For the purposes of this study, an armor layer of 3 times the critical diameter (D_c) was chosen. The results of the analysis are summarized in Tables 5-2 through 5-4.

5.2.1.1 Results of Armoring Analysis for Queen Creek

Incipient motion particle sizes ranged from approximately 3 to 10 mm and 4 to 13 mm for the 5- and 100-year flow events, respectively. Channel degradation ranges from approximately 0.4 feet to 2.0 feet for the 5-year flow and approximately 1.0 feet to 3.0 feet for the 100-year flow except for Reaches 2 and 3 where the bed will not armor. Bed material of sufficient size and quantity is not available for the formation of an armor layer in these reaches. Critical diameters estimated for Reaches 2 and 3 ranged from 3.2 to 5.3 mm and 4.0 to 6.8 mm for the 5- and 100-year flow, respectively. The D_{90} for both reaches is 0.7 mm.

5.2.1.2 Results of Armoring Analysis for Sanokai Wash - Main Branch

Incipient motion particle sizes for Sanokai Wash Main Branch ranged from approximately 1 to 4 mm and 2 to 6 mm for the 5- and 100-year flow events, respectively. Channel degradation ranged from approximately 0.2 feet to 1.3 feet for the 5- and 100-year flow, respectively for Reach 5. The channel will not armor in Reaches 1 through 4 and 6 for the 5 and 100-year flow. Bed material of sufficient size and quantity is not available for the formation of an armor layer in these reaches. Critical diameters estimated for these reaches ranged from 1.1 to 3.6 mm and 2.1 to 5.8 m for the 5- and 100-year flow, respectively. The D_{90} for these reaches ranges from 0.2 to 0.9 mm.

5.2.1.3 Results of Armoring Analysis for Sanokai Wash - East Branch

Incipient motion particle sizes for Sanokai Wash East Branch ranged from approximately 1 to 3 mm and 2 to 6 mm for the 5- and 100-year flow events, respectively. The channel will armor in all reaches for 5-year flow. Channel degradation ranges from approximately 0.4 feet to 3.6 feet for the 5-year flow. Channel degradation in Reach 4E is approximately 3.5 feet for the 100-year flow.

The channel will not armor in Reaches 1E through 3E for the 100-year flow. Critical diameters estimated for these reaches ranged from 1.7 to 2.4 mm for the 100-year flow. The D_{90} for these reaches ranges from 0.08 to 0.2 mm.

Table 5-2. Summary of Armoring Analysis for 5- and 100-Year Flows at Queen Creek

Preferred Alternative - 5-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Slope	Manning's "n"	D90 (mm)	Shields Armoring Size: Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)	Ya (ft)	% of bed material > Dc	Yd (ft)
7	72630.0000	72324.7	1557.35	2.39	4.74	0.005009	0.032	9.5	0.018	0.027	0.149	0.0308445	9.4	0.093	0.11	0.749
6	58771.0000	58479.79	1488.00	2.00	4.09	0.003843	0.032	9.5	0.018	0.029	0.118	0.0243904	7.4	0.073	0.15	0.421
5	45410.0000	45157	1436.80	3.42	5.34	0.003496	0.035	9.5	0.018	0.024	0.168	0.0347398	10.6	0.104	0.09	0.997
4	32084.0000	31827	1390.20	2.98	4.71	0.002891	0.035	1.17	0.012	0.013	0.068	0.014086	4.3	0.042	0.03	1.190
3	18366.0000	18303	1351.10	2.73	4.38	0.002752	0.035	0.7	0.011	0.011	0.051	0.0105626	3.2	0.032	0.00	inf
2	13159.0000	13070	1336.70	2.71	5.61	0.002608	0.025	0.7	0.011	0.011	0.084	0.0173396	5.3	0.052	0.00	inf
1	7666.0000	7510	1322.20	3.10	4.68	0.002290	0.035	0.5	0.011	0.009	0.050	0.0103325	3.1	0.031	0.02	1.928
			1305.00													

** Inf - Reach will not armor

Preferred Alternative - 100-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Slope	Manning's "n"	D90 (mm)	Shields Armoring Size: Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)	Ya (ft)	Ratio of bed material > Dc	Yd (ft)
7	72630.0000	72324.7	1557.35	2.69	5.36	0.005009	0.032	9.5	0.018	0.026	0.183	0.0379169	11.6	0.114	0.07	1.553
6	58771.0000	58479.79	1488.00	2.58	5.10	0.003843	0.032	9.5	0.018	0.027	0.168	0.0348312	10.6	0.104	0.09	1.104
5	45410.0000	45157	1436.80	4.25	6.16	0.003496	0.034	9.5	0.018	0.023	0.207	0.0429984	13.1	0.129	0.06	1.903
4	32084.0000	31827	1390.20	3.87	5.57	0.002891	0.033	1.17	0.012	0.012	0.087	0.018046	5.5	0.054	0.03	1.709
3	18366.0000	18303	1351.10	3.39	5.08	0.002752	0.034	0.7	0.011	0.010	0.064	0.0132192	4.0	0.040	0.00	inf
2	13159.0000	13070	1336.70	3.45	6.61	0.002608	0.026	0.7	0.011	0.010	0.107	0.0222506	6.8	0.067	0.00	inf
1	7666.0000	7510	1322.20	3.99	5.50	0.002290	0.034	0.5	0.011	0.009	0.063	0.0131246	4.0	0.039	0.01	3.055
			1305.00													

** Inf - Reach will not armor

Table 5-3. Summary of Armoring Analysis for 5- and 100-Year Flow at Sanokai Wash Main Branch

Preferred Alternative - 5-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Manning's "n"	D90 (mm)	Shields Armoring Size					Ya (ft)	Ratio of bed material > Dc	Yd (ft)
									Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)			
6	6142	37074.69	1432.60	1.78	4.71	0.008027	0.042	0.4	0.010	0.010	0.056	0.0116887	3.6	0.035	0.000	inf
5	30900	31012.55	1383.94	2.84	2.69	0.002469	0.042	0.9	0.012	0.012	0.021	0.0042781	1.3	0.013	0.064	0.188
4	27837	27642.55	1375.62	2.16	2.96	0.002114	0.042	0.25	0.010	0.008	0.018	0.0037005	1.1	0.011	0.001	12.349
3	20167	19930.98	1359.32	2.27	3.48	0.002573	0.035	0.19	0.009	0.008	0.022	0.0045779	1.4	0.014	0.000	inf
2	11120	10884.6	1336.04	1.94	3.30	0.003307	0.042	0.19	0.009	0.008	0.021	0.0043504	1.3	0.013	0.000	inf
1	2838	2603	1308.65	2.16	3.33	0.002213	0.035	0.19	0.009	0.008	0.021	0.004274	1.3	0.013	0.000	inf
			1302.89													

** Inf - Reach will not armor

Preferred Alternative - 100-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Manning's "n"	D90 (mm)	Shields Armoring Size					Ya (ft)	Ratio of bed material > Dc	Yd (ft)
									Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)			
6	6142	37074.69	1432.60	2.65	6.41	0.008027	0.042	0.4	0.010	0.009	0.091	0.0189598	5.8	0.057	0.000	inf
5	30900	31012.55	1383.94	5.54	4.88	0.002469	0.042	0.9	0.012	0.009	0.054	0.0112384	3.4	0.034	0.025	1.325
4	27837	27642.55	1375.62	4.20	4.56	0.002114	0.042	0.25	0.010	0.007	0.034	0.007039	2.1	0.021	0.000	inf
3	20167	19930.98	1359.32	3.82	5.55	0.002573	0.035	0.19	0.009	0.006	0.047	0.0098217	3.0	0.029	0.000	inf
2	11120	10884.6	1336.04	3.31	4.69	0.003307	0.042	0.19	0.009	0.007	0.035	0.0073536	2.2	0.022	0.000	inf
1	2838	2603	1308.65	4.11	5.11	0.002213	0.035	0.19	0.009	0.006	0.039	0.0081219	2.5	0.024	0.000	inf
			1302.89													

** Inf - Reach will not armor

Table 5-4. Summary of Armoring Analysis for 5- and 100-Year Flow at Sanokai Wash East Branch

Preferred Alternative - 5-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Manning's "n"	D90 (mm)	Shields Armoring Size					Ya (ft)	Ratio of bed material > Dc	Yd (ft)
									Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)			
4E	48164	17420.2	1444.13	1.93	3.25	0.003500	0.042	2.2	0.014	0.018	0.046	0.0095624	2.9	0.029	0.066	0.405
3E	44615	13831.8	1431.57	2.09	3.42	0.003500	0.042	0.075	0.008	0.006	0.016	0.0033435	1.0	0.010	0.003	3.606
2E	39100	8301	1412.21	2.09	3.42	0.003482	0.042	0.2	0.009	0.008	0.022	0.0046365	1.4	0.014	0.007	1.930
1E	33500	2700	1392.71	2.12	3.33	0.002648	0.042	0.2	0.009	0.008	0.021	0.0043749	1.3	0.013	0.008	1.602
			1385.56													

** Inf - Reach will not armor

Preferred Alternative - 100-yr Flow

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Upstream Elevation ft	Hydraulic Radius (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Manning's "n"	D90 (mm)	Shields Armoring Size					Ya (ft)	Ratio of bed material > Dc	Yd (ft)
									Grain Roughness n	Friction Factor f	Boundary Shear t_p (lb/ft^2)	Critical Diameter D_c (ft)	Critical Diameter D_c (mm)			
4E	48164	17420.2	1444.13	3.82	5.11	0.003500	0.042	2.2	0.014	0.014	0.091	0.0187995	5.7	0.056	0.016	3.497
3E	44615	13831.8	1431.57	3.51	4.83	0.003500	0.042	0.075	0.008	0.005	0.027	0.0056104	1.7	0.017	0.000	inf
2E	39100	8301	1412.21	3.51	4.83	0.003482	0.042	0.2	0.009	0.007	0.038	0.00778	2.4	0.023	0.000	inf
1E	33500	2700	1392.71	3.56	4.71	0.002648	0.042	0.2	0.009	0.007	0.036	0.0073634	2.2	0.022	0.000	inf
			1385.56													

** Inf - Reach will not armor

5.2.2 Sedimentation Transport Capacity

5.2.2.1 Theory

The placement of drop structures was determined using the procedure outlined in Simons, Li & Associates, Inc. (1985). The bed material discharge (q_s), for the dominant discharge (5-year) at various Reaches at Queen Creek and Sanokai Wash was estimated using the following equation:

$$q_s = aY_h^b V^c \qquad \text{Eq. 1}$$

where a, b and c are regression coefficients from tables, Y_h is the hydraulic depth in the reach and V is the average channel velocity. The hydraulic characteristics for each reach were obtained from the RAS hydraulic model of the system. The equilibrium slope (S_{eq}) for each reach was then calculated using the following equation:

$$S_{eq} = \left(\frac{a}{q_s}\right)^{\frac{10}{3(b-c)}} q^{\frac{2(2c+3b)}{3(b-c)}} \left(\frac{n}{1.49}\right)^2 \qquad \text{Eq. 2}$$

where a, b, c and q_s are from Eq. 1, q is the unit discharge, n is the Manning's n and S is the equilibrium slope in feet/feet. The elevation difference between existing slope (S_0) and equilibrium slope (S_{eq}) is calculated by:

$$\text{Elevation difference} = L(S_{eq} - S_0) \qquad \text{Eq. 3}$$

Sediment continuity analysis was done for both 5-year and 100-year discharge using equation (5.13):

$$Q_{s_{in}} - Q_{s_{out}} = \frac{dVol}{dt} \qquad \text{Eq. 4}$$

The unbulked sediment volumes (V_s) computed by Eq. 1 is converted to actual volume by equation (5.13)

$$V_t = \frac{V_s}{1-n} \qquad \text{Eq. 5}$$

where V_t is the bulked sediment volume and n is the porosity ($n = 4$).

For more detailed discussion for the theory the reader is directed to the above referenced document.

5.2.2.2 Sediment Transport Capacity for Queen Creek

Queen Creek was divided into 7 analysis reaches with similar hydraulic characteristics. The boundaries of the reaches are shown in Table 5-5 and Figure 4-1.

Table 5-5: Reaches for Queen Creek.

Reach	*Downstream Location*	*Upstream Location*	*Downstream cross section (from RAS)*	*Upstream cross section (from RAS)*
7	Kenworthy Road	CAP Canal	59,007	72,630
6	Rittenhouse Road	Kenworthy Road	45,490	59,007
5	Ellsworth Road	Rittenhouse Road	32,235	45,410
4	Sossaman Road	Ellsworth Road	18,432	32,084
3	Power Road	Sossaman Road	13,226	18,366
2	Recker Road	Power Road	7,786	13,159
1	EMF	Recker Road	166	7,666

For each of these reaches, the sediment transport capacity (q_s) was calculated using Eq. 1 and the hydraulic characteristics from the HEC-RAS model. The results are shown in Table 5-6.

Table 5-6: Bed material discharge for Queen Creek (5-year flow).

Total bed-material discharge in sand bed channel - 5-year									
Variable	Units	Source	Reach						
			1	2	3	4	5	6	7
Y_h	ft	HEC-RAS	3.105	2.74	2.73	3	3.45	2.015	2.41
V	ft/s	HEC-RAS	4.52	5.96	4.43	4.79	5.4	4.35	4.86
S_o	ft/ft	HEC-RAS	0.00228	0.00257	0.00280	0.00289	0.00354	0.00387	0.00524
D_{50}	mm	Sampling	0.33		0.29	0.275	0.7	1.25	
$D_{84.1}$	mm	Sampling	0.54		0.54	0.55	3.525	5.5	
$D_{15.9}$	mm	Sampling	0.19		0.19	0.15	0.255	0.41	
G			1.69	1.69	1.69	1.92	3.89	3.72	3.72
a		Table 5.6a	0.000068	0.000068	0.000068	0.000068	0.000043	0.000043	0.000043
b		Table 5.6a	0.315	0.315	0.315	0.315	0.324	0.324	0.324
c		Table 5.6a	3.83	3.83	3.83	3.83	3.7	3.7	3.7
q_s	cfs/ft		0.0314	0.0870	0.0279	0.0388	0.0330	0.0125	0.0199

Equilibrium slope was calculated based on the sediment transport capacities from Table 5-6 using Eq. 2. Upstream of Reach 7, the CAP channel crossing and the gravel mining will limit the supply to the downstream reaches. Therefore, sediment supply for Queen Creek was assumed to be equal to the capacity of Reach 7. The equilibrium slope calculations are shown in Table 5-7 and the resulting elevation differences due to the equilibrium slope are estimated in Table 5-8, using Eq. 3. As is seen in Table 5-8, Reach 6 will aggrade and other reaches will degrade. The armor calculations (Table 5-2) show that Reaches 5, 4 and 1 will armor. No grade control structures are needed for these reaches. It is recommended that a grade control structure with a drop of 3.6 feet be placed at Sossaman Road for Reach 3. For Reach 2 it is recommended that a grade control structure with a drop of 4.5 feet be placed at the multi-arch bridge and a grade control structure with a drop of 5 feet be placed at the Power Road.

Table 5-7: Equilibrium slope calculations for Queen Creek.

Equilibrium Slope									
Variable	Units	Source	Reach						
			1	2	3	4	5	6	7
Q	cfs	HEC-RAS	1974	1974	1978	1986	1896	1695	1695
n		HEC-RAS	0.035	0.025	0.035	0.035	0.035	0.032	0.032
P (RAS)	ft	HEC-RAS	137.88	87.7	165.5	128.14	105.97	215.19	170.42
A (RAS)	ft^2	HEC-RAS	423.22	384.745	451.28	421.615	354.96	414.475	357.42
B	ft		136.30	140.42	165.30	140.54	102.89	205.69	148.31
q	cfs/ft		14.5	14.1	12.0	14.1	18.4	8.2	11.4
qs (In)	cfs/ft	HEC-RAS	0.0199	0.0199	0.0199	0.0199	0.0199	0.0199	
Width over depth			44	51	61	47	30	102	62
So Equilibrium		Eq. 5.11	0.001535	0.000822	0.002096	0.001597	0.001907	0.006030	
So Existent		HEC-RAS	0.002284	0.002574	0.002800	0.002891	0.003536	0.003866	0.005243

Table 5-8: Elevation differences for Queen Creek.

Summary								
Reach	Existing Slope	Equilibrium Slope	Ratio	Equilibrium Slope is	Length ft	Existing Slope Elevation	Equilibrium Slope Elevation	Difference (ft)
6	0.003866	0.006030	1.56	Steeper	13,243	51.2	79.9	28.7
5	0.003536	0.001907	0.54	Shallower	13,180	46.6	25.1	-21.5
4	0.002891	0.001597	0.55	Shallower	13,464	38.9	21.5	-17.4
3	0.002800	0.002096	0.75	Shallower	5,143	14.4	10.8	-3.6
2	0.002574	0.000822	0.32	Shallower	5,440	14.0	4.5	-9.5
1	0.002284	0.001535	0.67	Shallower	7,290	16.7	11.2	-5.5

Sediment continuity analysis was performed to evaluate degradation as an input into the local scour evaluation for both the 5-year and the 100-year flow. Based on the hydrologic analysis, a 1-day hydrograph was chosen (Figure 3-3). Table 5-9 calculates the sediment transport capacity for the reaches for the 100-year discharge in the same way as Table 5-6 did for the 5-year discharge.

Table 5-9: Sediment transport capacity for Queen Creek based on the 100-year discharge.

Total bed-material discharge in sand bed channel - 100-year									
Variable	Units	Source	Reach						
			1	2	3	4	5	6	7
Y_h	ft	HEC-RAS	4.00	3.49	3.40	3.91	4.31	2.60	2.71
V	ft/s	HEC-RAS	5.29	6.71	5.08	5.57	6.16	5.10	5.36
S_o	ft/ft	HEC-RAS	0.002284	0.002574	0.002800	0.002891	0.003536	0.003866	0.005243
D_{50}	mm	Sampling	0.33		0.29	0.275	0.7	1.25	
$D_{84.1}$	mm	Sampling	0.54		0.54	0.55	3.525	5.5	
$D_{15.9}$	mm	Sampling	0.19		0.19	0.15	0.255	0.41	
G			1.69	1.69	1.69	1.92	3.89	3.72	3.72
a		Table 5.6a	0.000068	0.000068	0.000068	0.000068	0.0000431	0.0000431	0.0000431
b		Table 5.6a	0.315	0.315	0.315	0.315	0.324	0.324	0.324
c		Table 5.6a	3.83	3.83	3.83	3.83	3.7	3.7	3.7
q_s	cfs/ft		0.0619	0.1478	0.0505	0.0752	0.0578	0.0243	0.0297

Using Table 5-6 and Table 5-9 a sediment continuity analysis is presented in Table 5-10 based on Eq. 4. The resulting depths were used in the local scour depth analysis. As is seen in the table, Reach 2 has the greatest degradation potential according to the continuity analysis.

Table 5-10: Continuity Analysis for Queen Creek.

Sediment Budget 5-year flow									
Reach	qs	B	Qs	dQs		n	Vt	Length	Depth
	cfs/ft	ft	cfs	cfs			cfs	ft	ft
7	0.0199	148.3	3.0						
6	0.0125	205.7	2.6	0.4	Depos.	0.4	0.6	13,243	0.0
5	0.0330	102.9	3.4	-0.8	Scour	0.4	-1.4	13,180	-0.1
4	0.0388	140.5	5.4	-2.1	Scour	0.4	-3.4	13,464	-0.2
3	0.0279	165.3	4.6	0.8	Depos.	0.4	1.4	5,143	0.1
2	0.0870	140.4	12.2	-7.6	Scour	0.4	-12.7	5,440	-1.4
1	0.0314	136.3	4.3	7.9	Depos.	0.4	13.2	7,290	1.2
Sediment Budget 100-year flow									
Reach	qs	B	Qs	dQs		n	Vt	Length	Depth
	cfs/ft	ft	cfs	cfs			cfs	ft	ft
7	0.0297	192.5	5.7						
6	0.0243	215.7	5.2	0.5	Depos.	0.4	0.8	13,243	0.0
5	0.0578	110.5	6.4	-1.1	Scour	0.4	-1.9	13,180	-0.1
4	0.0752	151.0	11.4	-5.0	Scour	0.4	-8.3	13,464	-0.4
3	0.0505	192.0	9.7	1.7	Depos.	0.4	2.8	5,143	0.2
2	0.1478	174.2	25.7	-16.1	Scour	0.4	-26.8	5,440	-2.4
1	0.0619	153.0	9.5	16.3	Depos.	0.4	27.1	7,290	2.1

It is recommended that a sedimentation basin be placed at the confluence with EMF on Queen Creek. This is necessary as the slope of EMF is much flatter than the slope on Queen Creek and therefore it is necessary to remove material from the flow so it will not deposit in the EMF, downstream of the confluence. The size of the basin is found by using the capacity of the downstream reach of Queen Creek (Reach 1) for the 100-year discharge. As is seen in Table 5-9, the transport capacity is 0.0619 cfs/feet. Using a 1-day storm, it is estimated that a sedimentation basin be constructed to hold approximately 45,000 yd^3 of material. Table 5-11 and Figure 5-9 summarizes the location of drop structures for Queen Creek.

Table 5-11: Summary for Queen Creek.

Reach	*Armor Depth*	*Degradation/ Aggradation*	*Sedimentation Basin*	*Comment*
7	N/A	N/A	No	Supply reach
6	None	Aggradation	No	
5	1.0 feet	Degradation	No	
4	1.2 feet	Degradation	No	
3	None	Degradation	No	1. At Sossaman Road with 3.6 feet drop
2	None	Degradation	No	1. At multi-arch bridge with 4.5 feet drop 2. At Power Road with 5 feet drop
1	2.0 feet	Degradation	Yes	Sedimentation at the confluence with EMF

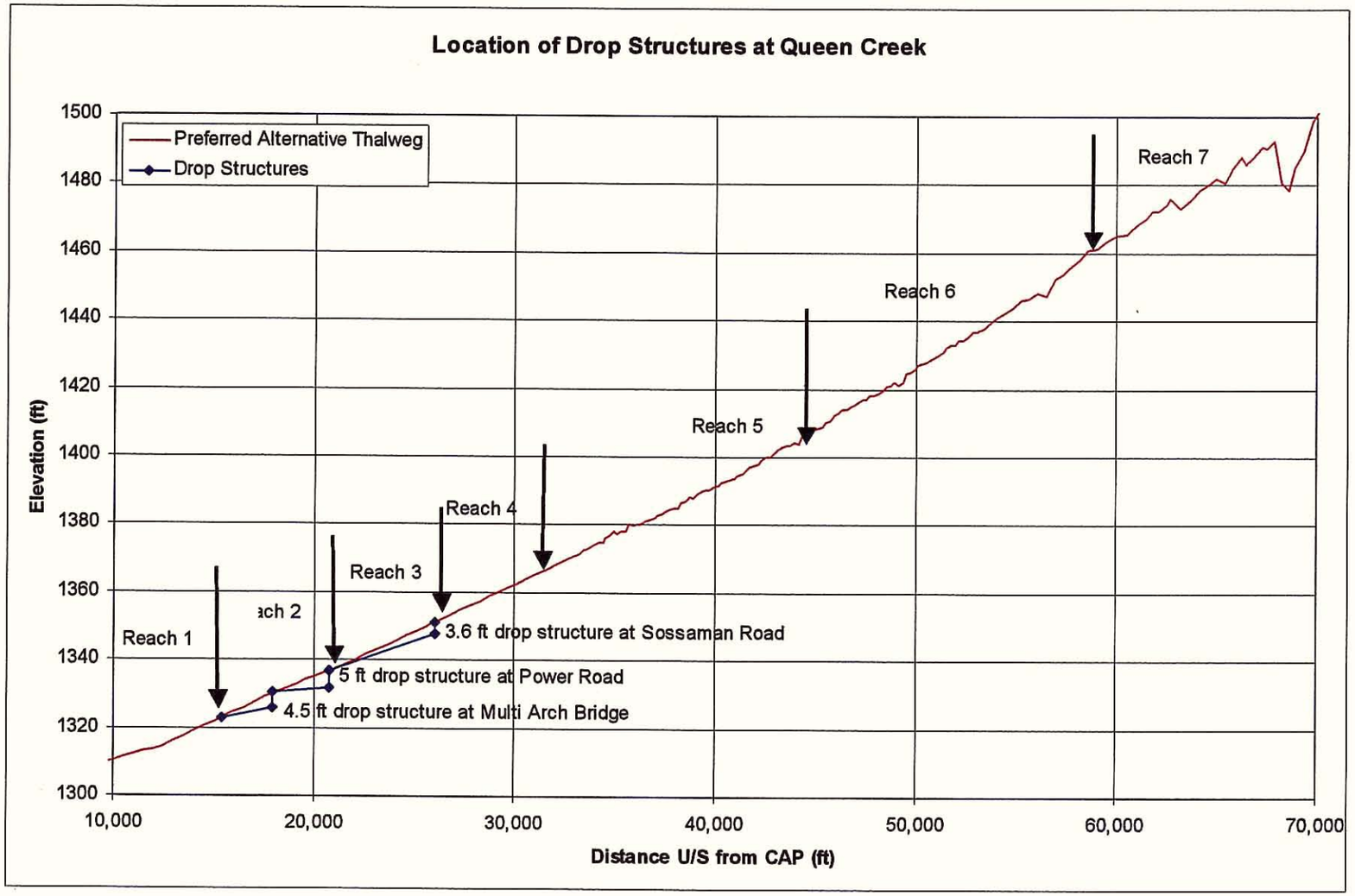

Figure 5-9. Location of Drop Structures in Queen Creek

5.2.2.3 Sediment Transport Capacity for Sanokai Wash Main and East Branch

The Sanokai Wash Main branch was divided into 5 Reaches of similar hydraulic characteristics. The East Branch of Sanokai Wash was divided into 4 Reaches (4E upstream to 1E at the confluence with the Main Branch) based on similarity in hydraulic characteristics. The Main Branch, upstream of the confluence with the East Branch, is one reach. The boundaries of the reaches are shown in Table 5-12.

Table 5-12: Reaches for Sanokai Wash

Reach		*Downstream Location*	*Upstream Location*	*Downstream Cross Section (from RAS)*	*Upstream Cross Section (from RAS)*
S of Riggs	6	Riggs Road	County Line	29,692	6,142
Main Stem	5	Hawes Road	Riggs Road	28,000	29,692
Main Stem	4	Sossaman Road	Hawes Road	20,242	28,000
Main Stem	3	Bend d/s of power road	Sossaman Road	11,602	20,242
Main Stem	2	Higley Road	d/s of Power Road	2,956	11,602
Main Stem	1	New SW Confluence	Higley Road	251	2,784
East Branch	4E	Signal Butte	County Line	45,015	48,164
East Branch	3E	Crismon Road	Signal Butte	39,474	45,015
East Branch	2E	Ellsworth Road	Crismon Road	33,900	39,474
East Branch	1E	Riggs Road	Ellsworth Road	31,400	33,900

For each of these Reaches, the sediment transport capacity (q_s) was calculated using Eq. 1 and the hydraulic characteristics from the HEC-RAS model. The results are shown in Table 5-13.

Table 5-13: Sediment transport capacity for Sanokai Wash for the 5-year discharge.

Total bed-material discharge in sand bed channel - 5-Year												
Variable	Units	Source	Reach - East Branch				Reach - Main Branch					
			4E	3E	2E	1E	6	5	4	3	2	1
T	ft	HEC-RAS	122.44	128.4	128.43	129.81	92.45	140.035	164.515	111.73	148.125	128.71
Y_h	ft	HEC-RAS	1.94	2.09	2.09	2.13	1.79	2.855	2.17	2.28	1.95	2.17
V	ft/s	HEC-RAS	3.23	3.42	3.42	3.23	4.82	3.19	2.92	3.56	3.24	3.63
S_o	ft/ft	HEC-RAS	0.00350	0.00350	0.00350	0.00325	0.00836	0.00247	0.00209	0.00252	0.00331	0.00221
D_{50}	mm	Sampling										
$D_{84.1}$	mm	Sampling										
$D_{15.9}$	mm	Sampling										
G			2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
a		Table 5.6a	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068
b		Table 5.6a	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315
c		Table 5.6a	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83
q_s	cfs/ft		0.0075	0.0095	0.0095	0.0077	0.0338	0.0081	0.0053	0.0114	0.0076	0.0121

Equilibrium slope was calculated based on the sediment transport capacities from Table 5-13 using Eq. 2. Sanokai Wash Main Branch, upstream of the confluence with the East Branch (Reach 6) has very high transport capacity compared to every other reach in the system. This is due to the steep slope as well as the relatively narrow channel. The culverts passing under Ellsworth Road at the upstream end of the reach may restrict the sediment supply to the reach. However, it will be assumed that the sediment supply into Reach 5 is the summation of the East Branch and Reach 6 capacities but in reality the input from Reach 6 is probably lower due to the

upstream constriction. Therefore it is recommended that the channel in Reach 6 should be lined to prevent erosion. Reach 5, which is the short span downstream of the confluence to Hawes Road, will at its downstream end experience deposition due to its low gradient between the landfill and the BMX track. It is recommended that the channel be excavated following flow events to maintain its capacity. This reach will therefore act as a natural sedimentation basin. The input to the downstream reaches was assumed to be the capacity of Reach 5. This leads to deposition in Reach 4 and Reach 2 and degradation in Reach 3 and slight degradation in Reach 1. It is recommended that two grade-control structures be placed in Reach 3, one with a drop of 3.5 feet at Sossaman Road and another at the midpoint between Power Road and Sossaman Road with a drop of 3.5 feet.

For the East Branch, the sediment capacity of the upstream most Reach (4E) was used as input for the downstream reaches. The equilibrium slope calculations are shown in Table 5-14 and the resulting elevation differences due to the equilibrium slope are shown in Table 5-15. For the East Branch, the three reaches will degrade. The armor calculations (Table 5-4) show that for Sanokai Wash East Branch Reaches 1E through 3E will armor for the 5-year flow. The depth of degradation for Reach 3E is 3.6 feet, which is close to the 3.8 feet drop estimated from the equilibrium slope calculations (Table 5-14). Therefore, it is recommended that a grade-control structure with a drop of 3.8 feet be placed at Signal Butte Road.

Table 5-14: Equilibrium slope calculations for Sanokai Wash.

Equilibrium Slope												
Variable	Units	Source	Reach - East Branch				Reach - Main Branch					
			4E	3E	2E	1E	6	5	4	3	2	1
Q	cfs	HEC-RAS	771	918	918	918	777	1073	1052	967	942	929
n		HEC-RAS	0.042	0.042	0.042	0.042	0.042	0.042	0.042	0.035	0.042	0.035
P (RAS)	ft	HEC-RAS	122.7	128.6	128.7	130.1	92.7	140.8	164.7	112.4	148.3	129.3
A (RAS)	ft^2	HEC-RAS	237.3	268.5	268.6	276.1	165.0	399.3	355.5	273.9	288.3	279.1
B	ft		122.3	128.5	128.5	129.6	92.2	139.9	163.8	120.1	147.9	128.6
q	cfs/ft		6.3	7.1	7.1	7.1	8.4	7.7	6.4	8.0	6.4	7.2
qs (In)	cfs/ft	HEC-RAS		0.0075	0.0075	0.0075		0.0413	0.0081	0.0081	0.0081	0.0081
Width over depth			63	61	61	61	52	49	75	53	76	59
So Equilibrium		Eq. 5.11		0.00276	0.00276	0.00280		0.01245	0.00354	0.00170	0.00358	0.00203
So Existent		HEC-RAS		0.00350	0.00350	0.00325	0.00836	0.00247	0.00209	0.00252	0.00331	0.00221

Table 5-15: Elevation differences for Sanokai Wash.

Summary								
Reach	Existing Slope	Equilibrium Slope	Ratio	Equilibrium Slope is	Length ft	Existing Slope Elevation	Equilibrium Slope Elevation	Difference (ft)
5	0.00247	0.01245	5.05	Steeper	3,206	7.9	39.9	32.0
4	0.00209	0.00354	1.69	Steeper	7,634	16.0	27.0	11.0
3	0.00252	0.00170	0.68	Shallower	8,565	21.6	14.5	-7.0
2	0.00331	0.00358	1.08	Steeper	8,166	27.1	29.3	2.2
1	0.00221	0.00203	0.92	Shallower	2,403	5.3	4.9	-0.4
3E	0.00350	0.00276	0.79	Shallower	5,141	18.0	14.2	-3.8
2E	0.00350	0.00276	0.79	Shallower	5,201	18.2	14.4	-3.8
1E	0.00325	0.00280	0.86	Shallower	2,200	7.2	6.2	-1.0

Sediment continuity analysis was performed to evaluate degradation as an input into the local scour evaluation for both the 5-year and the 100-year flow. It was assumed that the hydrograph was 1-day long (Figure 3-4). Table 5-16 calculates the sediment transport capacity for the reaches for the 100-year discharge in the same way as Table 5-13 did for the 5-year discharge.

Table 5-16: Sediment transport capacity for Sanokai Wash for the 100-year discharge.

Total bed-material discharge in sand bed channel - 100-Year												
Variable	Units	Source	Reach - East Branch				Reach - Main Branch					
			4E	3E	2E	1E	6	5	4	3	2	1
T	ft	HEC-RAS	201.6	188.1	188.1	190.4	132.4	168.1	240.0	130.025	199.52	148.16
Y_h	ft	HEC-RAS	3.83	3.52	3.52	3.57	2.66	5.59	4.21	3.86	3.32	4.15
V	ft/s	HEC-RAS	5.08	4.90	4.83	4.55	6.29	5.24	4.64	5.13	4.61	5.54
S_o	ft/ft	HEC-RAS	0.00350	0.00350	0.00350	0.00325	0.00836	0.00247	0.00209	0.00252	0.00331	0.00221
D_{50}	mm	Sampling										
$D_{84.1}$	mm	Sampling										
$D_{15.9}$	mm	Sampling										
G			2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
a		Table 5.6a	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068	0.000068
b		Table 5.6a	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.315
c		Table 5.6a	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83	3.83
q_s	cfs/ft		0.0523	0.0445	0.0421	0.0337	0.1060	0.0664	0.0382	0.0546	0.0347	0.0747

Using Table 5-13 and Table 5-16 a sediment continuity analysis is presented in Table 5-17 based on Eq. 4. The resulting depths were used in the local scour depth analysis.

Table 5-17: Continuity Analysis for Sanokai Wash.

Sediment Budget 5-year flow									
Reach	qs	B	Qs	dQs		n	Vt	Length	Depth
	cfs/ft	ft	cfs	cfs			cfs	ft	
6	0.0338	92.2	3.1					5,672	
5	0.0081	139.9	1.1	3.0	Depos.	0.4	5.0	3,206	1.0
4	0.0053	163.8	0.9	0.3	Depos.	0.4	0.4	7,634	0.0
3	0.0114	120.1	1.4	-0.5	Scour	0.4	-0.8	8,565	-0.1
2	0.0076	147.9	1.1	0.2	Depos.	0.4	0.4	8,166	0.0
1	0.0121	128.6	1.6	-0.4	Scour	0.4	-0.7	2,403	-0.2
4E	0.0075	122.3	0.9					3,188	
3E	0.0095	128.5	1.2	-0.3	Scour	0.4	-0.5	5,141	-0.1
2E	0.0095	128.5	1.2	0.0	Depos.	0.4	0.0	5,201	0.0
1E	0.0077	129.6	1.0	0.2	Depos.	0.4	0.4	2,200	0.1
Sediment Budget 100-year flow									
Reach	qs	B	Qs	dQs		n	Vt	Length	Depth
	cfs/ft	ft	cfs	cfs			cfs	ft	
6	0.1060	132.5	14.0					5,672	
5	0.0664	168.1	11.2	9.3	Depos.	0.4	15.5	3,206	2.5
4	0.0382	235.9	9.0	2.1	Depos.	0.4	3.6	7,634	0.2
3	0.0546	143.0	7.8	1.2	Depos.	0.4	2.0	8,565	0.1
2	0.0347	199.0	6.9	0.9	Depos.	0.4	1.5	8,166	0.1
1	0.0747	148.4	11.1	-4.2	Scour	0.4	-7.0	2,403	-1.7
4E	0.0523	201.7	10.5					3,188	
3E	0.0445	188.0	8.4	2.2	Depos.	0.4	3.6	5,141	0.3
2E	0.0421	188.0	7.9	0.5	Depos.	0.4	0.8	5,201	0.1
1E	0.0337	190.5	6.4	1.5	Depos.	0.4	2.5	2,200	0.5

It is recommended that a sedimentation basin be placed at the confluence with the EMF on Sanokai Wash. This is necessary as the slope of EMF is much flatter than the slope at Sanokai Wash in Reach 6 and therefore it is necessary to remove material from the flow so it will not deposit in the EMF downstream of the confluence. The size of the basin is found by using the capacity of the downstream most reach of Sanokai Wash (Reach 2 as Reach 1 is too short) for the 100-year discharge. As is seen in Table 5-16, the transport capacity is 0.0347 cfs/feet. Using a 1- day storm, it is estimated that a sedimentation basin be constructed to hold approximately 26,000 yd^3 of material.

Consideration should be given to using the existing channel and existing sedimentation basin. In addition to the downstream sedimentation basin, Reach 5, just upstream of Hawes Road, will act as a natural sedimentation basin as was discussed above. That portion of the wash should be excavated following large events to maintain its capacity during next storm. The amount that will deposit will depend on the sediment input from the main branch upstream. It has large capacity but it is not clear how much is delivered to that reach (Reach 6). Table 5-18, Figure 5-10 and Figure 5-11 summarize the location of drop structures for Sanokai Wash.

Table 5-18: Summary for Sanokai Wash Main and East Branch.

Sanokai Wash Main Branch					
Reach	Armor Depth	Drop Structures	Degradation/ Aggradation	Sedimentation Basin	Comment
6	None	N/A	N/A	No	Supply reach for Reach 5 - Channel should be lined
5	0.2 feet	Not needed	Aggradation	Yes	"Natural" sedimentation basin will form in the channel at d/s end – see discussion
4	None	Not needed	Aggradation	Yes	Combine with detention basin
3	None	2 x 3.5 feet	Degradation	No	Locate at Sossaman Road and midway between Sossaman and Power Road
2	None	No	Aggradation	No	
1	None	No	Even	Yes	At the confluence with EMF
Sanokai Wash East Branch					
Reach	Armor Depth	Drop Structures	Degradation/ Aggradation	Sedimentation Basin	Comment
4E	0.4 feet	N/A	N/A	No	Supply reach for East Branch
3E	3.6 feet	3.8 feet	Degradation	No	Locate at Signal Butte Road
2E	1.9 feet	Not needed	Degradation	No	
1E	1.6 feet	Not needed	Degradation	No	

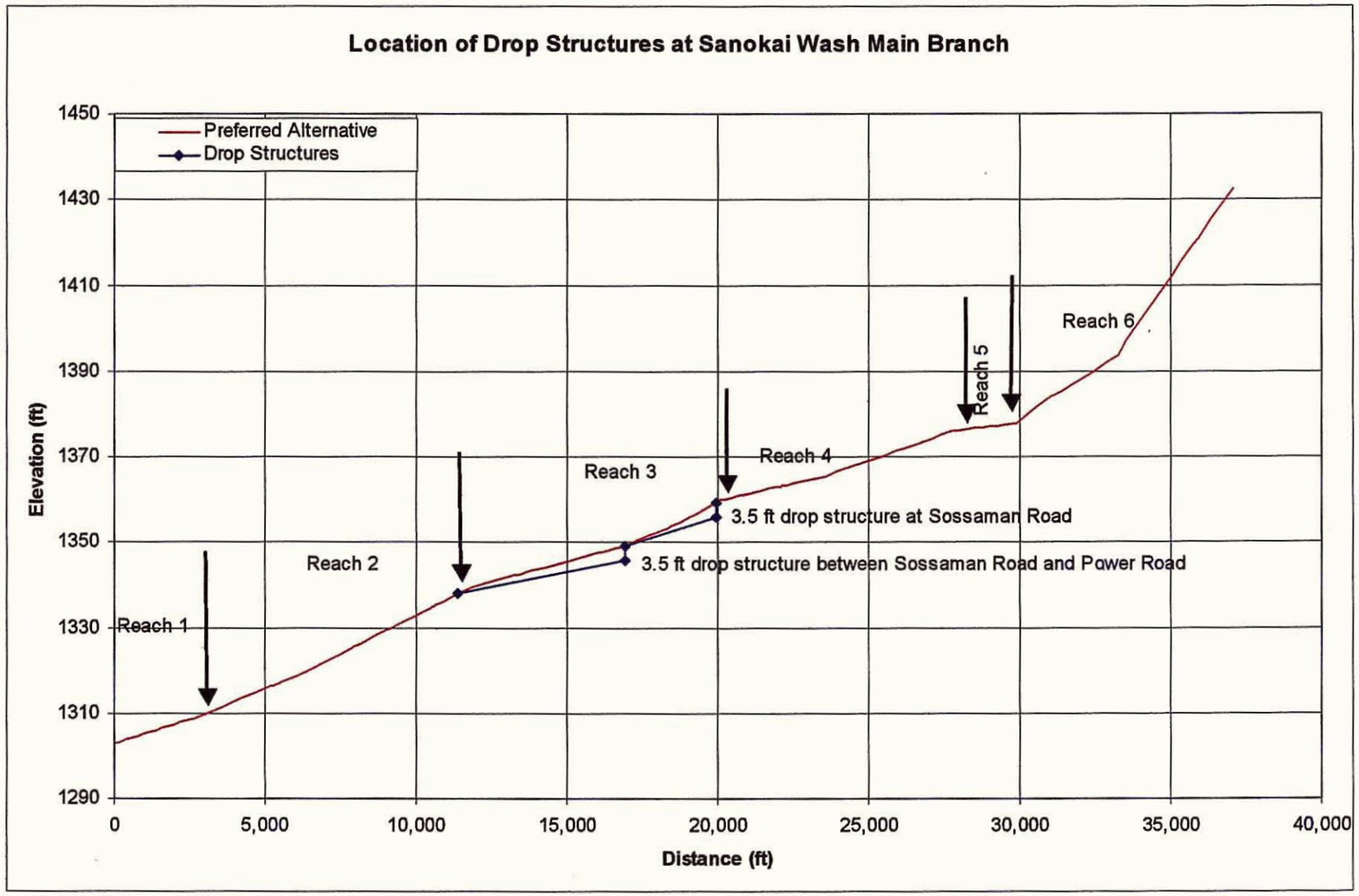

Figure 5-10. Location of Drop Structures in Sanokai Wash Main Branch

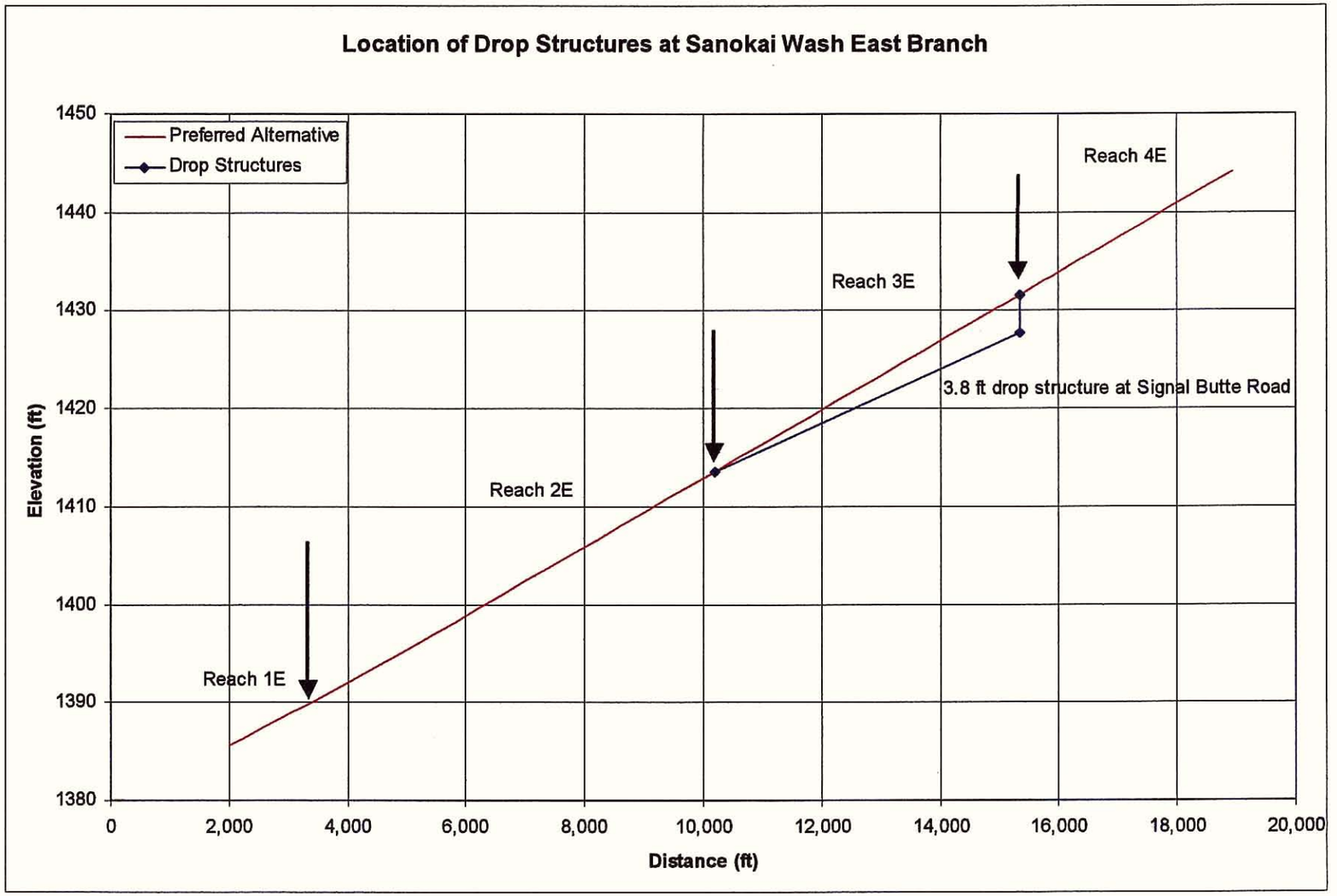

Figure 5-11. Location of Drop Structures in Sanokai Wash East Branch

5.2.3 Scour Potential

The total scour potential at Queen Creek and Sanokai Wash was estimated by calculating the local scour at various reaches and from the depth of degradation estimated using the sediment continuity analysis, in Section 5.2.2. The estimated total scour at various reaches of Queen Creek and Sanokai Wash are shown in Table 5-19 and Table 5-20.

The estimated total scour at Queen Creek ranged from 2.1 feet at Reach 7 to 15.6 feet at Reach 4. At Sanokai Wash Main Branch, the estimated total scour was 1.7 feet at Reach 1 and none was estimated at the East Branch of Sanokai Wash.

5.2.3.1 Pier Scour

Local scour is observed whenever an abrupt change in the direction of flow occurs, such as at bridge piers or embankments. The local scour at bridge piers is a result of the vortex systems developed at the pier. Local scour occurs when the capacity of the flow to remove or transport the bed material is greater than the rate at which the replacement material is supplied. The pier scour was determined using the Corps of Engineers River Analysis System standard-step backwater computer program (U.S. Army Corps of Engineers, 1997).

The estimated pier scour at various reaches of Queen Creek is shown in Table 5-19. The estimated pier scour at Queen Creek ranged from 2.1 feet at Schnepf Road to 11 feet at Power Road. As there are no bridges in Sanokai Wash Main and East Branches, pier scour was not estimated.

5.2.3.2 Contraction Scour

Contraction scour occurs when the flow area of a stream is reduced by a natural contraction or in the vicinity of the bridges that encroach on the flood plain. At a bridge crossing, many factors can contribute to the occurrence of the contraction scour. These factors may include: the main channel naturally contracts as it approaches the bridge opening; the road embankments at the approach to the bridges cause all or a portion of the overbank flow to be forced into the main channel; the bridge abutments are projecting into the main channel; the bridge piers are blocking a significant portion of the flow area; and a drop in the downstream tailwater which causes increased velocities and shear stress inside the bridge. More bed material is transported through the contracted section than is transported into the section. The contraction scour was determined using the Corps of Engineers River Analysis System standard-step backwater computer program (U.S. Army Corps of Engineers, 1997).

The estimated contraction scour at various reaches of Queen Creek is shown in Table 5-19. The estimated contraction scour at Queen Creek ranged from 0.27 feet at Hawes Road to 0.29 feet at Power Road. As there are no bridges in Sanokai Wash Main and East Branches, contraction scour was not estimated.

Table 5-19: Estimated Total Scour for 100-Year Flow at Queen Creek.

Reaches	Bridge/Culvert Location	Upstream Location	Pier Scour (feet)	Contraction Scour (feet)	Depths (Table 5-9) (feet)	Total Scour (feet)
7	Schnepf Road	CAP Canal	2.1	0.0	0.0	2.1
6	SPRR	Kenworthy Road	6.0	0.0	0.0	6.0
6	Rittenhouse Road	SPRR	3.9	0.0	0.0	3.9
5	Ellsworth Road	Rittenhouse Road	6.3	0.0	0.1	6.4
4	Ocotillo Road	Ellsworth Road	7.8	0.0	0.4	8.2
4	Hawes Road	Ocotillo Road	7.1	0.27	0.0	7.4
4	Sossaman Road	Hawes Road	0.0	0.0	0.0	0.0
3	Power Road	Sossaman Road	11.0	0.29	0.0	11.3
2	Recker Road	Power Road	0.0	0 .0	2.4	2.4
1	EMF	Recker Road	4.5	0.0	0.0	4.5

Table 5-20: Estimated Total Scour for 100-Year Flow at Sanokai Wash Main and East Branch.

Reaches	Downstream Location	Upstream Location	Pier Scour (feet)	Contraction Scour (feet)	Depths (Table 5-17) (feet)	Total Scour (feet)
6	Riggs Road	County Line	NA	NA	0.0	0.0
5	Hawes Road	Riggs Road	NA	NA	0.0	0.0
4	Sossaman Road	Hawes Road	NA	NA	0.0	0.0
3	Bend d/s of Power Road	Sossaman Road	NA	NA	0.0	0.0
2	Higley Road	d/s of Power Road	NA	NA	0.0	0.0
1	New SW Confluence	Higley Road	NA	NA	1.7	1.7
4E	Signal Butte	County Line	NA	NA	0.0	0.0
3E	Crismon Road	Signal Butte	NA	NA	0.0	0.0
2E	Ellsworth Road	Crismon Road	NA	NA	0.0	0.0
1E	Riggs Road	Ellsworth Road	NA	NA	0.0	0.0

5.2.4 Bend Scour

Bends associated with meandering channels will induce transverse or secondary currents which will scour sediment from the outside of a bend and cause it to be deposited along the inside of the bend. Downstream of the bend the velocity patterns will most likely cause lateral erosion of a channel bank. Bend sour at various reaches of Queen Creek and Sanokai Wash was determined using the procedure outlined in Simons, Li & Associates, Inc. (1985).

The estimated bend scour for Queen Creek ranged from 5.2 feet at Sossaman Road in Reach 3 to 16.4 feet at SPRR Bridge at Reach 6. Bend scour at Reaches 2 and 7 were not estimated, as they are straight reaches.

Table 5-21: Estimated Bend Scour for 100-Year Flow at Queen Creek.

River Reach	Bend Scour Location	Cumulative Channel Length (ft)	Maximum Depth Upstream ft	Hydraulic Depth (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Projection Angle (degrees)	Bend Scour (ft)
6	SPRR	58479.79	5.31	2.60	5.10	0.003841	5	16.4
5	Signal Butte Road	45157	5.37	4.31	6.16	0.003245	10	15.4
4	Hawes Road	31827	5.61	3.91	5.57	0.003063	15	6.1
3	Sossaman Road	18303	4.84	3.40	5.08	0.002895	15	5.2
1	Higley Road	7510	4.90	3.90	5.50	0.002911	5	14.9

The estimated bend scour for Sanokai Wash Main Branch ranged from 4.8 feet at downstream of Riggs Road in Reach 6 to 11.8 feet at Hawes Road in Reach 5. Bend scour within Reaches 1 and 2 were not estimated, as they are straight reaches.

Table 5-22: Estimated Bend Scour for 100-Year Flow at Sanokai Wash Main Branch.

River Reach	Bend Scour Location	Cumulative Channel Length (ft)	Maximum Depth Upstream ft	Hydraulic Depth (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Projection Angle (degrees)	Bend Scour (ft)
6	Riggs Road	37074.7	4.10	2.66	6.41	0.008027	45	4.8
5	Hawes Road	31012.6	6.10	5.59	4.88	0.002469	25	11.8
4	Sossaman Road	27642.6	5.20	4.21	4.56	0.002114	15	5.2
3	Power Road	19930.9	4.50	3.86	5.55	0.002573	25	11.1

The estimated bend scour for Sanokai Wash East Branch ranged from 7.5 feet at Crismon Road in Reach 3E to 9 feet at Riggs Road in Reach 1E. Bend scour at Reach 2E was not estimated, as this is a straight reach.

Table 5-23: Estimated Bend Scour for 100-Year Flow at Sanokai Wash East Branch.

River Reach	Upstream Cross Section number	Cumulative Channel Length (ft)	Maximum Depth Upstream ft	Hydraulic Depth (ft)	Mean Channel Velocity (ft/s)	Energy Slope	Projection Angle (degrees)	Bend Scour (ft)
4E	U/S of Signal Butte	17420.2	3.90	3.83	5.11	0.003500	25	8.2
3E	Crismon Road	13831.8	3.60	3.52	4.83	0.003500	25	7.5
1E	Riggs Road	2700	4.10	3.57	4.71	0.002648	25	9.0

5.2.5 Summary

The results of the armoring analysis for Queen Creek show that an armor layer will develop for all of the reaches except 2 and 3 for both the 5- and 100-year flow events. Reaches 2 and 3 will not armor because of the lack of sufficiently sized material to form an armor layer. Sanokai Wash Main Branch is seen to armor only in Reach 5 for both the 5- and 100-year flow events. All other reaches lack sufficiently sized material to form an armor layer. Sanokai Wash East Branch is seen to armor in all reaches for the 5-year flow event, but only in Reach 4E for the

100-year flow. Reaches 1E through 3E will not armor as there is not sufficiently sized material to form an armor layer.

The sediment transport capacity was estimated for each of the reaches defined for Queen Creek and Sanokai Wash using the hydraulic characteristics developed from the hydraulic model. The equilibrium slope was calculated based on the sediment transport capacities for each reach. These were used to determine the size and location of grade control structures.

Upstream of Queen Creek Reach 7, the CAP channel crossing and gravel mining will limit the supply to the downstream reaches. Therefore, sediment supply for Queen Creek was assumed to be equal to the capacity of Reach 7. From the equilibrium slope calculations for Queen Creek it is estimated that Reach 6 will aggrade but the other reaches will degrade. The armoring analysis showed that Reaches 5, 4 and 1 will armor. Therefore, no grade control structures are needed for these reaches. It is recommended that a grade-control structure with a drop of 3.6 feet be placed at Sossaman Road in Reach 3. It is further recommended that a grade-control structure with a drop of 4.5 feet be placed at the multi-arch bridge and another grade-control structure with a drop of 5 feet be placed at Power Road, both located in Reach 2. It should be noted that the original hydraulic model developed by Coe and Van Loo for Power Ranch (Meadow Brook Village) located in Reach 2 may not represent the actual channel geometry as channel modifications were made after the study was conducted. Additional analysis should be conducted during the design phase to determine the requirements for these drop structures.

The Sanokai Wash Main Branch upstream of the confluence with the East Branch (Reach 6) has very high transport capacity compared to every other reach in the system. This is due to the steep slope as well as the relatively narrow channel. The culverts passing under Ellsworth Road at the upstream end of the reach may restrict the sediment supply to the reach. However, it was assumed that the sediment supply into Reach 5 is the total of the East Branch and Reach 6 capacities. The input from Reach 6 is likely lower due to the upstream constriction. Therefore it is recommended that the channel in Reach 6 should be lined to prevent erosion. Reach 5, which is the short span downstream of the confluence to Hawes Road, will at its downstream end experience deposition due to its low gradient between the landfill and the BMX track. This reach will therefore act as a natural sedimentation basin. It is recommended that the channel be excavated following flow events to maintain its capacity. The input to the downstream reaches was assumed to be the capacity of Reach 5. Reaches 4 and 2 were shown to aggrade. Reach 3 was shown to degrade while Reach 1 was shown to have minimal degradation. It is recommended that two grade-control structures be placed in Reach 3, one with a drop of 3.5 feet at Sossaman Road and another at the midpoint between Power Road and Sossaman Road with a drop of 3.5 feet.

For the East Branch, the sediment capacity of the upstream most Reach (4E) was used as input for the downstream reaches. The East Branch shows that the three reaches will degrade. The armor calculations (Table 5-4) show that for Sanokai Wash East Branch Reaches 1E through 3E will armor but for Reach 3E it is recommended that a grade-control structure with a drop of 3.8 feet should be place at Signal Butte Road as the armoring depth is 3.6 feet.

It is recommended that a sedimentation basin be placed at the confluence with EMF on Queen Creek. The slope of the EMF is much flatter than the slope in Queen Creek Reach 1. Therefore, it is necessary to remove sediment from the flow so it will not deposit in the EMF downstream of the confluence. The size of the basin is found by using the capacity of the downstream most reach of Queen Creek (Reach 1) for the 100-year discharge event. It is estimated that a sedimentation basin be constructed to hold approximately 45,000 yd^3 of material.

It is also recommended that a sedimentation basin be placed at the confluence with EMF on Sanokai Wash. Similar to Queen Creek confluence, the slope of EMF is much flatter than the slope of the Sanokai Wash at the confluence. Therefore, it is necessary to remove material from the flow so it will not deposit in the EMF downstream of the confluence. The size of the basin is found by using the capacity of Reach 2 of Sanokai Wash for the 100-year discharge event, as Reach 1 is too short. It is estimated that a sedimentation basin be constructed to hold approximately 26,000 yd^3 of material.

Consideration should be given to using the existing channel and existing sedimentation basin. In addition to the sedimentation basin located at the confluence with the EMF, Reach 5 (just upstream of Hawes Road) will act as natural sedimentation basin. That portion of the wash should be excavated following large events to maintain its capacity. The amount that will deposit will depend on the sediment input from the main branch upstream. It has a large capacity but it is not clear how much is delivered to that Reach (Reach 6) from upstream sources. A detailed analysis defining sediment input to Sanokai Wash Main Branch should be considered.

Additionally, sediment continuity analyses were performed for Queen Creek and Sanokai Wash to evaluate channel degradation as an input into the local scour evaluation for both the 5-year and the 100-year flow.

The total scour potential at Queen Creek and Sanokai Wash was estimated by calculating the local scour at various reaches and from the depth of degradation estimated using the sediment continuity analysis. The estimated total scour at Queen Creek ranged from 2.1 feet at Reach 7 to 15.6 feet at Reach 4. At Sanokai Wash Main Branch the estimated total scour was 1.7 feet at Reach 1 and none was estimated at the East Branch of Sanokai Wash. The estimated pier scour at Queen Creek ranged from 2.1 feet at Schnepf Road to 11 feet at Power Road. The estimated contraction scour at Queen Creek ranged from 0.27 feet at Hawes Road to 0.29 feet at Power Road. As there are no bridges in Sanokai Wash Main and East Branches, pier scour and contraction scour was not estimated.

Additionally bend scour was estimated at various meandering locations of Queen Creek and Sanokai Wash. The estimated bend scour for Queen Creek ranged from 5.2 feet at Sossaman Road in Reach 3 to 16.4 feet at SPRR Bridge at Reach 6. Bend scour at Reaches 2 and 7 of Queen Creek were not estimated, as they are straight reaches. The estimated bend scour for Sanokai Wash Main Branch ranged from 4.8 feet downstream of Power Road in Reach 6 to 11.8 feet at Riggs Road in Reach 5. Bend scour at Reaches 1 and 2 of Sanokai Wash main Branch were not estimated, as they are straight reaches. The estimated bend scour for Sanokai Wash East Branch ranged from 7.5 feet at Crismon Road in Reach 3E to 9 feet at Riggs Road in Reach 1E. Bend scour at Reach 2E of East Branch was not estimated, as this is a straight reach.

6 LATERAL MIGRATION ANALYSIS

6.1 Aerial Photography Comparison

A comparison of aerial photographs taken of Queen Creek and Sanokai Wash in 1953, and 1999 was made to identify areas of significant channel change, to determine historical rates of channel migration and to relate changes in the watershed to channel changes. The scale of the aerial photographs for both time periods is approximately 1 inch =1,200 feet. Also available were aerial photographs from 1958, 1987 and 1993. However, these photographs were not used because of poor resolution and incomplete coverage. In the following sections observations of channel planform changes for each of the channel reaches within the study area are described.

6.1.1 Queen Creek

In the following sections the changes in channel planform for Queen Creek are discussed. Refer to Table 4-1 for channel reach locations.

6.1.1.1 Reach 7

Reach 7 has historically had a braided channel planform with top widths ranging from 200 feet in fairly straight sections to nearly 800 feet in the channel bends. Evidence of historical braiding and channel avulsions is apparent at numerous locations within the reach. There are several gravel pits within the reach including one that was observed in the main channel of Queen Creek during field reconnaissance. This pit as well as other pits adjacent to the channel could lead to potential channel instability both upstream and downstream of this reach. The potential for head cutting and downstream degradation could lead to future changes in the channel planform.

6.1.1.2 Reach 6

Historically, Reach 6 has had a braided channel planform with top widths ranging from 100 feet in fairly straight sections to nearly 550 feet in the channel bends. Evidence of historical braiding and channel avulsions is apparent at numerous locations within the reach. In the 1999 photos it is seen that Reach 6 has been channelized and has an approximate top width of 200 feet. The current channel is essentially straight except for bends at Vineyard Road and Rittenhouse Road. There are several gravel-mining operations along the banks of the channel that have redefined channel bank locations.

6.1.1.3 Reach 5

As seen in the 1953 photos, the upstream end of Reach 5 (downstream of Rittenhouse Road) historically had a braided pattern. However, this portion has been channelized. The current channel is essentially straight except for the bends at Signal Butte Road and Crismon Road. The current channel top width is approximately 100 feet.

6.1.1.4 Reach 4

The upstream (downstream of Ellsworth Road) and middle (downstream of Hawes Road) portions of Reach 4 historically had a meandering pattern. The meandering portions have been channelized. However, the present channel still retains a slightly sinuous pattern. The current channel top width is approximately 105 feet.

6.1.1.5 Reach 3

This entire reach had been channelized since at least 1953. Comparison of the aerial photographs showed no significant migration of the channel or erosion of banks. The current channel top width is approximately 100 feet and is essentially straight.

6.1.1.6 Reach 2

This entire reach had been channelized since at least 1953. However, recently, the channel has been realigned between Power Road and Recker Road as part of the construction of a golf course. The current channel top width is approximately 90 feet and has a slightly sinuous planform.

6.1.1.7 Reach 1

This entire reach had been channelized since at least 1953 and consists of four straight reaches. Comparison of the aerial photographs showed no significant migration of the channel or erosion of banks. The current channel top width is approximately 110 feet.

6.1.2 Sanokai Wash Main Branch

In the following sections the changes in channel planform for Sanokai Wash Main Branch are discussed. Refer to Table 4-2 for channel reach locations.

6.1.2.1 Reach 6

Flow in Sanokai Wash between the County Line and Riggs Road is essentially overland. There does not appear to be any developed channel at this location. More recently the section between the County Line and Santan Blvd has been channelized. The current channel top width of this section is approximately 45 feet.

6.1.2.2 Reach 5

The entire reach is fairly straight and shows no evidence of historical changes in channel planform. The current channel top width of this section is approximately 60 feet.

6.1.2.3 Reach 4

Historically the downstream end (upstream of Sossaman Road) of Reach 5 had a highly sinuous channel planform. The current channel top width of this section is approximately 100 feet and has slight sinuosity.

6.1.2.4 Reach 3

The middle portion of Reach 4 historically showed slight sinuosity with a channel top width of approximately 90 feet. The current channel top width of this section is approximately 70 feet and remains slightly sinuous.

6.1.2.5 Reach 2

This entire reach had been channelized since at least 1953. Comparison of the aerial photographs showed no significant migration of the channel or erosion of banks. The current channel top width is approximately 60 feet and is essentially straight.

6.1.2.6 Reach 1

This entire reach had been channelized since at least 1953. Comparison of the aerial photographs showed no significant migration of the channel or erosion of banks. The current channel top width is approximately 200 feet and is essentially straight.

6.1.3 Sanokai Wash East Branch

In the following sections the changes in channel planform for Sanokai Wash East Branch are discussed. Refer to Table 4-3 for channel reach locations.

6.1.3.1 Reaches 4, 3 and 2

The resolution of aerial photographs was insufficient for making historical planform comparisons

6.1.3.2 Reach 1

Historically, the channel was comprised of a series of multiple small parallel channels. This entire reach has recently been channelized. The current channel top width is approximately 20 feet and is essentially straight.

6.2 Allowable Velocity Analysis

Allowable velocity approach is applied to demonstrate the stability of channel material under 100-year flood conditions. Under this approach, the velocity of the 100-year peak flow within the watercourse adjacent to the site under consideration is compared to an "allowable" velocity (the velocity at and below which erosion is not expected to occur). The allowable velocity analysis for various sections of Queen Creek and Sanokai Wash to demonstrate the channel stability, was estimated using the procedure defined in Arizona Department of Water Resources State (ADWR) Standard 5-96, State Standard for Watercourse System Sediment Balance, dated September 1996.

The allowable velocity (V_a) can be estimated using the following equation:

$$Va = V_b(C_a)(C_b)(C_d)$$

Where V_b is the maximum allowable flow velocity and C_a, C_b and C_d are the correction factors for channel alignment, bank slope and flow depth, respectively. If the computed velocity exceeds the maximum allowable velocity, erosion may occur and bank protection will be needed. The results of the analysis are summarized in Tables 6-1 through 6-3.

6.2.1 Queen Creek

The allowable velocity for each channel station (of HEC-RAS) of Queen Creek was estimated based on the flow velocity, bank material and correction factors. Bank material samples were available for Reaches 7, 4 and 3 and for the others reaches bed material samples were used to estimate the allowable velocity. The results of the analysis are shown in Table 6-1. All locations except for portion of reaches 5, 6 and 7 of Queen Creek may require bank protection.

Table 6-1: Allowable Velocity Analysis for Queen Creek.

Reach	*Upstream River Station*	*Downstream River Station*	*Allowable Velocity (fps)*	*Bank Protection Required*
7	72630	69491	2.9	Yes
7	68947	68207	2.9	No
7	67809	62361	2.9	Yes
7	61927	60511	2.9	No
7	60427	59007	2.9	Yes
6	58771	56329	2.6	Yes
6	55770	-	2.6	No
6	55259	45490	2.6	Yes
5	45410	43663	3.6	Yes
5	43463	-	3.6	No
5	43263	32235	3.6	Yes
4	32084	18432	3.2	Yes
3	18366	13199	2.6	Yes
2	13159	7786	2.1	Yes
1	7666	166	2.4	Yes

6.2.2 Sanokai Wash Main Branch

The allowable velocity for each channel station (of HEC-RAS) of Sanokai Wash Main Branch was estimated based on the flow velocity, bank material and correction factors. Bank material samples were available for Reaches 5 and 4 and for the others reaches bed material samples were used to estimate the allowable velocity. The results of the analysis are shown in Table 6-2. Bank protection may be required for the entire length of Sanokai Wash Main Branch.

Table 6-2: Allowable Velocity Analysis for Sanokai Wash Main Branch.

Reach	*Upstream River Station*	*Downstream River Station*	*Allowable Velocity (fps)*	*Bank Protection Required*
6	6142	355	2.8	Yes
5	30900	28000	3.0	Yes
4	27837	20242	2.9	Yes
3	20167	11602	2.9	Yes
2	11120	2956	2.9	Yes
1	2838	251	2.9	Yes

6.2.3 Sanokai Wash East Branch

The allowable velocity for each river station (of HEC-RAS) of Sanokai Wash East Branch was estimated based on the flow velocity, bank material and correction factors. Bed material samples were used to estimate the allowable velocity as bank material samples were not available for the East Branch. The results of the analysis are shown in Table 6-2. Bank protection may be required for the entire length of Sanokai Wash East Branch.

Table 6-3: Allowable Velocity Analysis for Sanokai Wash East Branch.

Reach	*Upstream River Station*	*Downstream River Station*	*Allowable Velocity (fps)*	*Bank Protection Required*
4E	48164	45015	2.9	Yes
3E	44615	39474	2.9	Yes
2E	39100	33900	2.9	Yes
1E	33500	31400	2.9	Yes

6.3 Summary

The historic evidence suggests that Queen Creek has remained relatively stable since 1953. However, human influences such as channelization have occurred during this time period that would mask evidence of channel instability. The upstream reaches of Queen Creek (reaches 6 and 7) appear to have historically been the most unstable. The braided planform suggest that historically these reaches had an abundant supply of sediment from upstream. The break in channel slope reduced the transport capacity of the channel causing significant aggradation. The reduced upstream supply of sediment caused by the Sanoqui Flood Retarding Structure (SFRS) and the Central Arizona Project (CAP) canal have likely reduced the instability associated with excess sediment.

However, gravel mining within these reaches has created the potential for future instability. The locations of the gravel pits along the margins of the channel increase the risk on a future avulsion of the channel. Further, the gravel pit currently located in the channel will likely cause major channel instability as head cutting upstream will cause channel incision, bank failure and potentially failure of upstream structures such as bridges or road crossings. Downstream degradation due to loss of upstream supply could lead to further instability that may lead to increased bank erosion. It is suggested that further investigation of the impacts of the gravel mining on Queen Creek be conducted.

Sonakai Wash Main Branch appears to have remained relatively stable since 1953. Reach 4 appears to have been the most unstable. A large meander bend existed just upstream of Sossaman Road. This suggests that this area was historically the site of deposition. This section of channel appears to have been channelized. Future channel instability may be expected at this location.

The majority of Sanokai Wash East Branch is a poorly developed system of small ditches and washes. No planform evidence was available to make any conclusions about the stability of this system. However, the sediment size characteristics of the channel suggest that the majority of the downstream reaches are depositional.

Bank stability of Queen Creek and Sanokai Wash was estimated using the procedure defined in Arizona Department of Water Resources State (ADWR) Standard 5-96 the State Standard for Watercourse System Sediment Balance, dated September 1996. The allowable velocity for each channel station (of HEC-RAS) of Queen Creek and Sanokai Wash was estimated based on the flow velocity, bank material and correction factors for channel alignment, bank slope and flow depth. Allowable velocities typically ranged from 2.1 to 3.6 fps. Average channel velocities

typically ranged from 4.5 to 6.6 fps. All reaches of Queen Creek exceeded the allowable velocities except for a portion of Reaches 5, 6 and 7. These are the only portions that may not require bank protection. All reaches of Sanokai Wash Main and East Branch may require bank protection.

7 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations for this study include the following:

- Hydrologic analysis was conducted for Queen Creek and Sanokai Wash to estimate the various flood events (5-, 10-, 25-, and 50-year), required for the hydraulic and sediment transport analyses. As specified by the Flood Control District of Maricopa County, the Soil Conservation Service (SCS) Type II distribution curve was used for the future conditions 2-, 5-, 10-, 25-, and 50-year return period precipitation events. The Flood Control District of Maricopa County agreed that this was not the correct way to model other events, but was considered acceptable for a planning level analysis. Additional hydrologic analysis is recommended as part of the design phase.

- Hydraulic analysis of Queen Creek and Sanokai Wash was conducted for 2-, 5-, 10-, 25-, 50- and 100-year return period discharge events, to define the hydraulics of the preferred alternative. Reaches of similar hydraulic characteristics were defined based on hydraulic characteristics such as velocity, slope, top width and hydraulic depth and hydraulic controls such as bridges, culverts and dip road crossings. It should be noted that the hydraulic model might not represent the actual channel geometry in Reach 2. Additional hydraulic analysis using the actual channel geometry in Reach 2 should be conducted during the design phase. Additionally, issues were identified with the existing conditions hydraulic model developed by Huitt~Zollars, Inc. These were provided to Huitt~Zollars, Inc. for revision. Some revisions were made and the model was judged adequate for the analysis by Huitt~Zollars, Inc.

- Sediment size characteristics for Queen Creek and Sanokai Wash suggest that the bed material is generally finer in the downstream reaches as the sediment transport capacity of the channel is reduced. Significant deposition of coarse material occurs above the Sanoqui Flood Retarding Structure (SFRS) upstream of the study area. Additionally, coarse materials such as cobbles and gravels do not appear to be present within the study limits for Sanokai Wash, suggesting that deposition of these materials occurs further upstream. However, cobble and gravel sized materials are present within the upper reaches of Queen Creek. This material was likely transported to these reaches prior to the placement of the SFRS and the CAP canal. The major sediment size fractions transported through the study reaches of Queen Creek and Sanokai Wash are sands, silts and clays.

- Bed material sizes ranged from very coarse gravel to silts and finer in Queen Creek while bed material sizes for Sanokai Wash were slightly finer ranging from coarse gravel to silts and finer. Bank material is general finer than bed material and is non-cohesive. This would suggest that high flows with sufficient velocities could erode the banks along both Queen Creek and Sanokai Wash.

- The large amount of fine material forming the bed of Sanokai Wash in the vicinity of Riggs Road suggests that this area is depositional. The break in slope at this location reduces the channel velocities thus reducing the sediment transport capacity. The

reduced transport capacity causes sediment to deposit at this location. This area may require excavation to maintain flood conveyance.

- Gravel mining in and along Queen Creek has the potential for impacting the channel not only at the locations of the pits themselves but also at locations upstream and downstream of the site. Infrastructure such at the upstream bridge at Schnepf Road and the culverts through the CAP canal may be impacted by headcutting associated with the channel avulsing into the pits. Channel instability both upstream and downstream of the pits should be expected. It is suggested that further investigation should be done during the design phase to help quantify the impacts of the gravel mining on Queen Creek.

- Upstream of Queen Creek Reach 7, the CAP channel crossing and gravel mining will limit the supply to the downstream reaches. From the equilibrium slope calculations for Queen Creek, it is estimated that Reach 6 will aggrade while all the other reaches will degrade. The armoring analysis showed that Reaches 5, 4 and 1 would armor. Therefore, no grade control structures are needed for these reaches. It is recommended that a grade-control structure with a drop of 3.6 feet be placed at Sossaman Road in Reach 3. In Reach 2 it is recommended that grade control structures with drops of 4.5 feet and 5 feet be placed at the multi-arch bridge and at Power Road, respectively. As stated in the hydraulic analysis, the geometry for Reach 2 may not represent the actual channel geometry. Additional analysis should be conducted during the design phase to determine the requirements for the grade-control structures recommended for this reach.

- The Sanokai Wash Main Branch upstream of the confluence with the East Branch (Reach 6) has very high transport capacity compared to every other reach in the system. This is due to the steep slope as well as the relatively narrow channel. Further, the culverts passing under Ellsworth Road at the upstream end of the reach may restrict the sediment supply to the reach. Therefore it is recommended that the channel in Reach 6 be lined to prevent erosion. Reach 5, located between Hawes Road and Riggs Road, will experience deposition due to its low gradient. This reach will therefore act as a natural sedimentation basin. It is recommended that the channel be excavated as necessary to maintain its capacity. Reaches 4 and 2 were shown to aggrade. Reach 3 was shown to degrade while Reach 1 was shown to have minimal degradation. It is recommended that two grade control structures be placed in Reach 3, one with a drop of 3.5 feet at Sossaman Road and another at the midpoint between Power Road and Sossaman Road with a drop of 3.5 feet.

- The equilibrium slope calculations for Sanokai Wash East Branch estimated that Reaches 1E through 3E will degrade, however the armor calculations showed that Reaches 1E through 3E will armor. The degradation required to form an armor layer in Reach 3E is approximately 3.6 feet. Therefore it is recommended that a grade control structure with a drop of 3.8 feet should be placed at Signal Butte Road.

- It is recommended that a sedimentation basin be placed at the confluence with the EMF on Queen Creek. The slope of the EMF is much flatter than the slope in Queen Creek Reach 1. Therefore, it is necessary to remove sediment from the flow so it will not

deposit in the EMF downstream of the confluence. The sedimentation basin should be designed to hold approximately 45,000 yd^3 of material.

- It is recommended that a sedimentation basin be placed at the confluence with EMF on Sanokai Wash. The sedimentation basin should be designed to hold approximately 26,000 yd^3 of material. Consideration should be given to using the existing channel and existing sedimentation basin as the Preferred Alternative has the Sanokai Wash confluence located on the outside of a bend in the EMF. This may cause additional hydraulic and sedimentation problems.

- Reach 5 of Sanokai Wash Main Branch (just upstream of Hawes Road) will act as a natural sedimentation basin. That portion of the wash should be excavated following large events to maintain its capacity. The amount deposited will depend on the sediment input from the Reach 6. This reach has a large capacity but it is not clear how much is delivered to the reach from upstream sources. A detailed watershed sediment analysis defining sediment input to Sanokai Wash Main Branch should be undertaken as part of the design phase.

- The total scour potential at Queen Creek and Sanokai Wash was estimated by calculating the local scour at various reaches and adding it to the depth of degradation estimated using the sediment continuity analysis. The estimated total scour at Queen Creek ranged from 2.1 feet at Reach 7 to 15.6 feet at Reach 4. At Sanokai Wash Main Branch the estimated total scour was 1.7 feet in Reach 1 while 0.0 feet scour was estimated in the East Branch of Sanokai Wash. The estimated pier scour for Queen Creek ranged from 2.1 feet at Schnepf Road to 11 feet at Power Road. The estimated contraction scour for Queen Creek ranged from 0.27 feet at Hawes Road to 0.29 feet at Power Road. As there are no bridges in Sanokai Wash Main and East branches, pier scour and contraction scour were not estimated.

- Bend scour was estimated at various meandering locations of Queen Creek and Sanokai Wash. The estimated bend scour for Queen Creek ranged from 5.2 feet at Sossaman Road in Reach 3 to 16.4 feet at SPRR Bridge at Reach 6. The estimated bend scour for Sanokai Wash Main Branch ranged from 4.8 feet downstream of Power Road in Reach 6 to 11.8 feet at Riggs Road in Reach 5. The estimated bend scour for Sanokai Wash East Branch ranged from 7.5 feet at Crismon Road in Reach 3E to 9 feet at Riggs Road in Reach 1E.

- Historic evidence suggests that Queen Creek has remained relatively stable since 1953. However, human influences such as channelization have occurred during this time period that would mask evidence of channel instability. The upstream reaches of Queen Creek (reaches 6 and 7) appear to have historically been the most unstable among all reaches. The braided planform in the 1953 photos suggest that these reaches had an abundant supply of sediment from upstream. The break in channel slope reduced the transport capacity of the channel causing significant aggradation. The reduced upstream supply of sediment caused by the Sanoqui Flood Retarding Structure (SFRS) and the Central

Arizona Project (CAP) canal have likely reduced the instability associated with the excess sediment supply from upstream.

- Sonakai Wash Main Branch appears to have remained relatively stable since 1953. Reach 4 appears to have been the most unstable among all reaches. A large meander bend existed just upstream of Sossaman Road. This suggests that this area was historically the site of deposition. This section of channel appears to have been channelized. However, future channel instability may be expected at this location.

- The majority of Sanokai Wash East Branch is a poorly developed system of small ditches and washes. No planform evidence was available to make any conclusions about the stability of this system. However, the sediment size characteristics of the existing channel suggest that the majority of the downstream reaches are depositional. The Preferred Alternative sediment transport analyses indicates the planned channel configuration may cause degradation of Sanokai Wash East Branch.

- Bank stability for Queen Creek and Sanokai Wash was estimated by determining the allowable velocity for each channel station (of HEC-RAS). This was estimated based on the flow velocity, bank material and correction factors for channel alignment, bank slope and flow depth. Allowable velocities typically ranged from 2.1 to 3.6 fps. Average channel velocities typically ranged from 4.5 to 6.6 fps. All reaches of Queen Creek exceeded the allowable velocities except for portions of Reaches 5, 6 and 7. These are the only portions that may not require bank protection. All reaches of Sanokai Wash Main and East Branch may require bank protection. Bank stability is a problem, based on the analysis conducted to date and should be investigated during the design phase.

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