SONOQUI WASH CHANNELIZATION, PHASE III
RIGGS ROAD TO EMPIRE BOULEVARD

FCD 2009C011

DESIGN DATA REPORT

Prepared For:

Flood Control District of Maricopa County

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In Association with:

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March 21, 2012

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Expires 12/31/2014

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ADDENDUM 1

September 23, 2014

A. Introduction

This document is an addendum to *Sonoqui Wash Channelization, Phase III, Riggs Road to Empire Boulevard, FCD 2009C011, Design Data Report*, March 21, 2012 prepared for the Flood Control District of Maricopa County by JE Fuller/ Hydrology & Geomorphology, Inc. in association with Olsson Associates. It is intended to be an attachment to the 2012 report and is considered to be incorporated into that report as if it were a part of the original report. As such, references are included in this addendum to figures and tables from the original report without reproducing those figures and tables herein.

The purpose of this addendum is to document the result of overtopping from the 202nd Street detention basin within project Phase IIIB (Detention Basin A) and from the Happy Road detention basin within project Phase IIIA1 (Detention Basin B). Both basins are designed to contain the 100-year storm runoff without overtopping and are provided with additional storage volume to contain the predicted volume of sediment that would be deposited within the basin floor during a storm event without overtopping. Recognizing the potential for a storm greater than the design storm event or the plugging of the outlet works by debris or other unanticipated events that may cause the basins to overflow, the perimeter basin grading is designed to direct any overtopping flows along their historic flowpath. The overtopping considerations for the basins are described as follows:

B. 202nd Street Detention Basin

The existing condition, or pre-project condition runoff for the 202nd Street basin location is shown on Figure I-2 which depicts the results of a FLO-2D analysis of runoff for the 100-year storm event. Note that runoff generated in the mountainous watershed just south of Hunt Highway concentrates at approximately the 202nd Street alignment and flows over Empire Boulevard to the north. Small flows are conveyed through a culvert located about 660 feet west of 202nd Street and conveyed through a small wash northwesterly to a point along Hawes Road where it leaves the study area; however, when the capacity of this small wash is exceeded, the excess runoff makes its way north along the 202nd Street alignment, contributing to the flooding between Empire Boulevard and Riggs Road. The roadway profile of Empire Boulevard combined with the small culvert west of 202nd Street controls the nature and location of flows continuing to the north. Neither of these features are modified as part of this project.

The basin grading plan is entirely in an excavation condition with no constructed berms or embankments other than small landscape mounds. Therefore, there is no potential for increased downstream flood damage from failure of an embankment resulting from overtopping flows. The basin grading matches natural, existing ground elevations around its perimeter. As such the low point is along the north basin boundary in the vicinity of 202nd Street. If the water level within the basin rose to a point

where it could no longer be contained by the basin, it would flow over the basin edge in a sheet flow condition and make its way north to Empire Boulevard. At that point it would flow through the small culvert and weir over the road in a manner consistent with pre-project flow conditions and continue along the historic flowpath to the north.

C. Happy Road Detention Basin

The existing condition, or pre-project condition runoff for the Happy Road basin location is also shown on Figure I-2 which depicts the results of a FLO-2D analysis of runoff for the 100-year storm event. Note that runoff predominantly enters the area through the box culvert structure at the intersection of Ellsworth Road and Empire Boulevard and is conveyed northwest through a channel that is oriented diagonally through the area extending through the intersection of 206th Street and Happy Road and continuing to the intersection of San Tan Boulevard and 204th Street.

As with the 202nd Street basin, the basin grading plan is entirely in an excavation condition with no constructed berms or embankments other than small landscape mounds. Therefore, there is no potential for increased downstream flood damage from failure of an embankment resulting from overtopping flows. The basin grading matches natural, existing ground elevations around its perimeter. As such the low point is along the northwest basin boundary in the vicinity of 206th Street and Happy Road. If the water level within the basin rose to a point where it could no longer be contained by the basin, it would flow over the basin edge in a sheet flow condition and make its way northwest diagonally along the historic flowpath. The basin outlet pipes have been constructed within the alignment of the downstream pre-project channel, however a swale has been provided as a means to convey excess flows to San Tan Boulevard and 204th Street. Area inlets have also been provided along the alignment to allow surface runoff to enter the outlet pipes when capacity is available.

D. Conclusions

The construction of both basins will significantly reduce downstream flooding from storm events up to and exceeding a 100-year storm. Overtopping flows, if they occur, would be directed along their historic flowpath, and would be at a lower discharge than would occur if the basins had not been constructed, thereby providing a measure of flood mitigation even if flooding did occur from such an extreme event.

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SONOQUI WASH CHANNELIZATION, PHASE III
RIGGS ROAD TO EMPIRE BOULEVARD

FCD 2009C011

DESIGN DATA REPORT

I. INTRODUCTION

A. Background

This Design Data Report is prepared for the Flood Control District of Maricopa County as part of the Sonoqui Wash Channelization Phase III, Riggs Road to Empire Boulevard design project. The *Sonoqui Wash Floodplain Delineation Study* (Entellus, 2004) from Higley Road to Hunt Highway indicated that significant ponding and breakouts of flood flows occur along the wash. Results from the *Queen Creek / Sonoqui Wash Hydraulic Master Plan* (Huitt-Zollars, 2000) indicated that the most feasible solution to contain breakouts from Sonoqui Wash is to increase the cross section of the wash to contain the 100-year flood flows.

Phase I of the Sonoqui Wash channelization project extends from Queen Creek Wash to Chandler Heights Road and was constructed in early 2007. The project removed approximately 200 homes and 800 acres of land from the Federal Emergency Management Agency (FEMA) Regulatory Floodplain. The project was turned over to both the Towns of Queen Creek and Gilbert for operation and maintenance.

Phase II of the channelization project extends from the terminus of Phase I at Chandler Heights Road to Riggs Road (main branch) and along Riggs Road to Crismon Road (east branch). Phase II is currently under construction. The Phase II reach will be operated and maintained by the Town of Queen Creek.

The purpose of this Phase III project as originally scoped was to design, construct, operate and maintain a conveyance channel capable of containing a 100-year storm event within the designed alignment. The design concept was subsequently changed based on the results of the Pre-Design study alternative evaluation. The evaluation of alternative solutions and development of the recommended plan is presented in the *Sonoqui Wash Channelization, Phase III, Riggs Road to Empire Boulevard, Pre-Design Report*, prepared as part of this project. The revised concept consists of a system of two storm water storage basins and a combination of closed conduit and open channel conveyance systems to convey storm water to the system outfall at the Phase II channel. The District will own, operate, and maintain the Phase III facilities.

An upstream extension of the Sonoqui Wash main branch within Pinal County was originally designed by Pinal County as part of project #124400, *Hunt Highway Widening (Phase 1) – Empire Boulevard to Thompson Road*. However, due to funding limitations, the project was never constructed. The portion of the Hunt Highway Widening project that would have constructed a collector channel to direct runoff into the Sonoqui Wash system has been added to this project. The improvements will collect the upstream flows by extending the Main Branch of Sonoqui Wash from the inlet headwall of the box culvert improvements at the intersection of Ellsworth Road and Empire Boulevard to a point downstream from the Sonoqui Wash crossing at Hunt Highway. Facilities upstream of this point, which will direct flows into the Sonoqui Wash project, will be constructed by others.

B. Study Area

The project Study Area is shown on **Figure I-1** and consists of the area contained within Section 33 bounded by Riggs Road on the north, Ellsworth Road on the east, Empire Boulevard on the south, and Hawes Road on the west. The main branch of Sonoqui Wash enters the study area through an existing 5-10'x5' RCBC that extends through the intersection of Ellsworth Road and Empire Boulevard at the southeast corner of the study area. Flow from the intersection is conveyed in a sand bed channel that has been partially constructed by mounding up both the left and right banks. The channel extends diagonally across Section 33, from the intersection of Empire Boulevard and Ellsworth Road to the intersection of San Tan Boulevard and 204th Street. Containment of flow is lost at the downstream end of the diagonal channel, and runoff spreads out into an overland sheet flow condition. A portion of the overland flow is directed due north along the 204th Street alignment to Riggs Road, with the remainder of the flow conveyed northwest through a number of parcels to where it reaches Riggs Road at approximately the 202nd Street alignment. A number of residents within this area have reported repeated and frequent flooding of their homes and property, which has likely been exacerbated in recent years by construction of new residences within the impacted area.

This flooding condition was analyzed as part of the FEMA floodplain study resulting in a shallow floodplain with a large areal extent delineated along the channel alignment as shown on **Figure I-1**.

The watershed contributing runoff to the main branch of Sonoqui Wash extends into the San Tan Mountains in Pinal County and consists of a contributing drainage area of 4.6 square miles. A substantial portion of the watershed remains mostly undisturbed; however, new development is planned and under construction within the watershed.

In addition to the main branch of Sonoqui Wash entering the study area at Empire Boulevard and Ellsworth Road, a second flooding source was identified during the data collection and field reconnaissance phase of this project. Runoff generated in the smaller, mountainous watershed just south of Empire Boulevard concentrates at approximately the 202nd Street alignment and flows over Empire Boulevard into the study area. Small flows are conveyed through a culvert located about 660 feet west of 202nd Street and conveyed through a small wash northwesterly to a point along Hawes Road where it leaves the study area; however, when the capacity of this small wash is exceeded, the excess runoff makes its way north along the 202nd Street alignment, making a substantial contribution to the flooding between Empire Boulevard and Riggs Road.

In order to gain a better understanding of the actual flooding conditions within the study area, a 2-dimensional hydrologic/hydraulic computer model (FLO-2D) was used to route the incoming hydrographs through the study area. Model results identified the extent of flooding and associated flow depths from both flooding sources. In addition, the 2- dimensional flow analysis yielded significantly different results than those shown on the current regulatory floodplain map. The 2-dimensional flood limits are shown on **Figure I-2**.

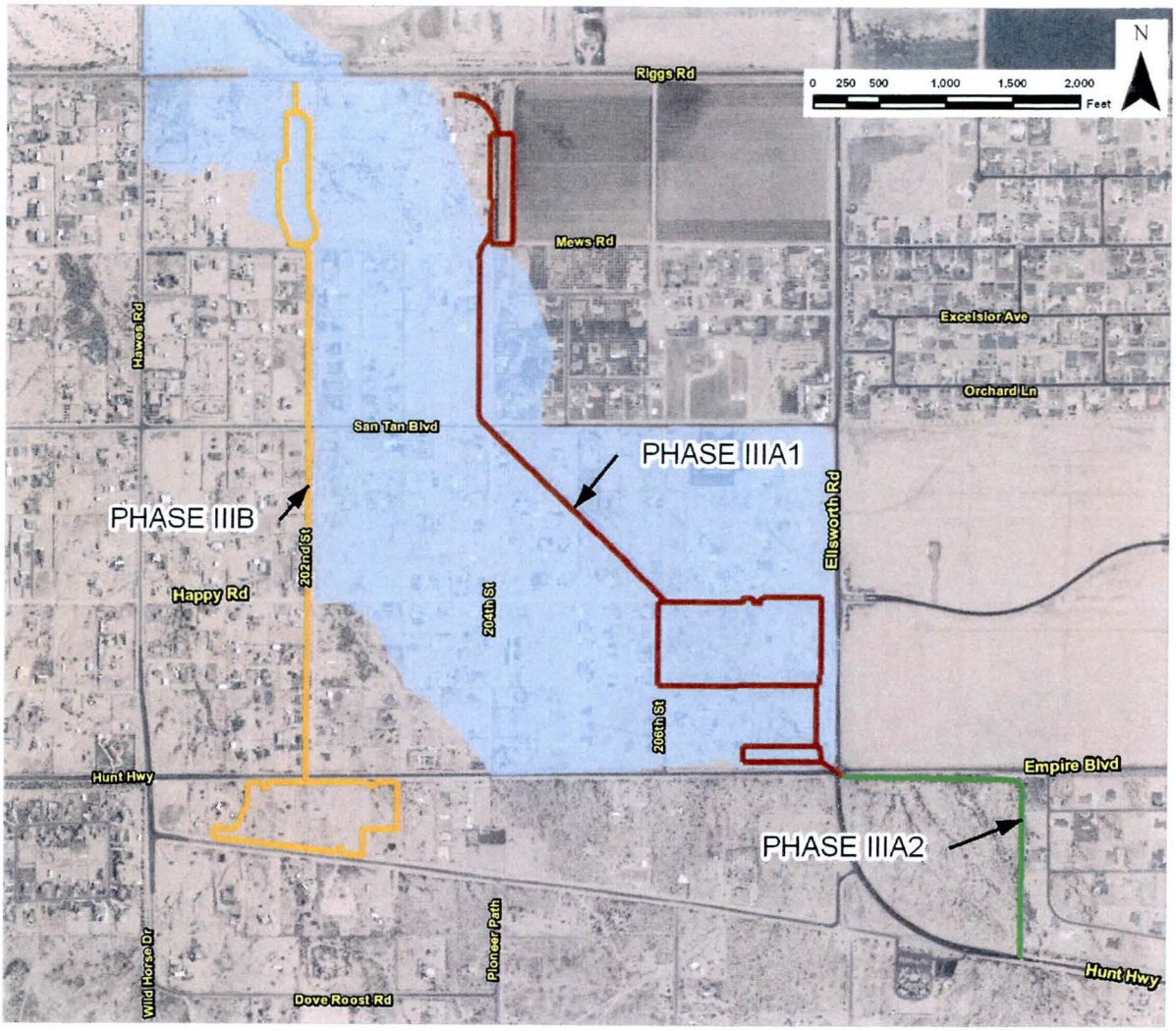


Figure I-1 - Study Area

Flood Control District of Maricopa County
www.fcd.maricopa.gov
2801 W. Durango Street
Phoenix, AZ 85009
602-506-1501

Flood Control District of Maricopa County 1959–2009

SONOQUI WASH CHANNELIZATION PHASE III

FCD#2009C011

LEGEND

Flow Depth At Cell (Feet)

- 0.5 - 0.59
- 0.6 - 1.0
- 1.1 - 1.5
- 1.6 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 8.0
- 8.1 - 9.0

DATA SOURCES

BASE MAP DATA SUPPLIED BY FCDMC

700 350 0
Feet

10/13/2010

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC

OLSSON ASSOCIATES

FIGURE I-2
FLO-2D ANALYSIS
EXISTING CONDITIONS

C. Existing Plans & Studies

The following plans, studies, and designs impact the project:

- Queen Creek/Sanokai Wash Hydraulic Master Plan, FCD 98-26, September 2000
- Sonoqui Wash Floodplain Delineation Study, FCD 2002C033, March 2004
- Sonoqui Wash Channelization, Phase IIA2, FCD 2010C039, 2011
- Hunt Highway Widening (Phase I) Empire Boulevard to Thompson Road, Pinal County Project 124400, October 2008
- Plans for the Construction of Ellsworth Road, Hunt Highway to Cloud Road, MCDOT Project TT178, August 2009
- Riggs Road Bridge at Sonoqui Wash, Design Concept Report (DRAFT), MCDOT Project TT254, June 2008

D. Project Features

The project has two distinct systems; a west system along the 202nd Street alignment and a main branch system along the existing Sonoqui Wash channel alignment. The west system collects, stores, and conveys the runoff generated by the west flooding source and the main branch system collects, stores, and conveys the runoff generated in the main branch watershed. The project will be constructed from the following three independent sets of plans and construction documents:

- Sonoqui Wash Channelization, Phase IIIA1, Main Branch Improvements – Riggs Road to Empire Boulevard, FCD 2011C010
- Sonoqui Wash Channelization, Phase IIIA2, Main Branch Improvements – Empire Boulevard to Hunt Highway, FCD 2011C012
- Sonoqui Wash Channelization, Phase IIIB, West Branch Improvements – Riggs Road to Old Hunt Highway, FCD 2011C011

This Design Data Report incorporates the design documentation for all three sets of construction documents. The two systems are shown in the attached Final Design Plans (**Appendix G**) and described as follows:

1. 202nd Street System

The 202nd Street system consists of a detention basin situated south of Empire Boulevard with an outlet storm drain and channel combination along the 202nd Street alignment discharging into the Sonoqui Wash Phase II channelization project at Marsh Road. The channel junction is just upstream of the Riggs Road crossing.

Detention Basin A is sized to capture approximately 78 acre-feet of runoff, attenuating the peak inflow discharge from 2,161 cfs to 560 cfs which is released into an 84-inch outfall pipe. The basin inflow reaches the basin as sheet flow across Hunt Highway which is immediately south of the basin. The sheet flow weirs over the road over a length of approximately 600 feet. The sheet flow will be collected with a

collector channel constructed on the north side of Hunt Highway. The collector channel will direct the runoff to the low point in the profile which is the existing road crossing dip section location. The collected runoff is then directed into the basin via a spillway constructed on the basin sideslope (Basin A inlet spillway calculations provided in **Appendix D**).

The basin outfall will be an 84-inch concrete pipe extending from the basin outlet to the north side of Mews Road where it will discharge into an open channel. The change from a closed pipe system to an open channel system is to allow discharge into the Phase II channel. The ground slope is relatively flat in the reach from Mews Road to Marsh Road and the available pipe cover is inadequate to allow a gravity discharge from an 84-inch pipe directly into the Phase II channel. Area inlets will be provided along the alignment for local runoff to enter the system.

The downstream limit of the Phase IIIB project will be at the inlet headwall to the triple 10-ft by 4-ft concrete box culvert proposed for Marsh Road as part of the Phase II project.

2. Main Branch System

The Sonoqui Wash main branch system will collect runoff from upstream of the intersection of Ellsworth Road and Empire Boulevard. The runoff will be directed into a detention basin at Happy Road and then discharged into an outlet storm drain and channel combination. The outlet system will extend along the existing wash alignment and along the 204th Street alignment to the Sonoqui Wash Phase II project at Riggs Road.

The runoff generated upstream of the intersection of Ellsworth Road and Empire Boulevard will be collected and conveyed to the intersection via the Phase IIIA2 extension project within a riprap channel which will extend east from the intersection for approximately 1300 ft along the south side of Empire Boulevard. It will then extend south to the downstream (north) side of Hunt Highway at the location of a planned 6 barrel 10'x3' box culvert crossing that will be constructed by Pinal County. The construction of the collector channel will convey runoff to the box culvert inlet at Ellsworth Road and Empire Boulevard, eliminating potential overtopping flows that could flow north over Empire Boulevard and then along the east side of Ellsworth Road contributing to downstream flooding.

The Phase IIIA1 system inflow at the Ellsworth Road / Empire Boulevard intersection is through box culverts passing under the intersection with significant sheet flow passing over Empire Boulevard west of the intersection. The box culverts have been upsized from the previously existing 3 barrel 10-ft by 5-ft configuration to a 5 barrel 10-ft by 5-ft system as part of the Ellsworth Road improvement project constructed by MCDOT in partnership with the Town of Queen Creek and Pinal County. The runoff will be conveyed approximately 700 feet in a triple 102-inch pipe system. Flow within the pipe system will enter Basin B via a stair-step energy dissipater structure (Basin B energy dissipater structure calculations provided in **Appendix D**).

Detention Basin B is sized to contain approximately 50 acre-feet of runoff, attenuating the peak inflow discharge from 2,350 cfs to 1,810 cfs, nearly a 25 percent reduction. The basin outflow is released into a double 108-inch concrete outfall pipe extending from the basin outlet at Happy Road to the north side of San Tan Boulevard where it will transition to a double barrel 10 foot by 8 foot box culvert system. The box culvert is selected due to minimal available cover and flat slopes in the reach from San Tan

Boulevard to Mews Road where it will discharge into an open channel. The open channel will be situated between the extension of Marsh Road along the 204th Street alignment and the Ranchos Legante development. The channel will join the Phase II channel at the point of tangency of the Marsh Road curve at 204th Street.

II. DESIGN CRITERIA

The analysis and design of flood control facilities is governed by the following District guidance documents:

- *Drainage Policies and Standards for Maricopa County*, April 2007
- *Drainage Design Manual for Maricopa County, Hydrology*, November 18, 2009
- *Drainage Design Manual for Maricopa County, Hydraulics*, March 2009 (Draft)
- *River Mechanics Manual for DDMSW*, September 2009 (Draft)

The design criteria adopted for this project is described in the following sections. General criteria contained in the aforementioned manuals will not be repeated herein.

A. Design Storm(s)

Regional flood control planning for the area was developed within the *Queen Creek/Sanokai Wash Hydraulic Master Plan* and is based on the 100-year, 24-hour storm utilizing the NOAA Atlas 2 rainfall depths. The same design storm has been used for subsequent design projects within the watershed which implement the recommendations of the master plan. Although the NOAA Atlas 2 rainfall data has since been superseded by newer rainfall data from NOAA Atlas 14, the earlier source is still used to maintain continuity of drainage design within the system.

The District standard criterion for the design storm requires that the 6-hour and 24-hour storms both be analyzed to determine which storm produces the highest peak discharges. Drainage facilities are then designed for the higher peak discharge producing storm. The 24-hour storm typically governs in large watersheds and the 6-hour storm typically governs in smaller watersheds. Although the downstream drainage system has been designed based on the 24-hour storm, since this Phase III project is near the top of the watershed, both storms were evaluated to determine which storm would produce the highest discharges for Phase III. The evaluation determined that the 6-hour storm produces higher peak discharges and is therefore adopted for this design.

As a result of these findings, a multiple storm design approach is used. The 100-year, 24-hr storm is used to evaluate the project impacts on downstream discharges and water surface elevations within the Phase I and Phase II channel reaches, and the 100-year 6-hour storm is used to develop the design discharges for design of the Phase III facilities.

Although the master plan backbone system is designed for the 100-year storm, the local runoff that reaches the system is evaluated for the 10-year storm. Local area inlets along the system are sized for the 10-year peak discharge.

B. Detention Basins

Detention basins are designed for the 100-year storm using a flow-through concept. Relatively large outfall pipes are provided to quickly drain the lower discharges from the basin to preserve storage volume for the high peak discharges at the highest intensity period of the design storm. In addition to the required storage volume for storm water runoff, additional storage volume is provided to capture and hold sediment washed into the basins from the upstream watershed.

C. Open Channels

Open channels are designed to be stable with the channel bed and banks designed to withstand the tractive shear forces of the design discharge.

D. Storm Drains

Storm drain slopes are set to limit the velocity to 15 feet per second or less for the entire range of flows. In some cases, where the maximum permissible pipe slope is flatter than the natural ground slope, drops are required. When needed, drops are incorporated into manhole structures and are limited to a maximum height of 3 feet per drop.

E. Local Drainage

The residual local runoff that will remain with the proposed project in place will be admitted into the storm drainage system at locations where the surface flows naturally reach the basin, channel, or pipe system. Local runoff inlets are located and sized based on a 10-year storm. In the case of local runoff being conveyed along streets to the system, when the local peak discharges are greater than can be collected in a standard inlet, or set of inlets, a 24-inch storm drain pipe stubout is provided to facilitate future construction of a storm drain lateral extension that could include construction of additional inlets along the road alignment to collect the full design discharge for ultimate discharge into the system. Local runoff inlet rating curve and inlet location exhibit are provided in **Appendix A**.

During a 100-year design storm event, the runoff reaching the local drainage system of area inlets would be expected to drain through the storm drain system several hours before the peak watershed runoff reaches the upstream detention basins. As a result, the local runoff system of inlets is not anticipated to be contributing to the peak discharges being conveyed through the storm drain system. Therefore, the local inlet flows are not added to the 100-year design discharges leaving the basins.

F. Roadway Design

Surface improvements through the project area primarily consist of constructing maintenance roads for access to the channels and storm drain manholes that are being constructed as a part of this project. At many locations along 202nd Street and 204th Street these will also be used by local residents for access to their homes. These locations are currently unpaved, but as a part of this project AB will be placed for dust control.

The horizontal and vertical design for the access road is based on a 35 mph design speed. AASHTO – *"Guidelines for Geometric Design of Very Low-Volume Local Roads"* recommends a nominal clear zone width of 6 feet beyond the edge of roadway. There are several locations where obstructions exist just off of the proposed roadway corridor; these obstructions consist of power poles, walls and fences, etc. These obstacles limit the corridor width to approximately 30' and as a result reduce the 6-foot clear

zone normally desired. However the AASHTO guide referenced provides guidance that states: *"Where provision of a clear zone is not practical, none is required. Site specific conditions and engineering judgment of the designer should be the two primary determinants of the appropriate clear zone width for very low volume local roads"*. Three locations exist along 202nd Street where the 6-foot clear zone criterion is currently not accommodated; these are at station 114+50 (5' from face of pole), station 116+50 (5' from face of pole) and station 118+20 (4.5' from face of pole).

III. FLOODING CONTEXT

A. Hydrology

1. Local Design Model

The design of the Sonoqui Wash Phase IIIA1 and IIIB projects is based on a 100-year, 6-hour storm (HEC-1: SNQI3_6HR.DAT). The decision to use the 6-hour storm rather than the 24-hour storm was based on the size of the contributing area, which is approximately 7.8 sq. mi. (5.9 sq. mi. and 1.9 sq. mi. for the Main and East Branches, respectively). To verify the validity of this decision, the 6-hour storm was compared to the 24-hour storm (HEC-1: SNQI3_24HR.DAT). The comparison showed that the 6-hour storm generally produced higher peak flows, while the 24-hour storm generally produced larger volumes of runoff. Although the 6-hour storm was the design storm of choice, Basins A and B were checked for volume capacity by routing hydrographs from the 6- and 24-hour storm through each basin. Due to hydrograph timing, the 6-hr storm produced the highest storage volumes in the detention basins. As a result the 100-yr, 6-hr storm is used for the local design discharges. HEC-1 modeling results for the 6-hour and 24-hour storms are provided in Table III-1. The HEC-1 schematic is shown on **Figure III-1**. Design HEC-1 modeling is provided in **Appendix A**.

The design of the Sonoqui Wash Phase IIIA2 project is based upon the design discharge from the *Hunt Highway Design Concept Report - Empire Boulevard to Franklin Road Project No. 124400* by RBF Consulting. The design discharge for this project was 2300 cfs and is based upon a 100-year-recurrence interval storm. A spatially comparable discharge based upon the local design model developed for Phases IIIA1 and IIIB is approximately 2100 cfs. An existing lateral ditch is present within the Phase IIIA2 area and was not explicitly accounted for in the DCR-level design. The design discharge for this ditch was developed by linearly scaling the contributing area for the ditch with respect to the contributing area of subbasin S330E in the Phase IIIA1 and IIIB project hydrology (HEC-1: SNQI3_6HR.DAT). Based upon this analysis a design discharge of 299 cfs was calculated (100-year, 6-hour).

2. Regional System Model

The Sonoqui Wash Phase III design was evaluated to ensure peak discharges downstream of Riggs Road are not increasing as a result of the proposed improvements. To accomplish this, the Detention Basins A and B and downstream pipes and open channels were coded into the Phase II design hydrologic model (HEC-1: ALTBASE.DAT). The results from this hybrid model (HEC-1: REGIONAL24.DAT) show an insignificant increase in peak flows downstream of Riggs Road. The pre- and post-project peak flows at strategic locations are shown in **Table III-2**. Regional HEC-1 modeling is provided in **Appendix A**.

3. Local Inflows

Within Phases IIIA1 and IIIB, local runoff is collected along the project corridor by a series of at-grade catch basins (MAG Standard Detail 537). Catch basin locations are identified along storm drain alignments approximately every 1/8 of a mile (660 feet), which also corresponds to roadway crossings. Given the shallow, unconfined nature of the flow within the project limits, FLO-2D was used to evaluate the effectiveness of the catch basins. Based on the FLO-2D results, 24-inch diameter storm drain stub-outs are shown where flow accumulation is still significant. The stub-outs are intended for future catch basin and lateral pipe improvements.

In Phase IIIA2, local inflows from the south are captured in transverse swales and directed to two grouted-rock downdrains. These down drains were sized based upon rational method calculations and are located coincidently with existing low points in the terrain. A scupper was added to the Empire Blvd improvements during construction and built during the design of the Sonoqui Wash Phase III project. A riprap channel and grouted rock downdrain have been provided to pass flow from the scupper around the 5-10'x5' RCBC headwall, and into the channel.

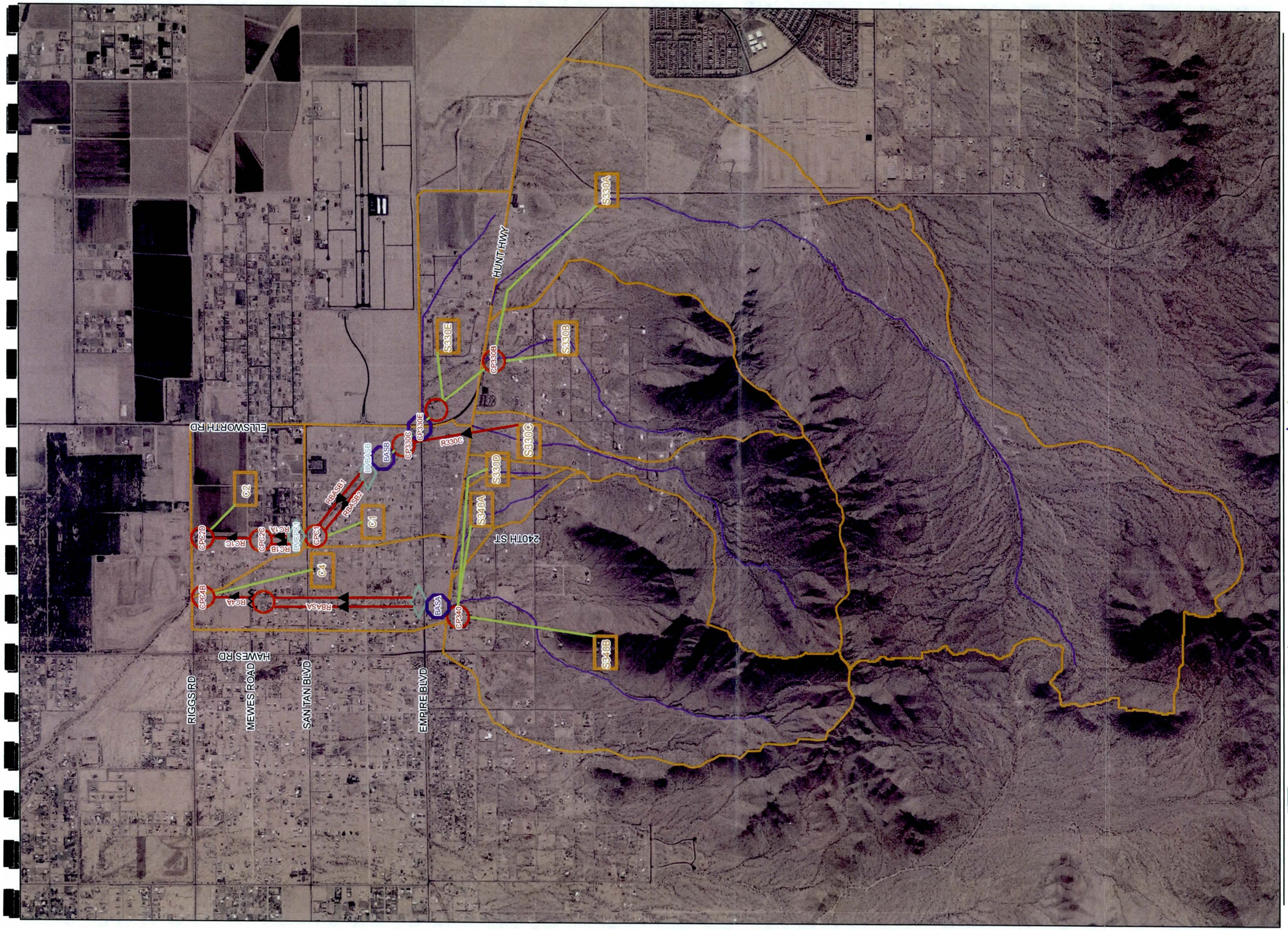
Table III-1. 100-Year, 24- and 6-Hour HEC-1 Modeling Results: Peak Flows and Runoff Volumes.

		Peak Flow (cfs)			*Volume (ac-ft)*		
HEC-1 ID	***Feature***	***24HR***	***6HR[2]***	***Difference =(6HR-24HR)***	***24HR***	***6HR[2]***	***Difference =(6HR-24HR)***
S330A	Sub Basin	1,915	1,986	71	298	291	-7
S330B	Sub Basin	1,170	1,501	331	70	72	2
CP330B	Conc. Pnt.	1,971	1,991	20	364	348	-16
S330E	Sub Basin	242	376	134	17	23	6
CP330E	Conc. Pnt.	2,062	2,037	-25	380	361	-19
S330C	Sub Basin	802	1,065	263	78	78	0
R330C	Routing Reach	761	1,024	263	78	78	0
CP330C	Conc. Pnt.	2,355	2,351	-4	453	424	-29
BASB	Level Pool	1,787	1,807	20	455	425	-30
DRBASB	Diversion	893	904	11	227	213	-14
DVBASB	Diversion	893	904	11	227	213	-14
RBASB1	Routing Reach	893	903	10	227	213	-14
DRBASB	Diversion	893	904	11	227	213	-14
RBASB2	Routing Reach	892	903	11	227	213	-14
C1[1]	Sub Basin	426	605	179	19	22	3
RETDIV (C1)[1]	Diversion	426	605	179	18	18	0
RETAIN (C1)[1]	Diversion	7	179	172	1	4	3
CPC1	Conc. Pnt.	1,783	1,802	19	453	423	-30
DRCPC1	Diversion	891	901	10	227	211	-16
DVCPC1	Diversion	891	901	10	227	211	-16
RC1A	Routing Reach	891	901	10	227	211	-16
DRCPC1	Diversion	891	901	10	227	211	-16
RC1B	Routing Reach	891	901	10	227	211	-16
CPC2C	Conc. Pnt.	1,783	1,802	19	453	423	-30

RC1C	Routing Reach	1,783	1,801	18	453	423	-30
C2[1]	Sub Basin	193	199	6	18	24	6
RETDIV (C2)[1]	Diversion	193	199	6	14	14	0
RETAIN (C2)[1]	Diversion	55	114	59	4	10	6
CPC2D	Conc. Pnt.	1,815	1,797	-18	454	423	-31
S330D	Sub Basin	50	74	24	2	2	0
S340A	Sub Basin	130	203	73	5	7	2
S340B	Sub Basin	2,044	2,105	61	139	135	-4
CP340	Conc. Pnt.	2,140	2,161	21	146	142	-4
BASA	Level Pool	531	561	30	146	142	-4
RBASA	Routing Reach	531	561	30	146	142	-4
RC4A	Routing Reach	531	560	29	145	142	-3
C4[1]	Sub Basin	191	275	84	17	19	2
RETDIV (C4)[1]	Diversion	191	275	84	17	18	1
RETAIN (C4)[1]	Diversion	0	28	28	0	1	1
CPC4B	Conc. Pnt.	531	560	29	145	142	-3
FINAL	Conc. Pnt.	2,288	2,290	2	585	532	-53

Notes: 1. HEC-1 card taken directly from modeling provided by FCDMC (6-Hr HEC-1: SNQ6.DAT; 24-Hr HEC-1: ALTBASE.DAT).

2. Design storm = 100-Year, 6-Hour storm.



Flood Control District of Maricopa County 1959–2009

SONOQUI WASH CHANNELIZATION PHASE III

FCD#2009C011

LEGEND

- Concentration Point
- Sub Basin Boundary
- Storage Area
- Diversion
- Routing Reach
- Direct Combination
- Flow Path

1,000 500 0 1,000 2,000

Feet

DATA SOURCES

AERIAL PHOTOGRAPHY SUPPLIED BY USDA/FSA - AERIAL PHOTOGRAPHY FIELD OFFICE. 2007.

JULY, 2010

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

OLSSON ASSOCIATES

FIGURE III-1
HEC-1 SCHEMATIC

Table III-2. The Pre- and Post-Project Peak Flows at Strategic Locations.

HEC-1 ID	Description	Pre-Phase 3 Project Discharge (cfs)	Post-Phase 3 Project Discharge (cfs)	Difference Between Pre and Post Project (cfs)
CP-C2B	Sonoqui Wash at Riggs Road	2562	2274	-288
CP-C16	Sonoqui Wash at Hawes Road	2540	2252	-288
CP-C7A	Sonoqui Wash at Sossman Road	2056	2067	+11
CP-C7M	Sonoqui Wash at Stage Stop Retention Basin	2190	2185	-5

4. Empire Boulevard Improvements

Flow crossing Empire Boulevard, west of Hunt Highway and Ellsworth Road, was evaluated in detail using FLO-2D (20100817_Sonoqui_wproj_25ft_rds_rough). This evaluation was completed to determine the hydrologic impact that will result upon completion of the Empire Boulevard improvements constructed by MCDOT. According to model results, the improvements will likely result in the following splitting of the approximate 1,000 cfs crossing Empire Boulevard at Ellsworth Road (see design hydrology):

- Approximately 550 cfs flows to the east, across Hunt Highway, before reaching Empire Boulevard. This flow is conveyed through the culverts under the intersection of Ellsworth Road and Empire Boulevard and then through the proposed 3-102" diameter pipes, eventually outfalling into Basin B.
- Approximately 300 cfs flows north over Empire Boulevard and is captured in the Phase III proposed collector channel located just north of Empire Boulevard. Once collected the flow is conveyed via proposed 3-102" diameter pipes to Basin B.
- Approximately 150 cfs is lost due to attenuation and/or a routing of flow to the west toward 206th Street.

With an understanding of the likely flow split, JEF performed HEC-1 modeling of the split to complete the design of the collector channel located just north of Empire Boulevard, west of Ellsworth Road (see Design Plans). The collector channel is an interim drainage facility that can be replaced upon parcel improvements. The above discussed FLO-2D and HEC-1 modeling of the split is provided in **Appendix A**.

B. Hydraulics

1. Boundary Conditions

The hydraulic modeling of the Sonoqui Wash Phase III design is dependent on the coincidental water surface elevation at the Phase II and Phase III confluence (near the Riggs Road Bridge). This coincidental water surface elevation was estimated by combining confluence hydrographs to determine the peak flow within the Sonoqui Wash downstream of the confluence near the Riggs Road Bridge, then modeling this cumulative peak flow in the Phase II design HEC-RAS model (HEC-RAS: SW2_100_Design.prj).

Given that the Phase III design is based on a 100-year, 6-hour storm, the FCDMC provided the 6-hour, existing condition HEC-1 model developed for the Sonoqui Wash Floodplain Delineation Study (HEC-1: SNQ6.DAT, Revised March 10, 2004) for use in addition of confluence hydrographs. The Phase III design hydrology (HEC-1: SNQI3_6HR.DAT) culminates in a concentration point (HEC-1 ID "FINAL") that combines the Main Branch (204th St. alignment) with the West Branch (202nd St. alignment). The "FINAL" hydrograph was added to the "RE9W" routing hydrograph from the existing condition hydrology. "RE9W" routes the Sonoqui Wash East Branch flow to a location near the Riggs Road Bridge crossing. It should be understood that no 6-hour future condition (design) hydrology had been developed as part of the Phase II project; and therefore, the 6-hour, existing condition hydrology provided by the FCDMC was assumed to be the best available model for estimating the coincidental peak flow at the Phase II and Phase III confluence. The resulting peak flow after combining of the confluence hydrographs is 2,294 cfs. The cumulative discharge of 2,294 cfs was input into the JEF-modified Phase II design HEC-RAS model (HEC-RAS: PhaseIIIDSWSEL.prj). The computed subcritical water surface elevations used as the hydraulic modeling downstream boundary conditions for the Phase III Main and East Branches are shown in **Table III-3**. Addition of the confluence hydrographs (20110413_6HR_HYDRGRPH_JEF&SCI.xlsx) is provided in **Appendix B**.

Table III-3. Downstream WSEL Boundary Conditions for Main and West Branches.

Phase II HEC-RAS River Station	Phase III Branch	Boundary Condition WSEL (ft)
30900	Main Branch (204th St. Alignment)	1390.82
29500	West Branch (202nd St. Alignment)	1385.65

The above computed water surface elevations at the Main and West Branches were used as downstream starting conditions for their respective HEC-RAS models (HEC-RAS Main Branch: MainBranch.prj; HEC-RAS West Branch: westbranch.prj). The HEC-RAS water surface elevations at the upstream limit of the models (outlet of proposed 202nd St. and 204th St. storm drains) were then used as downstream boundary conditions for the storm drain analysis for the Main and West Branches.

The downstream boundary condition for the Phase IIIA2 channel analysis is formed by the existing 5-10'x5' RCBC spanning the Empire Boulevard-Ellsworth Road intersection. The presence of the trash rack (Phase IIIA1, Structure 9) was included to account for potential debris accumulation. Based upon this analysis a downstream water surface elevation of 1443.68 ft was used in HEC-RAS modeling.

2. Open Channel Analysis & Design

HEC-RAS models were developed for the open channels along the 204th St. and 202nd St. alignments of the Phase III Main and West Branches, respectively as well as the Phase IIIA2 channel. As discussed in Section B.1, the downstream boundary conditions for the 204th St (Phase IIIA1) and 202nd St (Phase IIIB) models were set based on the cumulative hydrographs at the Phase II and Phase III confluence. Peak discharges for the modeling of the Main and West Branches were taken from the design hydrology (See Section III.A). The Main Branch design discharge is rounded to 1,810 cfs. The West Branch design discharge is rounded to 560 cfs. Roughness for armored reaches was calculated using the method by Phillips and Ingersoll presented in *Verification of Roughness Coefficients for Selected Natural and*

Constructed Stream Channels in Arizona (1998). Typical and maximum flow velocity and depth results are shown below in Table III-4. HEC-RAS workmaps and output are provided in **Appendix B**.

Table III-4. Typical and Maximum Flow Velocity and Depth Results from HEC-RAS Modeling of Phase III Open Channels.

Phase III Branch	Typ. Vel. (ft/s)	Typ. Depth (ft)	Max. Vel. (ft/s)	Max. Flow Depth (ft)
Main Branch (IIIA1)[3]	6.4	5.5	8.4[1]	5.8
Pinal Reach (IIIA2)	7 (Upper reach) 5 (Lower Reach)	5 (Upper Reach) 7 (Lower Reach)	7.7	8.03
West Branch (IIIB)[2,3]	1.8	5.2	2.5	5.3

Notes:

1. Cross-section in vicinity of confluence with Phase II Channel.
2. West Branch flow velocity and depth controlled by 3-10'x4' RCBC at Marsh Road.
3. Main Branch HEC-RAS Model: MainBranch.prj. West Branch HEC-RAS Model: westbranch.prj.

Modeling results assuming a riprap condition were used to check for available freeboard in the Main and West Branches. Per FCDMC criteria for subcritical channels, minimum freeboard was taken as 1-foot or 25 percent of the energy grade depth. Both the Main and West Branch Channels meet the required freeboard criteria. Freeboard estimate calculations are provided in **Appendix B**.

The West Branch HEC-RAS model results show the typical flow velocity below 5 ft/s (varies from 4.06-1.86 ft/s). This is a result of the mild slope and backwater from the proposed 3-10'x4' RCBC at Marsh Road. Given the relatively low velocities within this channel, a riprap channel lining with a D_{50} size of 6 inches is recommended. However, due to the likelihood of relatively high velocities at the outlet of the 84-inch diameter storm drain, a riprap apron with a D_{50} size of 18 inches has been designed at this location. Riprap calculations for the West Branch Channel are provided in **Appendix B.**

The Main Branch HEC-RAS model results, for the full-riprap channel condition, show the typical flow velocity around 5.8 ft/s (4.74-5.78 ft/s). This velocity range is maintained near the confluence with the Sonoqui Wash Phase II Channel, although JEF has not analyzed the Phase III confluence in detail. The downstream-most section of channel in the JEF-design HEC-RAS model has been designed by others as part of Phase II. Riprap size calculations were completed based on the Isbash Equation provided in the *River Mechanics Manual for DDMSW* (FCDMC, DRAFT – September 2009). Based on these calculations, a riprap D_{50} size of 12 inches is recommended for the majority of the Main Branch Channel. In addition, a riprap apron with a D_{50} size of 18 inches has been designed at the outlet of the 2-10'x8' RCBC storm drain. Riprap calculations for the Main Branch Channel are provided in **Appendix B**.

The Sonoqui Wash Phase IIIA2 channel, located within Pinal County, was modeled and analyzed similarly to the channels within Phases IIIA1 and IIIB. Due to right-of-way limitations, the downstream boundary condition, and existing infrastructure along the northern boundary of the channel, Flood Control District of Maricopa County channel design criteria cannot be satisfied along the entire channel reach. Notable deviations include 2:1 H:V riprap side slopes, inadequate freeboard along Empire Boulevard, and channel radiuses of curvature less than 3 times the channel top width. Channel curvature, in general, is equal to approximately 3 times the top width of the design flow. In the region where a ditch paralleling Empire Boulevard enters the channel, the top width of the channel increases markedly and the curvature ratio reduces significantly.

Riprap sizing in the Phase IIIA2 channel varies by location. In the upper reach (oriented North-South), D_{50}=12" riprap was sized, in the curve between the upper and lower reaches D_{50}=18" riprap was sized, and in the lower reach, which is dominated by backwater from the 5-10'x5' RCBC, D_{50}=6" riprap was sized. In the ditch paralleling Empire Blvd, riprap was extended 100 ft upstream to account for secondary currents or turbulence due to the confluence. Normal depth analysis of the ditch was performed using FHWA's Hydraulic Toolbox (v. 1.0). Normal depth analysis indicates flow within the ditch will be drowned-out by backwater from flow in the main channel during coincident flows. Freeboard and riprap calculations for the channel are provided in **Appendix B**.

An assessment of backwater impacts beyond the limits of Phase IIIA2 was undertaken after the inclusion of the Empire Boulevard ditch within the area of disturbance. Based upon the design discharge of 2300 cfs, the northwest corner of the property adjacent to the northeast corner of the project (APN 509-04-101) is partially inundated by backwater from the Phase IIIA2 main channel. Based upon aerial mapping, no habitable structures are inundated by the backwater. The backwater water surface elevation of 1445.18 is significantly lower than the water surface elevation documented in *Letter of Map Revision for Sonoqui Wash Between Empire Blvd and Valley View Rd* (Pinal County Project No. 124400, dated May 2, 2008 by RBF Consulting) which listed a water surface elevation between 1446.66 and 1447.69. Sedimentation within the existing Empire Boulevard ditch adjacent to this parcel may also pose a flood risk under 100-yr flows within the ditch, however, this area is outside the scope of this project.

3. Storm Drain Analysis & Design

Circular storm drains are designed as pre-fabricated reinforced concrete pipe (RCP). Rectangular sections are designed as cast-in-place reinforced concrete box culverts (RCBC). RCP and RCBC are analyzed using a Manning's n of 0.013 for pipe roughness. Alternate pipe materials will be considered based on the results and recommendations from the geotechnical report. Specifically, cast-in-place concrete pipe (CIPP) has been evaluated for use. Per FCDMC standards, CIPP equivalent pipe has been specified by increasing pipe diameters by 6" and specifying the appropriate wall thickness based upon ADOT pipe selection guidelines. CIPP has not been allowed within the Empire Boulevard ROW and at the upstream limit of Phase IIIA1 due to complicated junction requirements.

The hydraulic grade line (HGL) is evaluated under steady flow conditions for the peak design discharge with consideration for energy losses from pipe friction, drop manholes, and bends and is computed for each system starting at the downstream end and progressing upstream according to the following relationship:

$$HGL_2=HGL_1 + H_f + H_{mh} + H_b$$

Where;

HGL_2 = the upstream station hydraulic grade line elevation (ft),
HGL_1 = the downstream station hydraulic grade line elevation (ft)
H_f = Headloss due to friction (ft),
H_{mh}= Headloss due to manholes (ft), and
H_b= Headloss due to bends (ft)

The downstream starting HGL elevation is either the downstream water surface elevation from the HEC-RAS water surface profile analysis of the downstream channel section plus the outlet trash rack loss, if applicable, or the top of pipe, whichever is higher. The headloss due to friction (H_f) is computed using Manning's equation to compute the friction slope (S_f), which is then multiplied by the pipe length between stations to get the friction loss in feet. The MAG521 storm drain manhole, which applies to storm drain pipes of 51-inches and larger, consists of a pre-fabricated tee section with the 48-inch Tee pointed upward. The 8-foot long tee pipe section is encased in concrete and the manhole shaft is placed on top of the upward facing 48-inch pipe section. As a result, the mainline pipe extends continuously through the manhole section with no variation in cross-section that would create an additional energy loss beyond the pipe friction loss. Therefore, no manhole loss is added for these structures. There are several special manhole structures which are also designed as drop manhole structures, which are required to maintain the maximum allowable pipe slope. Losses are included for these structures estimated as the product of the manhole loss coefficient, K_{mh} and the velocity head in the downstream pipe. The value of K_{mh} for the drop manholes is computed using the FHWA Access Hole Method presented in HEC-22, *Urban Drainage Design Manual*, September 2009 and is estimated to be 0.6 for the 2-108-inch pipes. Similarly, bend losses are estimated as the product of the bend loss coefficient, K_b and the velocity head. K_b is assumed to be 0.14 for a 45-degree bend.

Design Discharges used in the calculations are shown in Table III-5. The hydraulic grade line analysis is contained in **Appendix C**.

Table III-5 - Storm Drain Design Discharges.

LOCATION	DESIGN DISCHARGE (CFS)
West Branch – 202nd St.	
1-84-inch RCP	560
Main Branch	
3-102-inch RCP	2350
2-108-inch RCP	1810
2-10'x8' RCBC	1810

Confluence Analysis

One junction location exists within the project limits which consists of 2-84" diameter pipes entering 1-102" diameter pipe at a 35 degree angle. FHWA's HEC-22 provides an analysis method for calculating headloss at conduit junctions based upon the momentum equation. The equation is shown below.

$$H_j = \frac{(Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)}{0.5\, g\, (A_o + A_i)} + h_i - h_o$$

where:

H_j	=	junction loss, m (ft)
Q_o, Q_i, Q_l	=	outlet, inlet, and lateral flows respectively, m^3/s (ft^3/s)
V_o, V_i, V_l	=	outlet, inlet, and lateral velocities, respectively, m/s (ft/s)
h_o, h_i	=	outlet and inlet velocity heads, m (ft)
A_o, A_i	=	outlet and inlet cross-sectional areas, m^2, (ft^2)
θ	=	angle between the inflow and outflow pipes (figure 7-4)

Several elements of the project complicate this analysis. Firstly, the 102" diameter conduit is parallel to two additional 102" diameter pipes. Secondly, the location of the pipe junctions is also a hydrologic junction with aerial reduction applied to the component discharges. Lastly, the 84" diameter laterals are currently designed in an interim condition of indeterminate duration. Improvements to the intersection of Empire Blvd and Ellsworth Road cause ponding of flows in the southwest quadrant of the intersection. Ultimately, the 84" diameter conduits should be extended beneath Empire Blvd to capture the ponded water, however, until that time, at the 100-year discharge, water will pond and overtop Hunt Highway with some flow captured in the channel and passed into the 84" diameter pipes.

For analysis, the system of conduits including both 84" diameter pipes and the three 102" diameter pipes was modeled. The presence of parallel conduits was addressed by adjusting discharges through the conduits until a uniform upstream headwater condition was developed among the three 102" diameter barrels of the trunk line. Additionally, Q_L was calculated at the time corresponding to the peak discharge in the system and reduced areally to generate the corresponding lateral discharge during peak flow in the trunkline system. A hydrologically conservative assumption of concurrent peaking in the lateral and trunkline was assumed for junction analysis. This analysis, utilizing 1007 cfs as the lateral flow and 2351 cfs as the total downstream discharge, generated the greatest headloss under the ultimate conditions. Interim condition results were used for backwater analysis of the collector channel in the northwest quadrant of the Empire Blvd/Ellsworth Rd intersection, while ultimate condition results were used for analysis of the existing 5-10'x5' RCBC at the intersection of Empire Blvd and Ellsworth Rd.

4. Detention Basin Analysis & Design

Hydraulic aspects of the detention basin design consist of the stage-storage-discharge relationships that are entered into the HEC-1 hydrology model for the storage routing routine. In addition to storm water storage for peak flow attenuation, the basins are also designed to serve as sediment storage basins. The total 5-year maintenance period sediment yield volume, described in the following section, is expected to settle out of the flow and deposit in the basin bottom as a result of the low velocity, ponded condition within the basin. To account for the storage volume lost to sediment, the maintenance sediment volume is subtracted from the bottom elevations of the stage storage relationship. As a result,

the basin volume that is designated for sediment storage is not used for the HEC-1 storage routing calculation.

The stage-discharge relationship for detention basin flow routing is computed using HY-8 for the outlet pipe and associated inlet configuration. The storm drain hydraulic grade line calculations described in the previous section do not include inlet losses for the basin outlet pipes. Those calculations are only used for the storm drain analysis and are not intended for use in the detention basin stage-discharge relationship. The inlet controlling water surface elevation computed with HY-8 is then adjusted to incorporate the trash rack losses. Trash rack loss calculations include a 50% plugging factor. For simplification, a constant trash rack loss is utilized for the full range of flows. The specified trash rack loss is the loss associated with the peak design discharge. The stage-discharge relationship which includes inlet and trash rack losses is used for the HEC-1 hydrograph routing. The HY-8 and trash rack loss calculations are contained in **Appendix D.**

Inlet and outlet structures for each basin were designed as structural elements. Both basins incorporate stepped drop structures at their primary inflow points to transition flow from upper elevations to the basin floor. While there are differences in concept between the two stepped drop structures, the analysis methods and general features are similar. Both structures incorporate an upstream weir to train flows prior to entering the stepped portion of the structure; the Main Branch basin incorporates a level, sharp-crested weir with central notch to bypass low-flows while the West Branch basin utilizes a broad-crested trapezoidal-cross-section weir. Analysis of the stepped portion of the structures follows methodology described in "Hydraulic Design of Stepped Cascades, Channels, Weirs, and Spillways" (Chanson, 1996) and utilizes the step height, tread width and inflow hydraulics to calculate friction losses, flow depth, length to achieve homogenous aeration, and energy loss. The West Branch basin structure is sufficiently steep to develop a fully aerated condition while the Main Branch structure is not. Immediately downstream of the stepped portion of the structure, stilling basins have been designed to facilitate additional energy loss as well as transition flow from super-critical to sub-critical prior to releasing it to a riprap apron immediately downstream. Both stilling basins incorporate an end sill with a height calculated to support the generation of a hydraulic jump on the stilling basin surface and an apron sufficiently long to contain the entire length of the hydraulic jump. Low-flow "breaks" in the end sill have been incorporated to allow passage of minor discharges without requiring underdrains or other features. The gaps in the end sills also facilitate maintenance access onto the stilling basin surface. Stilling basins were designed based upon guidance in "Hydraulic Design of Stilling Basins and Energy Dissipators" (Peterka, 1984).

Finally, riprap aprons abut the stilling basins to transition flow onto the basin floor; riprap in these areas was assessed for scour protection with the Schoklitsch equation. Another scour calculation method, Zimmerman and Maniak, was assessed and found to generate negative values which implies a non-scouring condition. Riprap apron length was first calculated to armor the scour hole depth calculated with the Schoklitsch equation (assuming 2:1 H:V slopes from the bottom of the scour hole, per riprap launching guidance in FHWA's HEC-11, this resulted in a minimum apron length of 2 times the scour calculated scour depth). Riprap apron lengths were increased to round numbers as well as based upon engineering judgment and varied in width for visual effect. While this method dictated the minimum

allowable riprap size, larger stone was specified at the edge of the basin inlet structure in Phase IIIA1. Design calculations are contained in **Appendix D.**

Per Chanson, 1996, the stepped basin inlet structure in Phase IIIA1 possesses a "shallow" slope of 0.1 ft/ft and the stepped basin inlet structure in Phase IIIB has a more typical average slope of 0.25 ft/ft. Despite differences in slope, both structures develop skimming flow, although homogeneous aeration of flow is not developed on the structure in Phase IIIA1. Fully aerated values were used to design structure wingwall heights, which represents a conservative estimate of flow depth.

Aesthetic considerations, such as curved steps and inset boulders (Phase IIIA1 only) have been incorporated into the inlet structure designs. These considerations have been accounted for in design calculations by considering arc lengths at structure limits and reduced effective areas due to boulder placement. Flow expansion and contraction on the structures due to these elements has not been explicitly considered as it is not well addressed in design literature, however, adding two-dimensional flow components to an otherwise one-dimensional feature and analysis suggests additional, and likely minor, energy dissipation may result. Variable step geometry, particularly in the Phase IIIA1 inlet structure, is non-typical in stepped structure analysis. To accommodate this variability, computations were performed for each step width independently.

"Rock chute" downdrains were designed to pass local runoff from the basin rims to the basin floors. Although grouted rock has been specified for the chute material, design methodology based upon USDA's *Design of Rock Chutes* (Robinson, 1998) was utilized which assumes loose rock for the construction material. Robinson, 1998 also describes a methodology for sizing and determining the extent of a downstream riprap apron which distributes the flow over the basin floor. All chutes were designed for one foot of depth and varying width to accommodate the design discharge. Design discharges were taken from FLO2D analysis, rational calculations, or HEC-1. While most chutes pass less than 20 cfs, one chute into the detention basin in Phase IIIB carries approximately 200 cfs – this chute is the widest in either project and utilizes D_{50} = 24" riprap at its base. Design calculations for the rock chutes are contained in **Appendix D**. These calculations include riprap sizing calculations for the two lateral collection channels (one in Phase IIIA1 at the southern end of the project adjacent to Empire Boulevard and the other in Phase IIIB near the northern end of the project between Mews Road and Marsh Road). A comparison of the maximum discharge indicated in the project FLO2D model versus the result of distributing the HEC-1 discharge over the edge of the channel was made to calculate the appropriate riprap size.

Headwall Aprons

A 2 ft wide riprap apron has been included along the back edge of headwalls within the project. The riprap apron is intended to prevent excessive rill formation due to localized nuisance flows concentrating and running along the back of the headwall. FCDMC has further requested JEF incorporate aprons with larger extents at detention basin inlets and outlets to prevent slope erosion due to secondary velocities. JEF investigated the magnitude, likelihood and extent of secondary velocities at basin inlets and outlets in multiple design guidelines published by the Federal Highways Administration, US Bureau of Reclamation, and the Flood Control District of Maricopa County, and found no design guidance related to the size and extent of protection in these areas. Based upon a macro-level analysis,

the velocity in a ponded condition, such as that found in a detention basin, is 0 ft/s and the energy grade line elevation is equal to the hydraulic grade line elevation. Based upon this scale of analysis, no protection would be warranted, however, experience dictates that erosion does occur in these areas when left unprotected. Therefore, FCDMC suggests that this additional protection may be warranted. Based upon this concern, D_{50}=6" riprap has been placed along the slope extending 15' from the back edge of the structures laterally and up-slope. The 15' apron was reduced to a 2' width above the 100-yr water surface elevation to conform to the rill-prevention strategy. Riprap with D_{50}=6" was selected for these aprons as it is the minimum practical riprap size recognized by FCDMC and larger stone was not warranted based upon engineering judgment. Based upon the Isbash methodology presented in FCDMC's Drainage Design Manual Hydraulics, D_{50}=6" riprap can withstand velocities up to 6.46 ft/s in this installation. In the absence of specific design guidance and this velocity threshold, D_{50}=6" riprap has been specified at basin inlet and outlet headwall aprons.

C. Sediment Control

The two detention basins, Basin A and Basin B, are designed to receive both storm water runoff and the sediment that is carried by the storm water. The sediment yield estimates at these two locations are used to evaluate the required basin capacity for storage of sediment. The sediment yield estimates are computed using the criteria specified in the Hydraulic Design Manual (FCDMC, 2010) and the Drainage Design Management System (DDMSW) Software Version 4.5.6. The sediment yield estimation involves computation of the washload component as well as the bedload component. The washload component of the sediment transport is estimated using the MUSLE procedure developed by the Natural Resources Conservation Service (NRCS) or its predecessor, the Soil Conservation Service (SCS) while the bedload component is computed using the Zeller-Fullerton sediment transport equation. The sediment delivery ratio (SDR) is applied to the MUSLE sediment yield estimates to arrive at the washload estimates for the basins. The washload and bedload estimates are performed to obtain the average annual sediment yield estimates as well as the 100-year flood event sediment yield. The total sediment volume estimates for both basins are then computed for 3-year and 5-year maintenance period scenarios. The total sediment storage volume proposed for the basins is the sum of the sedimentation that can occur during a single maintenance period plus a single 100-year event.

1. Sediment Yield

The DDMSW software Version 4.5.6 is used to perform the sediment yield computations. Descriptions of the data utilized to perform the analysis within DDSMW software are presented in the following sections.

Storm Runoff

The 100-year 6-hour HEC1 model is used to obtain the peak discharge and volume for the 100-year flood event. The values of the peak discharge and volume estimates for 2-year, 10-year, and 50-year flood frequency events are then computed using the results from the 100-year 6-hour HEC1 model and applying appropriate discharge-ratios to the peak flows. These discharge ratios are based on the Hydrology Design Manual of the Flood Control District of Maricopa County (FCDMC, 2010a).

Soil Erodibility Factor (K)

The soils data provided by the District is used to estimate the soil erodibility factor. The soil erodibility factor (K) for each soil is obtained from the soils coverage for the upper watershed areas. The soil survey also describes the composition of each map unit as a percentage of each soil series. The K value for a particular map unit is based on the "Dominant Condition" method as described in River Mechanics Manual (FCDMC, 2010c). Further, several map units cover each sub-basin. Thus, the value of K for a particular subbasin is a weighted average of the weighted map unit K-values occurring in the basin. These computations are performed internally within DDMSW software based on the subbasin coverage and soils coverage. The default values used by DDMSW for each map unit and the weighted average values computed for each subbasin are presented in **Appendix E**.

Topographic Factor

The 10m DEM USGS topographic mapping is used to compute the average slope and slope length in the upper watershed area. The slope length and slope angle calculations were performed using the GIS based procedure presented in Hickey, 2000. The calculations are performed on a rectangular point grid overlapping the subbasin area. The procedure can be briefly described by the following steps:

- Obtain elevation data in a grid format.
- Eliminate sinks in the elevation data.
- Determine flow directions at each grid point using maximum downhill slope to adjacent grid point.
- Identify high points as grid points with no net inflow (this was done using the flow directions at the adjacent grid points).
- Estimate a non-cumulative slope length at each grid point which is the slope length contributed by the cell area of the particular grid point.
- Track the flow path in the downstream direction from a high point using the flow directions. The tracking stops when the flow path reaches a pour point or the slope break is greater than 50%.
- Determine the slope length using the flow path and summing the non-cumulative slope lengths.

The elevation data are obtained within Arc GIS from the topographic mapping using 10m USGS DEM Rasters. The elevation data on the rectangular grid requires some smoothing to eliminate sinks which disrupt accurate computation of the slope lengths. The elimination of the sinks is performed using the algorithm provided in Planchon and Darboux (2001). Using the "adjusted" elevation data and the procedure outlined above, the slope length and slope angle calculations are performed for all the points that fall inside the subbasin area. Using these values, average values of the slope length and slope angle are obtained for all the subbasins. These values are then entered into DDMSW and used internally by DDMSW in the computation of topographic factors.

Cover and Management Practice

The cover and management factor, C, is categorized under three types of effects:

- C_I – Effects of canopy cover
- C_{II} – Effects of mulch or close-growing vegetation in direct contact with the soil surface
- C_{III} – Tillage and residual effects of the landuse.

The cover and management factor, C, is then defined as product of these three values ($C_I \times C_{II} \times C_{III}$). The District has developed a table from which DDMSW software automatically computes the cover and management factor. The values computed by DDMSW software are included in **Appendix E.**

Erosion Control Practice Factor (P)

The erosion control practice factor accounts for conservation practices such as contouring and terracing. In desert rangeland and open space areas it can be reasonably assumed that no such activities have taken place and the factor can be assigned a value of 1.0. A value of 1.0 is computed internally by DDMSW using the landuse coverage for all the subbasins.

Sediment Gradation

Sediment samples collected as part of the Sonoqui Wash Channelization Phase III project are used in locations upstream of the basin. The sediment gradation data was used in the estimation of D_{16}, D_{50} and D_{84} used in the computation of the Zeller Fullerton equation. The sediment gradation data are presented in **Appendix D**.

Channel Cross-Section for Bedload Computation

The 1-ft topographic mapping is used to estimate bedload sediment transport that occurs upstream of the basins. The cross-sections used in the computation of bedload transport are obtained using ArcGIS software with 3D Analyst extension. The cross-section data was entered into the DDMSW and is used internally to compute the hydraulic parameters using normal depth assumption. Three channels were identified as Channel A, B and C with Channel A and B draining into Basin B and Channel C draining into Basin A. The locations of these cross-sections are included in **Appendix E**.

2. Basin Sediment Storage Volume

The sediment volumes entering the two basins are determined by identifying the contributing subbasins for washload and contributing channel inflows for the bedload. The sediment yield estimates obtained for these subbasins and channel inflows are summed to provide the total sediment yield into the basins. The total sedimentation volume estimate is computed using the sum of the sedimentation that can occur during a single maintenance period and a single 100-year event. The computations are performed for both 3-year and a 5-year maintenance cycle and results are presented in **Table** III-6.

Table III-6 Sediment Yield Results

Sediment Source ID	Average Annual Sediment Yield (ac. ft.)	100 year Sediment Yield (ac. ft.)	3 Year Maintenance Total Sediment Yield (ac. ft.)	5 Year Maintenance Total Sediment Yield (ac. ft.)
S330A	0.133	0.807	1.206	1.472
S330B	0.068	0.412	0.616	0.752
S330E	0.004	0.025	0.037	0.045
S330C	0.077	0.467	0.698	0.852
CHNL A	0.107	0.53	0.851	1.065
CHNL B	0.037	0.246	0.357	0.431
Total sediment yield for BASIN B			**3.765**	**4.617**
S340B	0.069	0.416	0.623	0.761
S340A	0.002	0.01	0.016	0.02
S330D	0	0.004	0.004	0.004
CHNL C	0.3	1.553	2.453	3.053
Total sediment yield for BASIN A			**3.096**	**3.838**

D. FEMA Floodplain & CLOMR

The FEMA Conditional Letter of Map Revision (CLOMR) has been provided to the District and subsequently submitted to FEMA for review for Phase IIIA1. No CLOMR or LOMR documentation has been prepared for Phase IIIB and IIIA2. The Phase II and III CLOMR Technical Data Notebooks were submitted at the same time to facilitate the review by FEMA. This is partly due to the fact that the Phase III CLOMR is dependent on the Phase II CLOMR for its downstream tie-in. As a result the timing of review and approval of the two submittals is important. The Phase III CLOMR is documented in *"Sonoqui Wash Phase III- Sonoqui Wash from Riggs Road to Empire Boulevard, Conditional Letter of Map Revision (CLOMR) – Technical Data Notebook"*, FCD 2009C011, October 2011. The effect of the ultimate LOMR will be to contain the 100-year floodplain within the basins and pipes removing residents from the floodplain.

IV. COMMUNITY CONTEXT

A. Public Involvement

The project is located within unincorporated Maricopa County; therefore, there is no municipal funding partner for cost-sharing of the project. Even so, there are a number of project stakeholders who have an interest and will be impacted by the project including:

- Town of Queen Creek
- Pinal County

- Maricopa County Department of Transportation (MCDOT)
- Maricopa County Parks & Recreation Department

The Town of Queen Creek, Pinal County, and MCDOT are partnering on projects involving Ellsworth Road, Empire Boulevard, and Hunt Highway improvements that include the box culvert that is the southerly terminus of Sonoqui Wash Main branch.

The proposed design was presented to local residents at 3 public meetings held at various milestones during the development of the project design. The meetings were held at the Queen Creek Public Library on the following dates;

- July 29, 2010
- January 25, 2011
- July 19, 2011

Local residents potentially impacted by the project attended to go over the designs with the project team. Written comments were received and incorporated into the design as appropriate.

B. Aesthetics & Multi-Use

1. Open Channels

Most of the storm water is conveyed through a subsurface storm drain system that runs under roadways and through portions of residential parcels. Due to the alignment, there is no opportunity for recreation joint use potential including trails. While the 202nd Street channel is open at the northern outfall, there are constraints on the channel width. This resulted in a channel alignment that has very limited variation in horizontal or vertical differentiation. The Main Branch channel is open at the northern outfall and the grading concept was originally proposed to have a meandering alignment with varying widths of channel top and bottom and varying slopes. However, due to right-of-way limitations, the design was changed to a straight channel and context sensitive design goals were constrained. The main branch segment from Mews Road to the Phase II channel confluence will be incorporated into the Town of Queen Creek equestrian system. Design features to accommodate equestrian use include providing a 1 foot thick native soil layer over the riprap channel bottom and providing a 4 foot wide decomposed granite surface adjacent to the maintenance access ramp to allow equestrian ingress and egress from the channel.

2. Detention Basins

The basin sizing and design allows for grasses, wildflower mixes and trees within the basin. Due to the need for regular maintenance and removal of sediment in the low flow area, seed mix and tree planting is not provided in low flow channel areas. The basin size accommodates increased levels of flora within the basin side slopes. The basin form is sinuous with a meandering top and bottom slope. Any structural elements will reinforce the rural character that the residents preferred. Both basins are designed as a semi-soft structure. The primary landscape design theme will use Natural Lower Sonoran Desert Riparian landscape.

202nd Street Basin

Due to the limitation of available ROW, there may be only a limited opportunity for passive recreation. The grading and planting are designed to provide a buffer to adjacent residential development while providing view windows into the basin from the adjacent roadway.

Main Branch Basin

During preliminary design, the size of the basin seemed to eliminate the potential for active recreation. However, as we developed the final plans, an area that can accommodate a multi-use soccer field was integrated in the southwest corner. The area has been graded to accommodate this use, but will not be developed by the District. If a project partner is identified, this area can be developed into a multi-use field. The maintenance access roads have been designed to accommodate ADA access into the basin for potential future use. The low flow channel meanders along the bottom of the sediment basin to provide visual movement of the sediment basin bottom. Similar to the 202nd Street basin, grading and planting have been designed to buffer adjacent residential development while providing view windows into the basin as open space features. Collector channels have been incorporated along the north side of Hunt highway and will be designed with the same parameters as the channels and basins.

C. Right-of-Way & Easements

The project area is within a county island and is heavily subdivided. Most parcels are one acre in size or larger. The types of structures include temporary trailers, manufactured homes, small businesses, and newly constructed residences exceeding 2,000 square feet in size.

Roadway right of way (ROW) and parcel access are not uniformly applied within the project area. Dedicated paved roads on the site are limited to Riggs Road, Ellsworth Road, Empire Blvd, Hawes Road, and San Tan Blvd. The east/west roadways, Riggs Road, San Tan Blvd, Happy Road, and Empire Blvd are aligned within dedicated ROW. Mews Road ROW is evident from Hawes Road to the 204th street alignment. However, no public ROW has been dedicated for any of the north/south roadways within the study area. Many of these roadways are simple dirt roads receiving limited maintenance. Evidence that the north/south roadways double as storm water conveyance pathways is prevalent along 202nd Street.

Proposed easements will include both Temporary Construction Easements (TCE's) and Drainage Easements (DE's) along with an SRP Maintenance Access Easement (AE). Additionally, there are TCE's identified along the drainage channel/storm drain pipe alignment and DE's around some of the drainage structures. The TCE limits were laid out to provide the contractor vehicular access and material storage zones and were provided along both sides of the storm drain alignment depending on what created the smallest impact. The DE's were laid out to provide FCDMC access for maintenance to the drainage structures. The AE was laid out to provide SRP access to their utility poles. The 202nd Street storm drain TCE limits vary from 30 to 50 feet from the outer cut line with areas along the constrained side (existing overhead power/power poles and property structures) ranging from 2 to 20 feet from the outer cut line for additional work space area. The Main Branch storm drain alignment maintains 20 to 30 foot TCE limit swath from each side of cut line along the entire proposed drainage corridor.

D. Utilities

Existing water utilities were traced horizontally to determine the location of these facilities. It should be noted that services from the water meters to the homes appear to be plastic and could therefore not be

located using horizontal locating equipment. Additionally, there do not appear to be major utilities within the project limits with the majority being 6-inch and 8-inch waterlines. Where the drainage facilities cross water lines, the 6-inch and 8-inch water lines will be dipped under the concrete box culverts and concrete pipes and DIP will be used. Where the contractor comes in contact with previously unknown PVC water service lines, the contractor will simply relocate these lines vertically above the drainage pipes and box culverts.

SRP has a number of existing power poles located along the drainage facilities alignment, and these will be relocated by SRP prior to construction.

Additionally, septic tank permitting diagrams have been obtained through the Environmental Services On-Site Wastewater Program. These diagrams have been evaluated and sites along the Western Alignment and the Main Branch alignment were evaluated to determine if the leach field will be affected by TCE's and cut slopes, and there should be no critical impacts.

E. Public Access & Traffic Control

The proposed storm drains are to be constructed within rural roadways that provide access to residences on both sides of the roadway. The contractor will be required to provide pedestrian access to the homes at all times. Vehicular access shall be maintained to each home at all times except for the following; it will be allowed that each home not have vehicular access for one night provided that 2 days notice be provided, in writing, to the homeowners, and that the contractor provide a parking area not more than 100 yards away from the driveway. This requirement shall include any access to any homes directly adjacent to the storm drain construction. Access shall be maintained at all times for all homes not directly adjacent to the storm drain construction.

For all east-west streets two lanes of traffic shall be maintained at all times.

For all north-south streets two lanes of traffic shall be maintained at all times, or one lane of traffic will be allowed provided flaggers are provided.

V. LAND & RESOURCE CONTEXT

A. Geotechnical Conditions

The geotechnical conditions within the project limits have been investigated by Kleinfelder and documented in the *Geotechnical Study Report, Sonoqui Wash Channelization Project, Phase III*, dated September 10, 2010 and provided under separate cover. *Geotechnical Study Report* highlights include:

- Native alluvial soils consist of silty sand, clayey sand, silty-clayey sand, sandy clay, sandy-silty clay, and sandy silt were encountered to the maximum drilled depth of approximately 36.5 feet. Results of consolidation tests performed on the samples or the alluvial soils indicate they have low to moderate potential for collapse when wetted under a 1 ksf surcharge.

- Groundwater was not encountered.

- Conditions encountered in the exploratory borings suggest the majority of the soils to be encountered in excavations will consist of silty and clayey sand. These soils appear to have sufficient cohesion and they are expected to be relatively stable during the placement of storm drain pipe, including cast-in-place pipe. Though not noted in the borings, within the anticipated depth of excavation, clean sands that are prone to running may be encountered.

B. Hazardous Materials

The emphasis of this review is to obtain site specific data in evaluating the project area for the presence of recognized environmental conditions (REC's) resulting from past or present uses, as defined by ASTM Standard Practice for ESA's (E1527-05) which could potentially adversely impact each of the alternative alignments.

A *Phase I Environmental Site Assessment, Sonoqui Wash III* was completed by Converse Consultants on May 24, 2011 to determine the potential presence of any environmental impacts which may hinder the construction of the project. Results of the survey indicated that there was no evidence of recognized environmental conditions found within the proposed project alignment.

C. Environmental & 404 Permit Issues

Prior to the alternatives analysis and during the data collection activities, an inventory of environmental conditions including but not limited to the Clean Water Act Section 404 jurisdictional areas, hazardous material sites, cultural resource sites and the biological and ecological surveys was conducted by the district to identify potential adverse environmental impacts within the preferred project alignment and boundaries. The results of the inventory are described below:

1. Section 404 of the Clean Water Act

The regulatory program administers and enforces Section 10 of the Rivers and Harbors Act of 1899 and Section 404 of the Clean Water Act. Under Section 10, a Corps permit is required for work or structures in, over or under navigable waters of the United States. Under Section 404, a Corps permit is required for the discharge of dredged and fill material into waters of the United States.

A preliminary jurisdictional delineation was completed by the District and determined that the diagonal wash may be a "waters of the US". Consequently a Section 404 Permit application including a preliminary jurisdictional determination was submitted to the COE for the proposed impacts to the wash. Currently the permit application is still in review by the COE and approval is pending.

2. Cultural Surveys

The archeological work was being conducted in compliance with among others a) the National Historic Preservation Act of 1966, b) the Arizona Antiquities Act of 1960 (Arizona Revised Statute [A.R.S.] Section [§] 41-841 et seq.), and c) the State Historic Preservation Act of 1982 (A.R.S. § 41-861 et seq.). The principal concern of all three laws is the realistic protection of all important cultural properties that might otherwise be impacted or destroyed during various undertakings such as geotechnical work and project construction.

A cultural survey *An Archaeological Inventory of the Sonoqui Wash III Drainage Project Area in Maricopa and Pinal County, Arizona* was completed by Scientific Archaeological Services (SAS) on June 28, 2011.

The results of the survey determined that some isolate artifacts were found but the proposed project should have no effect upon any historic resource. In keeping with the compliance process a copy of the report was submitted to the COE and the SHPO for concurrence of "no effect" from the project.

3. Biological and Ecological Surveys

A list of federally listed Threatened and Endangered Species for Maricopa and Pinal County was obtained from the United States Fish and Wildlife Services (USFWS), to determine the presence of any of these species within the study area. Results of the survey indicated that there were no T&E species identified within the project footprint. Additional field reviews conducted by the District supported these findings and determined that there was scarce habitat within the footprint of the proposed project.

VI. PROJECT CONSTRUCTION

A. Survey & Construction Control

The survey for this project is tied into the Maricopa County Department of Transportation Geodetic Densification and Cadastral Survey (GDACS) network control. This system is based on the North American Vertical Datum of 1988 (NAVD 88).

To be consistent with the designs from the previous segments, the design for the Sonoqui project use NAVD 1988 vertical datum and the NAD 1983 horizontal datum. The horizontal data is provided in the Arizona State Plane Coordinates (Central Zone) and converted to ground using a factor of 1.000154783.