PRELIMINARY (STAGE II)
CROSS DRAINAGE REPORT
FOR
TANGERINE ROAD CORRIDOR STUDY –
INTERSTATE-10 TO LA CANADA DRIVE

VOLUME 1 OF 2

Location:
Portions of Sections 31 through 36 of Township 11 South, Range 12 East
Portions of Sections 31 through 34 of Township 11 South, Range 13 East
Portions of Sections 1 through 6 of Township 12 South, Range 12 East
Portions of Sections 3 through 6 of Township 12 South, Range 13 East
Pima County, Arizona

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SECTION 1.0 INTRODUCTION

This Stage II Drainage Report has been prepared to provide the Town of Marana, Pima County and Town of Oro Valley with quantitative information on existing surface drainage along Tangerine Road between Interstate 10 (I-10) and La Canada Drive, and to provide a cross-drainage plan for the proposed roadway widening. It's the second in a series of reports that will accompany and compliment a Design Concept Report (DCR) and roadway improvement plans that are being prepared for the project by Psomas. The report format is based on the suggested Drainage Report Table of Contents contained in Section 3.11 of the Pima County Roadway Design Manual, 2010 version (2010 RDM).

The current project scope is for completion of design reports and plans through the DCR (approx. 30% design) phase of the Tangerine Road improvement project. Future drainage design submittals will include a Stage IV Drainage Report that will address comments from this Stage II submittal, provide additional details for proposed drainage structures, and will refine the drainage design during the future construction-level stages of the project.

1.1 Project Description

The Tangerine Road improvement project is located in the Town of Marana, unincorporated Pima County, and the Town of Oro Valley, Pima County, Arizona. The roadway alignment follows along the southern boundary of Sections 31 through 36, Township 11 South, Range 12 East; Sections 31 through 34, Township 11 South, Range 13 East; and the northern boundary of Sections 1 through 6, Township 12 South, Range 12 East; and Sections 3 through 6, Township 12 South, Range 13 East, G&SRB&M. The limits span approximately 9.8 miles, extending from I-10 on the west to La Canada Drive on the east. A vicinity / location map for the project is presented as Figure 1.

The existing roadway section is a paved, non-curbed two-lane roadway. The prevailing drainage flows from northeast to southwest along and through the project limits. Sheet flow is the prevailing drainage pattern for the segment from I-10 to Dove Mountain Boulevard, while channel flow is the main drainage pattern for the segment from Dove Mountain Boulevard to La Canada Drive. There are several drainage culverts beneath Tangerine Road from Dove Mountain Boulevard to La Canada Drive, however, storm water flows over the road via at-grade crossings (dip sections) at most locations. Between I-10 to Dove Mountain Boulevard, there are only two below-grade drainage structures because sheet flow is present throughout the area.

The proposed project consists of improvement of Tangerine Road to a four-lane divided cross section. The roadway cross-drainage designs support a four-lane roadway. However, it is intended that the roadway will ultimately be improved to a six-lane section. The proposed Tangerine Road alignment will take the future six-lane section into consideration to minimize future roadway expansion costs. Therefore wherever possible, the outside limits to the north of the four-lane roadway will be the future six-lane roadway's north limit. Thus, drainage structures, such as interceptor channels, roadside ditches, and inlets to the cross-drainage culverts, along the north side of the four-lane roadway will not need to be re-constructed when it expands to six-lane roadway, and future expansion will incur much less costs associated with drainage structures.

1.2 Major Drainage Features

Major watersheds that intersect the project emanate from the Tortolita Mountains and include North Ranch Watershed, Canada Agua East Watershed, Canada Agua West Watershed, Prospect Canyon Watershed, Ruelas Canyon Watershed, and Wild Burro Watershed. The prevailing offsite drainage patterns are dispersed sheet flow from I-10 to Dove Mountain Boulevard, while mainly riverine or channel flow conditions prevail from Dove Mountain Boulevard to La Canada Drive. The Special Flood Hazard Area (SFHA) shown on the effective Flood Insurance Rate Map (FIRM) panels within the project limits supports the prevailing offsite drainage patterns discussed previously. Areas west of Dove Mountain Boulevard are predominately Federal Emergency Management Agency (FEMA) Zone AO, while areas east of Dove Mountain Boulevard are predominately riverine flow with isolated SFHAs at major wash crossings. FIRM panels, 04019C1045L, 04019C1065L, 04019C1080L, and 04019C1090L, with an effective date of June 16, 2011, cover the project limit. These FIRM panels are shown in Appendix B.

On the west portion of the project approaching I-10, an approximately 3300-foot long earthen berm, which is approximately 1 to 3 feet in height, is located adjacent to the south edge of the existing roadway. This berm prevents offsite sheet flow from north from being conveyed across the road right-of-way and onto adjacent lands to the south. Consequently, existing drainage at this location is diverted westerly on and alongside the roadway pavement next to the berm until reaching low-lying flat areas adjacent to the Union Pacific Railroad (UPRR), where it ponds and eventually drains back over Tangerine Road and on to the north along the railroad.

1.3 Proposed Improvements

The planned roadway improvements include reconstruction and widening of the existing roadway to a four-lane divided section with cross drainage structures, roadside landscaping, and pedestrian facilities for a portion of the roadway. Preliminary design parameters are that from I-10 to Thornydale Road, the proposed roadway will have a depressed median and no outside curbs; while from Thornydale Road to La Canada Drive, the proposed roadway will have a raised median and no outside curbs.

Cross drainage improvements include upgraded roadside drainage systems consisting of swales/channels designed to convey flows along the roadway to new cross drainage culverts that will replace existing undersized culverts, or existing at-grade dip crossings. New pavement drainage storm drain systems may be needed at roadway intersections and they may drain to the cross drainage culverts. At the west end of the project, a regional interceptor channel is necessary along the north side of the Tangerine Road to convey offsite runoff westerly and to direct it to the existing flow path along the UPRR. Drainage design alternatives for the west end of the project have been evaluated and are documented in a separate West End Regional Drainage Analysis that has been included in Appendix K of this report.

1.4 Design Criteria

The drainage design criteria for this project follow the standards outlined in the Pima County 2010 RDM. This reference establishes the hydrologic design frequency for cross drainage structures to be the 100-year return period event. A Technical Memorandum that documents the project hydrologic and hydraulic design parameters has been prepared and included in Appendix A.

As shown on Figure 1, the project limits fall within three governmental jurisdictions; Town of Marana, unincorporated Pima County, and Town of Oro Valley. The three jurisdictions agreed during early project scoping meetings that Pima County hydrology methods (PC-Hydro or HEC-1 per Pima County Regional Flood Control District (PCRFCD) Technical Policies TECH-015 & TECH-018) or FLO-2D two-dimensional flood modeling (based on the Town of Marana 2009 FEMA flood study) would be used for the project's hydrologic computations.

Drainage areas for offsite watersheds vary from approximately 4 acres to over 5600 acres. Drainage patterns for watersheds from I-10 to Dove Mountain Boulevard are prevailing sheet flow on the Tortolita Mountain alluvial fan; while the drainage patterns for watersheds from Dove

Mountain Boulevard to La Canada Drive are predominately riverine in nature. The large variation in watershed sizes and drainage patterns warrant using different methods to compute discharge rates for differing watersheds. These watersheds are divided into regional watersheds and local watersheds. The regional watersheds have drainage areas over 1 square mile, while the local watersheds have less than 1 square mile of drainage area.

For local watersheds, 100-year peak discharge rates were calculated using PC-Hydro, Version 5.4.2, which is developed in conformance with the Pima County Hydrology Procedures (Pima County, 1979, Revised March 2007). The PC-Hydro program incorporates rainfall depth information from the intensity-duration-frequency data from NOAA Precipitation-Frequency Atlas 14 of the Western United States, Volume I, Version 4, NOAA National Weather Service, Silver Spring, Maryland (G. M. Bonnin, et al., 2006). Watershed parameters for these local watersheds were determined according to the methods described in the Pima County Hydrology Procedures User's Guide (Pima County, 2007) and using engineering judgment guided by the topography, aerial photography, field reconnaissance, and GIS-based land use information. Local watersheds west of Dove Mountain Boulevard are generally represented by sheet flow conditions and thus a basin factor of 0.06 was used in computing their 100-year discharge rates. Local watersheds east of Dove Mountain Boulevard are generally represented by riverine flow conditions with low density residential development (<1 residence / acre) and thus a basin of 0.035 was used in computing their 100-year discharge rates.

For regional watersheds, their 100-year peak discharge rates were calculated using a modified FLO-2D model from the FEMA approved Town of Marana Tortolita Alluvial Fan Study or HEC-1 models. Parameters and methodologies used in hydrologic computations for the regional watersheds were determined according to PCRFCD's Technical Policies TECH-015 *Acceptable Methods for Determining Peak Discharges* & TECH-018 *Acceptable Model Parameterization for Determining Peak Discharges*.

Point rainfall depths were derived from the rainfall intensity-duration-frequency data from NOAA Atlas 14. Rainfall depths for the upper 90% confidence limit for 3-hour storm duration were determined and are summarized in a Figure (Rainfall Data and Aerial Reduction Factor for Major Watersheds) in Appendix C.

Areal reduction factors, based on the major watersheds listed in Section 1.2 above, were used to convert point rainfall to equivalent depths of rainfall spread over the watershed under design, based on procedures found in Section 3.3.3 of Arizona State Standard SS 10-07. The eastern Arizona depth-area reduction factors were applied to the point rainfall input.

For HEC-1 models, rainfall losses were computed using the Natural Resources Conservation Services (NRCS) curve number methodology (U.S. Department of Agriculture). Watershed runoff was computed using the NRCS's Technical Release-55 methodology (TR-55, 1986). Modified Puls normal depth storage routing was used to route runoff between sub-basins. A typical Modified Puls normal depth storage-outflow was applied in the HEC-1 routing. Eight-point cross sections were used to represent the routing channels.

For FLO-2D modeling, flood hydrographs that were developed by HEC-1 models at the apexes of the alluvial fans were input as inflow hydrographs. Rainfall was also provided on the FLO-2D study areas. A combined NRCS Runoff Curve Number / Green and Ampt Methodology available within FLO-2D was used for rainfall/runoff losses. This methodology uses the NRCS procedures to determine runoff depth and infiltration losses associated with direct rainfall on the fan. The Green and Ampt methodology are executed in the FLO-2D model when the depths of flow within grids exceed the accumulated rainfall depth.

Existing hydraulic conditions (inundation limits, flow depths and velocities) for at-grade crossings in the predominantly sheet flow alluvial fan areas west of Dove Mountain Boulevard were based on floodplain mapping depths determined by 2-dimensional FLO-2D hydraulic modeling from the Town of Marana Tortolita Floodplain Study. For at-grade crossings in the more riverine conditions found east of Dove Mountain Boulevard, 1-dimensional hydraulic modeling using HEC-RAS was used. Headwater depths and ponding limits for all existing culverts were determined using FHWA HY-8 computer software.

Proposed culverts were designed to convey the full 100-year flows beneath the roadway with the exception of the west end study area noted in Section 3.1 of this report. Maximum headwater elevations were kept at, or below roadway subgrade level. Wildlife crossings were incorporated into some of the drainage culvert locations, thus requiring special consideration regarding culvert size as well as inlet and outlet treatments.

Collector channel designs included conveyance capacity for 100-year storm events plus freeboard within erosion protected limits of the channels. This maximizes containment of ponding on the right-of-way and potential erosive longitudinal flows within the erosion protected cross section. Per the Town of Marana's instructions, interceptor channels will be further evaluated to provide a minimum 1-foot freeboard along the north bank during the project's final design phases.

Low head room and flat drainage conditions necessitate that the roadway and drainage design allow flows from larger storm events to overtop the road in the western approximate 1.3 miles of the project area. An interceptor channel drainage system design that provides roadway flood protection from lesser floods (up to the 10-year flood) has been proposed and is detailed in the West End Regional Drainage Analyses report in Appendix K. Preliminary West End Regional Interceptor Channel plans have been prepared and are included in the Tangerine Road 30% Roadway and Drainage Improvements plan set being submitted by Psomas.

SECTION 2.0 EXISTING CONDITIONS

2.1 Overview

The project is affected by drainage that flows southwesterly and across the roadway through either at-grade crossings or existing cross drainage culverts. The contributing watersheds north of the roadway originate from the Tortolita Mountains and foothills. The slopes for the watersheds in the vicinity of the project are generally from 1 to 3%.Existing development from I-10 to Dove Mountain Boulevard is minimal, except for those residential subdivisions constructed in conjunction with the Dove Mountain Master Planned community. Some of the developments provide detention/retention facilities, while others do not. All of those residential subdivisions fall within the regional watersheds. The Existing land uses east of Dove Mountain Boulevard are mainly low density development (<1 house per acre) or vacant land. Medium to high density developments within this segment include The Preserve at Dove Mountain subdivision, Tortolita Vistas subdivision, Tangerine Crossing subdivision, and several commercial centers adjacent to Dove Mountain Boulevard and Thornydale Road. These developments provide detention/retention facilities except for a portion of the Preserve at Dove Mountain subdivision. Future land uses for the watersheds that impact Tangerine Road are generally low density development (< 1 house per acre), except for the previously mentioned medium to high density developments. Future land uses are considered in the hydrologic computations.

Vegetation in the contributing watershed areas is Desert Brush with an estimated 20% vegetation cover. Soils in the Tortolita Mountain areas are prevailing hydrologic soil type D. Down slope, in the Tortolita alluvial fan areas and west of Dove Mountain Boulevard, soils are mainly hydrologic soil type B. Soils east of Dove Mountain Boulevard are predominately hydrologic soil types B and C in the vicinity of the project. Hydrologic soil groups were determined from the soil data provided on the Pima County RFCD GIS internet site. Hydrologic soil type information is shown on Figure 2.

2.2 Existing Conditions Hydrology

There are seventy points of concentration for existing drainage designated in the vicinity of the project site. See the Existing Conditions Off-site Watershed Maps, Figures 3 & 4. Figure 3 shows the locations and watershed boundaries of the regional watersheds. The Time of Concentration for the biggest watershed (CP-68), computations for which are included in

Appendix C, is 2.92 hours. Therefore, a rainfall distribution for a 3-hour Pima County (SCS modified) Type II storm was used for the regional watersheds. For the three regional watersheds east of Dove Mountain Boulevard (CP-13, -19, and -32), 100-year peak discharge rates were determined by HEC-1 models. Parameters used in the HEC-1 modeling followed Pima County Regional Flood Control Technical Policies.

For the regional watersheds west of Dove Mountain Boulevard, the FLO-2D model in the FEMA approved Town of Marana Tortolita Alluvial Fan Study was modified and used to compute their 100-year discharge rates. The FLO-2D model, including HEC-1 models that provide inflow hydrographs for the FLO-2D model, were revised to use 3-hour Type II storm rainfall distributions instead of the 24-hour Type I storm in the FEMA approved models. Rainfall data used in the associated HEC-1 models vary depending on the locations of the subwatersheds (Appendix C: Figure: Rainfall Data and Aerial Reduction Factor for Major Watersheds and Figure: HEC-1 Watershed Boundaries and FLO-2D Study Limits for Tangerine Road Project). For the FLO-2D model, only Rainfall ID F (3.10" for 3-hour storm) is within the FLO-2D study area. Therefore, rainfall data at Rainfall ID F and the areal reduction factor for Prospect Wash watershed (0.826) were used to generate the equivalent rainfall depth (2.56") in the FLO-2D model. This is a conservative approach, because the Prospect Wash watershed, which is the smallest major watershed in size among the three major watersheds (Wild Burro Canyon Wash watershed, Ruelas Canyon Wash watershed, Prospect Wash watershed) that drains to Tangerine Road within the FLO-2D study area, has the biggest areal reduction factor. Aerial reduction factors for rainfall used in HEC-1 models, which were used to provide inflow hydrographs, are based on the drainage areas for each major watershed when they intersect Tangerine Road.

The grid size for this FLO-2D model is 100' x 100'. This grid size was not small enough to pick up many of the gully/ridge features on the alluvial fan surfaces. In the preliminary FLO-2D model runs, runoff was observed to flow from one watershed to another at locations where the aerial photos and topographic data indicate that no flow exchanges should occur. This is mainly because the 100' x 100' grids were not small enough to represent many of the gully/ridge features on the alluvial fan surfaces. However, to utilize the base FLO-2D model to the greatest extent possible, it was desirable to use the 100' x 100' grid FLO-2D without major revisions. To eliminate unjustified flow exchanges between regional watersheds, the FLO-2D model was revised to block grids along the regional watershed boundaries unless the aerial photos or topographic data suggested that split flows actually exist on those grids. Discharge rates for CP-52, -67, -68, -69, and -70 are determined by the FLO-2D model.

For CP-62 (drainage area = 686.3 acres), the 100-year discharge rate determined by the FLO-2D is approximately 8.5 cfs, which is unrealistically low. This is very likely due to the grid size (100' x 100') was not small enough to pick up the gully features within this watershed and thus unreasonably high rates of attenuation were produced from this watershed in the FLO-2D model. Considering the drainage area associated with this watershed is only slightly over 1 square mile, it was treated as a local watershed and PC-Hydro was used to compute its 100-year discharge rate. The PC-Hydro unit discharge result for CP-62 compared favorably with unit discharges from the other local watersheds in the study area, and are therefore considered much more reasonable compared to that from FLO-2D.

Regional Regression Equations for estimating peak discharges developed by USGS (Open-File Report 93-419) were used to verify the 100-year discharges rates for the watersheds, whose 100-year discharge rates are obtained by either the HEC-1 models or the FLO-2D model. The Regional Regression Equations are obtained by using least-squares multiple-regression analyses to the available gaged data in each region. This project site is within Southern Arizona Region 13 according to the USGS report. The Regional Regression Equation for estimating the 100-year discharge rates for this region is shown as follows:

$$Q_{100} = 10^{(5.52 - 2.42 A^{-0.12})}$$

Where, Q_{100} = the 100-year discharge rate, in cubic feet per second
A = the drainage area, in square miles

Compared to the 100-year discharge rates obtained by Regional Regression Equation, HEC-1 models generated higher 100-year discharge rates and FLO-2D model generally generated lower 100-year discharge rates. The only exception is CP-52 (FLO-2D study area), whose 100-year discharge rate is higher than that obtained by Regional Regression Equation. Higher 100-year discharge rates in the HEC-1 study areas are mainly due to the following reasons: Type II rainfall distribution, upper 90% NOAA 14 rainfall data, and no transmission losses. Alluvial fan greater than average surface runoff storage and transmission losses are the main reasons for lower 100-year discharge rates in the FLO-2D study area. For CP-52 (Prospect Wash), the location of CP-52 is only 4800 feet away from the location of its contributing inflow hydrograph (HEC-1). In addition, the channel is well defined between CP-52 and the HEC-1 inflow hydrograph location. Therefore, peak flow attenuation at CP-52 is not as significant as at other CPs in the FLO-2D model.

The Pima County hydrology method (computer program PC-Hydro) was used to compute the peak discharge rates for the local watersheds. The latest available rainfall depth-duration-frequency data from NOAA Atlas 14 (upper 90%) were used for all the watersheds for this project. Locations of these local watersheds and their watershed boundaries are shown in Figure 4.

The existing conditions concentration point runoff rates are summarized in Table 1. The 100-year discharge rates for the HEC-1 and FLO-2D watersheds are listed in Table 2. Hydrologic computation sheets from the PC-Hydro, HEC-1, and FLO-2D models are provided in Appendix C.

Table 1: Summary of Existing Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Q100 per Acre (cfs/acre)
		B	C	D				
1	89.8	7%	53%	40%	0.035	5	422	4.7
2	180.9	34%	36%	30%	0.035	5	558	3.1
3	7.8	0%	53%	47%	0.035	5	59	7.6
4	265.9	31%	33%	36%	0.035	5	639	2.4
5	53.3	13%	52%	35%	0.035	5	278	5.2
6	3.5	0%	53%	47%	0.035	5	26	7.4
7	185.4	67%	31%	2%	0.035	5	470	2.5
8	30.8	58%	42%	0%	0.035	5	145	4.7
9	34.5	88%	12%	0%	0.035	5	135	3.9
10	28.3	73%	27%	0%	0.035	5	123	4.3
11	35.7	84%	16%	0%	0.035	5	172	4.8
12	12.3	100%	0%	0%	0.035	5	60	4.9
13*	1,202.40	34%	2%	64%	0.035	5	2,683	2.2
14	17.9	100%	0%	0%	0.035	5	77	4.3
15	6.7	100%	0%	0%	0.035	5	40	6.0
16	37.1	98%	2%	0%	0.035	5	153	4.1
17	25.6	95%	5%	0%	0.035	5	126	4.9
18	4.3	50%	50%	0%	0.035	5	28	6.5
19*	2,317.40	15%	9%	76%	0.035	5	4,678	2.0
20	31.6	50%	50%	0%	0.035	5	168	5.3
21	86.7	50%	50%	0%	0.035	5	316	3.6
22	474	62%	34%	4%	0.035	5	1,110	2.3
23	1.8	50%	50%	0%	0.025	5	12	6.7
24	26.1	50%	50%	0%	0.025	5	157	6.0
25	72.8	50%	50%	0%	0.025	5	339	4.7
26.1	19.7	53%	47%	0%	0.025	5	111	5.6
26.2	1.1	53%	47%	0%	0.025	5	7	6.4

Table 1: Summary of Existing Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Q100 per Acre (cfs/acre)
		B	C	D				
27	9.5	50%	50%	0%	0.025	5	63	6.6
28	321.4	66%	34%	0%	0.035	5	810	2.5
29	27.6	50%	50%	0%	0.035	5	127	4.6
30	27.6	50%	50%	0%	0.035	5	140	5.1
31	4.7	50%	50%	0%	0.035	5	31	6.6
32*	1,170.90	15%	18%	67%	0.035	5	2,574	2.2
33	8.5	0%	53%	47%	0.035	5	64	7.5
34	2.6	0%	53%	47%	0.035	5	20	7.7
35	166.5	27%	39%	34%	0.035	5	549	3.3
36	9.9	0%	53%	47%	0.035	5	63	6.4
37	4.2	0%	53%	47%	0.035	5	32	7.6
38	3.3	0%	53%	47%	0.035	5	25	7.6
39	166.4	34%	35%	31%	0.035	5	546	3.3
40	95.9	24%	40%	36%	0.035	5	372	3.9
41	10.2	50%	27%	23%	0.025	35	80	7.8
42	112.2	34%	35%	31%	0.035	10	399	3.6
43	12.2	0%	53%	47%	0.025	20	97	8.0
44	133	1%	53%	46%	0.032	20	688	5.2
45	22.9	1%	53%	46%	0.025	5	172	7.5
46	79.1	40%	32%	28%	0.025	15	435	5.5
47	8.2	100%	0%	0%	0.025	5	50	6.1
48	13.1	100%	0%	0%	0.025	20	88	6.7
49	19.4	100%	0%	0%	0.03	10	94	4.8
50	4.3	100%	0%	0%	0.035	5	26	6.0
51	15	100%	0%	0%	0.035	5	77	5.1
52**	4,262.5	29%	5%	66%	N/A	5	5,074	1.2
53	39.3	100%	0%	0%	0.06	0	118	3.0
54	59.2	100%	0%	0%	0.06	0	124	2.1
55	86.4	100%	0%	0%	0.06	0	157	1.8
56	145.9	100%	0%	0%	0.06	0	232	1.6
57	13.4	100%	0%	0%	0.06	0	50	3.7
58	9	100%	0%	0%	0.06	0	35	3.9
59	90.7	100%	0%	0%	0.06	0	164	1.8
60	231.6	100%	0%	0%	0.06	0	312	1.3
61	31.4	100%	0%	0%	0.06	0	73	2.3
62	686.3	100%	0%	0%	0.06	0	708	1.0
63	19.8	100%	0%	0%	0.06	0	58	2.9
64	94	100%	0%	0%	0.06	0	161	1.7
65	22.5	100%	0%	0%	0.06	0	60	2.7
66	638.7	100%	0%	0%	0.06	0	563	0.9

Table 1: Summary of Existing Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Q100 per Acre (cfs/acre)
		B	C	D				
67**	3,591.70	43%	0%	57%	N/A	0	1585	0.4
68**	5,651.80	26%	0%	74%	N/A	0	2454	0.4
69**	1,965.10	96%	0%	4%	N/A	0	1526	0.8
70**	N/A	N/A	N/A	N/A	N/A	N/A	1,874	N/A

*obtained from HEC-1 model using Pima County's 3-hour Type II rainfall distribution
**obtained from the revised FLO-2D model for Tortolita Study with Pima County's 3-hour Type II rainfall distribution

Table 2: HEC-1 and/or FLO-2D 100-Year Discharge Rate Comparison

Concentration Point	WS Area (acres)	Q100 (cfs)	Q100 (cfs) per Regression Equation	Q100 Difference***
13*	1,202.4	2,683	1,890	42%
19*	2,317.4	4,678	2,794	67%
32*	1,170.9	2,574	1,859	38%
52**	4,262.5	5,074	3,913	30%
67**	3,591.7	1585	3,568	-56%
68**	5,651.8	2454	4,536	-46%
69**	1,965.1	1526	2,540	-40%
70**	N/A	1,874	N/A	N/A

*obtained from HEC-1 model using Pima County's 3-hour Type II rainfall distribution

**obtained from the revised FLO-2D model for Tortolita Study with Pima County's 3-hour Type II rainfall distribution
*** Q100 Difference is computed as percentage differences between the Q100 in this Report and the Q100 obtained from Regression Equation

2.3 Existing Conditions Hydraulics

Existing floodplain conditions within the road right of way were analyzed by different methods depending on the type of flow present within each area. The primary purpose of these analyses were to compare pre-project and post-project floodplain limits and water surface elevations so as to determine whether or not adverse changes will occur.

Flooding conditions west of Dove Mountain Boulevard are characterized as wide spread sheet flow as the entire land area within the right of way is inundated to depths varying between a few inches and 2 feet. Flow patterns throughout this area are also quite variable as the flow crosses the road at some locations and flows parallel to it at other locations. FLO-2D was determined to be the most appropriate method hydraulic modeling for this area. In the Tortolita Mountain Regional Study, FLO-2D was used to determine the SFHAs within the road right-of-way, which are shown on the effective FEMA FIRM panels. Therefore, flow depths within the road right of way were obtained from the effective FEMA FIRM panels. The predominant SFHA is FEMA Zone AO1 (1 foot flooding depth) with Zone AH adjacent to the Railroad and Zone AO 2 (2 feet flooding depth) in the vicinity of Prospect Wash. Between roadway Sta. 599+00 and Sta. 645+85, the north side of the road is shown as FEMA Zone X (not subject to flooding in 100-year rainfall event), while the south side of the road is shown as FEMA Zone AO1.

HEC-RAS and HY-8 were used to determine flow depths and water surface elevations for washes located east of Dove Mountain Boulevard. The majority of the washes crossing Tangerine Road along this reach are contained within dip sections and/or culverts. Three to four cross-section HEC-RAS models were developed for each at-grade wash crossing to estimate 100-year flood inundation limits. The cross-sections were located at the right of way limits and at the centerline of the existing pavement. HY-8 was used to determine water surface elevations and flow spread limits for all existing culverts.

The results of the existing conditions floodplain analyses are exhibited on Figure 5 of this report. Water surface elevations are given for each HEC-RAS cross-section and for each HY-8 headwater computation. West of Dove Mountain Boulevard water surfaces are given as average flow depths per effective FEMA FIRM panels. Hydraulic computation sheets (HEC-RAS or HY-8) for the existing at-grade crossings and culverts are included in Appendix E. The existing conditions drainage crossing hydraulic information is summarized in Table 3.

Table 3: Existing Conditions Cross Drainage Hydraulic Summary

CP	Roadway Station	Wash Name	Existing Drainage / Structure Description	100-Year Flow (cfs)
UPRR R/W (Sta 441+87)				
70	442+87	100' NE of UPRR R/W	2-24" RCP	1874
69	460+00	Unnamed	Sheet Flow	1526
68	496+00	Wild Burro Wash	Sheet Flow	2454
67	522+50	Ruelas Wash	Sheet Flow	1585
66	526+80	Unnamed	Sheet Flow/Dip Section	563
Breaker Rd. (Sta 528+60)				
65	545+00	Unnamed	Sheet Flow	60
64	556+73	Unnamed	1-84" CMP	161
63	561+60	Unnamed	Sheet Flow	58
62	570+65	Unnamed	Sheet Flow	708
61	579+09	Unnamed	Sheet Flow	73
60	588+13	Unnamed	Sheet Flow/Dip Section	312
59	592+00	Unnamed	Sheet Flow/Dip Section	164
58	596+45	Unnamed	Sheet Flow	35

Table 3: Existing Conditions Cross Drainage Hydraulic Summary

CP	Roadway Station	Wash Name	Existing Drainage / Structure Description	100-Year Flow (cfs)
57	605+00	Unnamed	Sheet Flow/Dip Section	50
56	613+44	Unnamed	Sheet Flow/Dip Section	232
55	628+40	Unnamed	Sheet Flow/Dip Section	157
54	635+80	Unnamed	Sheet Flow/Dip Section	124
53	645+68	Unnamed	Sheet Flow/Dip Section	118
52	655+70	Prospect Wash	Sheet Flow/Dip Section	5074
	661+66		Sheet Flow/Dip Section	
	672+56		Sheet Flow/Dip Section	
51	678+50	Unnamed	Sheet Flow	77
50	685+17	Unnamed	Dip Section	26
49	689+34	Unnamed	Dip Section	94
48	694+47	Unnamed	Dip Section	88
47	699+44	Unnamed	2-53"x34" HERCP	50
Dove Mountain Blvd/Twin Peaks Rd. (Sta 700+00)				
46	706+64	Unnamed	Dip Section	435
45	711+74	Unnamed	Dip Section	172
44	716+00	Unnamed	Dip Section	689
43	722+75	Unnamed	Dip Section	97
42	726+38	Unnamed	Dip Section	399
41	731+52	Unnamed	Dip Section	80
40	735+73	Unnamed	Dip Section	372
39	745+70	Unnamed	1-36" RCP	571= 546 + 25
38	747+00	Unnamed	Flows to CP-39	25
37	750+53	Unnamed	1-36" RCP	85 = 32 + 53
Camino de Oeste (Sta 752+90)				
36	CDO	Unnamed	1-36" RCP (Under Cmo de Oeste to CP-37)	63 (53 pipe + 10 roadway)
35	757+56	Unnamed	Dip Section	549
34	763+50	Unnamed	Sheet Flow	20
33	769+08	Unnamed	1-36" RCP	64
32	772+40	Canada Agua West Wash	Dip Section	2574
Camino de Manana (Sta 774+67)				
31	777+54	Unnamed	Dip Section	31
30	781+84	Unnamed	Dip Section	140
29	787+63	Unnamed	Dip Section	127
28	794+30	Unnamed	4-8'x5' RCBC	810
27	801+50	Unnamed	3-30" RCP	63
Thornydale Rd. (Sta 804+64)				
26.2	805+50	Unnamed	1-24" RCP	7
26.1	809+00	Unnamed	2-36" RCP	111
25	811+71	Unnamed	1-10'x4' RCBC	339
24	818+23	Unnamed	Dip Section	157
23	825+13	Unnamed	Dip Section	12
22	828+00	Unnamed	Dip Section	1110
21	836+23	Unnamed	Dip Section	318
20	848+00	Unnamed	Dip Section	168

Table 3: Existing Conditions Cross Drainage Hydraulic Summary

CP	Roadway Station	Wash Name	Existing Drainage / Structure Description	100-Year Flow (cfs)
19	854+67	Canada Agua East Wash	Dip Section	4678
Shannon Rd. (Sta 857+50)				
18	860+00	Unnamed	Flows to CP-19	28
17	868+60	Unnamed	Dip Section	126
16	874+82	Unnamed	Dip Section	153
15	877+93	Unnamed	Dip Section	40
14	883+22	Unnamed	Dip Section	77
13	885+00	North Ranch Wash	Dip Section	2683
12	894+33	Unnamed	Dip Section	60
11	897+31	Unnamed	Dip Section	172
10	904+54	Unnamed	2-36" RCP	123
La Cholla Blvd (Sta 909+28)				
9	910+25	Unnamed	3-3.7'x2.3' HERCP	135
8	913+54	Unnamed	3-36" RCP	145
7	918+44	Unnamed	1-48"CMP	496 = 470 + 26
6	924+00	Unnamed	Flows to CP-7	26
5	931+87	Unnamed	Dip Section	278
4	934+00	Unnamed	Dip Section	639
3	941+55	Unnamed	Dip Section	59
2	947+08	Unnamed	1-48"CMP	558
1	955+49	Unnamed	1-48" RCP	422
La Canada Dr. (Sta 962+13)				

RCP = Reinforced Concrete Pipe
RCBC = Reinforced Concrete Box Culvert
HERCP = Horizontal Elliptical Reinforced Concrete Pipe
CMP = Corrugated Metal Pipe
R/W = Right of Way

2.4 Geomorphology

Tangerine Road lies on the Tortolita Mountain Piedmont which contains active and inactive alluvial fan washes. East of Dove Mountain Boulevard., most washes that cross the road can be characterized as being located on inactive alluvial fan surfaces. They are entrenched on well dissected topographical surfaces and channel avulsions generally do not occur within or upstream of the project environment. The slope of the washes ranges from 0.8% to 1.5% and the volume of sediment conveyed during floods is moderate.

West of Dove Mountain Boulevard., most washes that cross the road can be characterized as being located on active alluvial fan surfaces. The land surface is poorly dissected and aerial photographs show multiple channel systems that bifurcate and coalesce throughout the area. A review of historical aerial photographs found that the overall channel patterns have not changed, however, distribution of flow amongst the multiple channels changes from flood to flood as erosion and sedimentation processes alter channel capacity. The slope of the washes west of Dove Mountain Boulevard ranges from about 2% to 3% and the volume of sediment conveyed during floods is high. Available sediment data indicates that the median diameter of the bed sediments throughout the project area ranges from 0.8 to 1.8 millimeters.

Field observations of the channel reaches upstream and downstream of Tangerine Road has led to the conclusion that the existing channel slopes are near equilibrium where land development within the watershed is absent or of low density. Certainly, change is expected where active alluvial fans exist but processes such as severe head cutting were not found within the Tangerine Road right of way. Minor degradation (1' to 3') extending a few hundred feet downstream of the road was observed at many locations; this phenomena appearing to be the result of local changes in flow velocities and sediment transport at the culverts and dip crossings. More severe channel changes (degradation) were observed downstream of some of the street crossings within the Dove Mountain area north of Tangerine Road; probably due to changes in hydraulic conditions in the vicinity of the culverts and development within the contributing watersheds. . This observation indicates that culverts can cause significant local changes to the flow hydraulics which in turn can initiate degradation along downstream channel reaches.

The existing culverts along Tangerine Road and the channel sections north and south of the road were inspected to assess their sediment conveyance characteristic. Some culverts were found to be functioning well with regard to sediment conveyance while others were observed to contain significant sediment deposition. Table 4 below summarizes the observed condition of the culverts and upstream/downstream channel conditions.

Table 4: Summary of Existing Culverts and Inlet/Outlet Conditions

Location	Q100 (cfs)	Culvert Size	Culvert Slope (ft/ft)	Inlet Conditions	Outlet Conditions
CP-1	422	1-48" RCP	3.5%	ok	Conc. Apron - ok
CP-2	558	1-48" CMP	1.9%	Minor sediment blockage	Gabion Apron - ok
CP-7	470	1-48" CMP	2.6%	ok	ok
CP-8	145	2-36" RCP	0.8%	ok	ok
CP-9	135	3-29"x45"Oval RCP	0.3%	Drop Inlet with 50% sediment blockage	90 degree turn, 25% sediment blockage
CP-10	123	2-36" RCP	1.0%	Drop Inlet	No riprap, erosion
CP-25	339	10' x 4' RCBC	0.5%	Unstabilized inlet slope, 60% sediment blockage	50% sediment blockage, riprap basin buried
CP-26.1	118	2-36" RCP	-0.3%	25% sediment blockage	25% sediment blockage
CP-26.2		1-24" RCP	1.3%	ok	50% sediment blockage, riprap basin buried
CP-27	63	3-30" RCP	1.3%	ok	ok

Table 4: Summary of Existing Culverts and Inlet/Outlet Conditions

Location	Q100 (cfs)	Culvert Size	Culvert Slope (ft/ft)	Inlet Conditions	Outlet Conditions
CP-28	810	4-8'x5' Box Culvert	0.5%	Minor sediment deposition	Minor sediment deposition
CP-33	64	1-36" RCP	1.4%	ok	ok
CP-37	32	1-36" RCP	3.2%	ok	ok
CP-38	25	1-36" RCP	3.8%	ok	ok
CP-64	161	1-84"CMP	1.4%	Inlet blocked	ok

2.5 Summary of Existing Conditions

This study has determined that there are seventy points of concentration for off-site runoff within the project area. Offsite flows emanate from watersheds north of the project and generally flow southwesterly. Watershed boundaries were determined from 2000 topography and 2005 and 2010 aerial photography provided by Pima Association of Governments (PAG) and by field reconnaissance. For local watersheds with areas less than 1 square mile, the Pima County hydrology method and PC-Hydro computer program were used to compute the 100-year discharge rates. Watershed parameters for these local watersheds were determined according to the methods described in the Pima County Hydrology Procedures User's Guide (Pima County, 2007) and using engineering judgment guided by the topography, aerial photography, and field reconnaissance. For regional watersheds with areas more than 1 square mile, either FLO-2D or HEC-1 computer models were used to obtain the 100-year discharge rates. The Pima County Type II 3-hour design storm was used to for these regional watersheds.

Flooding conditions west of Dove Mountain Boulevard are characterized as wide spread sheet flow as the entire land area within the right of way is inundated to depths varying between a few inches and 2 feet. The SFHAs from the FEMA FIRM panels were used to determine the flooding depths within the right of way. For areas east of Dove Mountain Boulevard., HEC-RAS and HY-8 were used to determine flow depths and water surface elevations for washes located east of Dove Mountain Boulevard. The majority of the washes crossing Tangerine Road along this segment are contained within dip sections and/or culvert crossings.

The geomorphic assessment determined that the washes crossing Tangerine Road convey high sediment loads, particularly those washes west of Dove Mountain Boulevard where slopes are steep and sheet flow is wide spread. The some of the existing culverts have significant loss of capacity due to sedimentation. This deposition occurs due to inadequate culvert slope, inadequate culvert capacity or culvert inverts set below the channel profile. Providing new culverts which maintain continuity of velocity and hydraulic geometry of the channels is critical to minimizing erosion along downstream reaches and deposition with and at the culvert inlets.

SECTION 3.0 PROPOSED CROSS DRAINAGE IMPROVEMENTS

3.1 Offsite Drainage Approach

The prevailing drainage flows from northeast to southwest along and through the project limits. A goal of this study is that post-improvement offsite drainage patterns be maintained to match the existing patterns as closely as possible. The proposed roadway is generally raised above the existing grade, except for isolated cut sections. Roadside channels/swales may be needed to direct flow to the proposed cross culverts/bridges. Those culverts/bridges are placed at the locations with natural outfall watercourses south of the proposed roadway. Where new cross culverts have been proposed, they have been designed to minimize sediment deposition in the culverts, including culvert inlets and outlets, and with erosion control measures to limit erosion and scour damage from flood events with magnitudes up to, and including the 100-year design storm.

The western 1.7 miles of the project, from I-10/Union Pacific Railroad (UPRR) to approximate road Station 526+55 (west of Concentration Point 66 near Breaker's Water Park entrance road), currently experiences widely dispersed sheet flow conditions and periodic roadway inundation. The lack of topographic relief and adequate downstream outfall channels in this area make it impractical to construct conventional culvert crossings that provide full conveyance of 100-year flood flows under the roadway. Because of this situation, an expanded "regional west end" study has been added to the project to examine possible offsite drainage facilities, e.g. interceptor channel systems and stormwater detention basins, that could help alleviate some of the flooding concerns for the new roadway improvements in the near term until a permanent outfall channel system to the Santa Cruz River is constructed in the future. An interceptor channel, with 10-year runoff capacity, was proposed along north side of Tangerine Rd to collect overland sheet flow from the north and convey it to the area northeast of Tangerine Rd and the UPRR crossing. At this location, runoff is proposed to be returned to widespread sheet flow, the same as existing, to drain northwesterly along the UPRR ROW. Twenty-foot grid FLO-2D models were built to accurately characterize the terrain and were use to analyze proposed drainage facilities. Under proposed conditions, the FLO-2D model results indicate that Tangerine Road is dry in the 10-year rainfall event, even at the vicinity of UPRR and Tangerine Rd intersection. During the 100-year rainfall event, storm water does not overtop Tangerine Rd from roadway Sta. 452+00 to the east end of this project. Just east of the UPRR/Tangerine Rd crossing, approximately 900 feet of the road is inundated with flow depths of up to 1.4 feet, which are measured at the inside lane of Tangerine Rd. Inundation on roadway from Sta

444+50 to Sta. 450+00 resulted in flooding depths of 1.0 foot and above. The estimated duration for inundation with depths of 1.0 foot and above on Tangerine Rd is 3 hours during the 100-year storm. The West End Regional Drainage Analysis, which itself is a standalone report, has been included in Appendix K of this Drainage Report.

After the 15% Plans and Stage I drainage report were submitted to the agencies for review, the Regional Transportation Authority (RTA) conducted a Value Analysis Study (dated March 2012) for this project to recommend feasible cost saving measures. Three adopted recommendations by the Value Analysis Study team have significant impacts on cross drainage designs: 1) Shift the Tangerine Road alignment 25 feet north from the 15% Plan alignment between Thornydale Road and La Canada Drive, 2) Lower roadway profiles between culverts/bridges to reduce roadway fill costs, and 3) Revise all grouted riprap bank protection designs to colored shotcrete. As results of these three recommendations, culverts were shifted northerly along with the shifted roadway alignment. Lowered roadway profiles between culverts made more upstream roadside channels necessary as compared to the Stage I design due to reduced drainage conveyance capacities along roadway embankment. All grouted riprap bank protection applications were revised to incorporate colored shotcrete instead.

Two equestrian crossings were included in the roadway design at Stations 573+00 and 690+00. These equestrian crossings are not designed for cross drainage purposes with offsite runoff being diverted around them.

3.2 Proposed Conditions Hydrology

The proposed roadway will not have curbs, except for isolated roadway intersections. Generally, runoff on north half of the roadway will sheet flow to the north and will be directed to the cross culverts along the roadway embankment, while south half of the roadway will sheet flow to the south and leave the project site. The percentage of pavement areas in proposed conditions is relative small compared to the overall drainage areas of each offsite watershed. Therefore, proposed pavement drainage has negligible impact on 100-year discharge rates for most watersheds. The 100-year discharge rates at some concentration points were revised because of the proposed cross culvert locations or diversion of flows to other crossings. At those locations, watersheds were either further divided, or portions of contributing watershed areas were redirected to neighboring watersheds. Watersheds within the west end regional drainage study areas were also updated based on the twenty-foot grid FLO-2D models utilized for that study.

The proposed conditions concentration point runoff rates are summarized in Table 5. Only hydrologic computations for the proposed watersheds that are different from their existing ones have been revised. Proposed hydrologic computations are provided in Appendix D and the watersheds are shown on Figure 6.

Table 5: Summary of Proposed Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Revised From Existing?
		B	C	D				
1	89.8	7%	53%	40%	0.035	5	422	No
2	180.9	34%	36%	30%	0.035	5	558	No
3	7.8	0%	53%	47%	0.035	5	59	No
4	265.9	31%	33%	36%	0.035	5	639	No
5	53.3	13%	52%	35%	0.035	5	278	No
6	3.5	0%	53%	47%	0.035	5	26	No
7	185.4	67%	31%	2%	0.035	5	470	No
8	30.8	58%	42%	0%	0.035	5	145	No
9	34.5	88%	12%	0%	0.035	5	135	No
10	28.3	73%	27%	0%	0.035	5	123	No
11	35.7	84%	16%	0%	0.035	5	172	No
12	12.3	100%	0%	0%	0.035	5	60	No
13*	1,202.40	34%	2%	64%	0.035	5	2,683	No
14	17.9	100%	0%	0%	0.035	5	77	No
15	6.7	100%	0%	0%	0.035	5	40	No
16	37.1	98%	2%	0%	0.035	5	153	No
17	25.6	95%	5%	0%	0.035	5	126	No
18	4.3	50%	50%	0%	0.035	5	28	No
19*	2,317.40	15%	9%	76%	0.035	5	4,678	No
20	31.6	50%	50%	0%	0.035	5	168	No
21	86.7	50%	50%	0%	0.035	5	316	No
22	474	62%	34%	4%	0.035	5	1,110	No
23	1.8	50%	50%	0%	0.025	5	12	No
24	26.1	50%	50%	0%	0.025	5	157	No
25	72.8	50%	50%	0%	0.025	5	339	No
26.1	19.7	53%	47%	0%	0.025	5	111	No
26.2	1.1	53%	47%	0%	0.025	5	7	No
27 E	N/A						Est. 42	Yes
27	9.5	50%	50%	0%	0.025	5	63	No
28	321.4	66%	34%	0%	0.035	5	810	No
29	27.6	50%	50%	0%	0.035	5	127	No
30	27.6	50%	50%	0%	0.035	5	140	No
31	4.7	50%	50%	0%	0.035	5	31	No
32*	1,170.90	15%	18%	67%	0.035	5	2,574	No
33.1	3.5	0%	53%	47%	0.035	5	26	Yes

Table 5: Summary of Proposed Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Revised From Existing?
		B	C	D				
33.2	5.0	0%	53%	47%	0.035	5	38	Yes
34	2.6	0%	53%	47%	0.035	5	20	No
35	166.5	27%	39%	34%	0.035	5	549	No
36	10.3	0%	53%	47%	0.035	5	66	Yes
37	4.2	0%	53%	47%	0.035	5	32	No
38	3.3	0%	53%	47%	0.035	5	25	No
39	166.4	34%	35%	31%	0.035	5	546	No
40	95.9	24%	40%	36%	0.035	5	372	No
41	10.2	50%	27%	23%	0.025	35	80	No
42	112.2	34%	35%	31%	0.035	10	399	No
43	12.2	0%	53%	47%	0.025	20	97	No
44	133	1%	53%	46%	0.032	20	688	No
45	22.9	1%	53%	46%	0.025	5	172	No
46	79.1	40%	32%	28%	0.025	15	435	No
47	8.2	100%	0%	0%	0.025	5	50	No
48	13.1	100%	0%	0%	0.025	20	88	No
49.1	2.3	100%	0%	0%	0.030	10	14	Yes
49.2	17.1	100%	0%	0%	0.030	10	83	Yes
50	4.3	100%	0%	0%	0.035	5	26	No
51	15	100%	0%	0%	0.035	5	77	No
52.1**	Runoff at CP52.1 is 50% of that at existing conditions CP-52						2,537	Yes
52.2**	Runoff at CP52.2 is 80% of that at existing conditions CP-52						4,059	Yes
53	49.2	100%	0%	0%	0.060	0	148	Yes
54	59.2	100%	0%	0%	0.06	0	124	No
55	86.4	100%	0%	0%	0.06	0	157	No
55.1	4.5	100%	0%	0%	0.060	5	21	Yes
56	145.9	100%	0%	0%	0.06	0	232	No
57	13.4	100%	0%	0%	0.06	0	50	No
58	10.4	100%	0%	0%	0.060	0	41	Yes
59	89.3	100%	0%	0%	0.060	0	161	Yes
60	231.6	100%	0%	0%	0.06	0	312	No
61	31.4	100%	0%	0%	0.06	0	73	No
62.1	Runoff at CP62.1 is 90% of that at existing conditions CP-62						637	Yes
62.2	Runoff at CP62.2 is 10% of that at existing conditions CP-62						71	Yes
62.3	2.4	100%	0%	0%	0.060	5	15	Yes
63	19.8	100%	0%	0%	0.06	0	58	No
64	94	100%	0%	0%	0.06	0	161	No
65	22.5	100%	0%	0%	0.06	0	60	No
66	638.7	100%	0%	0%	0.06	0	563	No
67***	N/A	N/A	N/A	N/A	N/A	N/A	785	Yes

Table 5: Summary of Proposed Conditions 100-Year Peak Discharge Rates

Concentration Point	WS Area (acres)	Hydrologic Soils Group			Basin Factor (nb)	Impervious (%)	Q100 (cfs)	Revised From Existing?
		B	C	D				
67.1***	N/A	N/A	N/A	N/A	N/A	N/A	156	Yes
68***	Sheet Flow, See FLO-2D in Appendix K							Yes
69***	Sheet Flow, See FLO-2D in Appendix K							Yes
70***	N/A	N/A	N/A	N/A	N/A	N/A	1,404	Yes
FR1	3.2	100%	0%	0%	0.025	30	22	Yes
FR2	1.0	100%	0%	0%	0.025	30	7	Yes
FR3	1.7	100%	0%	0%	0.025	30	11	Yes
FR4	1.6	100%	0%	0%	0.025	30	11	Yes
Regional WS at Trico East Driveway***	N/A	N/A	N/A	N/A	N/A	N/A	1,506	Yes

*obtained from HEC-1 model using Pima County's 3-hour Type II rainfall distribution;
**obtained from the revised FLO-2D model for Tortolita Study with Pima County's 3-hour Type II rainfall distribution;
***obtained from the 20-ft grid FLO-2D model for Tangerine Rd West End Regional Drainage Analyses.
**** CP-68 and CP-69 are removed in the proposed hydrology table. Refer to the Tangerine Rd West End Regional Drainage Analyses for discharge rates.

3.3 Proposed Conditions Hydraulics

As was the case for existing conditions drainage structures, proposed channels were rated by normal depth analyses using Manning's equation and proposed culverts were analyzed using the FHWA Culvert Analysis program HY-8. Hydraulic computation sheets for the proposed culverts are included in Appendix F.

The roadway is generally raised above existing ground to allow 100-year offsite drainage to be conveyed within the proposed cross culverts. To minimize the amount of roadway fill, thirty cross culverts are designed to have drop inlets. The drop inlets will be concrete lined with 2:1 (H:V) drop slope, except for the drop inlet at Sta 613+05. This drop inlet, which is lined with shotcrete, has a drop slope of 10:1 to conform to the project medium category wildlife crossing criteria for wildlife approach conditions. Drop inlet details are shown on Figure 7. For the cross culverts with drop inlets, the drop inlets are modeled in HY-8. The widths of the drop inlet crests are determined from the geometry of the cross culverts, wingwall configuration, topography in the inlet vicinity, and drainage patterns at the inlets. The widths of the drop inlet crests were generally set as suggested in HY-8 Manual (up to reasonable widths) so that the crest widths would not impact the headwater elevations. Further increasing the crest widths may be difficult or not cost effective in certain situations such as extending wingwalls or simply not geometrically possible.

The proposed cross culverts call out extending existing culverts at two locations, Sta. 801+54, and Sta. 699+85. The culverts at these locations are in good condition in the vicinity of the inlets and outlets. The inlet and outlet for the existing cross culvert (4-8'x5' reinforced concrete box culvert (RCBC)) at Sta. 794+30 are also in good condition. However, this existing culvert has a very flat slope (0.0%), which contradicts the design criteria of minimum 1.0% slope. The flat slope on this culvert makes it not suitable for reuse considering sediment transport and hydraulic conveyance. Therefore, this existing 4-8'x5' RCBC (Sta. 794+30) is proposed to be removed and replaced with a new set of 4-8'x5' RCBC with a 1.5% longitudinal slope. Sheet flooding is the predominate drainage pattern for areas west of Dove Mountain Boulevard and some areas east of Dove Mountain Boulevard. Those areas lack natural topography that would direct offsite runoff to the proposed cross culvert inlets, so training berms are needed on the west (downstream) side to direct runoff to the cross culvert inlets. The minimum heights of the training berms have been set at 1 foot above the culvert headwater elevations. The stream sides of the training berms are lined with colored shotcrete to prevent erosion. Training berm details are shown on Figure 7.

Different culvert materials were considered for the crossings including HDPE Pipe, Spiral Rib Pipe (SRP), Corrugated Metal Pipe (CMP), Reinforced Concrete Pipe (RCP), Metal Plate Arches, Concrete Arches and RCBC (pre-cast and cast-in-place). HDPE and SRP are hydraulically interchangeable with RCP, so the comparison comes down to cost, structural loading concerns and product design life. CMP is less smooth than RCP and thus less efficient to convey water and sediment loads. Washes crossing Tangerine Road convey high sediment loads so roughness and slope are important for maintaining sufficient velocity to minimize sediment deposition. Metal Plate Arches were evaluated further at three representative culvert locations. The project Geotechnical Report indicated that soil corrosivity appears to be extremely low, except for three locations east of Thornydale Road. Cost analysis indicated that the construction costs for Metal Plate Arches are generally 40% less than those for RCBC. However, Metal Plate Arches have not been widely used on roadway projects within Pima County limits. More analyses to demonstrate its longevity and ease of maintenance maybe needed before Metal Plate Arches would be used on this project. Therefore, for the 15% and 30% designs, the project team was directed to use RCP and RCBC for the basic cross drainage designs.

HY-8 computations for cross culverts are provided in Appendix F. The proposed cross culverts, including structure types, training berms, drop inlet, hydraulic information, and drainage structure details are shown on Figure 7. Table 6 summarizes the proposed cross culverts. A more detailed cross culvert summary table is provided in Appendix F.

Table 6: Proposed Cross Drainage Structures

CP	Roadway Station	Design Flow (cfs)	Structure Description	Length (ft)	Slope (%)	Velocity (fps)	HW Elev (ft)	ROW Prp WSE	ROW Ext WSE	WSE Diff (Prp-Ext)**
Regional WS	487+44 Lt (Trico)	1,506	5-10'x4' RCBC	60	0.9	12.4	2,059.9	N/A	N/A	N/A
FR3	FR 104+06	18	2-24" RCP	64	1.0	7.2	2,145.2	N/A	N/A	N/A
FR2	541+06 Rt	7	1-24" RCP	69	1.0	6.7	2,152.5	N/A	N/A	N/A
FR1	FR 110+00	82	2-36" RCP	98	2.0	12.6	2,157.6	N/A	N/A	N/A
UPRR R/W (Sta 441+87)										
70	442+87	1,404	4-24" RCP	134	1.6	11.26	2,042.1	2042.1	2042.7	-0.6
69	460+00	Sheet Flow								
68	496+00	Sheet Flow								
67	515+35	785	4-10'x4' RCBC	178	1.9	15.4	2,098.5	2099.6	2099.6	0.0
67.1	524+28 Lt	156	4-30" RCP	52	1.0	10.2	2,116.1	2116.1	2116.1	0.0
66	526+65	563	3-8'x4' RCBC	200	1.9	16.1	2,123.0	2128.7	2128.7	0.0
Breaker Rd. (Sta 528+60)										
65	544+94	60	2-36" RCP	262	2.1	12.0	2,167.8	2174.0	2174.0	0.0
64	555+06	161	2-48" RCP	156	2.0	14.4	2,189.8	2190.8	2190.8	0.0
63	561+35	73	2-36" RCP	256	2.5	13.6	2,207.1	2211.1	2211.1	0.0
62.3	568+78 LT	15	1-24" RCP	56	2.0	9.4	2,222.9	2223.8	2223.8	0.0
62.2	570+97	71	2-36" RCP	163	2.2	13.6	2,229.2	2230.2	2230.0	0.2
62.1	575+67	637	1-Single Span 60' Bridge, See Table 9					2241.2	2240.0	1.2
61	579+09	73	2-36" RCP	186	3.5	15.3	2,250.0	2252.0	2252.0	0.0
60	588+63	312	2-8'x4' RCBC	151	2.2	15.5	2,272.4	2273.5	2273.5	0.0
59	592+82	161	2-48" RCP	171	1.5	13.1	2,281.5	2282.5	2282.5	0.0
58	596+45	91	2-42" RCP	170	1.6	12.1	2,288.9	2291.0	2291.0	0.0
57	605+00	50 cfs Diverted to CP-58						2311.5	2311.5	0.0
56	613+05	232	4-10'x6' RCBC	156	1.9	10.3	2,331.8	2333.9	2333.9	0.0
55.1	630+71 LT	21	2-24" RCP	50	1.0	7.5	2,377.4	2382.8	2382.8	0.0
55	627+84	157	2-48" RCP	138	1.0	11.7	2,372.9	2374.9	2374.9	0.0
54	635+38	124	2-42" RCP	162	1.8	13.2	2,394.5	2395.5	2395.5	0.0
53	644+50	148	3-42" RCP	154	1.3	11.5	2,419.4	2422.5	2422.5	0.0
52.2	661+84	4,059	1-Single Span 104' Bridge, See Table 9					2475.7	2472.0	3.7
52.1	672+50	2,537	5-10'x4' RCBC	169	3.1	19.80	2,492.10	2492.4	2491.0	1.4
51	678+50	77 cfs Diverted to CP-52						2511.5	2511.5	0.0
50	685+17	26 cfs Diverted to CP-52						2525.5	2525.5	0.0
49.2	689+34	80 cfs Diverted to CP-52						2534.0	2534.0	0.0
49.1	691+04	14	1-24" RCP	187	3.0	12.1	2,535.3	2535.3	2535.3	0.0
48	694+27	88	2-42" RCP	166	1.3	11.1	2,538.8	2539.7	2539.7	0.0
47	699+44	50	Extend Exist 2-53"x34" HERCP	299	Varies	10.9	2,549.0*	2550.0	2550.0	0.0
Dove Mountain Blvd/Twin Peaks Rd. (Sta 700+00)										
46	706+73	435	2-10'x4' RCBC	160	2.3	16.5	2,552.7	2553.8	2553.8	0.0

Table 6: Proposed Cross Drainage Structures

CP	Roadway Station	Design Flow (cfs)	Structure Description	Length (ft)	Slope (%)	Velocity (fps)	HW Elev (ft)	ROW Prp WSE	ROW Ext WSE	WSE Diff (Prp-Ext)**
45	711+44	172	2-48" RCP	149	1.5	13.2	2,558.5	2559.6	2559.6	0.0
44	716+38	1,185	4-10'x5' RCBC	135	3.7	19.9	2,566.9	2568.4	2568.4	0.0
43	722+75	97 cfs Diverted to CP-44						2583.0	2583.0	0.0
42	726+38	399 cfs Diverted to CP-44						2593.5	2593.5	0.0
41	730+92	80	2-42" RCP	186	2.8	14.0	2,597.1	2599.2	2599.2	0.0
40	735+69	372	4-10'x6' RCBC	157	3.1	14.6	2,604.5	2606.0	2606	0.0
39	744+40	571	3-10'x4' RCBC	132	2.5	15.9	2,620.7	2622.1	2622.1	0.0
38	747+00	25 cfs Diverted to CP-39						2625.2	2625.2	0.0
37	750+46	98	2-42" RCP	163	3.4	16.4	2,626.2	2626.2	2625.9	0.3
Camino de Oeste (Sta 752+90)										
36	CDO 29+05	66	2-36" RCP	91	1.5	10.3	2,631.2	2632.0	2632.0	0.0
35	756+84	607	2-10'x5' RCBC	192	1.9	17.3	2,635.1	2635.8	2635.8	0.0
34	763+50	20 cfs Diverted to CP-35						2650.0	2650.0	0.0
33.2	767+00	38 cfs Diverted to CP-35						2657.0	2657.0	0.0
33.1	768+72	26	1-36" RCP	232	2.6	12.4	2,655.7	2658.5	2658.5	0.0
32	772+32	2,574	3-24'X7' Arch (embedded 8")	138	4.0	21.8	2,662.9	2662.9	2660.2	2.7
Camino de Manana (Sta 774+67)										
31	777+54	31	1-36" RCP	158	3.0	14.6	2,666.1	2666.2	2666.2	0.0
30	781+97	140	2-48" RCP	180	3.2	17.2	2,667.2	2667.2	2665.5	1.7
29	787+65	127	2-48" RCP	137	3.0	15.9	2,671.1	2672.3	2672.3	0.0
28	794+30	810	4-8'x5' RCBC	121	1.5	14.3	2,677.0	2677.4	2677.4	0.0
27	801+54	63	Extend Exist 3-30" RCP	154	1.1	8.7	2,684.1	2686.5	2686.5	0.0
Thornydale Rd. (Sta 804+64)										
27 E	Thornydale 53+00	Est. 42	2-24" RCP	190	1.8	11.2	2692.7	2694.2	2694.2	0.0
26.2	805+64	7	1-24" RCP	156	1.0	6.7	2,689.5*	2693.0	2693.0	0.0
26.1	809+20	111	2-42" RCP	148	1.2	12.2	2,691.2	2693.0	2693.0	0.0
25	812+03	339	2-8'x4' RCBC	193	1.5	14.5	2,693.2	2695.0	2695.0	0.0
24	818+42	157	2-48" RCP	170	1.9	14.1	2,703.3	2703.5	2703.5	0.0
23	825+31	12	1-24" RCP	150	2.3	10.6	2,714.3	2714.3	2714.3	0.0
22	828+00	1,110	4-10'x6' RCBC	122	2.5	17.2	2,717.5	2717.5	2716.0	1.5
21	835+80	316	2-8'x4' RCBC	125	1.9	14.6	2,723.7	2724.8	2724.8	0.0
20	847+26	168	2-48" RCP	140	1.4	12.7	2,730.5	2730.5	2730.1	0.4
19	855+03	4,706	1-36"x9' Arch (embedded 8", 2-32'x8' Arch s/ paved invert)	181	1.4	18.4	2,737.6	2737.6	2736.1	1.5
Shannon Rd. (Sta 857+50)										
18	860+00	28 cfs Diverted to CP-19						2751.0	2751.0	0.0
17	868+80	126	2-48" RCP	144	2.3	14.1	2,758.7	2758.7	2757.5	1.2
16	874+87	153	2-48" RCP	140	2.9	15.9	2,762.0	2762.0	2760.3	1.7
15	878+25	40	1-42" RCP	157	1.7	11.9	2,763.3	2763.3	2763.3	0.0
14	883+00	77	3-36" RCP	133	1.1	9.2	2,767.3	2767.3	2767.3	0.0

Table 6: Proposed Cross Drainage Structures

CP	Roadway Station	Design Flow (cfs)	Structure Description	Length (ft)	Slope (%)	Velocity (fps)	HW Elev (ft)	ROW Prp WSE	ROW Ext WSE	WSE Diff (Prp-Ext)**
13	885+32	2,743	7-10'x6' RCBC	146	1.9	17.6	2,768.7	2768.7	2768.4	0.3
12	894+33	60 cfs Diverted to CP-13						2775.5	2775.5	0.0
11	897+65	172	3-48" RCP	157	1.0	10.6	2,775.1	2775.9	2775.9	0.0
10	904+62	123	3-36" RCP	162	1.9	12.5	2,777.6	2777.6	2777.6	0.0
La Cholla Blvd (Sta 909+28)										
8 & 9	La Cholla 534+93	280	6-42" RCP	191	1.6	12.3	2,767.0	2767.0	2766.8	0.2
9	910+44	135	4-45"x29" HERCP	149	2.3	13.2	2,781.7	2782.9	2782.9	0.0
8	913+63	145	4-36" RCP	160	2.5	14.3	2,781.3	2781.3	2781.3	0.0
7	918+72	470	3-8'x4' RCBC	143	2.9	16.9	2,781.7	2781.7	2783.9	-2.2
6	924+00	26 cfs Diverted to CP-5						2790.7	2790.7	0.0
5	930+96	304	2-6'x4' RCBC	128	2.9	17.2	2,777.7	2777.7	2777.3	0.4
4	932+94	698	3-10'x4' RCBC	239	2.4	17.8	2,779.0	2779.6	2779.6	0.0
3	941+55	59 cfs Diverted to CP-4						2793.6	2793.6	0.0
2	946+90	558	2-10'x5' RCBC	155	1.6	15.7	2,788.5	2788.5	2792.4	-3.9
1	955+50	442	2-10'x4' RCBC	148	2.7	17.0	2,787.9	2787.9	2792.6	-4.7
La Canada Dr. (Sta 962+13)										

*Grate inlet ponding elevation
** Drainage easement will be provided where the proposed water surface elevations at ROW are 0.1 foot higher than those in existing conditions.

3.4 Sedimentation

The purpose of the sedimentation analysis is to assess the impact of erosion and sedimentation processes on the ability of the proposed culverts to function as intended, to minimize culvert maintenance requirements, and to minimize the impact of the culverts on upstream and downstream erosion/sedimentation. Achieving these goals is challenging in a high sediment load alluvial fan environment, and compliance with cross-drainage design standards necessitates significant change to flow hydraulics for washes west of Dove Mountain Boulevard. where flow is highly disbursed and sediment load is very high. As such, the scope of the needed appurtenant improvements such as stabilized training berms and drop inlets will increase cost.

To minimize the probability of sediment deposition within the culverts, CMG recommends that the ratio of the inlet headwater depth to culvert height be 1.0 or less so there will be minimal change in the approach flow velocity; the purpose being to minimize or prevent sediment deposition at the culvert inlet. The height of drop inlets (if they are necessary) should be minimized as severe inlet deposition has been widely noted for drop inlet structures in high sediment load environments. An analysis of how the proposed culverts should function (listed in Table 7) found that culvert slope should be 1.0 % or more to minimize the risk of deposition with the structures. Smooth bore culverts are also recommended to minimize surface roughness and sediment deposits.

Minimizing the use of drop inlets, limiting inlet headwater depths and maintaining minimum culvert slopes decreases the likelihood of deposition that will compromise culvert function. Equation 11.9 of the City of Tucson Drainage Standards Manual was used to compare the sediment transport capacity of the proposed culverts to that of the upstream sediment supply.

$$\mathrm{R_s} = Q_{ac} / Q_p * (S_{ac} / S_p)^{1.66} * (n_{ac} / n_p)^{-1.55} * (R_{ac} / R_p)^{0.91}$$

Where,

R_s = sediment-transport ratio (channel to culvert)
Q_{ac} = discharge in approach channel (cfs)
Q_p = total culvert discharge (cfs)
S_{ac} = longitudinal slope of approach channel (ft/ft)
S_p = longitudinal slope of culvert (ft/ft)
n_{ac} = Manning's roughness coefficient for the approach channel
n_p = Manning's roughness coefficient for the culvert
R_{ac} = Hydraulic radius of flow in approach channel (ft)
R_p = Hydraulic radius of flow within the culvert (ft)

Table 7 summarizes the results of these computations. The limitation of this methodology is the assumption of a uniform distribution of flow and sediment amongst the culvert cells, a condition that seldom exists in the field. Nonetheless, the slopes of the culverts have been maximized subject to other constraints and design criteria, to provide sediment conveyance that exceeds the estimated sediment supply. The ratio of the estimated sediment supply to computed culvert sediment conveyance capacity (R_s) ranges from 0.013 to 0.965, which indicates a factor of safety for preventing culvert sediment deposition.

High sediment yield from the alluvial fan indicates that post-flood channel/culvert maintenance will be required at some locations to maintain design capacity. A sediment yield of 0.36 tons per acre per year (based on reference 20) was used to estimate drainage system maintenance needs. Yearly sediment yields vary at different locations because of differences in soil characteristics, contributing drainage areas, developments, and drainage patterns. Interceptor channels and roadside swales are generally susceptible to sediment deposition due to high sediment yields found in the project area. Annual and post-flood drainage system inspections should be conducted to maintain the system's design drainage capacities.

Table 7: Cross Culvert Sediment Transport Summary

CP	Qac(cfs)	Qp (cfs)	Sac (ft/ft)	Sp (ft/ft)	nac	np	Rac	Rp	Rs
1	422	422	0.009	0.027	0.04	0.012	0.6	0.97	0.016
2	558	558	0.012	0.016	0.035	0.012	1	1.32	0.091
4	698	698	0.026	0.0237	0.035	0.012	1.24	0.99	0.273
5	278	278	0.017	0.029	0.035	0.012	0.93	1.00	0.073
7	496	496	0.013	0.0294	0.035	0.012	2.33	0.83	0.126
8	145	145	0.022	0.025	0.035	0.012	0.35	0.63	0.091
9	135	135	0.032	0.0235	0.035	0.012	0.29	0.49	0.195
La Cholla 534+93	280	280	0.017	0.016	0.035	0.012	1.03	0.77	0.273
10	123	123	0.012	0.0185	0.035	0.012	0.94	0.72	0.118
11	172	172	0.009	0.01	0.035	0.012	0.64	0.92	0.115
13	2743	2743	0.0128	0.0192	0.035	0.012	1.83	1.36	0.128
14	77	77	0.014	0.0113	0.035	0.012	0.51	0.66	0.214
15	40	40	0.021	0.0166	0.035	0.012	0.3	0.72	0.127
16	153	153	0.021	0.0293	0.035	0.012	0.54	0.83	0.074
17	126	126	0.033	0.023	0.035	0.012	0.71	0.80	0.311
19	4706	4706	0.015	0.0139	0.035	0.015	1.22	1.90	0.204
20	168	168	0.022	0.0136	0.035	0.012	0.55	1.00	0.245
21	318	318	0.019	0.0192	0.035	0.012	0.34	0.92	0.075
22	1110	1110	0.0135	0.0254	0.035	0.012	1.2	1.06	0.075
23	12	12	0.027	0.0233	0.035	0.012	0.2	0.43	0.122
24	157	157	0.019	0.0188	0.035	0.012	0.54	0.91	0.120
25	339	339	0.021	0.015	0.035	0.012	0.92	1.02	0.303
26.1	112	112	0.03	0.0122	0.035	0.012	0.84	0.85	0.839
26.2	7.3	7.3	0.018	0.01	0.035	0.012	0.20	0.41	0.264
27 E	42	42	0.01	0.02	0.035	0.012	1.10	0.47	0.132
27	63	63	0.03	0.011	0.035	0.012	0.29	0.62	0.503
28	810	810	0.015	0.015	0.035	0.012	1.06	1.12	0.181
29	127	127	0.0133	0.0299	0.035	0.012	0.71	0.73	0.048
30	140	140	0.0185	0.0319	0.035	0.012	0.59	0.76	0.061
31	31	31	0.033	0.03	0.035	0.012	0.2	0.56	0.087
32	2574	2574	0.022	0.0399	0.035	0.012	1.38	1.16	0.083
33.1	26	26	0.029	0.025	0.035	0.012	0.2	0.56	0.096
35	607	607	0.03	0.0189	0.035	0.012	0.77	1.19	0.275
36	63	63	0.021	0.0147	0.035	0.012	0.38	0.69	0.199
37	98	98	0.023	0.0337	0.035	0.012	0.46	0.67	0.072
39	571	571	0.02	0.025	0.035	0.012	0.83	0.87	0.126
40	372	372	0.021	0.029	0.035	0.012	0.83	0.56	0.159

Table 7: Cross Culvert Sediment Transport Summary

CP	Qac(cfs)	Qp (cfs)	Sac (ft/ft)	Sp (ft/ft)	nac	np	Rac	Rp	Rs
41	80	80	0.024	0.028	0.035	0.012	0.45	0.64	0.106
44	1185	1185	0.02	0.037	0.035	0.012	0.76	0.99	0.054
45	172	172	0.019	0.015	0.035	0.012	0.53	0.99	0.160
46	435	435	0.015	0.023	0.035	0.012	1.10	0.96	0.106
47	50	50	0.025	0.025	0.035	0.012	0.65	0.57	0.215
48	88	88	0.025	0.013	0.035	0.012	0.65	0.79	0.473
49.1	14	14	0.025	0.0297	0.035	0.012	0.20	0.42	0.072
52	2537	2537	0.03	0.035	0.035	0.012	0.30	1.13	0.044
53	148	148	0.033	0.013	0.035	0.012	0.53	0.82	0.597
54	124	124	0.029	0.0181	0.035	0.012	0.33	0.85	0.177
55	157	157	0.027	0.01	0.035	0.012	0.48	1.03	0.492
56	232	232	0.027	0.0189	0.035	0.012	0.45	0.46	0.335
58	91	91	0.029	0.016	0.035	0.012	0.37	0.77	0.264
59	161	161	0.028	0.015	0.035	0.012	0.46	0.96	0.274
60	312	312	0.034	0.022	0.035	0.012	0.26	0.89	0.126
61	73	73	0.034	0.035	0.035	0.012	0.26	0.60	0.084
62.2	71	71	0.028	0.0221	0.035	0.012	0.25	0.63	0.121
62.3	15	15	0.022	0.0196	0.035	0.012	0.26	0.49	0.129
63	73	73	0.029	0.025	0.035	0.012	0.35	0.65	0.138
64	161	161	0.025	0.0202	0.035	0.012	0.72	0.91	0.219
65	60	60	0.03	0.021	0.035	0.012	0.53	0.62	0.298
66	563	563	0.036	0.019	0.035	0.012	1.04	1.01	0.564
67	785	785	0.02	0.0192	0.035	0.012	1.04	0.95	0.220
70	1404	1404	0.03	0.0164	0.035	0.012	0.53	0.73	0.384
FR1	82	82	0.015	0.02	0.035	0.012	1.10	0.68	0.182
FR2	7	7	0.015	0.01	0.035	0.012	1.10	0.39	0.965
FR3	18	18	0.015	0.01	0.035	0.012	1.10	0.47	0.815
Regional WS (Trico)	1506	1506	0.016	0.009	0.035	0.012	1.10	1.49	0.376

3.5 Channelization/Bank Protection

New channels are proposed to redirect drainage to or from the proposed cross drainage system, to prevent offsite runoff from overtopping the roadway, and/or to minimize the need for drainage easements outside of the right-of-way. The channels were designed to contain the 100-year runoff plus freeboard. In most cases, only a portion of the listed watershed (see Table 5 in Section 3.2) contributes runoff to proposed roadside channels. The discharge reaching a given section of channel was estimated by assuming it is proportional to the contributing drainage area within that watershed.

Channels were also evaluated to determine whether erosion protection is needed. Equation 1b-V of *Drainage and Channel Design Standards for Local Drainage by Pima County Department of Transportation and Flood Control District* was used to determine allowable velocities for unprotected earthen channels. Channel velocities were determined by Manning's Equation. Manning's values of 0.03 or 0.025 were used at the proposed earthen or colored shotcrete-lined swales and roadway embankment respectively; while natural drainage conveyance areas adjacent to the roadway were modeled with a Manning's value of 0.035. Channels will be armored to prevent erosion where design velocity exceeds allowable velocity. Some channels have been designed as interceptor channels to collect offsite runoff from the north side of the channel. Per the Town of Marana's instructions, interceptor channels will be further evaluated to provide a minimum 1-foot freeboard along the north bank during the project's final design phases. Roadway embankment bank protection is proposed at locations where significant offsite runoff is directed at the embankment then redirected westerly along the roadway. The proposed channels and roadway bank protection are shown on Figure 7, and are summarized in Appendix H.

3.6 Culvert Outlet Protection

Wire-tied riprap lined basin dissipators are proposed at outlets of all culverts larger than 24-inch diameter to dissipate velocity and minimize downstream degradation. Loose riprap aprons (HEC-14 Section 10.2) have been proposed at the 24-inch culvert outlets. Standard ADOT concrete cutoff walls will be provided to further protect the culverts should the riprap basins be undermined. The cutoff walls are shown on the culvert Plan & Profile sheets in the roadway plans (reduced copies also provided in Appendix F). The final geometry of the HEC-14 basin dissipators will be detailed on the project plans during future construction plan phases. Given the large number of culverts in the project and the uncertainty of where future channel degradation may occur, it's suggested that an on-going post-flood monitoring program be implemented after roadway construction to assess downstream channel changes over time to determine additional maintenance and/or scour mitigation measures that may be needed.

As stated above, energy dissipators and concrete cutoff walls will be needed at all culvert outlets to minimize downstream erosion and to protect the culverts from being undermined by long-term degradation. Most culvert outlet velocities are above 10 feet per second (fps) and are more than 1.5 times the downstream channel velocities. As stated in the Drainage and Channel

Design Standards for Local Drainage by PCDOT and PCRFCD, whenever culvert outlet velocities are more than 1.5 times the stream channel velocities and exceed 10 fps then grouted or wire-tied riprap should be used. Due to highly erosive soils in the project area, and because they have more natural environment matching characteristics, wire-tied riprap energy dissipating basins, as described in Chapter 10 of *Hydraulic Design of Energy Dissipators for Culverts and Channels* (HEC-14) by FHWA, are proposed to be used at the outlets of all culverts larger than 24-inch diameter. The riprap basin energy dissipators are based on armoring a pre-formed scour hole of sufficient depth and length to promote and contain a hydraulic jump at the culvert outlet, thereby reducing energy and returning culvert outlet velocities to approximate downstream pre-project channel velocities. Sediment deposited in the riprap basin from low flow events is scoured out during the next significant flood, allowing the energy dissipation to occur in the outlet basin during successive flood events.

The Town of Oro Valley has different standards for culvert outlet protection than the Pima County standards discussed above, e.g. plunge basin dissipators are discouraged for culvert outlet velocities exceeding 15 fps. Even though this project's hydrologic and hydraulic design criteria followed Pima County standards, the culvert outlet protection designs within the Oro Valley Town limits were put through a limited evaluation using Oro Valley standards. Based on hydrology from the recent La Cholla Boulevard corridor drainage study, Oro Valley hydrology peak discharges were found to be approximately 50% of peak discharges determined by the Pima County PC-Hydro method. CMG found that if the Oro Valley hydrology methods were used (at the assumed 50% of PC-Hydro peaks), the relevant culverts generated outlet velocities at, or below 15 fps, and thus would meet Oro Valley standards. Notwithstanding the limited Oro Valley evaluation describe above, the culvert hydraulic computations provided in Appendix F of this report are based on Pima County hydrology.

The sizes of outlet basins were computed by use of the HEC-14 methods and guidance found in the U.S. Army Corps of Engineers (USACE) research document, *Gabions for Streambank Erosion Control*, May 2000 and in Maccaferri Gabions manufacturer's design data. The recommended sizes of the outlet protection basins are summarized in Table 8 and are provided in Appendix I, along with an excerpt from the USACE research document. Cutoff wall references to the ADOT B Standards are also included in Table 8.

At the medium and large wildlife crossings, which were called out on Figure 7, the culverts/bridges outlets are customized to provide suitable walking surfaces (concrete) for wildlife. A conceptual outlet treatment exhibit for wildlife crossings is provided in Appendix I, and further details regarding wildlife crossing considerations are discussed below in Section 3.10.

Table 8: Cross Culvert Inlet and Outlet Protection Summary

CP	HEC-14 Wire-Tied Riprap Outlet Basin Dimensions(ft)				Cutoff Wall Per ADOT "B" Standard Drawings	
	hs	Ls	L_A	L_B	Culvert Inlet	Culvert Outlet
1	1.3	30.0	10.0	40.0	B-04.30	B-04.10
2	1.5	30.0	10.0	40.0	B-04.70	B-04.50
4	1.6	30.0	10.0	40.0	B-04.70	B-04.50
5	2.0	19.8	9.9	29.7	B-04.30	B-04.10
7	1.6	24.0	8.0	32.0	B-04.70	B-04.50
8	1.1	11.0	5.5	16.5	B-11.14	B-11.11
9	Culvert outflow to a shotcrete lined channel				B-11.14	B-11.11
La Cholla 534+93	1.5	15.1	7.6	22.7	B-11.11	B-11.11
10	1.5	14.5	7.3	21.8	B-11.14	B-11.11
11	1.5	15.0	7.5	22.5	B-11.14	B-11.14
13	1.6	30.0	10.0	40.0	B-04.70	B-04.50
14	1.2	11.9	5.9	17.8	B-11.11	B-11.11
15	1.3	13.2	6.6	19.8	B-11.11	B-11.11
16	1.8	17.7	8.9	26.6	B-11.14	B-11.14
17	1.7	16.6	8.3	24.9	B-11.14	B-11.14
19	2.2	36.0	12.0	48.0	B-04.80	B-04.60
20	1.8	18.5	9.2	27.7	B-11.14	B-11.14
21	1.2	24.0	8.0	32.0	B-04.30	B-04.10
22	1.8	30.0	10.0	40.0	B-04.30	B-04.10
23	Substitute 6'W x 10'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				B-11.11	B-11.11
24	1.8	17.9	9.0	26.9	B-11.14	B-11.14
25	1.3	24.0	8.0	32.0	B-04.70	B-04.50
26.1	1.1	10.9	5.5	16.4	B-11.14	B-11.11
26.2	Substitute 6'W x 10'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				N/A	B-11.11
27 E	Substitute 12'W x 10'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				B-11.14	B-11.11
27	0.9	8.7	4.4	13.1	B-11.14	B-11.11
28	1.5	24.0	8.0	32.0	B-04.30	B-04.10
29	1.7	16.8	8.4	25.1	B-11.14	B-11.14
30	1.9	19.0	9.5	28.4	B-11.14	B-11.14
31	1.4	13.9	6.9	20.8	B-11.11	B-11.11
32	2.4	30.0	12.0	42.0	B-04.30	B-04.10
33.1	1.4	13.9	6.9	20.8	B-11.11	B-11.11
35	1.5	30.0	10.0	40.0	B-04.70	B-04.50
36	Substitute 10'W x 25'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				B-11.11	B-11.11
37	1.3	13.4	6.7	20.0	B-11.11	B-11.11
39	1.4	30.0	10.0	40.0	B-04.30	B-04.10
40	1.1	30.0	10.0	40.0	B-04.70	B-04.50
41	1.4	14.4	7.2	21.5	B-11.11	B-11.11
44	1.6	30.0	10.0	40.0	B-04.30	B-04.10
45	1.7	16.8	8.4	25.2	B-11.14	B-11.14

Table 8: Cross Culvert Inlet and Outlet Protection Summary

CP	HEC-14 Wire-Tied Riprap Outlet Basin Dimensions(ft)				Cutoff Wall Per ADOT "B" Standard Drawings	
	hs	Ls	L_A	L_B	Culvert Inlet	Culvert Outlet
46	1.3	30.0	10.0	40.0	B-04.30	B-04.10
47	1.2	11.8	5.9	17.7	N/A	B-11.11
48	1.2	12.2	6.1	18.4	B-11.11	B-11.11
49.1	1.1	10.8	5.4	16.2	B-11.11	B-11.11
52.1	2.0	30.0	10.0	40.0	B-04.70	B-04.50
53	1.4	13.9	7.0	20.9	B-11.11	B-11.11
54	1.7	16.6	8.3	24.9	B-11.11	B-11.11
55	1.5	14.9	7.5	22.4	B-11.14	B-11.14
55.1	Culvert outflow to a shotcrete lined channel				B-11.11	B-11.11
56	0.7	30.0	10.0	40.0	B-04.70	B-04.50
58	1.5	14.7	7.4	22.1	B-11.11	B-11.11
59	1.6	16.0	8.0	24.0	B-11.14	B-11.14
60	1.4	24.0	8.0	32.0	B-04.70	B-04.50
61	1.6	15.6	7.8	23.4	B-11.11	B-11.11
62.2	1.5	15.4	7.7	23.1	B-11.11	B-11.11
62.3	Culvert outflow to a shotcrete lined channel				B-11.11	B-11.11
63	1.6	15.7	7.8	23.5	B-11.11	B-11.11
64	1.8	17.6	8.8	26.3	B-11.13	B-11.13
65	Culvert outflow to a shotcrete lined channel				B-11.11	B-11.11
66	1.4	24.0	8.0	32.0	B-04.70	B-04.50
67.1	Culvert outflow to a shotcrete lined channel				B-11.11	B-11.11
67	1.4	30.0	10.0	40.0	B-04.70	B-04.50
70	Substitute 17'W x 10'L x 3'T Riprap Outlet Apron, D50=18" underlain by Filter Fabric				B-11.11	B-11.11
FR1	1.4	14.3	7.2	21.5	B-11.11	B-11.11
FR2	Substitute 6'W x 10'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				B-11.11	B-11.11
FR3	Substitute 6'W x 10'L x 1'T Riprap Outlet Apron, D50=6" underlain by Filter Fabric				B-11.11	B-11.11
Sta 487+44 LT (Trico)	Substitute Full Channel Width x 65'L x 3'T Riprap Outlet Apron, D50=18" underlain by Filter Fabric				B-04.30	B-04.10

*h_s = the dissipater pool depth
L_S = Dissipator Pool
L_A = Apron Length
L_B = L_S + L_A

3.7 Right-of-Way Requirements

Insufficient right-of-way exists to contain several of the proposed drainage structures (primarily training berms). Proposed drainage easements are shown on the Tangerine Road Right-of-Way Plans, prepared by Psomas. These drainage easements are needed for drainage structures that extend beyond the proposed road right-of-way, or proposed ponding limits that extend outside the right-of-way and exceed pre-roadway flooding conditions beyond regulatory limits. To compare pre-roadway to post-roadway flooding conditions (approximate flooding depths and inundation limits), please refer to Figures 5 and 7 of this report.

3.8 Mitigation Measures

Per review of the Pima County RFCD GIS internet website, the proposed cross culverts at multiple locations may impact riparian habitat areas. Mitigation regulated under Section 16.30 of the Pima County Floodplain and Erosion Hazard Management Ordinance could be required if it is determined that the proposed construction impacts more than 1/3 acre of this habitat. Riparian mapping and mitigation for this project is being handled by the environmental consultant and is outside the scope of this report.

3.9 Permitting Requirements

The Pima County RDM includes a task to address whether or not the project encroaches on regulated floodplains and if a CLOMR and LOMR are required in the case of Federal Emergency Management (FEMA) floodplains.

The Tangerine Road drainage structures described in this report have been designed to contain water surface elevation and limits of flooding changes to within the 300-foot road right of way and/or within pre-project flood limits wherever possible. The flood hazard boundaries for FEMA-mapped watercourses east of Dove Mountain Boulevard are generally contained within well-defined watercourses and the proposed drainage structures will cause only minor changes to the channel location or floodplain limits. As such, there is limited benefit to revising the FEMA flood hazard mapping, unless required by local floodplain regulations.

West of Dove Mountain Boulevard, the FEMA flood hazard areas cover most of Tangerine Road west of Prospect Wash as a result of wide spread sheet flooding. The proposed roadway drainage structures along this reach include several training dikes, channels and an elevated roadway profile, which all function to collect sheet flow and divert it to the culvert openings. At some locations, these structures may result in changes to the downstream flooding limits, i.e. decrease the area of flooding. However, such changes occur as a result of the aforementioned structures, some of which may be considered by FEMA to be levee-like structures or non-accredited levees. FEMA policy does not allow modifications to the flood hazard boundaries based on structures that are not designed to comply with FEMA levee design and accreditation standards.

The exception to the above described areas west of Dove Mountain Boulevard is at the very west end of the study area near the UPRR crossing. Proposed west end regional drainage improvements will result in significant drainage impacts on FEMA floodplains. For example, 100-Year peak discharge rates, runoff volumes, and flooding depths are estimated to increase at the northeast corner of the UPRR/Tangerine Road. Effective FEMA floodplain zone designations in this area are Zone AO1 and Zone AH. In this area a Conditional Letter of Map Revision (CLOMR) should be considered to address the expected flood depth increases in this localized area.

A jurisdictional delineation to determine watercourses which are subject to Section 404 Clean Water Act regulations has been completed by the environmental consultant for this project. All Section 404 permitting for this project is being handled by the environmental consultant and is outside the scope of this study. Drainage system design information will be provided to the environmental consultant by CMG Drainage Engineering, Inc. or Psomas Inc. for use toward determining impacts to Section 404 jurisdictional waters.

3.10 Wildlife Crossing Considerations

As shown on the Wildlife Crossing & Fencing exhibit in Appendix I (Psomas, 04-24-2012), two large wildlife crossings and six medium wildlife crossings have been identified for the project by the TAC. Fences are proposed along certain portions of the roadway alignment to direct wildlife to the wildlife crossings. CMG recommends that where wildlife fencing is necessary, the north side fence alignment should extend from each culvert headwall to culvert headwall and be placed either at the top of, or on the 4:1 or 6:1 north roadway slopes, above the water surface elevations in the roadside channels, in between culverts. On the south side, we recommend that the fence be brought in to each culvert headwall, not just the box culverts or wildlife crossings, so as not to interfere with cross drainage flow.

Per directions from the TAC, at medium wildlife crossings, 4-inch high transverse concrete sills will be provided on the culvert floor within the culvert on 10-foot centers to promote natural substrate deposition on the culvert bottoms for wildlife usage. These culverts have been hydraulically modeled with a 4-inch embedment to confirm that headwater limits were not exceeded should sediment accumulate up to the sill levels. The culverts at the medium wildlife crossing locations all function under inlet control, so the internal sills have a minor effect on inlet headwater levels. The sills will likely produce higher Manning's n values for the culvert bottoms and have the effect of reducing outlet velocities. For this report, however, the culverts have been designed with the standard concrete Manning's n values to promote a more conservative approach to outlet protection design.

If drop inlets are proposed, 10:1 (H:V) slopes, which are armored with shotcrete, have been used to provide relatively mild slope for wildlife access. At the two large wildlife crossings, single span bridges are utilized for this purpose. It is expected that significant degradation could occur at the bridge outlets as a result of concentrating existing sheetflow and funneling it through the new bridge openings. Grade control structures (sills) are necessary to protect the bridge outlets. To accommodate large wildlife accessibility, two additional 6-foot deep cutoff walls, spaced approximately 20 feet apart, are proposed downstream of the bridge outlet grade control structures. Scour hole depths between the bridge grade control structures and the two cutoff walls are anticipated to be insignificant enough to not impede wildlife access. Details of the wildlife crossings are shown on Figure 7 (Sheet 6 of 6) and on details in Appendix I.

SECTION 4.0 Bridge Analysis

4.1 Overview

There are two proposed bridges within the project limits. One of the bridges is located at Sta. 661+84 at Prospect Wash, and the other is located at Sta. 575+67. The main purpose of the bridge at Sta. 575+76 is to provide a wildlife crossing; however, storm water runoff from the local watershed will be directed to this bridge to fully utilize its conveyance capacity.

Prospect Wash drains across Tangerine Road as sheet flow but there are three locations with well defined channels. The natural channels associated with the easternmost location and the one near Sta. 661+84 cross the road at mild skew angles (~30 degrees), while the westernmost channel has a skew angle of more than 80 degrees. This large skew angle requires the bridge length to increase due to poor hydraulic conditions. Therefore, only the two natural channel locations with mild skew angles (~30 degrees) were chosen to install cross drainage structures. The natural channel near Sta. 661+84 is a large wildlife crossing, while the easternmost location is not. Therefore, the cross drainage structure near Sta. 661+84 was proposed to be a bridge with minimum height of 9' (large wildlife crossing requirement), while a box culvert was proposed at the easternmost location. Because the two drainage structures (box culvert and a bridge) at prospect Wash are subject to widespread sheet flow from the same source and a portion of the runoff that drains to the box culvert breaks out and flows west to the bridge, the box culvert is discussed in this section along with the bridge at Prospect Wash.

4.2 Design Criteria

Under existing conditions, offsite drainage approaches Tangerine Road at the two bridge locations as widespread sheet flow. It is difficult to accurately quantify the amount of runoff approaching each of the bridges. CMG Drainage Engineering Inc. coordinated with the project design team and Town of Marana and agreed that flow distribution in the FLO-2D model, aerial photograph, topography, and geomorphology, be used as the basis for determining the design flow for the bridges. At Prospect Wash, the design flow for the east culvert (5-10'x4' RCBC) and west bridge are 50% and 80% of the100-year discharge rate (5074 cfs), respectively. In addition, it was decided that the west bridge at Prospect Wash should not be under pressure flow assuming 100% of the100-year discharge reaches this location. For the bridge at Sta. 575+67, an approximately 360-feet long training berm extending outside of the right of way will be needed to capture 90% of the runoff at CP-62.

The 200-year discharge, which is assumed to be 40% higher than 100-year discharge rates (per the Draft Bridge Design Guidelines prepared by PCRFCD and Pima County Department of Transportation, November 2011), will be used as the extreme event for the bridge scour analysis. Per the draft Bridge Design Guideline, the three bridges must provide a minimum 1-foot freeboard above the 100-year water surface elevation (the design requirement for bridges where the 100-year discharge is less than 5,000 cfs).

4.3 Hydraulic Modeling

HEC-RAS was used to analyze the bridges and the box culverts. Each HEC-RAS model consists of six cross sections, three upstream and three downstream of the bridges. Abutment slopes of 1 horizontal to 1 vertical (1:1) were proposed at the bridges because sloping abutments require less scour depths compared to vertical abutments. Ineffective flow areas upstream and downstream of the bridges were modeled using 2:1 expansion and 1:1 contraction ratios.

The natural longitudinal stream slopes in the vicinity of the bridges are generally 2% to 3%. It is very possible that supercritical flows could be observed within reaches of those steep streams and within the bridges. Therefore, mixed flow regime was used in HEC-RAS for scour analyses, which would provide the most conservative results for this purpose. However, the water surface elevations shown on Figure 7 are obtained from subcritical flow regime, because subcritical flow regime generally generates slightly higher water surface elevations (to be conservative) and is widely instructed for use on natural streams by agencies, including FEMA.

For the culvert at Sta. 672+50, the design flow is 2537 cfs. The culvert is 5-10’x4’ RCBC with a skew angle of 30 degrees at the roadway. An armored (colored shotcrete-lined upstream face) training berm will extend from the west culvert inlet wingwall to the north right of way line to prevent breakout to the west once flow nears the opening. Upstream of the right of way line, the channel banks are shallow so a significant amount of flow will break toward the west bridge (up to 1075 cfs according to the HEC-RAS split flow model). Most of the breakout flow will reach the road embankment between the bridges so a collector channel is needed to convey this flow (1075 cfs) to the west bridge at Station 661+84. The proposed channel has a top width of 49 feet, depth of up to 3.5 feet and will be lined with colored shotcrete.

For the bridge at Sta. 661+84, the design flow is 4059 cfs. The bridge is a single span bridge with a dimension of 104’ x 9’ (W x H) and a skew angle of 30 degrees to the roadway. An approximate 800-foot long training berm is proposed at the west side of the bridge to prevent flow from breaking out to the west. The stream side of the training berm will be armored with colored shotcrete to prevent erosion. A minimum 3 feet of freeboard is recommended for this training berm to prevent runoff from topping the embankment due to wave action or sediment deposition. A training berm detail is provided on Figure 7.

For the bridge at Sta. 575+67, the design flow is 637 cfs. The bridge is a single span bridge with a dimension of 60' x 9' (W x H) and a skew angle of 30 degrees to the roadway. An approximate 350-foot long training berm is proposed at the west abutment of the bridge to prevent flow from breaking to the west. The stream side of the training berm will be armored with colored shotcrete to prevent erosion. A minimum 1 feet of freeboard is recommended for this training berm to prevent runoff from topping the embankment. A training berm detail is provided on Figure 7.

The results of the bridge HEC-RAS analysis are summarized in Table 9, and are provided in more detail in Appendix J.

Table 9: Bridge/Culvert Hydraulic Design Summary Table

Bridge/Culvert Station	575+67	661+84	672+50
Location	Wildlife Crossing	Prospect Wash West	Prospect Wash East
100-Year Design Flow (cfs)	637	4059	2537
Bridge Dimension (W x H)	1-60'x9'	1-104'x9'	5-10'x4' RCBC
Outlet Velocity (fps)	8.5	11.8	12.3
Headwater Elevation (ft)	2241.0	2475.4	2492.1
Split flow to West (cfs)	N/A	N/A	1075

4.4 Scour Analysis

Scour computations were conducted for the bridge structures (not the culvert structure at Sta. 672+50) using procedures outlined in The FHWA Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges", using Chapter 6, Section 6.6 of the Standard Manual for Drainage Design and Floodplain Management in Tucson, Arizona (SMDDFM), and, ADWR Design Manual for Engineering Analysis of Fluvial Systems, Simons, Li and Associates, March 1985.

The design scour calculated using this method is the summation of several scour components, including general scour, antidune trough depth, low flow thalweg depth, bend scour, and scour caused by local hydraulic conditions. The total scour depth is the sum of the short-term scour components and the long-term aggradation/degradation component.

The total scour analysis also includes a long-term degradation component to account for the possibility of upstream or downstream changes that could affect the channel bed profile. Long-term Aggradation/Degradation for the cutoff wall downstream of the bridges was analyzed using Equation 5.12 of the ADWR Design Manual for Engineering Analysis of Fluvial Systems, while long-term degradation for the structures upstream of the bridges was analyzed using Equation 6.26 of the City of Tucson Standards Manual for Drainage Design & Floodplain Management. These procedures determine the equilibrium channel slope for the watercourse, and account for existing urbanization in the contributing watershed as well as local changes in hydraulic conditions associated with the proposed bridge structures. The reduction of sediment supply factor for urbanization was 1 to 5 percent since most of the watersheds have land use densities that are relatively low.

Bridges are proposed in the vicinity of roadway Stations 575+67 and 661+84. The bridge at 661+84 is located on the west branch of Prospect Wash. The proposed bridge at 575+67 will receive stormwater from a portion of Ruelas Canyon Wash. However, its primary function is to serve as a wildlife crossing. The type of scour occurring in the vicinity of the bridges includes (1) scour at the bridge abutments (including general scour, antidune trough depth, low flow thalweg depth, bend scour, contraction scour and long-term degradation), (2) scour along the roadway embankments where flow impinges and is turned toward the bridge aperture, and, (3) scour adjoining training berms that will be constructed north of the west bridge abutments to prevent flow from breaking out in that direction. Scour at the bridge abutments is based on the 200-year storm discharge as recommended in the draft, "Guidelines for Determining Freeboard and Scour Analysis for Bridges in Pima County", April 2011. Scour computations for the roadway embankment and the training berms are based on the 100-year storm discharge. Long-term degradation computations are based on the 10-year discharge or estimated discharge for dominant flow conditions. Long-term degradation is primarily expected to occur as a result of local changes in the hydraulic conditions caused by contracting flow in the vicinity of the bridges. This form of degradation is unavoidable because of the significant change is hydraulic and sediment transport conditions associated with the contraction. Grade control structures to be located between the east and west toes of the roadway embankment at the outlet of the bridges are recommended to limit long-term degradation at the abutments. Scour calculation results given below are based on a grade control at this location. Grade control structures depths would be either the sum of local and long term scour depths or the vertical drop scour depths, whichever is greater. Riprap aprons are also recommended to mitigate additional scour associated with development of a vertical drop on the downstream of these grade control structures. Riprap apron lengths were recommended to be 6 times of the vertical drop scour depths at the grade control structures. A conceptual bridge outlet treatment exhibit for wildlife crossings is provided in Appendix I. Computation sheets for the scour analyses are provided in Appendix J of this report.

Table 10: Bridge Scour Summary Table

Bridge at Station 575+67			
Location	**Design Storm**	**Design Discharge (cfs)**	**Scour Depth (ft)**
Bridge Abutments	200-year	892	7.1
Upstream Training Berm	100-year	637	7.4
Road Embankment	100-year	637	5.7
Long-Term Degradation Downstream of Grade Control Structure	10-year	320	5.5
Local Scour at Grade Control Structure	100-year	637	3.0
Vertical Drop Scour at Grade Control Structure	100-year	637	8.3
Minimum Grade Control Structure Depth	8.5 feet		
Recommended Riprap Apron Length Downstream of Grade Control Structure	50 feet (D50=18”)		
Bridge at Station 661+84			
Location	**Design Storm**	**Design Discharge (cfs)**	**Scour Depth (ft)**
Bridge Abutments	200-year	5683	12.2
Upstream Training Berm	100-year	4059	8.3
Road Embankment	100-year	1075	5.0*
Long-Term Degradation Downstream of Grade Control Structure	10-year	1600	8.3
Local Scour at Grade Control Structure	100-year	4059	3.7
Vertical Drop Scour at Grade Control Structure	100-year	4059	14.6
Minimum Grade Control Structure Depth	14.6 feet		
Recommended Riprap Apron Length Downstream of Grade Control Structure	90 feet (D50=18”)		

*no scour will occur along embankment if colored shotcrete channel is provided.

SECTION 5.0 QUALITY CONTROL

A quality control review of this report and supporting computations was performed per the Psomas project Quality Control Plan. A quality control Certificate of Compliance is included in Appendix L.

SECTION 6.0 REFERENCES

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