MONTANA VISTA DRAINAGE MASTER PLAN

FINAL SUBMITTAL

DOÑA ANA COUNTY FLOOD COMMISSION

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July 2017
Rev. July 2017
Smith Project No.: 116119

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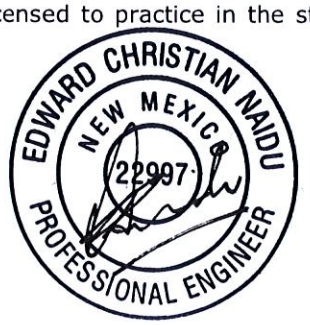
FINAL SUBMITTAL

DOÑA ANA COUNTY FLOOD COMMISSION

The technical material and data contained in this document were prepared under the supervision and direction of the undersigned, whose seal as a professional engineer licensed to practice in the state of New Mexico, is affixed below.



E. Christian Naidu, PE

Solutions for Today…

Vision for Tomorrow

July 5, 2017

John Gwynne, PE
Michael Garza, EI
Doña Ana County Flood Commission
County Government Center
845 N. Motel Blvd Room 1-250
Las Cruces, New Mexico 88007

Re: Montana Vista Drainage Master Plan

Smith #: 116119

Dear Mr. Gwynne and Mr. Garza:

I am pleased to submit the Final Drainage Report for the Montana Vista Drainage Master Plan. This report concludes findings based on analyses of the existing watershed conditions. It identifies areas of elevated risk and includes options for proposed improvements. The selected options have been refined and cost estimates for the recommended facilities are included. All comments from the 90% review have been incorporated into this final report.

Please feel free to contact me at any time with questions.

Sincerely,
Smith Engineering Company

E. Christian Naidu, PE
Project Manager

Enclosure: Montana Vista Drainage Master Plan Final Submittal

cc: Francisco Urueta, PE, Smith Engineering, Carl Lukesh, DACFC

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ACKNOWLEDGMENTS

DACFC for providing necessary digital files to perform the drainage study

EXECUTIVE SUMMARY

DESCRIPTION AND PURPOSE OF PROJECT

This Drainage Report was prepared by Smith Engineering Company (Smith) for the Doña Ana County Flood Commission (DACFC) to study the Montana Vista watershed. The Montana Vista watershed is approximately 24 miles south of Las Cruces. An existing conditions hydrologic model was developed. Based on the results of the existing conditions model, areas of potential flooding were identified and proposed drainage improvement options were developed to mitigate flooding. The hydrologic conditions were evaluated using the HEC-HMS hydrologic modeling software. Simulations were run for three storms as follows: 10 year, 50 year and 100 year return periods of 24 hour duration. The DACFC's design criteria for flood mitigation is the 10 year - 24 hour storm.

Description of Watershed and Existing Drainage Infrastructure

The Montana Vista watershed has a total drainage area of 13.7 square miles. The watershed is divided into two distinct sections by I-10. Approximately 75% of the watershed is east of I-10 which is undeveloped semi-arid rangeland with fair to extremely steep and rocky areas, particularly on the uppermost parts of the watershed. East of I-10 the watershed contains two existing dams, called Breedlove and Lauson Dams. The following table presents the embankment height and storage volume of the dams. **Figure E.1** shows their location in the watershed.

Dam Name	Design Storage Volume	Dam Height
Breedlove Dam	218 ac-ft.	23 ft.
Lauson Dam	471 ac-ft.	23 ft.

There are several large culverts under I-10 that convey flows from the east side of I-10 to west side of the watershed. These structures were evaluated for their maximum discharge capacity and the structures are shown on **Figure E.1**. The west side of the subbasin consists primarily of a mixture of agricultural fields and low density residential areas. Heavy industrial commercial areas are minimal. Most low density residential areas are on large acre lots that are predominantly pervious.

SUMMARY OF EXISTING PROBLEM AREAS AND PROPOSED OPTIONS

Figure E.1 shows the subbasins that would at the highest risk for flooding for the 10 year - 24 hour return period storm. These subbasins are vulnerable to offsite flows that are conveyed through the culverts under I-10. **Figure E.1** also has a summary table that compares the 10-year discharge versus the culvert capacity at the inflow points along I-10. A detailed summary of culvert capacities is discussed and presented in the report in **Table 1**. The two dams discussed above fully control the 10-year discharge. Reservoir routing results for the two dams are presented in **Table 2.**

Table 2

Detention Pond Routing Summary - Existing Ponds

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Top of Principal Spillway Elevation	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	ft	**ft**	**ft**
Breedlove Dam	Existing	Existing & Proposed	100 / 24	2739	891	273.0	273.0	218.0	125.0	**3929.3**	**3920**	3928	3911	23	18.3	3934	**-1.3**	**4.7**
Breedlove Dam	Existing	Existing & Proposed	50 / 24	1962	274	216.0	216.0	218.0	113.0	**3928.6**	**3920**	3928	3911	23	17.6	3934	**-0.6**	**5.4**
Breedlove Dam	Existing	Existing & Proposed	10 / 24	954	48	379.0	114.0	218.0	60.0	**3924.4**	**3920**	3928	3911	23	13.4	3934	**3.6**	**9.6**
Lauson Dam	Existing	Existing & Proposed	100 / 24	4916	548	472.0	472.0	471.0	283.0	**3964.7**	3956	3964	3947	23	17.7	3970	**-0.7**	**5.3**
Lauson Dam	Existing	Existing & Proposed	50 / 24	3608	109	379.0	379.0	471.0	267.0	**3964.1**	3956	3964	3947	23	17.1	3970	**-0.1**	**5.9**
Lauson Dam	Existing	Existing & Proposed	10 / 24	1838	46	212.0	212.0	471.0	201.3	**3958.8**	3956	3964	3947	23	11.8	3970	**5.2**	**11.2**

a - Refer to Existing Plans in Appendix B for Existing Dams / Pond Plans and contour maps, refer to Figures included in report text for Proposed Detention Pond Conceptual Grading Plans (AutoCAD drawings of these grading plans are included in this Appendix)

b - Refer to Appendix D for the HEC-HMS model output for the pond routing results. Dead storage was simulated for 2ft. below the principal spillway to account coservatively for heavy sediment loads therefore inflow volume is not equal to outflow volume

c - See Appendix C for all Elevation - Storage Volume - Discharge Data Tables

d- Negative number indicates the flow depth exceeds referenced elevation - no freeboard available

CONCLUSIONS AND RECOMMENDATIONS

Four ponds were simulated to attenuate peak discharges into the subbasins identified to be at elevated risk on the west side of I-10. **Figure E.2** shows the overview of all four ponds and the effect they have on peak discharge reduction. Based on the findings of the 60% report the DACFC provided input as far as what the priority for each of the ponds would be. The table below lists the ponds in the order of descending importance and the engineer's opinion of probable cost.

Facility Name	POND NAME	TOTAL STORAGE VOLUME (AC-FT.)	IS POND JURISDICTIONAL	TOTAL COST
Facility 1	Pond 2	13	NO	$719,000
Facility 2	Pond 3	27.9	NO	$1,154,000
Facility 3	Pond 4	31.8	NO	$1,351,000
Facility 4	Pond 1	26.2	NO	$1,193,000

Facilities 1-3 can fully control the 10-year discharge. However, facility 4 allows significant flow to spill through the emergency spillway. This is largely due to this facility having a tributary area of approximately 2 square miles with no upstream facilities to control discharge. The tributary area to the east of I-10 is part of the Bureau of Land Management's National Monument therefore no facilities can be ever constructed within those limits.

As such, any proposed facility built to fully contain the 10-year discharge volume will have to be a jurisdictional facility as the inflow volume for the **10-year storm is 62 ac-ft. At 62 ac-ft., the dam can remain non-jurisdictional if the height is less than 6 ft.**

Culvert Name / Location Description	CULVERT DATA FOR CULVERT MASTER						Culvert Capacity			HEC-HMS Analysis Point Name	10-yr 24-hr storm				50-yr 24-hr storm				100-yr 24-hr storm			
	Existing or Proposed	Comment on Inlet Sediment or Debris	No. of Culverts	Material	Culvert Rise	Culvert Span	Maximum Culvert Capacity from Culvert Master	Maximum Culvert Capacity assuming 15% Clogging Factor	Discharge Per Culvert		Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)	Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity)	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)	Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)
					feet	feet	cfs	cfs	cfs		cfs				cfs				cfs			
C1 CROSSING I-10	Existing	Fully open	6	CBC	4	6.00	1572	1336	223	J49	456	880	N	0.0	1057	279	N	0.0	1533	-197	Y	0.9
C2 CROSSING I-10	Existing	60% open	1	CMP	2	2.00	15	13	13	NA	13	0	Y	0.0	0	13	N	0.0	0	13	N	0.0
C3 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0	277	-31	Y	0.1	423	-177	Y	0.7
C3.1 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0	277	-31	Y	0.1	423	-177	Y	0.7
C4 I-10 CROSSING (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Existing	Fully open	6	CBC	4	8.00	2046	1739	290	JTC4	48	1691	N	0.0	275	1464	N	0.0	896	843	N	0.0
C4.1 WEST FRONTAGE RD (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Existng	Fully open	6	CBC	4	8.00	1777	1510	252	JTC4	48	1462	N	0.0	275	1235	N	0.0	896	614	N	0.0
C4.3 I-10 CROSSING	Existng	Fully open	3	CMP	4	4.00	226	192	64	JTC4.3	141	51	N	0.0	359	-167	Y	2.6	535	-343	Y	5.4
C4.4 I-10 CROSSING	Existng	Fully open	1	CMP	4	4.00	81	69	69	NA	0	69	N	0.0	0	69	N	0.0	0	69	N	0.0
C4.5 I-10 CROSSING (CONVEYS OUFLOW FROM LAUSON UNDER I-10)	Existng	Fully open	3	CBC	8	10.00	1361	1157	386	J36.1	110	1047	N	0.0	288	869	N	0.0	567	590	N	0.0
C4.6 WEST FRONTAGE RD (CONVEYS FLOWS FROM LAUSON UNDER FRONTAGE RD)	Existng	Fully open	1	CMP	6.5	6.50	185	157	157	JT_DSC4.6	128	29	N	0.0	320	-163	Y	1.0	568	-411	Y	2.6
C22 WEST FRONTAGE RD, NORTH OF INTERCHANGE	Existng	Fully open	1	CMP	3	3.00	37	31	31	NA	0	31	N	0.0	0	31	N	0.0	0	31	N	0.0
C9 ANTHONY DR BETWEEN MONTANA VISTA DR & E JOY DR	Existng	Fully open	3	CBC	4	6.00	580	493	164	J21	20	473	N	0.0	51	442	N	0.0	76	417	N	0.0

Legend

SymbolID

- C3 I-10 Culvert ID
- UpstreamBasin
- High Risk Subbasins for 10Yr Storm
- Montana Vista Subbasins
- Existing Dams

131 / 219.8 — SUBBASIN ID & SUBBASIN AREA (AC)

2014 DIGITAL ELEVATION MODELS, 2 FT. INTERVAL LIDAR AND ORTHOPHOTOGRAPHY WAS PROVIDED BY DONA ANA COUNTY FLOOD COMMISSION

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE E.1

Subbasins at High Risk for 10Yr-24Hr Storm

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PREPARED BY

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JULY, 2017

NOT TO SCALE

Detention Pond Routing Summary Proposed Ponds

Facility ID (Land and Construction ID)	Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Total Approximate Cost
			inches	yr / hr	cfs	cfs	
Facility 4.1 & 4.2	Pond 1	Proposed	Proposed	10 / 24	458	273	$ 1,193,000
Facility 1.1 & 1.2	Pond 2	Proposed	Proposed	10 / 24	140	27	$ 719,000
Facility 2.1 & 2.2	Pond 3	Proposed	Proposed	10 / 24	199	28	$ 1,154,000
Facility 3.1 & 3.2	Pond 4	Proposed	Proposed	10 / 24	138	53	$ 1,351,000

POND 1
Ex. Q10 = 458 cfs
Prop. Q = 273 cfs

POND 2
Ex. Q10 = 140 cfs
Prop. Q = 27 cfs

POND 3
Ex. Q10 = 199 cfs
Prop. Q = 28 cfs

POND 4
Ex. Q10 = 138 cfs
Prop. Q = 53 cfs

Legend

- Culvert ID
- National Monuments
- Montana Vista Subbasins
- Dams
- EBID Facilities

131
219.8
SUBBASIN ID & SUBBASIN AREA (AC)

2014 DIGITAL ELEVATION MODELS, 2 FT. INTERVAL LIDAR AND ORTHOPHOTOGRAPHY WAS PROVIDED BY DONA ANA COUNTY FLOOD COMMISSION

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MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE E.2

Overview of Options

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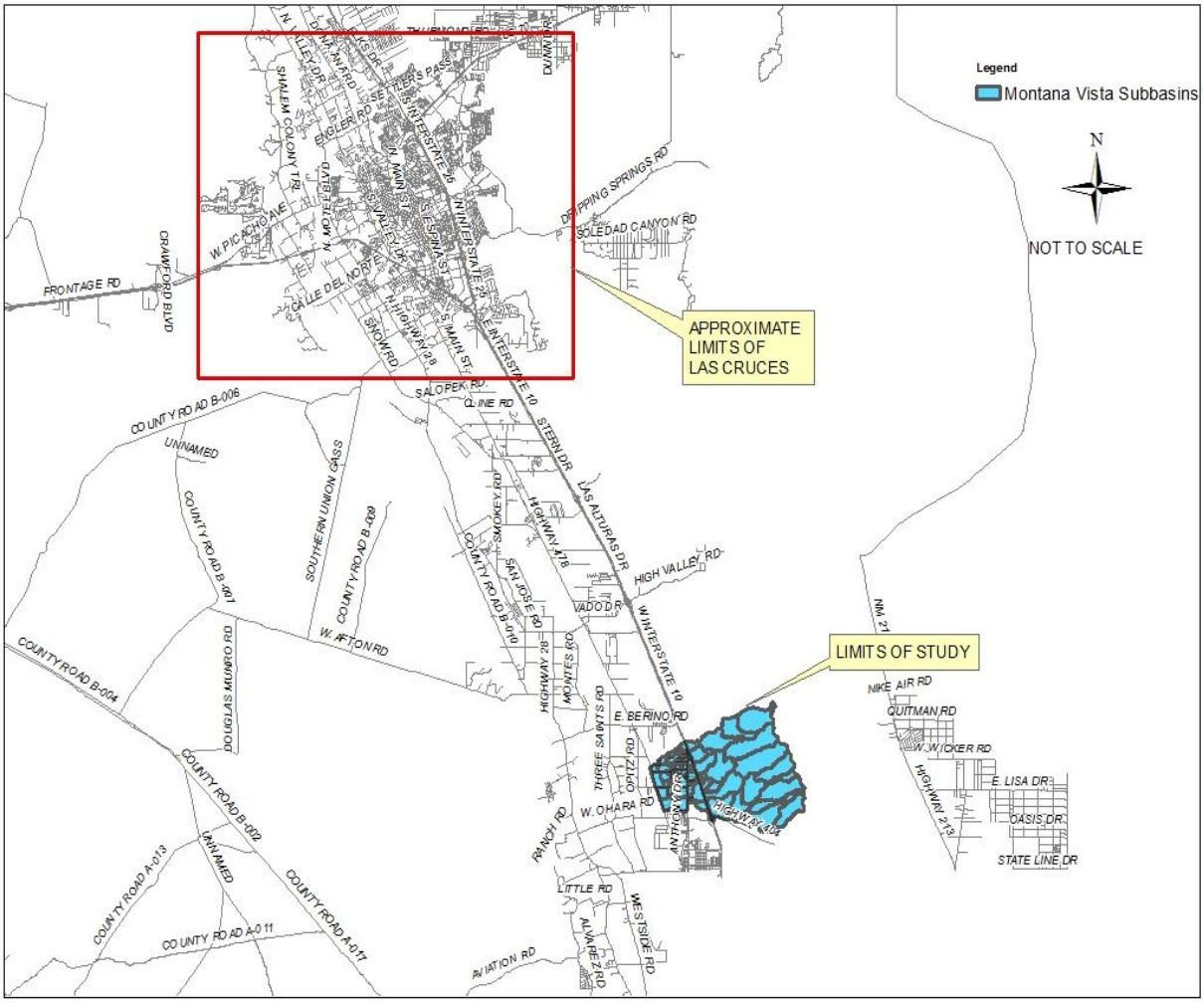
SECTION 1. GENERAL PROJECT INFORMATION

1.1 DESCRIPTION AND PURPOSE OF PROJECT

This drainage master plan was prepared by Smith Engineering Company (**Smith**) for the Doña Ana County Flood Commission (DACFC) to study the Montana Vista watershed. The Montana Vista watershed is approximately 24 miles south of Las Cruces. An existing conditions hydrologic model was developed. Based on the results of the existing conditions model, areas of potential flooding were identified and proposed drainage improvement options were developed to mitigate flooding. The hydrologic conditions were evaluated using the HEC-HMS hydrologic modeling software. Simulations were run for three storms as follows: 10 year, 50 year and 100 year return periods of 24 hour duration. **Figure 1** shows the project vicinity map.

Figure 1: Project Vicinity Map



1.2 FIELD OBSERVATION

Smith conducted field observations in December 2016. **Appendix A** contains annotated photographs of the various locations in the Montana Vista watershed, some of the I-10 culverts and of the two dams.

SECTION 2. EXISTING HYDROLOGIC AND HYDRAULIC ANALYSES

2.1 PREVIOUS STUDIES

No previous drainage studies were available for review for the entire watershed, however, DACFC provided Smith with design reports and final construction plans for both Breedlove and Lauson dams. These were used to build the elevation-storage-discharge rating curves to simulate them in the HEC-HMS model. A detailed description of all the assumptions and calculations are provided in **Appendix C**. The reports and plans for the dams are included digitally.

2.2 EXISTING FLOOD CONTROL AND CULVERT STRUCTURES

The Montana Vista watershed has two existing dams called Lauson and Breedlove dams that are under the jurisdiction of DACFC. Both are jurisdictional dams' due to their embankment height (23 ft.) and storage volume (471 & 218 ac-ft.). **Appendix B** contains record drawings and design reports for these dams. There are numerous culvert crossings under I-10. These were observed in the field and their maximum headwater depth was also estimated. Peak discharge capacities for these culvert structures were also computed based on maximum headwater depth. This information is presented in **Table 1 on page 8** and the dam and culvert locations are shown on **Figure 3 on page 9.**

2.3 DRAINAGE BASIN DESCRIPTION AND BASIN DELINEATION

A. Drainage Basin Description

The Montana Vista watershed has a total drainage area of 13.7 square miles. The basin is divided into two distinct sections by I-10. Approximately 75% of the watershed is east of I-10. The basin is undeveloped semi-arid rangeland with fair to extremely steep and rocky areas, particularly on the uppermost parts of the basin. The east side of the basin contains two dams, all located upstream and east of I-10. The west side of the basin consists primarily of a mixture of agricultural fields and low density residential areas. Heavy industrial commercial areas are minimal. Several Elephant Butte Irrigation District facilities are located on the western edge of the watershed. The watershed has small areas that are heavily impervious and most of residential areas are situated on large acre lots that are mostly pervious.

B. FEMA Floodplains

FEMA floodplains were downloaded from the FEMA website. Digital copies of panels are included in **Appendix B**.

C. Drainage Basin Delineation

The watershed limits were provided by the DACFC. The 2014 digital elevation models (DEMS) were then used to authenticate the outer watershed boundary. Once the boundary was refined, HEC-GeoHMS was used to delineate subbasins and refine subbasin boundaries. Analysis points were determined based on the following:

1. Outfall locations based on topography

2. Culvert locations
3. Existing features (dams, principal and emergency spillway outfall locations)
4. Drainage paths (soil or streets) within Montana Vista

Figure 2 on page 4 shows the overview of the subbasins for Montana Vista. **Figure 2.1 (Map Pocket)** presents the subbasins in more detail and better scale along with the location of the key culvert crossings. Culverts are identified as C1, C2, etc.

2.4 DRAINAGE ANALYSIS CRITERIA

A. Storms Evaluated

The DACFC requested that 10 year, 50 year and 100 year - 24 hour duration storms be simulated.

B. Design Storm

The DACFC requested that the design storm shall be 10 year 24-hour storm. The proposed options will not include design for the 50 year and 100 year - 24 hour storms, although the results will be included.

C. Hydrologic Computer Program

The US Army Corps of Engineers "HEC-HMS - Hydrologic Modeling System" program or commonly called "HEC-HMS" (Version 4.2.1) was selected for simulation of basin storm rainfall – runoff for existing basin and for the proposed options.

C. Existing Dams

For the existing and proposed options HEC-HMS models, both dams will be assumed to remain in place as they are certainly viable for the 10-year design storm. Details of the reservoir routing results for all dams will be provided later is this section for all storms simulated.

2.5 RAINFALL DATA

A. Rainfall Distribution

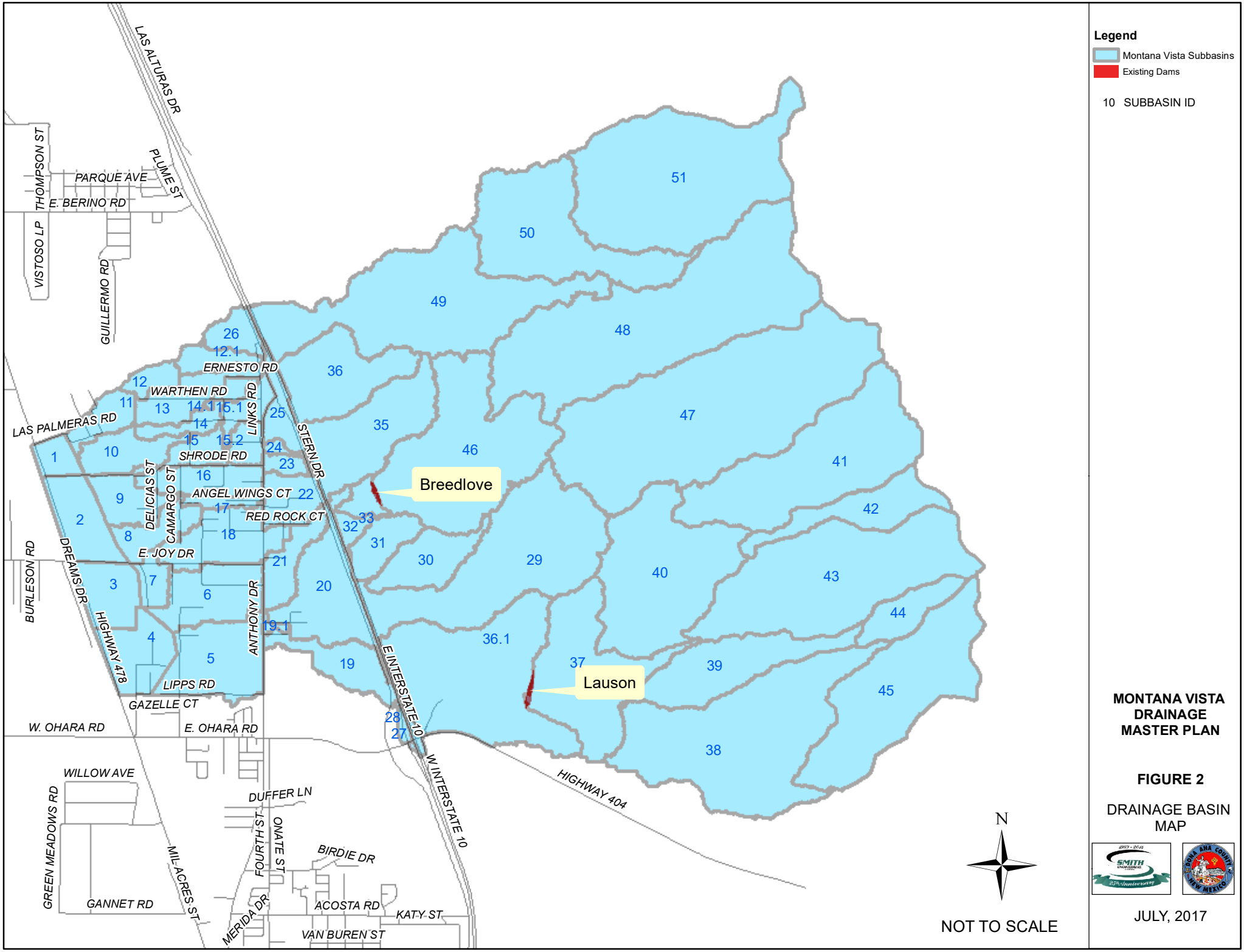
The Montana Vista watershed is located within the USDA Natural Resources Conservation Service (NRCS), previously the Soil Conservation Service (SCS) Type II rainfall distribution area. Please refer to **Appendix C** for Figure B-2 that illustrates the Type II boundaries. However, the DACFC dictated that the 25% Frequency Storm Distribution be adopted. That distribution is available in the HEC-HMS program and it places peak intensity of the rainfall in at 25% of the storm duration, or at 6 hours for a 24-hour storm.

A. Areal Reduction Factors

Areal reduction factors were considered from Figure 14 – NOAA Atlas 2, Vol. IV since NOAA 14 has not yet developed areal reduction factors. **Appendix C** contains a copy. Since the subbasin area was 13.7 square miles, no areal reduction was required.

B. Point Rainfall Data

Point rainfall data for was obtained from NOAA Atlas 14 website. **Table C1** in **Appendix C** contains the printouts from the NOAA Atlas 14-point rainfall data results.


Legend
Montana Vista Subbasins
Existing Dams
10 SUBBASIN ID
Breedlove
Lauson
LAS ALTURAS DR
PLUME ST
THOMPSON ST
PARQUE AVE
E. BERINO RD
VISTOSO LP
GUILLERMO RD
ERNESTO RD
WARTHEN RD
LINKS RD
LAS PALMERAS RD
SHRODE RD
DELICIAS ST
CAMARGO ST
ANGEL WINGS CT
RED ROCK CT
E. JOY DR
DREAMS DR
BURLESON RD
HIGHWAY 478
ANTHONY DR
LIPPS RD
GAZELLE CT
W. OHARA RD
E. OHARA RD
STERN DR
E INTERSTATE 10
W INTERSTATE 10
HIGHWAY 404
WILLOW AVE
GREEN MEADOWS RD
GANNET RD
MIL-ACRES ST
DUFFER LN
FOURTH ST
ONATE ST
BIRDIE DR
MERIDA DR
ACOSTA RD
KATY ST
VAN BUREN ST
N
NOT TO SCALE
MONTANA VISTA DRAINAGE MASTER PLAN
FIGURE 2
DRAINAGE BASIN MAP
JULY, 2017

2.6 SOILS DATA AND RUNOFF CURVE NUMBERS (CNS)

Natural Resources Conservation Service (NRCS) Web Soil Survey website was used to obtain soils data for the watershed.

http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

Appendix C contains the Web Soil Survey information. The Figures in **Appendix C** illustrate the soil map unit locations and tables that summarize the hydrologic soil groups and cover types for the various soil map units.

Table C2 (**Appendix C**) contains a summary of the CNs for each sub-basin and the areal weighted CN data and results for all sub-basins. The data and assumptions applied to develop **Table C2** are based on the following:

A. Antecedent Runoff Condition II (ARC II) is defined as the soil average runoff condition (moisture condition) by the NRCS. Antecedent Runoff Condition III (ARC III) is defined as the wetter soil condition. For all sub-basins denoted as "Arid and Semiarid Rangelands" with "Desert Shrub Cover Type" a composite (average) CN value between ARC II CN and ARC III CN was adopted.
B. Hydrologic Soil Group (A, B, C, or D) – Determined by the NRCS per soil map unit (**Appendix C** contains the Web Soil Survey Data).
C. Land Use Type is either – arid rangeland (most sub-basins), urban (within the community of Doña Ana) or cultivated agricultural land. The orthophotography as presented on the Drainage Basin Maps (map pocket) was used to make the land use type determinations. The CN tables are obtained from "Urban Hydrology for Small Watersheds, US Dept. of Agricultural Soil Conservation Service, Technical Release 55 (TR-55), June 1986. *
D. The TR-55 CN tables are listed here:
    Table 2-2a Runoff Curve Numbers for Urban Areas. *
    Table 2-2b Runoff Curve Numbers for Cultivated Agricultural Land. *
    Table 2-2c Runoff Curve Numbers for Other Agricultural Lands. *
    Table 2-2d Runoff Curve Numbers for Arid and Semiarid Rangelands. *

    *Copies are included in **Appendix C**

E. Cover Type, Hydrologic Condition and Percent Imperviousness

    Arid Rangeland - assumed Cover Type and Hydrologic Condition – Desert Shrub, etc., poor hydrologic condition (Table 2-2d applies)

    Urban - assumed Cover Type and Average Impervious Area – 1/8 acre 65%, impervious (Table 2-2a applies)

    Cultivated Agricultural Land - assumed Cover Type and Hydrologic Condition – Row Crops – Straight Row 65%, poor hydrologic condition (Table 2-2b applies)

F. CN selections were based on the previous data, assumptions and NRCS soils data / and Tables.

G. Areal weighted CNs were computed by areal weighting the CN per soil map unit by the acreage of that map unit relative to the total sub-basin acreage.

2.7 TRAVEL TIME (T_t), TIME OF CONCENTRATION (T_C), AND UNIT HYDROGRAPH LAG TIME (T_L) COMPUTATIONS AND UNIT HYDROGRAPH

A water course may have up to three sub-reaches that comprise the longest flow path. The upper overland/sheet flow reach, then a shallow concentrated flow reach followed by a channel reach. The NRCS TR-55 Tt and Tc method was applied to each water course. The time of concentration (Tc) for the watercourse equals the summation of travel times (Tt) from each sub-reach. **Appendix C** contains the TR-55 description and procedures.

The NRCS Unit Hydrograph Lag Time Method (T_L) was applied to the Tc to compute the unit hydrograph Time to Peak (Tp). Note that Lag Time = 0.6 Tc. **Appendix C** contains the reference pages from Part 630 Hydrology, National Engineering Handbook, May 2015, Chapter 15 that describes the lag time concept and method.

HEC-GeoHMS was used to delineate the longest flow path. Manning's Roughness Coefficients "n" assumptions were obtained from TR-55, by experience and by review of "n" value tables by Chow, 1959 (copies included in **Appendix C**).

Channel slopes were computed from elevations and length measurements from the drainage basin maps using the DACFC supplied imagery and LIDAR data (map pocket). Typical channel widths were also measured from the drainage basin maps and / or with Google Earth.

Tables C3 (**Appendix C**) summarizes the travel time, time of concentration and lag time data and results and **Figure 2.1** (**Map pocket**) shows the longest flow paths delineated for all the subbasins.

2.8 CHANNEL ROUTING

The "Muskingum-Cunge" channel routing method was applied to route hydrographs. Manning's "n" values were assumed based on experience and the Manning's "n" values from Chow, 1959 and locations of routing reaches as observed on the drainage basin maps. Bottom width assumptions were determined as the typical channel width from the DEM. **Table C4 (Appendix C)** presents the Muskingum-Cunge channel routing input data summary. Channel routing parameters were computed using HEC-GeoHMS and exported as a shapefile to form part of the background map in HEC-HMS to ease the review process.

Note that runoff losses to channel bed infiltration and percolation were assumed to be small and were therefore not simulated.

2.9 SEDIMENT BULKING

The HEC-HMS models simulate clear water hydrographs unless a "Flow Ratio" is applied to simulate sediment volume within hydrographs. This is called sediment bulking. Note that a sediment bulking value of about 17% is considered the limit before mud flow would occur.

Due to lack of site specific data, a sediment bulking factor of 10% or a factor of 1.10 was assumed for all undeveloped sub-basin hydrographs and a value of 1.05 was assumed for urbanized subbasin hydrographs. That assumption is based on review of information presented in Sediment and Erosion Design Guide, Nov. 2008, Mussetter Engineering Inc. **Appendix C** contains a copy of relevant pages from that document.

2.10 HYDROLOGIC DATA SUMMARY

Table C5 in **Appendix C** provides a summary of all the input variables required for the HEC-HMS model.

2.11 COMPUTATION TIME INCREMENT FOR HEC-HMS MODELS

While various procedures are available for assigning the computational time increment, DACFC prefers to use a time step of one minute. All simulations were run at a one minute time increment.

2.12 INFLOW DIVERSION FUNCTIONS AND UPSTREAM DETENTION AT CULVERT STRUCTURES

A. Inflow Diversion Functions

No inflow diversion functions were required for this study.

B. Upstream Detention at Culvert Structures

Typically, culvert structures that cross under major highways are built up against elevated embankments. This allows water to pond against the inlet structure. In some instances, the culverts are under capacity and cannot convey the peak discharges and as such, the embankments act as detention ponds where the water pools and spreads laterally**. The discharge rates to the downstream analysis points at these locations are therefore purely a function of maximum culvert capacity**. Any excess flows will pond at the embankment. In past versions, the program required an outflow curve that would include stage-storage-discharge data to perform reservoir routings. The discharge rating curve for the outlet structure had to be computed externally to HMS and then input as a paired data set. With the latest version of HEC-HMS V4.2.1, there are new features developed for reservoirs. The program now allows users to designate an outlet structure, for example a culvert outlet, as an outflow method. With the correct culvert parameters, HEC-HMS can compute an internal discharge rating curve based on inlet or outlet control flow regimes, however as in the past versions, the stage storage data must be computed externally. As such, upstream ponding was simulated using reservoirs for the following culverts: C1, C3, C4.2-4.3 and C4.6. Stage data was assigned based on measured maximum available headwater depth, storage was artificially manipulated so that the outlet discharge matched the computed discharge capacity of the culverts.

Upstream ponding due to under capacity culverts provides a significant benefit especially in the higher return period storms when the high peak discharges could significantly affect downstream areas. The locations of the culverts are presented on **Figure 2.1 (Map Pocket)** and **Figure 3 on page 9**.

2.13 RESERVOIR ROUTING DATA

The stage storage and discharge data for the existing dams are included in **Appendix C** along with a write up that documents all assumptions and calculations.

2.14 HEC-HMS HYDROLOGIC MODELS AND SUMMARY RESULTS

Unit peak discharges were computed and evaluated to ensure that the numbers are in the acceptable range for a watershed exhibiting the characteristics of semi-arid rangeland mixed with and low density urban development. Unit peak discharges were in the range of 1.1 – 3.1 cfs/acre which falls well within the acceptable range.

After evaluating the results from the existing conditions model, it was clear that the highest risk for potential flooding during a 10-year storm will be primarily be due to large inflows from the east side of the watershed. The existing large diameter culverts under I-10 will act as the inflow points. The local subbasins west of I-10 have small discharges in the 10-year storm and given that there is so much pervious area available for infiltration and spread, the potential

for flooding is very low. Breedlove and Lauson dams effectively control the 10-year storm. Culvert capacities and pond routings are discussed in more detail in the next section.

2.15 EXISTING DRAINAGE INFRASTRUCTURE HYDRAULIC CAPACITIES AND RISK

A. Existing Culvert Capacities

All existing culverts that convey flows under I-10 were evaluated for maximum discharge capacity. A 15% clogging factor was applied to account for debris. The peak inflow at these culverts was compared against their peak discharge capacity to see how much flow could be passed to the west side in the various storms. For some culverts, upstream ponding was simulated as discussed in Section 2.12.

Table 1 provides a performance summary of existing culverts. A comparison of the 10 year subbasin discharge at the culverts prove that many of the culverts have sufficient capacity to pass the 10 year flows from the east side of I-10 to the west side of I-10. As such, the subbasins on the west will be at higher risk for potential flooding in the 10-year storm. Based on the peak discharge capacity of the culverts and the ramifications downstream of I-10, a risk map (**Figure 3 on page 9)** was developed to show subbasins with the highest potential for flooding for the design storm.

Table 1

	CULVERT DATA FOR CULVERT MASTER						Culvert Capacity				10-yr 24-hr storm			
Culvert Name / Location Description	Existing or Proposed	Comment on Inlet Sediment or Debris	No. of Culverts	Material	Culvert Rise	Culvert Span	Maximum Culvert Capacity from Culvert Master	Maximum Cuvlert Capacity assuming 15% Clogging Factor	Discharge Per Culvert	HEC-HMS Analysis Point Name	Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity	Extra Culverts Required Y or N	No. of Extra Cuvlerts to pass flow (same as existing)
					feet	feet	cfs	cfs	cfs		cfs			
C1 CROSSING I-10	Existing	Fully open	6	CBC	4	6.00	1572	1336	223	J49	456	880	N	0.0
C2 CROSSING I-10	Existing	60% open	1	CMP	2	2.00	15	13	13	NA	13	0	Y	0.0
C3 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0
C3.1 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0
C4 I-10 CROSSING (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Existing	Fully open	6	CBC	4	8.00	2046	1739	290	JTC4	48	1691	N	0.0
C4.1 WEST FRONTAGE RD (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Exisitng	Fully open	6	CBC	4	8.00	1777	1510	252	JTC4	48	1462	N	0.0
C4.3 I-10 CROSSING	Exisitng	Fully open	3	CMP	4	4.00	226	192	64	JTC4.3	141	51	N	0.0
C4.4 I-10 CROSSING	Exisitng	Fully open	1	CMP	4	4.00	81	69	69	NA	0	69	N	0.0
C4.5 I-10 CROSSING (CONVEYS OUFLOW FROM LAUSON UNDER I-10)	Exisitng	Fully open	3	CBC	8	10.00	1361	1157	386	J36.1	110	1047	N	0.0
C4.6 WEST FRONTAGE RD (CONVEYS FLOWS FROM LAUSON UNDER FRONTAGE RD)	Exisitng	Fully open	1	CMP	6.5	6.50	185	157	157	JT_DSC4.6	128	29	N	0.0
C22 WEST FRONTAGE RD, NORTH OF INTERCHANGE	Exisitng	Fully open	1	CMP	3	3.00	37	31	31	NA	0	31	N	0.0
C9 ANTHONY DR BETWEEN MONTANA VISTA DR & E.JOY DR.	Exisitng	Fully open	3	CBC	4	6.00	580	493	164	J21	20	473	N	0.0

Culvert Name / Location Description	CULVERT DATA FOR CULVERT MASTER						Culvert Capacity			HEC-HMS Analysis Point Name	10-yr 24-hr storm				50-yr 24-hr storm				100-yr 24-hr storm			
	Existing or Proposed	Comment on Inlet Sediment or Debris	No. of Culverts	Material	Culvert Rise	Culvert Span	Maximum Culvert Capacity from Culvert Master	Maximum Culvert Capacity assuming 15% Clogging Factor	Discharge Per Culvert		Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)	Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity)	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)	Peak Discharge	Spill flow (Max. Capacity minus peak discharge) - positive means excess capacity	Extra Culverts Required Y or N	No. of Extra Culverts to pass flow (same as existing)
					feet	feet	cfs	cfs	cfs		cfs				cfs				cfs			
C1 CROSSING I-10	Existing	Fully open	6	CBC	4	6.00	1572	1336	223	J49	456	880	N	0.0	1057	279	N	0.0	1533	-197	Y	0.9
C2 CROSSING I-10	Existing	60% open	1	CMP	2	2.00	15	13	13	NA	13	0	Y	0.0	0	13	N	0.0	0	13	N	0.0
C3 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0	277	-31	Y	0.1	423	-177	Y	0.7
C3.1 I-10 CROSSING	Existing	Fully open	1	CBC	5	6.00	290	247	247	J35-36	106	141	N	0.0	277	-31	Y	0.1	423	-177	Y	0.7
C4 I-10 CROSSING (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Existing	Fully open	6	CBC	4	8.00	2046	1739	290	JTC4	48	1691	N	0.0	275	1464	N	0.0	896	843	N	0.0
C4.1 WEST FRONTAGE RD (CONVEYS FLOWS FROM BREEDLOVE UNDER I-10)	Existng	Fully open	6	CBC	4	8.00	1777	1510	252	JTC4	48	1462	N	0.0	275	1235	N	0.0	896	614	N	0.0
C4.3 I-10 CROSSING	Existng	Fully open	3	CMP	4	4.00	226	192	64	JTC4.3	141	51	N	0.0	359	-167	Y	2.6	535	-343	Y	5.4
C4.4 I-10 CROSSING	Existng	Fully open	1	CMP	4	4.00	81	69	69	NA	0	69	N	0.0	0	69	N	0.0	0	69	N	0.0
C4.5 I-10 CROSSING (CONVEYS OUFLOW FROM LAUSON UNDER I-10)	Existng	Fully open	3	CBC	8	10.00	1361	1157	386	J36.1	110	1047	N	0.0	288	869	N	0.0	567	590	N	0.0
C4.6 WEST FRONTAGE RD (CONVEYS FLOWS FROM LAUSON UNDER FRONTAGE RD)	Existng	Fully open	1	CMP	6.5	6.50	185	157	157	JT_DSC4.6	128	29	N	0.0	320	-163	Y	1.0	568	-411	Y	2.6
C22 WEST FRONTAGE RD, NORTH OF INTERCHANGE	Existng	Fully open	1	CMP	3	3.00	37	31	31	NA	0	31	N	0.0	0	31	N	0.0	0	31	N	0.0
C9 ANTHONY DR BETWEEN MONTANA VISTA DR & E JOY DR	Existng	Fully open	3	CBC	4	6.00	580	493	164	J21	20	473	N	0.0	51	442	N	0.0	76	417	N	0.0

Legend

SymbolID

- C3 I-10 Culvert ID
- UpstreamBasin
- High Risk Subbasins for 10Yr Storm
- Montana Vista Subbasins
- Existing Dams

131 / 219.8 — SUBBASIN ID & SUBBASIN AREA (AC)

2014 DIGITAL ELEVATION MODELS, 2 FT. INTERVAL LIDAR AND ORTHOPHOTOGRAPHY WAS PROVIDED BY DONA ANA COUNTY FLOOD COMMISSION

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 3

Subbasins at High Risk for 10Yr-24Hr Storm

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B. Existing Dams

Both Lauson and Breedlove dams fully control the 10-year discharge. However, while both dams spill through the emergency spillways for the 50 and 100 year storms, they still provide a significant reduction in peak discharge as shown in **Table 2.**

Table 2

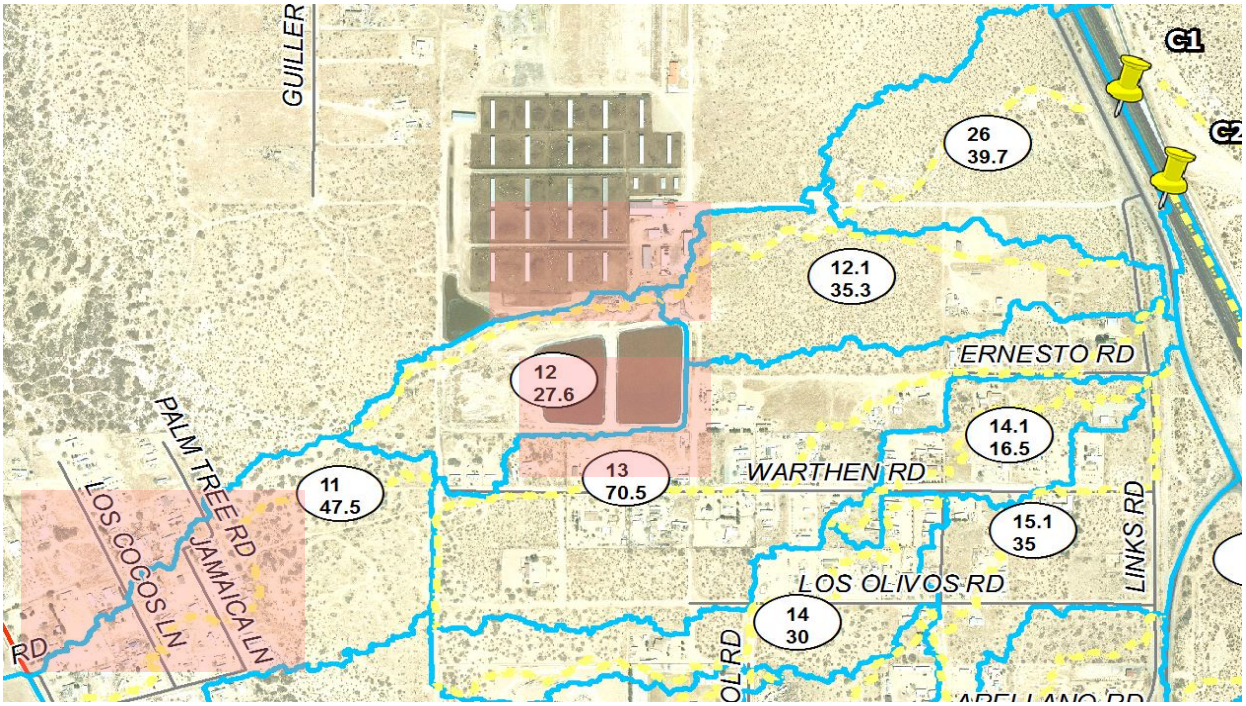
Detention Pond Routing Summary - Existing Ponds

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Top of Principal Spillway Elevation	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	ft	**ft**	**ft**
Breedlove Dam	Existing	Existing & Proposed	100 / 24	2739	891	273.0	273.0	218.0	125.0	**3929.3**	**3920**	3928	3911	23	18.3	3934	**-1.3**	**4.7**
Breedlove Dam	Existing	Existing & Proposed	50 / 24	1962	274	216.0	216.0	218.0	113.0	**3928.6**	**3920**	3928	3911	23	17.6	3934	**-0.6**	**5.4**
Breedlove Dam	Existing	Existing & Proposed	10 / 24	954	48	379.0	114.0	218.0	60.0	**3924.4**	**3920**	3928	3911	23	13.4	3934	**3.6**	**9.6**
Lauson Dam	Existing	Existing & Proposed	100 / 24	4916	548	472.0	472.0	471.0	283.0	**3964.7**	3956	3964	3947	23	17.7	3970	**-0.7**	**5.3**
Lauson Dam	Existing	Existing & Proposed	50 / 24	3608	109	379.0	379.0	471.0	267.0	**3964.1**	3956	3964	3947	23	17.1	3970	**-0.1**	**5.9**
Lauson Dam	Existing	Existing & Proposed	10 / 24	1838	46	212.0	212.0	471.0	201.3	**3958.8**	3956	3964	3947	23	11.8	3970	**5.2**	**11.2**

C. Areas of High Risk

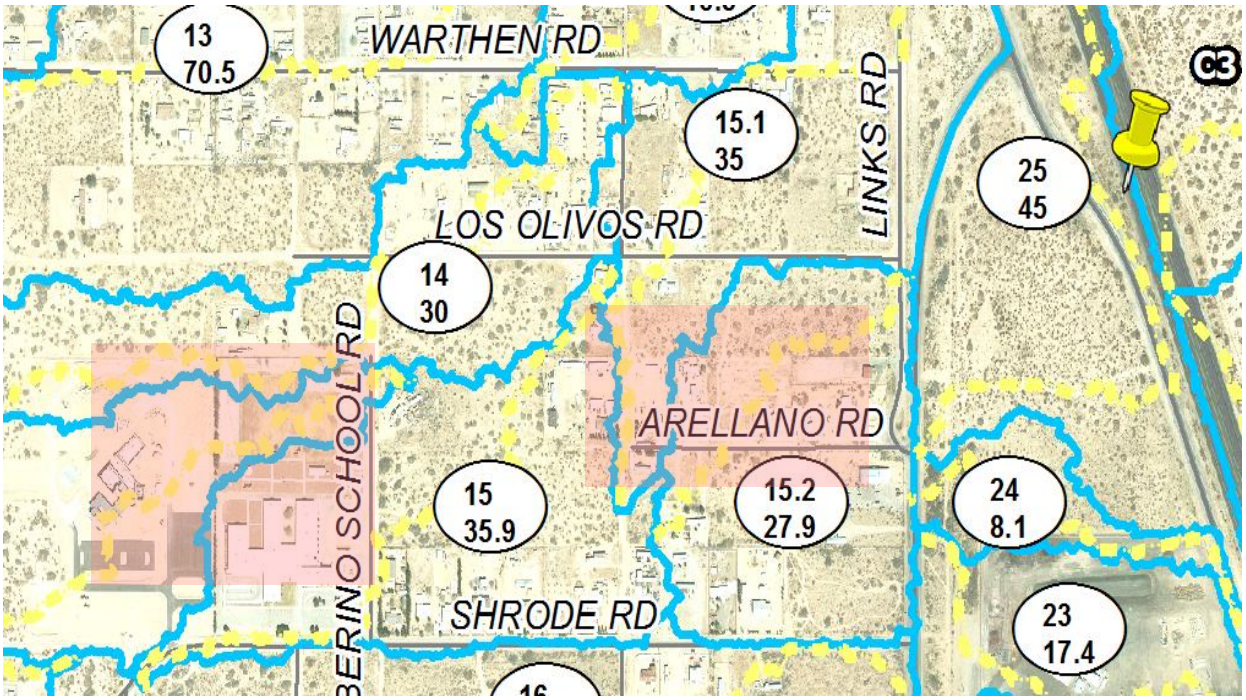
Culvert C1 has an uncontrolled tributary area of approximately 2 square miles. As such even the 10-year discharge and runoff volume are **456 cfs and 62 ac-ft**. respectively. Culvert C1, consisting of 6 – 4' X 6' CBCs, have a maximum discharge capacity of **1336 cfs** including a 15% clogging factor. As such, Culvert C1 will pass the entire peak discharge from the upstream subbasins during the 10-year storm. Inflow from Culvert C1 may cause damage to two storage lagoons. The contents from the storage lagoons may prove to be environmentally hazardous and further property damage may occur at Palm Tree Rd. The areas highlighted in **Figure 3.1** point out the properties mentioned above.

Figure 3.1



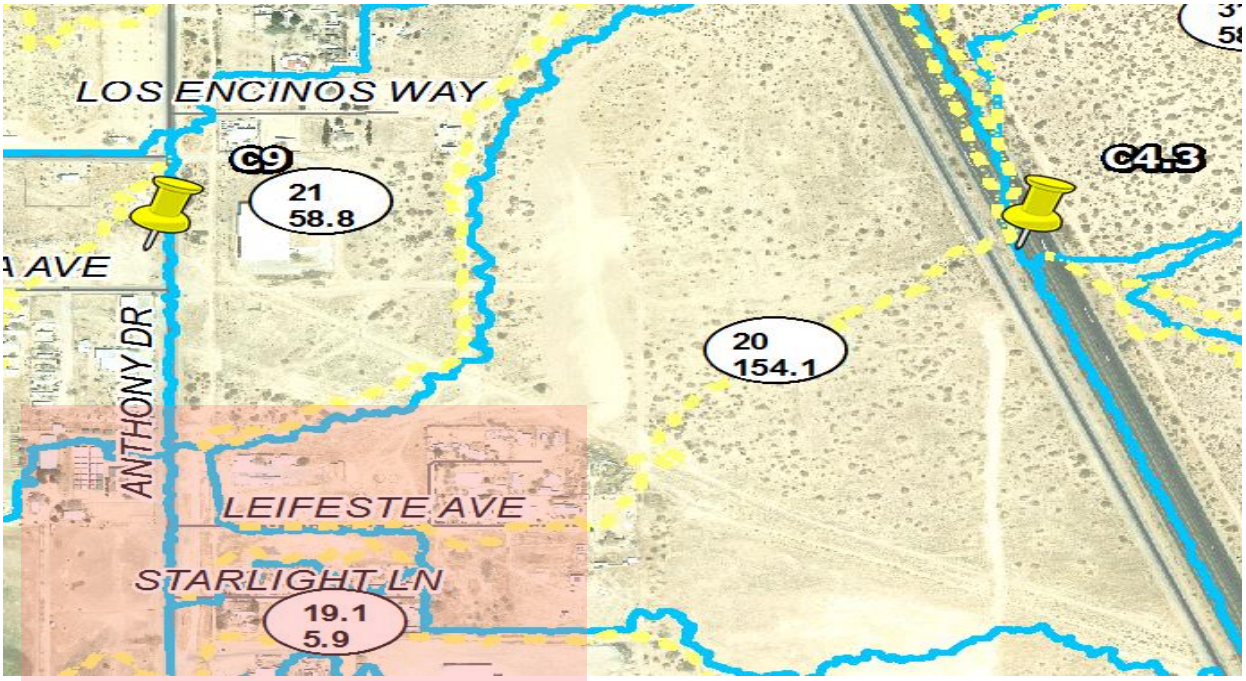
Inflows from Culvert C3 will spill over Links Rd. and eventually drain to Berino School Rd. as shown on **Figure 3.2.**

Figure 3.2



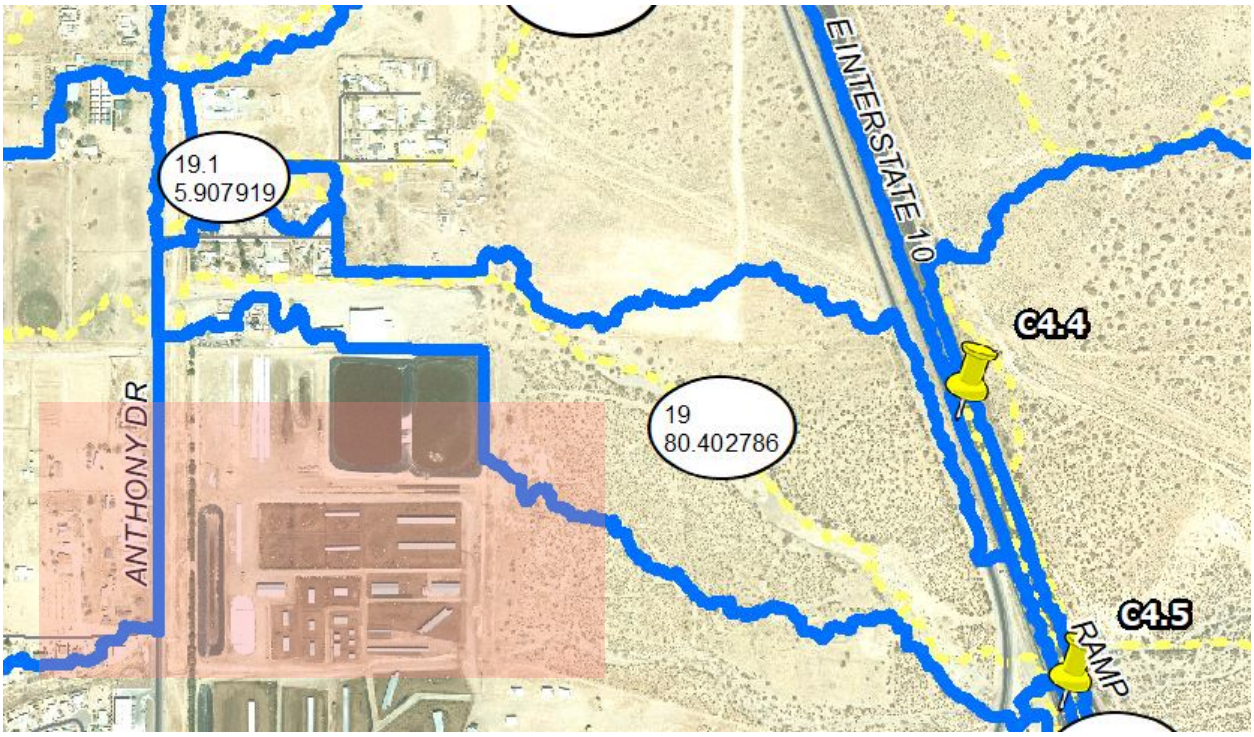
Inflows from Culvert C4.3 will affect properties along Anthony Dr. between Starlight Ln. and Montana Vista Ave. as shown on **Figure 3.3.**

Figure 3.3



The Lauson Dam spills in the 50 and 100 year storms and the outflow through Culvert C4.4 and C4.5 can adversely affect another pair of storage lagoons as shown on **Figure 3.4**.

Figure 3.4



SECTION 3. FLOODPLAIN MAPPING

Floodplain mapping was not part of the scope for this project however refer to **Appendix B** for digital copies of existing FEMA FIRM maps.

SECTION 4. PROPOSED OPTIONS HYDROLOGIC AND HYDRAULIC ANALYSES

4.1 PROPOSED OPTIONS HYDROLOGIC DATA

No modeling changes were made that would affect the two existing dams therefore the reservoir routing results remain unchanged from the existing conditions model. All other modeling parameters remained the same as the existing conditions model. The existing conditions HEC-HMS model was modified to simulate four proposed detention ponds. Conceptual level grading plans were developed for all four ponds. Based on these grading plans, stage-storage-discharge rating curves were developed and refined to simulate reservoir routings in HEC-HMS. **Appendix C** documents the data tables used for these rating curves. The proposed ponds were incorporated into the proposed model and differences in peak discharges were reevaluated. The final footprint of the proposed ponds was utilized to develop conceptual level of engineer's opinion of probable cost (EOPC) for land acquisition, pond construction and construction of appurtenances associated with the ponds.

4.2 CONCEPTUAL DESIGN OPTIONS

Based on the existing conditions results, four ponds are proposed to mitigate discharge for the 10-year storm for the subbasins labeled as high risk in **Figure 3 (page 9)**. **Figure 4A** provides an overview of the locations of the four ponds and the effect they have on peak discharge reduction for the design storm. **Figure 4A** also presents pertinent information about the facilities in terms of facility size and cost. In the following section, proposed ponds are categorized as facilities and prioritized in the order of most to least important based on direction provided by the DACFC. Facilities have been further categorized into two sub categories to help the DACFC with phasing of these projects and incorporating them into the capital improvements list. Since all four facilities are ponds, they are categorized into two sub facilities: land acquisition and pond construction. For cost estimating purposes, $2,500/acre was assumed for all facilities. All facility maps, **Figures 4 through 4.3** are presented within Section 4.3. Ponds were typically graded at a 1V:3H from the top of pond to the pond bottom to maximize volume yet minimizing the need for slope stabilization. All ponds were simulated to have a concrete reverse inclined ported riser structure for water quality. Principal outflow pipes were simulated to be a 36-inch CMP pipe. A larger diameter outfall pipe was selected to reduce chances of clogging from sediment and debris. The hydraulic calculations for the water quality structure are part of the stage-storage-discharge tables **(Tables C8-C11)** that are included in **Appendix C**. The emergency spillways were all sized to pass the 100 year -24 hour peak discharge. Emergency spillways would have to be made of reinforced concrete and were priced accordingly. All run-down structures will have to be wire enclosed riprap since the soil conditions in this area is cohesion less. Erosion control aprons at the outlet of all the ponds will be required.

Detention Pond Routing Summary Proposed Ponds

Facility ID (Land and Construction ID)	Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Total Approximate Cost
			inches	yr / hr	cfs	cfs	
Facility 4.1 & 4.2	Pond 1	Proposed	Proposed	10 / 24	458	273	$ 1,193,000
Facility 1.1 & 1.2	Pond 2	Proposed	Proposed	10 / 24	140	27	$ 719,000
Facility 2.1 & 2.2	Pond 3	Proposed	Proposed	10 / 24	199	28	$ 1,154,000
Facility 3.1 & 3.2	Pond 4	Proposed	Proposed	10 / 24	138	53	$ 1,351,000

POND 1
Ex. Q10 = 458 cfs
Prop. Q = 273 cfs

POND 2
Ex. Q10 = 140 cfs
Prop. Q = 27 cfs

POND 3
Ex. Q10 = 199 cfs
Prop. Q = 28 cfs

POND 4
Ex. Q10 = 138 cfs
Prop. Q = 53 cfs

Legend

- Culvert ID
- National Monuments
- Montana Vista Subbasins
- Dams
- EBID Facilities

131
219.8
SUBBASIN ID & SUBBASIN AREA (AC)

2014 DIGITAL ELEVATION MODELS, 2 FT. INTERVAL LIDAR AND ORTHOPHOTOGRAPHY WAS PROVIDED BY DONA ANA COUNTY FLOOD COMMISSION

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MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 4A

Overview of Options

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4.3 ANALYSES AND OPTIONS SUMMARY

Note: Summary in order of highest to lowest priority.

Facility 1: Pond 2

This facility was assigned the highest priority as it will mitigate flooding that would occur at Berino Elementary School as discussed in Section 2.15 and shown on **Figure 3.2**.

Facility 1.1: Land Acquisition (Cost - $19,000)

DACFC will be required to purchase approximately 7 acres of land currently owned by Bureau of Land Management (BLM) to construct a viable facility in this location. There may be potential for a land exchange as DACFC owns a 3-acre parcel just to the south of Pond 2. Smith evaluated the possibility of grading a pond on the parcel owned by DACFC however that option does not provide sufficient storage volume for the 10-year storm and would still require an easement from the BLM so that a diversion channel could be constructed to convey flows from the outlet of Culvert C 3.1 to the DACFC parcel. As such, the DACFC parcel was not considered.

Facility 1.2: Pond 2 Construction (Cost - $ 700,000)

Figure 4 on **page 16** shows the conceptual layout for Pond 2. It will be able to fully control the 10-year storm. Based on its embankment height of 6 feet and storage volume of 13 ac-ft., this pond will be non-jurisdictional. Reservoir routing results are presented below.

Table 3.1

Detention Pond Routing Summary - Proposed Pond 2

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 2	Proposed	Proposed	100 / 24	227	211	52.0	52.0	13.0	8.3	**3878.3**	3878.0	3874	6.0	4.3	3880.0	**-0.3**	**1.8**
Pond 2	Proposed	Proposed	50 / 24	193	173	40.0	40.0	13.0	8.1	**3878.2**	3878.00	3874	6.0	4.2	3880.0	**-0.2**	**1.8**
Pond 2	Proposed	Proposed	10 / 24	140	27	19.2	19.2	13.0	7.4	**3877.9**	3878.0	3874	6.0	3.9	3880.0	**0.1**	**2.1**

The total cost for Facility 1 is $719,000. Detailed construction items and costs are provided in Appendix F.

Detention Pond Routing Summary - Proposed Pond 2

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 2	Proposed	Proposed	100 / 24	227	211	52.0	52.0	13.0	8.3	**3878.3**	3878.0	3874	6.0	4.3	3880.0	-0.3	**1.8**
Pond 2	Proposed	Proposed	50 / 24	193	173	40.0	40.0	13.0	8.1	**3878.2**	3878.00	3874	6.0	4.2	3880.0	-0.2	**1.8**
Pond 2	Proposed	Proposed	10 / 24	140	27	19.2	19.2	13.0	7.4	**3877.9**	3878.0	3874	6.0	3.9	3880.0	**0.1**	**2.1**

Facility 1 Cost Summary		
Facility ID	Description	Approximate Cost
$ 1.1	Pond 2	$ 19,000
$ 1.2	Pond 2	$ 700,000
Total Cost		$ 719,000

EX CULVERT C 3.1 5' X 6' CBC PIPE

EX CULVERT C 3 - 5' X 6' CBC PIPE

FACILITY 1.1 LIMITS OF LAND ACQUISITION

RUNDOWN STRUCTURE

POND INVERT 3874'

FACILITY 1.2 POND 2

36" CMP PRINCIPAL SPILLWAY
INV IN = 3874.00'
INV OUT = 3872.00'

WATER QUALITY STRUCTURE

1:3 SLOPE TYP.

EMERGENCY SPILLWAY
SPILLWAY ELEV = 3878'
TOP OF EMBANKMENT = 3880'

1:5 SLOPE TYP.

EROSION CONTROL APRON

EXISTING PROPERTY LINE

0 50' 100'

NOTES:
1. CONCEPTUAL LEVEL DRAWING. NOT FOR CONSTRUCTION

NO.	REVISION DESCRIPTION	DATE	BY
1			
2			
3			
4			
5			

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 4

PROPOSED POND 2

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116119

JUNE, 2017

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Facility 2: Pond 3

This facility was assigned the second highest priority as it will mitigate flooding that would occur from Culvert C4.3 that will affect properties along Anthony Dr. between Starlight Ln. and Montana Vista Ave. as shown on **Figure 3.3** in Section 2.15.

Facility 2.1: Land Acquisition (Cost - $35,000)

DACFC will be required to purchase approximately 13 acres of privately owned land to construct a viable facility in this location. The limits of land acquisition are shown on **Figure 4.1**.

Facility 2.2: Pond 3 Construction (Cost - $ $1,119,000)

Figure 4.1 on page 18 shows the conceptual layout for Pond 3. It will be able to fully control the 10-year storm. Based on its embankment height of 6.5 feet and storage volume of 27.9 ac-ft., this pond will be non-jurisdictional. Reservoir routing results are presented below. Pond 3 will also require a rundown to channel the water from Culvert C4.3 into the pond. **Table 3.2** summarizes the reservoir routing for Pond 3.

The total cost for Facility 2 is $1,154,000.

Table 3.2

Detention Pond Routing Summary - Proposed Pond 3

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 3	Proposed	Proposed	100 / 24	451	209	92.0	92.0	27.9	27.0	**3862.4**	3862.0	3856	6.5	6.4	3862.5	**-0.4**	**0.1**
Pond 3	Proposed	Proposed	50 / 24	357	174	71.0	71.0	27.9	26.7	**3862.3**	3862.0	3856	6.5	6.3	3862.5	**-0.3**	**0.2**
Pond 3	Proposed	Proposed	10 / 24	199	28	35.0	35.0	27.9	14.9	**3860.0**	3862.0	3856	6.5	4.0	3862.5	**2.0**	**2.5**

Detention Pond Routing Summary - Proposed Pond 3

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	ft	ft	ft	ft	ft	ft	ft	ft
Pond 3	Proposed	Proposed	100 / 24	451	209	92.0	92.0	27.9	27.0	**3862.4**	3862.0	3856	6.5	6.4	3862.5	-0.4	0.1
Pond 3	Proposed	Proposed	50 / 24	357	174	71.0	71.0	27.9	26.7	**3862.3**	3862.0	3856	6.5	6.3	3862.5	-0.3	0.2
Pond 3	Proposed	Proposed	10 / 24	199	28	35.0	35.0	27.9	14.9	**3860.0**	3862.0	3856	6.5	4.0	3862.5	2.0	2.5

Facility 2 Cost Summary

Facility ID	Description	Approximate Cost
$ 2.1	Pond 3	$ 35,000
$ 2.2	Pond 3	$ 1,119,000
Total Cost		$ 1,154,000

FACILITY 2.1 LIMITS OF LAND ACQUISITION

RUNDOWN STRUCTURE

EX CULVERT C 4.3: 3 - 4' DIA. CMP PIPE

EXISTING PROPERTY LINE

POND INVERT 3856'

FACILITY 2.2 POND 3

WATER QUALITY STRUCTURE

1:6 SLOPE TYP.

1:3 SLOPE TYP.

EMERGENCY SPILLWAY
SPILLWAY ELEV = 3862'
TOP OF EMBANKMENT = 3862.5'

36" CMP
PRINCIPAL
SPILLWAY
INV IN = 3858.00'
INV OUT = 3856.00'

EROSION CONTROL APRON

0 100' 200'

NOTES:
1. CONCEPTUAL LEVEL DRAWING. NOT FOR CONSTRUCTION

NO.	REVISION DESCRIPTION	DATE	BY
1			
2			
3			
4			
5			

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 4.1

PROPOSED POND 3

SMITH ENGINEERING COMPANY
NEW MEXICO

116119

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Facility 3: Pond 4

This facility was assigned the third highest priority as it will mitigate flooding from the Lauson Dam that spills in the 50 and 100 year storms. The outflow through culvert C4.5 and C4.6 can adversely affect another pair of storage lagoons as shown on **Figure 3.4**. in section 2.15. Pond 4 will act as an off-channel structure that will require a concrete diversion weir built in the main arroyo which will divert the 10 year flows into the pond. The existing channels bottom width and slope were computed. The normal depth for the 10-year peak discharge of 138 cfs was computed to be 0.4 ft. using FlowMaster. Based on this data Smith determined that the diversion weirs crest elevation should be 0.5 ft. higher than the existing channel invert in order to divert the 10-year peak discharge into the pond. Any flows greater than the 10-year storm will overtop the diversion weir and continue downstream. For the purposes of this master plan, the entire inflow hydrographs for the 50 and 100 year storms were routed through the pond to be conservative. Further refinement of the model will be required if this project goes to final design.

Facility 3.1: Land Acquisition (Cost - $35,000)

DACFC will be required to purchase approximately 13 acres of privately owned land to construct a viable facility in this location. The limits of land acquisition are shown on **Figure 4.2**.

Facility 3.2: Pond 4 Construction (Cost - $ 1,316,000)

Figure 4.2 on page 20 shows the conceptual layout for Pond 4. It will be able to fully control the 10-year storm. Based on its embankment height of 7.5 feet and storage volume of 31.8 ac-ft., this pond will be non-jurisdictional. Reservoir routing results are presented below.

The total cost of Facility 3 is $1,351,000.

Table 3.3

Detention Pond Routing Summary - Proposed Pond 4

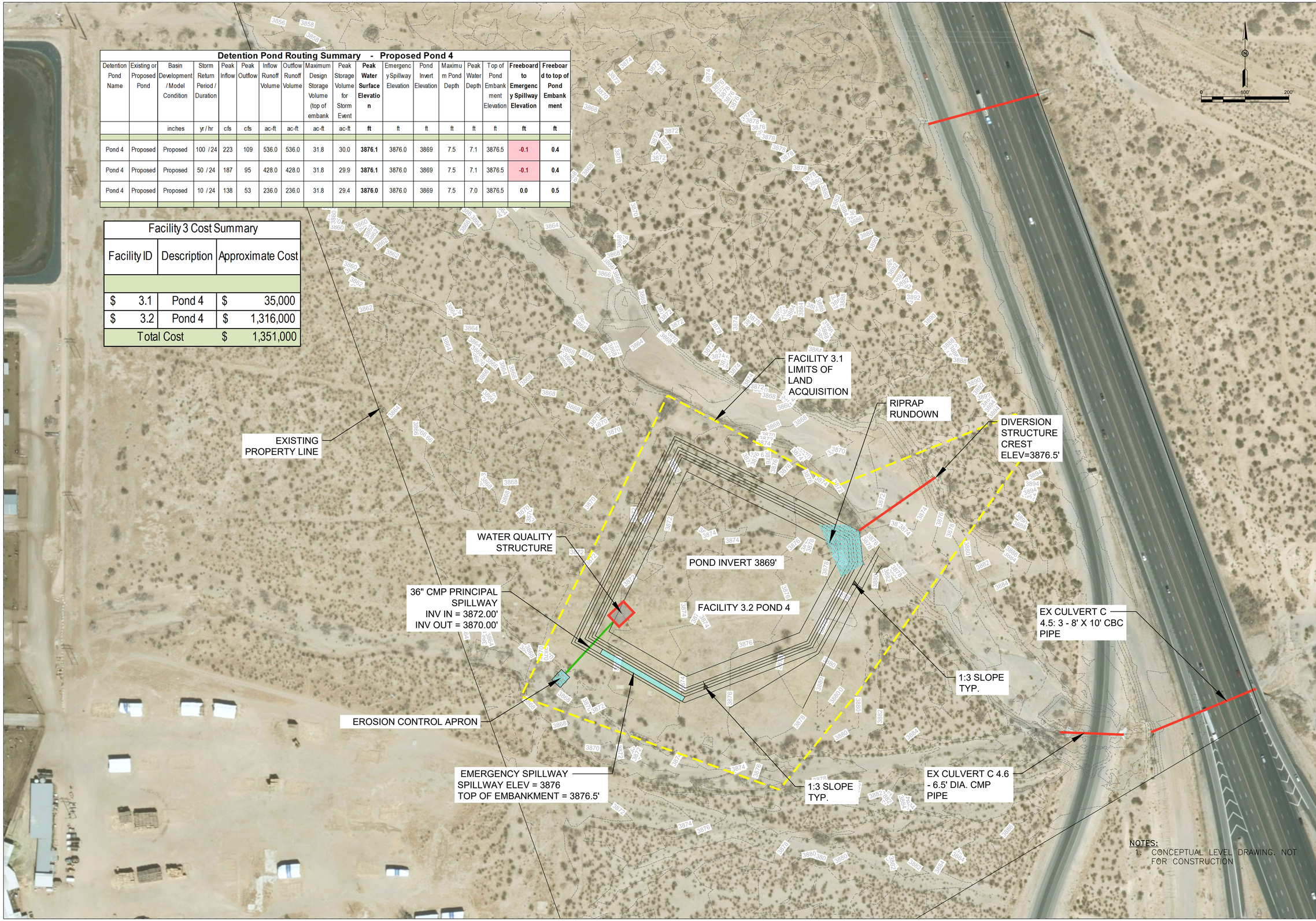
Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 4	Proposed	Proposed	100 / 24	223	109	536.0	536.0	31.8	30.0	**3876.1**	3876.0	3869	7.5	7.1	3876.5	**-0.1**	**0.4**
Pond 4	Proposed	Proposed	50 / 24	187	95	428.0	428.0	31.8	29.9	**3876.1**	3876.0	3869	7.5	7.1	3876.5	**-0.1**	**0.4**
Pond 4	Proposed	Proposed	10 / 24	138	53	236.0	236.0	31.8	29.4	**3876.0**	3876.0	3869	7.5	7.0	3876.5	**0.0**	**0.5**

Detention Pond Routing Summary - Proposed Pond 4

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 4	Proposed	Proposed	100 / 24	223	109	536.0	536.0	31.8	30.0	**3876.1**	3876.0	3869	7.5	7.1	3876.5	**-0.1**	**0.4**
Pond 4	Proposed	Proposed	50 / 24	187	95	428.0	428.0	31.8	29.9	**3876.1**	3876.0	3869	7.5	7.1	3876.5	**-0.1**	**0.4**
Pond 4	Proposed	Proposed	10 / 24	138	53	236.0	236.0	31.8	29.4	**3876.0**	3876.0	3869	7.5	7.0	3876.5	**0.0**	**0.5**

Facility 3 Cost Summary

Facility ID	Description	Approximate Cost
$ 3.1	Pond 4	$ 35,000
$ 3.2	Pond 4	$ 1,316,000
Total Cost		$ 1,351,000



NOTES:
1. CONCEPTUAL LEVEL DRAWING. NOT FOR CONSTRUCTION

NO.	REVISION DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 4.2

PROPOSED POND 4

SMITH ENGINEERING COMPANY
NEW MEXICO

116119

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Facility 4: Pond 1

Inflows from culvert C1 and C2 may cause damage to two storage lagoons. The contents from the storage lagoons may prove to be environmentally hazardous and further property damage may occur at Palm Tree Rd. The areas highlighted in **Figure 3.1** point out the properties as discussed in Section 2.15. Pond 1 will be an inline structure that will attenuate the peak discharge from culvert C1 and C2. However, the inflow peak discharges and volumes for the 3 return period storms are extremely large due to the lack of upstream improvements. Upstream improvements are not possible because most of the area upstream (east) of I-10 is part of the BLM National Monument boundary. As such any kind of mitigation even for the 10 year storm will require a jurisdictional facility. For example, just the 10-year inflow volume is 62 ac-ft. **Figure 4A** shows the limits of the National Monument areas.

Facility 4.1: Land Acquisition (Cost - $32,000)

DACFC will be required to purchase approximately 12 acres of privately owned land to construct a viable facility in this location. The limits of land acquisition are shown on **Figure 4.3**.

Facility 4.2: Pond 1 Construction (Cost - $ 1,161,000)

Figure 4.3 shows the conceptual layout for Pond 1. Based on its embankment height of 6 feet and storage volume of 26 ac-ft., this pond will be non-jurisdictional. Due to the large and uncontrolled upstream drainage area, the 10 year flows discharge through the emergency spillway as shown in the reservoir routing summary table. The pond footprint would have to be increased significantly in order to fully control the 10-year discharge. However, this may cause the pond to become jurisdictional in size and greatly increase the cost. Due to its low priority in the list of facilities, further grading and routing efforts were not considered. Reservoir routings for the pond are provided below.

The total cost of Facility 4 is $1,193,000.

Table 3.4 summarizes the reservoir routing results for Pond 1.

Table 3.4

Detention Pond Routing Summary - Proposed Pond 1

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 1	Proposed	Proposed	100 / 24	1202	965	111.0	111.0	26.1	20.8	**3887.0**	3886.0	3882	6.0	5.0	3888.0	**-1.0**	**1.0**
Pond 1	Proposed	Proposed	50 / 24	1061	893	126.0	126.0	26.1	20.4	**3886.9**	3886.0	3882	6.0	4.9	3888.0	**-0.9**	**1.1**
Pond 1	Proposed	Proposed	10 / 24	458	273	62.0	62.0	26.1	16.7	**3886.3**	3886.0	3882	6.0	4.3	3888.0	**-0.3**	**1.7**

Detention Pond Routing Summary - Proposed Pond 1

Detention Pond Name	Existing or Proposed Pond	Basin Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embank	Peak Storage Volume for Storm Event	**Peak Water Surface Elevation**	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	**Freeboard to Emergency Spillway Elevation**	**Freeboard to top of Pond Embankment**
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft**	**ft**
Pond 1	Proposed	Proposed	100 / 24	1202	965	111.0	111.0	26.1	20.8	**3887.0**	3886.0	3882	6.0	5.0	3888.0	**-1.0**	**1.0**
Pond 1	Proposed	Proposed	50 / 24	1061	893	126.0	126.0	26.1	20.4	**3886.9**	3886.0	3882	6.0	4.9	3888.0	**-0.9**	**1.1**
Pond 1	Proposed	Proposed	10 / 24	458	273	62.0	62.0	26.1	16.7	**3886.3**	3886.0	3882	6.0	4.3	3888.0	**-0.3**	**1.7**

Facility 4 Cost Summary

Facility ID	Description	Approximate Cost
4.1	Pond 1	$ 32,000
4.2	Pond 1	$ 1,161,000
Total Cost		$ 1,193,000

FACILITY 4.1 LIMITS OF LAND ACQUISITION

RIPRAP RUNDOWN

EXISTING: 6 - 4' X 6' CBC PIPE

POND INVERT 3882'

FACILITY 4.2 POND 1

EMERGENCY SPILLWAY ELEV = 3886' TOP OF EMBANKMENT = 3888'

WATER QUALITY STRUCTURE

EROSION CONTROL APRON

36" CMP PRINCIPAL SPILLWAY INV IN = 3882.00' INV OUT = 3880.00'

1:6 SLOPE TYP.

1:3 SLOPE TYP.

EXISTING PROPERTY LINE

0 60' 120'

NOTES:
1. CONCEPTUAL LEVEL DRAWING. NOT FOR CONSTRUCTION

NO.	REVISION DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

MONTANA VISTA DRAINAGE MASTER PLAN

FIGURE 4.3

PROPOSED POND 1

SMITH ENGINEERING COMPANY NEW MEXICO

116119

JUNE, 2017

SECTION 5. CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSIONS AND RECOMMENDATIONS

Most of the subbasin discharges for the areas west of I-10 didn't display significant problems for the design storm largely due to rural, pervious nature of the watershed. Therefore, no improvements were proposed for the residential areas west of I-10. The culvert discharges, as discussed in Section 4 for the design storm were of concern and therefore the four facilities discussed in Section 4 are recommended. These facilities are summarized here in order of descending order along with the approximate cost for land acquisition and construction in the table below.

Facility ID	Pond Name	Total Cost
Facility 1	Pond 2	$719,000
Facility 2	Pond 3	$1,154,000
Facility 3	Pond 4	$1,351,000
Facility 4	Pond 1	$1,193,000