UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

DOÑA ANA COUNTY FLOOD COMMISSION

VOLUME 1 OF 2

Smith Engineering Company
2201 San Pedro Drive, NW
Building 4, Suite 200
Albuquerque, NM 87110
www.smithengineering.pro

March 2020

Smith Project No.: 817103-04

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FINAL

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.

KELLI BERMAN
NEW MEXICO
25766
PROFESSIONAL ENGINEER

3-19-2020

Kelli Berman, PE

March 19, 2020

Michael Garza, EIT, CFM
Doña Ana County Flood Commission
845 N. Motel Blvd.
Las Cruces, New Mexico 88007

Re: Updated Doña Ana Area Drainage Master Plan

Smith #: 817103-04

Dear Mr. Garza:

I am pleased to submit the Updated Doña Ana Area Drainage Master Plan for the Doña Ana County Flood Commission (DACFC). The purpose of this report was to update the current DACFC HECHMS model using Type II CNs and incorporating additional ponds and storm conveyance systems. We have refined sub-basins, hydrologic models and alternatives to evaluate further drainage improvement projects for the County's consideration. Five options were modeled and are being presented in this report. Each option includes various facilities that will allow the County to improve their drainage network in small increments while steadily working on the entire system. Each facility was prioritized and placed in a phasing schedule. These recommendations were further refined based on feedback from the 90% review meeting. Also included in this submittal is the Engineer's Opinion of Probable Cost (EOPC) for all facilities and options presented.

All comments from the 90% review were incorporated/addressed in this submittal. Thank you for allowing us to work on this project.

Please feel free to contact us at any time with questions. We look forward to your comments.

Sincerely,
Smith Engineering Company

Kelli Berman, PE
Project Manager
Enclosure: Updated Doña Ana Area Drainage Master Plan final

CC: John Gwynne, PE, CFM
Andrew Guerra, PE, CFM,

ACKNOWLEDGMENTS

DACFC for providing necessary digital files to perform the drainage study and local insight into the watershed.

The Community of the Doña Ana Area for invaluable historical accounts of flooding and input regarding areas of concern.

EXECUTIVE SUMMARY

In 2017, Smith Engineering completed the Doña Ana Drainage Master Plan (2017 DMP). A location of this watershed is shown in **Exhibit 1**. The Doña Ana Flood Commission directed Smith to update the original DMP with changes to curve numbers from ARC III to ARC II. The changes in curve numbers preceded to updating the HEC-RAS 2D and HEC-HMS model. Which led to proposing further drainage improvements east of Interstate-25 (I-25).

The modeled storm events include 5-yr, 10-yr, 50-yr and 100-yr 24-hr for existing and proposed conditions in HEC-HMS. The curve numbers and time of concentrations did not change from existing to proposed conditions. **Exhibit 2** shows the subbasin boundaries for the entire watershed that drains from east to west through existing culverts that pass under I-25.

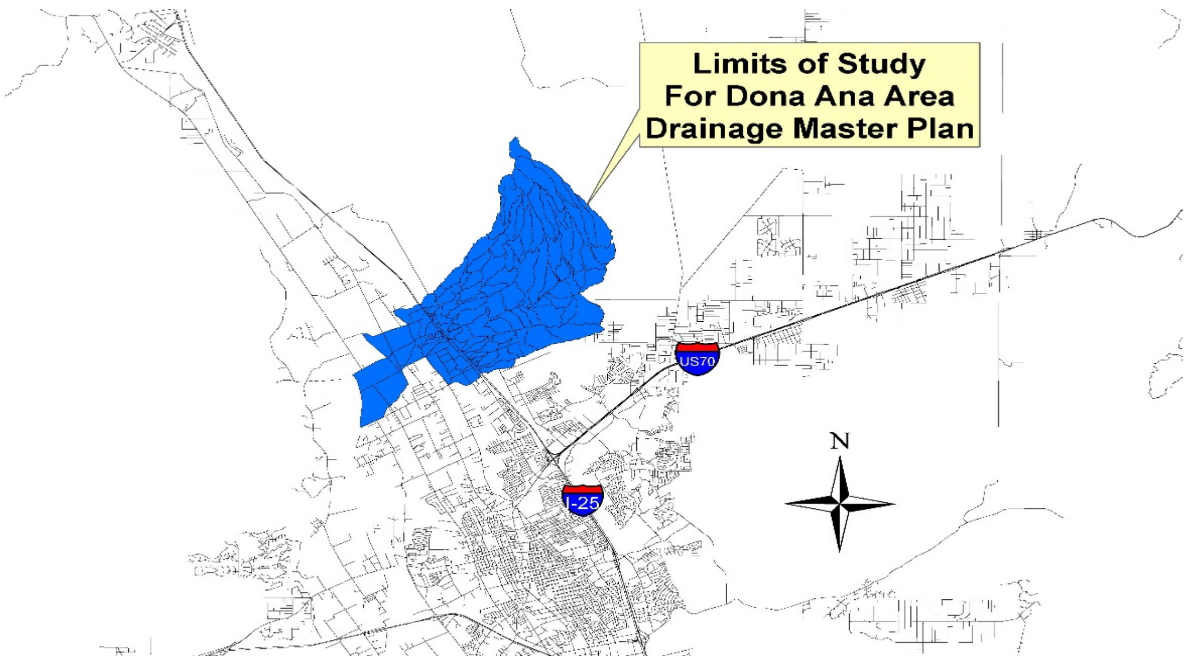
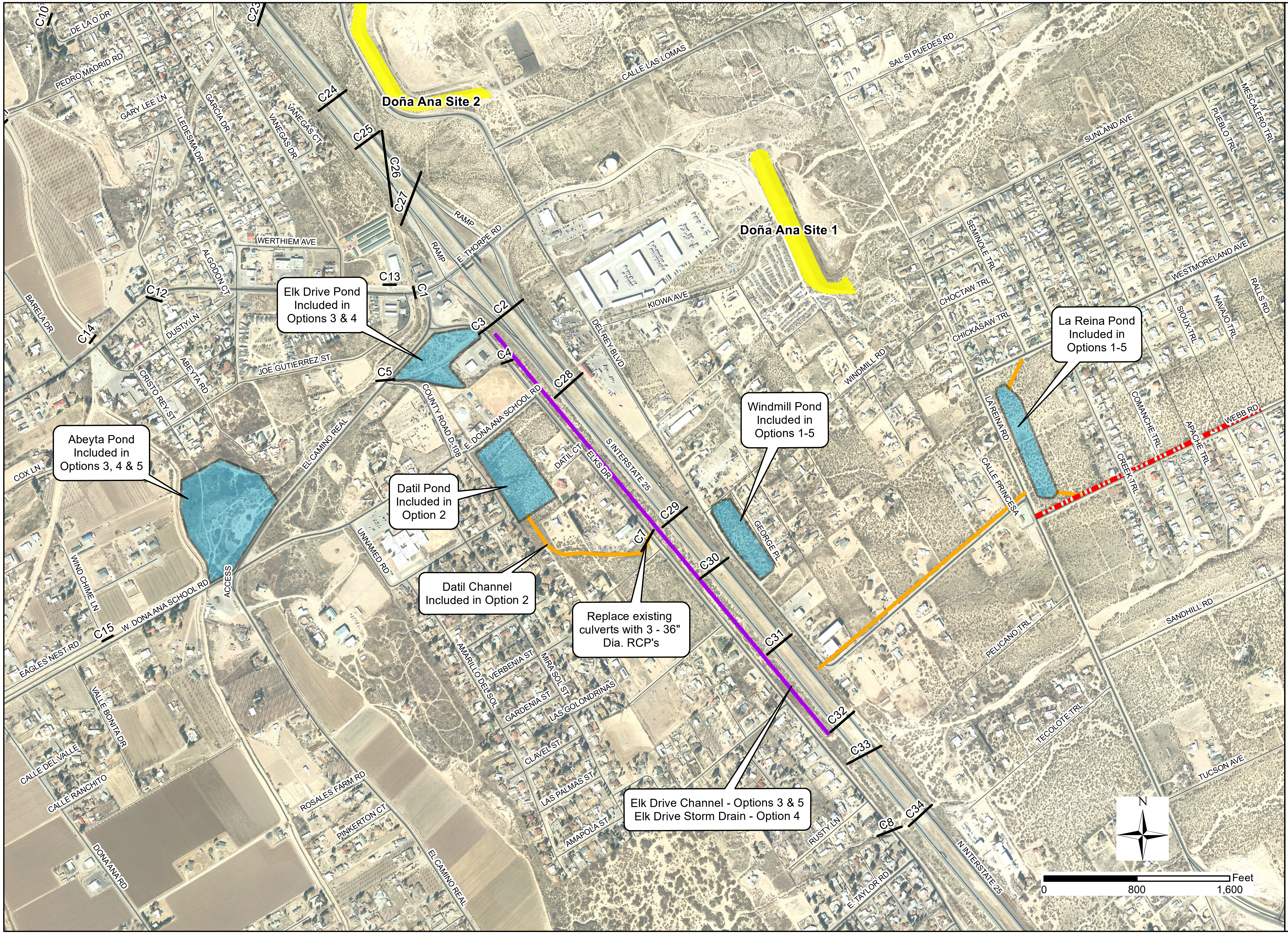


Exhibit 1: Overall Vicinity Map

The proposed improvements in this report are broken into 7 Facilities which include detention ponds, earth lined channels, a storm drain system and improvements to existing culverts. Each facility will improve the drainage on their own and will work as a drainage network with the other facilities. To show the impact of the drainage improvements as a network 5 options were model in HEC-HMS. Figure 18: Overview of Proposed Options (shown below and included in the body of the report) shows the flow path for these options. The tables on the following pages summarize the cost for each option and the cost for the corresponding facilities.



Legend

- Proposed Ponds
- Existing Dams
- Proposed Elk Channel and Storm Drain
- Proposed Webb Road Curb and Gutter
- Proposed Channels
- Existing Culverts
- Roads

Updated Dona Ana Area Drainage Master Plan

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Figure 18
Overview of Proposed Options

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Summary of Cost For All Options Updated Dona Ana County Drainage Master Plan			
Option No.	**Facility No.**	**Description**	**Cost**
Option 1	1	La Reina Pond, Proposed Rundown, Earth Lined channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
	2	Windmill Pond	$ 455,000
		Total Cost For Option 1	**$ 1,638,000**
Option 2	1	La Reina Pond, Proposed Rundown, Earth Lined Channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
	2	Windmill Pond	$ 455,000
	3	Datil Pond, Datil Channel, Remove/Replace Existing Private Culverts, Channel from Culvert C32 to C7, Replacement of Culvert C7, Rebuild Existing Wall and Maintain Existing Channel	$ 1,621,000
		Total Cost For Option 2	**$ 3,259,000**
Option 3	1	La Reina Pond, Proposed Rundown, Earth Lined channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
	2	Windmill Pond	$ 455,000
	4 (a)	Elk Drive Channel, and Channel From Culvert C28 to C3	$ 243,000
	5	Elk Drive Pond	$ 1,187,000
	7	Abeyta Pond	$ 1,321,000
		Total Cost For Option 3	**$ 4,389,000**
Option 4	1	La Reina Pond, Proposed Rundown, Earth Lined channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
	2	Windmill Pond	$ 455,000
	4 (b)	Elk Drive Storm Drain, and Channel From Culvert C28 to C3	$ 1,178,000
	5	Elk Drive Pond	$ 1,187,000
	7	Abeyta Pond	$ 1,321,000
		Total Cost For Option 4	**$ 5,324,000**
Option 5	1	La Reina Pond, Proposed Rundown, Earth Lined channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
	2	Windmill Pond	$ 455,000
	4 (a)	Elk Drive Channel, and Channel From Culvert C28 to C4	$ 243,000
	6	El Camino Real Culvert Replacement	$ 191,000
	7	Abeyta Pond	$ 1,321,000
		Total Cost For Option 5	**$ 3,393,000**

Each facility was prioritized based on discussion and after analyzing the drainage impacts and costs. The table below lists the facility and corresponding phase and priority level. This table will be further refined after the 90% review.

Facility No.	Priority	Phase	Description	Cost
Facility 1	5	3	La Reina Pond, Proposed Rundown, Earth Lined Channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
Facility 2	4	3	Windmill Pond	$ 455,000
Facility 3	6	4	Datil Pond, Datil Channel, Remove/Replace Existing Private Culverts, Channel from Culvert C32 to C7, Replacement of Culvert C7, Rebuild Existing Wall and Maintain Existing Channel	$ 1,621,000
Facility 4 (a)	1	1	Elk Drive Channel, and Channel From Culvert C28 to C3	$ 243,000
Facility 4 (b)	1	1	Elk Drive Storm Drain, and Channel From Culvert C28 to C3	$ 1,178,000
Facility 5	2	2	Elk Drive Pond	$ 1,187,000
Facility 6	2	2	El Camino Real Culvert Replacement	$ 191,000
Facility 7	3	2	Abeyta Pond	$ 1,321,000
Total Cost of Phased Capital Improvement Projects with Facility 4.2.a				**$ 6,201,000**
Total Cost of Phased Capital Improvement Projects with Facility 4.2.b				**$ 7,136,000**

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SECTION 1. GENERAL PROJECT INFORMATION

1.1 DESCRIPTION AND PURPOSE OF PROJECT

In 2017, Smith Engineering completed the Doña Ana Drainage Master Plan (2017 DMP). The purpose of this project is to update the drainage plan to include revisions to the CNs (From ARC III to ARC II) and to incorporate additional drainage improvements east of I-25. **Figure 1** presents the Doña Ana Vicinity Map for this project.

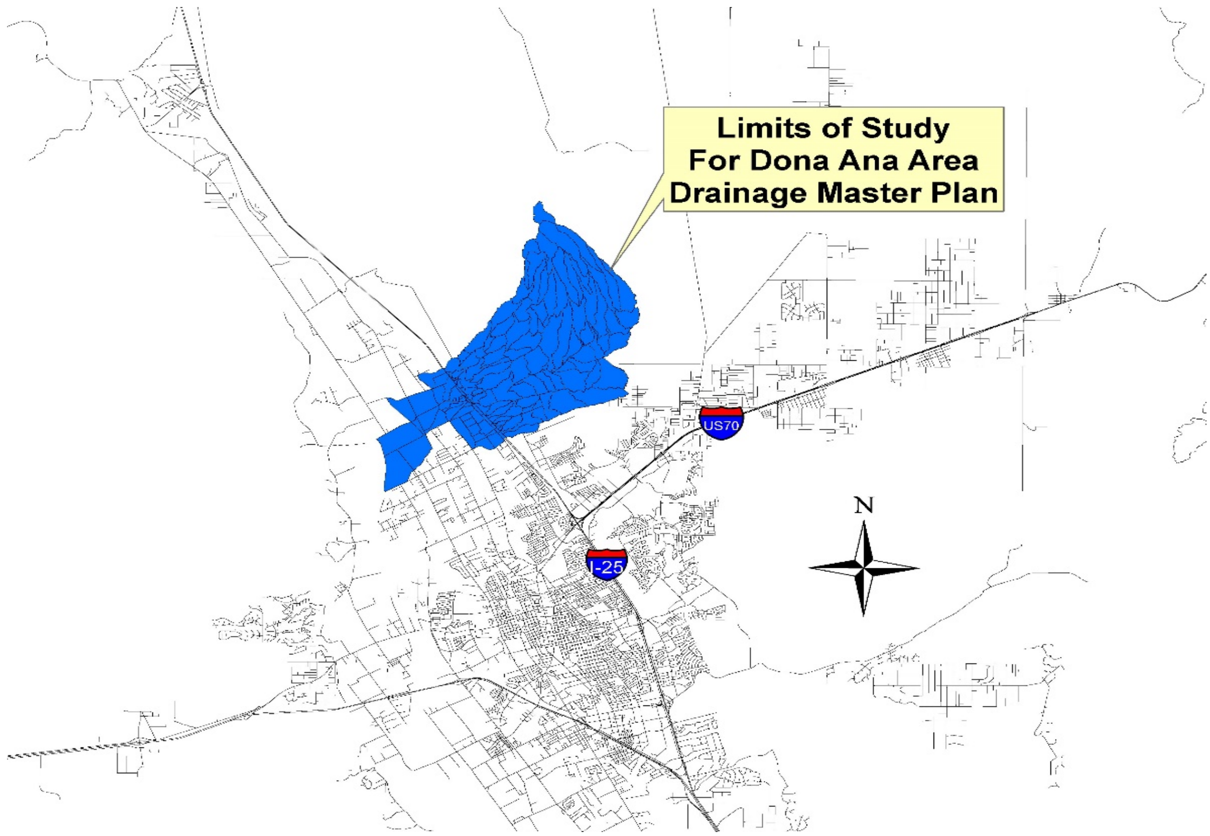


Figure 1: Project Vicinity Map

1.2 FIELD OBSERVATION

Smith conducted two field observations for this project in July and August 2019. **Appendix A** contains annotated photographs from the 2019 and 2016 field visits. These photos are taken at various locations in the Doña Ana area and show existing drainage infrastructure.

SECTION 2. EXISTING HYDROLOGIC AND HYDRAULIC ANALYSES

2.1 PREVIOUS STUDIES

Doña Ana Area Drainage Master Plan, completed in March 2017, prepared by Smith Engineering Company.

In 2017 Smith Engineering completed and submitted the Original Doña Ana Area Drainage Master Plan. The Original DMP developed analyses required to support the development of drainage improvement facilities for the community of Doña Ana and surrounding areas. The plan contains recommendations and conceptual level Engineers Opinion of Probable Costs. The analyses were conducted for the 5, 10, 50, and 100-yr 24-hr storm events. HEC-RAS 2D models were completed to simulate inundation limits for overland flow. These models were then used to develop drainage improvement.

The Original DMP is the basis of the 2020 Update to the Doña Ana Area Drainage Master Plan.

Drainage Report for NM 320 (Thorpe Road from I-25 to EBID Ditch, Control No. CN C1150921, completed in September 6, 2016, Occam Engineers Inc.

The Drainage Report for NM 320 was general in nature, with limits described from Thorpe Road from I-25 to the Elephant Butte Irrigation District (EBID) Ditch. The report provides recommendations for drainage improvements to be included as part of a proposed roadway improvement project for NM 320 in Doña Ana County. The project will extend from I-25 to the EBID ditch.

The report proposes roadway sections that include roadside ditches designed to convey runoff generated within the right-of-way. In addition to the runoff generated within the right-of-way, runoff entering the right-of-way from offsite basins were also evaluated. Through review of the existing topography, the report found that runoff from offsite drainage basins on the north and south sides of NM 320 (Thorpe Road) will reach the NMDOT right-of-way.

The runoff generated in the offsite drainage basins east of Ledesma Road was determined to be negligible. The runoff entering the right-of-way east of Ledesma Road, however, was determined to be beyond the capacity of roadside ditches (10-yr storm peak runoff rate of 477 cfs). Per the EBID, 50 cfs may be discharged into the existing EBID ditch. The report states that limiting the peak runoff rate to 50 cfs will require 5.5 acre-feet of storage within the offsite drainage basins west of Ledesma Road.

Master Drainage Study for Vista Rancho (Phase I) Subdivision. November 8 – Revision 3, complete November 8, 2016, Prepared by David B. Church.

"This is a proposed residential subdivision located just north of Taylor Rd. and 1200 feet west of Elks Rd. The off-site flows will be conveyed through the site and also detained. On-Site flows will be detained a number of on-site ponding locations.

This project will provide a net reduction of storm runoff flows from the site as a result of the total volume of storm water storage areas provided as part of the project.

The off-site flows will be conveyed through the site and also detained. On-Site flows will be detained a number of on-site ponding locations.

This project will provide a net reduction of storm runoff flows from the site as a result of the total volume of storm water storage areas provided as part of the project". Prepared by David B. Church, PE, 11-8-2016.

2.2 EXISTING FLOOD CONTROL STRUCTURES

Flood Control Dams

The Doña Ana Area Basin contains four dams as shown on **Figure 2.** The US Soil Conservation Service designed and built the North and South Doña Ana Dams in cooperation with the Elephant Butte Irrigation District (EBID). The Soil Conservation Service designed and built Alvillar Dams 1B and 1C in 1975 and the DACFC took over ownership in 1989. **Appendix B** contains the Construction Plans for the North and South Dams and the most recent topographic survey of Alvillar Dams 1B and 1C. The dam names and basic data are presented in the following table.

Table 1: Summary of Existing Dams

SUMMARY OF FOUR EXISTING DAMS							
Dam Name / Owner	Year Built	Drainage Area	Pond Depth to Top of Dam	Maximum Storage Volume to Top of Dam	Principal Spillway Pipe Diameter	Emergency Spillway Length	Is this a Jurisdictional dam?
a		sq mi	ft	ac-ft	inches	ft	Yes/No
Dona Ana North Dam (Site 2) / EBID	1960	2.155	29	507	24	300	Yes
Dona Ana South Dam (Site 1) / EBID	1960	6.3598	29.5	761	24	300	Yes
Alvillar Dam 1B / DACFC	1975	0.1557	10	15.8	18	22	No
Alvillar Dam 1C / DACFC	1975	0.9469	16	50.9	18	46	Yes/No[b]

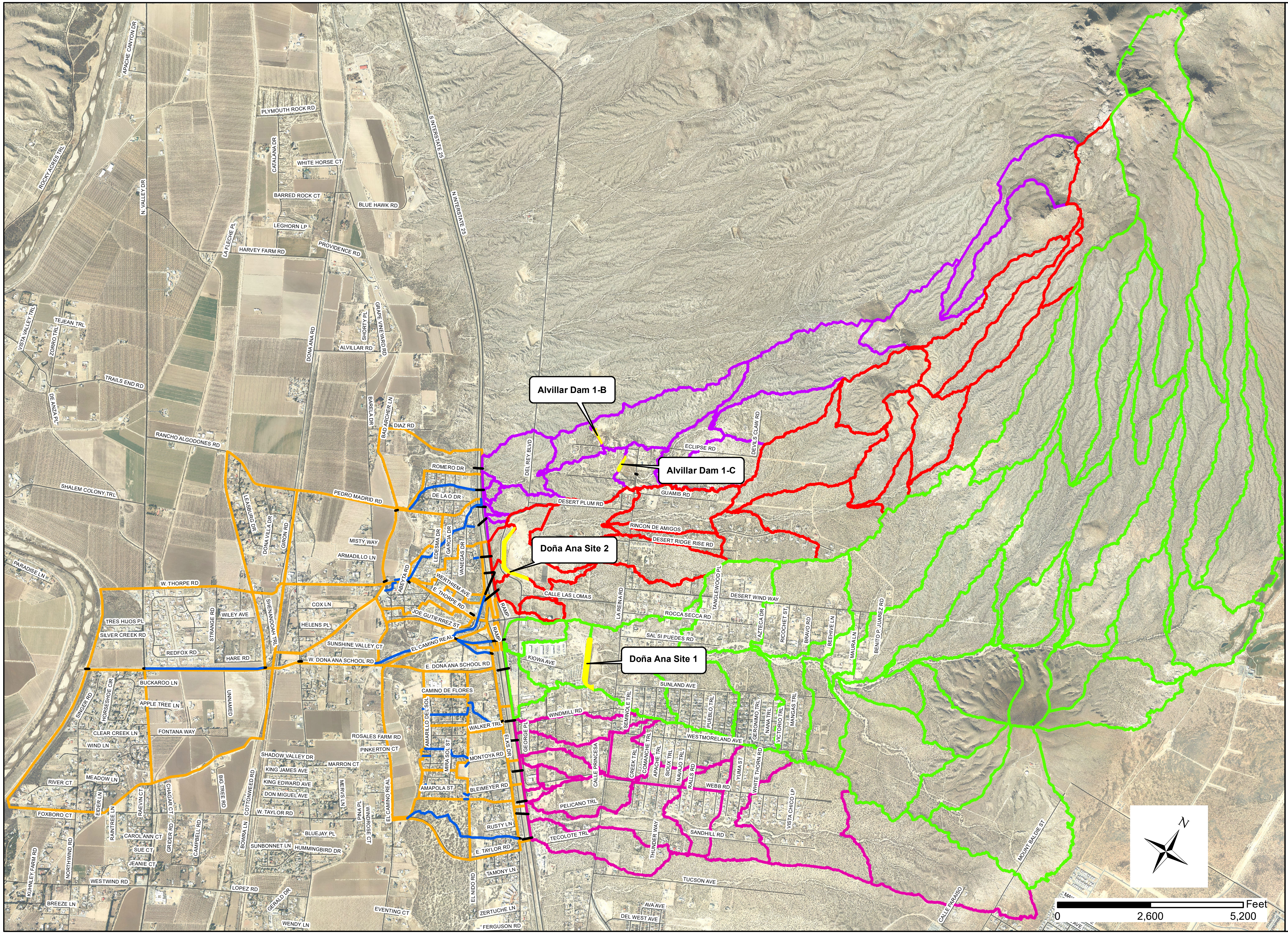
a - EBID - Elephant Butte Irrigation District

a - DACFC - Dona Ana County Flood Commission

b - The Alvillar Dam 1C is on the limits of being considered a jurisdictional dam. With a pond depth of 16-ft the dam would need to be at or under 50 ac-ft to maintain a non-jurisdictional title.

The North and South Dams are considered “high hazard jurisdictional dams” as defined by the current criteria and regulations as specified by the NM State Engineers Dam Safety Bureau (NMOSE DSB, December 31, 2010). Due to various deficiencies such as emergency spillway design and capacity (originally designed to pass only the 50-yr flood through the emergency spillways) these dams would be non-viable dams (NMOSE DSB). These dams would therefore require major upgrades to satisfy the NMOSE DSB such as passing the Probable Maximum Flood through the emergency spillway without dam failure.

Alvillar Dam 1B is non-jurisdictional and Alvillar 1C is at the limit of the NMSOE DSB criteria based on its dam height and storage volume that are 16 ft tall and about 51 ac-ft. At this dam height the pond volume needs to be limited to 50 ac-ft to remain non-jurisdictional.



Legend

- 100 Subbasin Boundaries
- 200 Subbasin Boundaries
- 300 Subbasin Boundaries
- 400 Subbasin Boundaries
- West of I-25 Subbasin Boundaries
- Existing Dams
- Existing Culverts
- Existing Channels
- Roads

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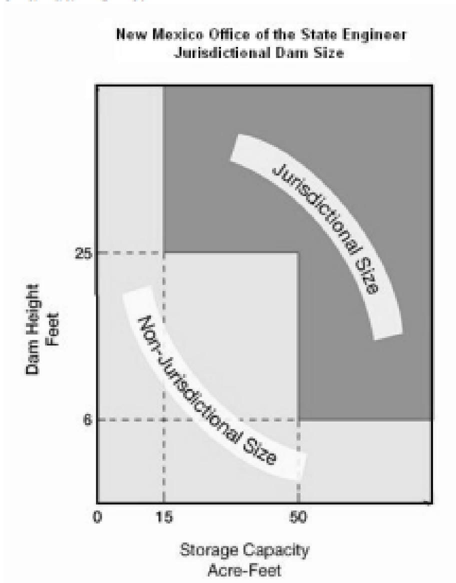
Figure 2 Overview of Existing Conditions Drainage Basin Map

2020

Below is a copy of the NMOSE definitions and graph that illustrates the basic dam height and storage volume requirements that define a jurisdictional or non-jurisdictional dam.

(1) Dam: A man-made barrier constructed across a watercourse or off-channel for the purpose of storage, control or diversion of water.

(a) Jurisdictional dam: A dam 25 feet or greater in height, which impounds more than 15 acre-feet of water or a dam that impounds 50 acre-feet or more of water and is 6 feet or greater in height. For purposes of these regulations, reference to a dam means a jurisdictional dam unless otherwise noted. See figure of jurisdictional dam size.



(b) Non-jurisdictional dam: Any dam not meeting the height and storage requirements of a jurisdictional dam. The state engineer does not regulate the design, construction and operation of a non-jurisdictional dam unless the dam is unsafe and there is a threat to life or property, as determined by the state engineer. Waters impounded by a non-jurisdictional dam may not be exempt from water right permit requirements; therefore a separate state engineer water right permit for the water impounded in the reservoir created by a non-jurisdictional dam may be required. Non-jurisdictional dams shall meet the requirements of 19.26.2.15 NMAC unless otherwise exempt. The structures listed below are considered non-jurisdictional dams:

Other Significant Structures

Please refer to **Figure 12**, which identifies the locations of other significant structures which include the following:

1. Channel 4 – has two reinforced concrete grade control or “drop structures”
2. Channel 5 – has four reinforced concrete grade control structures
3. Channels 1, 2, 9, 10 and 11 convey flows through residential subdivisions.
4. Wasteway No. 5 (Channel 6) is the outfall channel for the North and South Doña Ana Dams and other local sub-basins.
5. I-25 has many culverts that convey flows east to west under I-25.
6. **Appendix A** contains annotated photographs of some of the structures.

2.3 DRAINAGE BASIN DESCRIPTION, DELINEATION, AND MODELING CRITERIA

Drainage Basin Description

Refer to **Figure 3** Drainage Basin Maps (map pocket). The total basin area is about 15 square miles and drains from east to west. The sub-basin group located east of I-25 where the land use is primarily undeveloped range land with mild to steep topography. However, the southern portion of this sub-basin does contain residential development and a commercial development. The sub-basins located west of I-25 consist of residential, commercial and agricultural land use.

FEMA Floodplains

FEMA has developed Flood Insurance Rate Maps (FIRMs) for the Doña Ana area and these are dated July 6, 2016 (a copy of these are included in **Appendix B)**. Note that only Approximate A Flood zones have been delineated and the maps are at a small scale. The "A" Zone on Panel 0900 does not appear to reflect Wasteway No. 5 being used as an outlet for the Doña Ana North and South Dams. Further analysis would be required to determine the detailed extents of the floodplain beyond the El Camino Real/ Doña Ana School Road intersection.

Drainage Basin Delineation

Figure 3 (map pocket) present the drainage basin and sub-basin delineations. The orthophotography date is 2018 and date of the Lidar one-foot contours development is 2018.

The sub-basin numbering scheme was assigned as listed here:

<u>Sub-basins numbered 1 through 45 (West of I-25 Sub-basin Series)</u>

These are located west of I-25 and are uncontrolled meaning they do not outfall into any of the four dams. This subbasin series is noted in orange on **Figure 2**. Some of these sub-basins are in agricultural fields that are depressed below adjacent irrigation laterals or canals. Due to the terrain, these sub-basins capture flow, preventing it from leaving the site. These sub-basins are considered closed sub-basins and are noted with the letter C after the sub-basin number. These closed sub-basins were simulated but the hydrographs are not added to any other sub-basin.

<u>Sub-basins numbered in the 100 series</u>

These sub-basins are shown in green on **Figure 2**. These subbasins are located on the east side of I-25 and outfall into Doña Ana South Dam (Site 1). Eventually the flows in this subbasin series are conveyed underneath I-25 thru Culverts C2 and C28 into the West of I-25 Sub-basins series.

<u>Sub-basins numbered in the 200 series</u>

These sub-basins are shown in red on **Figure 2**. These subbasins are located on the east side of I-25 and outfall into Doña Ana North Dam (Site 2). Eventually the flows in this subbasin series are conveyed underneath I-25 thru Culverts C24, C25, C26, and C27 into the West of I-25 Sub-basins series.

Sub-basins numbered in the 300 series

This series is shown in purple on **Figure 2**. These sub-basins are located on the east side of I-25 and outfall into Alvillar Dams 1B and 1C. Eventually the flows in this subbasin series are conveyed underneath I-25 thru Culverts C20, C21, C22, and C23 into the West of I-25 Sub-basins series.

Sub-basins numbered in the 400 series

This series is shown in pink on **Figure 2**. These sub-basins are located on the east side of I-25 and are the most southern basins of the study. Eventually the flows in this subbasin series are conveyed underneath I-25 thru Culverts C29, C30, C31, C32, C33, and C34, into the West of I-25 Sub-basins series.

Analysis points were determined based on the following:

1. Outfall locations based on topography
2. Culvert and drainage channel locations
3. Existing features (dams, principal and emergency spillway outfall locations)
4. Drainage paths (soil or streets) within Doña Ana
5. Known drainage problem locations
6. Street locations
7. Hydrograph divide locations

The total area of all sub-basins is about 15 square miles.

Drainage Analysis Criteria

Storms Evaluated:

The DACFC requested that four storms be simulated that are the 5, 10, 50 and 100-yr 24-hr duration storms.

Design Storm:

The design storms for this update is the 100-yr 24-hr storm.

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.

Existing Dams:

For the existing and proposed options HEC-HMS models, all four dams will be assumed to remain in place as they are certainly viable for the 5-yr and 10-yr design storms because all dams detain and attenuate the 5-yr and 10-yr. Details of the reservoir routing results for all dams will be provided later is this section for all four storms simulated.

2.4 RAINFALL DATA

The study basin is located within the USDA Natural Resources Conservation Service (NRCS) (previously the Soil Conservation Service (SCS)). **Figure 4** illustrates the Type II boundaries. The DACFC directed Smith to use the 25% Frequency Storm Distribution storm to simulate the type II-75 rainfall distribution which is supported by Figure R1 and R2 in **Appendix C**. This distribution is available in the HEC-HMS program and it places peak intensity of the rainfall at 25% of the storm duration, or at 6 hours for a 24-hour storm. The 25% Frequency Distribution Storm also distributes approximately 80% of the cumulative rainfall depth at 6 hours in the 24-hr storm. The SCS Type II on the other hand only distributes 40% of the cumulative rainfall depth over the same time. As a result, the peak discharges resulting from a 25% Frequency Storm will be higher.

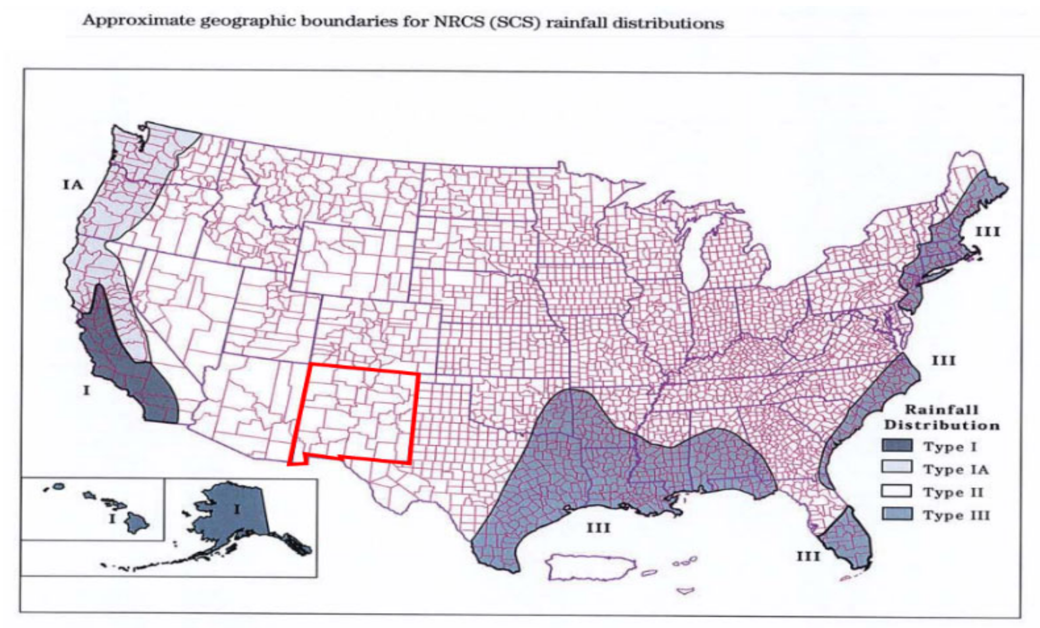


Figure 4: Approximate Geographic Boundaries for NRCS Rainfall Distribution

Figure 5 was adopted from a recent document by the Albuquerque Metropolitan Arroyo and the Flood Control Authority (AMAFCA), called the *State of Practice for Hydrology, Migrating from AHYMO'97 to HEC-HMS (and USEPA SWMM)* by Mr. Charles Easterling P.E., dated June 2018. This figure provides a graphical comparison between the different 24-hour rainfall distributions.

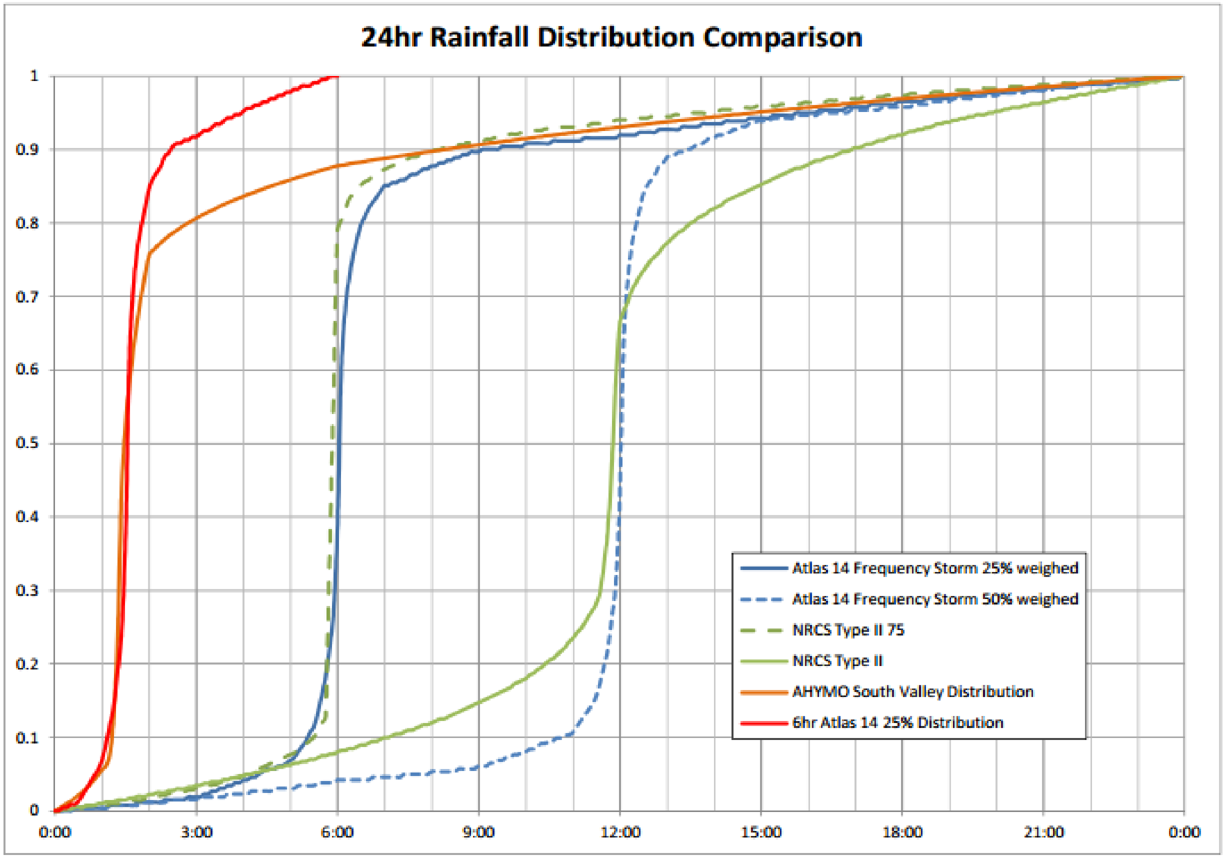


Figure 5: 24 Hour Rainfall Distribution Comparison

Point Rainfall Data

Point rainfall data for the 5-yr, 10-yr, 50-yr and 100-yr 24-hr return period storms for various durations were obtained from NOAA Atlas 14 website for the lower basin (west of I-25) and also for the upper basin (east of I-25).

Appendix C contains the printouts from the NOAA Atlas 14-point rainfall data results. The point rainfall depths are basically identical with the upper basin depths slightly larger, between the lower and upper basins, therefore, the upper basin point depths were assumed applicable to the entire basin model.

The 24-hr total point rainfall depth values are follows:

Return period	Depth (inches)
5 year	1.90
10 year	2.24
50 year	3.10
100 year	3.51

2.5 SCS RUNOFF CURVE NUMBERS (CNS)

Background Information

The Natural Resources Conservation Service (NRCS) Runoff Curve Number (CN) Method was used to simulate the excess precipitation (storm runoff) as a function of the rainfall initial abstraction loss and rainfall infiltration loss. The CN value is selected based on the following items:

1. Hydrologic Soils Group (HSG)
2. Land use and cover type (imperviousness, vegetation type, agriculture, etc.)
3. Hydrologic Condition that is an estimate of ground cover density (poor, fair, good)
4. Antecedent Runoff Condition (ARC).

Appendix C contains the soil survey information used for this project. This soil data was obtained from the NRCS website: ***http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx.*** **Figure 6** illustrates the HSGs for the drainage basin. The HSG are classified based on their minimum infiltration rate and are broken into five categories: A, B, C, D and unknown. HSG A has the highest infiltration rate (pervious soils with high hydraulic conductivity) while HSG D has the lowest infiltration rate (poor soils with low hydraulic conductivity) of the four HSGs. If an HSG is labeled as unknown, site photos, surrounding soils and site conditions are used to assign a soil group.

When land is developed the soil is impacted as areas are partially covered with impervious areas (such as pavement and concrete) and virgin soil conditions are altered to meet density requirements for construction. To illustrate the impact development has on runoff the NRCS method also uses Land use and cover type as a parameter. This parameter is illustrated on Tables 2-2a through Table 2-2d which show various land use and cover types. Land use types range from fully developed urban areas to undeveloped native land. The land use and cover type for this project was taken from aerial photography provided by the DACFC. Typically, if there are multiple land use types in a subbasin an aerial weighted CN is used. However, experience has shown that when curve numbers have a greater disparity than 10 are weighted together, the peak discharges predicted are significantly lower than what's expected. This is largely illustrated in the subbasins that are largely undeveloped and have small pockets of development. For these subbasins an additional percent impervious was applied to the model. This percent was obtained from the TR-55 Table 2-2a and was applied to the urban districts and residential districts land use types. For example, if a Subbasin A has 90% of undeveloped rangeland and 10% of 1-acre residential, then from Table 2-2a it shows that the average percent impervious area for this land type is 20% Therefore, the model would show a percent imperviousness of 2% (which is 20% of 10%). This is discussed in further detail below.

In addition to HSG and Land use/ cover type, the hydrologic condition of the land should be taken into consideration. The hydrologic condition is indicated by the amount of vegetative cover. For this project the drainage basin was in poor hydrologic condition with the native/undeveloped land having less than 50% ground cover.

The ARC is a term that represents an assumption as to the watershed moisture condition at the time of the storm event. CN values have been developed by the NRCS for three moisture conditions defined as either dry (ARC I), average (ARC II) or wet (ARC III).

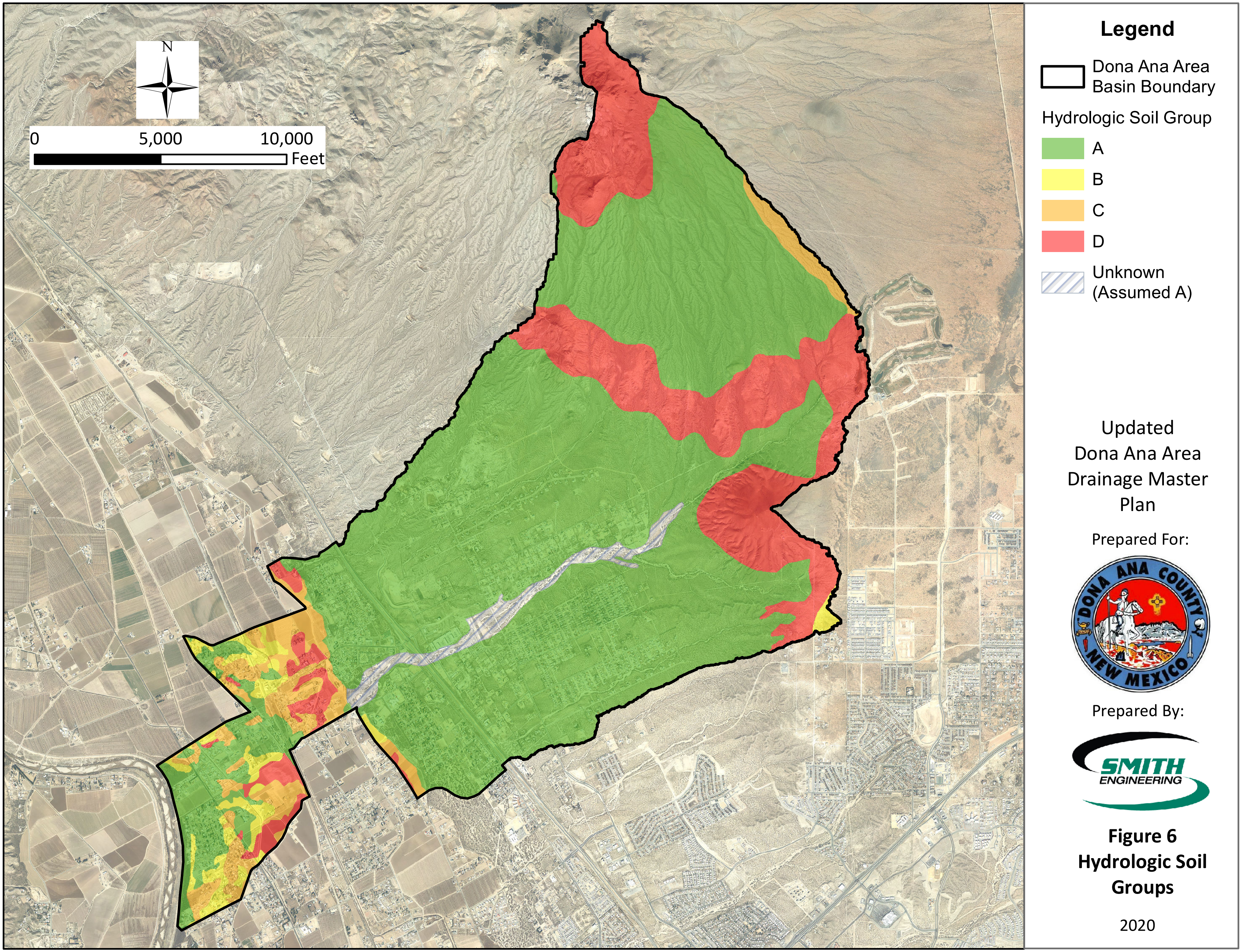


Figure 6
Hydrologic Soil Groups

2020

Establishing ARC II CNs

The 2017 DMP used an average CN value between ARC II CN and ARC III CNs, which is a conservative approach. In June of 2019 DACFC approached Smith to revise the CNs to ARC Type II and update the hydrologic and hydraulic models accordingly.

The NRCS Web Soil Survey results for the various HSGs often does not account for development and much of the data is outdated often representative of virgin soil conditions. Subdivisions that are built often change the virgin conditions by the implementation of fill and compaction of non-native soils.

When updating the CNs it was noted that the CNs for Soil Type A in residential communities were lower than the CN for Soil Type A in arid rangelands. The CN for Soil Type A, 1-acre lot residential is 51 while the CN for Soil Type A in arid rangelands desert shrub, poor conditions, is 63 (based on TR-55, Chapter 2, Tables 2-2a and 2-2d, include in **Appendix C**). Consequently, the peak discharges from developed areas were lower than the undeveloped semi-arid rangeland. Based on Smith's extensive experience in studying watersheds in the Doña Ana County and on actual video evidence from last summer's storm events (as recorded by residents in the vicinity of Windmill Rd) the models peak discharges were incongruent with the evidence at hand.

It was clear that a sensitivity analysis was in order. As such Smith developed HEC-HMS models using various CNs to illustrate how the runoff is being impacted. The table, below, is a summary of four HEC-HMS models that modeled an arbitrary subbasin. The first two subbasins was assumed to have 100% HSG A and the same subbasin area. Subbasin CN-51 is a 100% developed subbasin with 1 acre lots and has a corresponding CN of 51. Subbasin CN-63 is a 100% undeveloped, arid rangeland, desert shrub in poor hydrologic conditions and has a corresponding CN of 63. The only difference in the model between the two subbasins is their CNs. As shown the flow from subbasin CN-63 has almost 5 times the flow of subbasin CN-51. Which means, the undeveloped subbasin has less infiltration occurring than that of the developed subbasin. One of the reasons for this outcome is the assumptions that the developed conditions (shown on the TR-55 Table 2-2a, located in **Appendix C**) were created on. These assumptions are:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition
2. Impervious areas have a CN of 98 and are directly connect to the drainage system.

Unfortunately, the watershed being analyzed does not meet assumption 1. The area is not in good hydrologic condition and it is not in pastureland. CN Type II are standard and are used throughout the southwest to approximate initial abstractions. But it is also logical to look at and realize that the flow will not be accurately portrayed in developed areas within HSG Type A. To come up with a solution that would not cause over design but also not be under designed Smith Engineering went through several iterations and came up with a multi solution.

Table 2: Comparison of CNs

Subbasin	Area (sq. miles)	% HSG A	% HSG B	CN	Q_{100} (cfs)
CN—51	0.2718	100		51	20.7
CN—63	**0.2718**	**100**		**63**	103.9
CN—68	**0.2718**		**100**	**68**	163.1
CN—63—5%	0.2718	100		63	133

If the subbasins were primarily developed and located in HSG A, the CN for HSG B was used instead to account for impacts due to construction and landscaping. In the above table the subbasin CN-68 is a 100% developed, 1-acre lots subbasin that has 100% of HSG B. This increased the flow from 21 to 163 cfs, which is higher flow than the undeveloped land runoff of 104 cfs (which it should be since the developed land has more impervious area, resulting in less infiltration and a higher runoff). Changing the soil type in developed basins, is not that unusual as the soil will naturally be more compacted and varied due to human traffic, landscaping, and vehicle compaction making the soil less permeable.

If the subbasin was primarily underdeveloped but had small portions that were developed a percent impervious was used rather than a CN from a different soil type. This method was applied to the subbasins in groups 400 and 300. Using a percent impervious based on land types shown in the TR-55 manual on Table 2-2a was applied to the developed portions of each subbasin. The table above, shows the flows for the above mentioned arbitrary subbasin. This subbasin CN-63-5% has a 63 CN (undeveloped, arid rangeland, desert shrub in poor hydraulic conditions) and has a percent imperviousness of 5. This increases the flow from 104 to 133 cfs.

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

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

The TR-55 method (Cronshey, 1986, pp. 3-1 – 3-4) was used to compute the time of concentration for the sub-basins. A water course may have up to three sub-reaches that comprise the longest flow path as defined by the TR-55 method, including:

- **Sub-Reach 1** defined as an upper overland sheet flow reach not to exceed 300 ft in length. The method allows the engineer to exercise judgement on the appropriate reach length based on watershed characteristics. For the sub-basins in the Doña Ana Watershed, a typical length of 100 ft was selected.
- **Sub-Reach 2** defined as a shallow concentrated flow reach not to exceed 2000 ft. The maximum length of 2000 ft was selected for computations.
- **Sub-Reach 3** defined as a channel flow reach that comprises of the remainder of the flow path.

The time of concentration (T_c) for the watercourse equals the summation of travel times for each sub-reach. T_c is defined as the time required for water to travel from the most hydraulically remote point in a sub-basin to the point of interest or the outlet of the sub-basin. The NRCS Unit Hydrograph Lag Time Method (T_L) was applied to the T_c to compute the unit hydrograph Time to Peak (T_p). Note that Lag Time = 0.6 T_c. **Appendix C** contains the reference pages from National Engineering Handbook (NRCS, 2010, Chapter 15) describing the TR-55 method procedures used for each water course sub-reach previously described.

Using spatial analysis, elevations, lengths and slopes were extracted from the DEM. Typical channel widths were measured from the ortho-imagery provided by the DACFC. Manning's Roughness Coefficients "n" were chosen based on guidance provided in "Open Channel Hydraulics" (Chow, 1959). Copies of "n" value tables are included in **Appendix C**.

For more detail output see **Table C3** (**Appendix C**) and **Figure 3 (in map pocket**) which summarizes the travel time, time of concentration, and lag time data.

2.7 CHANNEL ROUTING

The "Muskingum-Cunge" channel routing method was applied to route hydrographs. Manning's Roughness Coefficients were chosen based on guidance provided in the Open Channel Hydraulics textbook (Chow, 1959), included in **Appendix C**. Channel routing lengths, slopes and typical bottom width were assumed based on the DEM and orthophotography. **Table C4 (Appendix C)** presents the Muskingum-Cunge channel routing input data summary. Arroyo bed runoff losses from infiltration and percolation were assumed to be small and were not considered or simulated.

2.8 SEDIMENT BULKING

The HEC-HMS models simulate clear water hydrographs unless a "Flow Ratio" is applied to simulate sediment volume within hydrographs. This parameter is also called sediment bulking. A sediment bulking value of about 19% is considered the limit before mud flow would occur. Refer to the "References" section of **Appendix C** that contains a portion of the "Sediment and Erosion Design Guide, and Figure 3.8 (Mussetter Engineering Inc. Nov. 2008), that is the basis of the 19% limit.

For this basin, Flow Ratio values were assumed based on engineering experience and judgement. The surrounding area is largely undeveloped with field conditions showing, even on roads, sediment deposits. Based on this and on visual estimates based on the orthophotography presented in the Drainage Basin Map, **Figure 3**, a 10% sediment bulking factor was used. That assumption is based on review of information presented in Mussetter Engineering, Inc. (2008). **Appendix C** contains a copy of relevant pages from that document. **Table C5** included in **Appendix C** represents the flow ratio assumptions for each sub-basin.

2.9 HYDROLOGIC DATA SUMMARY

Tables C5 in **Appendix C** provides a summary table for all the input data required for the HEC-HMS model.

2.10 COMPUTATION TIME INCREMENT FOR HEC-HMS MODELS

The computation increment assumed within a HEC-HMS model may make a large difference in model peak discharge results particularly for large drainage basins. Guidance on computation intervals was found in the HEC-HMS manual and **Appendix C** contains a copy of that guidance. For this project, a 5-minute computational time step was utilized.

2.11 INFLOW-DIVERSION FUNCTIONS

Inflow-Diversion (Inf-Div) Function provides the capability to divide a sub-basin hydrograph into two hydrographs. The inflow hydrograph is divided by the Inf-Div rating curve values and stored in the “diversion” or “connection” hydrograph (shown as a dashed line in the HMS schematic) and the remainder of the inflow hydrograph in excess of the rating curve is stored in the “main” or “downstream” hydrograph (shown as a solid line in the HMS schematic). These Inf-Div functions were applied to several locations for specific reasons as provided here:

1. Divide the North Dam and South Dam hydrographs based on the principal and emergency spillway rating curves to determine the peak discharge at each structure.
2. Divide the South Dam hydrographs combined with local sub-basin hydrographs below the dam, near the I-25 large box culverts (2 – 8 ft. rise X 10 ft. span). The purpose is to simulate the fraction of the flow below the South Dam that will spill north along the east side of I-25 to Thorpe Rd. This was determined based on the HEC-RAS 2D modeling (presented later) that illustrates that some flow will spill as described.
3. Divide the North Dam principal spillway hydrograph at W. Thorpe Rd. based on the limited culvert capacity (Culvert C1) that outfalls south of Thorpe Rd. into the El Camino Real Rd. Channel.
4. Divide the flows at the Thorpe Rd. inlets and storm drain (inlets located under I-25) to simulate a fraction of the total discharge that will outfall to the El Camino Real Channel from that storm drain and to account for flows in excess of the inlet / storm drain capacity that will spill west on Thorpe Rd. as this location was initially assumed to have flow spill west on Thorpe Rd. However, the HEC-RAS 2D model results indicate that even the 100-yr flows at these inlets (which far exceeds the inlet capacities) will spill south to the El Camino Real Channel (near the storm drain pipe outfall from the inlets) and none will spill west on W. Thorpe Rd.
5. Divide a few hydrographs at the east side of I-25 that have capacity limitations to provide for ponding that will occur due to the I-25 roadway embankment at these culverts. This will provide a more accurate hydrologic analysis of the flow that will outfall from some of the I-25 culverts to the west side of I-25.

6. Divide a few hydrographs at other locations in the basin to simulate assumed divide locations.

Tables C6.1 and **C6.2** included in **Appendix C** contain descriptions of these Inf-Div function locations and associated hydrograph divide values. **Table C6.3** (**Appendix C**) includes the I-25 upstream embankment ponding areas elevation – storage – discharge rating curves applied to account for upstream ponding at limited culvert capacity locations.

2.12 RESERVOIR ROUTING DATA

Elevation-Area- Storage- Discharge data, assumptions and computations for each dam are summarized in tables within **Appendix C.** Discussion of some aspects of the sources and data are presented here.

Doña Ana North and South Dams

Appendix B contains copies of the final plans provided by the EBID (owner) for these dams. These plans provide the elevation storage data and state that these dams were each designed for the 50-yr flood that would spill through the emergency spillway. Note that the current storage volumes far exceed those stated in the construction plans.

Table 3: Summary of Storage Volumes for North Dam Sites 1 and 2

Dam Name	Storage Volume to top of dam from 1957 "Final" Plans	Storage Volume to top of dam from 2016 topo survey / 2010 Lidar contours
	ac-ft	ac-ft
North dam (Site 2)	346	507
South dam (Site 1)	660	761

The current Elevation-Area- Storage volumes were computed based on a recent limited topographic survey provided by DACFC that was completed mainly for the lower reservoir areas to obtain better information due to recent excavations of the reservoir lowest elevations performed by the EBID to remove sediment. **Appendix B** contains copies of the limited topographic survey contour maps. That information was combined with the Lidar 2010 contours that were adopted for the higher reservoir elevations that were not included in the topographic survey. The principal spillway rating curves were adopted as presented in the original plans. The emergency spillway rating curves were computed by Smith Engineering as these were not provided in the original construction plans. However limited data regarding the emergency spillway lengths and weir coefficient "C" values were adopted to assist in development of the emergency spillway rating curves. **Appendix C** contains the information and various computation tables developed to provide the current elevation – storage- discharge rating curves applied in the HEC-HMS models.

Alvillar Dams 1B and 1C

Appendix B contains copies of the final plans provided by the DACFC (owner) for these dams. The elevation – area – storage data from these dams were developed from DACFC topographic surveys provided to Smith Engineering that have the following dates.

Alvillar Dam 1B Dam Topo Date 3-26-2004, revised 8-12-2011

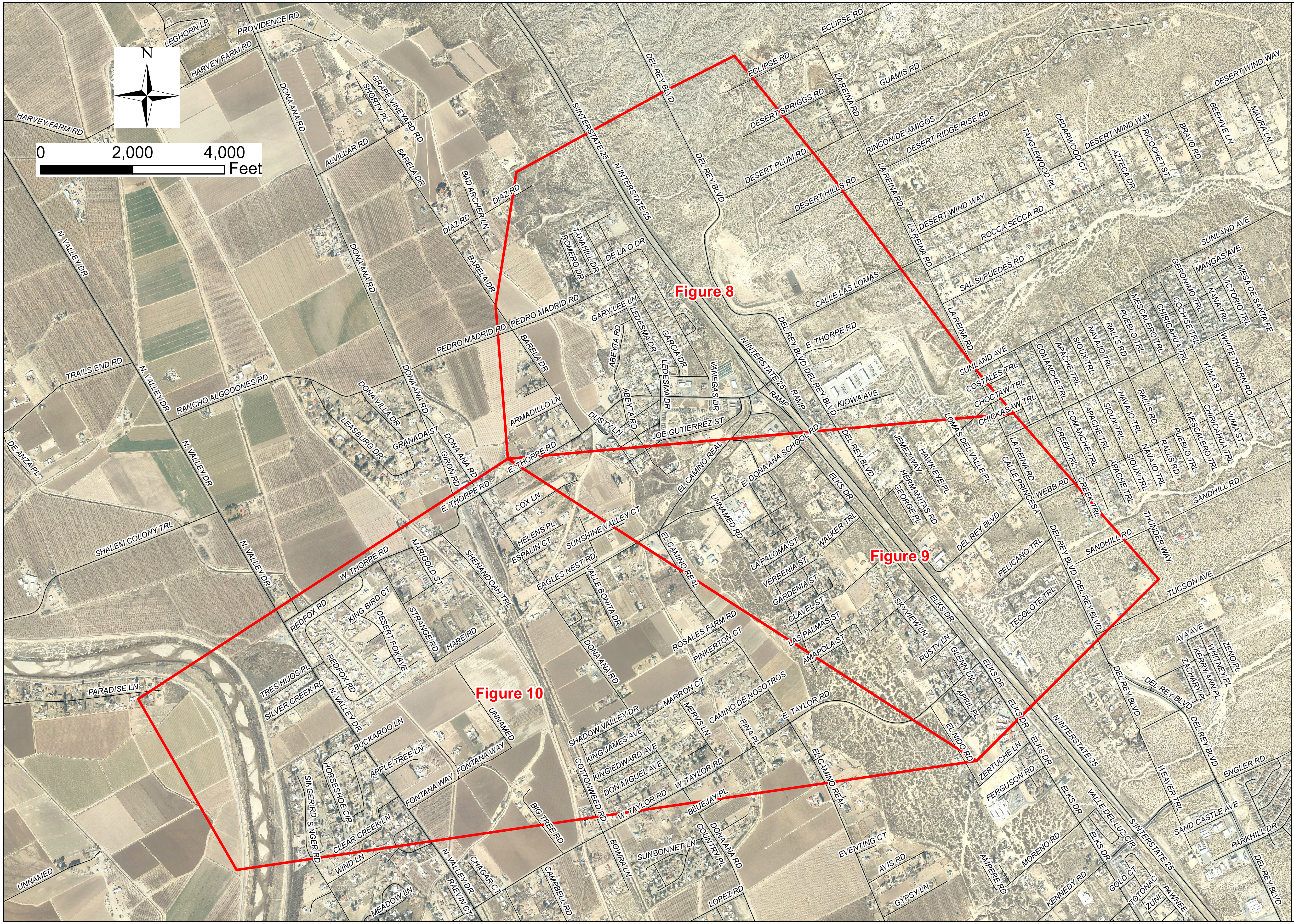
Alvillar Dam 1C Dam Topo Date: 4-9-2003, revised 8-12-2011

Appendix B contains copies of these topographic maps and **Appendix C** contains the elevation-area-storage-discharge data and associated computation tables.

2.13 HEC-HMS SUMMARY RESULTS

Appendix E contains the HEC-HMS existing and proposed HMS models and the associated summary output.

Figure 7 thru **Figure 10** on the following pages note peak discharges at select locations and illustrate the discharges that were divided based on the HEC-RAS 2D modeling results and existing culvert capacities.



Legend

Key Map

Roads

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Figure 7

Existing Peak Discharge Schematic Overview

2020

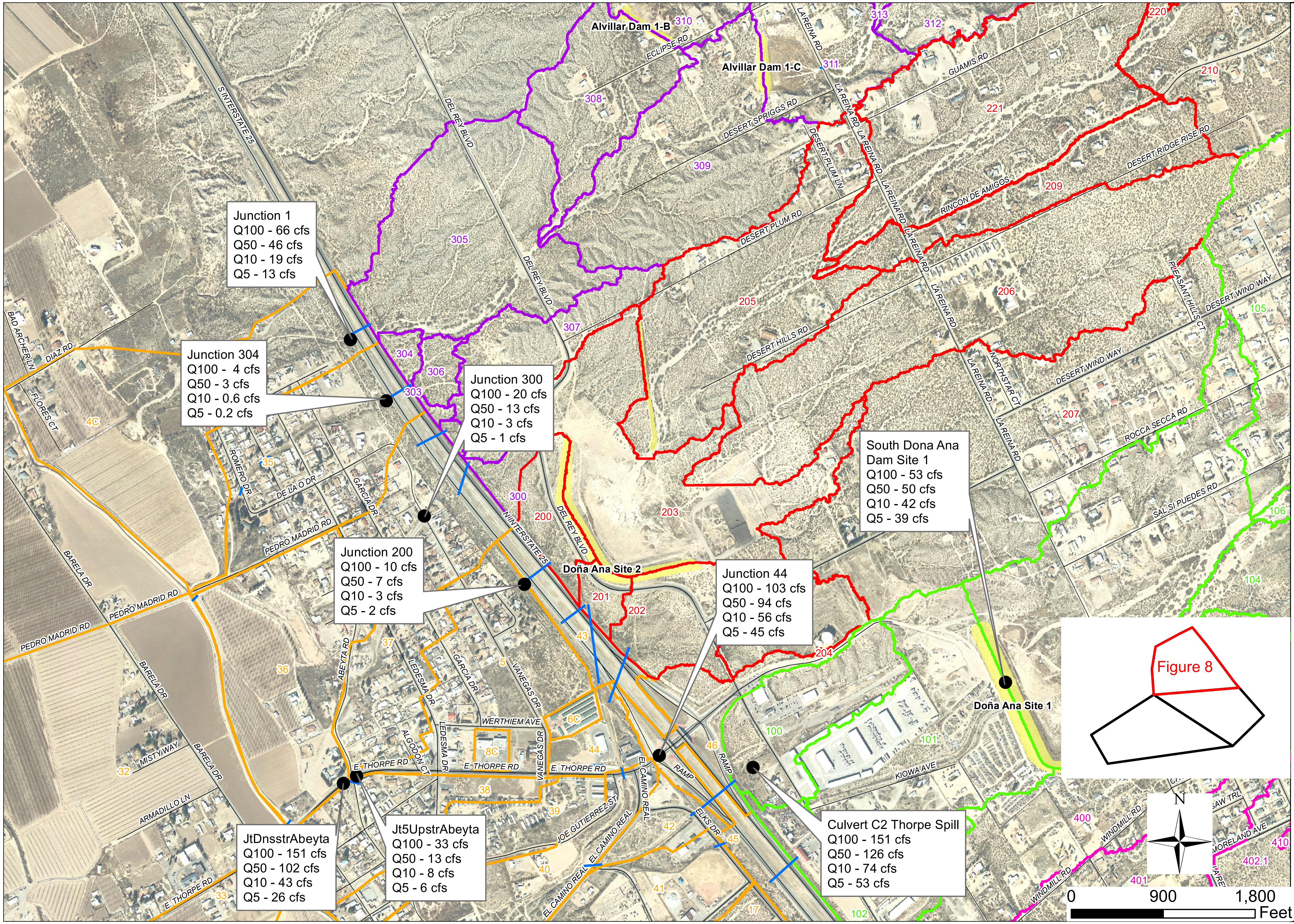

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Figure 8
Existing Peak Discharge Schematic
1 of 3

2020

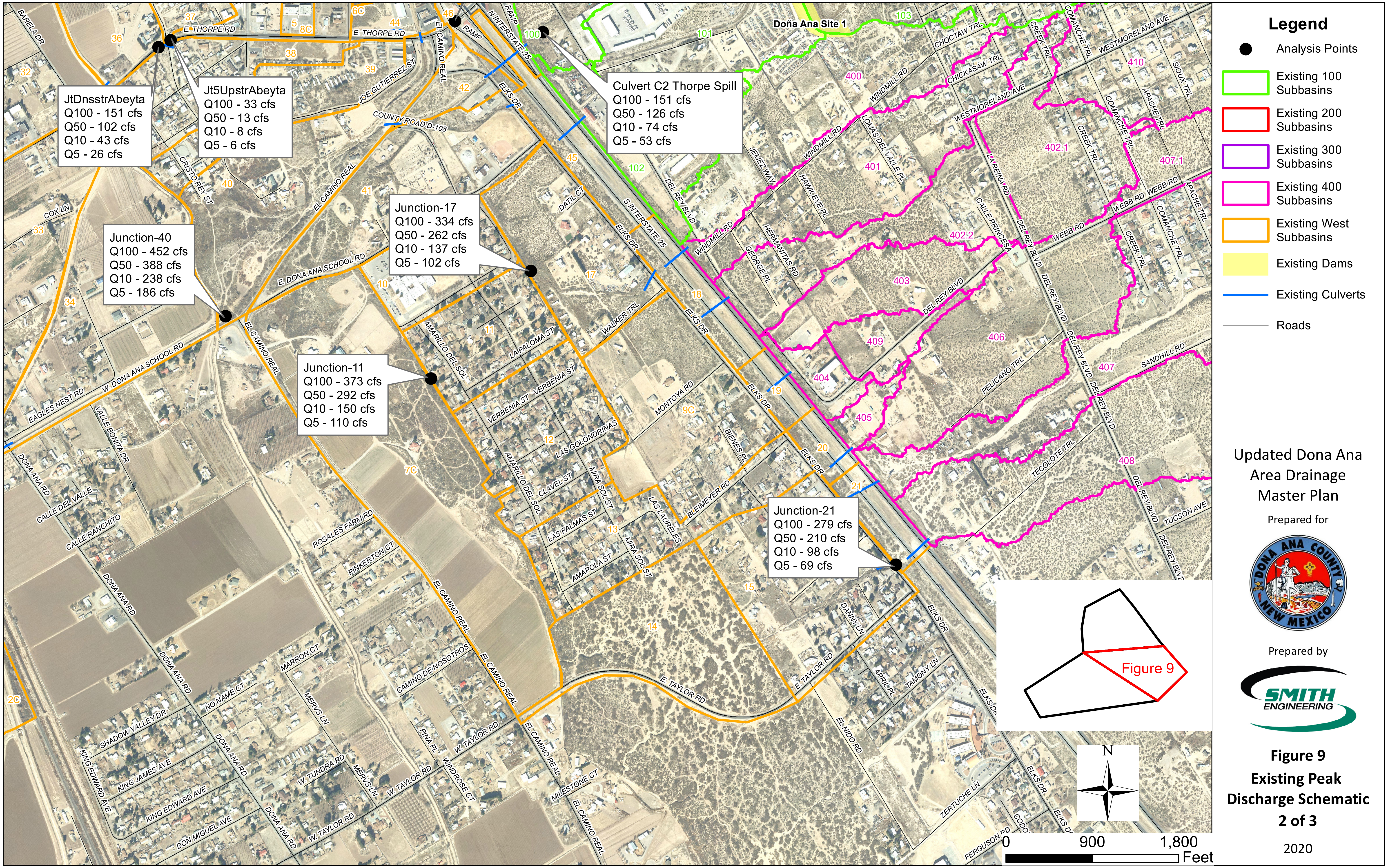
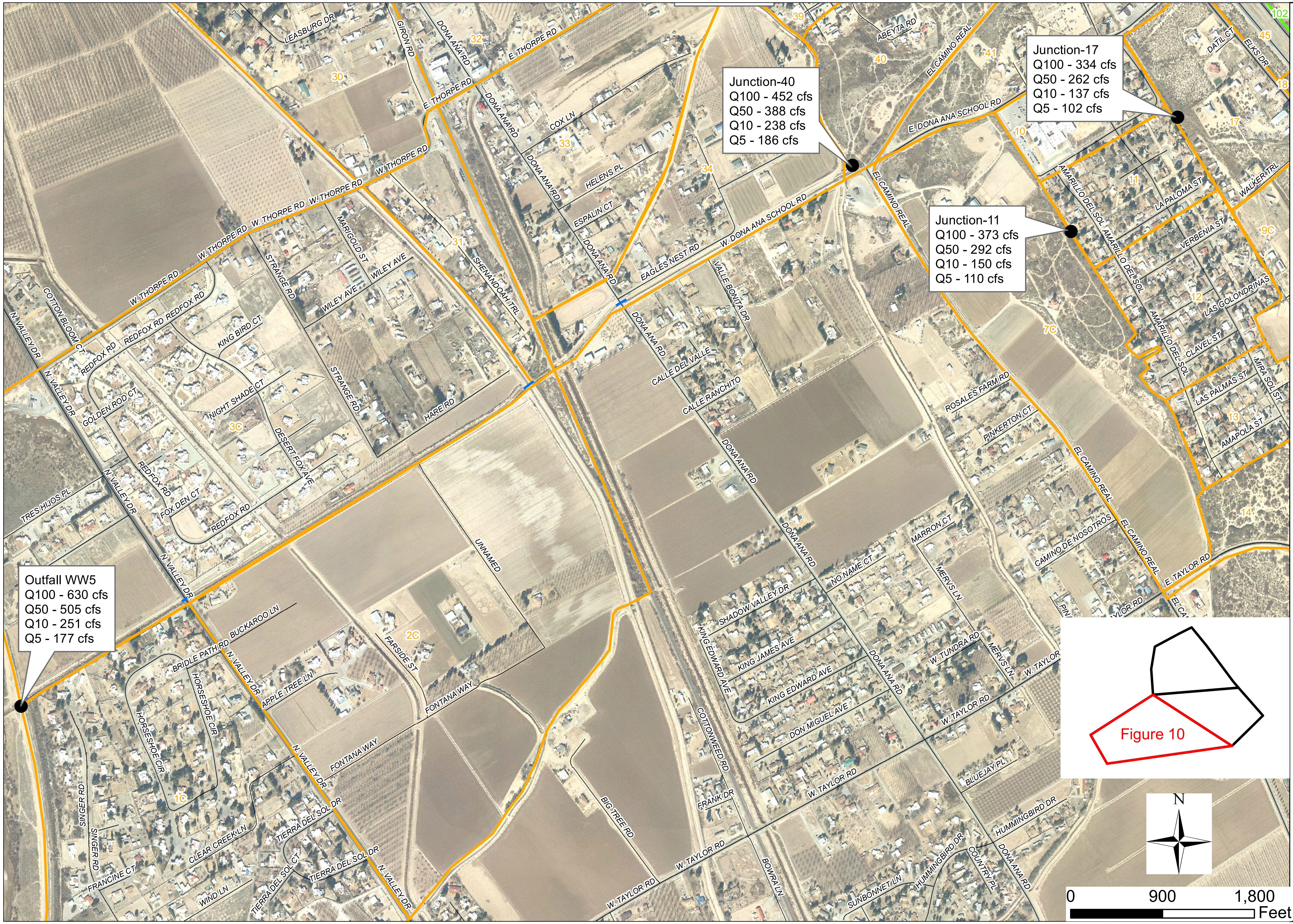


Figure 9
Existing Peak Discharge Schematic
2 of 3

2020



Legend

- Analysis Points
- Existing 100 Subbasins
- Existing 200 Subbasins
- Existing 300 Subbasins
- Existing 400 Subbasins
- Existing West Subbasins
- Existing Dams
- Existing Culverts
- Roads

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Figure 10
Existing Peak Discharge Schematic
3 of 3

2020

2.14 PERFORMANCE OF EXISTING DRAINAGE INFRASTRUCTURE

Existing Dams (Reservoir) Routing Summary Results

North and South Dams Brief Summary

The North and South Dam contains the 100-yr and smaller floods below the emergency spillway.

Alvillar Dams Brief Summary

Alvillar Dam 1B contains the 100-yr and smaller floods below the emergency spillway.

Alvillar Dam 1C spills through the emergency spillway as follows:

100-yr = 5.4 cfs 50-yr = 0 cfs, 10-yr = 0 cfs 5-yr = 0 cfs.

DACFC Retention Pond Brief Summary (used Lidar Contours)

DACFC has a retention pond located west of I-25 and west of Elks Dr. that retains the 100-yr and smaller storms.

Tables C7.1 thru **C7.4,** located in **Appendix C**, contains additional information such as peak stage, freeboard and the discharges from the principal and emergency spillways.

Existing Drainage Infrastructure

The existing drainage infrastructure (excluding the four dams) in the vicinity of Doña Ana include culverts and channels. Culverts were labeled using the letter C followed by a number, for example C1 is Culvert 1. The same naming convention was used for labeling channels, for example CH1 is Channel 1. These structures are labeled on **Figure 11** and **Figure 12.**

Culverts

For the significant culverts, culvert at critical locations and the I-25 culverts, Smith measured and recorded the maximum allowable headwater (MAH) depth from culvert upstream invert to the edge of the adjacent road. Since the 2 ft contour Lidar elevations may be plus or minus 1 ft in elevation, the following culvert analysis method and assumptions were adopted as the basis to computed culvert maximum capacity.

1. Adopt relative elevations
2. All upstream invert elevations will be set at elevation 100
3. Culvert downstream elevations will be computed based on the culvert length at a 1% slope.
4. Culvert capacity will be computed based on the MAH depth to the edge of the road, Example, if the MAH depth is 7 ft. from culvert invert to edge of road, then the MAH elevation = 107 (100+7).
5. Compare the culvert capacity at MAH elevation to the 5-yr 10-yr, 50-yr and 100-yr peak discharges (24-hr storms).
6. Reduce the actual capacity to 85% to account for sedimentation and debris clogging the culvert.

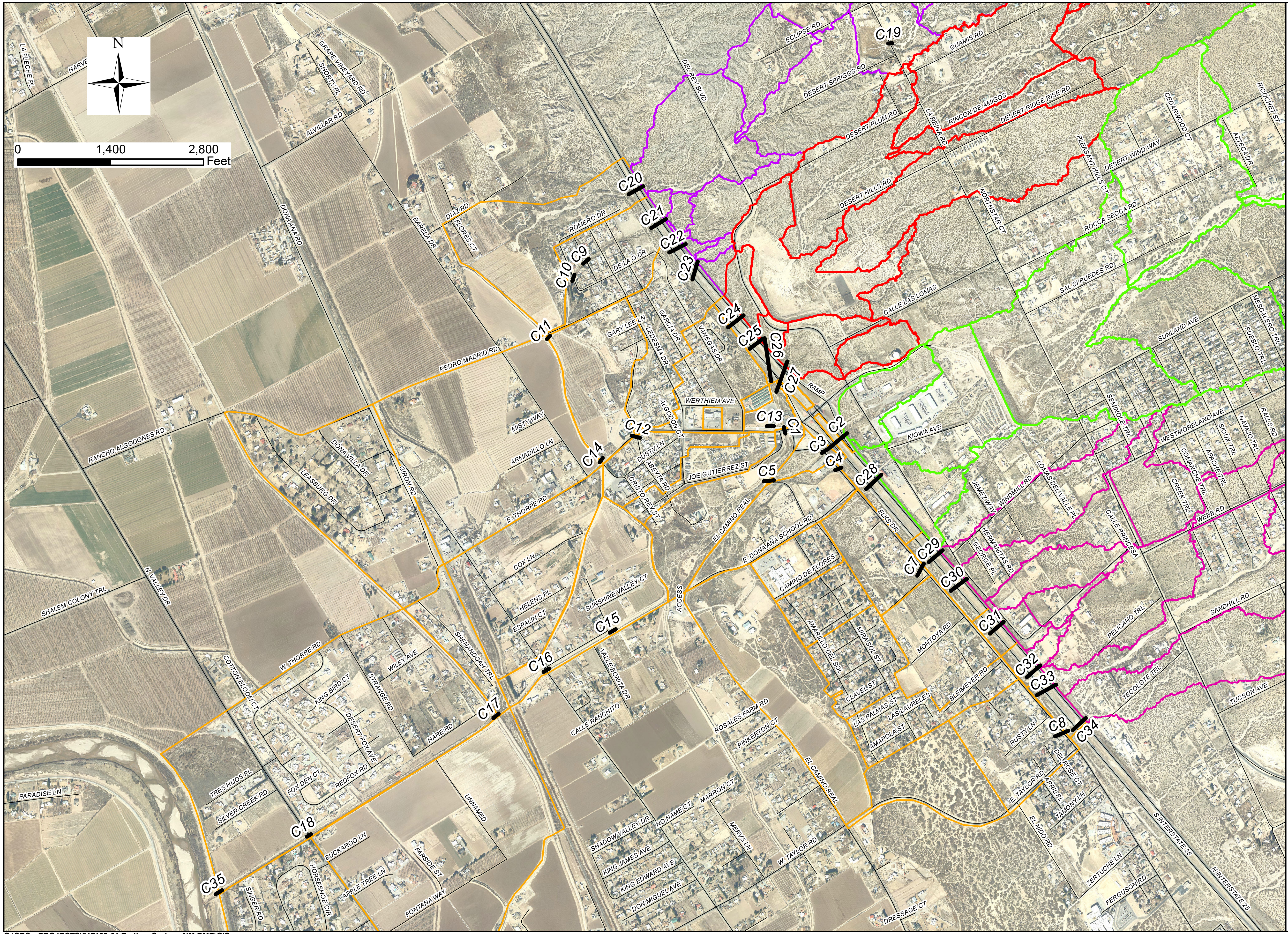
7. Assume additional culverts will be the same size and material as the existing culverts.
8. Compute the number of additional culverts required to pass discharges for each storm. Note that fractional culverts were computed (to the tenths place), for example 0.2 additional culverts. *Fractional culverts were computed to allow decision making as to whether additional culverts should be recommended if the existing culverts is slightly under capacity.*

NMDOT Roadways

NMDOT criteria was used to evaluate the I-25 culverts. According to this criteria the 50-yr flood may be allowed to reach the edge of shoulder elevation and the 100-yr flood may be allowed to spread into the roadway to either half of a driving lane or 1 full driving lane, depending on the road classification and number of lanes.

Culvert Data and Results

Figure 11 shows the location of the existing culvert that were analyzed for this project. **Table G1**, in **Appendix G** contains the culvert hydraulic data, assumptions and results. The table provides the number of additional culverts that may be required to pass each storm peak discharge (5-yr, 10-yr, 50-yr and 100-yr).



Q:\SEC---PROJECTS\817103-01 Radium Springs NM DMP\GIS

Legend

- Existing 100 Subbasins
- Existing 200 Subbasins
- Existing 300 Subbasins
- Existing 400 Subbasins
- Existing West Subbasins
- Existing Culvert
- Roads

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Figure 11

Existing Culvert Locations

2020

Channels

Figure 12 shows the locations of the various channels that were evaluated for maximum capacity. Channel slopes were computed based on the Lidar elevations at the upstream and downstream ends of the reach.

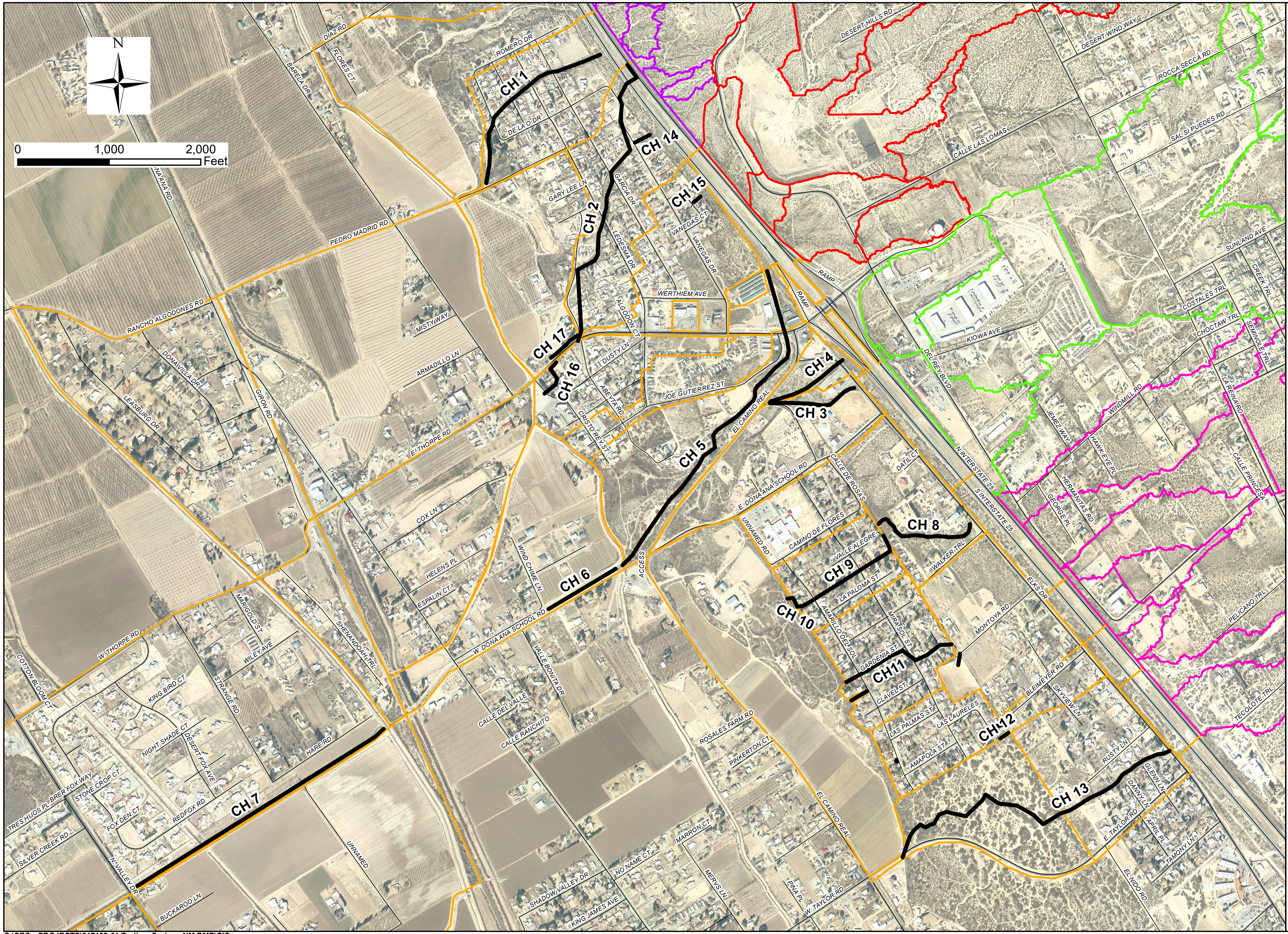
Google Earth combined with the contour data extracted from the DEMS generated from Lidar were used to estimate bottom widths, side slopes and depths. For some of the defined channels Smith field verified the dimensions such as bottom width and channel height and shape (rectangular or trapezoidal). Manning's Roughness coefficients "n" values, were estimated based on field observation, photographs taken during the field work and experience with open channel hydraulic analyses

The FlowMaster program was applied to compute the maximum channel capacities. The capacities were compared against the 5-yr 10-yr 50-yr and 100-yr storm peak discharges to determine those channels with deficiencies.

Table G2, in **Appendix G** contains the data applied and the capacity results. Since all open channel computations were based on estimated sections and parameters, freeboard was not addressed for this conceptual level analysis.

The comparison of channel flow capacities to the storm peak discharges indicates that all channels have adequate capacity. It should be noted that Channel 8, Walker trail has had been modified by landowners and the alignment and capacity of the channel may have been impacted. The calculations included in Appendix G are based on the topo provided by the County. The improvements to this channel are discussed in Facility 3 Datil Pond and in Option 2.

The analysis used for the channels assumes uniform flow which does not account for obstructions and possible backwater effects. Therefore, the analysis should be viewed as an approximate channel capacity evaluation.



Legend

- Existing 100 Subbasins
- Existing 200 Subbasins
- Existing 300 Subbasins
- Existing 400 Subbasins
- Existing West Subbasins
- Existing Channels
- Roads

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Figure 12

Existing Channel Locations

2020

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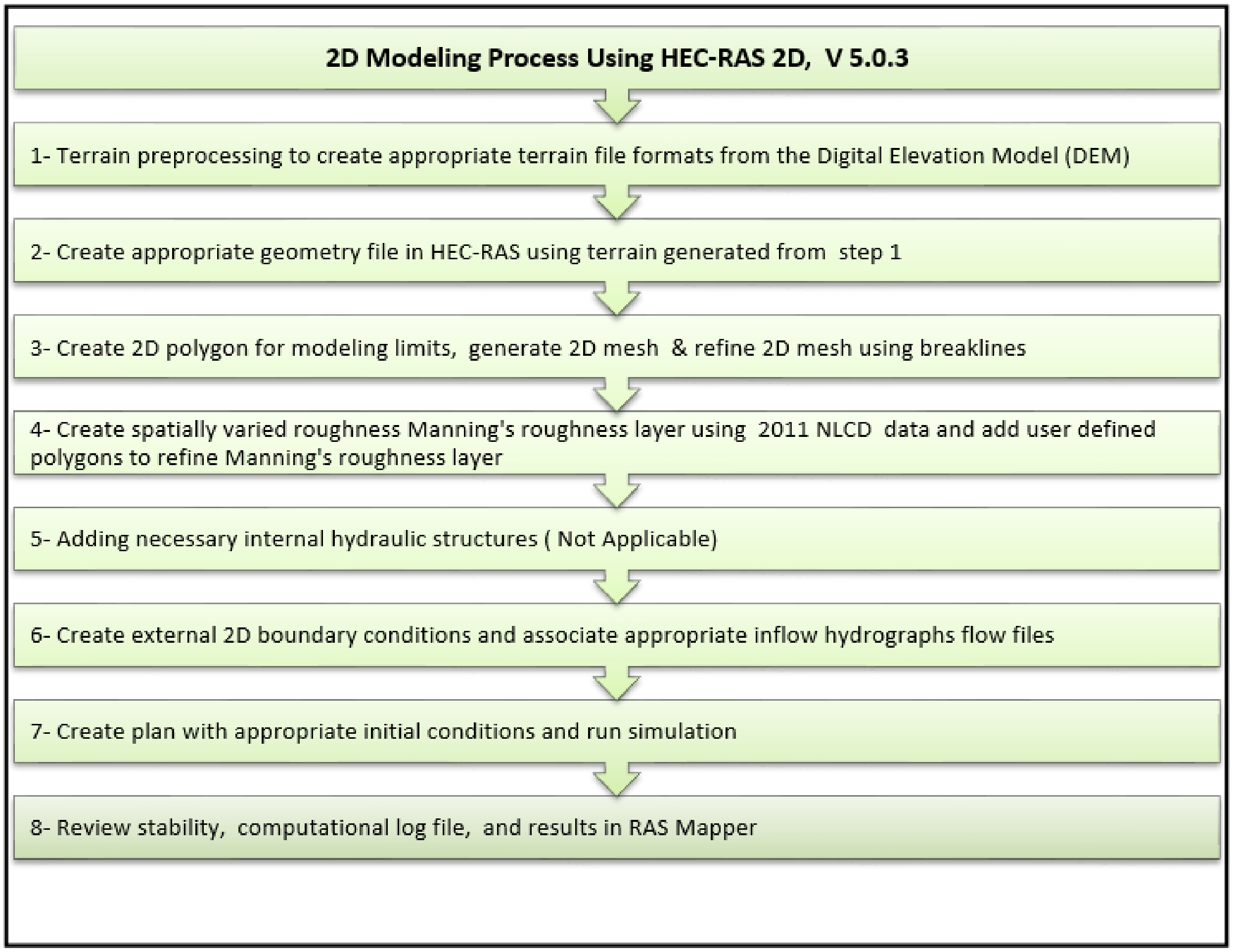
SECTION 3. FLOODPLAIN MAPPING

3.1 HEC-RAS 2D MODEL FLOODPLAIN MODELING

DACFC tasked Smith Engineering to develop a 2-dimensional model for two areas west of I-25. Both these areas receive offsite flows from the east side of I-25 and have drainage channels that intertwine through the residential areas. The two channels identified as CH 1 and CH 2 are shown on the drainage basin map (**Figure 12**). However, after performing the hydrologic analysis and understanding the complicated nature of the drainage through the area, it seemed wise to model the entire area west of I-25, instead of two separate discontinuous models. This will help to understand the full impact of the inflows from the sub-basins from the east side of I-25. The figure below outlines the general process to create a 2D model, however only the critical elements will be discussed in this section.



The 2D model covers approximately 3917 acres.

Standard HEC-RAS Limitations

1- 2D Ineffective Flow Areas

Programs such as GeoHECRAS allow the user to define 2D ineffective flow areas using polygons and they will automatically refine the 2D mesh to accommodate the ineffective flow area shape. The U.S. Army Corps of Engineers' version of HEC-RAS does not include this capability. Ineffective flow areas can be assigned to the 2D mesh to account for regions where the water is not actively being conveyed. The water will pond in such areas, and its velocity in the downstream direction will be close to zero. These defined regions are included in the storage calculations but are not included as part of the active flow area. When using ineffective flow areas, no wetted perimeter friction is included at the boundary between the ineffective flow area and active flow area. This HEC-RAS limitation results in high velocities within depressions and small ponding areas, where in fact, the velocity should be close to zero.

2- HEC-RAS Uniform 2D Mesh

HEC-RAS can create a mesh that contains squares and rectangles of uniform size and shape. Where breaklines are defined, the HEC- RAS meshing engine will refine the mesh using irregular mesh elements so that the mesh cell faces align with the breaklines. Refined regions can also be created with newer versions of HEC-RAS to provide smaller or larges cells where appropriate. However, part of the difficulty with a uniform mesh is that for regions where a refined mesh is necessary, there is not an automated way of refining the mesh. Programs such as GeoHECRAS offer adaptive mesh creation as described below. Current versions of HEC-RAS do not allow for adaptive mesh.

For a large and complex 2D flow study area, an adaptive 2D mesh can be used in place of a 2D uniform (i.e., square cells) for a faster and more accurate simulation. Adaptive 2D meshing allows to determine what size and shape element should be used, based upon the underlying terrain elevation change and user-defined breaklines.

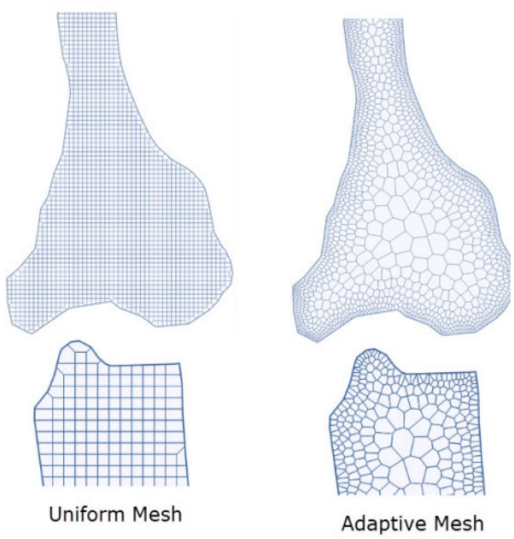


Figure 13: HEC RAS Mesh Type Comparison

The completed 2D adaptive mesh will have automatically refined elements (i.e., creating smaller elements) in areas where additional detail is required, and relaxed elements (i.e., creating larger elements) in areas where the 2D flow is relatively uniform and not much change is occurring. This mesh type is also significantly better at modeling changes in flow direction since the cell faces are generally perpendicular to any flow entering or leaving the cell. HEC-RAS does not automatically adjust the cell faces and cells are not necessarily placed perpendicular to flow directions.

Because of the inherent advantages of the 2D adaptive mesh, the HEC-RAS 2D flow simulations tend to be more stable (because of smaller element sizes where sudden changes occur) and run quicker (because of smaller total number of elements) than an equivalent 2D uniform mesh. A comparison of mesh types is shown in **Figure 13**.

Floodplain Analysis with Doña Ana North and South Dams (Sites 2 and 1)

2D Mesh Generation

After completing the terrain preprocessing as outlined in Chapter 2 of the user manual, the data was incorporated as part of the geometry file in HEC-RAS. The bounding polygon for the study area was defined as shown in **Figure 14** (Page 34). Using the bounding polygon, a 2D mesh was generated that consists of user-defined grids. The user defines the grid size to control the level of detail for the analysis. Since this was an analysis at a master plan level, a 50 ft X 50 ft grid cell size was chosen. This would equate to an average grid size of 0.06 acres. The terrain model was further refined using break lines to simulate the high points in the terrain that would act as a barrier to flow or to refine over banks of channels. Using a 50 ft X 50 ft grid won't define channel embankments or berms accurately since most channels or berms do not have 50 ft top widths. Therefore, using a smaller grid size around break lines was used to help define the channel and berm embankments. The break line grid size spacing ranged from 20 to 40 ft depending on size of channel or embankment.

Spatially Varied Manning's Roughness Layer

HEC-RAS 2D allows the user several options to account for friction losses for overland flow. Multiple options may be used for a single 2D grid and the user has control over which option takes precedence. In case of this model, user defined polygons were drawn within the geometry editor of HEC-RAS to model solid obstructions like buildings. RAS Mapper now allows the user to import aerial imagery from the ArcGIS World Imagery within the geometry editor. This feature was used to draw the user defined polygons where necessary. These polygons were typically assigned an 'n' value of 10 except for the area on the downstream of South Doña Ana Dam Site 1. This is an area that has significant walls, a junkyard and commercial/industrial buildings combined with open space. The 'n' values were modified to 3 and 0.3 respectively. Note that the buildings within the 2D limits were not extruded on the terrain model. This would take a significant amount of effort to recreate the DEM from the original point cloud which was not appropriate given the level of detail for this model.

The 2011 National Land Cover Dataset (NLCD_2011) for the Doña Ana area was downloaded from the Natural Resources Conservation Service geospatial data gateway website. This raster data set provides a spatially varying 'n' value based on land use and classification created from a unique Value and Name assigned within the raster data set. The program is than able to apply the data to the 2D mesh as it performs the 2D flow computations. The table below summarizes the NLCD_2011 data. Some of the 'n' values had to be artificially increased in order to lower the velocities shown in the earlier simulations.

Table 4: NLCD 2011 Mannings 'n' Values

Value	Name	Mannings 'n' Value
1	barren land	0.06
2	cultivated crops	0.06
3	developed high intensity	10
4	developed low intensity	0.1
5	developed medium intensity	0.5
6	developed open space	0.06
7	herbaceous	0.06
8	shrub	0.06

Finally, the program applies a default 'n' value of 0.06 across the board for all the cells. Within RAS Mapper, the user can set the precedence for 'n' values.

For this model, the order of importance was:

1. User Defined Polygons
2. NLCD_2011 Raster Data
3. HEC-RAS – default

Internal Hydraulic Structures

An additional feature that was simulated in this model was a hydraulic structure inside of the 2D mesh. This was done to simulate culverts C2, two 10 ft S X 8 ft R concrete box culverts (CBCs) that drain flows from the south east corner of the I-25 and Doña Ana exit. The initial simulations demonstrated that the overflow from the emergency spillway from South Doña Ana Dam Site 1 would get to these culverts. The flows would split based on culvert capacity. The culvert capacity is a function of how deep the water would stack up on the upstream side of the culverts and the adjacent terrain. Once the water depth becomes greater than surrounding terrain, whatever flows that were not passed through the culvert would eventually drain north to the underpass of I-25 and Thorpe Rd. and flow through the underpass to drain west along Thorpe Rd. The 2D model also demonstrated that once on the west side of I-25 and Thorpe Rd the flows would further split into channel CH 5 to the south and Thorpe Rd. It was necessary to observe the water depths at culvert C2 and in order to refine the diversion rating curves that model the flow splits in the HEC-HMS hydrologic model. Please see the Hydrology section that describes how these rating curves function to split an inflow hydrograph. It was important to have reasonable flow splits at these specific locations due to the impact that has on the analysis of existing facilities and the design of proposed facility improvements.

The current version of HEC-RAS 2D prevents the user to simulate internal hydraulic structure across several cells. The structure must cross only 2 cells in a 2D mesh. Since the culvert is approximately 400 ft. long, the grid cell size had to be modified to 500 ft. X 500 ft. at this location to meet the restrictions imposed by the program. The internal structures available for simulation are weirs, weir - gate combination and weir - culvert combination. For this model the weir – culvert combination was used. The station elevation data for the weir was coded to match the profile of the edge of the road to preserve the correct amount of hydraulic head available to the culverts.

External 2D Flow Area Boundary Conditions

The 2D flow area must have an upstream and downstream boundary condition specified. For areas where flow leaves the model, normal depth was specified. Since the downstream areas are typically flat agricultural fields, a typical energy slope of 0.1% was specified. The upstream boundary conditions basically simulate locations where flows are added into the mesh.

Please refer to **Figure 15 thru Figure 17** (pages 35-37) for locations where flows were added into the 2D mesh. These locations simulate outfall of culverts that add flow on the west side of I-25 into the study area. The emergency spillway for the South Doña Ana Dam Site 1 also simulated.

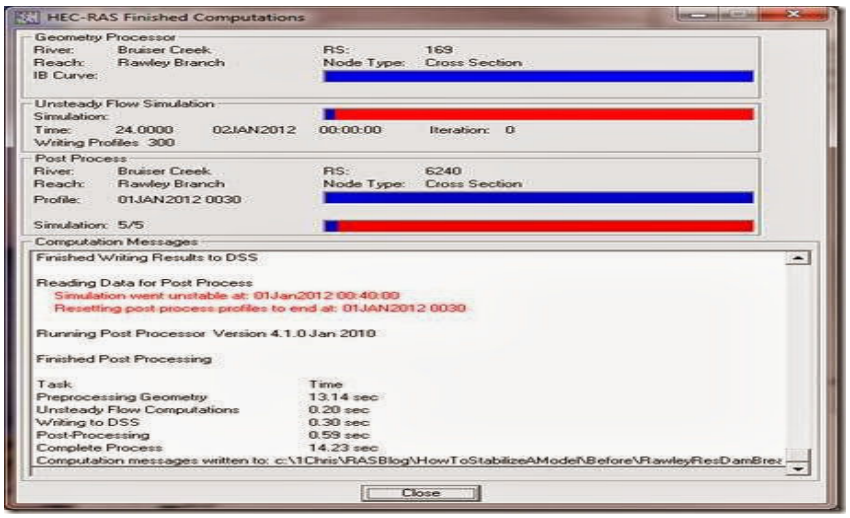
The hydrographs from the HEC-HMS hydrologic model at the appropriate junctions were imported into an unsteady flow file in HEC-RAS. Once again, the energy slope must be specified within the unsteady flow file, which was assumed at 1%.

Setting Up Plan Initial Conditions

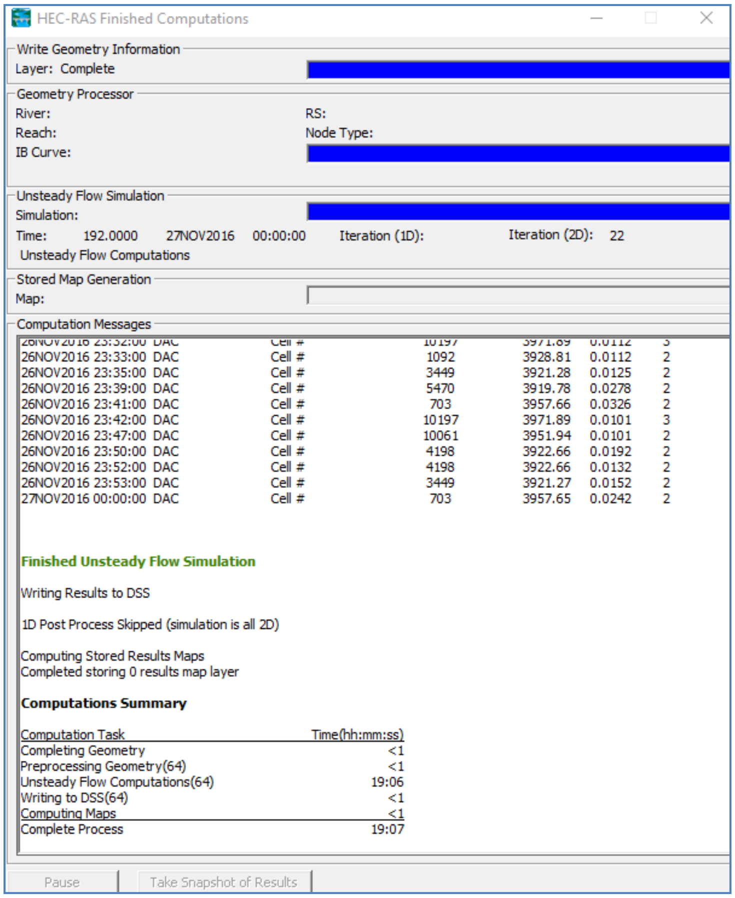
An unsteady analysis plan was then set up and initial conditions for the 2D analysis were defined. All the default values for 2D flow options were assumed. The 2D area was assumed to have dry initial conditions. The program allows the 2D computations to be based on either the Diffusion Wave equation or the Full Momentum equation. There are guidelines in the user manual for when to pick the Full Momentum equation vs. Diffusion Wave. In this instance, the diffusion wave equation was selected initially in order to reduce simulation time and instability issues. Once the kinks in the model were resolved, the solving regime was switched to Full Momentum. The Full Momentum equation became default because it simulates dynamic flood waves, wave propagation and super elevation around bends much more accurately. All of the above were true for the study area. Selecting the correct time step the last part of model building process. Based on the guidelines for Full Momentum Equation, a time step of 15 second was selected. At this point, the hydraulic properties for the cells within RAS Mapper were computed.

Simulation Run and Results

The results from the 2D analysis are best viewed dynamically in RAS Mapper to see how the flow distributes over the terrain over the duration of the hydrograph. There are many variables that can be queried within RAS Mapper. The three that are provided by default are depth, velocity and water surface elevation. Typically, if the model has 2D mesh errors or incorrect simulation time step interval it will be unable to converge the solution for the 2D mesh and go unstable which then a message appears as shown.



In this case, this window did not show up proving the model was performing the computations and achieving convergence for all the cells. Upon completing the simulation run successfully, this window (below) opens up indicating that results are now ready to be viewed in RAS Mapper.



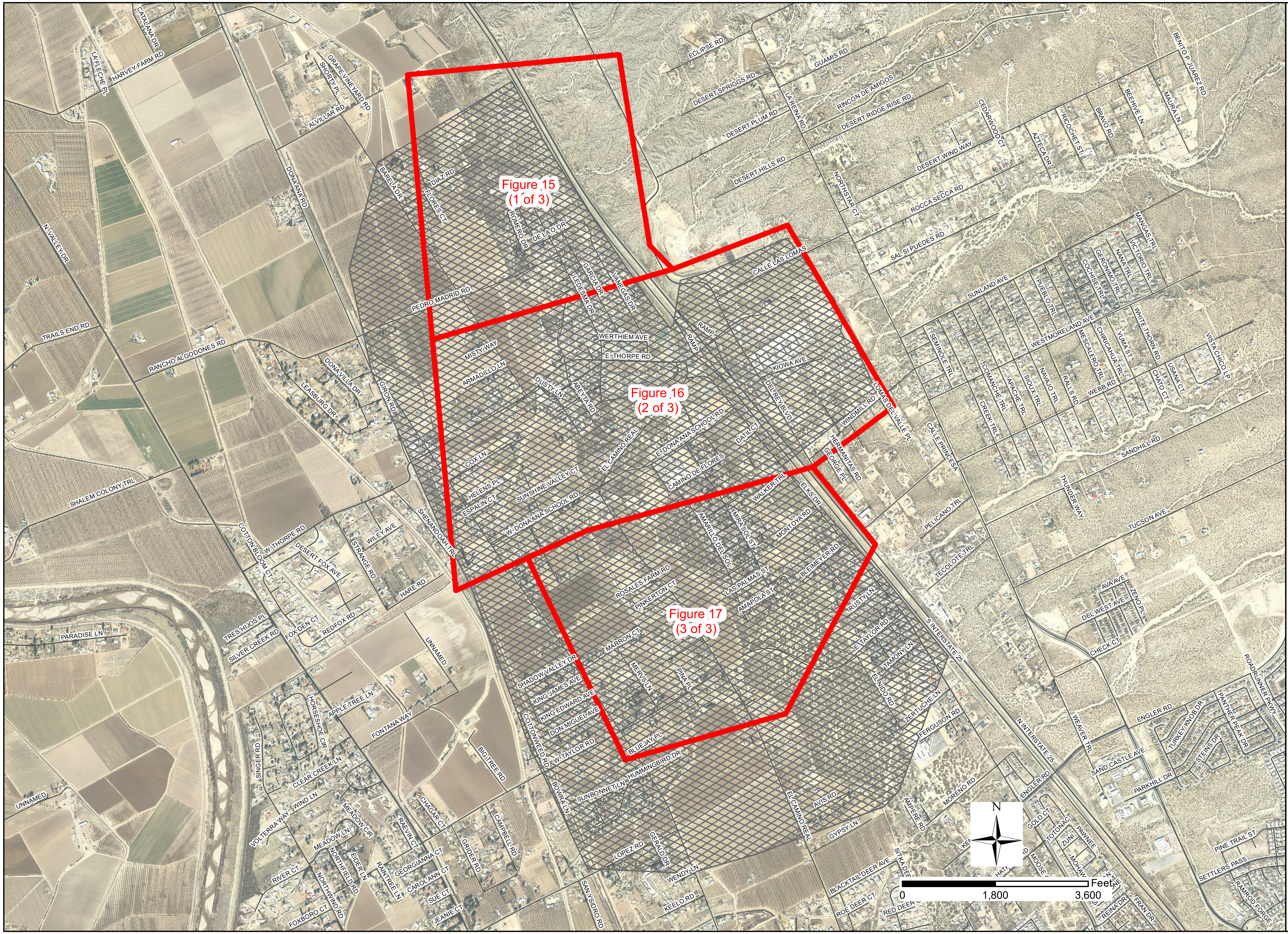
The next check was to view the computational log file which is accessed through the Options tab in the Unsteady Flow Analysis window. This does a volume continuity check for the simulation. The key number here is the percent error during the run shown in the red box shown below. This number should be very small if the model is running correctly. The log should look like below:

Volume Accounting for 2D Flow Area in Acre Feet

2D Area	Starting Vol	Ending Vol	Cum Inflow	Cum Outflow	Error	Percent Error
*******	************	**********	**********	***********	*****	************
DAC		83.85	685.5	602.8	1.110	0.1619

In case of this unsteady state simulation the percent error was 0.1619 percent which is very low and well within the acceptable range for full momentum analysis. Please refer to the HEC-RAS 2D User Manual by USACE, February 2016 for further reference.

Figure 15 thru Figure 17 (pages 35-37) show the limits of inundation during the 100-yr 24-hr storm at the peak of the storm. The inundation limits for the 10-yr 24-hr storm are included in **Appendix F**. As expected, Culvert C2 can convey most of the flows from the emergency spillway under I-25 and into channel CH5. The small overflow that gets to the west side of I-25 on Thorpe also flows into channel CH5. The downstream side of the CBCs tie into 2 X 60 in. RCP pipes that conveys the flows into channel CH4. This 2D model does not consider factors like back watering from the downstream culverts or blockages that may impede flow upstream of the CBC's. Therefore, allowing some flow to split north through the underpass is a conservative assumption.



Legend

- Key Map
- 2D Flow Area
- Roads

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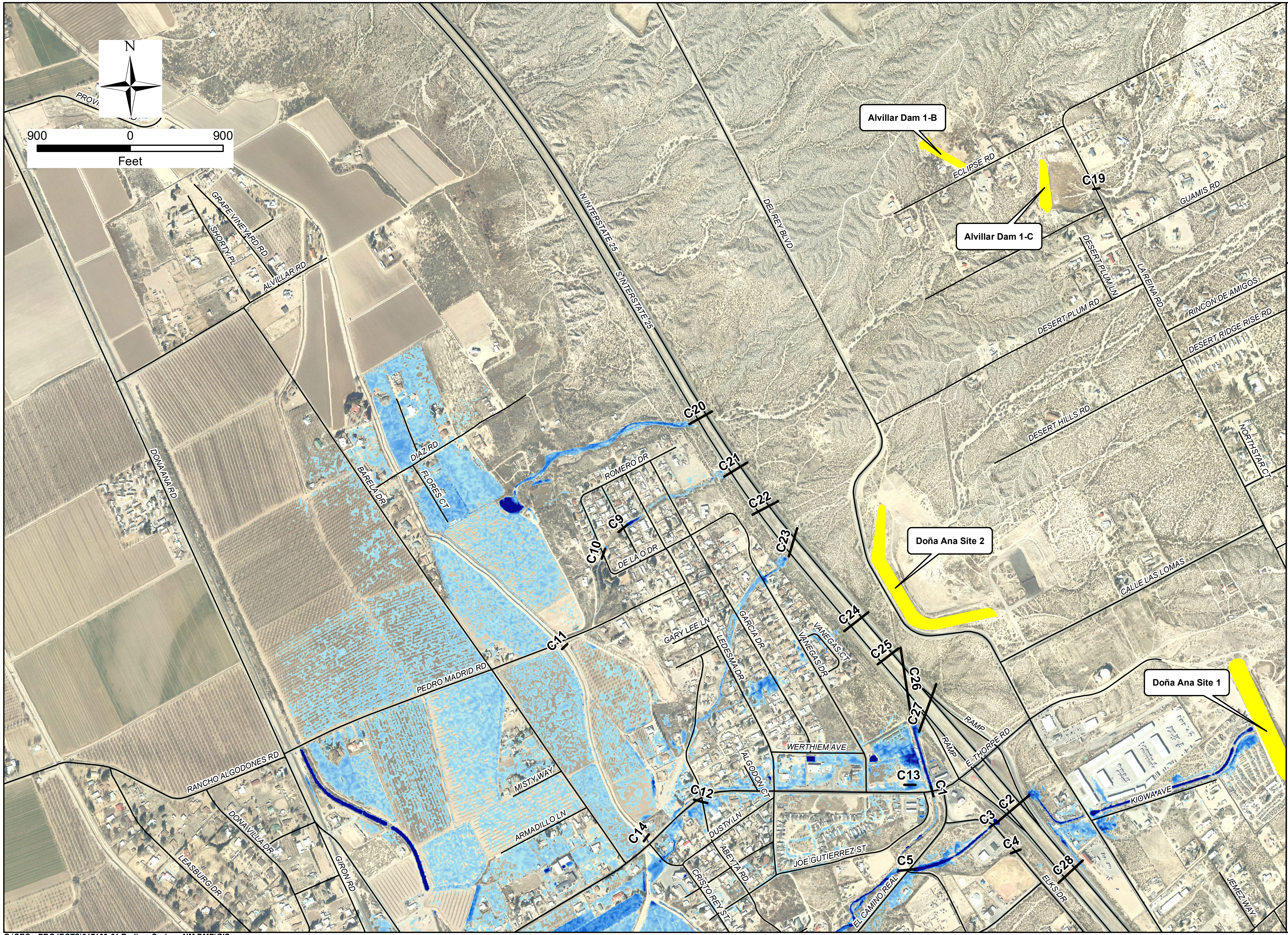
Figure 14
Limits of 2D Model For 10yr & 100yr-24hr Storm Events

2020



Legend

- Existing Culverts
- Existing Dams
- Roads

Depth

- High : 13.0 ft
- Low : 0.5 ft

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Figure 15 Inundation Limits of the 100yr-24hr Storm (1 of 3)

2019

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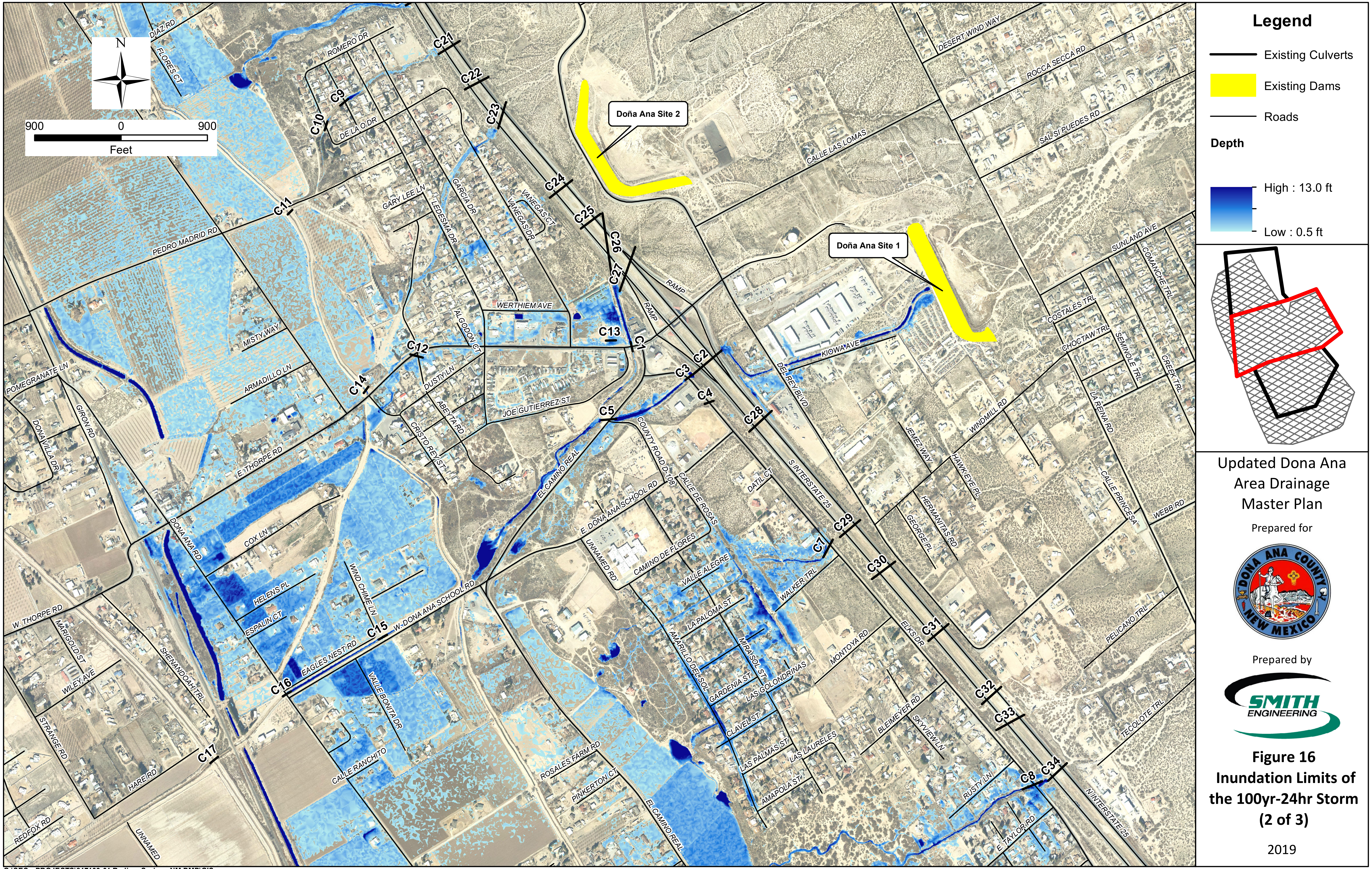

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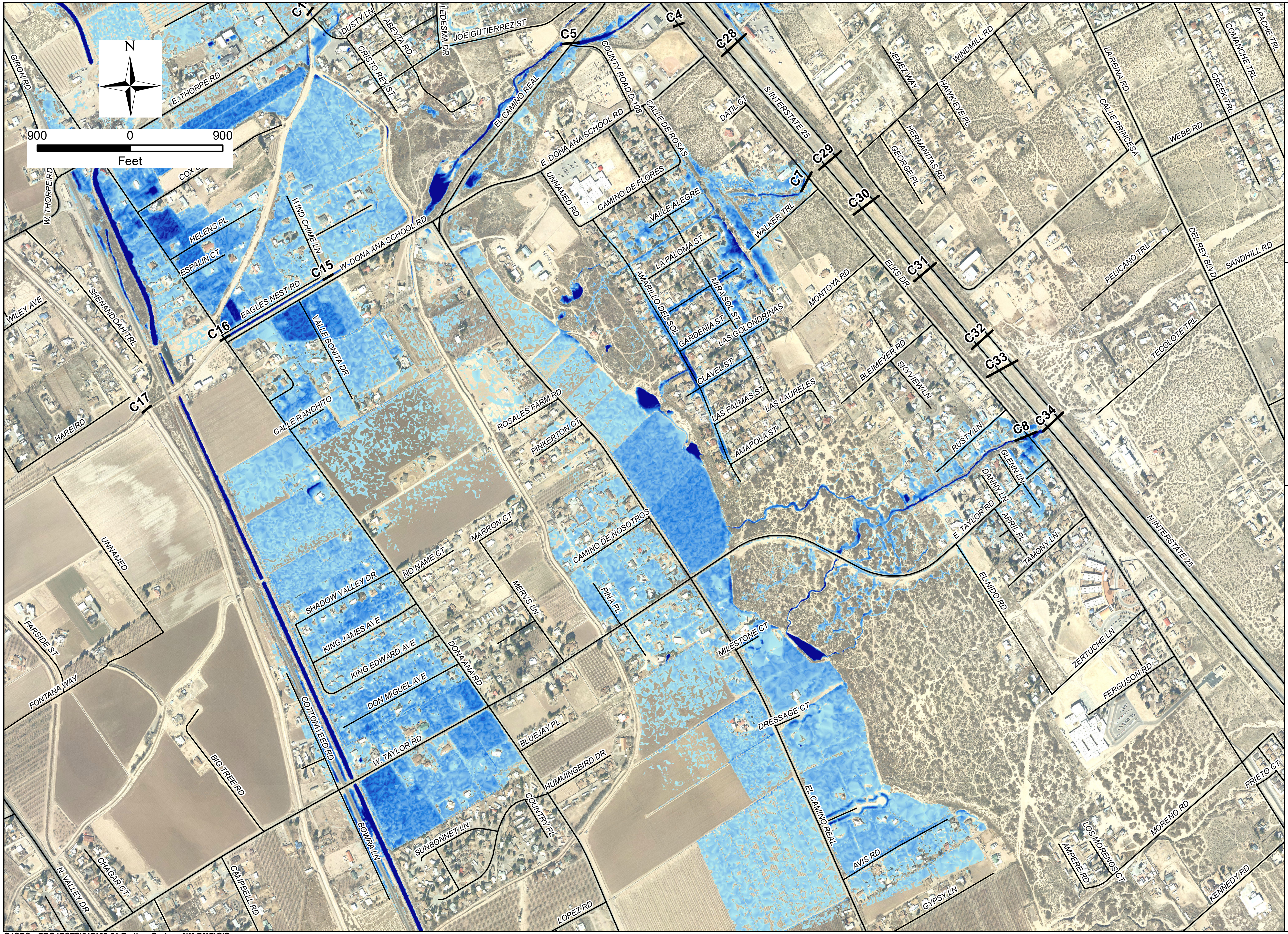

Legend

Existing Culverts

Existing Dams

Roads

Depth

High : 13.0 ft

Low : 0.5 ft

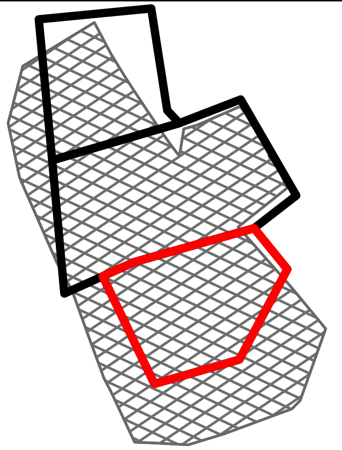

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Figure 17
Inundation Limits of the 100yr-24hr Storm (3 of 3)

2019

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Downstream of Culvert C20 at the very north end of town, the flows are contained within the limits of the arroyo. Channel CH1 also demonstrates that the flows are contained within the limits of the channel if culverts and the primary flow areas are maintained. The same holds true for Channel CH2. Culvert capacities for these locations are discussed and summarized in **Table G2** in **Appendix G.** Channel CH 15 is unique in the sense that it drains west of Vanegas Dr. and then disappears within one of the properties. It may be beneficial to either cutoff the flows from junction 200 on the east side of I-25 or to divert the flows either north or south using a shallow swale.

Culvert C1 east of the intersection of Thorpe Rd. and Joe Gutierrez St. show lack of capacity to convey the 100-yr and 50-yr flows. The overflow from this channel will flow down Thorpe Rd.

Junction 17, at channel CH8 spills and spreads out around Walker Ln. These flows eventually spill over and enter the subdivision downstream. The walls that divide the mobile home park east of the subdivision have several weirs notched into them. There also appears to be a drainage way at the cul-de-sac of Verbania St. at the intersection of Verbania St. and Mirasol Dr. Most of these features are on private property so thorough field investigation of where the overland flows would spill was not possible during field work.

Culverts C8 just north of Taylor Rd and Elks Dr. discharge flows into channel CH13. The 2D model does show flows spreading through the neighborhood at the culvert. A typical cross section of CH13 was completed and the flowmaster results indicate CH13 can handle the 100-yr flows. However, Culvert C8 is undersized to fully convey the 100-yr event to the CH13, which results in ponding and the flow not being fully contained in the channel.

Typical flood depths range from 1-1.5 ft through these neighborhoods with depths increasing to 2.5 ft. at the western end of the neighborhoods which is the bottom of the subdivision.

The following conclusions were developed from the results of the 2D models.

- The channels intertwined within the residential areas north of Thorpe Rd demonstrate sufficient capacity to convey the 100-yr 24-hr storm runoff. The limiting factor for these channels will be culvert capacity and channel maintenance.

- The north and south dams provide a significant benefit for the community west of I-25 during the 100-yr 24-hr storm. Most of the flows from the emergency spillway from South Dam will be conveyed under I-25 via culvert C2. However, the model does not account for debris or blockages on the upstream culvert entrance or backwater conditions on the downstream side that could be created from the 2 X 60 in. culverts. In the worst-case scenario, if the CBCs fail, the flows would flow north through the underpass.

- I-25 does help to detain the flows in the absence of the dams and provide a certain measure of protection against the incoming peak flows.

- The concentration of flows that arrive at Walker Ln. create a significant flooding problem for the downstream subdivision.

To successfully simulate the HEC-RAS Model for the various simulations discussed above, please refer to the table below that describes the plans to use.

10-yr-24-hr Storm using Full Momentum Equation						
Project	Dona_Ana_Update					
Plan	Plan 05	Plan set up to run Full Momentum Equation				
Geometry File	DAC_2	Breaklines, Polygons, NLCD_2011				
Flow File	Q10_Rev	Updated Q's from Latest HMS Model				

100-yr-24-hr Storm using Full Momentum Equation		
Project	Dona_Ana_Update	
Plan	Plan 06	Plan set up to run Full Momentum Equation
Geometry	DAC_2	Breaklines, Polygons, NLCD_2011
Flow File	Q100_Rev	Updated Q's from Latest HMS Model

10-yr 24-hr Floodplains with Dams

The 10-yr 24-yr storm floodplain analysis (HEC-RAS 2D) model results that includes the existing dams east of I-25 are presented in **Appendix F.**

SECTION 4. PROPOSED FACILITIES AND COST ESTIMATES

4.1 PROPOSED ANALYSIS

The developed areas of Doña Ana are the most adversely affected by storm events. This is due to the lack of engineered facilities within the developed areas to handle stormwater runoff. There are several conveyance facilities not designed to handle any certain storm event, but only to help alleviate the affects to adjacent properties. The primary focus of the Proposed Facilities is to intercept stormwater runoff upstream of the developed areas and detain and reduce peak discharges and / or improve conveyance facilities such as channels and culverts. Specific problem locations are described with the proposed facility that will mitigate the flooding problem at that location.

Many of the hydrologic data and assumptions such as flow divides, etc. developed in the "Existing" HEC-HMS basin model were replicated in the HEC-HMS Proposed "Options" Models. The basic assumptions carried over from the "Existing" basin model are presented below:

A. Model computation time increment – 5 minutes

B. No additional Sub-Basins were created in the proposed options models

C. Runoff Curve Numbers and lag time values remain the same

D. The storm event (precipitation) models in the existing conditions model are the same events used to create the proposed options models

<u>Design Storm</u>

The conceptual designs were prepared to accommodate the 100-yr return period storm of 24-hr duration based on direction from the DACFC. **Appendix E** contains the hydrologic summary of all "Option" basin models simulated for the 5-yr, 10-yr, 50-yr and 100-yr 24-hr duration storms. All detention ponds were simulated to pass the 100-yr 24-hr storm through the emergency spillways without overtopping the pond embankment.

<u>Proposed Facility</u>

The HEC-HMS models are comprised of various Facilities. Each proposed Facility is a drainage improvement that can be completed on its own and will help improve the capture and conveyance of runoff in a more controlled manner. Although the facilities will be able to work on their own, if implemented in conjunction with each other the entire drainage system is affected. To help understand what the proposed facilities are Section 4.2 discusses each facility in detail.

<u>Proposed Option HEC-HMS Models</u>

There are 5 proposed HEC HMS models that were completed for this project. The models were named as Options 1 through 5 and contain different combinations of the proposed facilities mentioned above. An overview of the proposed Facilities is shown on **Figure 18**. The five HEC-HMS "Option" basin models are described in further detail in Section 4.3.

4.2 PROPOSED FACILITIES

A Facility is a proposed improvement. Each Facility contains Sub-Facilities, which are drainage structures or actions required to develop the complete Facility. The Facility Number naming convention is such that Facility 1 represents the main Facility Number. The various Sub-Facilities are represented as decimal numbers after the main facility number.

For example:

Facility 1 - Proposed Pond 1 (Main Facility)

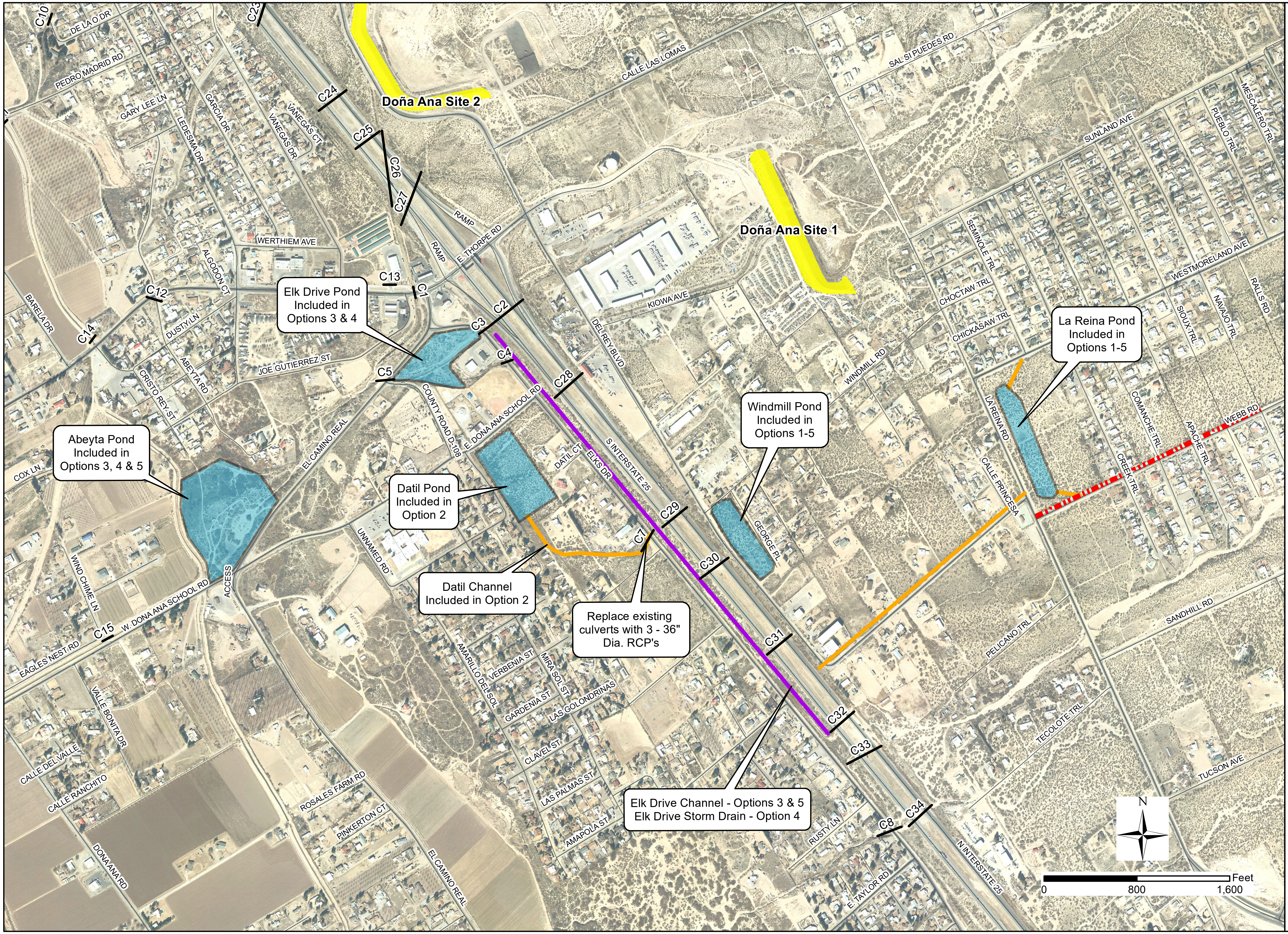
Facility 1.1 - Land Acquisition (Sub-Facility)

Facility 1.2 - Pond Construction (Sub-Facility)

The development of distinct Sub-Facilities will allow for planning, funding, and construction of smaller projects.

Conceptual level grading plans have been developed for each proposed detention pond. Note that the initial grading plans have been designed to achieve the maximum storage volumes within the given site based on anticipated right-of-way acquisition and adjacent elevation constraints. The pond routing results indicate that some ponds may be designed smaller than shown on the conceptual grading plans in this DMP. If adopted for final design, these ponds will require refinements that may result in costs savings.

A summary table of the conceptual level Engineer's Opinion of Probable Costs (EOPC) is located below. The detailed EOPC for each Facility is in **Appendix H**. The total cost will include a construction costs, contingency costs, bid allowances and the current 2020 New Mexico Gross Receipts Tax for Doña Ana County. Construction Phase Services is not included.



Legend

- Proposed Ponds
- Existing Dams
- Proposed Elk Channel and Storm Drain
- Proposed Webb Road Curb and Gutter
- Proposed Channels
- Existing Culverts
- Roads

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Figure 18
Overview of Proposed Options

2020

Facility 1 La Reina Pond

Facility Description

La Reina Pond is located just north of Del Rey Blvd between Westmoreland Avenue and Webb Road. The area just downstream of the proposed pond has documented flooding. The purpose of this Facility is to detain the upstream flows and convey the flows in a controlled manner to I-25. Which will help alleviate flooding in the surrounding residential areas.

La Reina Pond is comprised of a detention pond, curb and gutter, and a channel. The Facility is in the 400 Drainage Sub-basin Series. An overview of the Facility is shown on **Figure 19.** The pond was designed with a total volume of 11.2 ac-ft and a maximum depth of 7 ft. Flows from Westmoreland Avenue and Webb Road are directed into the proposed detention pond through rundown structures. La Reina Pond outlets into a proposed channel system that conveys the water through Culvert C31 to the westside of I-25.

Contributing Sub-basins

Sub-basins 402.1, 407.1, and 410 contribute flows to the proposed detention pond for a total drainage area of 0.1992 sq. mi. The design of the pond was done assuming 100% of these sub-basins are conveyed to the pond either naturally (through existing non-engineered channels) or through the proposed Sub-Facilities listed below. The peak inflow into the pond is 143 cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

La Reina Pond was designed to handle the 100-yr 24-hr storm event with minimal to no use of the emergency spillway. The pond has a minimum bottom slope of 0.5% and was designed with 4:1 side slopes.

The principal spillway is designed as a 6" reversed inclined ported riser with an 18" RCP principal spillway outlet.

The emergency spillway is designed to handle the 100-yr 24-hr storm event. It has a length of 35-ft and height of 1.5-ft. The emergency spillway is located to direct flows, if used, into the proposed channel, noted as Sub-Facility 1.3.

HEC-HMS Modeled Options

Each facility was modeled in various options. Facility 1 is the most upstream facility and will impact the flows to all other facilities. Facility 1 – La Reina Pond was modeled in the Option 1 -5 HEC-HMS models. A description of each option is included in Section 4.3 of this report.

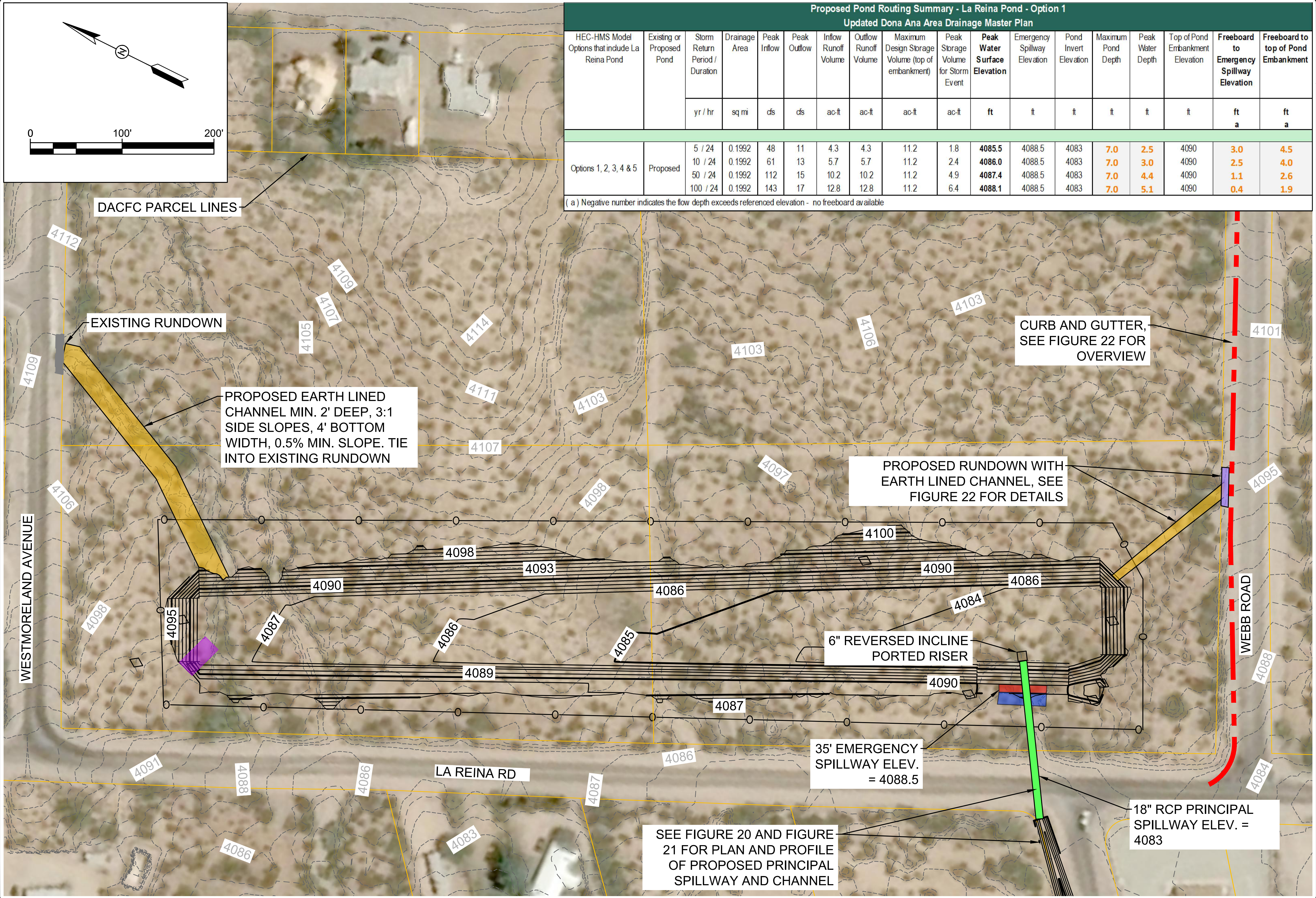
Pond Routing Results

The proposed pond routing summary table is located on **Figure 19.** During the 100-yr storm event the pond has 0.4-ft of freeboard to the emergency spillway. The peak outflow from La Reina pond during the 100-yr 24-hr storm event is 17 cfs.

Proposed Pond Routing Summary - La Reina Pond - Option 1
Updated Dona Ana Area Drainage Master Plan

HEC-HMS Model Options that include La Reina Pond	Existing or Proposed Pond	Storm Return Period / Duration	Drainage Area	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embankment)	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**
		yr / hr	sq mi	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft a**	**ft a**
Options 1, 2, 3, 4 & 5	Proposed	5 / 24	0.1992	48	11	4.3	4.3	11.2	1.8	**4085.5**	4088.5	4083	7.0	2.5	4090	3.0	4.5
		10 / 24	0.1992	61	13	5.7	5.7	11.2	2.4	**4086.0**	4088.5	4083	7.0	3.0	4090	2.5	4.0
		50 / 24	0.1992	112	15	10.2	10.2	11.2	4.9	**4087.4**	4088.5	4083	7.0	4.4	4090	1.1	2.6
		100 / 24	0.1992	143	17	12.8	12.8	11.2	6.4	**4088.1**	4088.5	4083	7.0	5.1	4090	0.4	1.9

(a) Negative number indicates the flow depth exceeds referenced elevation - no freeboard available



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LEGEND:

- CHAIN LINK FENCE
- PARCEL LINES
- PROPOSED CURB AND GUTTER
- SOIL CEMENT PAD
- PROPOSED EMERGENCY SPILLWAY
- PROPOSED CHANNELS
- EXISTING RUNDOWN
- PROPOSED RUNDOWN
- PROPOSED OUTLET
- MAINTENANCE ROAD

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 19

PROPOSED LA REINA POND OVERVIEW

PROJECT NO:
817103-04

DATE:
2020

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Sub-Facilities

Sub-Facility 1.1 – Purchase property for La Reina Pond and appropriate drainage easements.

Sub-facility 1.2 – Build La Reina Pond including the principal spillway, outfall pipe, emergency spillway, erosion control and fence. Build the proposed rundown structure on Webb Road. Build the proposed earth lined channels tying each rundown structure into La Reina Pond. This is shown on above on **Figure 19.**

Sub-Facility 1.3 – Build La Reina Channel and driveway culverts, as shown on **Figure 20 and Figure 21**. The typical cross section for the channel is located on **Figure 20**. The 100-yr peak outflow from the La Reina pond, was used to design the La Reina channel and the corresponding driveway culverts. The channel is modeled with 2:1 side slopes, 3-ft bottom width and a minimum depth of 1-ft. The culverts were place at each driveway. The outlet of the channel is the corner where Del Rey turns to run parallel with the Interstate. At this location the water will flow into an existing channel which then conveys water to Culvert C31.

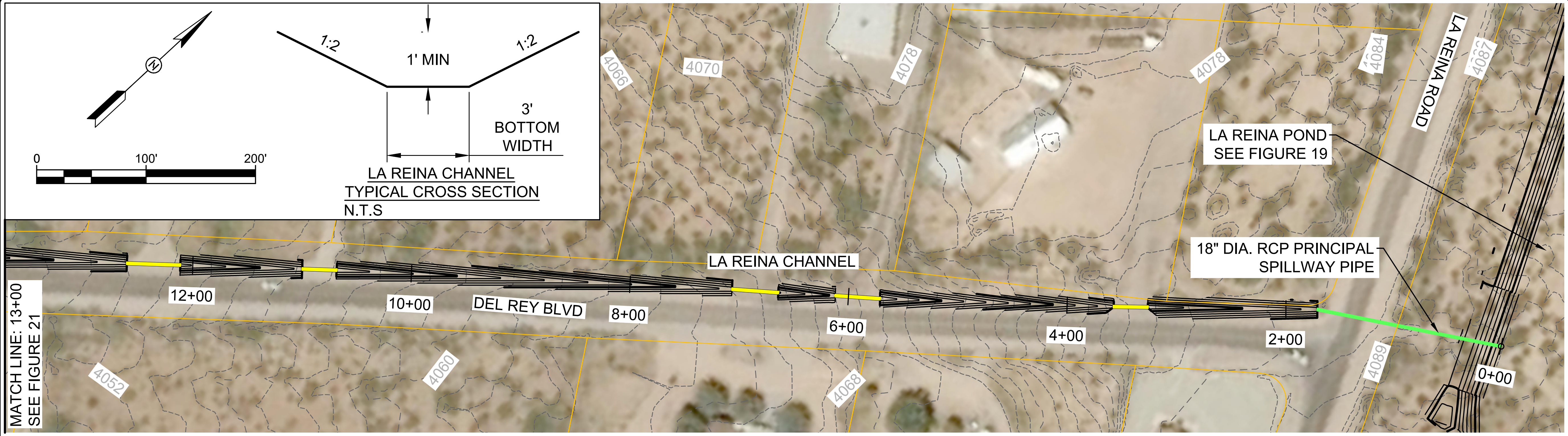
Sub-Facility 1.4 – Build the proposed curb and gutter along Webb Road. The curb and gutter will need to tie into the proposed rundown structure, Sub-Facility 1.2. This is shown on **Figure 22**.

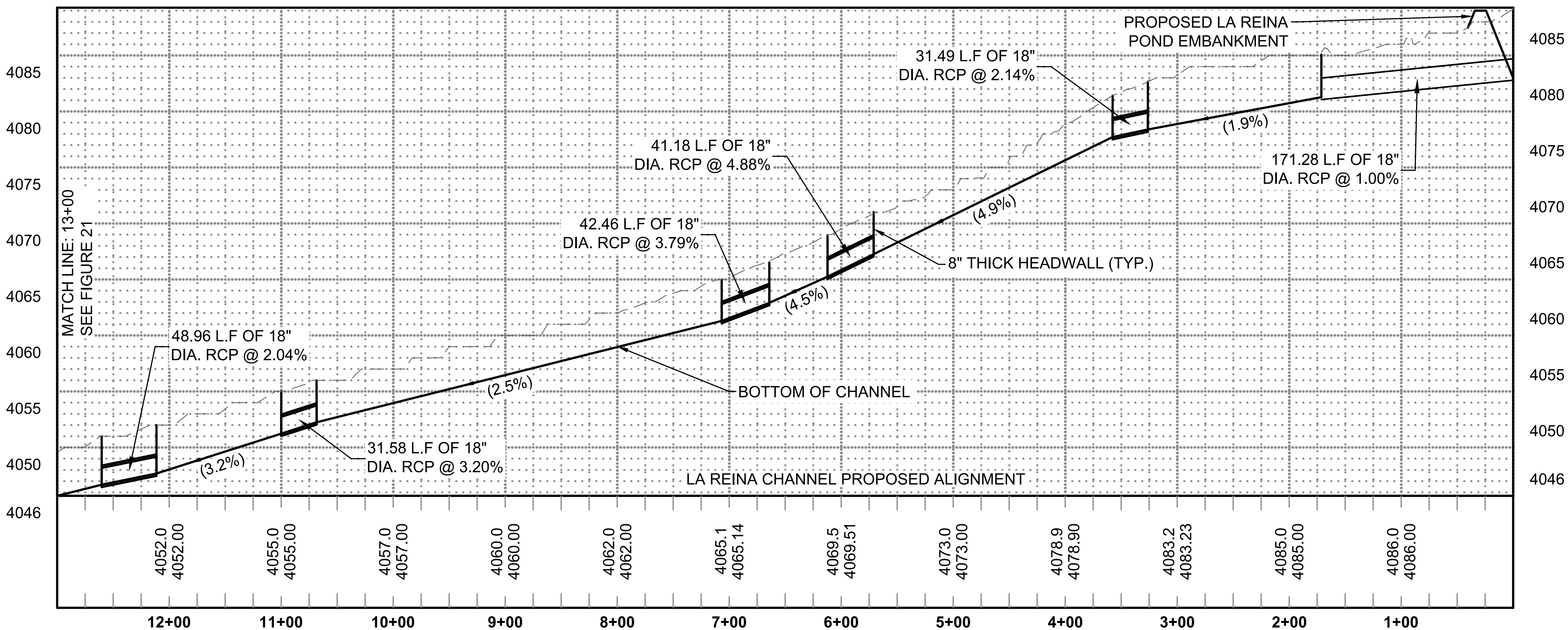
Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. The land acquisition cost assumes $2,500 per acre. Other costs were derived from recent City of Albuquerque and NMDOT bids. The total cost for Facility 1 is $708,000.

Table 5: Summary of Engineer's Opinion of Probable Cost for Facility 1

Sub Facility	Description	Cost
1.1	Property Acquisition	$ 500,000
1.2	La Reina Pond, Proposed Rundown, Earth Lined Channels	$ 446,000
1.3	La Reina Driveway Culverts	$ 143,000
1.4	Webb Road Curb and Gutter	$ 94,000
	Total Cost for Facility 1	**$ 1,183,000**





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LEGEND:

PARCEL LINES

PROPOSED OUTLET

PROPOSED CULVERT

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 20

LA REINA CHANNEL
STA. 0+00-13+00

PROJECT NO:
817103-04

DATE:
2020

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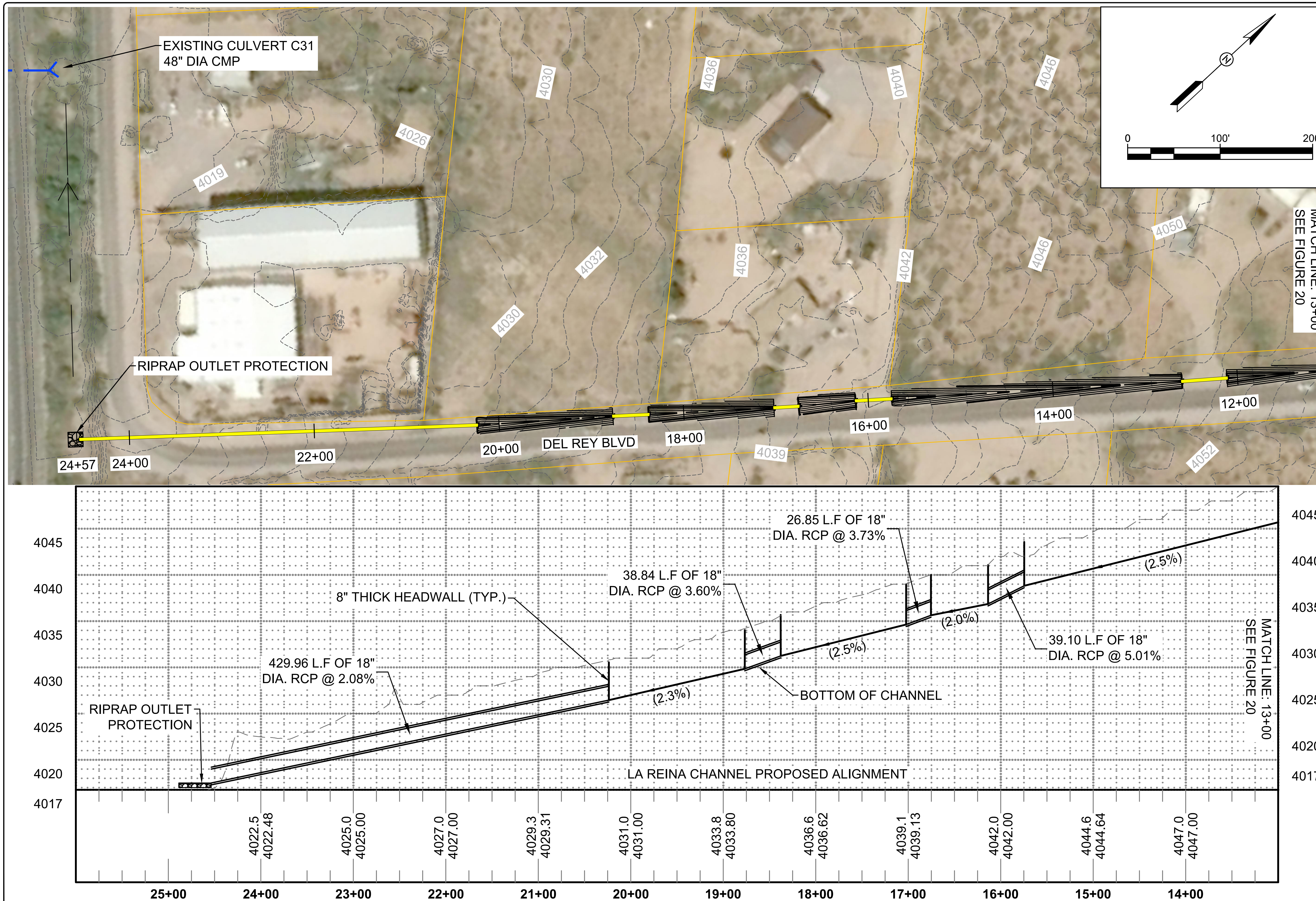

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LEGEND:

PARCEL LINES

PROPOSED CULVERT

FLOW ARROWS

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 21

LA REINA CHANNEL
STA. 13+00-24+57

PROJECT NO:
817103-04

DATE:
2020

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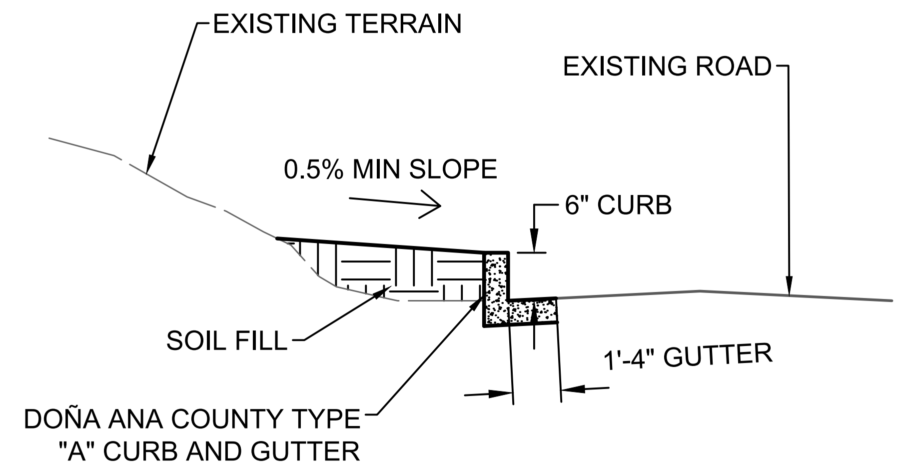
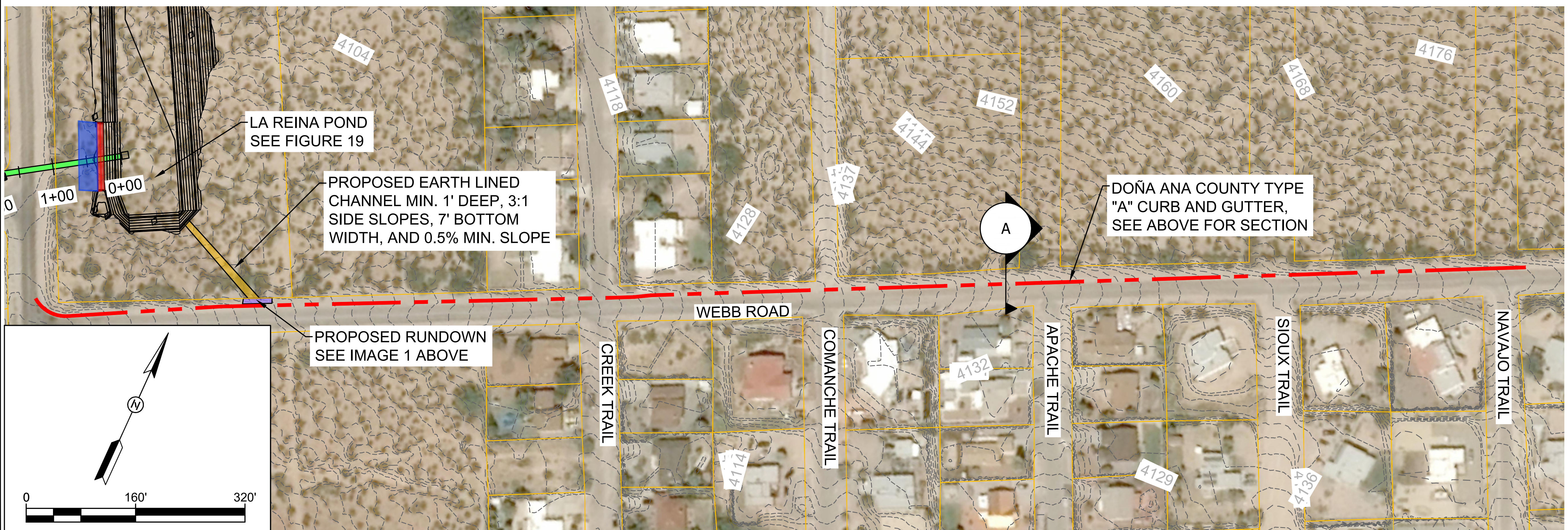


(A) TYPICAL CURB AND GUTTER
N.T.S

IMAGE 1: THE EXISTING RUNDOWN, ABOVE, IS LOCATED ON WESTMORE AVENUE. THE PROPOSED RUNDOWN, BELOW, HAS SIMILAR DESIGN FEATURES AND IS LOCATED ON WEBB ROAD. SEE BELOW FOR RUNDOWN LIMITS.



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LEGEND:
PARCEL LINES
PROPOSED CURB AND GUTTER
SOIL CEMENT PAD
PROPOSED EMERGENCY SPILLWAY
PROPOSED CHANNEL
PROPOSED RUNDOWN
PROPOSED OUTLET

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 22

PROPOSED WEBB CHANNEL WITH ROAD CURB AND GUTTER OVERVIEW

PROJECT NO:
817103-04

DATE:
2020

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Facility 2 Windmill Pond

Facility Description

The proposed Windmill Pond is located on the southeast corner of the Del Rey Blvd. and Windmill Road intersection. The primary purpose of this pond is to act as a sediment pond. Capturing sediment upstream will help improve and reduce maintenance cost of the downstream facilities.

Figure 23 shows an overview of the proposed Facility along with its proximity to the other Facilities.

Contributing Sub-basins

Sub-basins 401, 402.2 and 403 contribute flows to the proposed detention pond for a total drainage area of 0.1193 sq. mi. The design of the pond was done assuming 100% of these sub-basins are conveyed to the pond either naturally (through existing natural channels) or through the proposed Sub-Facilities listed below. The peak inflow into the pond is 83 cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

Windmill Pond was designed to handle the 100-yr 24-hr storm event with minimal to no use of the emergency spillway. The Pond has a minimum bottom slope of 0.5% and was designed with 4:1 side slopes.

The principal spillway is designed as a 24" RCP discharge pipe.

The emergency spillway is designed to handle the 100-yr 24-hr storm event. It has a length of 20-ft and height of 1.5-ft. The emergency spillway is located to direct flows, if used, into the existing Culvert C30 that runs underneath I-25.

HEC-HMS Modeled Options

Each facility was modeled in various options. This Facility is modeled in the Options 1 thru 5 HEC-HMS models. A description of each option is in Section 4.3 of this report.

Pond Routing Results

The proposed pond routing summary table is located on **Figure 23**. During the 100-yr storm event the pond has 1.7-ft of freeboard to the emergency spillway and a peak outflow of 15 cfs.

Sub-Facilities

Sub-Facility 2.1 – Purchase property for Windmill Pond and appropriate drainage easements.

Sub-facility 2.2 – Build Windmill Pond including the principal spillway, emergency spillway, erosion control and fence. This is shown in **Figure 23**.

Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. The land acquisition cost assumes $2,500 per acre. Other costs were derived from recent City of Albuquerque and NMDOT bids. The total cost for Facility 2 is $265,000.

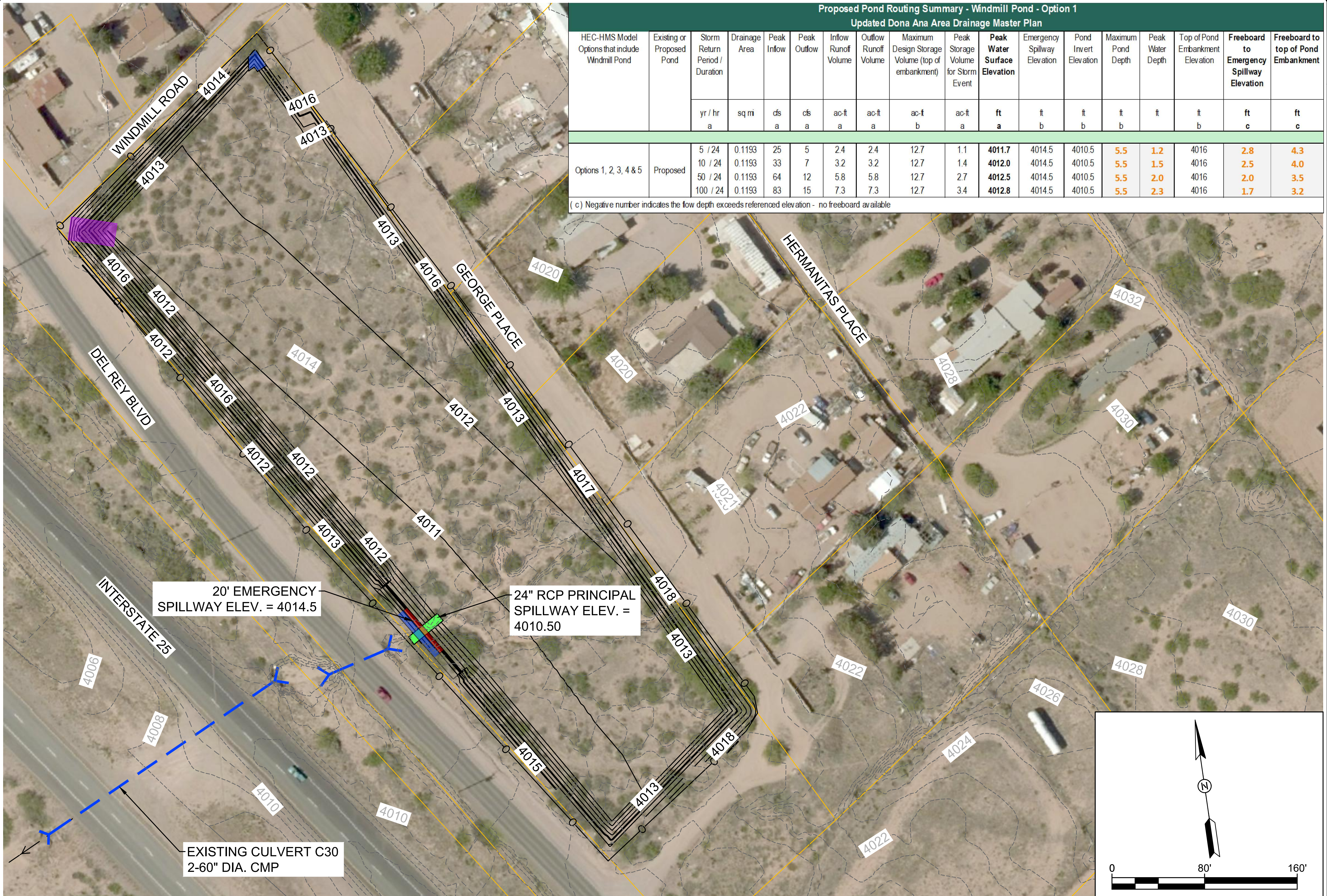
Table 6: Summary of Engineer's Opinion of Probable Cost for Facility 2

Sub Facility	Description	Cost
2.1	Property Acquisition	$ 200,000
2.2	Windmill Pond	$ 255,000
	Total Cost for Facility 2	**$ 455,000**

Proposed Pond Routing Summary - Windmill Pond - Option 1
Updated Dona Ana Area Drainage Master Plan

HEC-HMS Model Options that include Windmill Pond	Existing or Proposed Pond	Storm Return Period / Duration	Drainage Area	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embankment)	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**
		yr / hr a	sq mi	cfs a	cfs a	ac-ft a	ac-ft a	ac-ft b	ac-ft a	**ft a**	ft b	ft b	ft b	ft	ft b	**ft c**	**ft c**
Options 1, 2, 3, 4 & 5	Proposed	5 / 24	0.1193	25	5	2.4	2.4	12.7	1.1	**4011.7**	4014.5	4010.5	5.5	1.2	4016	2.8	4.3
		10 / 24	0.1193	33	7	3.2	3.2	12.7	1.4	**4012.0**	4014.5	4010.5	5.5	1.5	4016	2.5	4.0
		50 / 24	0.1193	64	12	5.8	5.8	12.7	2.7	**4012.5**	4014.5	4010.5	5.5	2.0	4016	2.0	3.5
		100 / 24	0.1193	83	15	7.3	7.3	12.7	3.4	**4012.8**	4014.5	4010.5	5.5	2.3	4016	1.7	3.2

(c) Negative number indicates the flow depth exceeds referenced elevation - no freeboard available



SMITH ENGINEERING

LEGEND:

CHAIN LINK FENCE

PARCEL LINES

FLOW ARROW

EXISTING CULVERT

SOIL CEMENT PAD

PROPOSED EMERGENCY SPILLWAY

PROPOSED OUTLET

MAINTENANCE ROAD

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 23

PROPOSED WINDMILL POND OVERVIEW

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Facility 3 Datil Pond

Facility Description

The proposed Datil Pond is located along Doña Ana School Road between Elks Road and Calle De Rosas Road. Currently, there is an existing channel that takes flows from Culvert C7 to the proposed pond site. There is no pond in this area. However, flows can enter an existing channel that runs along the residential homes, parallel to Calle De Rosas Road. The existing channel then conveys the flows to run parallel with Camino De Flores. The purpose of the pond is to help detain the flows from the upper watershed and have a more controlled outflow from the pond that will then utilize the existing channel system.

Facility 3 is comprised of a detention pond, proposed channels, culvert improvements, and maintenance to the existing channel. The Facility is located in the West of I-25 Drainage Sub-basin Series. An Overview of the Facility is shown on **Figure 24**. The pond was designed with a total volume 19.6 ac-ft and a maximum depth of 6 feet.

Contributing Sub-basins

The entire 400 Drainage Sub-basin Series contributes to Datil Pond through Culverts C32, C31, C30 and C29. Along with the 400 series, sub-basins 18, 19, 20, 21 and 9C contribute to the Datil Pond inflow volume. The total drainage area contributing to Datil Pond is 0.63-sq. mi. The design of the pond was done assuming 100% of the above-mentioned sub-basins are contributing to the pond either naturally (through existing natural channels) or through the proposed Sub-Facilities listed below. The peak inflow into the pond is 202-cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

Datil Pond was designed to handle the 100-yr 24-hr storm event with minimal to no use of the emergency spillway. The Pond has a minimum bottom slope of 0.5% and was designed with 3:1 side slopes.

The principal spillway is designed as a 36” RCP discharge pipe.

The emergency spillway is designed to handle the 100-yr 24-hr storm event. It has a length of 80-ft and height of 1.0-ft. The emergency spillway is located to direct flows, if used) into the existing channel that runs parallel with the residential community. This channel noted as Sub-Facility 3.5 will need some minor improvements and maintenance to ensure the channel properly conveys the flow from Datil Pond.

HEC-HMS Modeled Options

Each facility was modeled in various options. This Facility was modeled in the Option 2 HEC-HMS model. A description of each option is located in Section 4.3 of this report.

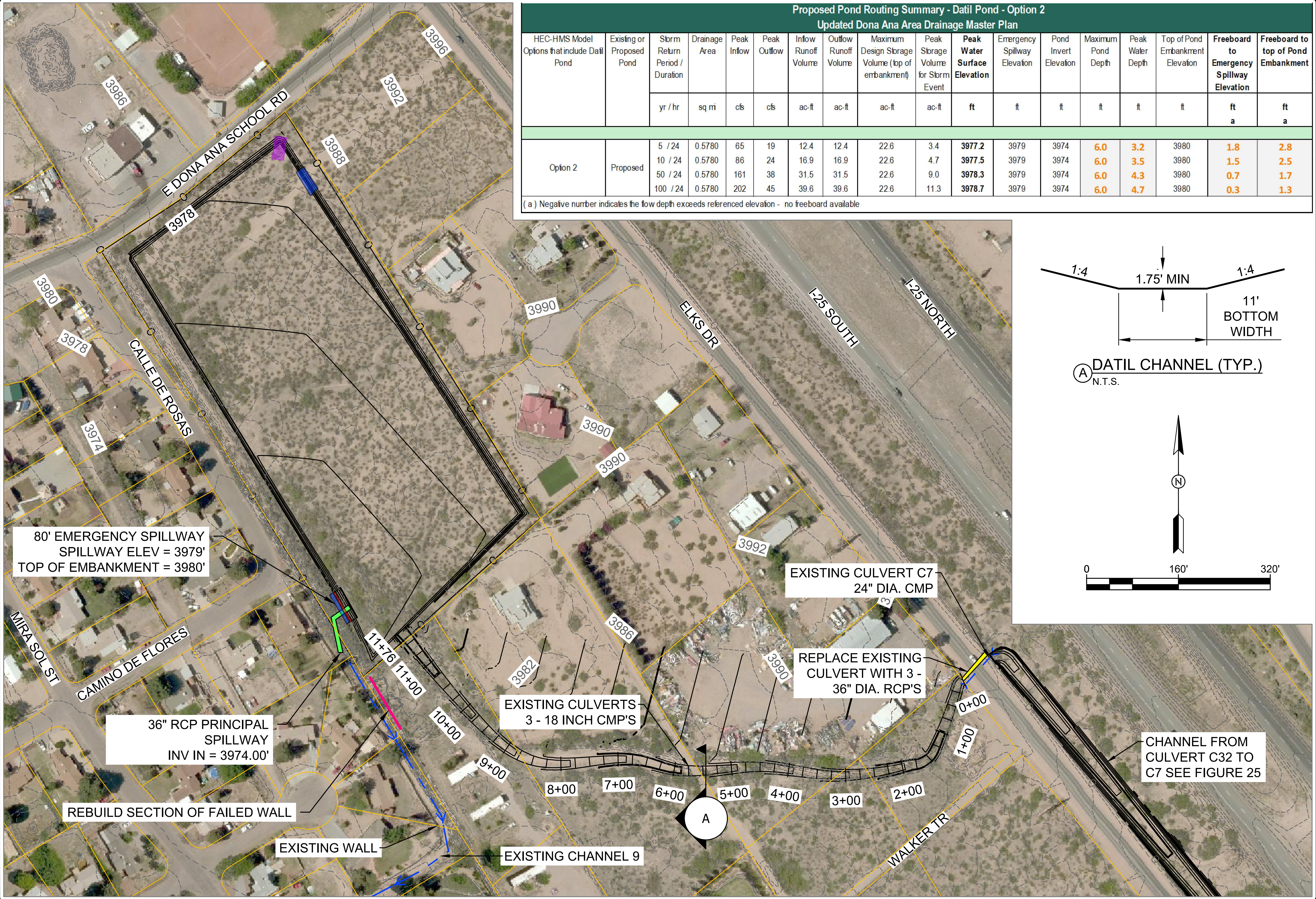
Pond Routing Results

The proposed pond routing summary table is located on **Figure 24**. During the 100-yr storm event the pond has 0.6-ft of freeboard to the emergency spillway. The peak outflow from Datil pond during the 100-yr 24-hr storm event is 39-cfs.

Proposed Pond Routing Summary - Datil Pond - Option 2
Updated Dona Ana Area Drainage Master Plan

HEC-HMS Model Options that include Datil Pond	Existing or Proposed Pond	Storm Return Period / Duration	Drainage Area	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embankment)	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**
		yr / hr	sq mi	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	**ft**	ft	ft	ft	ft	ft	**ft a**	**ft a**
Option 2	Proposed	5 / 24	0.5780	65	19	12.4	12.4	22.6	3.4	**3977.2**	3979	3974	6.0	3.2	3980	1.8	2.8
		10 / 24	0.5780	86	24	16.9	16.9	22.6	4.7	**3977.5**	3979	3974	6.0	3.5	3980	1.5	2.5
		50 / 24	0.5780	161	38	31.5	31.5	22.6	9.0	**3978.3**	3979	3974	6.0	4.3	3980	0.7	1.7
		100 / 24	0.5780	202	45	39.6	39.6	22.6	11.3	**3978.7**	3979	3974	6.0	4.7	3980	0.3	1.3

(a) Negative number indicates the flow depth exceeds referenced elevation - no freeboard available



SMITH ENGINEERING

LEGEND:

- CHAIN LINK FENCE
- PARCEL LINES
- EXISTING FENCE
- REBUILD EXISTING WALL
- FLOW PATH
- EXISTING CULVERT
- SOIL CEMENT PAD
- PROPOSED EMERGENCY SPILLWAY
- PROPOSED OUTLET
- PROPOSED CULVERT
- MAINTENANCE ROAD

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 24

PROPOSED DATIL POND OVERVIEW

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Sub-Facilities

Sub-Facility 3.1 – Purchase property for Datil Pond and appropriate drainage easements.

Sub-facility 3.2 – Build Datil Pond including the principal spillway, emergency spillway, erosion control and fence. This is shown on the above **Figure 24**.

Sub-Facility 3.3 – Build the Datil Channel and replace/remove existing private culvert, as shown on **Figure 24**. There is currently a natural channel in this area. These improvements should be done to ensure the channel is properly designed to handle the 100-yr storm event. The typical cross section for the channel is located on **Figure 24**. The Datil Channel was designed to hold the 100-yr storm event and outlets into Datil Pond. The channel was modeled with 4:1 side slopes, 11-ft bottom width and minimum depth of 1.75-ft. In the proposed Datil Pond a resident has constructed "driveway" culvert crossing consisting of 3-18-in CMPS. The photo below, taken during a field visit, shows the culverts and the obstruction they are causing in the field. If the crossing is to remain, it is suggested that the existing culverts be replaced with 3-36-in RCPs and a headwall be constructed.

Sub-Facility 3.4 – Build Channel and replace culvert C7, as shown on **Figure 25 and Figure 26**. The typical cross section for the channel is located on **Figure 25**. The Channel was designed to hold the 100-yr storm event. The proposed channel ends shortly after Culvert C7. At this point a headwall structure will be necessary to ensure the flow is directed into the proposed culverts. The headwall design should be done so that the top of headwall is at least a half-foot below the existing channel. This will allow for the headwall to act as a weir. If overtopping were to occur (due to the culverts being plugged) the water would continue down to Culverts C3 and C4, instead of overtopping the road and inundating the residents located near Culvert C7. The culvert locations are shown on **Figure 24**. The channel was designed to have 4:1 side slopes and minimum depth of 2-ft. Culvert C7 should be replaced with 3-36-in RCPs. The proposed culverts were sized to handle the 100-yr storm event.

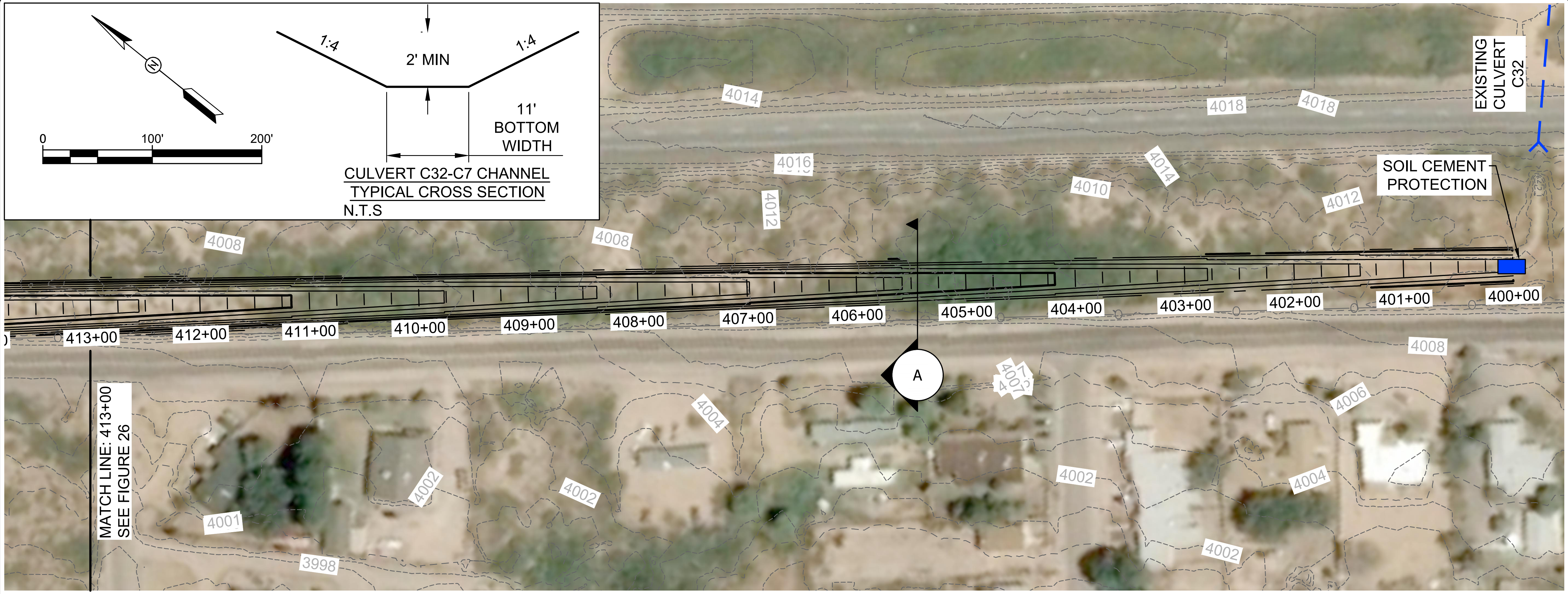
Sub-Facility 3.5 – Existing channel (Channel 9) maintenance needs to be done on the outlet channel, which runs parallel with Calle De Rosas. To ensure the channel has maximum capacity a portion of the wall located just south of the principle spillway needs to be rebuilt. The approximate location of the wall is shown on **Figure 24**.

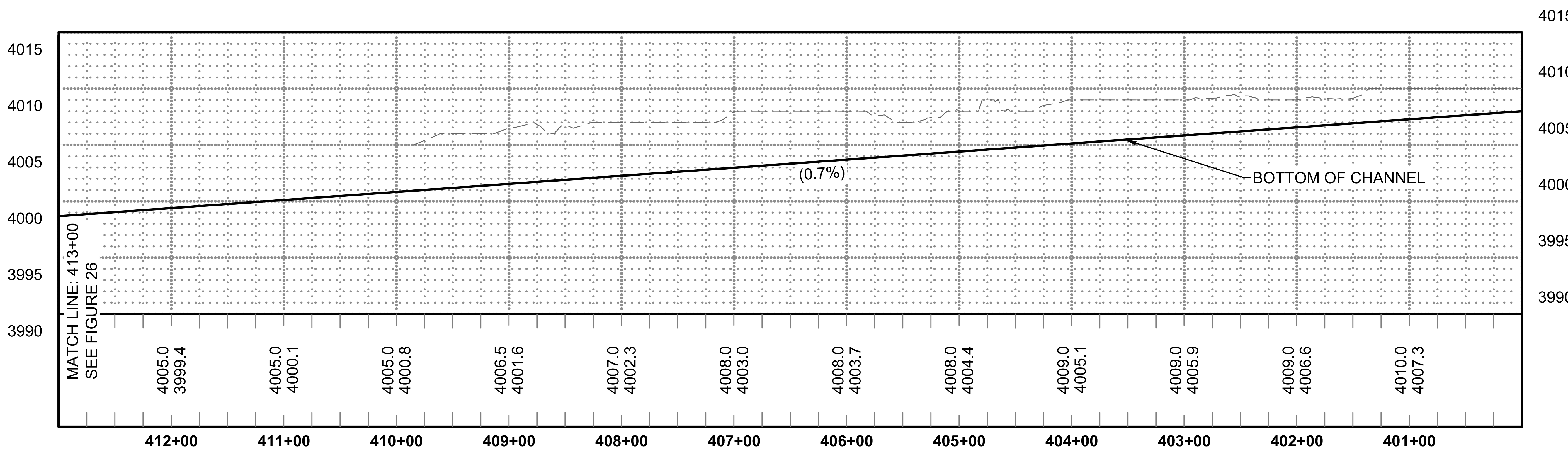
Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. The land acquisition cost assumes $2,500 per acre. Other costs were derived from recent City of Albuquerque and NMDOT bids. The total cost for Facility 3 is $1,056,000.

Table 7: Summary of Engineer's Opinion of Probable Cost for Facility 3

Sub Facility	Description	Cost
3.1	Property Acquisition	$ 570,000
3.2	Datil Pond	$ 830,000
3.3	Datil Channel, Remove/Replace Existing Private Culverts	$ 29,000
3.4	Replacement of Culvert C7 and Channel from Culvert C32 to C7	$ 172,000
3.5	Rebuild Existing Wall and Maintain Existing Channel	$ 20,000
	Total Cost for Facility 3	**$ 1,621,000**





SMITH ENGINEERING

LEGEND:

- EXISTING CULVERT
- EXISTING CHAIN LINK FENCE
- EXISTING PROFILE
- PROPOSED PROFILE
- RIPRAP PROTECTION

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 25

PROPOSED CHANNEL FROM CULVERT C32 TO C7
STA. 400+00-413+00

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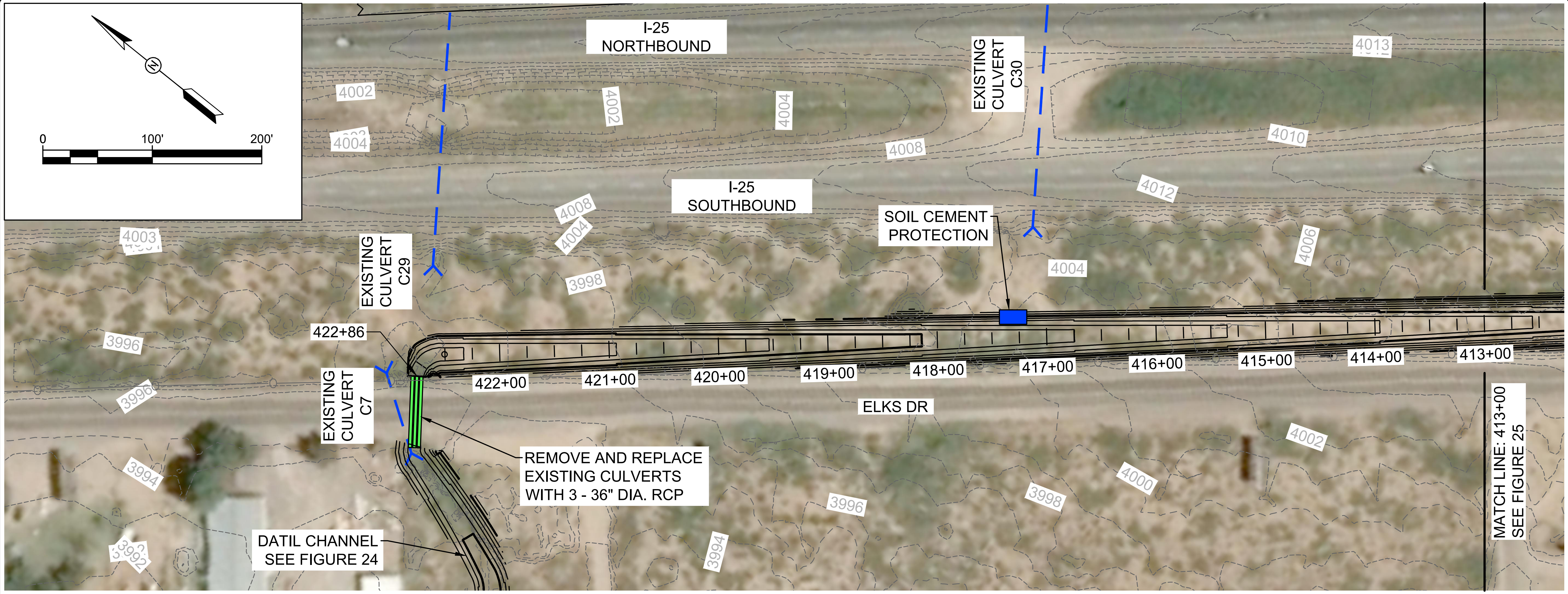

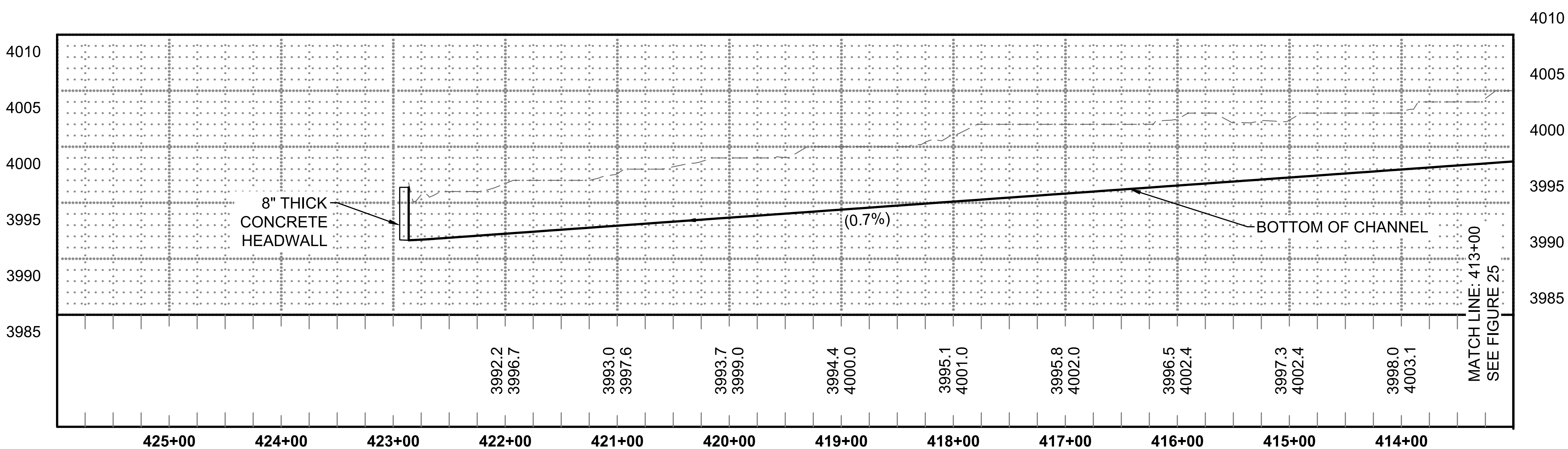

SMITH ENGINEERING

LEGEND:
- EXISTING CULVERT
- EXISTING CHAIN LINK FENCE
- EXISTING PROFILE
- PROPOSED PROFILE
- RIPRAP PROTECTION
- PROPOSED CULVERTS

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 26

PROPOSED CHANNEL FROM CULVERT C32 TO C7 STA 413+00-422+86.14

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Facility 4 Elk Drive Storm drain and Channel

Facility Description

Currently the flows from Culverts C32, C31, C30, C29, and C28 outlet into an existing natural channel that is not designed. The result is the water ponds in certain areas and overtops in other areas. Facility 4 consists of two alternatives to effectively convey the water downstream, reducing ponding and flooding along Elks Dr.

The first alternative is a proposed channel and storm drain system. For this alternative the channel starts at Culvert C32 and ends at Culvert C29. At this point a storm drain will convey the water from Culvert C29 to Culvert C3. Additionally, there is another proposed channel (**Figure 32**) that starts at Culvert C28 and flow to Culvert C3 working in parallel with the channel and storm drain. The second alternative consists of a channel that continues at Culvert C29 and ends at Culvert C3. The approximate location for Facility 4 is shown on **Figure 27 thru Figure 33.** The Sub-Facilities are designed to handle the 100-yr storm event and to stay within the existing right of way.

Contributing Sub-basins

The entire 400 Drainage Sub-basin Series contributes to the Facility through Culverts C32, C31, C30 and C29. Along with the 400 series, Sub-basins 18, 19, 20, 21 and 9C contribute to the Facility. The total drainage area contributing to Facility 4 is 7.14 sq. mi. The design of the storm drain and channel(s) was done assuming 100% of these sub-basins are conveyed to them. The peak inflow into Facility 4 is 342 cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

Elk Drive Channels were designed to handle the 100-yr 24-hr storm event. The slope of the channels varies with a minimum slope of 0.5%. The channel is modeled with 3:1 side slopes, a bottom width range of 2 to 4-ft and depth range of 1 to 1.5-ft. The storm drain alternative was designed using stormCAD and culvertmaster. The storm drain was modeled as a 54-in RCP with a minimum slope of 0.5% and a minimum 1-ft headcover.

The storm drain is designed to handle the 100-yr 24-hr storm event. The storm drain is modeled as a 54-in RCP, with a minimal slope of 0.5%, headcover of 1-ft and manholes set at a maximum of every 500 LF.

HEC-HMS Modeled Options

Each facility was modeled in various options. This channel alternative for Facility 4 is modeled in the Option 3 and 5 HEC-HMS models. The storm drain alternative for Facility 4 is modeled in the Option 4 HEC-HMS model. A description of each option is in Section 4.3 of this report.

Pond Routing Results

There is no pond for this Facility.

Sub-Facilities

Sub-Facility 4.1 – Purchase drainage easements for the proposed channels and storm drain, if necessary. The alignment of the storm drain and channel were placed between the road and the existing NMDOT ROW fence. This was done to minimize drainage easements and land acquisition.

Sub-Facility 4.2.a – This is the first alternative for Facility 4 and is shown on **Figure 27**. This Sub-Facility consists of on continuous channel from Culvert C32 to Culvert C3. **Figure 27** shows the proposed channel. The outlet of the Channel will vary depending on which downstream facility is chosen.

Sub-Facility 4.2.b – This Sub-Facility is shown on **Figure 27 thru Figure 31.** As mentioned, there are two alternatives for this Facility. Once the Channel has been built to Culvert C29, a decision needs to be made to either continue the Channel or to build a storm drain. Sub-Facility 4.2.b is the proposed storm drain alternative and is shown on **Figure 28 thru Figure 31**. Once the Channel ends at Culvert C29 the storm drain begins and conveys the flows downstream to the chosen proposed Facility. The proposed storm drain is a 54" RCP. The channel leading to the storm drain will need to be designed to allow for ponding and to direct flow towards the storm drain opening. A grate should be place on the storm drain to ensure no entrance can be made by the public and to block large debris from entering the storm drain system. The channel bottom should be a foot lower than the culvert invert to allow for sediment deposit. The outlet of the Channel will vary depending on which downstream Facility is chosen.

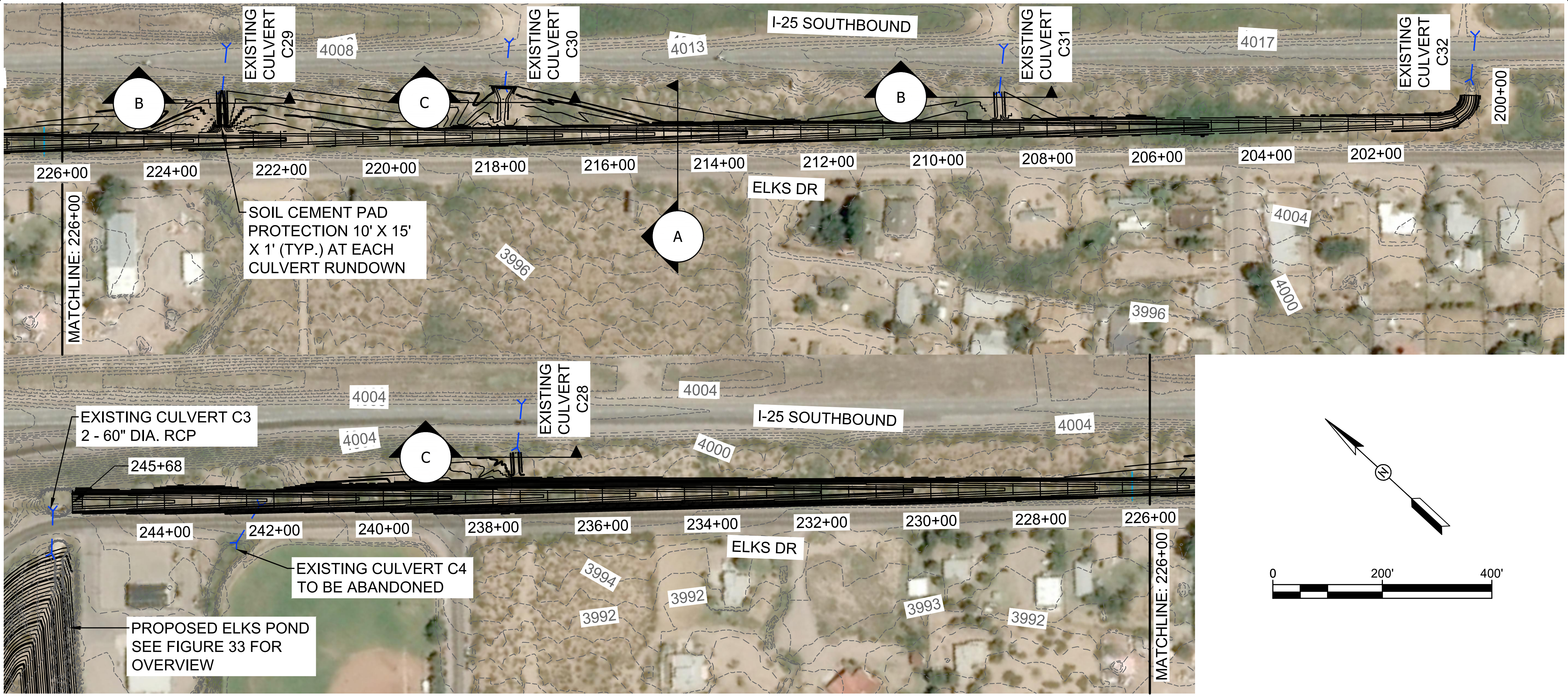
Sub-Facility 4.3 – This Sub-Facility, shown on **Figure 32,** would be used in conjunction with the storm drain system. Currently the flows from Culvert C28 flow to Culvert C4 naturally. It is suggested that an engineered channel be placed between Culvert C28 and C3. To ensure the flows from these culverts do not pond and are conveyed effectively downstream in a more controlled manner. The Channel was design with a minimum 1-ft depth, 0.5% slope, 3:1 side slopes, and a 5-ft bottom width. The outlet of the Channel will vary depending on which downstream Facility is chosen.

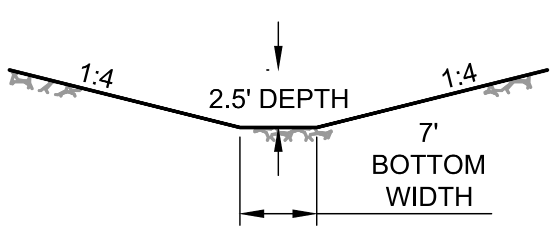
<u>Cost</u>

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. The land acquisition cost assumes $2,500 per acre. Other costs were derived from recent City of Albuquerque and NMDOT bids. The table breaks the costs down into two options. If the option of Sub-Facility 4.2a is chosen, the total EOPC for Facility 4 is $234,000. If Sub-Facility 4.2.b is chosen, the total EOPC for Facility 4 is $1,633,000.

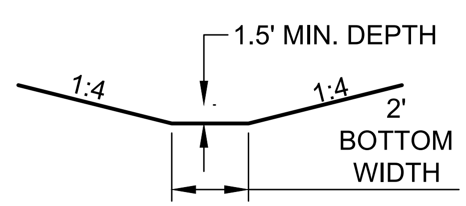
Table 8: Summary of Engineer's Opinion of Probable Cost for Facility 4

Sub Facility	Description	Cost
4.1	Property Acquisition	$ -
4.2.a	Elk Drive Channel	$ 233,000
4.3	Channel From Culverts C28 to C3	$ 10,000
	Total Cost for Facility 4.2.a	**$ 243,000**
4.1	Property Acquisition	$ -
4.2.b	Elk Drive Storm Drain	$ 1,168,000
4.3	Channel From Culverts C28 to C3	$ 10,000
	Total Cost for Facility 4.2.b	**$ 1,664,000**

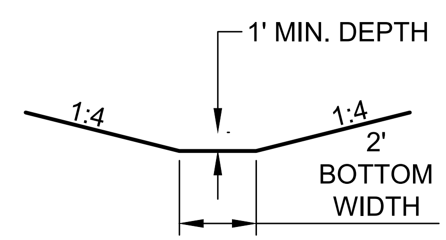




(A) ELKS CHANNEL TYPICAL CROSS SECTION
SCALE: 1" =10'



(B) CULVERTS C29 AND C31 RUNDOWN CROSS SECTION
SCALE: 1" =5'



(C) CULVERTS C28 AND C30 RUNDOWN CROSS SECTION
SCALE: 1" =5'

SMITH ENGINEERING

LEGEND:
EXISTING CULVERT

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 27

PROPOSED ELK CHANNEL OVERVIEW

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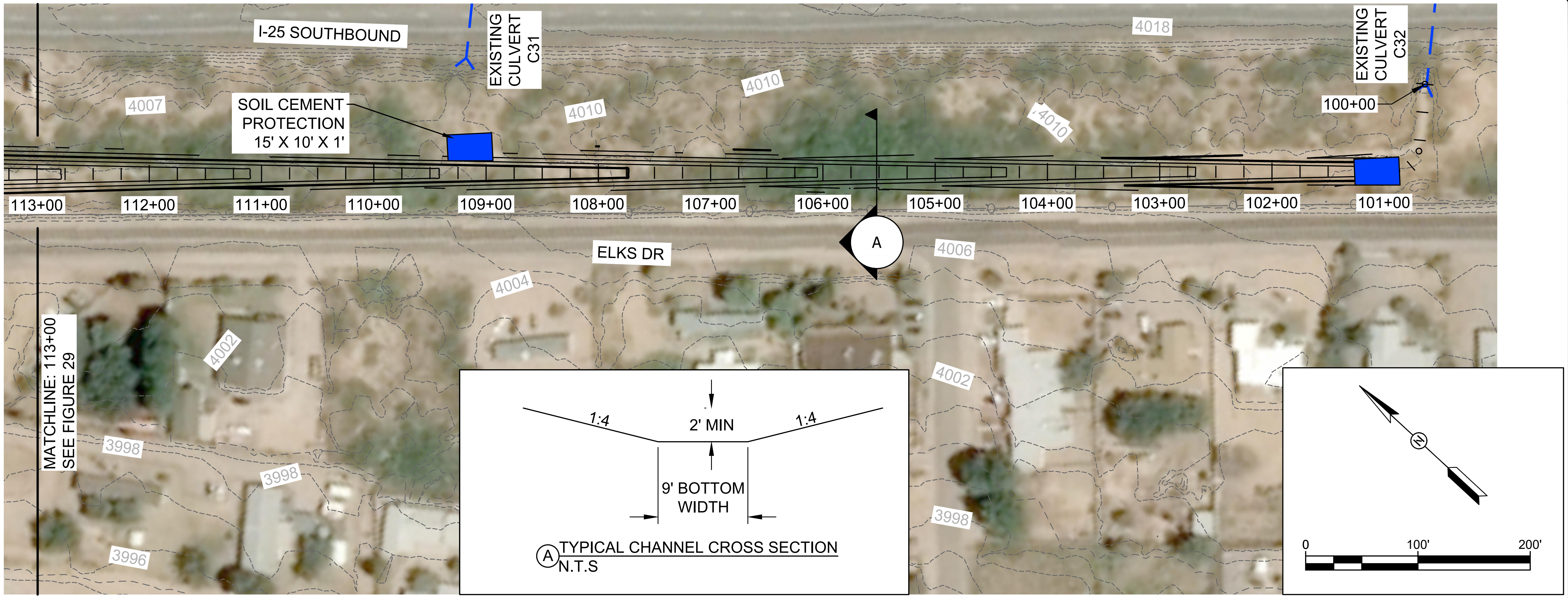

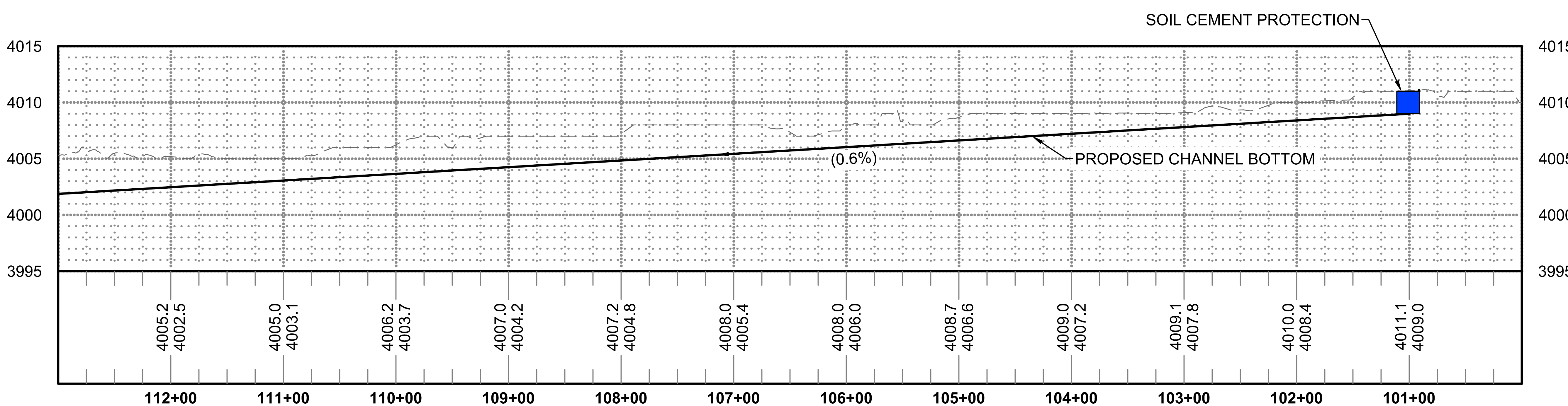

SMITH ENGINEERING

LEGEND:

- EXISTING CULVERT
- EXISTING PROFILE
- PROPOSED PROFILE
- SOIL CEMENT PROTECTION
- EXISTING CHAIN LINK FENCE

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 28

ELK STORM DRAIN STA. 100+00-113+00

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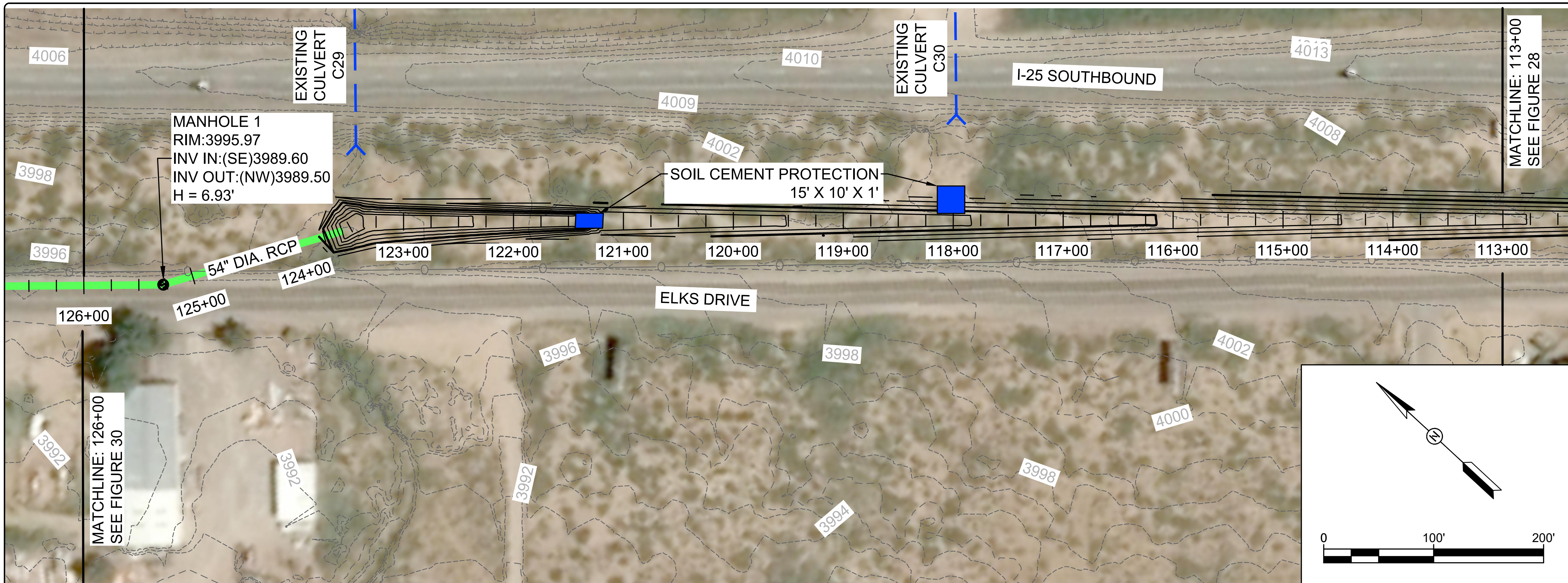

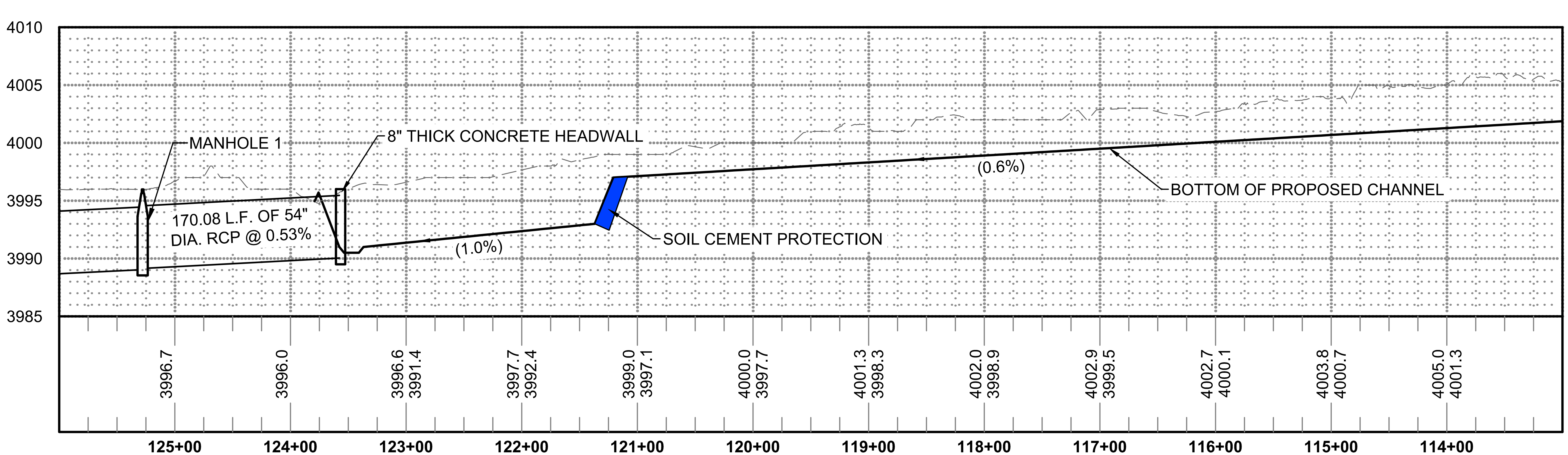

SMITH ENGINEERING

LEGEND:

- EXISTING CULVERT
- PROPOSED STORM DRAIN
- SOIL CEMENT PROTECTION
- EXISTING CHAIN LINK FENCE

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 29

ELK STORM DRAIN STA. 113+00-126+00

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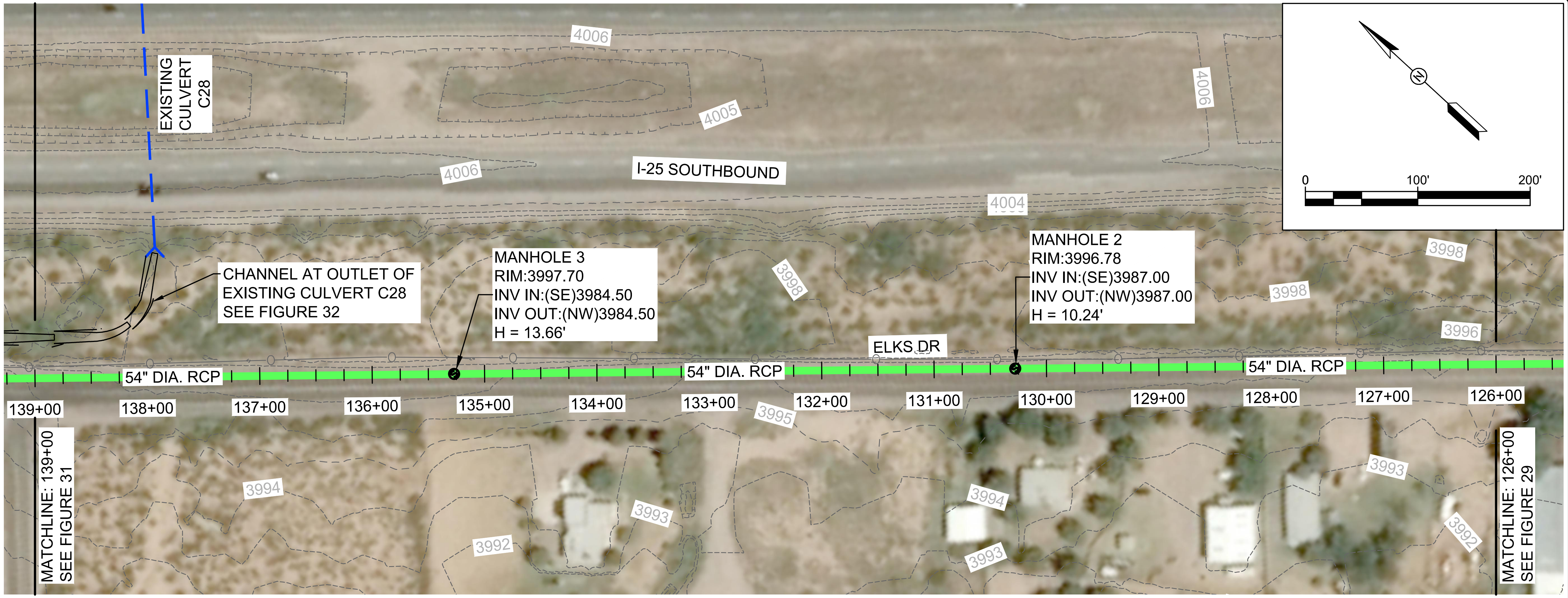

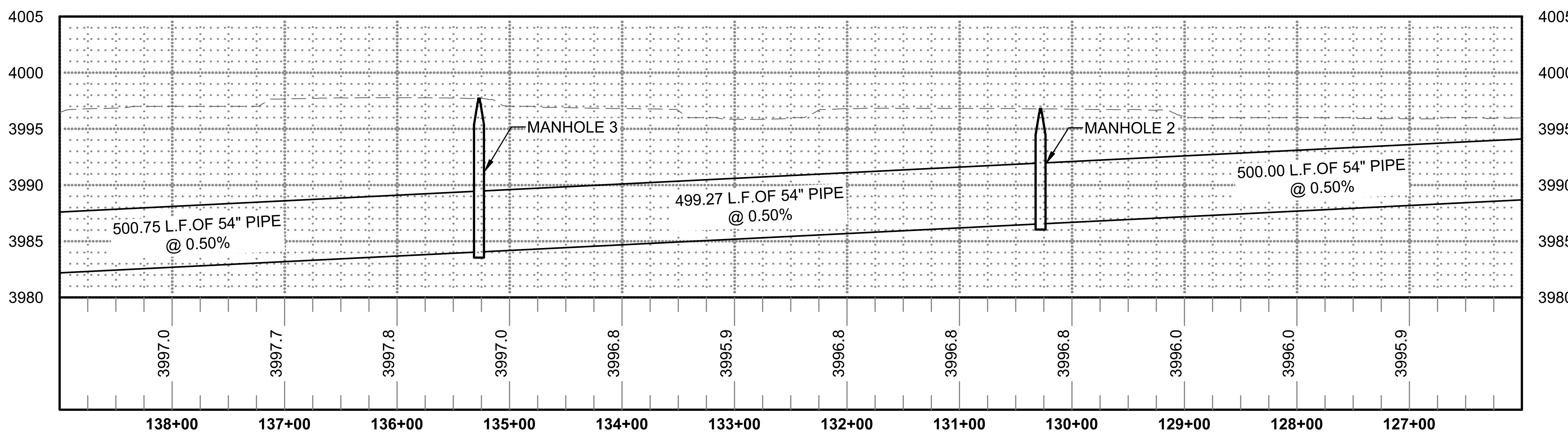

SMITH ENGINEERING

LEGEND:

- EXISTING CULVERT
- PROPOSED STORM DRAIN
- EXISTING CHAIN LINK FENCE

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 30

ELK STORM DRAIN
STA. 126+00-139+00

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2020

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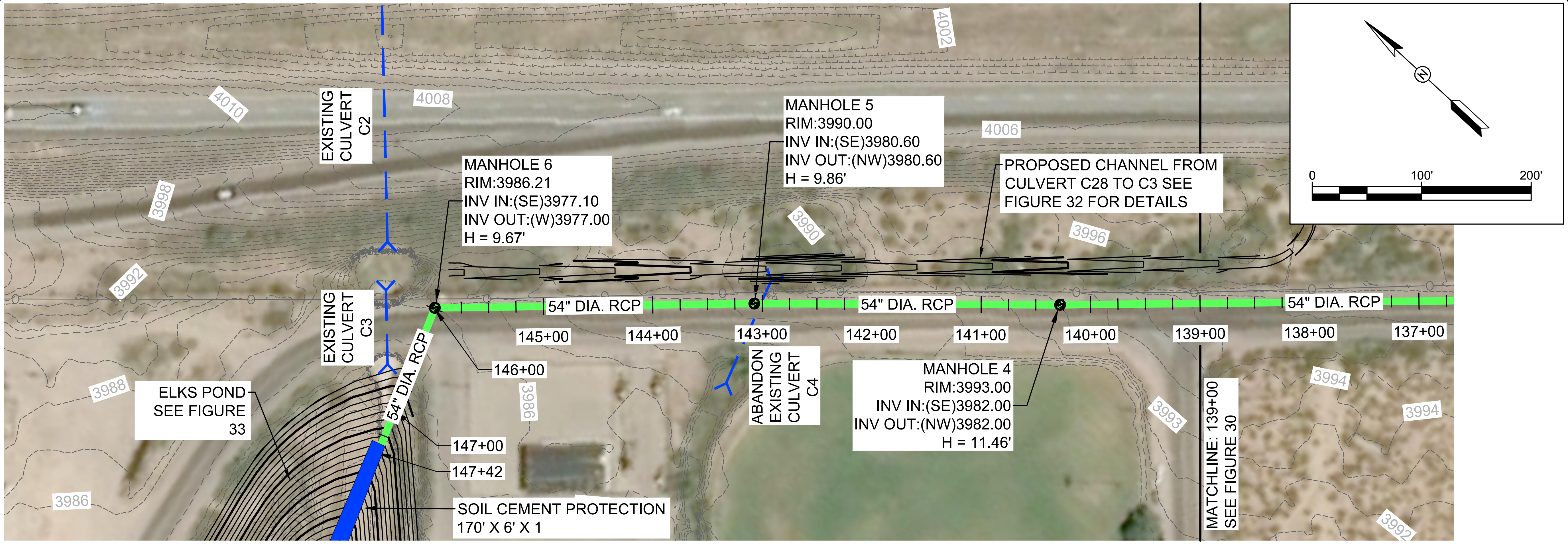

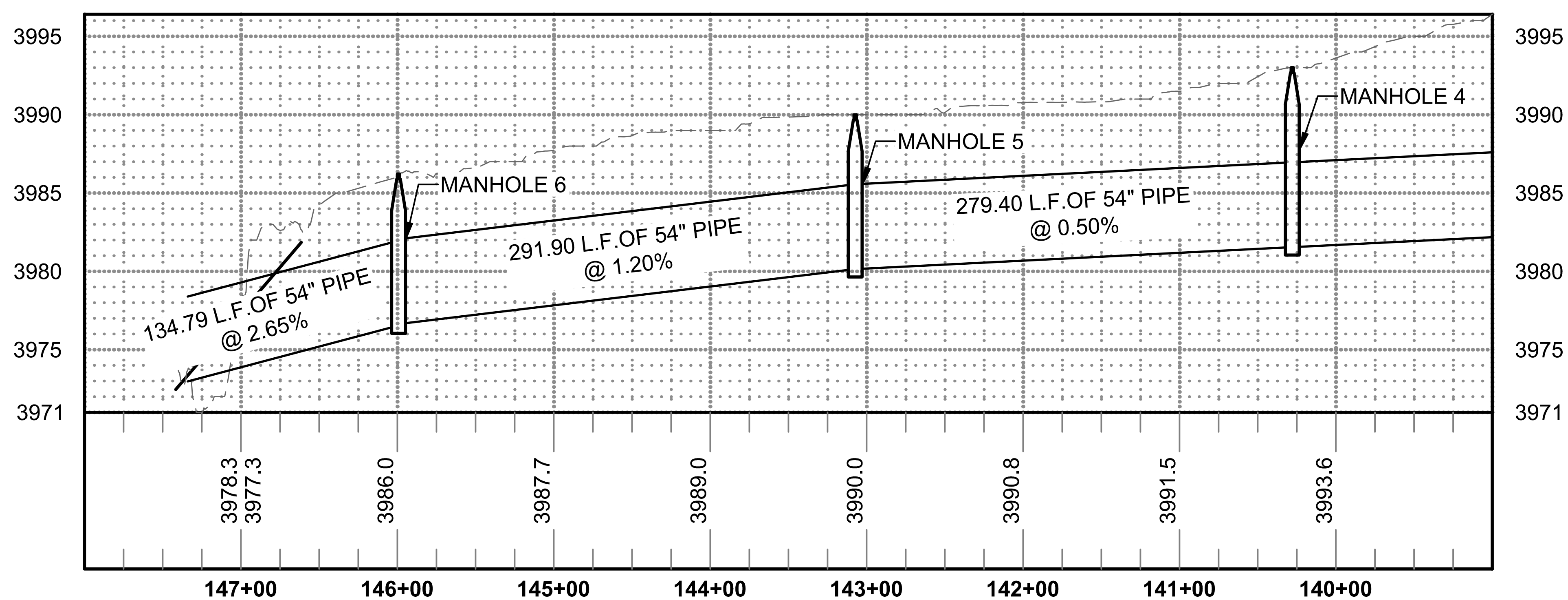

SMITH ENGINEERING

LEGEND:

EXISTING CULVERT

PROPOSED STORM DRAIN

SOIL CEMENT PROTECTION

EXISTING CHAIN LINK FENCE

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 31

ELK STORM DRAIN STA. 139+00-147+42

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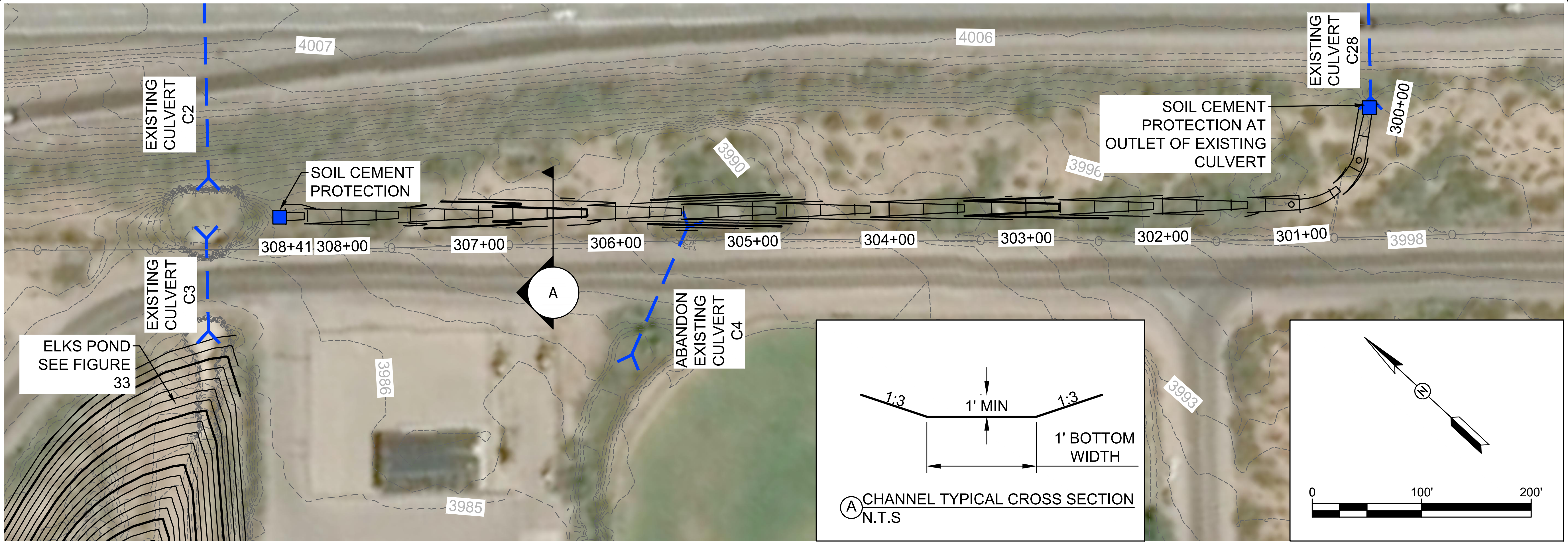

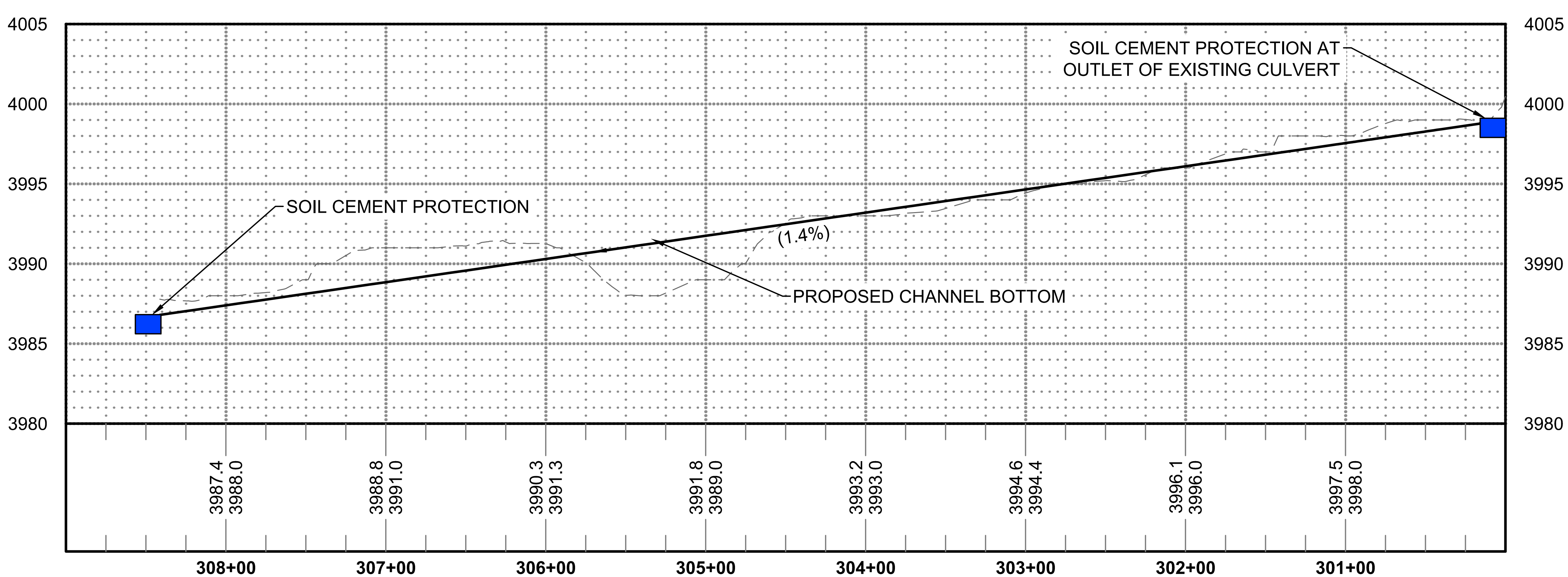

SMITH ENGINEERING

LEGEND:

- EXISTING CULVERT
- EXISTING CHAIN LINK FENCE
- EXISTING PROFILE
- PROPOSED PROFILE
- SOIL CEMENT PROTECTION

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 32

PROPOSED CHANNEL FROM EXISTING CULVERT C28-C3

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Facility 5 Elk Drive Pond

Facility Description

At the southeast corner where El Camino Real and Elks Dr intersect is an open space that has a non-engineered channel (**Figure 34)** that conveys water from Culvert C3 to Culvert C5. This is a primary collection area, where water from culvert C3, C28, C29, C30, C31 and C32 flow into and underneath El Camino Real, which is a main arterial for the Don Ana Community. Once conveyed underneath El Camino Real the water flows to the proposed Facility 7 - Abeyta Pond (**Figure 35**). The purpose of this facility is to contain the flows entering the site and convey them in a control manner under El Camino Real Dr.

This Facility includes a proposed detention pond and associated structures (**Figure 33**). The Elk Drive Pond would help to detain the water and outlet into the existing channel located on the north side of El Camino Real Dr. This channel flows into Facility 7. The benefits of this Facility are with both Facility 5 and 7 in place neither pond utilizes their emergency spillway. Along with this, the flows are further detained with two ponds and have a higher impact on lowering flows downstream of the ponds.

Facility 5 is in the West I-25 Sub-basin Series. An Overview of the Facility is shown on **Figure 33**. The Sub-Facilities were designed to handle the 100-yr 24-hr storm event.

Contributing Sub-basins

The entire 100 and 400 Drainage Sub-basin Series contributes to the Facility through Culverts C32, C31, C30 and C29. Along with the 400 series, sub-basins 18, 19, 20, and 21 contribute to the Facility. The total drainage area contributing to the Facility 5 is 7.1419 sq. mi. The design of the Elks Pond assumes the Doña Ana Dam Site 1 is in place and assumes 100% of the upstream sub-basins are conveyed to the pond. The peak inflow into Elk Pond is 342 cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

Elk Pond was designed to handle the 100-yr 24-hr storm event with minimal to no use of the emergency spillway. The pond has a minimum bottom slope of 0.5% and was designed with 4:1 side slopes.

The principal spillway is a 36” RCP outlet pipe.

The emergency spillway is designed to handle the 100-yr 24-hr storm event. It has a length of 75-ft and height of 1.5-ft. The emergency spillway is located to direct flows, if used, onto El Camino Real. There is an existing drainage channel that runs on the northside, parallel with El Camino Real.

HEC-HMS Modeled Options

Each facility was modeled in various options. This Facility was modeled in the Options 3 and 4 HEC-HMS models. A description of each option is in Section 4.3 of this report.

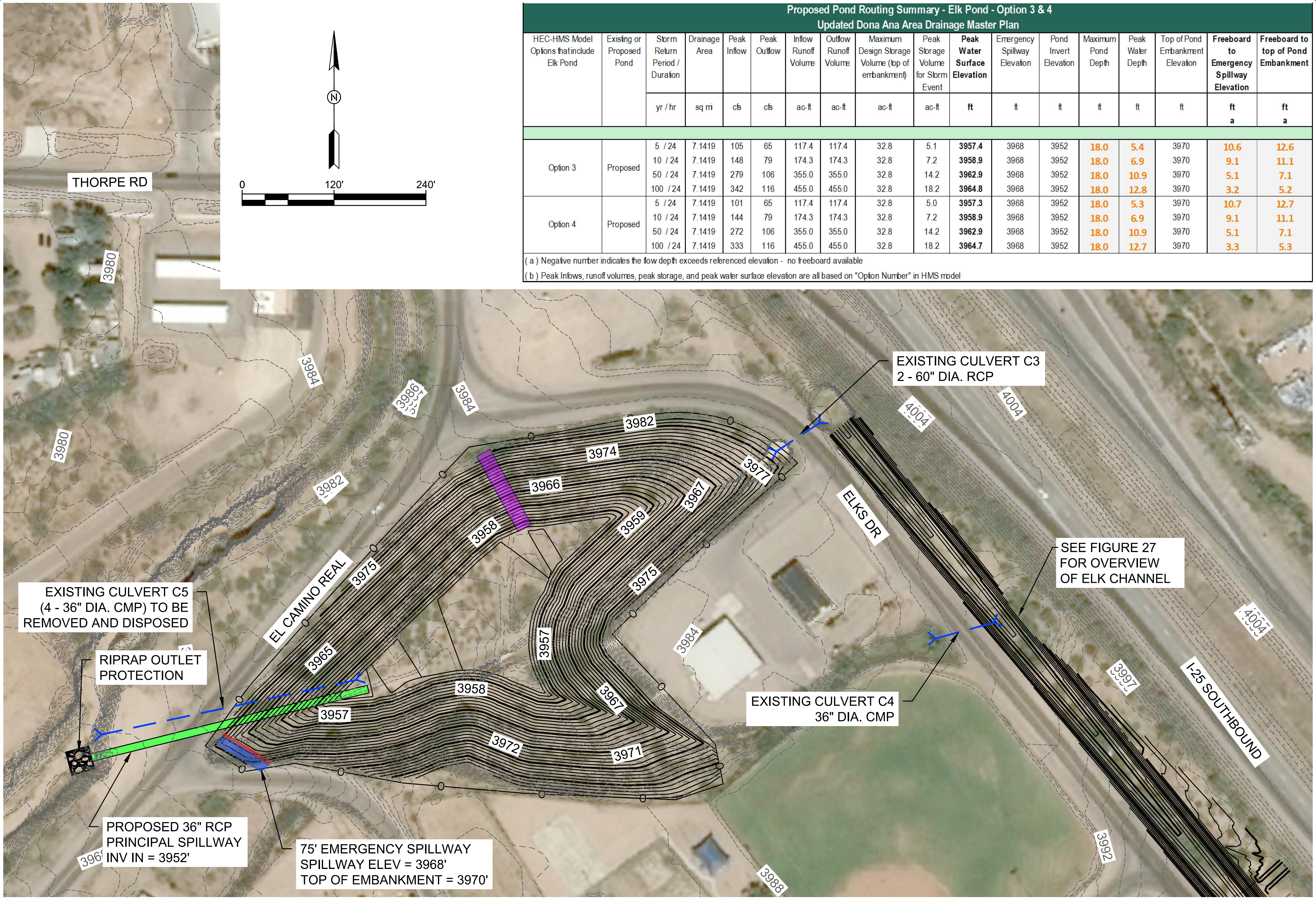
Pond Routing Results

The proposed pond routing summary table is located on **Figure 33**. During the 100-yr storm event the pond has 3.2 to 3.3-ft of freeboard to the emergency spillway and a peak outflow of 116 cfs.

Proposed Pond Routing Summary - Elk Pond - Option 3 & 4 Updated Dona Ana Area Drainage Master Plan																	
HEC-HMS Model Options that include Elk Pond	Existing or Proposed Pond	Storm Return Period / Duration	Drainage Area	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embankment)	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**
		yr / hr	sq mi	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	ft	ft	ft	ft	ft	ft	ft a	ft a
Option 3	Proposed	5 / 24	7.1419	105	65	117.4	117.4	32.8	5.1	**3957.4**	3968	3952	18.0	5.4	3970	10.6	12.6
		10 / 24	7.1419	148	79	174.3	174.3	32.8	7.2	**3958.9**	3968	3952	18.0	6.9	3970	9.1	11.1
		50 / 24	7.1419	279	106	355.0	355.0	32.8	14.2	**3962.9**	3968	3952	18.0	10.9	3970	5.1	7.1
		100 / 24	7.1419	342	116	455.0	455.0	32.8	18.2	**3964.8**	3968	3952	18.0	12.8	3970	3.2	5.2
Option 4	Proposed	5 / 24	7.1419	101	65	117.4	117.4	32.8	5.0	**3957.3**	3968	3952	18.0	5.3	3970	10.7	12.7
		10 / 24	7.1419	144	79	174.3	174.3	32.8	7.2	**3958.9**	3968	3952	18.0	6.9	3970	9.1	11.1
		50 / 24	7.1419	272	106	355.0	355.0	32.8	14.2	**3962.9**	3968	3952	18.0	10.9	3970	5.1	7.1
		100 / 24	7.1419	333	116	455.0	455.0	32.8	18.2	**3964.7**	3968	3952	18.0	12.7	3970	3.3	5.3

(a) Negative number indicates the flow depth exceeds referenced elevation - no freeboard available

(b) Peak Inflows, runoff volumes, peak storage, and peak water surface elevation are all based on "Option Number" in HMS model



SMITH ENGINEERING

LEGEND:

CHAIN LINK FENCE

PARCEL LINES

EXISTING CULVERT

SOIL CEMENT PAD

PROPOSED EMERGENCY SPILLWAY

PROPOSED OUTLET

MAINTENANCE ROAD

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 33

PROPOSED ELK POND OVERVIEW

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Sub-Facilities

Sub-Facility 5.1 – Build Elk Pond including the emergency spillway, erosion control and fence. Remove the existing Culvert C5 and replace with the principal spillway outfall pipe. Abandon existing Culvert C4 and add erosion protection to help prevent scour occurring from Culvert C3. Regrade, if necessary, the channel from Culvert C4 to the Elk Pond. This is shown above on **Figure 33.**

Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. There is no land acquisition cost for Facility 5. Costs for this Facility were estimated based on recent City of Albuquerque and NMDOT bids. The total cost for Facility 5 is $1,171,000.

Table 9: Summary of Engineer's Opinion of Probable Cost for Facility 5

Sub Facility	Description	Cost	
5.1	Elk Drive Pond	$	1,187,000
	Total Cost for Facility 5	**$**	**1,187,000**

Facility 6 El Camino Real Culvert Replacement

Facility Description

At the southeast corner where El Camino Real and Elks Dr intersect is an open space that has a non-engineered channel that conveys water from Culvert C3 to Culvert C5. This is a primary collection area, where water from Culverts C3, C28, C29. C30, C31 and C32 flow into and underneath El Camino Real. Once conveyed underneath El Camino Real the water flows to the proposed Facility 7 - Abeyta Pond. The purpose of Facility 6 is to properly convey the flow under El Camino Real Dr.

This Facility includes replacing Culvert C5 with proposed 60-in. RCP culverts that can convey the 100-yr storm event under the roadway with no overtopping. There are two existing channels (noted as Channels 3 and 4) that connect Culvert C3 and C4 to Culvert C5 as shown on **Figure 34**. According to the FlowMaster calculations these existing channels are adequately sized to handle the 100-yr storm event. Culvert C5, however, is not sized adequately to handle the 100-yr storm event and should be replaced with a larger culvert(s). This small improvement will allow for the water to be effectively and efficiently conveyed to the proposed Abeyta pond (Facility 7). **Figure 34** shows an overview of this Facility.

Contributing Sub-basins

The entire 100 and 400 Drainage Sub-basin Series contributes to the Facility through Culverts C32, C31, C30 and C29. Along with the 100 and 400 series, Sub-basins 18, 19, 20, and 21 contribute to the Facility. The total drainage area contributing to Facility 6 is 7.17 sq. mi. The design of the El Camino Real Culvert assumes the Doña Ana Dam Site 1 is in place and assumes 100% of the sub-basins are conveyed to the culvert. The peak inflow into culvert is 418 cfs during the 100-yr 24-hr event.

Design Criteria and Assumption

The proposed culvert was designed to handle the 100-yr 24-hr storm event. The Culvert has a minimum bottom slope of 0.5% and 1-ft cover and was modeled as 2-60-in RCPs.

HEC-HMS Modeled Options

Each facility was modeled in various options. This Facility was modeled in the Option 5 HEC-HMS model. A description of each option is in Section 4.3 of this report.

Pond Routing Results

There is no pond in this Facility.

Sub-Facilities

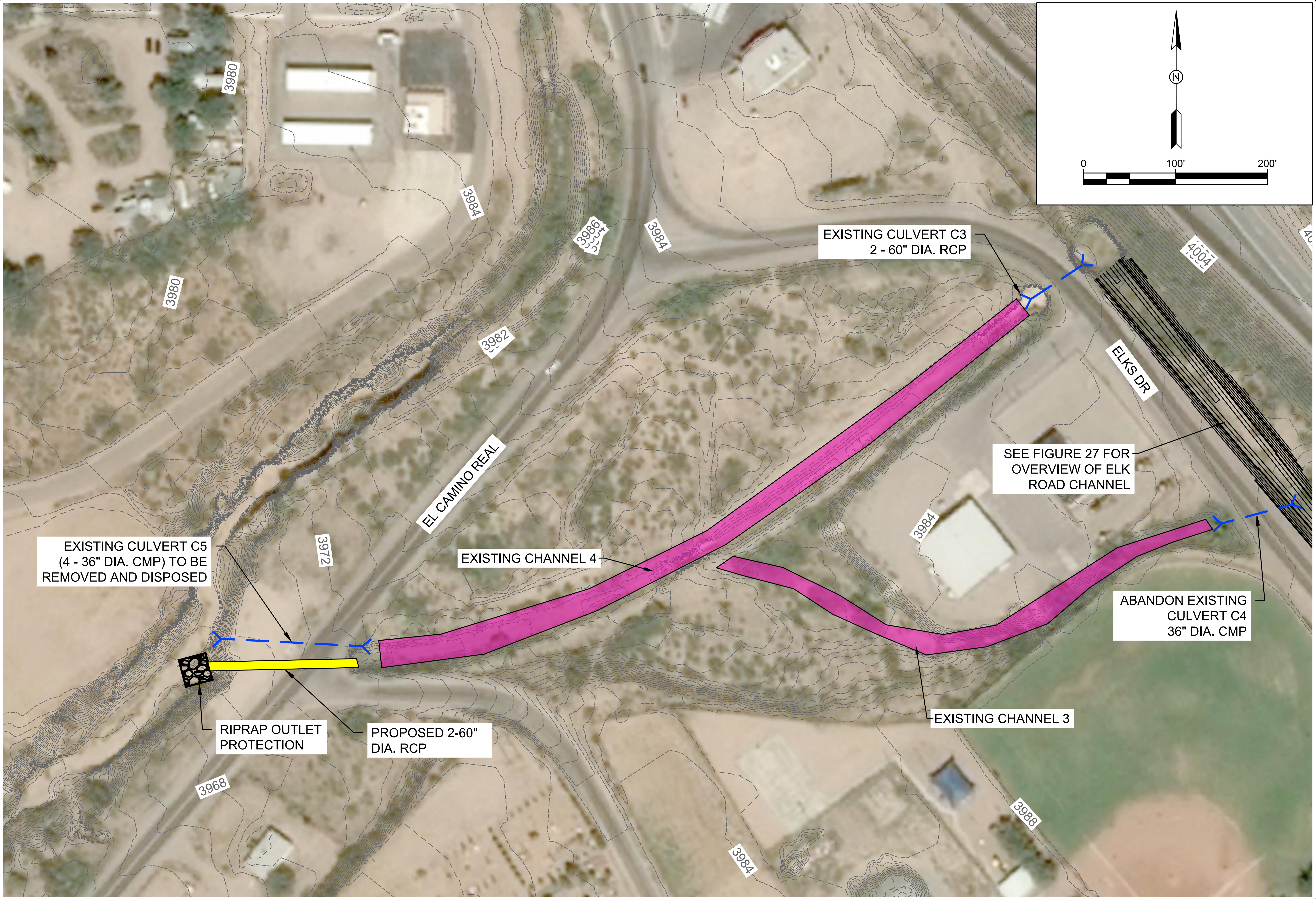
Sub-Facility 6.1 – **Figure 34** shows an overview of the proposed improvement locations. Remove and replace the existing Culvert C5 with 2 – 60in RCPs. Preform any maintenance necessary on existing Channels 3 and 4.

Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. There is no land acquisition cost for Facility 6. Costs for this Facility were estimated based on recent City of Albuquerque and NMDOT bids. The total cost for Facility 5 is $186,000.

Table 10: Summary of Engineer's Opinion of Probable Cost for Facility 6

Sub Facility	Description	Cost
6.1	El Camino Real Culvert Replacement	$ 191,000
	Total Cost for Facility 6	**$ 191,000**



SMITH ENGINEERING

LEGEND:

EXISTING FENCE

PARCEL LINES

EXISTING CULVERT

EXISTING CHANNEL

PROPOSED CULVERT

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 34

OVERVIEW OF CULVERT IMPROVEMENTS ALONG EL CAMINO REAL

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Facility 7 Abeyta Pond

Facility Description

Abeyta Pond (**Figure 35**) is the most downstream pond of the drainage improvements put forth in this report. The pond is a collection point for a portion of the 100, 200 and 400 sub-basins. The pond is downstream of the Doña Ana Dam Sites 1 and 2. At the proposed location there is an existing pond that is currently acting as more of a sediment trap and retention pond vs. detention pond. The existing pond has received minimal maintenance and has filled in with sediment.

The purpose of this Facility is to detain the large amount of flows the existing pond receives. By detaining the flow upstream, the downstream impact will be less flooding and ponding.

Contributing Sub-basins

The entire 100 and 400 Drainage Sub-basin Series and a portion of 200 Sub-basin Series contributes to the Facility. Along with these sub-basin series, sub-basins 18, 19, 20, 21, 42, 43, 44, 46, contribute to the Facility. The total drainage area contributing to the Facility 7 is 7.2752 sq. mi. The design of the Abeyta Pond assumes the Doña Ana Dam Site 1 and 2 are in place and 100% of the above-mentioned sub-basins are conveyed to the pond. The peak inflow into the pond is 294 to 550 cfs (depending on which Option) during the 100-yr 24-hr event.

Design Criteria and Assumption

Abeyta Pond was designed to handle the 100-yr 24-hr storm event with minimal to no use of the emergency spillway. The bottom of the pond has a minimum slope of 0.5%, 4:1 side slopes and is 8-ft deep.

The principal spillway is designed as a 48" RCP pipe. The principal spillway outlets into existing Channel 6-Wasteway 5.

The emergency spillway is designed to handle the 100-yr 24-hr storm event. It has a length of 64-ft and height of 1.5-ft. The emergency spillway is located to direct flows, if used, into the existing Channel 6-Wasteway 5.

HEC-HMS Modeled Options

Each facility was modeled in various options. This Facility is modeled in the Option 3, 4, and 5 HEC-HMS models. A description of each option is in Section 4.3 of this report.

Pond Routing Results

The proposed pond routing summary table is located on **Figure 35**. The pond routing for Abeyta Pond is different depending on which option is selected. Below is a brief summary of the ponding results.

Options 3 and 4 –Abeyta Pond does not use the emergency spillway during the 100-yr design storm, having a 0.1-ft freeboard to the emergency spillway and peak discharge of 142 cfs.

Options 5 – For this HEC-HMS model Elk Pond is not modeled. Therefore, the Abeyta pond has a higher inflow. For this option the emergency spillway is used during the 100-yr storm event, but the pond is not overtopped, see **Figure 35** for a detailed pond routing summary. The peak discharge from the pond during the 100-yr event is 185-cfs.

Abeyta pond outlets into the existing Channel 6-Wasteway 5. Channel 6-Wssteway 5 was modeled in FlowMaster to estimate the capacity of the existing channel. According to the FlowMaster output the existing channel can handle the discharge for Options 3, 4 and 5 without overtopping. **Appendix G** contains the FlowMaster model summary for Channel 6-Wasteway 5.

Sub-Facilities

Sub-Facility 7.1 – Purchase property for Abeyta Pond and acquire necessary drainage easements.

Sub-facility 7.2 – Build Abeyta Pond including the emergency spillway, principal spillway, erosion control and fence.

Cost

The costs for each sub-facility is summarized in the table below. A detail EOPC is in **Appendix H** of the report. The land acquisition cost assumes $2,500 per acre. Other costs were estimated based on recent bids from the City of Albuquerque and NMDOT. The total EOPC for Facility 7 is $871,000.

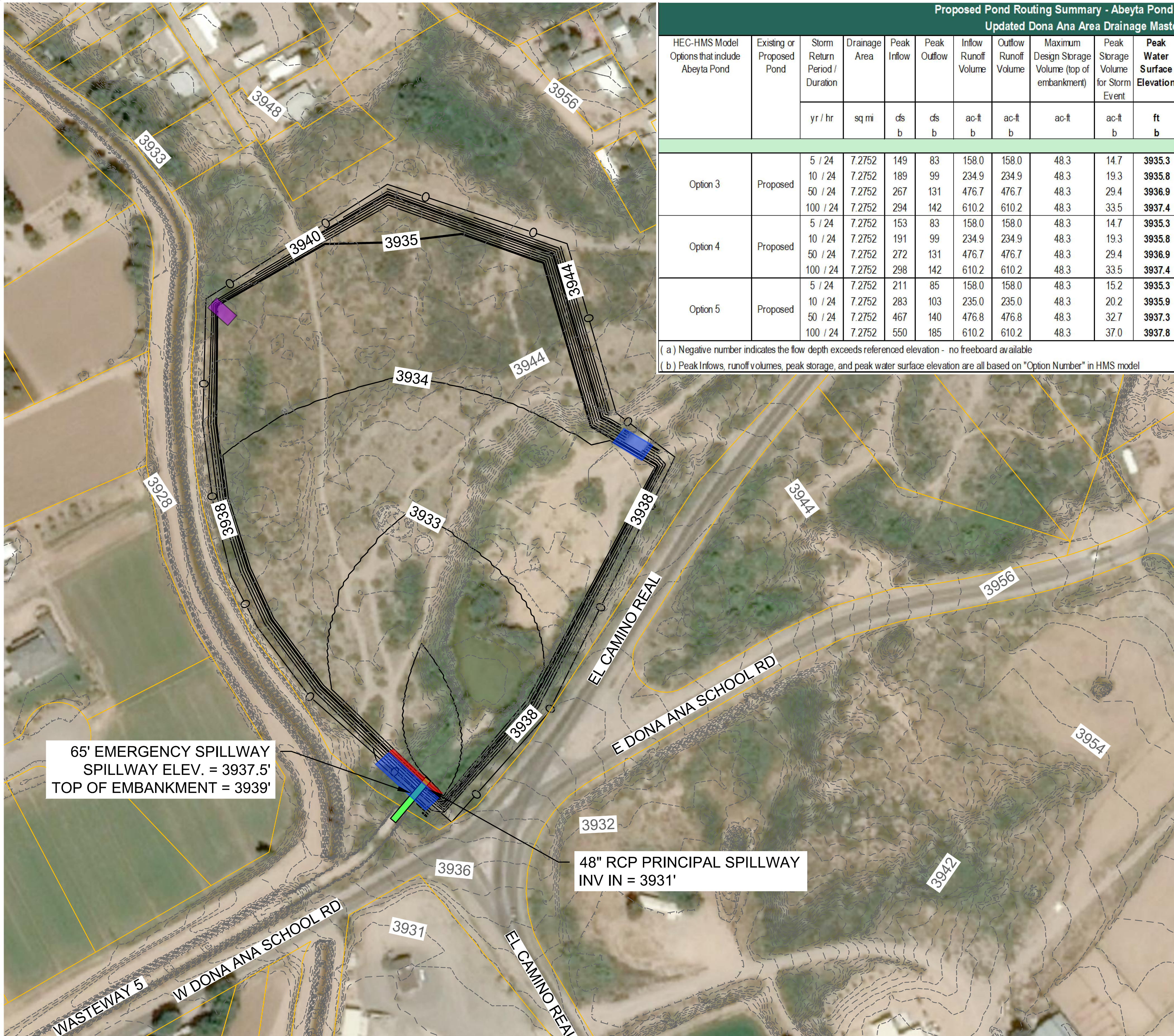
Table 11: Summary of Engineer's Opinion of Probable Cost for Facility 7

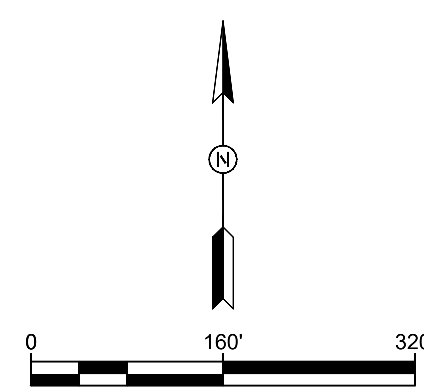
Sub Facility	Description	Cost
7.1	Property Acquisition2	$ 480,000
7.2	Abeyta Pond	$ 841,000
	Total Cost for Facility 7	**$ 1,321,000**

Proposed Pond Routing Summary - Abeyta Pond - Options 3, 4 & 5 Updated Dona Ana Area Drainage Master Plan																	
HEC-HMS Model Options that include Abeyta Pond	Existing or Proposed Pond	Storm Return Period / Duration	Drainage Area	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Design Storage Volume (top of embankment)	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**
		yr / hr	sq mi	cfs b	cfs b	ac-ft b	ac-ft b	ac-ft	ac-ft b	**ft b**	ft	ft	ft	ft	ft	**ft a**	**ft a**
Option 3	Proposed	5 / 24	7.2752	149	83	158.0	158.0	48.3	14.7	**3935.3**	3937.5	3931	**8.0**	**4.3**	3939	**2.2**	**3.7**
		10 / 24	7.2752	189	99	234.9	234.9	48.3	19.3	**3935.8**	3937.5	3931	**8.0**	**4.8**	3939	**1.7**	**3.2**
		50 / 24	7.2752	267	131	476.7	476.7	48.3	29.4	**3936.9**	3937.5	3931	**8.0**	**5.9**	3939	**0.6**	**2.1**
		100 / 24	7.2752	294	142	610.2	610.2	48.3	33.5	**3937.4**	3937.5	3931	**8.0**	**6.4**	3939	**0.1**	**1.6**
Option 4	Proposed	5 / 24	7.2752	153	83	158.0	158.0	48.3	14.7	**3935.3**	3937.5	3931	**8.0**	**4.3**	3939	**2.2**	**3.7**
		10 / 24	7.2752	191	99	234.9	234.9	48.3	19.3	**3935.8**	3937.5	3931	**8.0**	**4.8**	3939	**1.7**	**3.2**
		50 / 24	7.2752	272	131	476.7	476.7	48.3	29.4	**3936.9**	3937.5	3931	**8.0**	**5.9**	3939	**0.6**	**2.1**
		100 / 24	7.2752	298	142	610.2	610.2	48.3	33.5	**3937.4**	3937.5	3931	**8.0**	**6.4**	3939	**0.1**	**1.6**
Option 5	Proposed	5 / 24	7.2752	211	85	158.0	158.0	48.3	15.2	**3935.3**	3937.5	3931	**8.0**	**4.3**	3939	**2.2**	**3.7**
		10 / 24	7.2752	283	103	235.0	235.0	48.3	20.2	**3935.9**	3937.5	3931	**8.0**	**4.9**	3939	**1.6**	**3.1**
		50 / 24	7.2752	467	140	476.8	476.8	48.3	32.7	**3937.3**	3937.5	3931	**8.0**	**6.3**	3939	**0.2**	**1.7**
		100 / 24	7.2752	550	185	610.2	610.2	48.3	37.0	**3937.8**	3937.5	3931	**8.0**	**6.8**	3939	-0.3	**1.2**

(a) Negative number indicates the flow depth exceeds referenced elevation - no freeboard available

(b) Peak Infows, runoff volumes, peak storage, and peak water surface elevation are all based on "Option Number" in HMS model





SMITH ENGINEERING

LEGEND:

CHAIN LINK FENCE

PARCEL LINES

SOIL CEMENT PAD

PROPOSED EMERGENCY SPILLWAY

PROPOSED OUTLET

MAINTENANCE ROAD

UPDATED DOÑA ANA AREA DRAINAGE MASTER PLAN

FIGURE 35

PROPOSED ABEYTA POND OVERVIEW

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4.3 PROPOSED OPTIONS

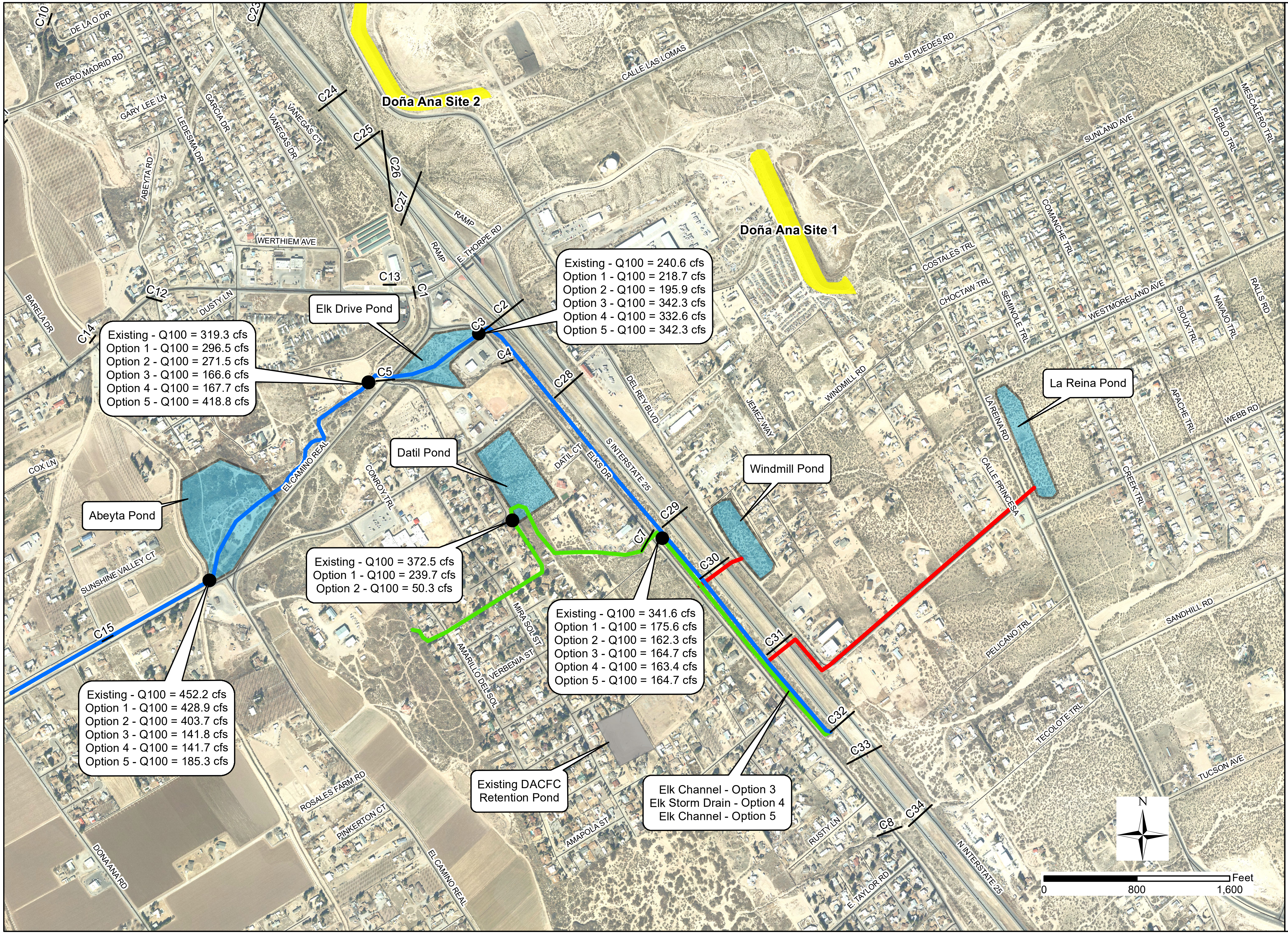
There are 5 options being present in this section. Each option is comprised of various combinations of the proposed facilities discussed above and listed below. The options were completed to show the hydrologic impacts each facility has as an individual component and as it works in conjunction with the other proposed facilities. The option name is the name of the corresponding HEC-HMS model. The output of these models is in **Appendix E**. The facilities are listed below as a quick reference.

- Facility 1 – La Reina Pond
- Facility 2 – Windmill Pond
- Facility 3 – Datil Pond
- Facility 4 – Elk Drive Storm drain and Channel
- Facility 5 – Elk Pond
- Facility 6 – El Camino Real Culvert Replacement
- Facility 7 – Abeyta Pond

Shown on **Figure 36** is a schematic overview of the impact each option has on the community. The 100-yr flows are shown at various junction points for the options presented. Along with flows, a flow path for each option is shown, to help illustrate how each option is functioning and how/where the water is being conveyed.

Below is a list of the options, stating which Facilities are included in each option.

- Option 1
    - Facility 1 – La Reina Pond
    - Facility 2 – Windmill Pond
- Option 2
    - Facility 1 – La Reina Pond
    - Facility 2 – Windmill Pond
    - Facility 3 – Datil Pond
- Options 3 and 4
    - Facility 1 – La Reina Pond
    - Facility 2 – Windmill Pond
    - Facility 4 – Elk Drive Storm Drain and Channel
    - Facility 5 – Elk Pond
    - Facility 7 – Abeyta Pond
- Option 3, 4 & 5
    - Facility 1 – La Reina Pond
    - Facility 2 – Windmill Pond
    - Facility 4 – Elk Drive Storm Drain and Channel
    - Facility 6 – El Camino Real Culvert Replacement
    - Facility 7 – Abeyta Pond



Legend

- Analysis Points for Proposed Options
- Existing Culverts
- Options 3, 4 & 5 Flow Path
- Option 2 Flow Path
- Option 1 Flow Path
- Existing Retention Pond
- Existing Dams
- Proposed Ponds
- Roads

Updated Dona Ana Area Drainage Master Plan

Prepared for

NEW MEXICO DONA ANA COUNTY

Prepared by

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Figure 36 Flow Overview of the Proposed Options

2020

Option 1

Option 1 is comprised of two facilities. Facility 1 and Facility 2. These facilities are the most upstream improvements being proposed. These Facilities do not feed into one another but do detain flows for the proposed Facilities 3 through 7. These Facilities are included in each option. However, to show the downstream impact of Facilities 1 and 2 by themselves, they are present in Option 1 with no other proposed improvements.

In the existing conditions HEC-HMS model the flows at C29 is 341.6 CFS. By implementing Option 1 these flows are reduced to 176 cfs. The further implications of implementing Option 1 are shown on **Figure 36**.

The EPOC for this option is $973,000.

Option 2

Option 2 is comprised of three facilities. Facility 1, Facility 2 and Facility 3. This option takes the flows from Facilities 1 and 2 and conveys them to the proposed Datil Pond, where the flows are further detained, discharging into an existing channel that runs along a residential community. Ultimately taking the flow to southwest. The ultimate outfall of this conveyance system would need to be look at further.

The EPOC for this option is $2,030,000.

Options 3 and 4

Options 3 and 4 are comprised of five Facilities; Facilities 1 and 2 along with Facilities 4, 5 and 7. The difference between the two options are the routing alternatives discussed in Facility 4. These alternatives (a storm drain system vs. a channel) were modeled in HMS to show the hydrologic difference between the alternatives. Option 3 models the all channel option. Option 4 is the alternative with the storm drain and channel. The outputs of the models show little hydrologic difference between the models. This is not surprising as both alternatives presented propose to capture 100% of the 100-yr 24-hr storm event. The main difference between the options is cost, maintenance issues, and the overall goal of the DAFC.

The total EPOC for Option 3 is $3,249,000.

The total EOPC for Option 4 is $4,180,000.

Option 5

Option 5 consists of five facilities; Facilities 1 and 2 along with Facilities 4, 6 and 7. For this option the Elk pond is not implemented. Instead the culverts at El Camino Real Dr are upgraded to handle the 100-yr storm event. The water is then routed to Facility 7 - Abeyta Pond through an existing channel. This channel is not an engineered channel. A typical cross section of channel does indicate that it has the capacity to handle to 100-yr event with no overtopping. The Abeyta pond will outlet into the existing Channel 6 – Wasteway 5. The capacity of this channel is also capable of handling the peak outflow from Abeyta Pond. For this option, the proposed facilities are design to handle the 100-yr flow with use of the emergency spillway.

The total EOPC for Option 5 is $3,249,000.

SECTION 5. PRIORITIZATION OF OPTIONS

5.1 VIABLE FACILITIES

From communication with DACFC all the options present in this report are viable. In **Table 12** each facility was broken into four phases and prioritized. The phasing and priority of the facilities were based on discussion with the DACFC, costs, and overall benefit to the community. The priority and phasing will be further refined after the 90% review.

5.2 CONCLUSIONS AND RECOMMENDATIONS

Each of the Facilities presented in this report will improve the drainage system of the Doña Ana area. Below is a table that summarizes the EOPC for each Facility.

Table 12: Summary of Engineer's Opinion of Probable Cost for All Facilities

Facility No.	Priority	Phase	Description	Cost
Facility 1	5	3	La Reina Pond, Proposed Rundown, Earth Lined Channels, La Reina Driveway Culverts, Webb Road Curb and Gutter	$ 1,183,000
Facility 2	4	3	Windmill Pond	$ 455,000
Facility 3	6	4	Datil Pond, Datil Channel, Remove/Replace Existing Private Culverts, Channel from Culvert C32 to C7, Replacement of Culvert C7, Rebuild Existing Wall and Maintain Existing Channel	$ 1,621,000
Facility 4 (a)	1	1	Elk Drive Channel, and Channel From Culvert C28 to C3	$ 243,000
Facility 4 (b)	1	1	Elk Drive Storm Drain, and Channel From Culvert C28 to C3	$ 1,178,000
Facility 5	2	2	Elk Drive Pond	$ 1,187,000
Facility 6	2	2	El Camino Real Culvert Replacement	$ 191,000
Facility 7	3	2	Abeyta Pond	$ 1,321,000
Total Cost of Phased Capital Improvement Projects with Facility 4.2.a				**$ 6,201,000**
Total Cost of Phased Capital Improvement Projects with Facility 4.2.b				**$ 7,136,000**

For Phase 1 it is suggested that Facility 4 be completed. This will have an immediate impact on the drainage, there is existing facilities downstream that are in place to handle the flow. Although there are existing downstream facilities, they are undersized and need maintenance. The second part of phase 1 focuses on upgrading these facilities (Facilities 6 and 7). Phase 1 will create a complete drainage network and will help mitigate flooding/ponding in the area. However. Facility 7 will utilize the emergency spillway during the 100-yr storm event.

Phase 2 focuses on the drainage east of I-25. For this phase it is suggested that Facilities 1 and 2 be completed. Facility 1 will help detain the flows and provide drainage improvements to the surrounding community while Facility 2 will be very beneficial for maintenance of the proposed Facilities in Phases 1.

The third phase would be the completion of Elk Drive Pond (Facility 5). If in place, Facility 7 will not utilize the emergency spillway during the 100-yr storm event.

The final phase is the completion of Datil Pond. This pond would detain flows and direct them into an existing channel. However, the final outlet of this channel is open space with no controlled outlet. This pond may be deemed necessary depending on how well the other facilities function and if there is future development in the area.

SECTION 6. REFERENCES

1. NOAA Atlas 14 Point Precipitation Frequency Estimates Output (printed from NOAA Atlas 14 internet site).

2. Figure 14, Depth-Area Curves (Source: NOAA Atlas 2 Vol. IV, New Mexico 1973).

3. Urban Hydrology for Small Watersheds, U.S. Department of Agricultural Soil Conservation Service, Technical Release 55, June 1986.

Approximate Geographic Boundaries for SCS Rainfall Distributions (FOR REFERENCE ONLY – The HEC-HMS Rainfall 25% Frequency Distribution was adopted).

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.

Chapter 3 - Time of Concentration and Travel Time Computation Procedure

4. National Engineering Handbook, Part 630, Chapter 15 - Time of Concentration. Natural Resources Conservation Service. May 2010. (Documentation that Lag Time = 0.6 Time of Concentration).

5. Sediment Bulking Factors were assumed based select pages - Figure 3.8 within - Sediment and Erosion Design Guide, November 2008. Prepared by Mussetter Engineering Inc. Prepared for the Southern Sandoval County Arroyo Flood Control Authority.

6. HEC-HMS Computation Time Interval Guidance.

7. Manning's "n" Values from - Open Channel Hydraulics, Ven T. Chow, 1959.

8. Soils Data Summary for: Soil Map Unit Descriptions and Hydrologic Soil Groups from Natural Resources Conservation Service (NRCS) Web Soil Survey – National Cooperative Soil Survey.

APPENDIX A: ANNOTATED PHOTOGRAPHS

APPENDIX B: PREVIOUS PLANS, TOPOGRAPHIC SURVEYS, AND REPORTS

APPENDIX C: EXISTING HYDROLOGIC DATA TABLES, COMPUTATIONS, AND REFERENCES

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