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DATE 10/15/15 BY GDS

ASARCO Mission Mine Complex
Pima County, Arizona

DRAINAGE REPORT:
Diversion Channel Re-Establishment Project – South Dump to Tailings Storage Facilities 6 & 7

SEPTEMBER 2015

TETRA TECH

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SEPTEMBER 2015

Prepared for:
ASARCO Groupo Mexico
Mission Complex
4201 W. Pima Mine Road
Sahuarita, AZ 85629

Prepared by:

TETRA TECH

4801 East Broadway Blvd., Suite 521, Tucson, AZ 85711-3652
Tel 520.623.7900 Fax 520.884.5278
tetratech.com
(our)facewater.com

TETRA TECH

Mr. Thomas Klempel, P.E.
Environmental Manager - Mission Complex
4201 W. Pima Mine Road
Sahuarita, AZ 85629
(520) 648-4588

Subject: Floodplain-Use-Permit - Drainage Report for Mission Stormwater Control (Diversion Channel Re-Establishment - South Dump to Tailing Storage Facilities 6 & 7)

Dear Mr. Klempel:

Tetra Tech has completed its Hydrologic and Hydraulic investigation regarding the Mission Complex Diversion Channel Re-Establishment Project from the South Dump to Tailing Storage Facilities (TSF) 6 & 7. Enclosed with this letter you will find documentation for the assumptions, analyses, and results for the hydrology, hydraulics, and sediment-transport/scour components to support re-establishment of the Mission Stormwater Controls.

Sincerely,

Registered Professional Engineer (Civil)
CERTIFICATE NO.
49108
JONATHAN D. ELSLAGER
Date Signed 3/28/2018
ARIZONA U.S.A.
Expires 3/31/2018

Jonathan Elslager, PE, CFM
Project Manager
Tetra Tech, Inc.

4801 East Broadway Blvd., Suite 521, Tucson, AZ 85711-3652
Tel 520.623.7980 Fax 520.884.5278
tetratech.com |
tisurfacewater.com

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1.0 INTRODUCTION

1.1 Project Location

The Project is located within the State of Arizona, Gila and Salt River Meridian (G&SRM), Township 17 South, Range 12 East, Section 12; and Township 17 South and Range 13 East, Sections 7, 8, 9, and 10. The Mission Complex Drainage Channel Re-establishment Project Site (the Project) is located along West Helmet Peak Road and lies less than one mile from the Town of Sahuarita, in Pima County Arizona.

Figure 1: Vicinity Map

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1.2 Objective

The existing diversion channels at the Project predominantly convey flows from west to east toward the Santa Cruz River around the southern end of the ASARCO Mission Complex along the north side of West Helmet Peak Road. Over time, the existing diversion channels have been damaged from storm water flows. This study was conducted in support of rehabilitation maintenance operations required under a 404-Permit Application to the US Army Corps of Engineers (USACE) for the Project, which is located in unincorporated Pima County, Arizona.

Tetra Tech has been contracted to provide supporting engineering services related to the re-establishment of stormwater controls at the Mission Mine Complex. ASARCO will be restoring damaged controls which are within a jurisdictional 404-delineated watercourse as defined by the United State Army Corps of Engineers (USACE). Westland Resources, Inc. (Westland) is the firm that has been contracted to coordinate the 404-Permit process with the USACE. ASARCO will be restoring stormwater controls for up to the 10-year-recurrence interval storm, based on the current requirements outlined by the USACE.

The South Dump and Tailings Storage Facilities (TSF) 6 & 7 Diversion Channels convey floodwaters from west to east along the southern part of the Mission Mine complex (adjacent and parallel to West Helmet Peak Road), which is located approximately 18 miles south of Tucson, Arizona (Figure 1). The objective of this report is to document the current and proposed drainage conditions for the South Dump and TSF 6 & 7 Diversion Channels and provide design guidance which, once implemented, will re-establish separation of contact and non-contact stormwater runoff.

2.0 SURVEY AND MAPPING

Watershed delineations and elevations for input into applicable regression equations were based on USGS 7.5 minute quadrangle maps.

For hydraulic analyses, both 2-foot and 5-foot-contour mapping of the Project area was provided to Tetra Tech by ASARCO. The detailed mapping is based on field-survey data collected by Environmental Field Service, LLC. (2015). Horizontal control is based on a local mine grid, and is in units of feet. All elevations are based upon North American Vertical Datum of 1988 (NAVD 88).

3.0 HYDROLOGIC ANALYSIS

This section describes the documented offsite hydrology for the Mission Complex. Hydrologic data for the offsite watersheds were developed and evaluated as part of a previous task, as presented in a 2015 Hydrology and Design Criteria Technical Memorandum (Tetra Tech, 2015). Results of this study are summarized in the following section for all offsite watersheds, however, the focus of the report is on the offsite watersheds impacting the diversion channels along the Mission Complex's South Dump and TSF 6 & 7. This hydrology was used as an aid in developing a recommended diversion capacity to provide to the contractor in the design specifications.

3.1 Hydrologic Method Overview (NRCS Method)

The NRCS CN procedure was developed for general hydrologic analysis and allows for various storm distributions and durations to be analyzed. This method is nationally accepted, as well as locally accepted within Pima County, Arizona. The method is applicable to the analysis of large complex watershed systems, where landscape conditions may change over time. This method was selected for the existing-conditions hydrologic modeling because of the local and national acceptance and the amount of readily available hydrologic data (i.e., soil data, topography, and land cover) to characterize the watershed areas.

Watershed characteristics were processed using geospatial data from USDA, USGS, and Pima County. Soil data was derived from NRCS Soil Data, as locally modified and available from Pima County.

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The analysis was performed using Hydrologic Engineering Center's Hydrologic Modeling Software (HEC-HMS), Version 3.5. HEC-HMS is a hydrologic modeling software package developed by the U.S. Army Corps of Engineers (USACE). HEC-HMS incorporates the NRCS CN procedure and allows for the analysis of complex and integrated systems (i.e., multiple sub-basins, reservoir and channel routing, etc.).

The primary input variables for determining the peak flow associated with stormwater runoff are:

- Precipitation;
- Rainfall Losses and Curve number;
- Basin delineation (i.e., watershed area); and
- Time of concentration or lag time.

The NRCS unit-hydrograph method was selected by Tetra Tech to account for appropriate losses (i.e., infiltration) and to estimate stormwater runoff (both peak and volume). It was also used to develop a relationship of runoff versus time for a specific basin, presented as a hydrograph. The area under a hydrograph represents the expected total runoff volume of the basin, and the apex of the hydrograph represents the estimated peak flow rate of the basin.

The estimated values for existing-conditions peak runoff will be used for the hydraulic design of the channels and their corresponding elements.

3.1.1 Precipitation

Precipitation data was obtained for the 24-hour, 100-year event from the National Oceanic Atmospheric Administration (NOAA) Atlas 14; and the upper 90-percent confidence-interval precipitation value was used. The coordinates used to obtain precipitation data for the Project were based on the centroid of the watersheds delineated. Precipitation point values were used to create rainfall hyetographs. Note that short-duration precipitation events are likely to produce larger runoff peak discharges due to the intensity of the storm events. Therefore, in addition to evaluating the 24-hour, 100-year storm required for prescriptive ADEQ BADCT requirements, the evaluation of short-duration intense precipitation events was also conducted by incorporating a 3-hour, 100-year precipitation event in the HEC-HMS model as well. For each watershed, a rainfall hyetograph was prepared using a tool developed by the Pima County Regional Flood Control District.

Table 1. NOAA Atlas 14 Precipitation Point Values

Event Frequency (year)	Event Duration (hour)	Precipitation (inches)[1]
100	24	4.40
100	3	3.32

1. Upper bounds of the 90% confidence interval shown

3.1.2 Rainfall Losses - Curve Number

As noted previously, the NRCS has developed a widely used CN procedure for estimating runoff from storm events. This procedure was used to estimate the direct runoff values for each basin and was selected due to its wide application and incorporation into many hydrologic modeling programs. Initial rainfall infiltration losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land-use numbers to assign runoff factors known as Curve Numbers (CNs). These CNs represent the runoff potential of a soil type or surface area (i.e., the higher the CN, the higher the runoff potential). It should be noted that long-duration events have lower CNs; however, such adjustments were not made for the purposes of this study. Not making such adjustments is considered to be conservative (safe) for purposes of this study effort.

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A. Soil Group

In order to determine the runoff potential, the NRCS classifies soils as "A," "B," "C," or "D," based on their hydrologic soil group. Type "A" soils, such as sandy soils, have a very low runoff potential (note: to be conservative (safe), Type "A" soils are generally replaced with Type B soils when encountered in Pima County). Heavy clay and mucky soils, as well as shallow/rocky soils, are classified as Type "D" soils, and have a very high runoff potential. The United States Department of Agriculture (USDA) maintains a database of hydrologic soil data that has been compiled as a part of their soil surveys, and provides this information through maps.

Because each basin is assigned one CN value, regardless of the quantity of soil groups and land uses that it might contain, a composite CN gives a more accurate representation of the "true" conditions at the site. Composite CNs are obtained by following the same procedures above; however, each composite CN is then weighted with respect to how much area of the basin it occupies. These weighted CNs are then averaged to arrive at the final CN value. Table 2 shows the Hydrologic Soil Groups, and includes a brief description regarding their properties.

Table 2. Hydrologic Soil Groups (Excluding Type "A" Soils)

Hydrologic Soil Group	Description*
Type B (Moderately low runoff potential)	These soils have a moderate infiltration rate when thoroughly wetted. They chiefly are moderately deep to deep, moderately well drained to well drained, soils that have moderately fine to moderately coarse textures. They have a moderate rate of water transmission (4 to 8 mm/hr), and are generally described as silty loam and loam.
Type C (Moderately high runoff potential)	These soils have a slow infiltration rate when thoroughly wetted. They chiefly have a layer that impedes downward movement of water or have moderately fine to fine texture. They have a slow rate of water transmission (1 to 4 mm/hr), and are generally described as sandy clay loam.
Type D (High runoff potential)	These soils have a very low infiltration rate when thoroughly wetted. They chiefly consist of clay soils that have a high swelling potential; soils that have a permanent high water table; soils that have a claypan or clay layer at or near the surface; and shallow soils over nearly impervious material. They have a very slow rate of water transmission (0 to 1 mm/hr), and are generally described as clay loam and silty clay loam.

*Descriptions obtained from the Pima County Regional Flood Control District PC-Hydro User Guide.

3.1.3 Basin Delineations

Unique contributing offsite watershed areas were delineated at key point along the western side of the Mission Complex where diversion channels are located. Each offsite drainage basin, was analyzed as part of the design and are shown in Figure 2, in addition to being depicted on the HEC-HMS Schematic in Figure 3.

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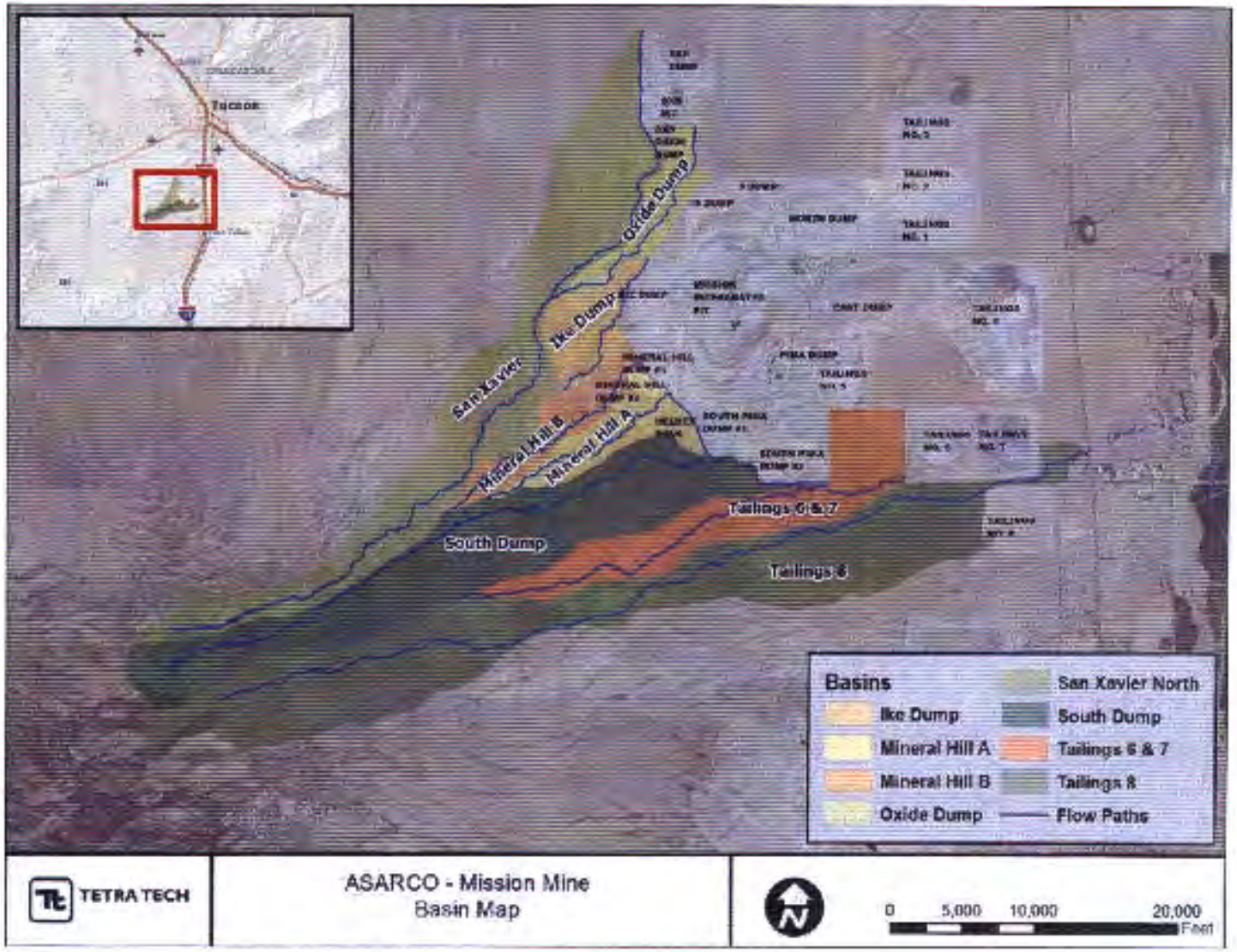

Figure 2: Offsite Watershed Basin Map

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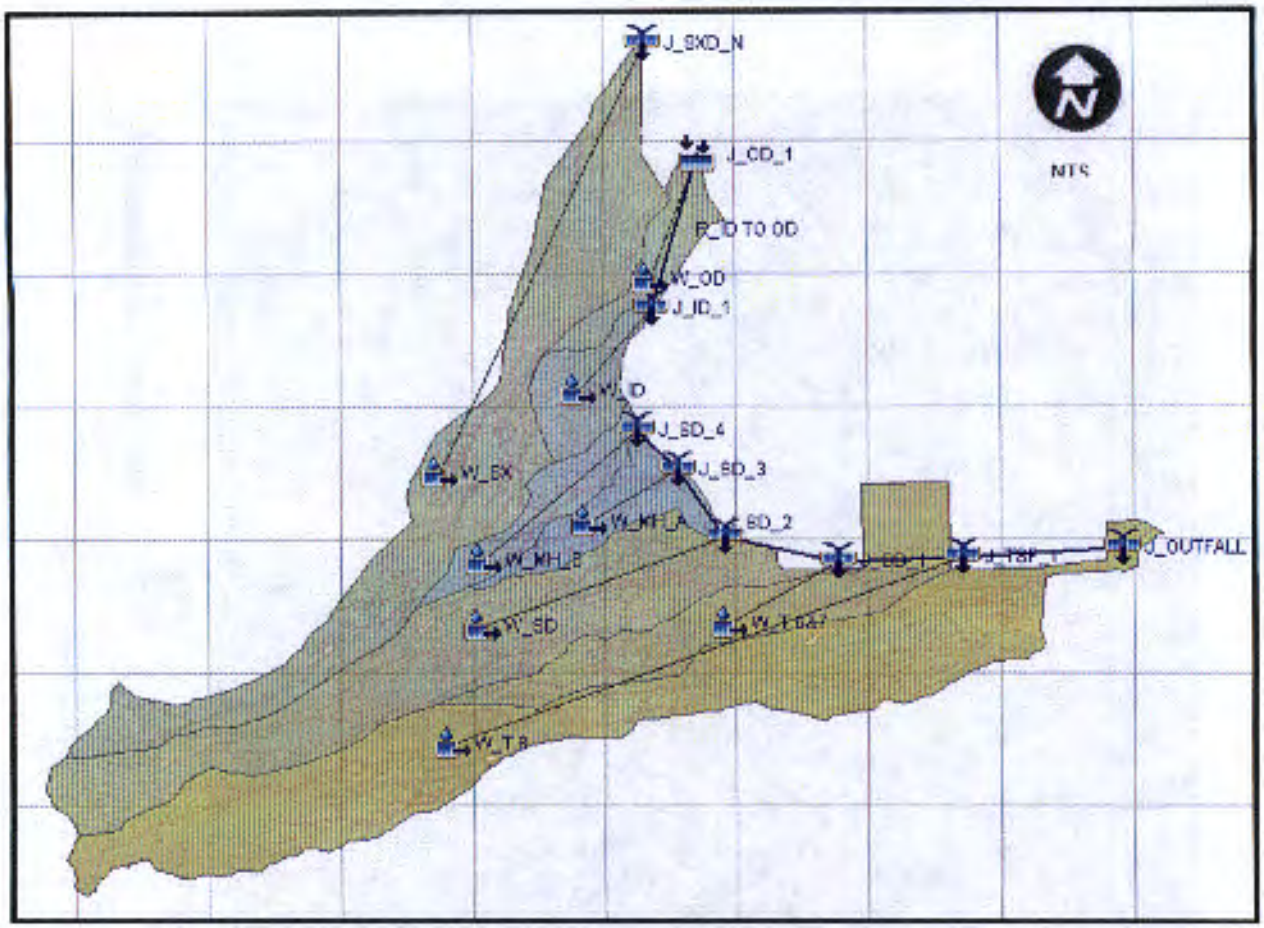

Figure 3: HEC-HMS Model Schematic

3.1.4 Rainfall Runoff Excess

The NRCS method estimates rainfall runoff excess as a function of cumulative precipitation, soil cover, land use, and the Antecedent Moisture Condition (AMC) using the following relationships:

$$Pe = \frac{(P - Ia)^2}{P - Ia + S}$$

$$S = \frac{1000}{CN} - 10$$

Where:

Pe = the accumulated precipitation excess, in inches;
P = the accumulated precipitation depth, in inches;
S = the maximum soil water retention parameter, in inches;
Ia = the initial abstraction, in inches; and
CN = the curve number.

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From an analysis of many small experimental watersheds, NRCS developed an empirical relationship between the initial abstraction and the maximum soil water-retention parameter given by the following expression:

$$Ia = 0.2 * S$$

Therefore, the determination of accumulated precipitation excess can be rewritten as:

$$Pe = \frac{(P - 0.2S)^2}{P + 0.8S}$$

3.1.5 Time of Concentration/Lag Time

The time of concentration (T_c) is the travel time for runoff to travel from the hydraulically most distant point in the basin to the outlet during a period of the most intense rainfall excess. The time of concentration was determined by considering the most hydraulically distant flow path for each basin.

Lag time for this project was computed using NRCS TR-55 (1986) methods for sheet flow, shallow concentrated flow, and channel flow.

The hydraulic length and watershed slope required were developed using ArcGIS and available topographic data. The lag times were then used in HEC-HMS to establish the peak flows at the selected concentration points. Table 3 presents the calculated lag times.

Table 3. Summary of Lag Time Parameters and Calculations

Sub-basin	Hydrologic Element	Lag Time (min)
South Dump	W_SD	123.58
Mineral Hill A	W_MH_A	51.79
Mineral Hill B	W_MH_B	48.31
Tailings 6 & 7	W_T 6 & 7	78.81
Tailings 8	W_T 8	157.38
San Xavier	W_SX	132.61
Ike Dump	W_ID	28.67
Oxide Dump	W_OD	60.81

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3.1.6 Routing Method

"Channel routing" is the term applied to the NRCS method to account for the effects of channel storage on the runoff hydrograph as the storm event moves through a channel reach. Channel routing is used to translate and attenuate an upstream runoff hydrograph into a downstream hydrograph during the specific design storm event. In this regard, the Kinematic Wave Method was used for the routing of flow through and along the channel sections, which have slopes at approximately 12%.

The physical channel characteristics used in the routing are presented in Table 4.

Table 4: Summary of Channel Routing Parameters

Routing Reach	Length (ft)	Slope (ft/ft)	Manning N-Value	Reach Segments	Shape
R_SD4 TO SD3	3200	0.002	0.035	2	Trapezoid
R_SD3 TO SD2	5200	0.004	0.035	2	Trapezoid
R_SD2 TO SD1	9000	0.013	0.035	8	Trapezoid
R_SD1 TO TSF	6000	0.0275	0.043	3	Trapezoid
R_TSF TO OUTFALL	16000	0.0195	0.037	12	Trapezoid
R_ID TO OD	10000	0.025	0.041	2	Trapezoid

3.2 Existing Conditions Hydrologic Results

Table 5 summarizes the cumulative peak-runoff results from the hydrologic analysis using the 100-year, 24-hour event, as well as the 100-year, 3-hour event.

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Table 5: Summary of Existing HMS Output

Hydrologic Element	Drainage Area (sq mi)	100-Year, 3-Hour Event			100-Year, 24-Hour Event		
		Peak Discharge (cfs)	Time of Peak (hour:min)	Volume (ac-ft)	Peak Discharge (cfs)	Time of Peak (hour:min)	Volume (ac-ft)
W_SD	7.19	2253.7	03:40	281.9	1993.1	12:06	1101.3
W_MH_A	1.36	1183.9	02:20	142.3	746.5	10:48	240.7
W_MH_B	1.25	1043	02:15	120.9	650.1	10:42	203.1
J_SD_4	1.25	1043	02:15	120.9	650.1	10:42	203.1
R_SD4 TO SD3	1.25	1038.8	02:45	90.4	645	11:24	201.7
J_SD_3	2.61	1933.6	02:40	232.7	1237.9	11:12	442.5
R_SD3 TO SD2	2.61	1929.2	02:50	218.5	1220.3	11:18	442.2
J_SD_2	9.8	3665	03:00	500.4	2958.9	11:36	1543.5
R_SD2 TO SD1	9.8	3662.3	03:05	454.9	2955.7	11:42	1543
W_T 6 &7	3.53	1492	02:50	210	1234.3	11:18	520.9
J_SD_1	13.33	5105.6	03:00	664.9	4114.7	11:36	2063.9
R_SD1 TO TSF	13.33	5104.9	03:05	628.5	4110	11:42	2063.8
W_T 8	10.01	2143.5	04:00	216.1	2307.2	12:48	1486.1
J_TSF_1	23.34	6675.5	03:20	844.7	5995.4	11:54	3549.9
R_TSF TO OUTFALL	23.34	6670.8	03:30	710.5	5993.8	12:06	3547.2
J_OUTFALL	23.34	6670.8	03:30	710.5	5993.8	12:06	3547.2
W_SX	7.76	2241.8	03:50	265.9	2131.1	12:18	1222.4
J_SXD_N	7.76	2241.8	03:50	265.9	2131.1	12:18	1222.4
W_ID	1.2	1389.9	01:55	116.6	829.9	10:24	195
J_ID_1	1.2	1389.9	01:55	116.6	829.9	10:24	195
R_ID TO OD	1.2	1376.3	02:05	113.1	821.7	10:30	194.5
W_OD	1.09	608.5	02:30	79.7	444.3	11:00	160.8
J_OD_1	2.29	1836.6	02:05	192.7	1202.4	10:36	355.4

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3.3 Estimated Regional Regression Flows

Peak discharge of stormwater runoff computed using any mathematical model should be compared to regional regression relationships to assess the reasonableness of results. Due to its remote location and the absence of a previous hydrologic study and local gage data, for calibration purposes the peak discharges for the Project were initially computed using a regional regression method described in USGS Fact Sheet 111-98, The National Flood-Frequency Program – Methods for Estimating Flood Magnitude and Frequency in Rural Areas in Arizona (USGS, 1999). Regional regression methodology is based on a regression analysis of a number of similar hydrologic regions.

The project area lies within the boundary of USGS Region 13, which is based on the above-referenced study. Table 6 identifies the estimated peak discharges for specified return intervals based upon the application of USGS Region 13 Regional Regression Equations at the project site. Comparison to the HMS Study results at four concentration points were analyzed for correlation purposes.

The four concentration points are located at key points within the model as follows. The San Xavier North receives contributions from the San Xavier watershed. Whereas, the South Dump received contributions from the South Dump watershed in addition to the Mineral Hill A and Mineral Hill B watersheds. Similar to San Xavier, the Ike Dump receives flow contribution from the Ike Dump Watershed. The watershed downstream of the Ike Dump, labeled as the Oxide Dump, was not a focus of this study; and was not indicated as a concentration point in Table 5, below. The concentration point identified as TSF 6-7 (I-19 Outfall) actually includes the upstream watershed contributing to the South Dump in addition to TSF 6 &7 and the Tailings 8 watersheds.

Table 6: Summary of Existing Condition 100-Year Stormwater Flows

Name	Hydrologic Element	Area (sq mi)	USGS RRE (Region 13 – cfs)	HMS Calculated 24-Hour	HMS Calculated 3-Hour
South Dump	J_SD_2	9.8	4,784	2,959	3,665
Tailings 6 & 7	J_SD_1	13.3	5,578	4,115	5,106
Tailings 8	J_TSF_1	23.3	7,274	5,995	6,676
Outfall	J_Outfall	23.3	7,274	5,994	6,671
Ike Dump	J_ID	1.2	1,420	830	1,390
San Xavier North	J_SXD_N	7.8	4,253	2,131	2,242

Runoff events with more frequent recurrence intervals (e.g., 2-year, 5-year, 10-year, 25-year, and 50-year events) can be approximated by using a locally accepted ratio to the computed 100-year peak discharge as presented in the *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona* (SLA, 1989). Table 7 below shows the ratios as applicable to the Mission Mine Complex.

Table 7: Ratios of More Frequent Floods to the 100-Year Flood[1]

Predominant Watershed Type	Recurrence Interval				
	2 Year	5 Year	10 Year	25 year	50 Year
Natural/Rural	0.10	0.23	0.37	0.58	0.77

[1]As excerpted from Table 4.5, pg. 4.08 (COT, 1989)

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3.4 Design Discharges

As summarized above, hydrologic data for the offsite watersheds were based on the 2015 Hydrology and Design Criteria Technical Memorandum (Tetra Tech, 2015). The referenced Technical Memorandum documented the existing-conditions hydrology for all offsite watersheds, however, peak discharges impacting the South Dump and TSF 6 & 7 Diversion Channels are the focus of this diversion channel re-establishment project and will be carried forth.

HEC-HMS peak-flow results for the 100-year, 3-hour event were found to compare well with flood peaks calculated using USGS Regional Regression Relationships (RRE) specifically developed for determining 100-year flood peaks in southern Arizona. This validation of the results provides confirmation that the computed peak discharges will be suitable for use as the basis of the future modeling on this project. Once determined, the peak-flow result for the 100-year, 3-hour event was then used in combination with locally acceptable ratios to determine peak flows for the more frequent recurrence-interval runoff events (e.g., 2-year, 5-year, 10-year, 25-year, and 50-year events). Table 8 provides a summary of the computed peak discharges generated from the upland watersheds. Location points are shown in Exhibit 1, along with the corresponding recurrence intervals used in the hydraulic analysis for the referenced South Dump and the TSF 6 & 7 Diversion Channels.

Table 8: Diversion Design Flow Data

Concentration Point	River Station	Peak Discharge (cfs)						Incremental Watershed Contribution Area (Sq. Mi.)	Cumulative Drainage Area (sq. mi)
		2 Year	5 Year	10 Year	25 Year	50 Year	100 Year		
CP1	27991	193	445	716	1,122	1,489	1,934	1.35	1.35
CP2	27330	367	843	1,356	2,126	2,822	3,665	7.19	8.54
CP3	17935	511	1,174	1,889	2,961	3,931	5,106	3.54	12.07
CP4	10057	668	1,535	2,470	3,872	5,140	6,676	10.01	22.08

4.0 HYDRAULIC ANALYSIS

4.1 Water-Surface Profiles

Steady-state water-surface profiles were computed using the River Analysis System software (HEC-RAS, v4.1, January 2010) developed by the USACE Hydrologic Engineering Center (HEC). The steady-state computation procedure is based on the one-dimensional (1-D) energy equation, with energy losses evaluated by friction (the Manning equation) and by contraction/expansion (velocity head). The HEC-RAS 1-D steady state simulation assumes that flow is steady, gradually varied, and has a stream slope less than 10 percent. A limitation to 1-D flow assumption is that velocity computations are only accounted for in the direction of flow (USACE, 2010). HEC-RAS is an approved method of floodplain delineation and analysis within Pima County (Pima County, 2007). A summary of the model inputs are included in the sections below, and are accepted methods of parameterization as described in Pima County Tech Policy 19 (Pima County, 2012).

4.2 Existing Conditions

USACE HEC developed the GeoRAS extension (USACE, 2009) to integrate within ESRI Geographic Information System (GIS) software ArcGIS (Version 10), which was used to develop the HEC-RAS input files for this project. As verification of model development, sensitivity analysis was also conducted to

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document the impact on computed water-surface elevations of the selected boundary conditions and the selected Manning roughness coefficients.

4.2.1 Cross-Sectional Geometry

Using the HEC-GeoRAS extension within ESRI ArcGIS (Version 10), the west-to-east flowing South Dump and TSF 6 & 7 Diversion Channels were characterized by utilizing a Triangulated Irregular Network (TIN) surface. The TIN was used for determining channel cross-section elevations, and was based on topographic survey data provided by Environmental Field Service, LLC. (2015). Cross-sections defined in GIS are imported to HEC-RAS by HEC-GeoRAS, Version 10 (USACE, 2009). The approximately 5.3-mile-long reach includes 102 cross-sections spaced at 100-foot intervals (on average). Bank stations were selected beginning at the geomorphic bank, which was based upon review of cross-section geometry. A channel cross-section map is included as Exhibit 2.

4.2.2 Manning Roughness Coefficients

A Manning roughness coefficient (n) can be calculated by choosing a base value (n_b) for the channel bed materials and then applying physical adjustments to account for channel irregularities, changes in alignment, flow obstructions, the presence of vegetation, and other factors (Phillips and Tadayon 2006). Due to the different land-cover types found at different cross sections along the reach, roughness coefficients are, in part, calculated according to the land-cover types at different cross sections. Table 9 shows the Manning roughness coefficients (n) for the right overbank, main channel, and left overbank for cross-section River Station 27991 to 17935 and cross-section River Station 11880 to 1584. At these cross sections, the left overbanks are covered by a combination of riprap and vegetation. Table 10 shows the Manning roughness coefficients (n) for the right overbank, main channel, and left overbank for cross section River Station 17455 to 12078 and cross-section River Station 1330 to 69. The left overbanks of these cross-sections are covered by vegetation only. For the main channel and right overbanks, there is no obvious variation in the land-cover types observed; so, for purposes of this study, it was assumed that the roughness coefficients for the main channel and right overbank remain consistent throughout the entire reach. The equation used for the Manning n calculation is shown below:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m$$

Where,

n_b = the base value of n along a straight, uniform, smooth channel in natural materials
n_1 = a correction factor to account for the effect of surface irregularities
n_2 = a value to account for variations in shape and size of the channel cross-section
n_3 = a value to account for obstructions
n_4 = a value to account for vegetation and flow conditions
m = a correction factor to account for meandering of the channel

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Table 9 Manning Roughness Coefficients (RS 27991-17935) and (RS11880-1584).

Left Over Bank	Selection	Description
n_b	0.03	Unstable channel, median grain size 1 millimeter
n_1	0.0015	Minor Irregularity
n_2	0	Alternating Occasionally
n_3	0	Negligible Obstructions
n_4	0.025	Medium, scattered shrubs, heavy weeds
m	1	Minor meandering
Selected n-value	0.0565	

Main Channel	Selection	Description
n_b	0.026	Unstable channel, median grain size 1 millimeter
n_1	0.001	Minor Irregularity
n_2	0.001	Alternating Occasionally
n_3	0	Negligible Obstructions
n_4	0.015	Small vegetation
m	1	Minor meandering
Selected n-value	0.043	

Right Over Bank	Selection	Description
n_b	0.026	Riprap Banks
n_1	0.003	Minor Irregularity
n_2	0.001	Negligible
n_3	0	Negligible
n_4	0.025	Limited to No Vegetation
m	1	Minor meandering
Selected n-value	0.055	

Table 10 Manning Roughness Coefficients (17455- 12078) and (1330-69).

Left Over Bank	Selection	Description
n_b	0.026	Unstable channel median grain size 1 millimeter
n_1	0.003	Minor Irregularity
n_2	0.001	Alternating Occasionally
n_3	0	Negligible Obstructions
n_4	0.025	Medium, scattered shrubs, heavy weeds
m	1	Minor meandering
Selected n-value	0.055	

Main Channel	Selection	Description
n_b	0.026	Unstable channel, median grain size 1 millimeter
n_1	0.001	Minor Irregularity
n_2	0.001	Alternating Occasionally
n_3	0	Negligible Obstructions
n_4	0.015	Small vegetation
m	1	Minor meandering
Selected n-value	0.043	

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Right Over Bank	Selection	Description
n_0	0.026	Riprap Banks
n_1	0.003	Minor Irregularity
n_2	0.001	Negligible
n_3	0	Negligible
n_4	0.025	Limited to No Vegetation
m	1	Minor meandering
Selected n-value	0.055	

As shown in Table 11, the Manning coefficients in Table 9 and Table 10 are consistent with investigations conducted by the United States Geological Survey (USGS) in the early 1970s (B.N. Aldridge and J.M Garret,1973), as well as those used in a previous study of the channel system that was completed within the study region (Tetra Tech, 2014).

Table 11 Comparison of Manning Roughness Coefficients

Study	Channel	Overbank
USGS 1973[1]	0.03-0.05	0.03-0.05
Previous Study[2]	0.033-0.064	0.05-0.07
This Study	0.038	0.043-0.046

1. Brawley Wash (B.N. Aldridge and J.M Garret,1973)
2. Range of values (Tetra Tech, 2014)

4.2.3 Flow Regime

A preliminary mixed-flow-regime model was executed with results indicating near critical flow conditions or transitional flow for each section. A rule of thumb for soft-bottom channels and alluvial fan hydraulics is that the Froude number should not exceed 1.0 (Grant, 1977). Therefore, subcritical flow conditions were selected to analyze the flow characteristics of the South Dump and TSF 6 & 7 Diversion Channels. This approach will provide a conservative baseline condition when comparing the impacts of the proposed improvements on flood depths, and is the typical standard used when modeling floodplains or encroachments. However, due to the transitional nature of the diversion channel flow, potentially high variations in velocities will need to be accounted for when designing erosion protection.

4.2.4 Steady-Flow Data

Four different drainage basins contribute to the total flow along the study reach including: (1) Mineral Hill watersheds (A and B) that are diverted south around the mine, (2) the South Dump Drainage Basin, (3) the TSF 6 & 7 Drainage Basin, and (4) the TSF 8 Drainage Basin. The drainage-basin boundaries are shown in Figure 2. In general, the active conveyance channels in the approaching watersheds do not have capacity to convey the larger, less-frequent events. Once the bankfull capacity of the washes are surpassed, flow extends in the overbank and is conveyed downstream as shallow sheet flow. Flow in the washes and the shallow overbank sheet flow is intercepted by the mine stormwater diversion channels, the accumulation of this flow is required to be managed. For conservative (safe) design flow purposes, in the RAS model the flow from each drainage basin is added to a channel cross-section at the upstream end where the mine intersects the initial contribution of each drainage basin.

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4.2.5 Boundary Conditions

The South Dump and TSF 6 & 7 Diversion Channels have a channel bed slope of approximately two percent. Based on the near-critical-flow nature of the diversion channel, selecting the critical depth downstream boundary condition for the model was believed to be valid for this analysis.

4.2.6 Results

Water-surface elevations, depths, and velocities for steady-state water-surface profiles were computed for the 2-year through 100-year events and are shown in Table 12 and Table 13, respectively.

Maximum channel depths for each profile are shown in Table 12 (bold values denote reach maximum) for rainfall events with recurrence intervals of 2, 5, 10, 25, 50 and 100 years. The maximum cross-section depths in the study reach range from 7.11 feet (2-year event) to 15.88 feet (100-year event).

The average channel velocities are shown in Table 13 (bold values denote reach maximum). The range of flow velocities for all runoff profiles is between 10.93 ft/sec and 21.02 ft/sec.

Table 12. Maximum Channel Depths

	HEC-RAS Channel Depth (ft)					
River Station (ft)	**2 Year**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**100 Year**
27991	4.03	5.58	6.69	7.92	8.79	9.63
27901	3.62	5.09	6.18	7.4	8.24	9.05
27822	3.23	4.68	5.77	7.01	7.86	8.65
27722	3.75	5.15	6.24	7.47	8.32	9.11
27591	2.89	3.87	4.37	5.18	5.89	6.69
27508	3.28	4.22	4.79	5.37	5.73	6.07
27330	1.9	2.25	2.47	2.75	3	3.27
27175	0.59	1.03	1.41	1.87	2.2	2.56
27087	1.14	1.65	2.05	2.5	2.82	3.15
26743	0.81	1.23	1.61	2.05	2.35	2.66
26456	1.17	1.59	1.87	2.18	2.41	2.67
26177	0.84	1.27	1.56	1.95	2.26	2.6
25876	0.96	1.52	1.97	2.5	2.9	3.31
25463	1.13	1.73	2.16	2.61	2.96	3.31
25131	3.7	4.71	5.35	5.98	6.35	6.72
24739	0.81	1.22	1.55	2.01	2.39	2.8
24418	1.32	1.92	2.42	3.03	3.47	3.92
24133	1.74	2.47	2.91	3.39	3.81	4.4
23737	2.22	3.44	4.35	5.47	6.35	7.3
23523	2.78	4.19	5.14	6.25	7.1	8.02
23238	2.29	3.37	4.3	5.42	6.27	7.15
22913	2.73	4.27	5.48	6.76	7.68	8.67
22507	1.79	3.05	4.1	5.38	6.35	7.33
22153	2.77	4.12	5.12	6.25	7.1	8.07

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River Station (ft)	HEC-RAS Channel Depth (ft)					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
21855	2.39	3.74	4.8	6	6.93	7.88
21582	2.83	4.63	6.16	7.87	9.07	10.32
21229	2.29	3.45	4.28	5.26	6.15	7.08
20936	1.64	2.6	3.45	4.44	5.23	6.13
20643	1.52	2.48	3.23	4.29	5.12	6.01
20250	1.44	1.99	2.47	3.13	3.62	4.07
19882	1	1.47	1.8	2.14	2.41	2.72
19436	1.94	2.63	3.14	3.71	4.13	4.55
18923	0.16	0.56	0.86	1.25	1.54	1.88
18559	0.91	1.25	1.53	1.86	2.13	2.41
18196	0.82	1.22	1.53	1.94	2.23	2.56
17935	0.96	1.25	1.47	1.74	1.96	2.22
17455	1.76	2.45	2.95	3.33	3.4	3.7
16945	2.05	3	3.74	4.53	5.15	5.87
16520	2.34	3.48	4.24	5.26	5.95	6.62
16294	1.79	2.69	3.4	4.22	4.84	5.49
15754	1.5	2.06	2.58	3.2	3.69	4.14
15289	1.77	2.64	3.23	3.89	4.4	4.86
14886	1.88	2.73	3.37	4.07	4.57	4.88
14212	1.27	1.94	2.47	3.09	3.58	4.27
13750	1.06	1.67	1.83	2.39	2.87	3.35
13197	2.39	3.84	5	5.63	6.07	6.47
12864	1.74	2.28	2.8	3.21	3.66	4
12525	1.69	2.72	3.57	4.87	5.48	6.57
12239	1.3	1.91	2.44	3.1	3.55	3.7
12078	1.37	2.14	2.77	3.54	4.14	5.02
11880	**7.11**	**8.63**	9.86	11.29	12.63	14.19
11627	4.91	6.17	7.2	8.4	9.28	10.07
11365	5.81	6.92	7.81	8.84	9.58	10.19
11123	1.16	1.73	2.21	2.84	3.32	4.14
10859	4.15	5.31	5.78	6.66	7.54	8.6
10700	1.75	3.04	3.52	4.75	5.76	6.88
10520	2.15	3.25	4.29	5.59	6.63	7.77
10372	2.75	3.96	5.03	6.31	7.33	8.45
10057	1.99	3.21	4.24	5.35	6.25	7.24
9795	1.95	3	3.8	4.67	5.24	5.8
9512	1.27	2.11	2.79	3.61	4.27	5.01
9268	2.51	4.04	5.32	6.85	7.99	9.23
9179	3.95	5.87	7.3	8.67	9.61	10.58

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River Station (ft)	HEC-RAS Channel Depth (ft)					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
9100	2.57	4.04	5.29	6.85	8.05	9.64
8922	3.28	4.77	6.05	7.47	8.39	9.55
8656	2.79	4.11	5	6.21	7.22	8.09
8414	2.97	4.29	5.19	6.28	7.06	7.93
8152	2.9	4.1	4.87	6.17	7.25	7.75
7997	4.08	6.81	8.07	9.13	9.95	10.77
7911	4.96	6.58	8.33	9.39	10.17	10.89
7736	4.67	7.97	9.01	9.97	10.71	11.48
7566	3.78	5.69	6.4	7.27	7.95	8.69
7455	2.93	4.39	5.29	6.38	7.11	7.9
7317	3.38	4.48	5.52	6.38	6.95	7.69
7054	2.44	3.45	4.2	5.23	6.04	6.97
6684	3	4.52	5.65	6.99	7.95	9
6457	2.86	4.76	6.37	8.55	10.03	12.38
6223	2.57	3.95	4.62	5.89	6.92	8.5
5998	2.7	3.94	5.55	6.43	7.06	7.84
5842	3.55	5.21	6.01	7	7.55	8.72
5734	2.85	3.58	4.18	5.24	5.84	6.52
5631	2.39	4.16	5.39	6.28	7.17	8.31
5342	2.07	3.09	4.1	5.54	4.81	5.24
4967	1.5	2.33	3.06	3.92	4.59	5.3
4555	1.92	2.98	3.74	4.64	5.36	5.92
4263	2.17	3.36	4.41	5.57	6.54	7.75
4008	1.79	2.97	3.91	5.07	5.96	7.15
3672	2.84	4.64	5.56	7.24	7.91	8.57
3333	3.9	6.35	8.64	9.57	10.23	10.91
3121	4.26	6.27	7.77	10.54	11.56	12.39
2820	4.88	8.06	10.49	12.28	12.89	15.03
2522	5.29	8.06	**10.91**	**12.95**	**14.75**	**15.88**
2192	4.13	6.78	8.92	10.89	12.11	13.05
1864	5.46	7.47	8.27	9.52	10.76	12.58
1584	3.64	5.61	6.99	8.04	8.74	8.74
1330	3.59	5.3	6.15	7.14	7.92	8.44
907	3.51	5.63	7.4	8.78	9.01	9.01
613	2.3	3.76	4.8	6.5	7.47	8.52
355	2.61	4.2	5.9	6.68	7.14	7.62
171	2.06	3.31	4.52	5.72	6.17	6.64
69	1.83	3.09	4.76	5.5	5.95	6.42

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Table 13. Average Channel Velocities

River Station (ft)	HEC-RAS Channel Velocities (ft/sec)					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
27991	3.94	5.19	6.05	6.98	7.67	8.42
27901	3.91	5.1	5.88	6.7	7.32	8.02
27822	3.88	4.89	5.53	6.18	6.71	7.34
27722	3.71	4.83	5.48	6.13	6.62	7.19
27591	4.42	6.41	8.46	9.99	10.67	11.18
27508	3.31	4.57	5.5	6.72	7.78	9.02
27330	3.06	4.15	5.05	6.02	6.56	7.06
27175	2.94	3.57	4.04	4.6	5.06	5.53
27087	2	2.75	3.32	3.97	4.5	5.06
26743	3.61	4.54	4.92	5.35	5.76	6.24
26456	2.22	3.06	3.84	4.8	5.5	6.23
26177	3.42	3.79	4.32	4.87	5.26	5.68
25876	2.24	3.02	3.58	4.19	4.63	5.09
25463	4.98	5.77	6.28	7.04	7.53	8.11
25131	5.04	6.19	6.84	7.7	8.6	9.52
24739	2.49	3.67	4.44	5.14	5.55	5.94
24418	1.66	2.04	2.36	2.73	3.04	3.39
24133	3.66	4.61	5.54	6.7	7.24	7.29
23737	3.91	4.81	5.14	5.24	5.24	5.24
23523	4.15	5.06	5.86	6.76	7.39	7.92
23238	6.66	8.67	9.74	10.83	11.51	12.23
22913	7.94	9.57	10.33	11.29	12.02	12.61
22507	5.4	6.47	7.09	7.71	8.11	8.57
22153	6.23	8.16	9.53	10.99	11.95	12.67
21855	7.41	9.02	10.02	11.13	11.78	12.45
21582	8.65	10.71	11.64	12.57	13.24	13.84
21229	5.64	7.67	9.29	11.02	11.77	12.5
20936	5.07	6.77	7.71	8.81	9.45	9.97
20643	6.04	8.03	9.6	10.94	11.88	12.83
20250	4.32	6.32	7.47	8.33	8.87	9.67
19882	4.67	5.85	6.59	7.71	8.51	9.22
19436	3.26	4.7	5.76	7.01	7.89	8.81
18923	1.05	2.45	3.71	4.94	5.78	6.58
18559	3.08	4.35	5.29	6.46	7.29	8.17
18196	4.1	5.15	5.95	6.66	7.2	7.71
17935	4.14	5.28	6.25	7.32	8.01	8.62
17455	5.06	6.24	6.94	6.41	8.19	9.18
16945	6.3	7.54	8.28	9.26	9.85	10.22

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River Station (ft)	HEC-RAS Channel Velocities (ft/sec)					
16520	6.62	7.84	8.88	9.39	9.78	10.21
16294	6.08	7.47	8.24	9.07	9.51	9.84
15754	4.81	6.56	7.46	8.39	9	9.82
15289	5.96	6.91	7.79	8.79	9.33	10.11
14886	4.08	5.33	6.21	7.23	7.97	9.35
14212	5.3	6.53	7.31	8.12	8.58	8.26
13750	3.69	4.92	7.1	8.1	8.56	9.12
13197	7.64	9.16	8	9.04	9.75	10.62
12864	5.37	6.79	7.35	9.06	9.78	11.11
12525	6.61	8.34	9.45	9.43	10.25	9.41
12239	3.69	5.01	5.9	6.9	7.77	9.62
12078	5.71	7.17	8.09	9	9.58	9.18
11880	3.39	5.82	7.62	9.66	9.21	7.65
11627	2.44	4.27	5.68	7.35	8.59	10.03
11365	1.71	3.12	4.26	5.64	6.68	7.96
11123	4.97	6.45	7.5	8.59	9.36	9.27
10859	1.27	2.06	2.93	3.76	4.19	4.18
10700	6.1	4.92	4.47	3.24	2.97	2.86
10520	1.77	1.96	2.09	2.3	2.45	2.61
10372	2.21	2.78	3.17	3.65	3.99	4.31
10057	5.07	6.59	7.45	8.55	9.26	9.92
9795	4.2	5.95	7.3	9.01	10.44	12
9512	5.96	7.54	8.62	9.85	10.68	11.42
9268	8.06	10.12	11.4	12.74	13.7	14.52
9179	8.9	10.49	11.17	12.57	13.72	14.9
9100	7.94	10.22	11.72	13.26	14.31	14.58
8922	5.76	8.02	9.39	11.11	12.62	13.77
8656	7.14	9.42	11.56	13.58	14.81	16.63
8414	6.82	8.75	10.1	11.52	12.69	13.77
8152	7.8	9.53	11.28	11.62	12.15	13.39
7997	9.95	9.95	10.67	12.4	13.57	14.85
7911	7.77	11.07	10.49	12.12	13.27	14.73
7736	10.81	9.62	10.83	12.67	13.86	15.07
7566	**10.93**	9.5	11.15	12.79	13.88	14.88
7455	8.04	9.72	11.06	12.36	13.51	14.58
7317	8.01	10.22	10.93	12.47	13.88	14.95
7054	6.2	8.07	9.67	11.19	12.27	13.26
6684	7.91	9.42	10.44	11.36	12.05	12.59
6457	8.9	11.34	12.89	13.78	14.5	12.67
6223	6.37	8.08	10.56	11.99	12.87	12.55

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River Station (ft)	HEC-RAS Channel Velocities (ft/sec)					
5998	8.42	11.2	10.34	12.05	13.25	14.33
5842	8.58	9.56	11.28	13.08	15.24	15.45
5734	7.13	9.13	10.44	10.6	11.8	12.9
5631	8.24	9.23	9.68	11.37	11.93	12.09
5342	6.72	9.37	10.61	11.13	**18.64**	**21.02**
4967	5.89	7.8	8.9	10.28	11.25	12.33
4555	6.06	7.36	8.59	9.8	10.56	11.96
4263	6.85	9.1	10.11	11.42	12.14	12.1
4008	7.05	8.73	9.83	10.91	11.51	11.29
3672	7.85	8.85	10.76	10.13	11.04	12.06
3333	10.08	12.35	9.97	11.52	12.69	13.91
3121	9.23	10.91	12.25	11.2	11.75	12.78
2820	10.76	12.74	12.72	**14.43**	17.36	15.55
2522	10.53	**13.85**	**13.06**	14.3	13.8	14.12
2192	10.22	12.26	12.88	14.25	15.69	17.91
1864	6.81	8.52	10.79	12.02	12.02	11.01
1584	9.16	11.04	10.76	12.66	14.56	18.91
1330	7.62	8.92	10.28	11.39	11.71	12.88
907	9.08	10.67	10.32	10.75	4.73	6.15
613	7.03	9.2	11.05	11.17	11.77	12.02
355	8.25	10.56	8.54	9.47	10.47	11.49
171	7.27	9.37	10.24	9.02	10.01	10.96
69	7.26	9.26	8.01	8.99	9.91	10.88

4.3 Proposed Conditions

In order to re-establish the stormwater controls along the south side of the Mission Complex, Tetra Tech has developed a proposed grading plan that, once constructed, will safely convey the 10-year flows around the project site. An overview of the proposed improvements relative to the hydraulic model cross-sections are shown on Exhibit 3.

4.4 Channel Confluence Design

The existing project site serves as a collector channel which diverts offsite stormwater around the Mission Complex. Confluences occur as the diversion channel intercepts tributary watersheds. At the project site many of the confluences occur as shallow overland flow; however, at a few locations the primary channel of the tributary feeds directly into the existing diversion channel. Where this occurs, it can cause both vertical and horizontal erosion. For the re-establishment design, where confluences are being re-graded it is recommended that the junction angle, as measured between the centerline of the approach flows with the origin at the confluence, be less than 12 degrees and adequately armored (COT, 1989).

RCC-CW010042

4.5 Reconstructed Basin Crest

Under existing conditions, the stormwater enters a wide and shallow basin immediately upstream of TSF 6 & 7. This basin has experienced degradation at the downstream extents, virtually obliterating the channel and outlet crest. This has historically compromised underground infrastructure in the area connecting TSF 6 & 7 with TSF 8, located south of Helmet Peak Road. A rigid basin outlet crest is necessary to provide grade stabilization in the area. This feature will need to have adequate toe-down and revetments to reduce the failure potential during high flow events. This crest was modeled as an inline structure in HEC-RAS at River Station 10120. While the invert of the low flow channel will remain at an elevation of 3115, the crest will have an elevation of 3123 feet. The proposed embankment slopes will be approximately 2.5H:1V (Horizontal:Vertical - upstream), and 6H:1V (downstream). For convenience, a maintenance ramp is proposed to be located at the right-hand side of the spillway looking downstream. Along the spillway, the shape of the rigid (revetted) crest is designed to be trapezoidal in shape, with a top width of 65 feet and bottom width of approximately 32 feet. Proposed dimension are intended to accommodate the 10-year design flow to flow through the existing basin un-impeded. Note, there may be some attenuation through the existing basin however, for conservative reasons, this was not accounted for in the steady-state modeling conducted.

4.6 Split Flow

Downstream of TSF 6 & 7, the existing channel currently flows in a northeasterly direction and outlets immediately adjacent to the Rancho Resort Residential community. In order to help alleviate the direct impingement of flow on this community, a portion of the flow contained within the primary diversion channel is proposed to be laterally split off from the main channel to lower the water-surface elevations at the Rancho Resort community. This split flow is proposed to be conveyed to the Rancho Resort collector channel system, located further south, via a new wide and shallow channel.

Within the proposed conditions model, a split reach was added to simulate a distributary flow at river station 1800. The designed split reach lies to the east side of the main reach, and the length was roughly about 1850 feet long. The split-reach cross-sectional geometry was developed using the available topography. The main-stream flow and the spilt-flow discharges at the joint cross section were calculated using the Manning equation, and the computed discharges were then used as input into the HEC-RAS model.

4.6.1 Boundary Conditions

Based on the near-critical-flow nature of both the main reach and diversion reach, selecting the critical depth downstream boundary condition for the model was believed to be valid for this analysis.

4.7 Comparison of Existing and Proposed Conditions

Water-surface elevations (WSELs), depths, and velocities for steady-state water-surface profiles were computed for the 2-year through 100-year events. The 10-year and 100-year-profile results were selected for the comparison of existing and proposed conditions and are shown on Exhibit 4.

The WSEL comparisons of the 10-year profile are shown in Table 14. From the results, the WSELs at the downstream end significantly decreased due to the proposed split-flow reach. The WSEL comparisons of the 100-year profile are shown in Table 15.

RCC-CW010043

Table 14. Comparisons of 10-Year Water-Surface Elevations

10-YR	*WSEL (ft)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft)*
27991	3422.69	3422.69	0
27901	3422.18	3422.18	0
27822	3421.77	3421.77	0
27722	3421.24	3421.24	0
27591	3419.37	3419.37	0
27508	3419.01	3419.01	0
27330	3416.47	3416.46	-0.01
27175	3414.23	3414.23	0
27087	3413.67	3413.7	0.03
26743	3410.36	3410.24	-0.12
26456	3406.87	3407.43	0.56
26177	3403.53	3403.53	0
25876	3400.97	3401.38	0.41
25463	3396.24	3397	0.76
25131	3385.35	3385.91	0.56
24739	3381.55	3381.8	0.25
24418	3380.42	3381.2	0.78
24133	3378.74	3380.45	1.71
23737	3376.35	3378.16	1.81
23523	3375.14	3375.37	0.23
23238	3371.3	3370.54	-0.76
22913	3364.48	3363.39	-1.09
22507	3357.1	3357.06	-0.04
22153	3352.12	3352.12	0
21855	3346.8	3346.8	0
21582	3340.16	3340.16	0
21229	3331.28	3331.28	0
20936	3327.34	3327.34	0
20643	3322.07	3322.07	0
20250	3314.47	3312.98	-1.49
19882	3307.1	3303.45	-3.65
19436	3294.14	3291.97	-2.17
18923	3287.67	3282.76	-4.91
18559	3280.53	3276.95	-3.58
18196	3272.65	3270.52	-2.13
17935	3265.37	3265.44	0.07
17455	3252.98	3252.98	0

RCC-CW010044

Table 14. Comparisons of 10-Year Water-Surface Elevations

10-YR	*WSEL (ft)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft)*
16945	3240.74	3240.74	0
16520	3230.24	3230.24	0
16294	3224.97	3224.97	0
15754	3211.58	3211.58	0
15289	3200.23	3200.23	0
14886	3192.37	3192.37	0
14212	3181.34	3181.34	0
13750	3170.42	3170.42	0
13197	3162	3162	0
12864	3154.7	3154.7	0
12525	3148.5	3148.5	0
12239	3142.22	3142.21	-0.01
12078	3139.16	3139.17	0.01
11880	3125.86	3125.27	-0.59
11627	3125.2	3123.94	-1.26
11365	3124.81	3122.52	-2.29
11123	3123.21	3122.94	-0.27
10859	3122.15	3122.94	0.79
10700	3121.52	3122.94	1.42
10520	3121.29	3122.93	1.64
10372	3121.03	3122.91	1.88
10057	3119.24	3119.43	0.19
9795	3116.68	3116.68	0
9512	3112.57	3112.57	0
9268	3102.32	3102.32	0
9179	3100.7	3100.7	0
9100	3090.72	3090.72	0
8922	3088.62	3088.67	0.05
8656	3085	3084.95	-0.05
8414	3081.64	3080.94	-0.7
8152	3076.87	3074.38	-2.49
7997	3073.07	3068.32	-4.75
7911	3070.33	3065.3	-5.03
7736	3067.5	3061.12	-6.38
7566	3061.4	3056.56	-4.84
7455	3057.29	3053.64	-3.65
7317	3054.52	3051.75	-2.77

RCC-CW010045

Table 14. Comparisons of 10-Year Water-Surface Elevations			
10-YR	*WSEL (ft)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft)*
7054	3048.2	3047.33	-0.87
6684	3040.88	3040.88	0
6457	3035.16	3035.16	0
6223	3029.25	3029.25	0
5998	3025.55	3025.54	-0.01
5842	3021.67	3021.55	-0.12
5734	3017.18	3017.17	-0.01
5631	3015.39	3015.38	-0.01
5342	3007.67	3007.69	0.02
4967	2999.56	2999.53	-0.03
4555	2990.85	2990.9	0.05
4263	2985.41	2985.41	0
4008	2979.91	2979.91	0
3672	2972.56	2972.58	0.02
3333	2966.64	2966.74	0.1
3121	2957.77	2957.71	-0.06
2820	2950.49	2950.07	-0.42
2522	2943.91	2943.54	-0.37
2192	2935.92	2935.7	-0.22
1864	2930.27	2926.12	-4.15
1584	2926.99	2923.77	-3.22
1330	2922.15	2920.82	-1.33
907	2916.4	2913.98	-2.42
613	2908.8	2907.3	-1.5
355	2904.48	2902.3	-2.18
171	2899.76	2898.09	-1.67
69	2897.76	2895.69	-2.07

Table 15. Comparisons of 100-Year Water-Surface Elevations			
100-YR	**WSEL (ft)**		**Difference (Proposed-Existing)**
Cross Section	**Existing**	**Proposed**	**(ft)**
27991	3425.63	3425.63	0
27901	3425.05	3425.05	0
27822	3424.65	3424.65	0
27722	3424.11	3424.11	0
27591	3421.69	3421.69	0

RCC-CW010046

Table 15. Comparisons of 100-Year Water-Surface Elevations

100-YR	WSEL (ft)		Difference (Proposed-Existing)
Cross Section	Existing	Proposed	(ft)
27508	3420.29	3420.29	0
27330	3417.27	3417.25	-0.02
27175	3415.38	3415.42	0.04
27087	3414.77	3414.87	0.1
26743	3411.41	3411.37	-0.04
26456	3407.67	3408.46	0.79
26177	3404.57	3404.86	0.29
25876	3402.31	3403	0.69
25463	3397.39	3398.35	0.96
25131	3386.72	3387.03	0.31
24739	3382.8	3383.51	0.71
24418	3381.92	3383	1.08
24133	3380.23	3382.14	1.91
23737	3379.3	3381.05	1.75
23523	3378.02	3378.08	0.06
23238	3374.15	3373.53	-0.62
22913	3367.67	3366.47	-1.2
22507	3360.33	3360.3	-0.03
22153	3355.07	3355.07	0
21855	3349.88	3349.88	0
21582	3344.32	3344.32	0
21229	3334.08	3334.08	0
20936	3330.02	3330.02	0
20643	3324.85	3324.85	0
20250	3316.07	3315.21	-0.86
19882	3308.02	3306.36	-1.66
19436	3295.55	3294.05	-1.5
18923	3288.69	3285.2	-3.49
18559	3281.41	3279.39	-2.02
18196	3273.68	3273.13	-0.55
17935	3266.12	3266.19	0.07
17455	3253.73	3253.73	0
16945	3242.87	3242.87	0
16520	3232.62	3232.62	0
16294	3227.06	3227.06	0
15754	3213.14	3213.14	0
15289	3201.86	3201.86	0

RCC-CW010047

Table 15. Comparisons of 100-Year Water-Surface Elevations

100-YR	WSEL (ft)		Difference (Proposed-Existing)
Cross Section	Existing	Proposed	(ft)
14886	3193.88	3193.88	0
14212	3183.14	3183.14	0
13750	3171.94	3171.94	0
13197	3163.47	3163.47	0
12864	3155.9	3155.9	0
12525	3151.5	3151.5	0
12239	3143.48	3143.48	0
12078	3141.41	3141.41	0
11880	3130.19	3130.12	-0.07
11627	3128.07	3127.8	-0.27
11365	3127.19	3124.11	-3.08
11123	3125.14	3125.47	0.33
10859	3124.97	3125.48	0.51
10700	3124.88	3125.47	0.59
10520	3124.77	3125.43	0.66
10372	3124.45	3125.36	0.91
10057	3122.24	3122.18	-0.06
9795	3118.68	3118.68	0
9512	3114.79	3114.79	0
9268	3106.23	3106.23	0
9179	3103.98	3103.98	0
9100	3095.07	3095.07	0
8922	3092.12	3092.12	0
8656	3088.09	3088.09	0
8414	3084.38	3083.5	-0.88
8152	3079.75	3077.06	-2.69
7997	3075.77	3070.8	-4.97
7911	3072.89	3067.89	-5
7736	3069.97	3063.64	-6.33
7566	3063.69	3059.07	-4.62
7455	3059.9	3056.5	-3.4
7317	3056.69	3054.5	-2.19
7054	3050.97	3049.74	-1.23
6684	3044.23	3044.23	0
6457	3041.17	3041.17	0
6223	3033.13	3033.13	0
5998	3027.84	3027.83	-0.01

RCC-CW010048

Table 15. Comparisons of 100-Year Water-Surface Elevations

100-YR	WSEL (ft)		Difference (Proposed-Existing)
Cross Section	Existing	Proposed	(ft)
5842	3024.38	3024.18	-0.2
5734	3019.52	3019.65	0.13
5631	3018.31	3017.9	-0.41
5342	3008.81	3009.01	0.2
4967	3001.8	3001.67	-0.13
4555	2993.03	2993.07	0.04
4263	2988.75	2988.75	0
4008	2983.15	2983.06	-0.09
3672	2975.57	2975.6	0.03
3333	2968.91	2969.29	0.38
3121	2962.39	2962.36	-0.03
2820	2955.03	2954.48	-0.55
2522	2948.88	2948.01	-0.87
2192	2940.05	2940	-0.05
1864	2934.58	2928.7	-5.88
1584	2928.74	2925.8	-2.94
1330	2924.44	2922.88	-1.56
907	2918.01	2917.43	-0.58
613	2912.52	2910.15	-2.37
355	2906.2	2905.13	-1.07
171	2901.88	2900.76	-1.12
69	2899.42	2898.33	-1.09

The flow velocities in the channel were also compared for the 10-year and 100-year events. Table 16 shows the comparisons of the 10-year channel velocities; while Table 17 shows the comparisons of the 100-year channel velocities.

Table 16. Comparisons of 10-Year Channel Velocities

10-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
27991	6.05	6.05	0
27901	5.88	5.88	0
27822	5.53	5.53	0
27722	5.48	5.48	0
27591	8.46	8.46	0
27508	5.5	5.49	-0.01
27330	5.05	5.24	0.19

RCC-CW010049

Table 16. Comparisons of 10-Year Channel Velocities

10-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
27175	4.04	4.04	0
27087	3.32	3.23	-0.09
26743	4.92	5.62	0.7
26456	3.84	3.86	0.02
26177	4.32	4.1	-0.22
25876	3.58	4.58	1
25463	6.28	7.22	0.94
25131	6.84	6.13	-0.71
24739	4.44	4.71	0.27
24418	2.36	2.2	-0.16
24133	5.54	4.36	-1.18
23737	5.14	4.78	-0.36
23523	5.86	8.37	2.51
23238	9.74	9.92	0.18
22913	10.33	10.15	-0.18
22507	7.09	7.05	-0.04
22153	9.53	9.53	0
21855	10.02	10.02	0
21582	11.64	11.64	0
21229	9.29	9.29	0
20936	7.71	7.71	0
20643	9.6	9.6	0
20250	7.47	8.53	1.06
19882	6.59	9.11	2.52
19436	5.76	8.32	2.56
18923	3.71	8.96	5.25
18559	5.29	8.06	2.77
18196	5.95	9.11	3.16
17935	6.25	7.08	0.83
17455	6.94	6.94	0
16945	8.28	8.28	0
16520	8.88	8.88	0
16294	8.24	8.24	0
15754	7.46	7.46	0
15289	7.79	7.79	0
14886	6.21	6.21	0
14212	7.31	7.31	0

RCC-CW010050

Table 16. Comparisons of 10-Year Channel Velocities

10-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
13750	7.1	7.1	0
13197	8	8	0
12864	7.35	7.35	0
12525	9.45	9.46	0.01
12239	5.9	5.91	0.01
12078	8.09	8.05	-0.04
11880	7.62	8.39	0.77
11627	5.68	7.19	1.51
11365	4.26	6.74	2.48
11123	7.5	1.12	-6.38
10859	2.93	0.52	-2.41
10700	4.47	0.56	-3.91
10520	2.09	0.83	-1.26
10372	3.17	1.18	-1.99
10057	7.45	8.94	1.49
9795	7.3	7.3	0
9512	8.62	8.62	0
9268	11.4	11.4	0
9179	11.17	11.17	0
9100	11.72	11.72	0
8922	9.39	9.29	-0.1
8656	11.56	11.73	0.17
8414	10.1	9.36	-0.74
8152	11.28	9.55	-1.73
7997	10.67	9.17	-1.5
7911	10.49	9.36	-1.13
7736	10.83	9.28	-1.55
7566	11.15	9.23	-1.92
7455	11.06	5.03	-6.03
7317	10.93	9.04	-1.89
7054	9.67	8.61	-1.06
6684	10.44	10.44	0
6457	12.89	12.89	0
6223	10.56	10.56	0
5998	10.34	10.34	0
5842	11.28	10.7	-0.58
5734	10.44	10.44	0

RCC-CW010051

Table 16. Comparisons of 10-Year Channel Velocities

10-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
5631	9.68	9.66	-0.02
5342	10.61	10.54	-0.07
4967	8.9	8.55	-0.35
4555	8.59	8.04	-0.55
4263	10.11	10.11	0
4008	9.83	9.78	-0.05
3672	10.76	10.67	-0.09
3333	9.97	10.19	0.22
3121	12.25	12.27	0.02
2820	12.72	11.32	-1.4
2522	13.06	11.55	-1.51
2192	12.88	12.19	-0.69
1864	10.79	7.84	-2.95
1584	10.76	6.06	-4.7
1330	10.28	8.85	-1.43
907	10.32	10.49	0.17
613	11.05	8.67	-2.38
355	8.54	9.95	1.41
171	10.24	8.89	-1.35
69	8.01	8.78	0.77

RCC-CW010052

Table 17. Comparisons of 100-Year Channel Velocities

100-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
27991	8.42	8.42	0
27901	8.02	8.02	0
27822	7.34	7.34	0
27722	7.19	7.19	0
27591	11.18	11.18	0
27508	9.02	9.02	0
27330	7.06	7.45	0.39
27175	5.53	5.42	-0.11
27087	5.06	4.81	-0.25
26743	6.24	7.16	0.92
26456	6.23	6.41	0.18
26177	5.68	6.1	0.42
25876	5.09	6.52	1.43
25463	8.11	9.79	1.68
25131	9.52	9.43	-0.09
24739	5.94	5.65	-0.29
24418	3.39	3.49	0.1
24133	7.29	5.85	-1.44
23737	5.24	4.66	-0.58
23523	7.92	11.6	3.68
23238	12.23	12.26	0.03
22913	12.61	12.86	0.25
22507	8.57	8.54	-0.03
22153	12.67	12.67	0
21855	12.45	12.45	0
21582	13.84	13.84	0
21229	12.5	12.5	0
20936	9.97	9.97	0
20643	12.83	12.83	0
20250	9.67	10.79	1.12
19882	9.22	10.46	1.24
19436	8.81	10.95	2.14
18923	6.58	11.77	5.19
18559	8.17	10.86	2.69
18196	7.71	9.58	1.87
17935	8.62	9.97	1.35

RCC-CW010053

Table 17. Comparisons of 100-Year Channel Velocities

100-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
17455	9.18	9.18	0
16945	10.22	10.22	0
16520	10.21	10.21	0
16294	9.84	9.84	0
15754	9.82	9.82	0
15289	10.11	10.11	0
14886	9.35	9.35	0
14212	8.26	8.26	0
13750	9.12	9.12	0
13197	10.62	10.62	0
12864	11.11	11.11	0
12525	9.41	9.41	0
12239	9.62	9.62	0
12078	9.18	9.18	0
11880	7.65	7.79	0.14
11627	10.03	10.4	0.37
11365	7.96	13.07	5.11
11123	9.27	2.22	-7.05
10859	4.18	1.05	-3.13
10700	2.86	1.12	-1.74
10520	2.61	1.67	-0.94
10372	4.31	2.35	-1.96
10057	9.92	12.36	2.44
9795	12	12	0
9512	11.42	11.42	0
9268	14.52	14.52	0
9179	14.9	14.9	0
9100	14.58	14.58	0
8922	13.77	13.77	0
8656	16.63	16.63	0
8414	13.77	12.67	-1.1
8152	13.39	12.95	-0.44
7997	14.85	12.72	-2.13
7911	14.73	12.83	-1.9
7736	15.07	12.88	-2.19
7566	14.88	12.74	-2.14
7455	14.58	7.79	-6.79

RCC-CW010054

Table 17. Comparisons of 100-Year Channel Velocities

100-YR	*Velocity (ft/s)*		*Difference (Proposed-Existing)*
Cross Section	*Existing*	*Proposed*	*(ft/s)*
7317	14.95	11.13	-3.82
7054	13.26	13.26	0
6684	12.59	12.59	0
6457	12.67	12.67	0
6223	12.55	12.55	0
5998	14.33	14.52	0.19
5842	15.45	14.85	-0.6
5734	12.9	12.5	-0.4
5631	12.09	13.09	1
5342	21.02	19.76	-1.26
4967	12.33	11.78	-0.55
4555	11.96	11.3	-0.66
4263	12.1	12.1	0
4008	11.29	11.07	-0.22
3672	12.06	12.26	0.2
3333	13.91	13.81	-0.1
3121	12.78	12.74	-0.04
2820	15.55	12.59	-2.96
2522	14.12	12.37	-1.75
2192	17.91	15.45	-2.46
1864	11.01	11.29	0.28
1584	18.91	10.08	-8.83
1330	12.88	10.93	-1.95
907	6.15	10.53	-1.62
613	12.02	9.9	-2.12
355	11.49	9.28	-2.21
171	10.96	8.61	-2.35
69	10.88	8.57	-2.31

5.0 SEDIMENT-CONTINUITY ANALYSIS

The Mission Complex diversion channels are relatively steep washes capable of transporting sands, gravels, cobbles, and debris during a typical flood event.

A cursory sediment-continuity evaluation was conducted to semi-quantitatively identify what the trends in bed-profile changes along the study reach could be expected.

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5.1 Sediment Gradations

Experience has dictated that for similar washes in Pima County a good assumption for the median particle diameter is approximately 2 millimeters (D50 = 2 mm) with a Gradation Coefficient, G, of 5. For this study, since no data regarding sediment gradations was available, this assumption was considered valid for a cursory sediment-continuity evaluation.

5.2 Transport Function Selection and Rating Curve Development

The sediment-transport equation selected was one developed by Zeller and Fullerton in 1986 for application to sand bed channels in arid regions, and particularly in southern Arizona. This sediment-transport equation was selected because of the straightforward applicability and capability of the user to determine quantitative values of the hydraulic variables in the equation.

The Zeller-Fullerton Sediment-Transport Equation is:

$$Q_s = \frac{0.0064 n^{1.77} V^{4.32} G^{0.45}}{Y_h^{0.30} D_{50}^{0.61}} W_e$$

Where,

- Q_s = Unbulked sediment transport, in cubic feet per second (cfs)
- n = Manning n-value
- V = Channel velocity, in feet/second
- G = Gradation coefficient = $1/2(D_{50}/D_{16}) + 1/2(D_{84}/D_{50})$
- Y_h = Hydraulic depth of flow, in feet
- D_{50} = Median diameter of sediments, in millimeters (by weight)
- W_e = Effective width of flow, in feet.

5.3 Sediment-Continuity Volumes

The project was separated into 12 sub-reaches based on similarity of hydraulic properties (Table 18). In order to evaluate the sediment balance, the sediment rating curves for each sub-reach was multiplied by ordinates of the event hydrographs in order to determine a total volume of sediment transported for each time step.

Table 18 – Channel Sub-Reaches

Sub-Reach	From HEC-RAS Cross-Section:	To HEC-RAS Cross-Section:
1	27508	27991
2	24133	27330
3	20250	23737
4	18196	19882
5 and 6	12525	17938
7 and 8	9795	12239
9	6223	9512
10	4008	5998
11	2192	3672
12	69	1864

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The upstream supply is generally taken as the upstream supply to the entire project reach. Summing the ordinates can provide the total sediment transported over the selected event. Each reach could then be compared reach to reach. If the total supply (upstream sub-reach) is greater than the transport capacity of the sub-reach of interest, the sub-reach is classified as aggradational. However, if the total supply is less than the transport capacity, the sub-reach is classified as degradational. Typically, this is routed by size fraction; however, without a valid gradation this analysis was completed on a net basis without regard for size class of sediment. This limitation was considered adequate for the level of detail of this study.

5.4 Sediment-Continuity Results

Based on this limited detail evaluation, an approximate average aggradation/degradation depth for each sub-reach can be computed by dividing the aggradation/degradation volume by the sub-reach length times its average width. For the purposes of this study, sub-reaches 5 and 6 and sub-reaches 7 and 8 were combined based on their short sub-reach lengths and similar hydraulics. For this analysis, a sediment-continuity evaluation was conducted for the design event of a 10-year flood, and results are summarized in Table 17, as shown below:

Table 19 –Continuity Analysis Results 10-Year Event

Sub-Reach	Continuity Analysis Capacity Loads (tons/event)	Continuity Analysis Aggradation/Degradation[1,2] (feet)
1	1,492	0
2	2,478	-0.06
3	9,692	-1.59
4	7,358	0.3
5 and 6	31,149	-0.78
7 and 8	1,881	3.84
9	26,433	-4.66
10	27,460	-0.24
11	27,516	-0.03
12	25,001	0.59

[1]Assumed maximum bulked sediment unit weight = 130 lb/cu. ft. (USDA, 1983)
[2]Note that assuming a minimum bulked sediment unit weigh would have the effect of amplifying the aggradation/degradation amounts but would not change the relative trend.

The results show that Sub-reach 7 and 8 could continue to aggrade while the system itself appears primarily degradational. This is consistent with the location of the existing basin located upstream of TSF 6 & 7. Historical observations, as described by Mission Complex staff, suggest that this basin area is routinely maintained due to the accumulation of sediments.

One limitation of the sediment-continuity approach is that static hydraulic data based on the existing-conditions hydraulics was used as the basis for analysis. Since the procedure does not account for adjustments in hydraulics and bed-material gradation as the channel responds to aggradation and degradation during passage of a flow event, the analysis is mainly useful for determining approximate values and trends, as opposed to the actual magnitude of channel bed-elevation adjustments. The aggradation and degradation response determined from a sediment-continuity analysis often over-predicts the actual channel response to the passage of a flow event.

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This sediment-continuity analysis primarily identifies a few locations, specifically those downstream of Sub-reaches 7 and 8, where a trend of degradation could compromise the toes of TSF 6 & 7, in addition to any underground infrastructure located within this reach of the Project.

6.0 SINGLE-EVENT SCOUR ANALYSIS

Single-event scour is typically computed as the sum of general scour, bed-form depth, low-flow channel incision, local scour, and bend scour. The following subsections describe the estimation of each of these components of single-event scour as applied to the design flow (10-year event) and the regulatory flow (100-year event).

6.1 General Scour

The general scour estimated by the procedure in this section is localized and is a temporary form of channel bed degradation that typically occurs during passage of a single flood event. The equation is intended to provide an upper limit on this potential form of scour. It does not represent degradation that can occur over the long term as a result of a continued sediment deficit, such as would occur downstream of a dam. It is reasonable to assume that general scour processes would be strongly correlated to the unit discharge of flow within the channel of an alluvial watercourse. Therefore, for purposes of this document General Scour, Z_{gs}, has been quantified in terms of unit discharge, q, as shown below (Tetra Tech, 2011):

$$Z_{gs} = 0.293q^{2/3}\left[q^{1/15} - 1.073\right]$$

Where:

Z_{gs} = General scour depth, in feet;
q = Average peak discharge per unit width of the channel, in cfs/ft.

It is noted that when general scour is computed to be less than 0.1 feet, by default it should be assumed that the general scour component is equal to 0.1 (i.e., if $Z_{gs} < 0.10$ feet, assume $Z_{gs} = 0.10$ feet).

6.2 Bed-Form Depth

For the purposes of evaluating an upper envelope for temporary scour that can occur during the passage of flood flows, differentials in streambed gradient associated with channel bed formations were considered. Bed forms are a second type of scour that can occur in sand-bed channels during a flood event. In order to evaluate the maximum streambed changes during the passage of a single flood event, two main bed forms, dunes or anti-dunes, were considered. In general, dunes typically form in lower regime flows (highly subcritical), and anti-dunes develop in upper regime flows (at or near critical). Essential to properly characterizing the single-event scour, a determination of upper or lower flow regime was made, using applicable charts (ASCE 2006).

The bed-form scour component in upper regime flow is customarily considered to be one-half the anti-dune height, from crest to trough as shown in the diagram below, but cannot exceed 1/2 the depth of flow.

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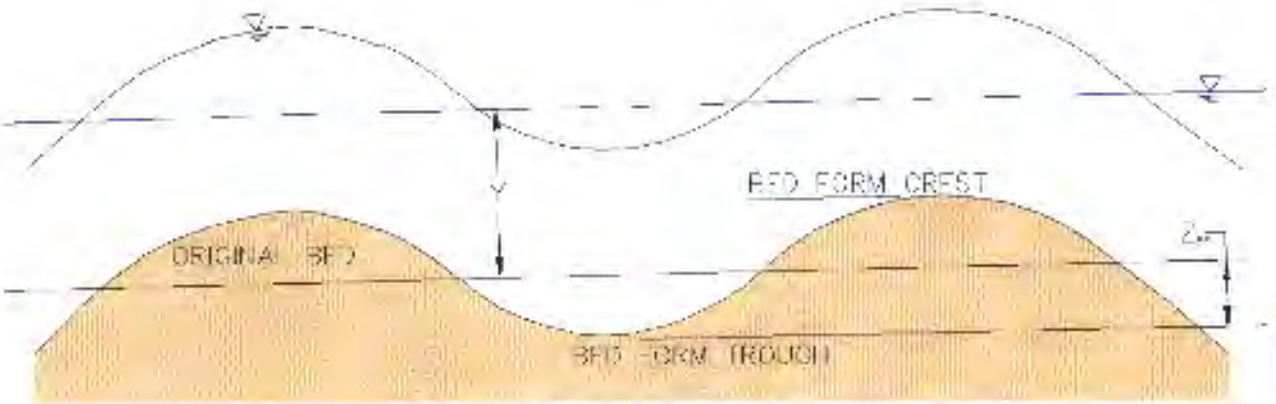

Figure 4: Bed Form Scour Schematic

Based on this relationship, the following equation was developed for upper regime flow (SLA 1982):

$$Z_a = \frac{1}{2}(0.14)\frac{2\pi V_m^{\ 2}}{g} = 0.0137 V_m^{\ 2}$$

Where:

G = gravitational acceleration, in ft/s^2
V_m = Median velocity, in ft/s
Z_a = Anti-dune scour, feet.

Similarly, for lower regime flow, one-half the dune height, from crest to trough, is typically used as the bed-form scour component. This relationship is represented by the following equation for lower regime flow (Julien and Klassen 1995):

$$Z_d = \frac{1}{2}(2.5Y)\left(\frac{D_{50}}{Y}\right)^{0.3} = 1.25Y^{0.7}D_{50}^{\ 0.3}$$

Where:

Y = Flow depth, in feet
D_{50} = Median particle diameter, in feet
Z_d = Dune scour, feet.

It was determined from the channel hydraulics that for the purposes of this investigation, the conditions of the entire Project reach can be considered as containing upper regime flow (Tetra Tech 2009).

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6.3 Low-Flow Channel Incision

Low-flow channel incision is the formation of a low-flow channel within a main channel in which low discharges are carried. There is no known methodology for predicting low-flow incision depth. If the presence of a low-flow thalweg is predicted, it should be assumed to be at least 1.0-foot deep within non-regional (local) watercourses, unless field observations indicate otherwise (Zeller 1981). It was determined, for the purposes of this investigation, that a low-flow incision could develop over time, following proposed channel improvements. Therefore, this term was considered equal to 1.0.

6.4 Local Scour

Local scour is observed whenever an abrupt change in the direction of flow occurs.

6.4.1 Overfall Scour

Local scour at drops will only occur on the downstream side of hard control points, such as grade control structures. Local scour at drops usually occurs when long-term degradation is manifest immediately below a hard control point, and flow is forced to plunge over the grade separation. This scenario creates additional localized scour in the immediate vicinity of the drop.

Grade-control structures can behave as either free-overfall or submerged structures. Accordingly, the procedures for computing scour below either a free-overfall (Z_{lsf}) or a submerged drop (Z_{lss}) are presented below (COT, 1989):

For free-overfall ($h/Y_d \geq 1$), $$Z_{lsf} = 1.32 q_u^{0.54} (H_t)^{0.225} - Y_d$$

For submerged ($h/Y_d < 1$), $$Z_{lss} = 0.581 q_u^{0.667} \left(\frac{h}{Y_d}\right)^{0.411} \left(1 - \frac{h}{Y_d}\right)^{-0.318}$$

Where,

- Z_{lsf} = Depth of scour below the streambed downstream of a free-overfall drop, in feet;
- Z_{lss} = Depth of scour below the streambed downstream of a submerged drop, in feet;
- q_u = average peak discharge per unit of width of the channel during the 100-year flood (average Q/Bottom width);
- H_t = Total drop in head, measured from the upstream energy grade line to the downstream energy grade line, in feet;
- h = drop height, in feet; and,
- Y_d = Downstream depth of flow, in feet (i.e., tailwater depth).

Note that when h/Yd is > 0.85 the scour should be computed under both free-overfall and submerged conditions, and the smaller of the two should be used for design purposes. The scour hole created immediately below the drop will extend for a distance of Ls with a parabolic shape, where the maximum scour depth can be expected at a distance of Xs downstream of the drop as presented below:

Ls = 12(Zlsf), or 12(Zlss)

Xs = 6(Zlsf), or 6(Zlss)

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Where all terms are as previously defined.

6.5 Bend Scour

Bend scour typically occurs along the outside of bends and is caused by spiral, transverse currents that form within the flow as the water moves through the bend. Currently, there is no single procedure that will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, an equation has been developed for estimating bend scour in sand-bed channels based on the assumption of the maintenance of constant stream power within the channel bend (COT, 1989):

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.6} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2(\alpha/2)}{\cos\alpha} \right)^{0.2} - 1 \right]$$

Where:

Z_{bs} =	Bend-scour component of total scour depth, in feet
Y_{max} =	Maximum depth of flow immediately upstream of bend, in feet
V_m =	Maximum velocity of flow immediately upstream of bend, in feet per second
Y_h =	Maximum hydraulic depth of flow immediately upstream of bend, in feet
S_e	Maximum energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot
α =	Angle formed by the projection of the channel centerline from the point of curvature to a point that meets a line tangent to the outer bank of the channel, in degrees

The bend scour should be assumed to be zero for bends with deflection angles less than 17.8 degrees. This project is fairly straight; however, some channel curvature exists in several locations. In these locations, bend scour was computed for inclusion in the total scour estimates.

6.6 Total Vertical Scour (Single Event)

Total vertical scour for a single event is computed as the sum of general scour, bed-form (anti-dune trough) depth, low-flow channel incision, local scour, and bend scour.

$$Z_t = 1.3\left(Z_{gs} + Z_{bf} + Z_{lf} + Z_{bs} + Z_{ls}\right)$$

Where all terms are as previously defined.

Table 20 shows the maximum total single-event scour computed for the Project reach both for the Design 10-year event and for the 100-year event.

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Table 20 – Summary of Total Single-Event Scour (10-year)

HEC-RAS River Station	General Scour Depth, Z_{gs} (feet)	Bed-Form (Anti-Dune Trough) Depth, Z_{bf} (feet)	Local Scour (Drops), Z_{ds} (feet)	Thalweg Depth, Z_{th} (feet)	Bend Scour Depth, Z_{bs} (feet)	Sum of Components, $\sum Z_i$ (feet)	Total Depth, $Z_s = 1.3\sum Z_i$ (feet)
247+39	0.10	0.27	0.00	1	0.00	1.37	1.78
244+18	0.10	0.37	0.00	1	0.00	1.18	1.53
241+33	0.11	0.42	0.00	1	0.00	1.53	1.99
237+37	0.10	0.56	0.00	1	0.00	1.46	1.90
235+23	0.26	0.63	0.00	1	0.00	1.73	2.25
232+38	0.48	1.30	0.00	1	0.00	2.78	3.62
229+13	0.59	1.46	0.00	1	0.00	3.05	3.96
225+07	0.36	0.69	0.00	1	0.00	2.05	2.66
221+53	0.56	1.24	0.00	1	0.00	2.80	3.64
218+55	0.55	1.38	0.00	1	0.00	2.92	3.80
215+82	0.88	1.86	0.00	1	0.00	3.74	4.86
212+29	0.51	1.18	0.00	1	0.00	2.69	3.50
209+36	0.35	0.81	0.00	1	0.00	2.16	2.81
206+43	0.47	1.26	0.00	1	0.00	2.73	3.55
202+50	0.13	0.76	0.00	1	0.00	1.89	2.46
198+82	0.10	0.59	0.00	1	0.00	1.69	2.20
194+36	0.11	0.45	0.00	1	0.00	1.56	2.03
189+23	0.10	0.19	0.00	1	0.00	1.29	1.68
185+59	0.10	0.38	0.00	1	0.00	1.48	1.93
181+96	0.10	0.49	0.00	1	0.00	1.59	2.06
179+35	0.10	0.54	0.00	1	0.00	1.64	2.13
174+55	0.10	0.66	0.00	1	0.00	1.76	2.29
169+45	0.27	0.94	0.00	1	0.00	2.21	2.88
165+20	0.35	1.08	0.00	1	0.00	2.43	3.16
162+94	0.26	0.93	0.00	1	0.00	2.19	2.85
157+54	0.15	0.76	0.00	1	0.00	1.91	2.48
152+89	0.15	0.83	0.00	1	0.00	1.98	2.57
148+86	0.19	0.53	0.00	1	0.00	1.72	2.24
142+12	0.16	0.73	0.00	1	0.00	1.90	2.46
137+50	0.14	0.69	0.00	1	0.00	1.83	2.39
131+97	0.10	0.88	0.00	1	0.00	1.98	2.57
128+64	0.12	0.74	0.00	1	0.00	1.86	2.42
125+25	0.43	1.22	0.00	1	0.00	2.66	3.45
122+39	0.16	0.48	0.00	1	0.00	1.63	2.12
120+78	0.25	0.90	0.00	1	0.00	2.14	2.78
118+80	0.86	0.99	0.00	1	0.00	2.66	3.46
116+27	0.57	0.80	0.00	1	0.00	2.02	2.62
113+65	0.41	0.84	0.00	1	0.00	1.65	2.15
111+23	0.18	0.77	0.00	1	0.00	1.95	2.54
108+59	0.16	0.68	0.00	1	0.00	1.28	1.67
107+00	0.10	0.48	0.00	1	0.00	1.37	1.79
105+20	0.10	0.55	0.00	1	0.00	1.16	1.51
103+72	0.15	0.62	3.32	1	0.00	4.61	5.99
100+57	0.38	0.76	5.83	1	0.00	7.97	10.36
97+95	0.39	0.73	5.81	1	0.00	7.93	10.31
95+12	0.31	1.02	7.13	1	0.00	9.46	12.30

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Table 20 – Summary of Total Single-Event Scour (10-year)

HEC-RAS River Station	General Scour Depth, Z_{gs} (feet)	Bed-Form (Anti-Dune Trough) Depth, Z_{bt} (feet)	Local Scour (Drops), Z_{ls} (feet)	Thalweg Depth, Z_{th} (feet)	Bend Scour Depth, Z_{bs} (feet)	Sum of Components, $\sum Z_i$ (feet)	Total Depth, $Z_t = 1.3\sum Z_i$ (feet)
92+68	0.82	1.78	7.83	1	0.00	11.43	14.85
91+79	0.48	1.71	8.08	1	0.00	11.27	14.65
91+00	0.88	1.88	7.97	1	0.00	11.73	15.25
89+22	0.76	1.21	6.99	1	0.00	9.95	12.94
86+56	0.57	1.83	7.86	1	0.00	11.26	14.63
84+14	0.40	1.40	7.20	1	0.00	10.00	13.01
81+52	0.41	1.74	7.71	1	0.00	10.87	14.13
79+97	0.36	1.56	7.98	1	0.00	10.90	14.17
79+11	0.40	1.51	7.94	1	0.00	10.84	14.09
77+36	0.39	1.61	8.23	1	0.00	11.23	14.59
75+66	0.36	1.70	7.90	1	0.00	10.96	14.25
74+55	0.39	1.68	7.67	1	0.00	10.73	13.95
73+17	0.30	1.64	7.64	1	0.00	10.58	13.75
70+54	0.47	1.28	6.94	1	0.00	9.69	12.60
66+84	0.61	1.49	7.43	1	0.00	10.53	13.69
64+57	1.07	2.28	8.70	1	0.00	13.04	16.96
62+23	0.64	1.53	7.36	1	0.00	10.53	13.69
59+98	0.33	1.46	7.37	1	0.00	10.17	13.22
58+42	0.35	1.74	7.89	1	0.00	10.99	14.28
57+34	0.28	1.49	7.30	1	0.00	10.08	13.11
56+31	0.28	1.28	7.03	1	0.00	9.59	12.47
53+42	0.44	1.54	7.39	1	0.00	10.37	13.48
49+67	0.31	1.09	8.31	1	0.00	10.70	13.91
45+55	0.29	1.01	6.50	1	0.00	8.80	11.44
42+63	0.55	1.40	7.14	1	0.00	10.09	13.12
40+08	0.49	1.32	7.06	1	0.00	9.88	12.84
36+72	0.65	1.59	7.57	1	0.00	10.81	14.05
33+33	0.25	1.36	7.73	1	0.00	10.34	13.44
31+21	1.02	2.06	8.69	1	0.00	12.77	16.60
28+20	0.76	2.22	9.45	1	0.00	13.43	17.46
25+22	0.82	2.34	9.70	1	0.00	13.86	18.02
21+92	0.80	2.27	0.00	1	0.06	4.13	5.37
18+64	0.36	1.60	0.00	1	0.06	3.01	3.91
15+84	0.42	1.59	0.00	1	0.05	3.05	3.97
13+30	0.25	1.45	0.00	1	0.04	2.74	3.56
9+07	0.31	1.46	0.00	1	0.88	3.65	4.75
6+13	0.75	1.67	0.00	1	0.59	4.01	5.21
3+55	0.12	1.00	0.00	1	0.68	2.80	3.64
1+71	0.20	1.44	0.00	1	0.00	2.63	3.42
0+69	0.10	0.88	0.00	1	0.00	1.98	2.57

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Table 21 – Summary of Total Single-Event Scour (100-year)

HEC-RAS River Station	General Scour Depth, Z_{gs} (feet)	Bed-Form (Anti-Dune Trough) Depth, Z_{bf} (feet)	Local Scour (Drops), Z_{ls} (feet)	Thalweg Depth, Z_{th} (feet)	Bend Scour Depth, Z_{bs} (feet)	Sum of Components, $\sum Z_i$ (feet)	Total Depth, $Z_t = 1.3\sum Z_i$ (feet)
247+39	0.16	0.48	0.00	1	0.00	1.65	2.14
244+18	0.12	0.16	0.00	1	0.00	1.28	1.66
241+33	0.20	0.73	0.00	1	0.00	1.93	2.50
237+37	0.19	0.38	0.00	1	0.00	1.57	2.04
235+23	0.34	0.86	0.00	1	0.00	2.20	2.86
232+38	1.02	2.05	0.00	1	0.00	4.07	5.29
229+13	1.12	2.18	0.00	1	0.00	4.30	5.58
225+07	0.76	1.01	0.00	1	0.00	2.77	3.60
221+53	1.15	2.20	0.00	1	0.00	4.35	5.66
218+55	1.09	2.12	0.00	1	0.00	4.22	5.48
215+82	1.51	2.62	0.00	1	0.00	5.14	6.68
212+29	1.10	2.14	0.00	1	0.00	4.24	5.52
209+36	0.82	1.36	0.00	1	0.00	3.18	4.13
206+43	1.20	2.26	0.00	1	0.00	4.46	5.80
202+50	0.28	1.28	0.00	1	0.00	2.57	3.34
198+82	0.20	1.16	0.00	1	0.00	2.37	3.08
194+36	0.26	1.06	0.00	1	0.00	2.33	3.02
189+23	0.20	0.59	0.00	1	0.00	1.79	2.33
185+59	0.20	0.91	0.00	1	0.00	2.11	2.75
181+96	0.18	0.81	0.00	1	0.00	2.00	2.60
179+35	0.11	1.02	0.00	1	0.00	2.12	2.76
174+55	0.23	1.15	0.00	1	0.00	2.39	3.10
169+45	0.45	1.43	0.00	1	0.00	2.88	3.74
165+20	0.57	1.43	0.00	1	0.00	3.00	3.90
162+94	0.50	1.33	0.00	1	0.00	2.82	3.67
157+54	0.38	1.32	0.00	1	0.00	2.70	3.51
152+89	0.32	1.40	0.00	1	0.00	2.72	3.53
148+86	0.48	1.20	0.00	1	0.00	2.68	3.49
142+12	0.19	0.93	0.00	1	0.00	2.13	2.77
137+50	0.33	1.14	0.00	1	0.00	2.47	3.22
131+97	0.19	1.55	0.00	1	0.00	2.73	3.55
128+64	0.32	1.69	0.00	1	0.00	3.01	3.92
125+25	0.19	1.21	0.00	1	0.00	2.40	3.12
122+39	0.48	1.27	0.00	1	0.00	2.75	3.58
120+78	0.23	1.15	0.00	1	0.00	2.38	3.09
118+80	0.49	0.80	0.00	1	0.00	2.29	2.97
116+27	1.16	1.38	0.00	1	0.00	3.53	4.59
113+65	0.99	0.87	0.00	1	0.00	2.86	3.71
111+23	0.54	1.18	0.00	1	0.00	2.71	3.53
108+59	0.22	0.24	0.00	1	0.00	1.46	1.90
107+00	0.17	0.11	0.00	1	0.00	1.28	1.67
105+20	0.24	0.84	0.00	1	0.00	2.08	2.71
103+72	0.48	0.25	4.40	1	0.00	6.14	7.98
100+57	0.99	1.35	7.46	1	0.00	10.79	14.03
97+95	0.82	1.97	8.18	1	0.00	11.98	15.57
95+12	0.66	1.79	7.79	1	0.00	11.24	14.61
92+68	1.71	2.89	10.06	1	0.00	15.66	20.36

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Table 21 – Summary of Total Single-Event Scour (100-year)

HEC-RAS River Station	General Scour Depth, Z_p (feet)	Bed-Form (Anti-Dune Trough) Depth, Z_{bf} (feet)	Local Scour (Drops), Z_{ls} (feet)	Thalweg Depth, Z_{th} (feet)	Bend Scour Depth, Z_{bs} (feet)	Sum of Components, $\sum Z_i$ (feet)	Total Depth, $Z_t = 1.3\sum Z_i$ (feet)
91+79	1.03	3.04	10.52	1	0.00	15.59	20.27
91+00	1.52	2.91	10.17	1	0.00	15.61	20.29
89+22	1.68	2.60	9.78	1	0.00	15.06	19.57
86+56	1.36	3.79	10.73	1	0.00	16.88	21.95
84+14	1.03	2.60	9.43	1	0.00	14.06	18.27
81+52	0.50	2.46	9.22	1	0.00	13.17	17.12
79+97	0.95	3.02	10.54	1	0.00	15.51	20.17
79+11	1.06	2.97	10.51	1	0.00	15.54	20.20
77+36	1.07	3.11	10.79	1	0.00	15.97	20.76
75+66	1.01	3.03	10.10	1	0.00	15.14	19.69
74+55	1.01	2.91	9.79	1	0.00	14.71	19.13
73+17	0.79	3.06	9.91	1	0.00	14.75	19.18
70+54	1.25	2.41	8.99	1	0.00	13.65	17.75
66+84	0.85	2.17	9.10	1	0.00	13.12	17.05
64+57	0.55	2.20	9.77	1	0.00	13.52	17.57
62+23	0.72	2.16	8.98	1	0.00	12.86	16.71
59+98	0.81	2.81	9.66	1	0.00	14.28	18.57
58+42	0.73	3.27	10.37	1	0.00	15.36	19.97
57+34	0.75	2.28	8.73	1	0.00	12.76	16.58
56+31	0.79	2.00	8.72	1	0.00	12.52	16.27
53+42	0.87	2.62	11.75	1	0.00	16.25	21.12
49+67	0.88	2.08	8.24	1	0.00	12.21	15.87
45+55	0.80	1.96	8.19	1	0.00	11.95	15.53
42+63	0.67	2.01	8.61	1	0.00	12.29	15.97
40+08	0.51	1.75	8.11	1	0.00	11.37	14.78
36+72	0.70	1.99	8.76	1	0.00	12.45	16.18
33+33	0.70	2.65	10.12	1	0.00	14.47	18.81
31+21	0.68	2.24	9.83	1	0.00	13.74	17.86
28+20	0.83	3.31	11.69	1	0.00	16.83	21.88
25+22	0.65	2.73	11.10	1	0.00	15.48	20.12
21+92	1.67	4.39	0.00	1	0.10	7.16	9.31
18+64	0.70	1.66	0.00	1	0.09	3.45	4.48
15+84	0.87	4.37	0.00	1	0.07	6.31	8.20
13+30	0.57	2.27	0.00	1	0.06	3.91	5.08
9+07	0.28	0.52	0.00	1	0.97	2.77	3.60
6+13	0.53	1.98	0.00	1	1.06	4.57	5.94
3+55	0.39	1.81	0.00	1	0.94	4.14	5.38
1+71	0.35	1.65	0.00	1	0.00	3.00	3.90
0+69	0.35	1.62	0.00	1	0.00	2.97	3.86

6.7 Long-Term Degradation

Channelization of a watercourse or urbanization within a watershed can alter the sediment supply to a watercourse and create a discontinuity related to the sediment-transport capacity of a channel. As a result, over the long term, bed degradation can occur until the channel bed slope becomes "stable." Long-term degradation can then occur under the two conditions: armoring or stable slope.

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6.7.1 Degradation Controlled by Streambed Armoring

In the case of streambed armoring, the finer sediments are washed away coarsening the streambed surface. Based on previous sediment analyses conducted for Pima County watercourses conducted by Simons, Li, and Associates, the approximate D_{90} particle size for the watercourses was consistently determined to be approximately 13 mm (i.e., slightly > 1/2 inch)[1]. NRCS Web Soil Survey[2] data available for the study area indicated that for soils consistent with the wash bottoms seen during the site visit and other projects within the region the D_{50} for riverwash soils (comprised of gravelly-sand) can range between 5 and 19 mm. As a result, for this study, the D_{90} size for the watercourses under investigation is considered to be 13 mm. Thus, based upon flow hydraulics, when the armoring diameter for a watercourse is computed to be 13 mm, or less, the long-term channel degradation is presumed to be controlled by streambed armoring. This can be evaluated using Shields Incipient Motion criteria and the following equation:

$$D_{Critical} = \frac{\tau_o}{\gamma_s' F_*}$$

Where,

$D_{Critical}$ = Critical Diameter where particle size is mobile
T_o = Channel Shear
F^* = Shield's Parameter
γ_s' = Unit weight of sediments minus unit weight of water

A cursory investigation identifies that during a 10-year event and during a 100-year event all particles are capable of being transported, and thus bed degradation would not be controlled by development of an armor layer.

6.7.2 Degradation Controlled by Stable Slope

When there are not enough large particles (> 13 mm) within the streambed sediments to armor the streambed surface, channel bed degradation will continue to adjust the channel slope until the sediment-transport rate becomes equal to the incoming sediment supply, thus creating a "stable slope." Typically this involves the flattening of the stream gradient, which can cause streambed discontinuities within the watershed. Under such circumstances, for the Project study area the following formula should be applied (COT, 1998) to determine equilibrium-slope (stable-slope) conditions:

$$S_{eq} = \left(\frac{Q_{u,10}}{Q_{n,10}}\right)^{1.1} (1 - R_s)^{0.7} S_n$$

Where,

S_{eq} = the stable slope (feet per foot),
S_n = the existing natural slope along the control point (feet per foot),
$Q_{u,10}$ = the urban 10-year peak discharge (cfs),
$Q_{n,10}$ = the natural 10-year peak discharge (cfs), and
R_s = a reduction factor in upstream sediment supply (usually assumed equal to the ratio of impervious area to total area of the contributing upstream watershed).

[1] Personal communication with Michael E. Zeller, P.E., P.H. D_{50} equal to 2 mm and the Gradation coefficient is 5.0
[2] http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

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For a conservative estimate on this project the R_c was assumed to be 10 percent. Based on this evaluation, the anticipated adjustment within the channel system could be on the order of 0.0036 feet of change per foot in the streamwise direction. In other words, in a worst-case scenario, the channel is capable of degrading around 19 ft over a mile. These adjustment can be interrupted by localized caliche lenses and rock outcroppings, where drops would become visible on the downstream side as discussed previously.

6.7.3 Degradation Mitigation Measures

For this project, based on the results of the continuity analysis, sub-reaches downstream of the existing basin (sub-reaches 7 and 8) show the largest potential for degradation. Once a stable slope has been determined, long-term degradation (Z_{td}) can be estimated at a given point of interest from the following equation, (PCRWRD, 2011).

$$Z_{td} = \frac{8}{13}\left(S_n - S_{eq}\right)L_{dc}$$

Where L_{dc} is the estimated distance to a downstream control (or distance between grade-control structures, in feet) and all other terms are as previously defined.

Based on the preceding analysis, the recommended frequency of grade-control structures (designed to accommodate a 3-foot drop) used to stabilize the vertical profile of the channel and protect the toes of the Mission Complex facilities should be spaced every 833 feet, and be adequately embedded to resist being undermined or laterally flanked.

6.8 Lateral Erosion Hazard Setback

At tie-in points, flood-control structures are subject to flanking from the river by lateral erosion or meandering. Therefore, to prevent this type of failure it is recommended that levee toe-protection depths extend sufficiently below the maximum total vertical scour depth (Z_t) and be set back an adequate distance from the banks beyond the potential lateral-erosion limits at tie-in points. Zeller proposed a minimum setback distance along alluvial watercourses to account for lateral channel migration, which can be estimated from the product of an empirical coefficient times the square root of the 100-year discharge. This methodology is similar to that discussed in the Level 1 analysis section of the Arizona State Standards 5-96 (ADWR 1996).

For straight reaches, the following lateral erosion setback was proposed for watercourses draining less than 30 square miles of watershed area (Zeller 1981):

$$SB \geq (Q_o)^{0.5}$$

Straight Reaches ($R_c/T > 10$).

For reaches with R_c/T the following lateral erosion setback was proposed (Zeller 1981):

$$SB \geq 2.5(Q_o)^{0.5}$$

Curvature Reaches ($R_c/T < 5$),

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Where:

SB = Minimum Setback, in feet, measured from the top edge of the highest channel bank or from the edge of the top of the n-year water surface elevation, whichever is closer to the channel centerline;
Q_n = Peak discharge of n-year event, in cubic feet per second;
R_c = Radius of curvature of channel centerline, in feet; and,
T = Channel top width, in feet.

The relationship between rate of curvature (α = angle formed by projection of centerline from the point of curvature to the intersection of a tangent with the bank line) and radius over top-width (R_c/T) is defined by the following equation:

$$\frac{R_c}{T} = \frac{\cos\alpha}{4 * \sin^2\left(\alpha/2\right)}$$

Where all terms are as previously defined.

Table 22 – Summary of Lateral Erosion Setback (Straight Reaches)

Straight Reaches	Downstream of RS 279+91		Downstream of RS 273+30		Downstream of RS 179+35		Downstream of RS 103+72	
YEAR	Q	Setback (ft)	Q	Setback (ft)	Q	Setback (ft)	Q	Setback (ft)
2-yr	193	13.89	367	19.16	511	22.61	668	25.85
10-yr	716	26.76	1356	36.82	1889	43.46	2470	49.70
100-yr	1934	43.98	3665	60.54	5106	71.46	6676	81.71

Table 23 – Summary of Lateral Erosion Setback (Curved Reaches)

Curved Reaches	Downstream of RS 279+91		Downstream of RS 273+30		Downstream of RS 179+35		Downstream of RS 103+72	
YEAR	Q	Setback (ft)	Q	Setback (ft)	Q	Setback (ft)	Q	Setback (ft)
2-yr	193	34.73	367	47.89	511	56.51	668	64.61
10-yr	716	66.90	1356	92.06	1889	108.66	2470	124.25
100-yr	1934	109.94	3665	151.35	5106	178.64	6676	204.27

7.0 RESULTS AND CONCLUSION

As previously discussed, the existing South Dump and TSF 6 & 7 Diversion Channels around the southern end of the ASARCO Mission Complex along the north side of West Helmet Peak Road require maintenance and upgrades. In support of the Mission Complex re-establishing the diversion channels, rehabilitation maintenance operations are required under approval of a 404-Permit Application.

The proposed project will resolve capacity issues in the existing diversion channel by channelizing segments that currently are too shallow to convey the design 10-year event. Additionally, channel

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confluences will be improved and the existing basin will be cleaned out and a new outlet crest elevation armored to resist erosion. At the eastern limits of TSF 6&7 the water will be distributed across a wider floodplain by construction of a diversion structure or junction to split a portion of the water away from the Rancho Resort community allowing it to weir over into the Rancho Resort collector channel over a longer length. This is not intended to resolve the drainage issues that may be present at the Rancho Resort.

The proposed project exists in a relatively steep stream capable of transporting sands and gravels during a typical flood event. The proposed channel configuration is sufficiently capable of conveying both water and sediment, as it is delivered without concern for excessive sediment removal maintenance with the exception of Sub-reach 7 and 8 (existing basin), where this condition presently exists. Based on the results of the continuity analysis, grade-control structures are recommended to be placed approximately every 833 feet downstream of the existing basin and upstream of the diversion structure.

All constructed revetments, bank protection, and grade-controls should be designed with adequate toe-downs that extend sufficiently beyond the predicted scour depth and keyed into the banks sufficiently beyond the potential erosion hazards, as recommended herein.

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