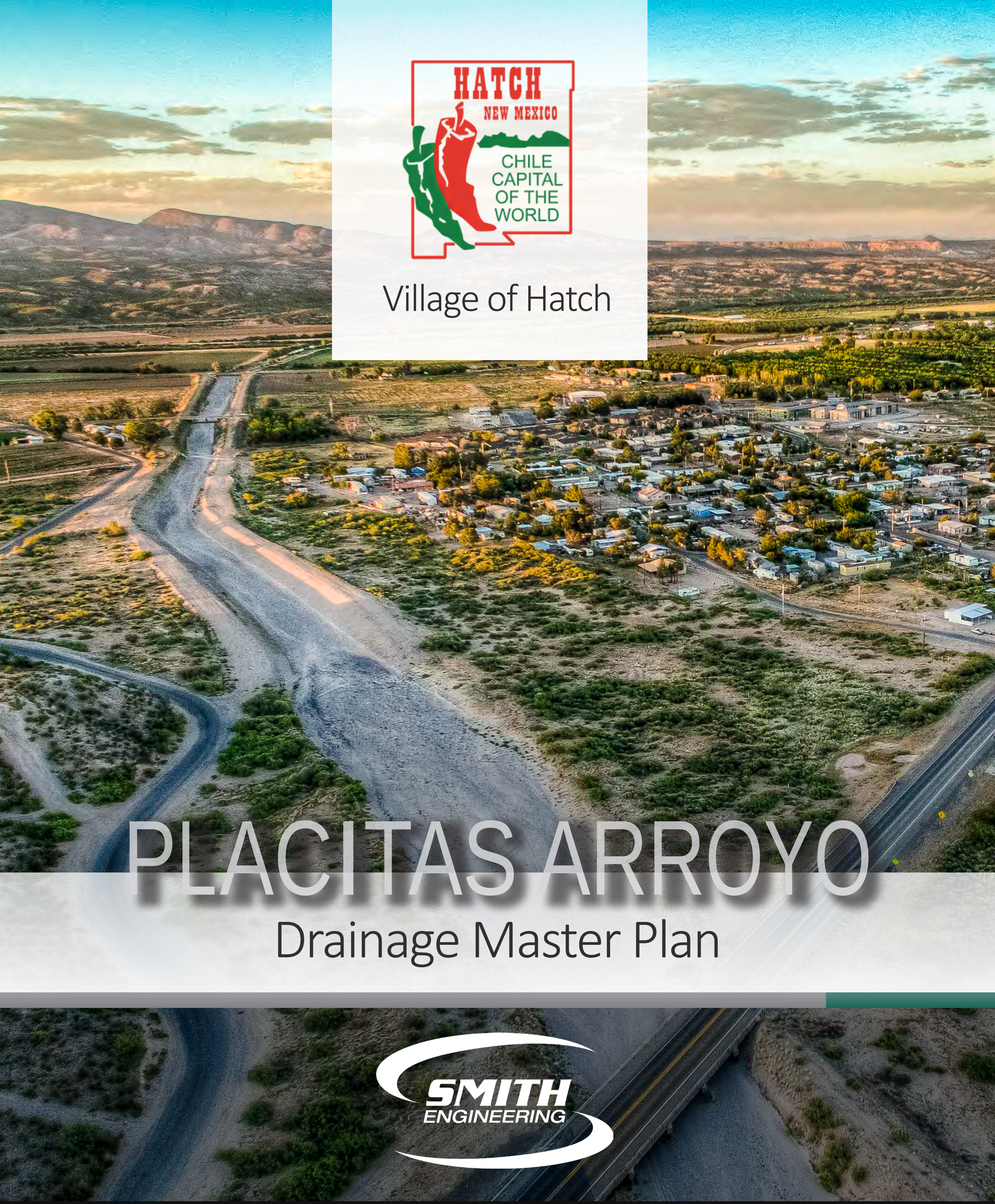

Village of Hatch

PLACITAS ARROYO

Drainage Master Plan

FINAL REPORT | JULY, 2018

201 N. CHURCH ST., SUITE 200A | LAS CRUCES, NM 88001 | 575-523-2395

PLACITAS ARROYO DRAINAGE MASTER PLAN

FINAL REPORT

PREPARED FOR THE VILLAGE OF HATCH IN CONJUNCTION WITH THE DOÑA ANA COUNTY FLOOD COMMISSION

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April 2018
Rev. July 2018
Smith Project No.: 117112

PLACITAS ARROYO DRAINAGE MASTER PLAN

FINAL REPORT

PREPARED FOR THE VILLAGE OF HATCH IN CONJUNCTION WITH THE DOÑA ANA COUNTY FLOOD COMMISSION

FINAL

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

E. CHRISTIAN NAIDU P.E.

July 2018

ACKNOWLEDGMENTS

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

The Village of Hatch Community for invaluable historical accounts of flooding and input regarding areas of concern.

EXECUTIVE SUMMARY

DESCRIPTION AND PURPOSE OF PROJECT

The Placitas Arroyo Drainage Master Plan (DMP) was prepared by Smith Engineering Company (Smith) for the Village of Hatch (VOH) in conjunction with the Doña Ana County Flood Commission (DACFC) to study the Placitas Arroyo Watershed, located in northern Doña Ana County. As part of this study, an existing conditions hydrologic model was developed to determine peak runoff rates and discharge volumes. The hydrologic conditions were evaluated using the U.S. Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) V4.2.1. Simulations were run for the following storms: 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year return periods of 24-hour duration. The hydraulic analysis of the Placitas Arroyo was completed using the U.S. Army Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) V5.0.3. Due to the complex hydraulic nature of the Placitas Arroyo, both 1D HEC-RAS and 2D HEC-RAS models were utilized. Based on the results of the modeling efforts, areas of potential flooding were identified, and proposed drainage improvement options were developed to mitigate flooding. The flood mitigation options are designed for the 100-year 24-hour return period storm. Prioritization of the options were finalized after 90% review meetings with the VOH and DACFC.

SUMMARY OF EXISTING BASIN AND EXISTING DRAINAGE INFRASTRUCTURE

The Placitas Arroyo Watershed has a total drainage area of 31.6 square miles. The subbasins are undeveloped semi-arid rangelands with fair to extremely steep topography and poor vegetative cover. The Placitas Arroyo has three major roadway crossings: a concrete bridge at NM 26, a concrete bridge at NM 187, and concrete box culverts at Canal Road. These structures were evaluated for their maximum discharge capacity relative to the inflow peak discharges from the various return period storms.

SUMMARY OF EXISTING PROBLEM AREAS AND PROPOSED OPTIONS

The capacity of the Arroyo through the Village is limited by the existing channel geometry as well as the road crossings. Based on the results from the existing conditions model, flood control improvements were conceptually designed and simulated for the watersheds. Upper watershed flood reduction improvements such as ponds were considered and modeled. These options were not pursued due to their lack of impact on the peak discharges. Lower watershed improvements are based on the potential options for conveyance through channel improvements. Two alternatives for channel and levee improvements are presented. Both alternatives will require permitting from the USACE.

Alternative 1 assumes that Nationwide Permits will be pursued. This alternative evaluated channel and levee development under two scenarios. Scenario 1 assumes a riprap lined channel. Scenario 2 assumes a shotcrete lined channel. The channel and levee will be designed so that the Ordinary High Water Mark (OHWM) is not disturbed for the majority of the project reach.

Alternative 2 assumes that the OHWM will be disturbed and a full Individual Permit will be acquired. This alternative also evaluated channel and levee development under two scenarios. Scenario 1 assumes a riprap lined channel. Scenario 2 assumes a shotcrete lined channel.

Hydraulic analysis of the channel configurations under both scenarios were performed to optimize conveyance. Cost estimates for both alternatives and scenarios were performed. In addition, future value computations were developed to forecast how the project costs will increase due to inflation over a 20-year period.

The **Table E1** below summarizes the differences in cost for the two alternatives.

Table E1: Cost Summary for Alternatives

Alternative Description	Scenario 1: Riprap Lined Channel Including Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 1: Riprap Lined Channel Excluding Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 2: Shotcrete Lined Channel Including Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 2: Shotcrete Lined Channel Excluding Alternate Sediment Facility at Outlet of Placitas Arroyo
Alternative 1: Benched Channel Design Assuming Nationwide Permit	$36,817,000	$27,673,000	$30,453,000	$23,242,000
Alternative 2: Trapezoidal Channel Design Assuming Individual Permit	$36,937,000	$27,782,000	$30,485,000	$23,263,000

CONCLUSIONS AND RECOMMENDATIONS

These alternatives and costs were reviewed by the DACFC and VOH to determine the final direction of the master plan.

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SECTION 1. GENERAL PROJECT INFORMATION

1.1 DESCRIPTION AND PURPOSE OF PROJECT

The Placitas Arroyo Drainage Master Plan (DMP) was prepared by Smith Engineering Company (Smith) for the Village of Hatch (VOH) in conjunction with the Doña Ana County Flood Commission (DACFC) to study the Placitas Arroyo Watershed. The Placitas Arroyo Watershed is approximately 40 miles northwest of Las Cruces. **Figure 1** shows the project vicinity map. The Placitas Arroyo's headwaters are in the Sierra de Las Uvas mountains and it drains northeast towards the Rio Grande. This ephemeral arroyo begins as a natural arroyo. However, upon passing under NM 26, it becomes a perched channel with un-engineered earthen levees on both banks. The leveed section of the Arroyo passes under three bridge crossings at NM 26, NM 187, and Canal Road before flowing into the Rio Grande.

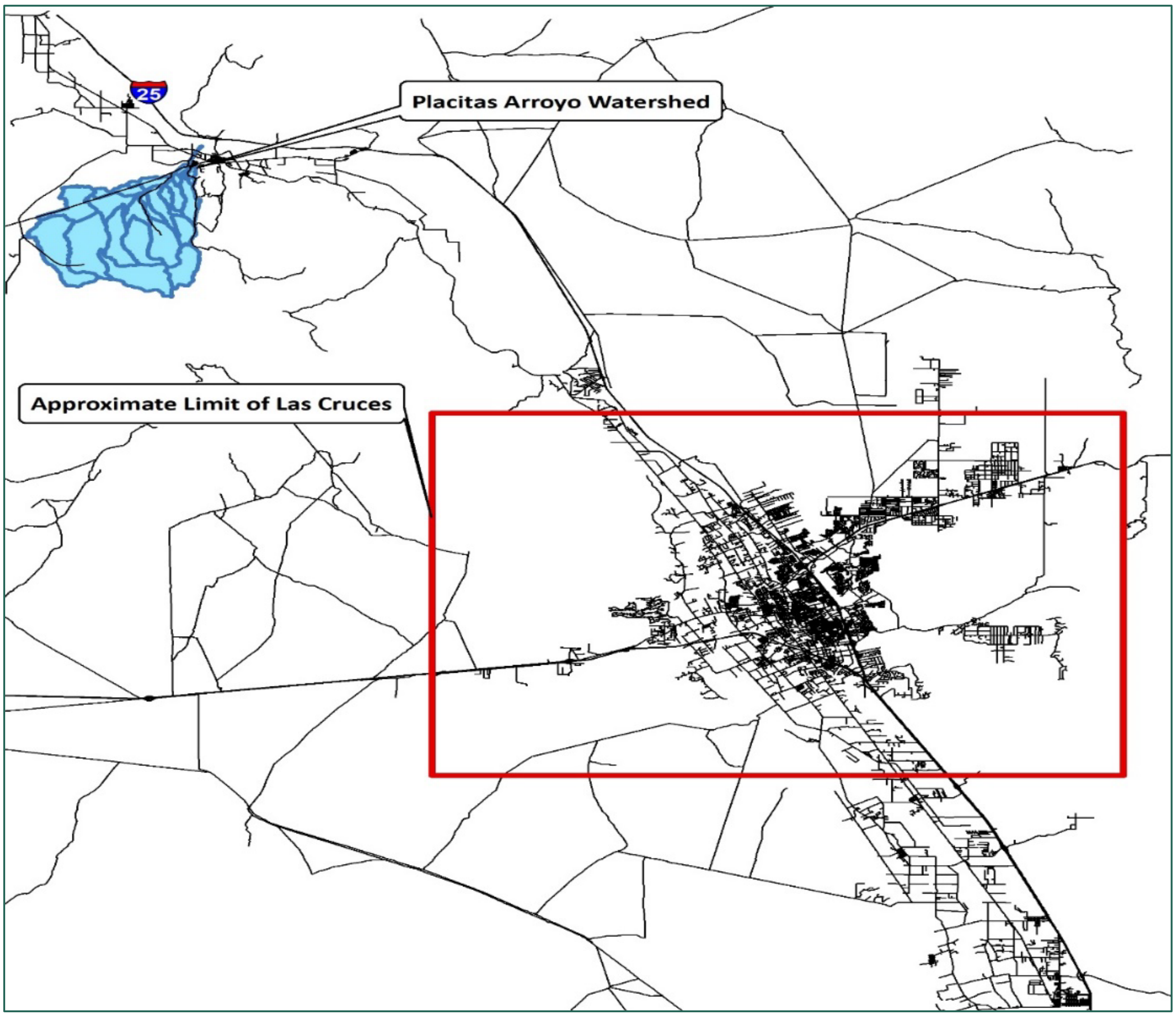


Figure 1: Project Vicinity Map

The last severe flood event occurred on August 15, 2006, when approximately 3.5 inches of rain fell on the south-southwest portions of the watershed, causing a breach in the Placitas Arroyo levee section, about 0.27 miles north of NM 26. A 6-foot flood wave washed through the village, leaving many homes, apartments, and businesses under up to 3 feet of water and sediment deposition (**Photo 1**). Numerous homes and businesses experienced flood damage; many families lost their belongings and were displaced from their homes for several months.

Photo 1: Flooding in Downtown Hatch, NM in August 2006 *(Photo: Jett Loe / Sun-News)*

Photo 2 shows a monument located outside the Village of Hatch Administrative Offices that documents the high-water mark from the flood event in 2006. The high water mark shown is approximately 22 inches.

Photo 2: Village of Hatch Municipal Building Water Mark Monument, 2006 Flood (*Photo: Jett Loe / Sun-News*)

The Placitas Arroyo (**Photo 3**) drains north from the Sierra de Las Uvas to the Rio Grande. The Placitas Arroyo Watershed has a tributary drainage area of 31.6 square miles. The downstream reach of this Arroyo lies along the western municipal limit of the Village of Hatch. The purpose of this DMP is to quantify runoff rates from the Placitas Arroyo Watershed, analyze the hydraulic characteristics of the Placitas Arroyo, identify points of restriction, and propose options that will convey the design flows safely through the VOH limits.

Photo 3: Placitas Arroyo Looking South from Approximately 0.1 Mile North of NM 187

1.2 FIELD OBSERVATION

Smith conducted several field observations in December 2017 and March 2018. The purpose of the field work was to observe the physical characteristics of the Placitas Arroyo Watershed, the channel characteristics of the Placitas Arroyo, the location of the breach in 2006, and take measurements of the various bridge crossings. **Appendix A** contains annotated photographs of the various locations in the Placitas Arroyo Watershed, existing drainage infrastructure, and bridge/culvert crossings. **Figure A1** in **Appendix A** provides an overview map of where the photos were taken. A topographic survey for the channel was also performed to obtained spot elevation and typical cross sections of the channel.

1.3 ACRONYMS

The following acronyms are common throughout the report:

BLM – Bureau of Land Management

DACFC – Dona Ana County Flood Commission

EBID – Elephant Butte Irrigation District

IBWC – International Boundary and Water Commission

OHWM – Ordinary High Water Mark

Smith – Smith Engineering Company

USACE – United States Army Corps of Engineers

VOH – Village of Hatch

SECTION 2. EXISTING HYDROLOGIC ANALYSES

2.1 PREVIOUS STUDIES

From the DACFC

In the aftermath of the flood event of 2006, the DACFC conducted several reconnaissance studies to better understand the storm event that caused the levees to breach. Independent sources were utilized to quantify rainfall and runoff from the 2006 storm event. The analysis summarized that the total rainfall depth over a 24-hour period was approximately 3.5 inches beginning on August 14th continuing to the 15th. Using the Snyder Unit Hydrograph Method, the peak discharge for the event was simulated to be around 10,000 cfs.

Larkin Group (2007), *Hydrologic and Hydraulic Analysis of Placitas Arroyo at NM 187.*

The Larkin Group performed the drainage analysis for the redesign of the NM 187 bridge. The Larkin report predicted the peak discharge for the 100-year – 24-hour storm to be approximately 11,022 cfs. The model used was HEC-1. The hydrologic analysis was based on the SCS Unit Hydrograph method which utilizes curve numbers to estimate the infiltration losses. While the new bridge has been constructed for future channel expansion, the existing channel geometry with a bottom width of 73 ft will only pass approximately 7,000 cfs.

A digital copy of this Report is included in **Appendix B**.

U.S. Army Corps of Engineers (2013), *Updated Hydraulic Evaluation of Placitas Arroyo HEC-RAS Modeling for Proposed Improvements.*

During an in-progress review discussion about the Section 205 Project, USACE South Pacific Division (SPD) requested that the Albuquerque District conduct an evaluation to determine if there will be residual flooding from Placitas Arroyo in the Village, even if the proposed flood control dam on Spring Canyon is constructed. The evaluation indicated that there could be residual flooding from the Placitas Arroyo in the Village even if the proposed project on Spring Canyon is constructed. This problem is exacerbated further given the "perched channel" configuration of the Placitas Arroyo.

The USACE study using the Snyder Unit Hydrograph method predicted a peak discharge for the 100-year – 24-hour storm to be approximately 9,550 cfs. The HEC-RAS Model developed as part of the evaluation reflected that the current conveyance capacity of the Placitas Arroyo from just upstream of NM 26 to the Rio Grande appears to handle a 10-year storm event safely. The results indicate that flows for a 25-year storm of 6,230 cfs would overtop all three existing bridges.

A digital copy of this Report is included in **Appendix B.**

Louis Berger (2016), *Final Drainage Report for Placitas Arroyo Crossing at NM 26 (CN1101270).*

This drainage report was prepared to investigate the conveyance capacity of the Bridge No. 7043 and to provide recommendations on countermeasure design options for the bridge abutments. The model used was HEC-1. Hydrologic analysis was based on the SCS Unit Hydrograph method, utilizing curve numbers to estimate the infiltration losses. The peak discharge for the 100-year – 24-hour storm was predicted to be approximately 9,532 cfs. The curve numbers utilized were based on agricultural lands and/or rangeland with greater than 65% ground cover. As such, the curve numbers utilized for weightings ranged from as low as 35 to maximum of 79. Selection of curve numbers has a tremendous impact on peak discharge. A digital copy of this Report is included in **Appendix B.**

U.S. Army Corps of Engineers (2017), *Preliminary Draft Detailed Project Report and Environmental Assessment, Continuing Authorities Program Section 205 Small Flood Risk Management Project, Hatch, New Mexico*.

Under the authority of Section 205 of the Flood Control Act of 1948, the U.S. Army Corps of Engineers (USACE) in cooperation with, and at the request of the DACFC, was authorized to study, design, and implement the Small Flood Risk Management Project that would reduce the potential for flood damage in the VOH. The proposed project consists of a flood detention structure (Hatch Dam) which would detain flows from Spring Canyon. The proposed earthen embankment dam on Spring Canyon would be located upstream of the village, adjacent to the Spring Canyon Arroyo. The dam site is just south of where the Colorado Drain and the Rodey Lateral meet.

A digital copy of this Report is included in **Appendix B.**

The record drawings for the NM 187 (Bridge No. 9447) and NM 26 (Bridge No. 7043) bridges were also reviewed. **Appendix B** includes digital copies of the reports and record drawings.

2.2 EXISTING FLOOD CONTROL STRUCTURES

There are no flood control structures present to detain flows. The existing leveed section of the Placitas Arroyo was compromised in 2006 and from field work, it appears that the channel maintains a trapezoidal geometry with varying side slopes and bottom widths due to un-engineered backfilling and repair work performed by unknown parties.

2.3 DRAINAGE BASIN DESCRIPTION

A. Drainage Basin Description Based on Historical and Existing Conditions

The Placitas Arroyo Watershed has a total drainage area of 31.6 square miles. The watershed is largely undeveloped semi-arid rangeland with poor landcover as shown in **Photo 4**. Field observation of the watershed provided evidence of a fair amount of rock and boulders present on the hillsides. The watershed's average slope was computed to be approximately 0.4 % reaching upwards of 50% on the uppermost parts of the watershed where the terrain becomes predominantly rocky outcrop. Average slopes were computed using spatial analysis of the DACFC supplied digital elevation models (DEMs) from 2014.

Photo 4: Placitas Arroyo Watershed Poor Landcover

A Historical Narrative on the Placitas Arroyo Watershed

The watershed conditions were observed based on aerial imagery from 1935 acquired from the New Mexico Resource Geographic Information System (RGIS). In 1935, the Placitas Arroyo and the Spring Canyon Arroyo formed a confluence where the existing Placitas Colonia is currently situated. There is no discernable evidence of human occupation at the confluence in 1935. NM 26 did not appear to cross the Placitas Arroyo either. Downstream of the confluence, Placitas Arroyo was basically a 1,000 ft wide floodplain. **Figure 2** shows the aerial imagery from 1935.

With the inception of the railroad, NM 26, and the settlement of the Placitas Colonia, the natural drainage path of this region was altered significantly. Spring Canyon Arroyo now drains directly into the Village of Hatch whereas the Placitas Arroyo has been constricted down to the perched leveed channel.

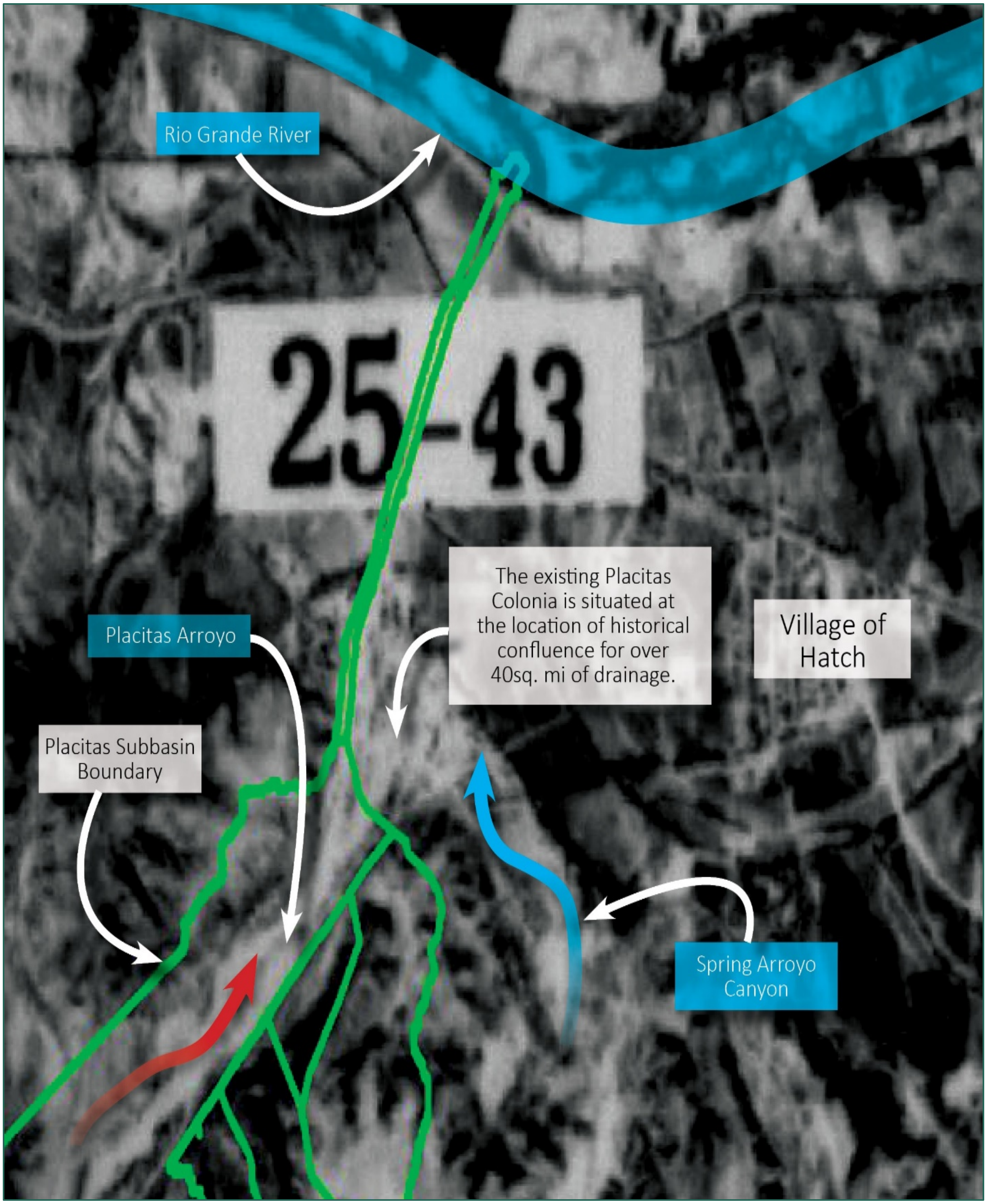


Figure 2: Village of Hatch Area 1935

Placitas Arroyo

The Placitas Arroyo is a meandering watercourse that is approximately 9 miles long. In the upper reaches of the watershed, the bottom width of the Arroyo varies from 100 to 500 ft. See **Photo 5.**

Photo 5: Upper Placitas Arroyo Upstream of NM 26

Upon crossing NM 26, the Arroyo is channelized into a trapezoidal channel (manmade) with the bottom width varying from 50 and 75 feet. The channelized portion of the Arroyo is just over a mile long from NM 26 to the Rio Grande with a mild slope of approximately 0.45 percent. This portion of the Arroyo has become perched over time due to earth levees being pushed up on both the east and west banks. The Arroyo's bed material primarily consists of a mix of clayey sand, with a high deposition of rocks. Human development has had a significant impact on the Placitas Arroyo because flows that used to spread out over hundreds of feet in the lower floodplain reaches of the Placitas Arroyo now must pass through a narrow channelized leveed section. See **Photo 6 and Photo 7**.

Photo 7: Rock Deposition on Channel Bed

Photo 6: Channelized Placitas Arroyo West of NM 187

A. FEMA Floodplains

Much of VOH is classified under FEMA Flood Zone A. FEMA floodplains were downloaded from the FEMA website. The flood map panels (dated July 6, 2016) listed here are included digitally in **Appendix B**.

- No. 35013C0400G
- No. 35013C0405G
- No. 35013C0410G
- No. 35013C0415G
- No. 35013C0420G
- No. 35013C0600G
- No. 35013C0625G

B. Existing Drainage Features

The existing leveed section of the Placitas Arroyo has three major road crossings at NM 26, NM 187 (New Mexico Department of Transportation (NMDOT) Jurisdiction), and Canal Road (Doña Ana County Jurisdiction). The NM 26 bridge, Bridge No. 7043 was built in 1968 when the road crossing was modified from a dip cross to a concrete bridge. Since Bridge No. 7043 is 50 years old, the NMDOT recently performed bridge rehabilitation on NM 26 to further extend the design life of the bridge. **NM 26 was designed to convey 4,350 cfs.** The bridge has five spans and is built at a 45-degree skew to the flow path of the Placitas Arroyo. **There is evidence of debris building up against the bridge piers as shown below.**

Photo 8: NM 26 Bridge Crossing at the Placitas Arroyo (Bridge No. 7043)

There is a pair of 48-inch culverts (**Photo 9**) crossing NM 26 located at the northeast corner of the intersection of Placitas Avenue and NM 26. Maximum headwater depth was measured. Mr. Mike Castillo, the VOH Public Works Director, mentioned that the intersection of Placitas Avenue and NM 26 usually becomes a ponding area during storm events as this area is in a low spot. **The culvert capacity of these 48-inch culverts is approximately 123 cfs.**

The next major roadway crossing structure spanning over the Placitas Arroyo is Bridge No. 9447 located on NM 187 (**Photo 10**). This bridge was constructed in 2007 to replace a timber bridge (Bridge No. 1672) and was designed to accommodate a future channel bottom width of 146 ft. However, under existing conditions, **per the as-built drawings**, **the existing bridge section can only pass 7,000 cfs.**

Photo 9: 2 X 48-inch CMP Culverts on the West Side of NM 26

Photo 10: NM 187 Bridge (Bridge No. 9447)

The final crossing structure is the Canal Road Bridge (**Photo 11**), which consists of 8 – 10 ft span X 6 ft rise concrete box culverts. The Canal Road bridge has a long history of being overtopped. The bridge is owned by Dona Ana County. **Smith estimates the existing capacity of the culverts to be around 3,200 cfs without any clogging factors applied.**

The County maintenance crews have reportedly seen the culverts clogged with debris by as much as 50%, reducing the actual conveyance capacity to as low as 1,600 cfs. Smith's preliminary hydraulic analysis shows that blockages or disruptions to flow near the Canal Road Bridge will cause a significant backwater effect at the NM 187 Bridge.

Photo 11: Canal Road Bridge

There is currently an EBID flow gage situated approximately 700 ft south of NM 187 (left side of **photo 12**, also see **photo 7**) and an irrigation siphon on the north side of Canal Road Bridge (right side of **photo 12**). Every time Canal Road Bridge is overtopped, sediment and debris are deposited into the EBID canal located just downstream of Canal Road Bridge. This has become a major maintenance concern for the EBID. See **Photo 12.**

Photo 12: Elephant Butte Irrigation District (EBID) Facilities.

2.4 DRAINAGE BASIN DELINEATION AND ANALYSIS CRITERIA

A. Drainage Basin Delineation

The overall watershed area of 31.6 square miles was subdivided into 23 subbasins as shown on **Figure 3.** Subbasins range from 0.03 square miles to 4.3 square miles. Subbasins were delineated using Arc Hydro version 10.2 and U.S. Army Corps of Engineers Hydrologic Engineering Center's Geospatial Hydrologic Modeling Extension (HEC-Geo-HMS) version 10.2, in conjunction with ESRI ArcGIS Version 10.2.2. Arc Hydro tools were used to perform spatial analysis on the 2014 digital elevation model (DEM) to derive the necessary data sets that collectively determine the drainage patterns of the watershed. The Arc Hydro tools process the terrain model, delineates the outer watershed boundary, and generates the stream network required to determine the hydrologic connectivity of the watershed. Once the terrain processing is completed, HEC-GeoHMS was used to merge further and refine subbasin boundaries. The subbasin boundaries delineated by the geospatial processing were field-verified during the site visits. **Figure A** (included in the Map Pocket) presents the subbasins in greater detail along with topographic data.

B. Storms Evaluated

The VOH and DACFC requested that 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year - 24-hour duration storms be simulated. However, the 2-year storm was added as well as to quantify sediment volumes to size sediment control facilities.

B. Design Storm

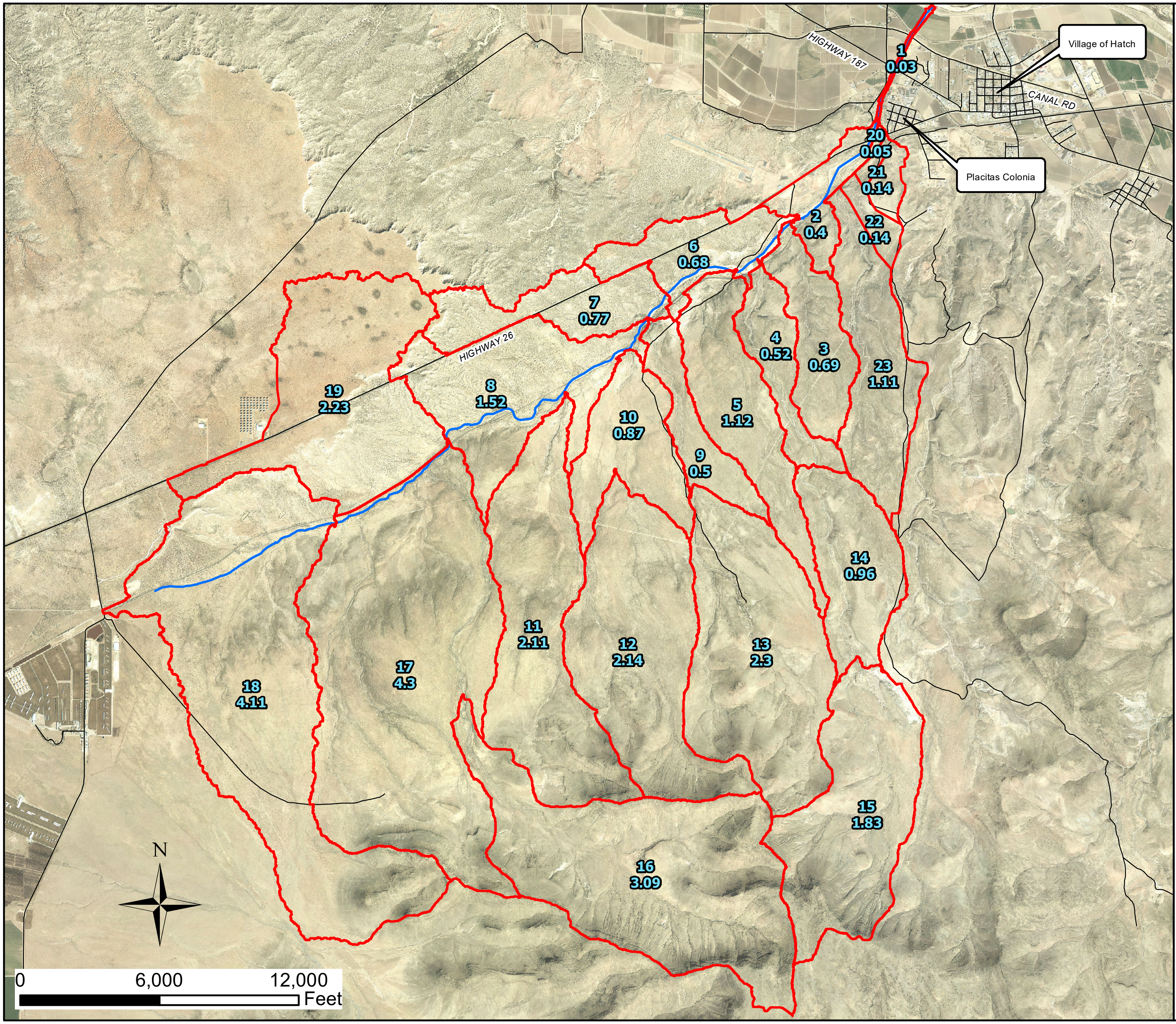
The VOH and the DACFC directed that the design storm shall be 100-year 24-hour storm.

C. Hydrologic Computer Program

The U.S. Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) Version 4.2.1 was selected for hydrologic modeling. The data derived from the geospatial processing described in section 2.4.A was directly imported into the HEC-HMS to build a comprehensive hydrologic numerical model to simulate the various return period storms and associated runoff.

C. Hydraulic Analysis Computer Program

The U.S. Army Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) Version 5.0.3 was used to perform open channel analysis for the Arroyo reach for various peak discharges. In addition to 1D hydraulic analysis, a 2-dimensional water surface model was also built to determine the inundation limits from the overtopping of NM 26, simulate the impact of inline sediment storage basins, quantify depths, velocities and shear stresses of the Placitas Arroyo under both existing and proposed conditions.



Legend

Subbasin Boundary

Placitas Arroyo

Roads

22
0.14
Subbasin ID & Area In Sq Miles

Placitas Arroyo Drainage Master Plan

Prepared for

HATCH NEW MEXICO CHILE CAPITAL OF THE WORLD

Prepared by

SMITH ENGINEERING

Figure 3 Drainage Basin Map

July 2018

Document Path: Q:\SEC---PROJECTS\117112\GIS\20171213_DrainageBasinMap.mxd

2.5 RAINFALL DATA

A. Rainfall Distribution

The Placitas Arroyo Watershed is located within the Natural Resources Conservation Service (NRCS), previously the Soil Conservation Service (SCS), Type II rainfall distribution area as defined by the NRCS. **Figure 4** illustrates the Type II boundaries. However, the DACFC directed Smith to use the 25% Frequency Storm Distribution storm.

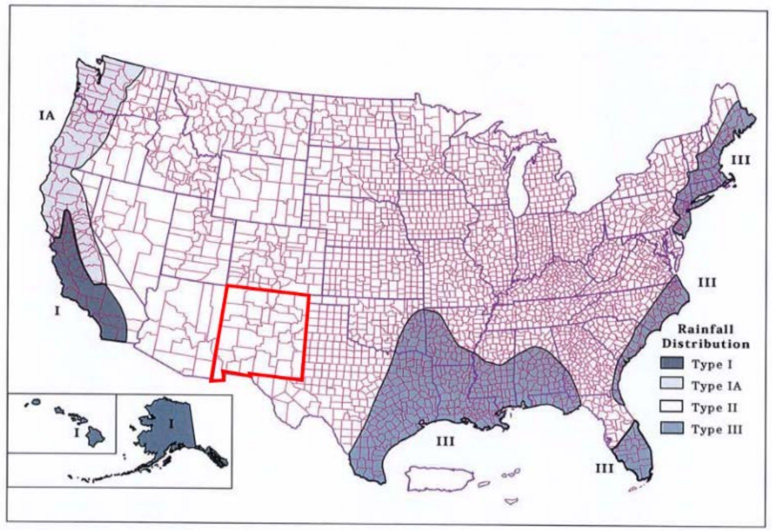


Figure 4: Approximate Geographic Boundaries for NRCS Rainfall Distribution

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. 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.

A recent document that was adopted 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., provides a graphical comparison between the different 24-hour rainfall distributions as shown in **Figure 5**. Note that the NRCS Type II 75 is very similar to the Atlas 14 Frequency 25% Weighted storm.

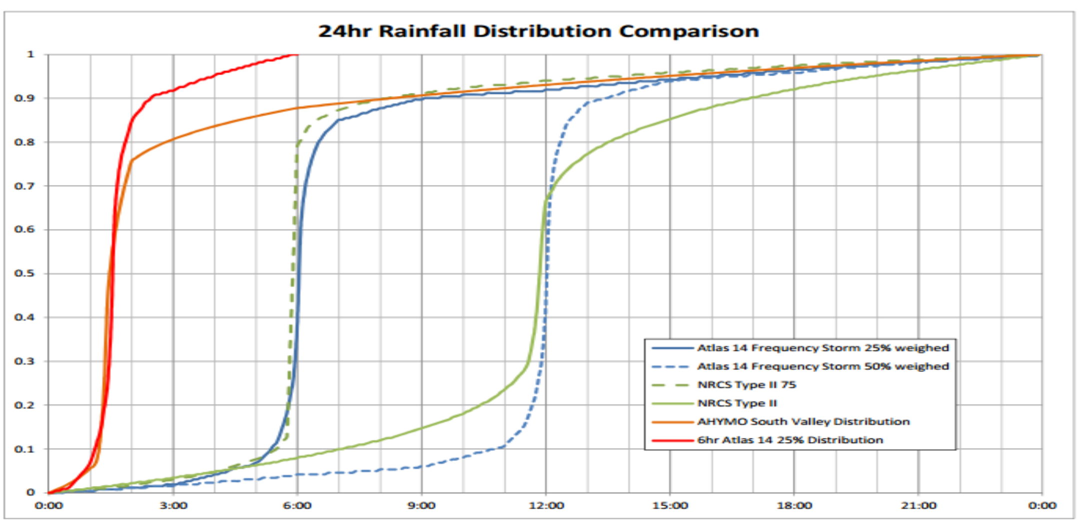


Figure 5: 24 Hour Rainfall Distribution Comparison

B. Point Rainfall Data

Point rainfall data was obtained from the NOAA Atlas 14 website. The watershed was broken up into four quadrants and point rainfall data was collected at the centroid of each quadrant. Point rainfall data was also collected at the centroid of the entire watershed. The estimated 100-year - 24-hour precipitation depth varied from 3.44 inches near the watershed centroid to 3.60 inches at the southwest portion of the watershed. Given that during the storm event of 2006, the most intense part of the storm was localized in the southwest part of the watershed, the point precipitation data for the southwest quadrant was used. **Table C1** documents the point precipitation depths used as input for the HEC-HMS model. **Appendix C** also contains the printouts from the NOAA Atlas 14 point rainfall data.

C. Areal Reduction Factors

The total watershed area is approximately 31.6 square miles. Therefore, areal reduction factors were applied as shown on the Depth-Area Curves in Figure 14, Depth-Area Curves (Miller et al., 1973) and **Table C1** both of which are included in **Appendix C.**

2.6 SOILS DATA AND RUNOFF CURVE NUMBERS (CNs)

A. Hydrologic Soil Group (HSG) and Curve Number Calculations

The NRCS runoff curve number (CN) method was used to estimate the initial abstraction loss to determine excess precipitation (direct runoff). The CN defines soil characteristics in terms of potential runoff, soil type, drainage conditions, land use, and vegetative cover. For the Placitas Arroyo Watershed, CNs were selected for arid and semi-arid rangelands as defined in Table 2-2D of the TR-55 document (Cronshey, 1986, p. 2-8). The typical cover as observed in the field was poor or less than 20%. The following photos illustrate the watershed conditions as observed in the field.

Photo 13: Looking East Towards NM 26 Bridge

Photo 14: Poor Vegetative Landcover in Watershed

Photo 15: Sparse Vegetation and Rocky Terrain in the Upper Placitas Arroyo Watershed

The CN for the subbasins are a function of the HSG, vegetation, and cover. Hydrologic Soil Groups are classified from HSG Type A to HSG Type D. HSG C and D have a slow infiltration rate (high runoff potential). Therefore, HSG C and D will generate higher levels of runoff whereas HSG A and B promote the most infiltration. Information on the watersheds soils characteristics was obtained from the Natural Resources Conservation Service (NRCS) Web Soil Surveys as follows: ***http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx.***

The information obtained from NRCS Websoil Survey was carefully evaluated against observations made in the field regarding watershed conditions and amount of vegetative cover. **Appendix C** contains a detailed soil report obtained from the NRCS Web Soil Survey information including the soil map unit locations, distribution of (HSG), and cover types for the various soil map units **Figure 6** also shows the distribution of HSG (A, B, C, and D as defined by the NRCS

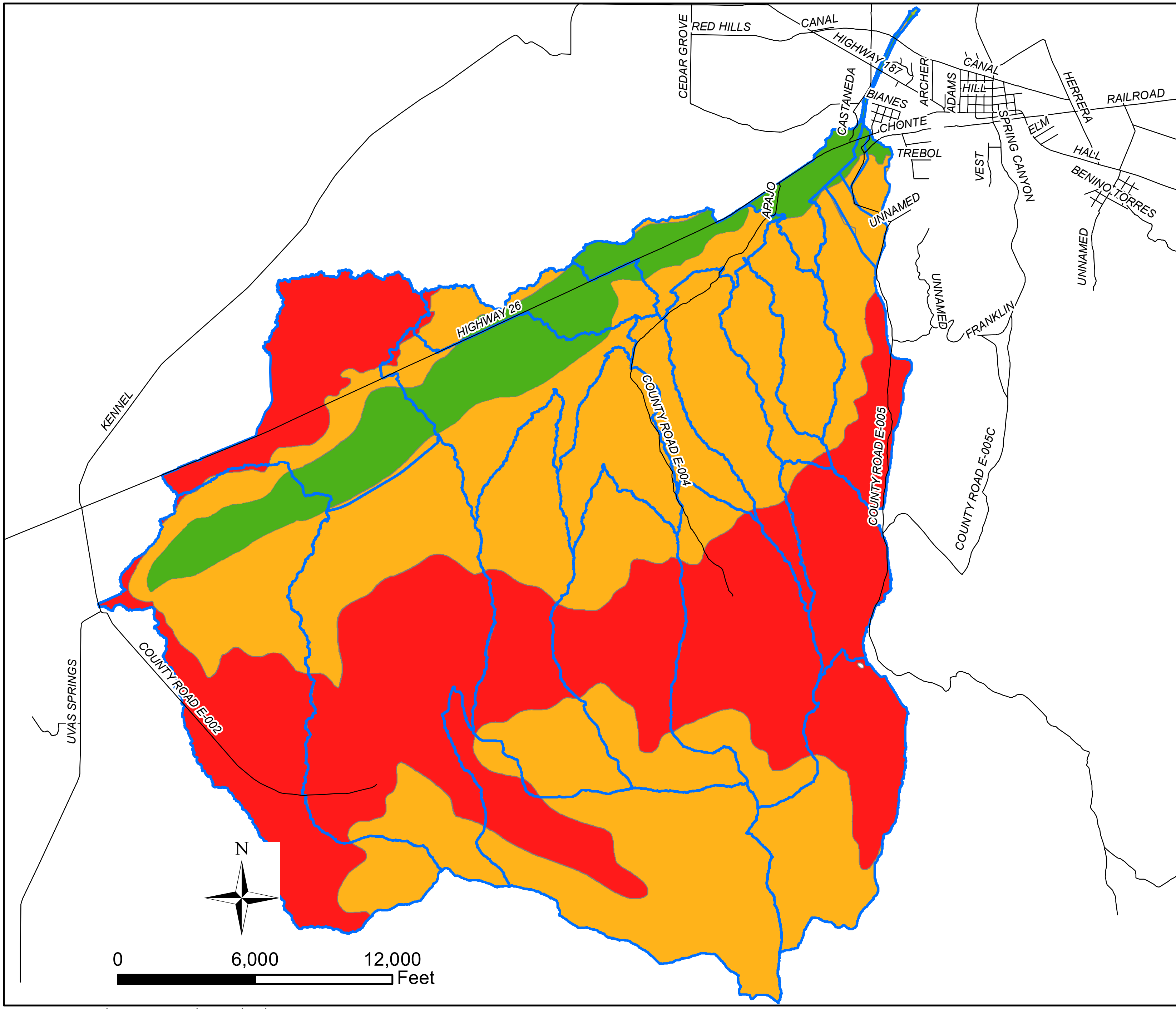
Web Soil Survey) map for the Placitas Arroyo Watershed. As shown on **Figure 6**, the Placitas Arroyo Watershed consists of approximately 80% HSG type C and D soils which provide very little initial abstraction and infiltration. The soils observed in the field also demonstrate a tendency to form a rigid hydrophobic crust when undergoing repetitive wetting and drying cycles. As such, if a large storm event occurs after several small storms, there would be significantly less infiltration due to the formation of this hydrophobic crust.

The NRCS categorizes curve numbers based on the Antecedent Runoff Conditions (ARC) also known as Antecedent Moisture Condition (AMC) of the watershed. ARC conditions represent watershed moisture conditions. The NRCS National Engineering Handbook categorizes curve numbers into three classes of Antecedent Runoff Conditions. ARC I represents dry watershed conditions. This is the least conservative scenario where there is high infiltration and low runoff. ARC II represents moderate moisture watershed conditions. This is a somewhat conservative scenario where there is medium infiltration and medium runoff. ARC III represents very moist watershed conditions. This is the most conservative scenario where there's low infiltration and high runoff. The DACFC has had numerous flood events throughout the County over the last several years. Based on their observation, most of the flood damage has occurred under ARC III conditions. However, designing flood control facilities for ARC III conditions significantly increases the footprint of the facilities, as well as the cost. As such, the DACFC directed Smith to use curve numbers based on an average of ARC II and ARC III CNs. **Table 1** documents the calculations for computing the average CN based on ARC II and ARC III.

Due to the variability in the peak discharges from the various reports done by others, Smith performed a series of sensitivity analysis to determine the difference in the magnitude in peak discharge between ARC II and Average ARC II and III curve numbers. The difference in peak discharge between ARC II and ARC III conditions is significant, particularly since the HSG is predominantly HSG C and D. In fact, it was observed for the Placitas Arroyo Watershed, that increasing the curve number by 5 (which is the average difference in curve numbers between ARC II and ARC III for this watershed) increased the overall discharge by approximately 25%. Further efforts were conducted to understand the use of the Snyder Unit Hydrograph Method that was used by the USACE in their Updated Hydraulic Evaluation of Placitas Arroyo HEC-RAS Modeling for Proposed Improvements (USACE, 2007) Study. The Snyder Method requires the initial loss parameter to be defined which determines the amount of rainfall losses and ultimately excess precipitation or direct runoff. The USACE analysis assumed a uniform initial loss of 0.9 inches for all subbasins. To determine the correlation between the initial loss (initial abstraction) and curve number, The TR-55 document (Cronshey, 1986) was reviewed, which presents several mathematical relationships between initial losses and curve numbers. **Table 5-1** in **Appendix C** presents the initial abstraction for curve numbers ranging from 40 to 98.

For an initial abstraction of 0.9 inches, the corresponding curve number is approximately equal to 68. Referring to Table 2-2d, the corresponding HSG for desert shrub would be an HSG B with a good hydrologic condition. The Placitas Arroyo Watershed does not fit in that category of land cover or hydrologic soil group and is unlikely to experience that level of initial abstraction due to the watershed conditions described earlier. However, it does explain why the USACE and Louis Berger models predicted significantly lower 100-year-24-hr peak discharges of around 9,500 cfs. The Larkin report from 2007, on the other hand, adopted an average curve number of 89 with poor conditions and HSG C and D soils. The Larkin report only delineated 10 subbasins where one subbasin was almost 14 square miles or almost 50% of the entire watershed. Smith delineated 23 subbasins with the largest subbasin approximately 4 square miles. The areal distribution of subbasins and the length of the time of concentration flow path for subbasins have a significant impact on the peak discharge.

The tools and techniques available for performing spatial analysis on soils distribution have evolved tremendously since 2007 which allows for more refined curve number computations. This explains the difference in the peak discharges on record for the Placitas Arroyo Watershed.



Legend

Roads

Subbasin Boundary

Hydrologic Soils Group

A

B

C

D

Placitas Arroyo Drainage Master Plan

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Figure 6 Hydrologic Soil Groups

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Table 1: Runoff Curve Number Assumptions and Calculations

Basin No.	Basin Area	Basin Area	Area of HSG A	Area of HSG B	Area of HSG C	Area of HSG D	Runoff Curve Number for HSG A	Runoff Curve Number for HSG B	Runoff Curve Number for HSG C	Runoff Curve Number for HSG D	Basin Description	Areal Weighted CN	Runoff Curve Number Based on AMC II Conditions	Runoff Curve Number Based on AMC III Conditions	Runoff Curve Number Based on Average between AMC II & AMC III
	sq mi	acres	acres	acres	acres	acres									
1	0.0287	18.35	14.37	2.16	1.83	0.00	63	77	85	88	Desert Shrub - Poor Condition	67	67	83	75
2	0.4190	268.16	131.18	0.00	81.37	0.00	63	77	85	88	Desert Shrub - Poor Condition	57	57	78	67
3	0.6897	441.42	0.00	0.00	441.42	0.00	63	77	85	88	Desert Shrub - Poor Condition	85	85	94	90
4	0.5177	331.32	0.00	0.00	320.73	10.59	63	77	85	88	Desert Shrub - Poor Condition	85	85	94	90
5	1.1217	717.87	0.00	0.00	623.27	94.60	63	77	85	88	Desert Shrub - Poor Condition	85	85	94	90
6	0.6757	432.47	238.24	0.00	194.23	0.00	63	77	85	88	Desert Shrub - Poor Condition	73	73	87	80
7	0.7654	489.85	255.94	0.00	218.99	14.92	63	77	85	88	Desert Shrub - Poor Condition	74	74	88	81
8	1.5178	971.40	462.97	0.00	497.30	11.13	63	77	85	88	Desert Shrub - Poor Condition	75	75	88	81
9	0.4989	319.31	0.00	0.00	316.88	2.44	63	77	85	88	Desert Shrub - Poor Condition	85	85	94	90
10	0.8746	559.77	0.00	0.00	559.77	0.00	63	77	85	88	Desert Shrub - Poor Condition	85	85	94	90
11	2.1132	1352.47	0.00	0.00	930.92	421.55	63	77	85	88	Desert Shrub - Poor Condition	86	86	94	90
12	2.1397	1369.40	0.00	0.00	704.77	664.64	63	77	85	88	Desert Shrub - Poor Condition	86	86	94	90
13	2.3007	1472.46	0.00	0.00	366.82	1105.64	63	77	85	88	Desert Shrub - Poor Condition	87	87	95	91
14	0.9573	612.66	0.00	0.00	20.32	592.33	63	77	85	88	Desert Shrub - Poor Condition	88	88	95	91
15	1.8326	1172.84	0.00	0.00	610.71	562.13	63	77	85	88	Desert Shrub - Poor Condition	86	86	94	90
16	3.0929	1979.45	0.00	0.00	1540.63	438.82	63	77	85	88	Desert Shrub - Poor Condition	86	86	94	90
17	4.3027	2753.73	0.00	0.00	1184.49	1569.24	63	77	85	88	Desert Shrub - Poor Condition	87	87	95	91
18	4.1079	2629.07	324.61	0.00	1115.33	1189.13	63	77	85	88	Desert Shrub - Poor Condition	84	84	93	88
19	2.2320	1428.48	350.64	0.00	191.00	886.84	63	77	85	88	Desert Shrub - Poor Condition	81	81	92	87
20	0.0330	21.12	7.93	0.00	13.19	0.00	63	77	85	88	Desert Shrub - Poor Condition	77	77	86	81
21	0.1439	92.10	17.35	0.00	74.74	0.00	63	77	85	88	Desert Shrub - Poor Condition	81	81	92	86
22	0.1423	91.07	4.50	0.00	86.58	0.00	63	77	85	88	Desert Shrub - Poor Condition	84	84	93	88
23	1.1049	707.12	2.31	0.00	358.95	345.86	63	77	85	88	Desert Shrub - Poor Condition	86	86	94	90

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

The TR-55 method (Cronshey, 1986, pp. 3-1 – 3-4) was used to compute the time of concentration for the subbasins in the Placitas Arroyo Watershed. 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 subbasins in the Placitas Arroyo 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 subbasin to the point of interest or the outlet of the subbasin. 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 previously described.

The longest flow path for each subbasin was generated by the HEC-GeoHMS. 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 were chosen based on guidance provided in Open Channel Hydraulics (Chow, 1959). Copies are included in **Appendix C**.

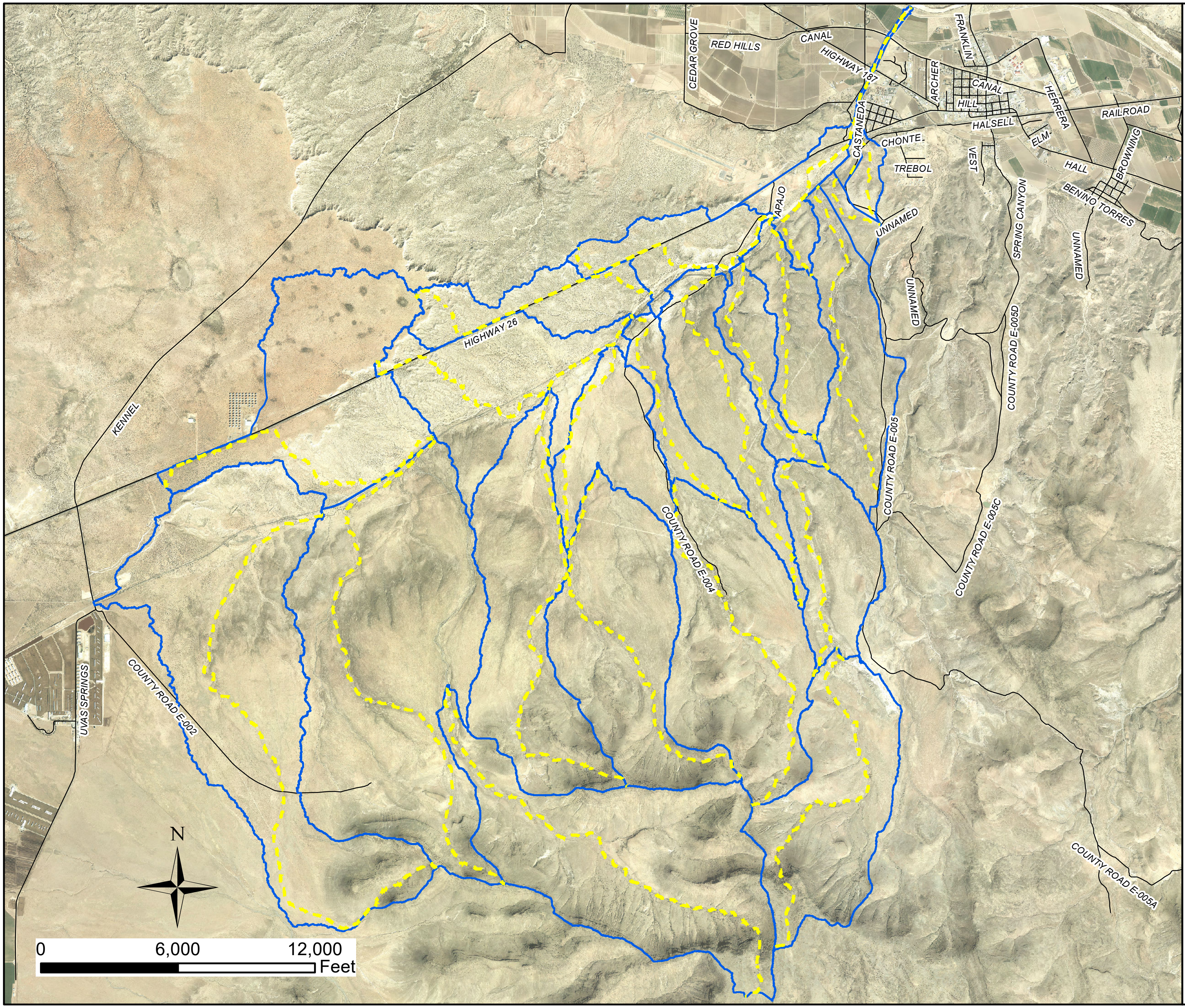
Table C3 (**Appendix C**) summarizes the travel time, time of concentration, lag time data and results. **Figure A** in the Map Pocket shows the longest flow paths delineated for all the subbasins. See **Figure 7** for a graphical representation of the delineated Tc flow paths for the Placitas Arroyo Watershed.

2.8 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. Runoff losses to channel bed infiltration and percolation were assumed to be small and were not simulated. See **Figure 8** for routing reaches.

2.9 SEDIMENT BULKING

The HEC-HMS models simulate clear water hydrographs unless a "Flow Ratio" is applied to simulate sediment volume within hydrographs. This parameter is also called sediment bulking. A sediment bulking value of about 19% is considered the limit before mud flow would occur. Several regional sediment yield documents were reviewed. Based on annual sediment yields, a 4% value was allocated for sediment bulking. A detailed explanation of how this value was computed is provided in **Table C7**, included in **Appendix C.**



Legend

Subbasin Boundary

Tc Flowpath

Roads

Placitas Arroyo Drainage Master Plan

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Figure 7

Tc Flowpaths

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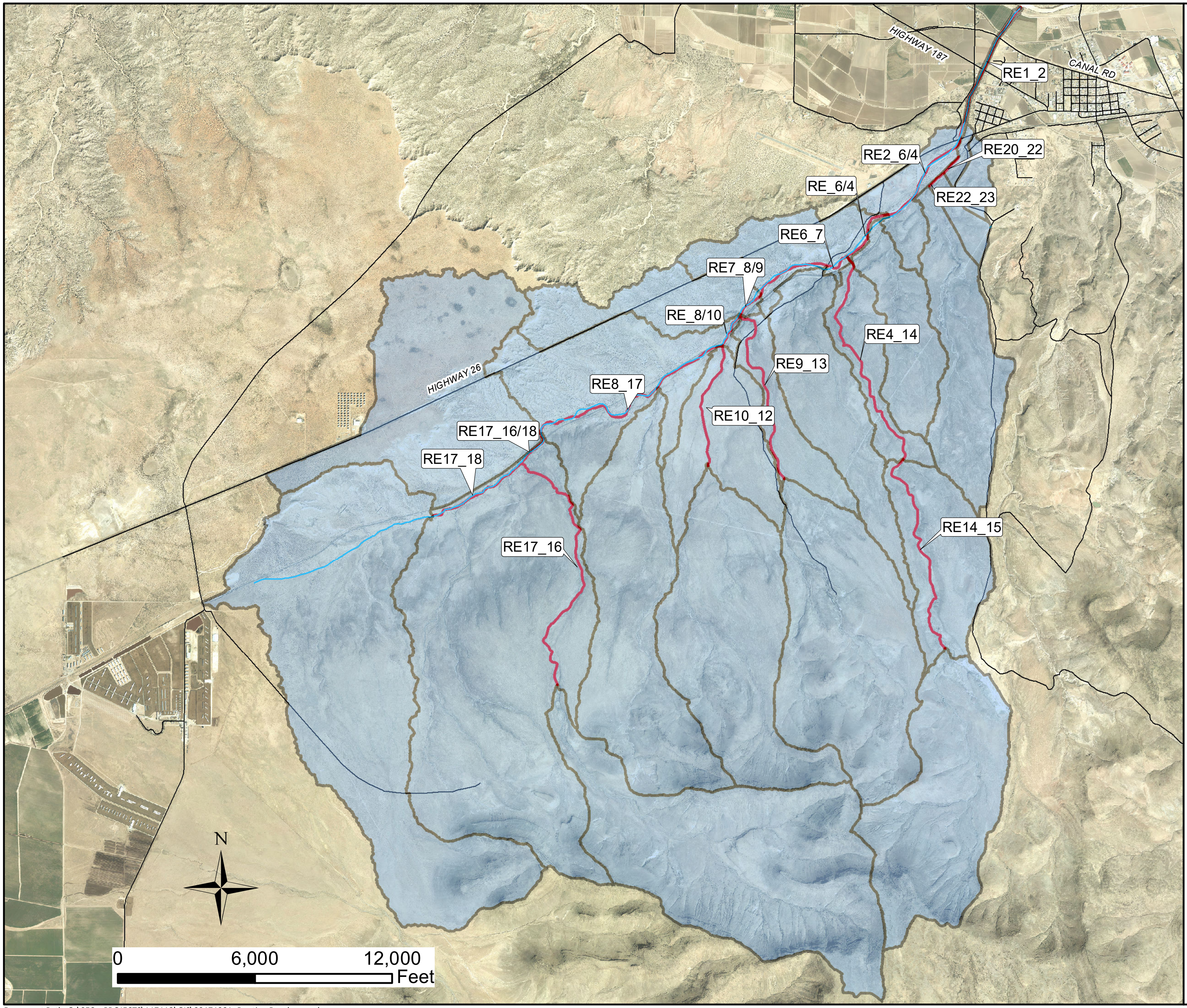

Legend

- Subbasin Boundary
- Placitas Arroyo
- Routing Reaches
- Roads

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Figure 8
Routing Reaches

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2.10 COMPUTATION TIME INCREMENT FOR HEC-HMS MODELS

When studying large watersheds where the time of concentration tends to be longer, the computational time increment is an important parameter. As such, HEC-HMS recommends using a time increment that is 17% of the time of concentration.

The average time of concentration for subbasins in the Placitas Arroyo Watershed was 76 minutes which equates to a computational time step of 13 minutes. HEC-HMS allows the users to select a default value of either 10 or 15 minutes. In this case, a 15-minute time increment was selected.

2.11 SUMMARY OF HYDROLOGIC ANALYSIS

The following table summarizes the predicted peak discharges for the Placitas Arroyo Watershed and provides a comparison with other reports that have been prepared. Based on final direction from the DACFC and VOH, the peak flows from the average ARC II and III conditions model will be used for design flows.

Table 2: Summary of Peak Discharge from Previous Reports and Existing Analysis

Summary of Peak Discharge for the Placitas Watershed Placitas Arroyo Drainage Master Plan							
Report	Basin Area	Peak Discharge 2 Yr - 24 Hr	Peak Discharge 5 Yr - 24 Hr	Peak Discharge 10 Yr - 24 Hr	Peak Discharge 25 Yr - 24 Hr	Peak Discharge 50 Yr - 24 Hr	Peak Discharge 100 Yr - 24 Hr
	sq mi	cfs	cfs	cfs	cfs	cfs	cfs
Larkin Group (2007). *Hydrologic and Hydraulic Analysis of Placitas Arroyo at NM 187.*	31.60	2,841	-	5,878	-	9,599	11,022
U.S. Army Corps of Engineers (2013). *Updated Hydraulic Evaluation of Placitas Arroyo HEC-RAS Modeling for Proposed Improvements.*	31.60	-	-	3,992	6,230	7,854	9,550
Louis Berger (2016). *Final Drainage Report for Placitas Arroyo Crossing at NM 26 (CN1101270).*	31.60	849	1,398	3,364	5,385	7,235	9,532
Smith Engineering Company (2018). *Placitas Arroyo Drainage Master Plan Using ARC II Curve Numbers.*	31.60	2,393	4,545	6,464	9,200	11,623	14,214
Smith Engineering Company (2018). *Placitas Arroyo Drainage Master Plan Using Average ARC II & III Curve Numbers.*	31.60	3,886	6,594	8,875	12,148	14,824	17,594

The curve number assumptions, rainfall distribution, areal uniformity of subbasins and the variance in the length of the time of concentration for subbasins are all factors that have varied significantly between the studies listed in Table 2. These are all crucial factors that have had a significant impact on the differences in peak discharge for the Placitas Arroyo Watershed. The Placitas Arroyo Watershed has always been a conveyance-based watershed. Excess runoff has always drained to the Rio Grande with little or no detention. The direct runoff volumes and the sediment volumes generated from the watershed are extremely high.

For instance, the 2-year – 24-hour volume at Junction J2, just upstream of NM 26, is approximately 1,151 ac-ft. Out of this direct runoff volume, approximately 4% is sediment yield which equates to about 46 ac-ft. To have any kind of peak discharge reduction will require several large and very expensive jurisdictional dams. **Appendix E** contains the HEC-HMS models along with the output summary tables from HEC-HMS.

SECTION 3 HYDRAULIC ANALYSIS OF THE PLACITAS ARROYO

3.1 GENERAL ASSUMPTIONS AND LIMITATIONS FOR HEC-RAS HYDRAULIC MODELS

Due to the complex hydraulic nature of the Placitas Arroyo, both 1D HEC-RAS and 2D HEC-RAS were utilized. General assumptions for modeling are described below. Please refer to the HEC-RAS User Manual for detailed explanations. HEC-RAS Version 5.0.3 was used to perform hydraulic analysis of the Placitas Arroyo. HEC-RAS 1D requires several data inputs that are utilized to perform hydraulic analysis as discussed below:

- River Reach: The centerline of the channel being evaluated must either be defined manually or extracted from a DEM for existing proposed terrain data.
- Cross Section Data: Cross section data defines cross sectional geometry of the channel. This should reflect any low flow channels, overbanks, floodplains, and levees. Cross section data requires the user to define where the channel bank stations are and how the Manning's 'n' values vary throughout the cross section. Cross section data is extracted from either existing or proposed terrain data.
- Geometry Data for Hydraulic Structures: If the reach being studied has bridges or culverts, these can be defined within the software. Information for structures can be obtained from record drawings, survey data, and supplemented with field measurements.
- Flow Data: Flow can be specified as steady flow (single peak discharge value), quasi-steady flow (discretized hydrograph required only for sediment transport analysis) or an unsteady flow discharge (a flow hydrograph with respect to time).
- Boundary Conditions: Required to establish starting water surface elevations at the ends of the river system to successfully perform computational analysis. Assorted options are available. However, for this analysis normal depth was selected. The model uses the defined boundary conditions to solve the Energy Equation iteratively in what is called the "standard step method".
- Flow Regime: 1D HEC-RAS demonstrates instability while performing an unsteady flow analysis if the flow regime is at critical depth or at super critical depth. The HEC-RAS solver was developed to deal with primarily sub-critical flows.
- 1D HEC-RAS will allow only a maximum of 10 culvert structures at any given roadway/bridge crossing.
- 1D HEC-RAS does not account for the effects of momentum around sinuous channels as it is primarily designed to solve the Energy Equation where the solutions are averaged between defined cross-section. As such, engineering judgement needs to be exercised in determining when a 2D analysis is more appropriate

to account for factors associated with momentum and super elevation in a sinuous channel to appropriately predict shear stresses and velocities around bends.

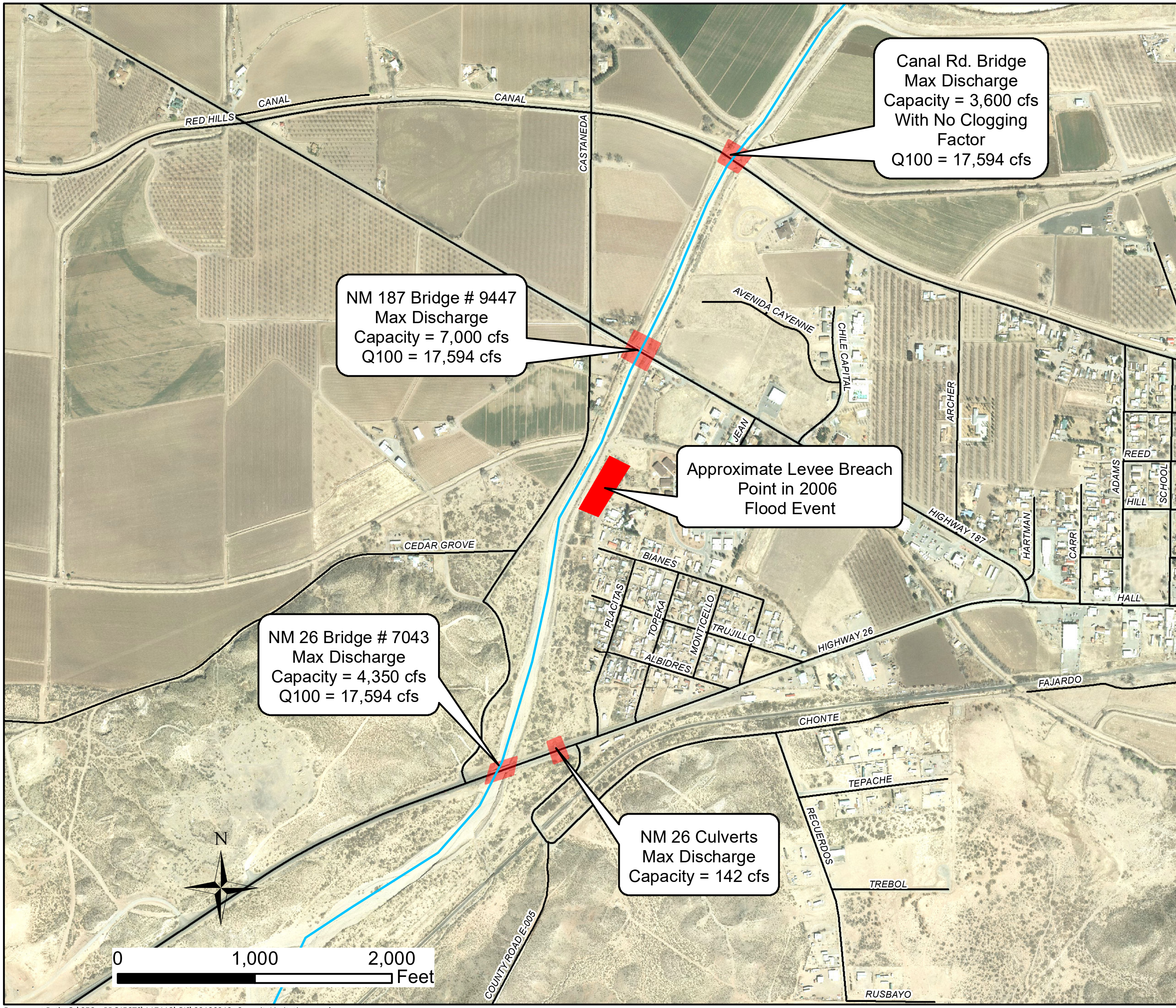
3.2 EXISTING CONDITIONS HEC-RAS MODEL

The existing conditions model was developed. The results confirmed the findings from previous studies that bridge crossings at NM 26 and NM 187 are severely under capacity. The box culverts at Canal Road were not incorporated into the HEC-RAS model. Culvert analysis was performed independently to determine the structures maximum conveyance capacity. The existing capacity of the 8 box culverts is approximately 3,600 cfs without applying any debris clogging factor. The NMDOT typically requires a 15% clogging factor. The culverts, therefore, fail at the 2-year storm event. The hydraulic analysis of Placitas Arroyo indicates that the leveed section of the channel, due to its mild slope, has very sensitive hydraulic characteristics. The Canal Road box culverts create a severe backwater effect that completely destabilizes the channels conveyance. As the box culverts become plugged with debris, the flow velocity of the channel upstream drops significantly, causing larger suspended solids to settle. The settlement also reduces the available flow area upstream. This is particularly detrimental for the bridges' upstream that are already below capacity. Consequently, the water surface elevation in the channel begins to rise. The following table provides a summary of water surface elevations at the bridge crossings for existing conditions. The values shown in red indicate all water surface elevations that exceed the high chord elevations of the bridge.

Table 3: Summary of Existing Bridge Hydraulic Capacity

SUMMARY OF BRIDGE HYDRAULIC CAPACITY Placitas Arroyo Drainage Master Plan											
Bridge Name/Location a	Bridge Number a	Design Capacity cfs a	Skew Degree a	Average High Chord Elevation @ Roadway Centerline ft	Average Low Chord Elevation @ Roadway Centerline ft	Water Surface Elevation 2Yr-24Hr ft b	Water Surface Elevation 5 Yr-24Hr ft b	Water Surface Elevation 10 Yr-24Hr ft b	Water Surface Elevation 25 Yr-24Hr ft b	Water Surface Elevation 50 Yr-24Hr ft b	Water Surface Elevation 100 Yr-24Hr ft b
NM 26	7043	4350	45.00	4093.92	4092.42	4089.00	4091.80	4093.69	4095.30	4094.80	4096.52
NM 187	9447	7000	0.00	4076.80	4075.30	4073.95	4079.32	4081.13	4083.37	4084.87	4086.13

Placitas Arroyo gets overtopped between the 10 and 25-year storms. Once breached, the flows inundate the Village of Hatch which occurred in the 2006 flood event. The capacity of the leveed section of the Placitas Arroyo through the Village of Hatch is completely insufficient to convey flows greater than 9,000 cfs. The backwater effect from Canal Road Bridge and NM 187 creates a breach point exactly where the levee failed in 2006. Based on analysis of the existing conditions, **Figure 9** shows the locations that act as choke points in the leveed section of the Placitas Arroyo. Since a steeper slope is impossible to attain, the only other alternative is to provide a larger cross-sectional area to safely convey design flows. **Appendix F** contains the 1D HEC-RAS model as well as the output for the 100-yr-24-hr analysis. This includes the existing cross section plots and profile plots for the Placitas Arroyo.



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Placitas Arroyo

Capacity Limits

Roads

Placitas Arroyo Drainage Master Plan

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Figure 9
Capacity Limitations of Placitas Arroyo

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3.2 HEC-RAS 2D MODELING ASSUMPTIONS

Placitas Arroyo is sinuous with a severe contraction as the channel approach transitions under Bridge No. 7043. Based on the 1D analysis, there is significant overtopping over the bridge high chord. The only way to model and understand the implications of these factors was to develop a 2D model. A 2D surface water model can predict flow depths, velocities, and surface flow direction, based on analysis of existing or proposed terrain as a function of flow hydrographs. The inundation from the bridge overtopping needed to be studied to determine if the Placitas Colonia could be inundated during a large storm event. The following flowchart describes the general process to build a 2D model in HEC-RAS.

2D Modeling Process Using HEC-RAS 2D, V 5.0.3

1- Terrain preprocessing to create appropriate terrain file formats from the Digital Elevation Model (DEM)

2- Create appropriate geometry file in HEC-RAS using terrain generated from step 1

3- Create 2D polygon for modeling limits, generate 2D mesh & refine 2D mesh using breaklines

4- Create spatially varied roughness Manning's roughness layer using 2011 NLCD data and add user defined polygons to refine Manning's roughness layer

5- Adding necessary internal hydraulic structures (Not Applicable)

6- Create external 2D boundary conditions and associate appropriate inflow hydrographs flow files

7- Create plan with appropriate initial conditions and run simulation

8- Review stability, computational log file, and results in RAS Mapper

A. 2D Mesh Generation

Terrain preprocessing as outlined in Chapter 2 of the HEC-RAS User Manual was performed, after the data was incorporated as part of the geometry file into the HEC-RAS model. Using the bounding polygon, a 2D mesh (**Photo 16**) was generated that consists of grids that are defined by the user to be a certain size. The adaptive mesh feature was chosen to determine grid size. 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. The 2D mesh was then saved as a geometry file to be used within the HEC-RAS.

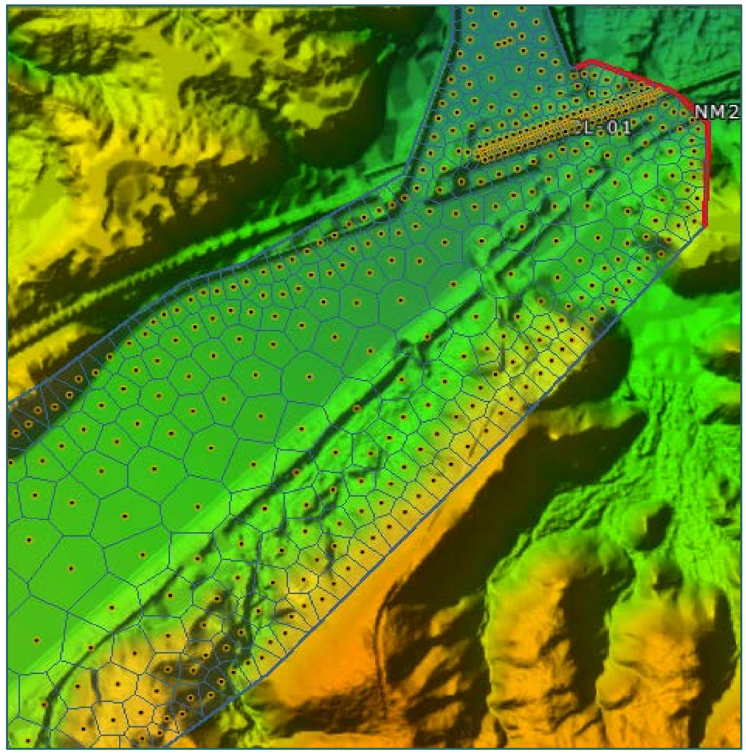


Photo 16: 2D Mesh Generated for NM 26

B. External 2D Flow Area Boundary Conditions

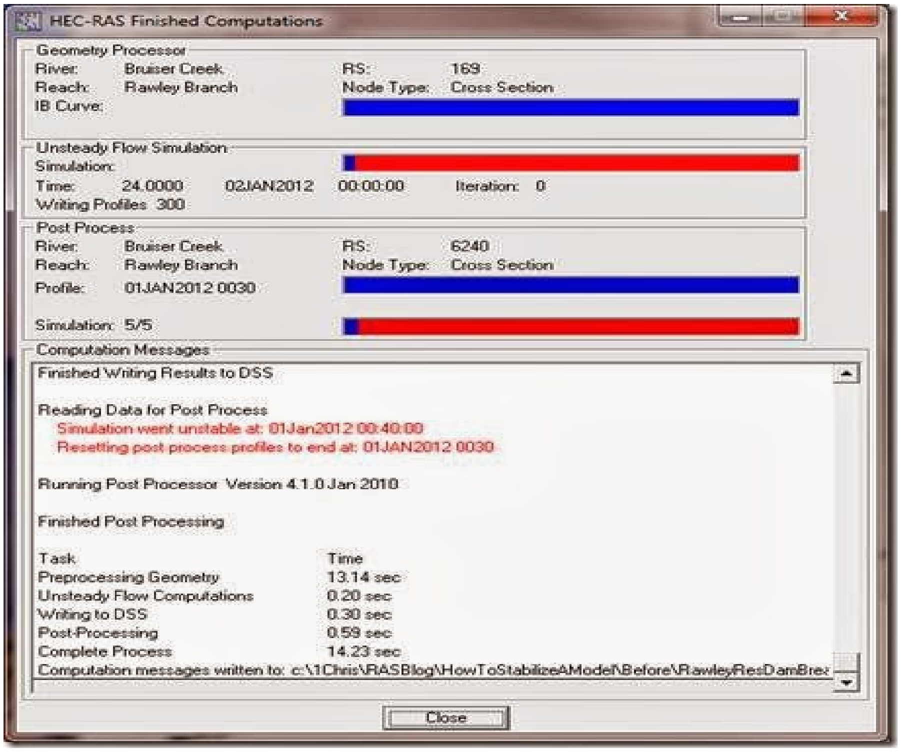
The 2D flow area must have upstream and downstream boundary conditions specified. For areas where flow leaves the model, normal depth was specified. Since the downstream areas are typically flat agricultural fields, a typical energy slope of 0.5% was specified. The upstream boundary conditions simulate locations where flows are added into the mesh. The hydrographs from the HEC-HMS hydrologic model, at Junction J2, were used for the inflow hydrograph.

C. 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 HEC-RAS 2D on when to use the Full Momentum equation vs. Diffusion Wave. In this instance, the full momentum was used. Based on the guidelines for Full Momentum equation, a time step of 1 minute was selected. At this point, the hydraulic properties for the cells within RAS Mapper were computed.

D. 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 the RAS Mapper. The ones 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 become unstable, and a message appears as shown.



In this case, the above window did not occur proving the model was performing the computations and achieving convergence for all the cells. Upon completing the simulation successfully, this window opens 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. The analysis 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 Village of Hatch 2D model had errors below 0.5% which is acceptable. The log should look like the image below:

Volume Accounting for 2D Flow Area in Acre Feet

2D Area	Starting Vol	Ending Vol	Cum Inflow	Cum Outflow	Error	Percent Error
FLOWAREA		11.09	46.08	35.15	0.1655	0.3592

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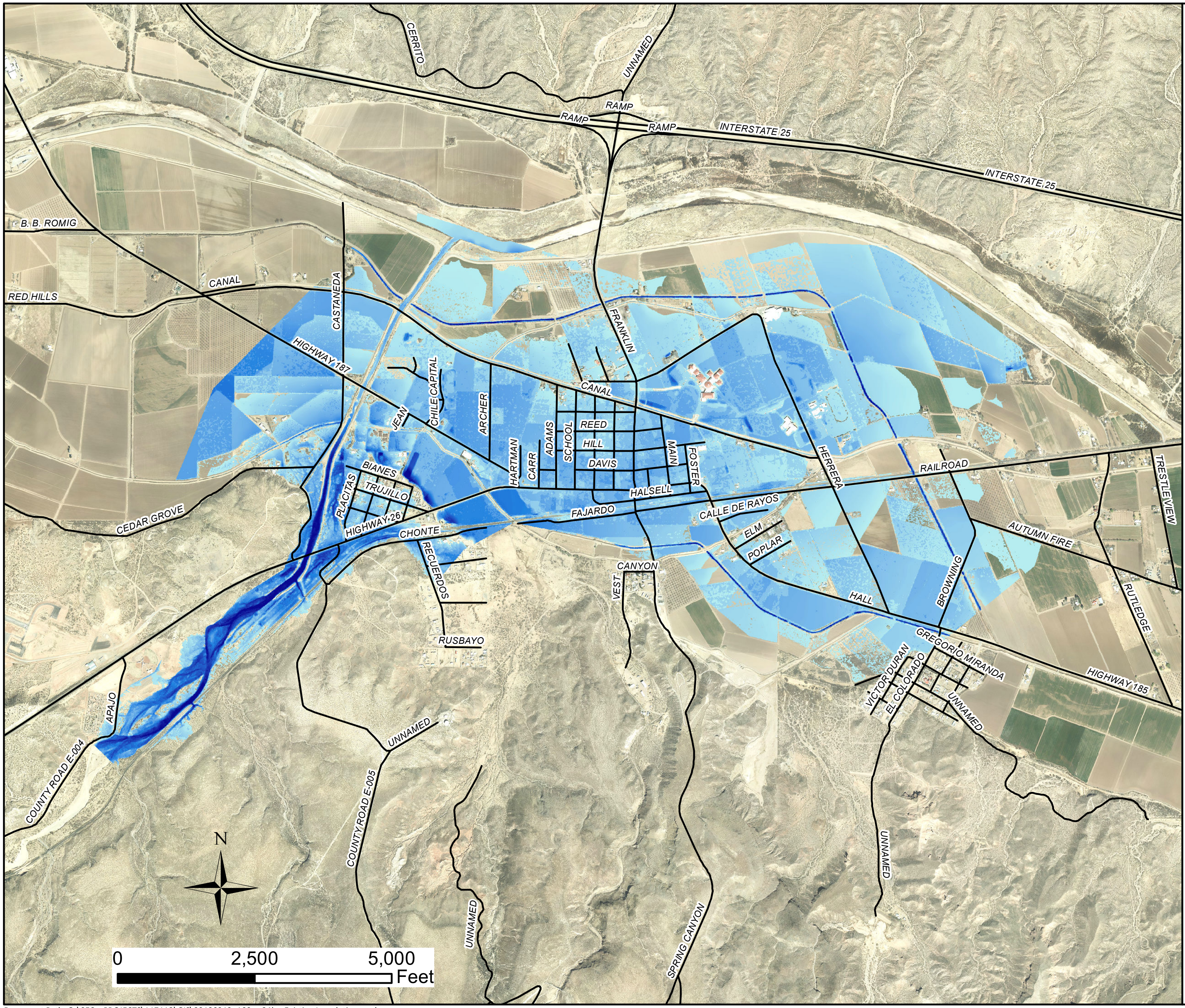
3.3 CONCLUSIONS FROM EXISTING CONDITIONS HYDROLOGIC AND HYDRAULIC ANALYSIS

The Placitas Arroyo Watershed has always been a conveyance-based watershed. Predevelopment evidence reveals that a large floodplain at the base of the watershed allowed the large runoff rates to drain to the Rio Grande unimpeded. Post-development conditions have constricted the Placitas Arroyo to a narrow channel that has been bermed up to form a perched leveed channel. The existing Placitas Arroyo through the limits of the VOH is incapable of conveying flows greater than 9000 cfs. The levees are un-engineered and have breached in the past, leading to severe flooding and will continue to do so until engineered facilities are constructed. The lack of a properly designed conveyance facility will continue to pose a serious flooding hazard for the Village. The channels lack of capacity is further exacerbated by Canal Road box culverts and in larger storm events, NM 187. Inundation from the 2D model is shown based on 100-yr-24-hr flows in **Figure 10.** The inundation map also shows that overtopping from the NM 26 bridge will cause surface flows to drain into the Placitas Colonia.



Legend

Roads

Depth

High : 12.3 (ft)

Low : 0.50 (ft)

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Figure 10 Existing Conditions Inundation From 100yr-24hr Storm

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SECTION 4. PROPOSED HYDROLOGIC AND HYDRAULIC ANALYSES

4.1 OVERVIEW OF PROPOSED IMPROVEMENTS

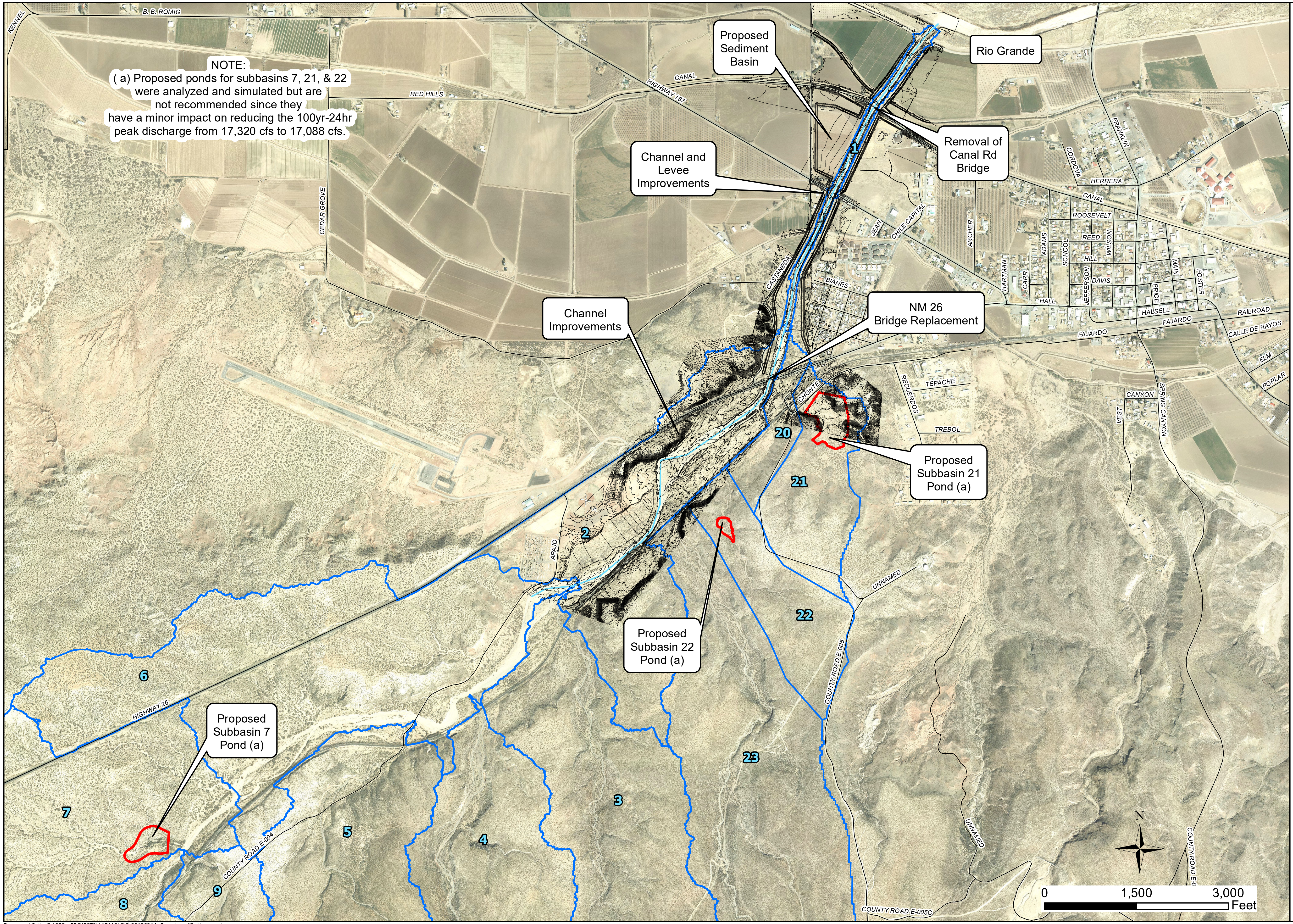
The magnitude of direct runoff reaching the Placitas Arroyo leaves very few options to provide effective flood control. The two main options evaluated were detention facilities and conveyance facilities. Three proposed ponds were simulated in subbasins 7, 21, and 22. The total volume of all three ponds was about 76 ac-ft. All ponds were designed to be below jurisdictional limits with embankment heights less than 6 feet and storage volumes less than 50 ac-ft. With the initial pond routings, the peak 100-yr-24-hr discharge dropped from 17,320 cfs to 17,088 cfs at the Rio Grande.

These pond routings proved that several jurisdictional dams would be required to provide any meaningful reduction in peak discharge. The DACFC in conjunction with the VOH and USACE are currently in the process of designing a jurisdictional dam called the 'Hatch Flood Control Dam'. This dam will control the 8 square mile drainage area for Spring Canyon. The approximate cost of this facility is expected to be around $12 million. As a rule of thumb for gravity-based detention facilities, to achieve meaningful attenuation, the target is to design a facility that has a design volume of approximately 60% of the watersheds direct runoff volume. The direct runoff volume for the Placitas Watershed is approximately 4062 ac-ft. The detention facilities would have to be designed to have a total design volume of 2437 ac-ft to achieve reasonable attenuation. That would equate to at least 10 jurisdictional dams of approximately 243 ac-ft each.

Hypothetically, if each structure cost approximately $12 million, the minimum total project cost would be $120 million. For a small community like Hatch, it would be impossible to meet the cost to benefit ratio for these structures. Apart from the cost factor, the railroad runs parallel to the Placitas Arroyo. Due to the railroad's proximity to the Arroyo, it would be difficult to design a large flood control dam in the main channel without posing a risk to the railroad. The upper watershed has many archeological/cultural sites that are protected by the Bureau of Land Management. Considering all this information, flood control dams were not explored any further. Since these ponds were simulated but are not recommended, the stage-storage-discharge tables, conceptual grading plans and reservoir routing summary tables for these ponds are included in **Appendix D**.

Figure 11 shows an overview of all the options that were considered.

Various configurations of proposed conveyance facilities were graded in Civil 3D. The proposed surfaces were imported into HEC-RAS and proposed models were simulated with varying bottom widths, side slopes, and levee heights. Since the grade from NM 26 to the river is fixed at approximately 0.45%, the only option to improve conveyance through the Village limits was to widen the leveed section of the arroyo and raise the levee height to maintain freeboard. **Figure 12** shows the overview of the proposed conveyance alternative.



Legend

- Proposed Contours
- Proposed Channel Centerline
- Subbasin Proposed Pond Locations
- Roads
- Subbasin Boundary

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Figure 11 Overview of Proposed Improvements

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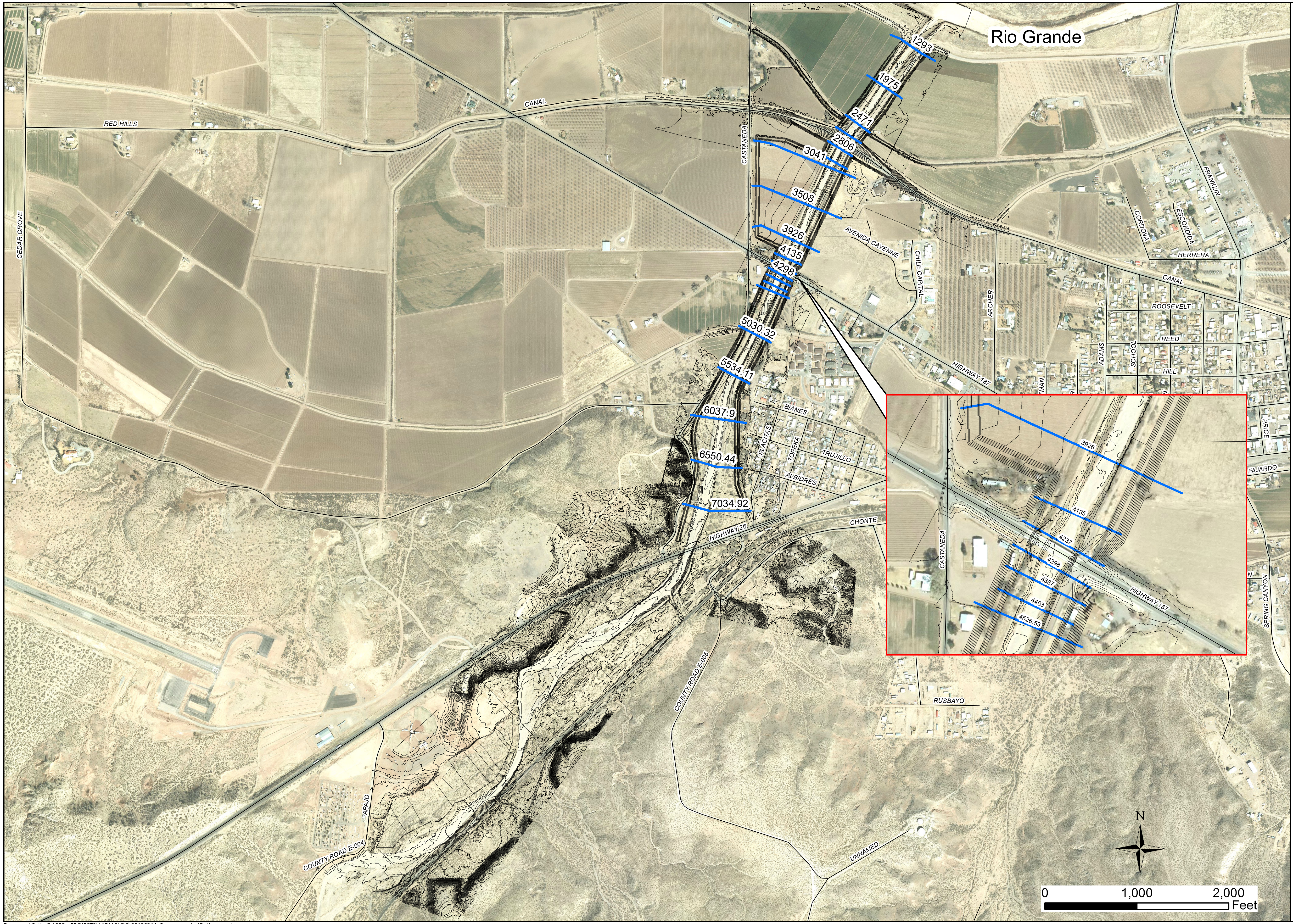

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HEC-RAS Cross Sections

Proposed Contours

Roads

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Figure 12 Proposed Conveyance Alternative

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4.2 DESIGN CONSIDERATIONS AFFECTING PROPOSED CONVEYANCE ALTERNATIVES

There were several conditions that affected the process of creating a conceptual design that would provide effective flood control for the VOH.

A. Permitting

The Placitas Arroyo, having its terminus at the Rio Grande, is considered as part of the 'Navigable Waters of the United States'. Any channel improvements and construction will require a 401/404 permit. As such, discussions were held with the USACE's regulatory division located in Las Cruces to determine what the permitting process would involve.

Depending on the extent of the project such as lineal feet of disturbance or acres of disturbance within what the USACE defines as the 'Ordinary High-Water Mark' (OHWM), the permit will be classified under a Nation-Wide Permit or an Individual Permit. The OHWM as defined in 33 CFR 328.3(e), "The line on the shore established by the fluctuations of water and indicated by physical characteristics such as a clear natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas. This is typically the region of the channel that experiences flow in the more frequent small events. The engineer for the project must define and justify the OHWM for the arroyo in question which then must be approved by the local USACE office. Furthermore, if the improvements are located outside the OHWM, then the permitting process is no longer applicable. While the Nation-Wide Permit is easily obtainable, it has restrictions on the extents of the project within the OHWM. For instance, channel disturbance must be limited to within 500 linear feet within the OHWM.

In contrast, an Individual Permit, while valid for 10 years, requires an arduous amount of time and money to obtain. There are stringent requirements on mitigation and remediation for the channel that is being disturbed. An initial assessment with the USACE was indicative of at least a two-year permitting period with the cost of the permit approximated to as much as $2 million. Based on this information, Smith performed successive field reconnaissance trips to determine if the OHWM was easily identifiable and what steps could be taken to minimize the disturbance of the OHWM to eliminate the need for an Individual Permit. Field observation proves that the Placitas Arroyo has a very defined OHWM throughout the limits of the proposed improvements. Therefore, an alternative conceptual design for the channel was performed so that the disturbance to the OHWM would be minimal. Channel design will be discussed further in the next section.

Photos 17 and 18 show the typical OHWM for the Placitas Arroyo where the break in region of low flow from the channel overbank is characterized by the vegetative line and change in soil structure.

Photo 18: Typical OHWM for the Placitas Arroyo

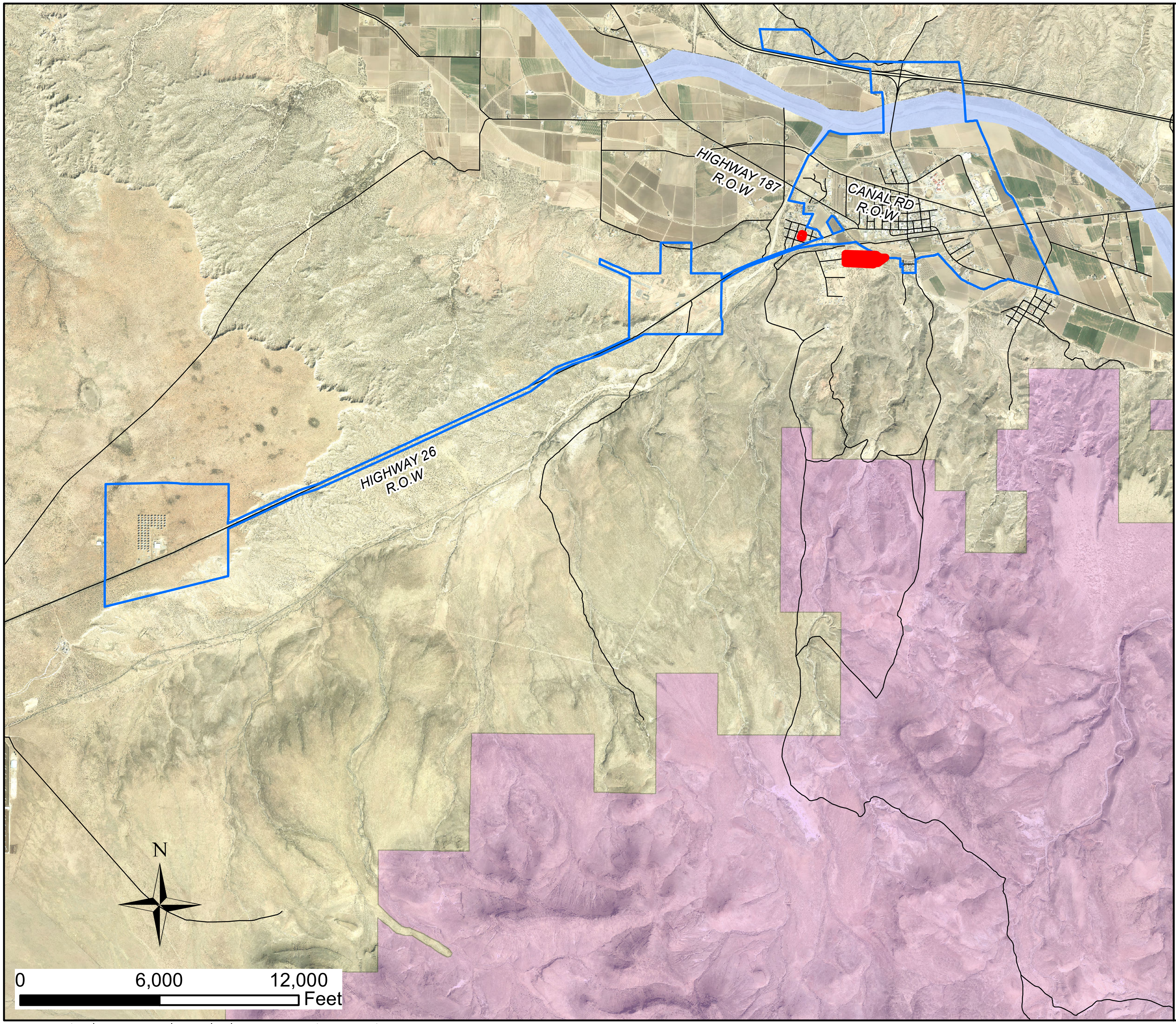
Photo 17: Typical OHWM for the Placitas Arroyo

B. Project Limits within NMDOT Jurisdiction

Since NM 26 is significantly under capacity, a new bridge structure will be required to pass the design flows without overtopping the bridge and flooding the Placitas Colonia. This phase will have to be coordinated with the NMDOT. Meetings were held with the NMDOT District 1 engineers, and the NMDOT Bridge and Drainage Bureau to discuss the potential options at this crossing. The first option considered was an array of large overflow box culverts east of the bridge to prevent a full bridge replacement. Initially, both 1 and 2D models were developed to comprehensively simulate the hydraulics of these proposed overflow culverts. Unfortunately, due to flows approaching critical or super critical depth, the 2D model could not converge to provide reasonable results while running an unsteady time-based analysis. Therefore, a 2D model without structures was built based on the proposed channel grading surface. Maximum headwater depths were obtained from the 2D model at the location of the overflow culverts. Culvert computations using Culvert Master were performed based on the maximum headwater depth obtained from the 2D model to verify the need for 16 – 12’S X 7’R box culverts. HEC-RAS 1D could not be utilized to model the full array of culverts as it only can simulate a maximum of 10 culverts at a time. A preliminary cost analysis was performed based on the NMDOT standard drawings for a bank of quadruple 12’ x 7’ box culvert structures. Preliminary estimates indicated that this overflow structure could cost as much as $5 million. NMDOT indicated that a full bridge replacement would be more feasible however that would occur in the next 10 years based on their capital improvements program. In addition to the cost, there are maintenance issues accompanied with debris clogging at the culvert openings. Based on discussions and jurisdictional issues with NMDOT, redesign of the bridge at NM 26 fell outside the scope of this project. Further coordination and collaboration will be required between the DACFC, VOH and the NMDOT when the NMDOT prepares for replacing NM 26. **The primary design assumption for the proposed Placitas Arroyo in this DMP is that NM 26 will be able to pass 100 % of the total runoff approximately 17,594 cfs.** NM 187 is the other NMDOT highway that crosses the Placitas Arroyo. Fortunately, when the bridge was replaced in 2007, NMDOT had the foresight to expand the span of the bridge so that the bottom width is 147 ft. The two spans constructed for future expansion are located on the west bank of the channel. This will enable channel improvements to tie into the full bridge section. Further coordination between NMDOT and the VOH will be required at that point.

C. ROW for Other Agencies

The Placitas Arroyo has ROW belonging to the IBWC and the BLM that will be affected by either the channel improvements or sediment control facility. **Figure 13** shows an overview of the ROW limits for the various entities. Coordination with IBWC was done to determine if work within their ROW was feasible for the design of outlet facilities. IBWC is willing to work with VOH and the DACFC as the projects proposed in this DMP come to fruition.



Legend

- DACFC
- Village of Hatch
- Roads
- IBWC
- BLM

Placitas Arroyo
Drainage
Master Plan

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Figure 13
Limits of ROW
for Various
Stake Holders

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4.3 DESCRIPTION OF CONVEYANCE ALTERNATVIES

From the existing hydraulic analysis, it is abundantly clear that the existing channel section is severely under capacity with failure anticipated between the 7000 to 9000 cfs. After several iterations, two conveyance alternatives were developed. The two alternatives are a function of which permitting option the VOH and DACFC will pursue.

4.3.1 ALTERNATIVE 1: NATIONWIDE PERMIT WITH MINIMAL DISTURBANCE TO THE OHWM

This alternative proposes a benched channel design which predominantly leaves the existing OHWM undisturbed. Channel redesign would occur outside the OHMW creating a non-uniform channel geometry that will create a main low flow channel and a benched overbank channel as shown in **Figure 14** starting from NM 26 to the Rio Grande. The proposed design would maintain the existing 55 ft main channel while adding an additional 150 ft of channel on both banks with a leveed section that would be at a 1V:2H side slope, making the total channel width approximately 210 ft. The levee would be 18 ft high to provide the required 3 ft of freeboard for the 100-year flow of 17,954 cfs. The main low channel would remain unlined and would be expected to erode over time. The side slopes on the stream side of levee would have to be lined due to the high shear stresses experienced during the peak flow. This design alternative also proposes an inline overflow ineffective flow area upstream of Canal Road that will reduce channel velocity and promote sediment deposition. Furthermore, two scenarios were modeled to simulate different channel surfaces. Scenario 1 assumes a wire-enclosed riprap channel. **Figure 14** shows a 3D rendition of what the proposed channel would look like, Scenario 2 assumes a smoother lining material such as shotcrete.

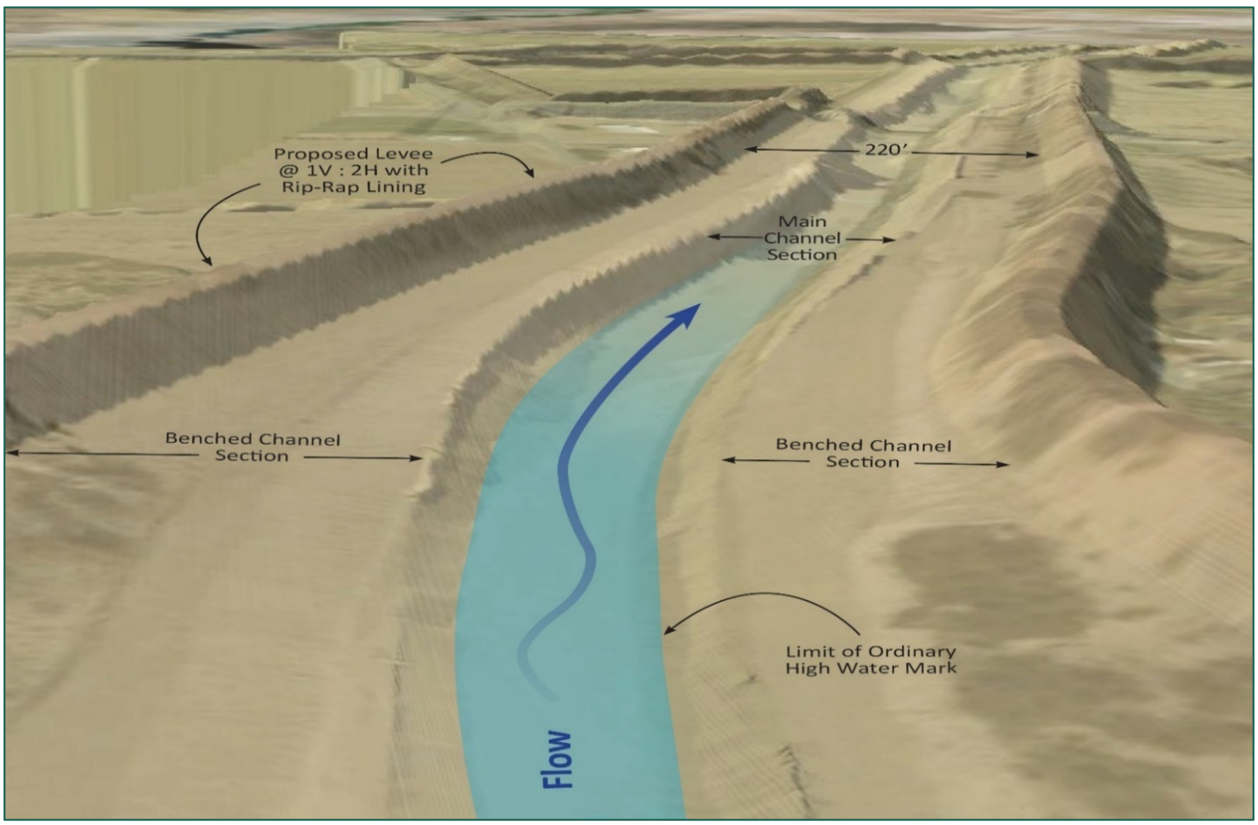


Figure 14: Typical Channel Section for Alternative 1

Since the channel has a complex geometry combined with an inline overflow area, a 2D model was constructed to cross check the results obtained from the 1D RAS model. The results from both 1 and 2D models corelated very well. The table below summarizes key results from the 1D-2D hydraulic analysis.

Table 4: Summary of Critical Hydraulic Parameters for Alternative 1 (Nationwide Permit with minimal distance to OHWM)

Channel Parameters	Scenario 1 Riprap HEC-RAS 1D	Scenario 2 Shotcrete HEC-RAS 1D	Scenario 3 Riprap HEC-RAS 2D
Channel Surface (Manning's n value)	0.035	0.027	0.035
Average Channel Depth (ft)	12	9	14
Average Channel Velocity (ft/s)	12	16	13

4.3.2 ALTERNATIVE 2: INDIVIDUAL PERMIT WITH UNRESTRICTED CHANNEL IMPROVEMENTS

Alternative 2 assumes that an Individual Permit will be obtained which allows unrestrained construction access and disturbance within the OHWM. This alternative proposes a uniform channel section with a 146 ft. bottom width, leveed sections at 1V:2H that are also 18 ft. high to meet the 3 ft. of freeboard. This provides a more efficient hydraulic section as is evident by the results presented in **Table 5.**

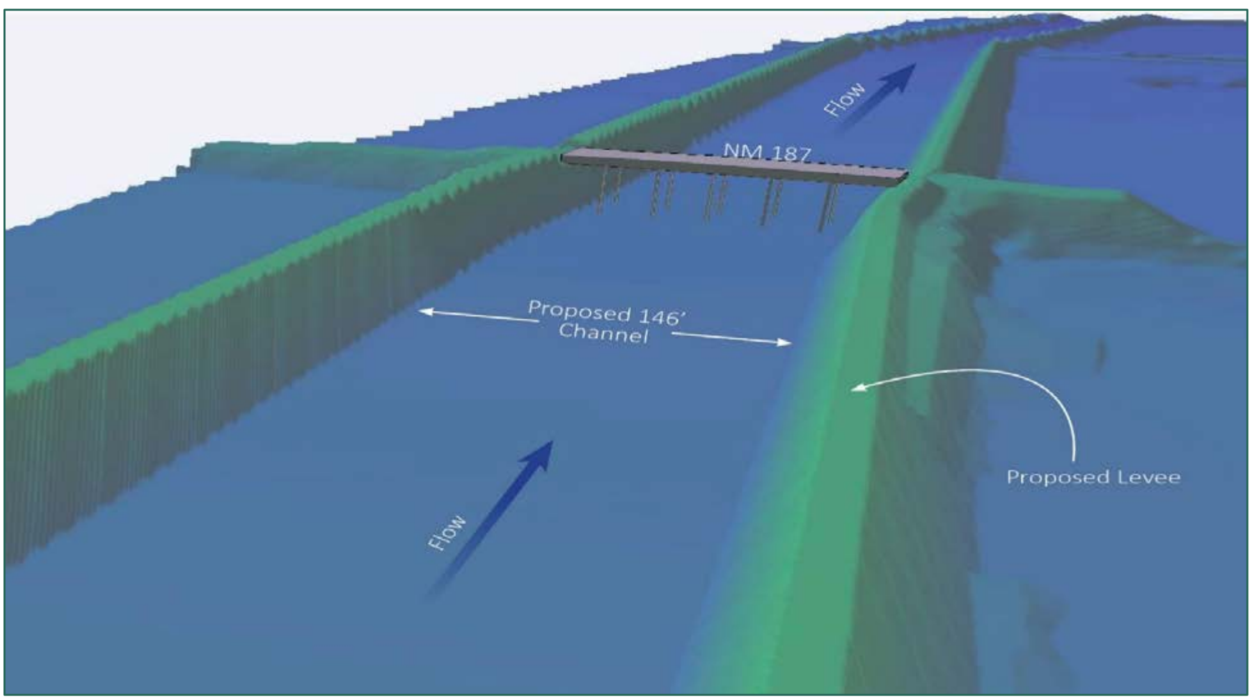


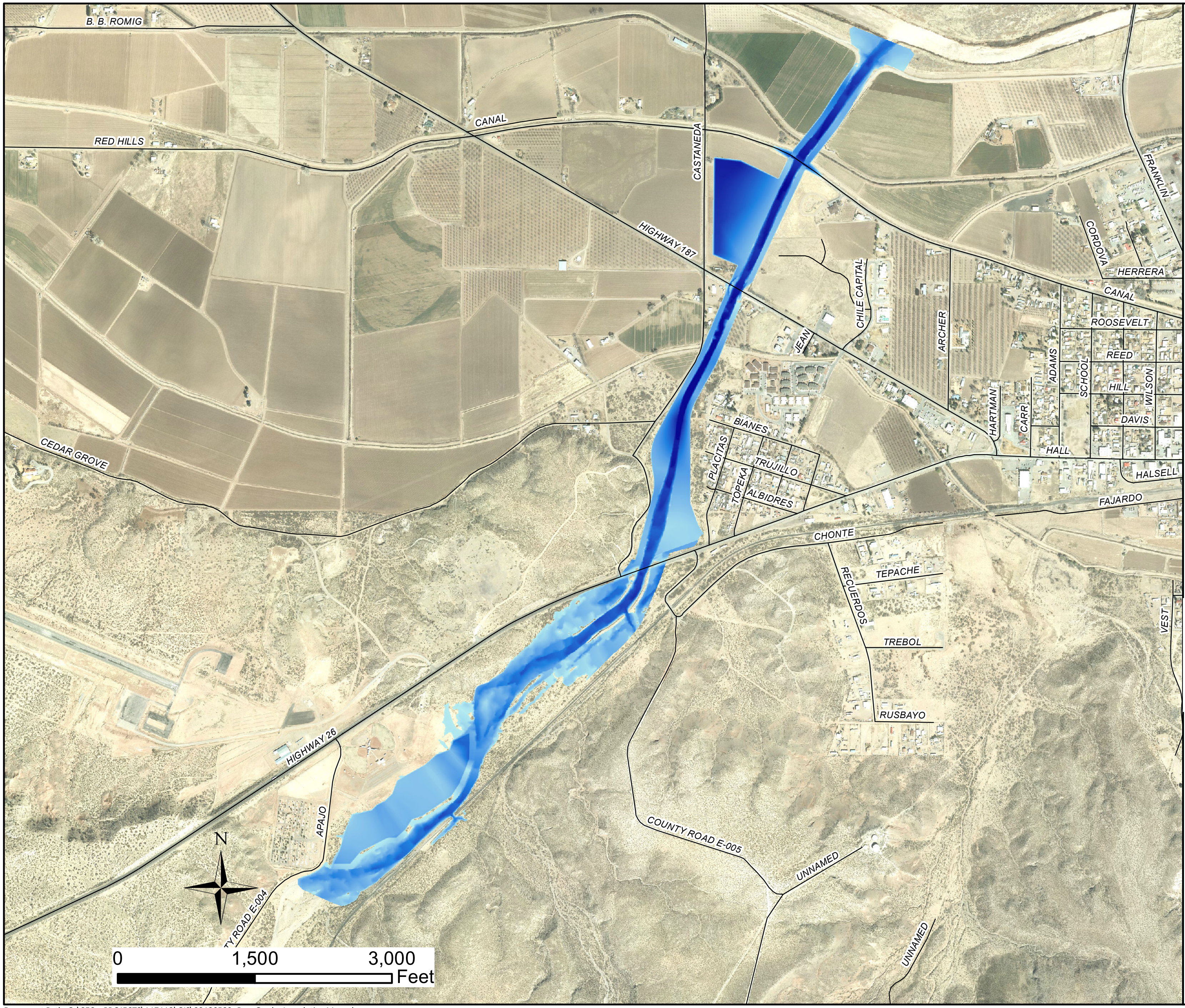
Figure 15: Typical Channel Section for Alternative 2

Table 5: Summary of Critical Hydraulic Parameters for Alternative 2
(Individual permit with unrestricted channel improvements)

Channel Parameters	Scenario 1 Riprap HEC-RAS 1D	Scenario 2 Shotcrete HEC-RAS 1D	Scenario 3 Riprap HEC-RAS 2D
Channel Surface (Manning's n value)	0.035	0.027	0.035
Average Channel Depth (ft)	11	8	14
Average Channel Velocity (ft/s)	10	16	12

In both alternatives, using the riprap lined channel raises the water surface enough to where NM 187 has no freeboard. The peak water surface elevation upstream of the bridge is at 4075.36. The low chord of the bridge is at 4075 while the high chord elevation is at 4076.29. At the peak duration of the hydrograph, water will build against the upstream side of the bridge without overtopping the structure itself.

Figures 16 and 17 show the distribution of depths and velocities based on a riprap lined channel.



Legend

Roads

Depths (ft)

High : 14.0

Low : 0.50

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Figure 16 Alternative 2 Scenario 3 Flow Depths Based on 2D Surface Water Modeling

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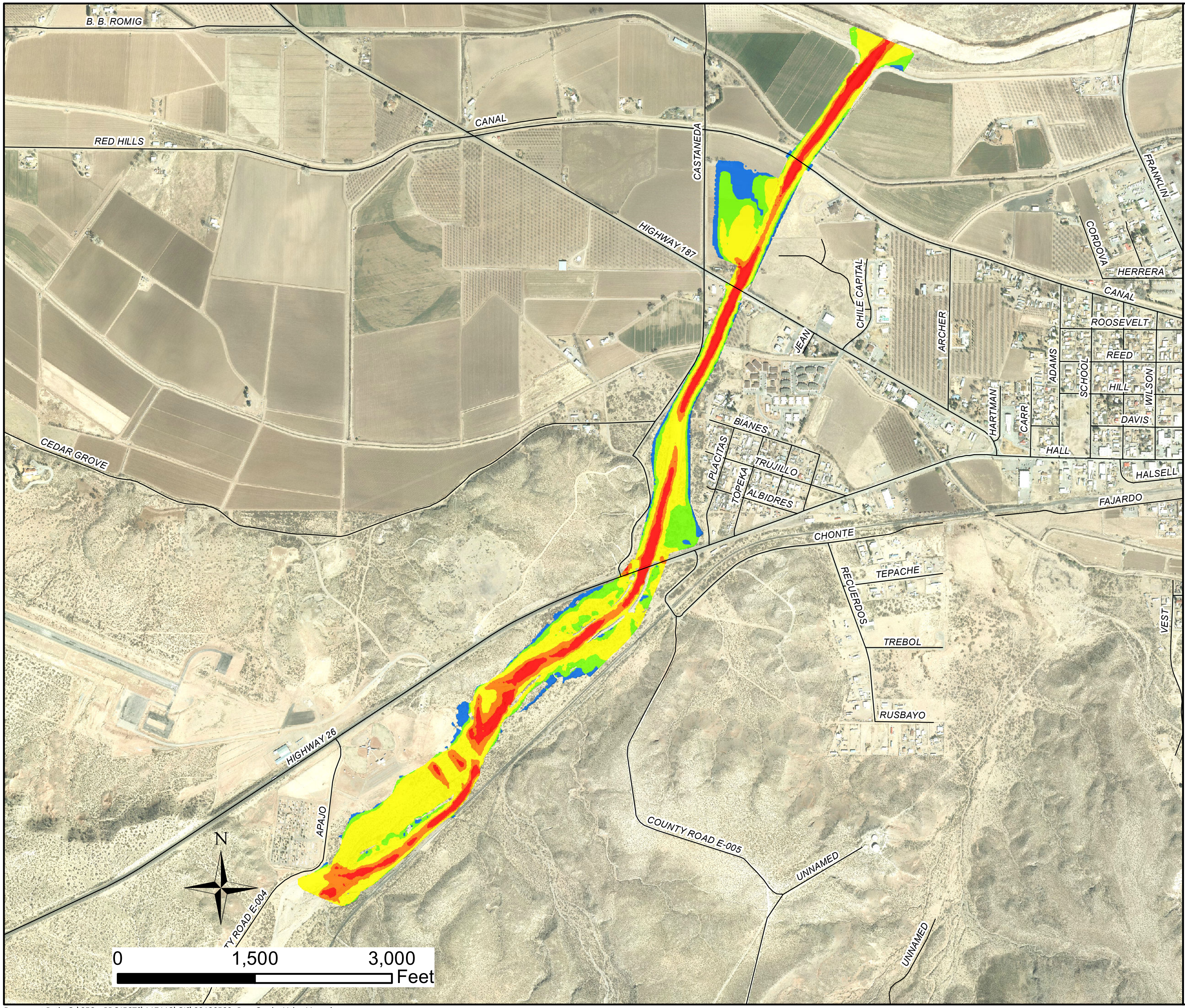

Legend

Roads

Velocity (ft/sec)

- 1-3
- 3-5
- 5-10
- 10-12
- 12-17

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Figure 17 Alternative 2 Scenario 3 Average Velocities Based on 2D Surface Water Modeling

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4.3.4 CHANNEL LINING, VERTICAL AND HORIZONTAL STABILITY

A. Channel Lining

Both alternatives will require the channel 300 ft upstream and 200 ft downstream of the bridge at NM 187 to be lined with either concrete or shotcrete to make the conveyance more efficient so that the bridge does not over top. The shear stress on the stream side of the levee face ranges from 1.1 – 3.2 lb./ft^2 for all alternatives. This will rule out an earth lined levee as per Table 2.3 (shown below) in FHWA Hydraulic Engineering Circular 15 (HEC-15), 'Design of Roadside Channels with Flexible Linings'. The maximum permissible shear stress for bare soil is approximately 0.14 lb./ft^2. This number can be increased to between 2.4 - 4.8 lb./ft^2 for riprap lined channels where the D_{50} for the riprap ranges from 0.5 – 1 ft. Since the shear stress for the channel is in the range of 2-4 lb./ft^2, a wire enclosed riprap lining will be required to prevent the levee from eroding. A smooth concrete lined channel was considered, however due to the significant increase in channel velocities between 18-25 ft/s, significant energy dissipating structures at the outlet of Placitas Arroyo would be required. Shotcrete is a possibility as well, however, impact damage from the large boulders that form part of the live load of the flows must be considered.

Table 2.3. Typical Permissible Shear Stresses for Bare Soil and Stone Linings

Lining Category	Lining Type	Permissible Shear Stress N/m^2	Permissible Shear Stress lb/ft^2
Bare Soil[1] Cohesive (PI = 10)	Clayey sands	1.8-4.5	0.037-0.095
	Inorganic silts	1.1-4.0	0.027-0.11
	Silty sands	1.1-3.4	0.024-0.072
Bare Soil[1] Cohesive (PI $\geq$ 20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil[2] Non-cohesive (PI < 10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.24
Gravel Mulch[3]	Coarse gravel D_{50} = 25 mm (1 in)	19	0.4
	Very coarse gravel D_{50} = 50 mm (2 in)	38	0.8
Rock Riprap[3]	D_{50} = 0.15 m (0.5 ft)	113	2.4
	D_{50} = 0.30 m (1.0 ft)	227	4.8

[1]Based on Equation 4.6 assuming a soil void ratio of 0.5 (USDA, 1987).
[2]Based on Equation 4.5 derived from USDA (1987)
[3]Based on Equation 6.7 with Shield's parameter equal to 0.047.

B. Vertical Stability

The vertical stability was evaluated based on the Equilibrium Slope Method Outlined in the 'Sediment and Erosion Guide'. Using equations 3.33 – 3.35, the equilibrium slope for Placitas Arroyo was computed to be approximately 0.52%.

$Ss = CQ_D^{-0.133}$ (Equation 3.33)

$C = 18.28n^2F_D^{0.133}F_r^{2.133}$ (Equation 3.34)

$C = 18.28(0.035)^2(40)^{0.133}(0.735)^{2.133}$ = 0.019

$Ss = 0.019(17594.2)^{-0.133}$ = 0.0052 ft/ft

Where equation variables are:

S_s = maximum stable slope, ft/ft
n = Manning's roughness coefficient
F_r = Froude Number (0.7 to 1.0)
Q_D = dominant discharge, cfs
F_D = width-depth ratio of the flowing water, ft

Based on survey elevations, the existing bed slope from NM 26 to the river is 0.5% indicating that the Placitas Arroyo is at equilibrium slope. These results make logical sense as the channel cannot degrade any further than its existing slope and elevation otherwise it will not be able to drain to the river by gravity. Through the duration of a hydrograph, the channel bed of an unlined channel will undergo dynamic changes in its vertical profile due to considerable variations in the sediment loads, shear stresses, and changing velocities. However, at the end of the hydrograph, a channel that has attained its maximum stable slope will always return to its state of equilibrium. That is the case for the Placitas Arroyo. As such, there is no need for grade control structures in the channel bottom or vertical stability.

Flood wall scour should be considered at preliminary and final design stages to make sure that the toe of the levee on the channel side does not get compromised. The 'Sediment and Erosion Guide' also has an equation that predicts scour parallel to a flood wall and local scour at revetments, spurs, and abutments.

Equations 3.61 and 3.62 were utilized to predict approximate limits of scour along the toe of the levee on the channel side using average values from all four scenarios. The result from Equation 3.61 predicts a depth of 3 ft whereas Equation 3.62 predicts 9.5 ft of scour along the toe of the levee.

$\frac{Y_S}{Y_1} = 4F_{r1}^{0.33}, \frac{a}{Y_1} = Y_1$ (Equation 3.61)

$Y_S = 4(0.735)^{0.33}\left(\frac{10}{10}\right) = 3.6$ ft

$\frac{Y_S}{Y_1} = 0.73 + 0.14\pi F_r^2$ (Equation 3.62)

$Y_S = (0.73 + 0.14\pi 0.735^2)(10) = 9.7$ ft

Where equation variables are:

Y = equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole), ft
Y_1 = average upstream flow depth in the main channel, ft
a = abutment and embankment or wall length projecting into main channel, ft
F_{r_1} = upstream Froude Number.

These are empirical methods, so the results require considerable engineering judgment and field observation. From field observation a 10 ft scour depth along the proposed flood wall would be unlikely.

Given the many unknowns at this planning stage in terms of final channel design, it's safe to assume a minimum of 5 ft for local scour along the toe of the proposed levee. Since the side slopes must be lined with wire enclosed riprap revetment, for the purposed of this DMP, it will be assumed that the revetment will be extended 5 ft below the toe of the proposed channel bed. Alternatively, sheet piles may also be a consideration. All variables for the calculations were extracted from the HEC-RAS 2D model.

C. Horizontal Stability

The proposed channel from NM 26 to the river should have no lateral migration issues since it will be confined by the levees. However, upstream of NM 26, Placitas Arroyo is still in a natural meandering state. The channel also meanders on its approach to NM 26. The area of concern in this region is the west bank at the cemetery. During the event in 2006, the arroyo migrated laterally towards the west almost to the point of compromising the cemetery. To prevent complete embankment failure at the cemetery, armoring of the west bank is recommended. Initially bend way weirs were considered but due to the magnitude of the flows, wire enclosed riprap armoring will be more efficient and less maintenance intensive.

Figure 18: Limits of Embankment Protection at Cemetery

4.3.4 GEOTECHNICAL CONSIDERATIONS FOR THE PROPOSED LEVEE

Smith solicited the services of Yedoma Consultants LLC, a geotechnical engineering company to provide a conceptual level geotechnical evaluation of the levee design so that it would be compliant of USACE document “EM1110-2-1913”. Yedoma Consultants considered the following parameters as part of their scope of work:

- Slope Stability
- Levee Construction
- Seepage Analysis

D. Conceptual Levee Design Considerations

The levee embankment could re-use existing levee material, to offset borrow quantities. It is assumed that the soils will be predominately sand with variations of silt and clay (fines) content. The stream side of the levee is anticipated to have a safe slope at a 2H: 1V ratio. The facing of the slope is recommended to have a cementitious material to provide increased resilience to scour, reducing the likelihood of breaches, and costs of maintenance of the structure.

The levee should be constructed with moisture conditioning and compaction control, with a relatively impermeable core. The core material could be comprised of driven metal sheets, clay, or engineered soil-cement mixture. A life-cycle cost analysis was beyond the scope of this study, however, for concept level considerations, we recommend the design should incorporate a soil-cement core for the remainder of the levee. The crest of the levee will be a minimum of 10 feet wide providing for ease of construction will using heavy machinery to develop the soil mass. The land side of the levee is anticipated to have a minimum slope angle of 3H: 1V ratio to extend the toe of slope far enough away from the channel to resist uplifting pressure during the peak flow events. Wire-enclosed riprap is an excellent erosion protection method for surfacing of the stream side slope. It is anticipated that local sources of material are available and NMDOT bid tabs from District 1 projects will provide a good estimate of costs.

Articulated mats can also provide a higher-class of slope protection. They tend to be quite costly and usually not readily available. They tend to reduce construction time and provide a strong protection of slope soils. Typically cement mixed soils are more economical in construction and provide a good amount of levee protection in the event of a flood. Cement stabilizer should be mixed with the soil and cover the stream side of levee structure to increase resilience from water wear. Added strength and resistance to soil erosion will reduce the maintenance of the structure over long-term. There may be drawbacks or constructability issues with implementing cement-mixed soil on a relatively steep slope. The wire-enclosed riprap is the recommended option for the conceptual levee design option. The design of granular filters should follow the guidelines outlined in the USACE Engineering Manual as well as the recommendation suggested by Terzaghi. The purpose of the filters is to prevent the migration of soil particles which could cause piping in the levee walls. This piping strips the structure of the fine grade particles which reduces cohesion of soil grains. Water is allowed to flow through the filter to drain out of the structure. The filters will require cleaned coarse aggregate that is uniformly sorted. The openings cannot be 6.5 times larger than the soil particles, otherwise stripping of fines will occur.

E. Geotechnical Analyses

A concept levee design option assuming a phreatic surface at 16 feet above the existing ground surface. The levee was modeled with 2H: 1V stream side and 3H: 1V land side slopes and an impermeable core. As the basis for developing geotechnical parameters, it is assumed that the levee will be constructed out of AASHTO A-2-4 or A-1 material (native soils).

Subgrades are to be scarified, moisture conditioned, and compacted to 95% of the modified proctor (T-180) maximum dry density and +/-2% of optimum moisture. Locally, sub-excavation may be necessary for unstable foundation material, such as organics, high plasticity clays, landfills, or other loose/soft material. Additionally, Table 6 summarizes key parameters assumed to perform the slope stability and seepage analysis.

Table 6: Slope Stability and Seepage Analysis

Subunit	Unit Weight, lbs/ft^3	Phi angle
Levee (engineering fill)	120	32
Native Soils	110	30

F. Slope Stability

The levee slope was modeled based on peak discharge event for global stability. The critical failure surface was determined based on our understanding of a failure which would lead to an unacceptable damage to the levee. The critical failure had a factor of safety of 1.8 for the stream side and 1.5 for the land side. The figures below show the failure planes computed based on the safety factors for stream side and land side respectively.

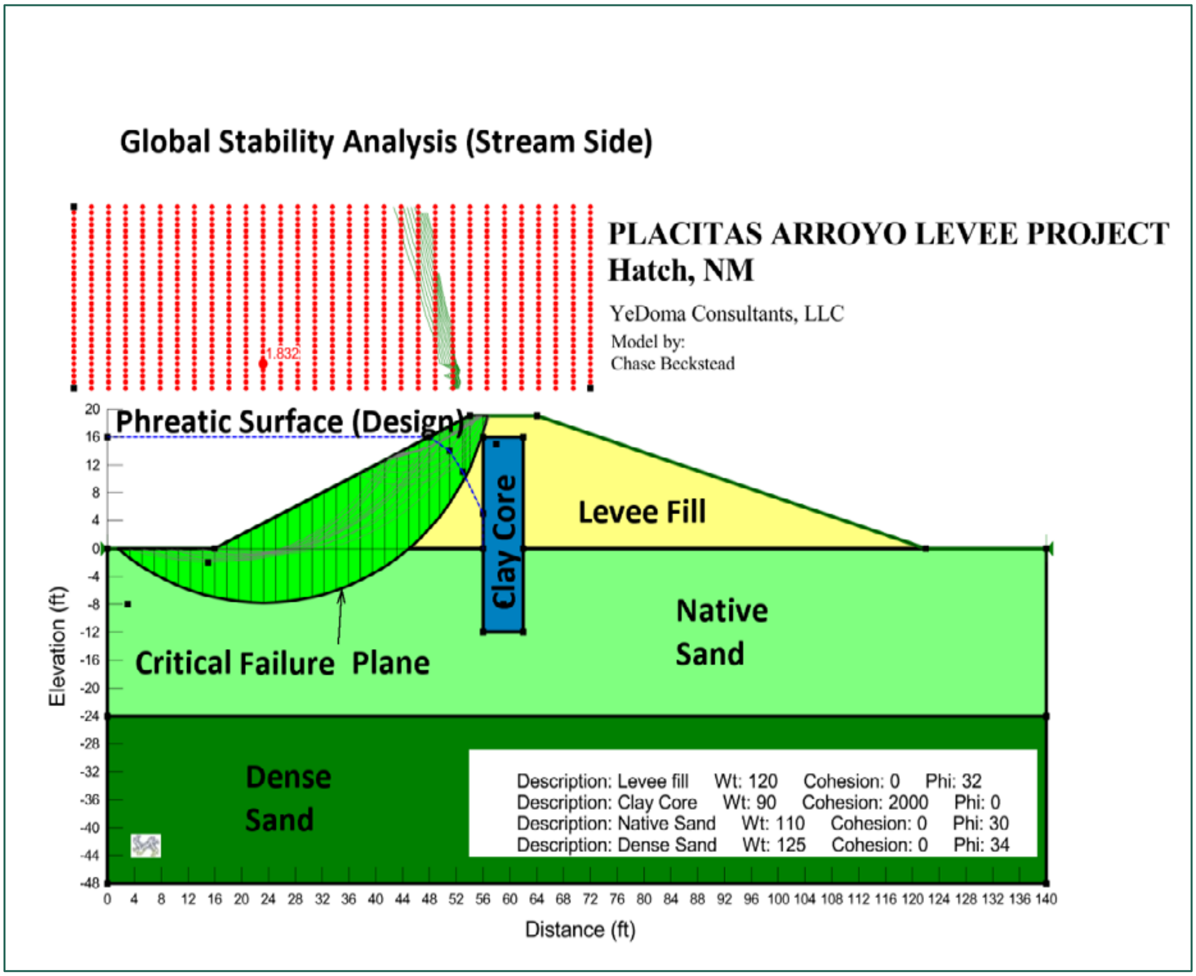


Figure 19: Failure Plane for Stream Side of Proposed Levee

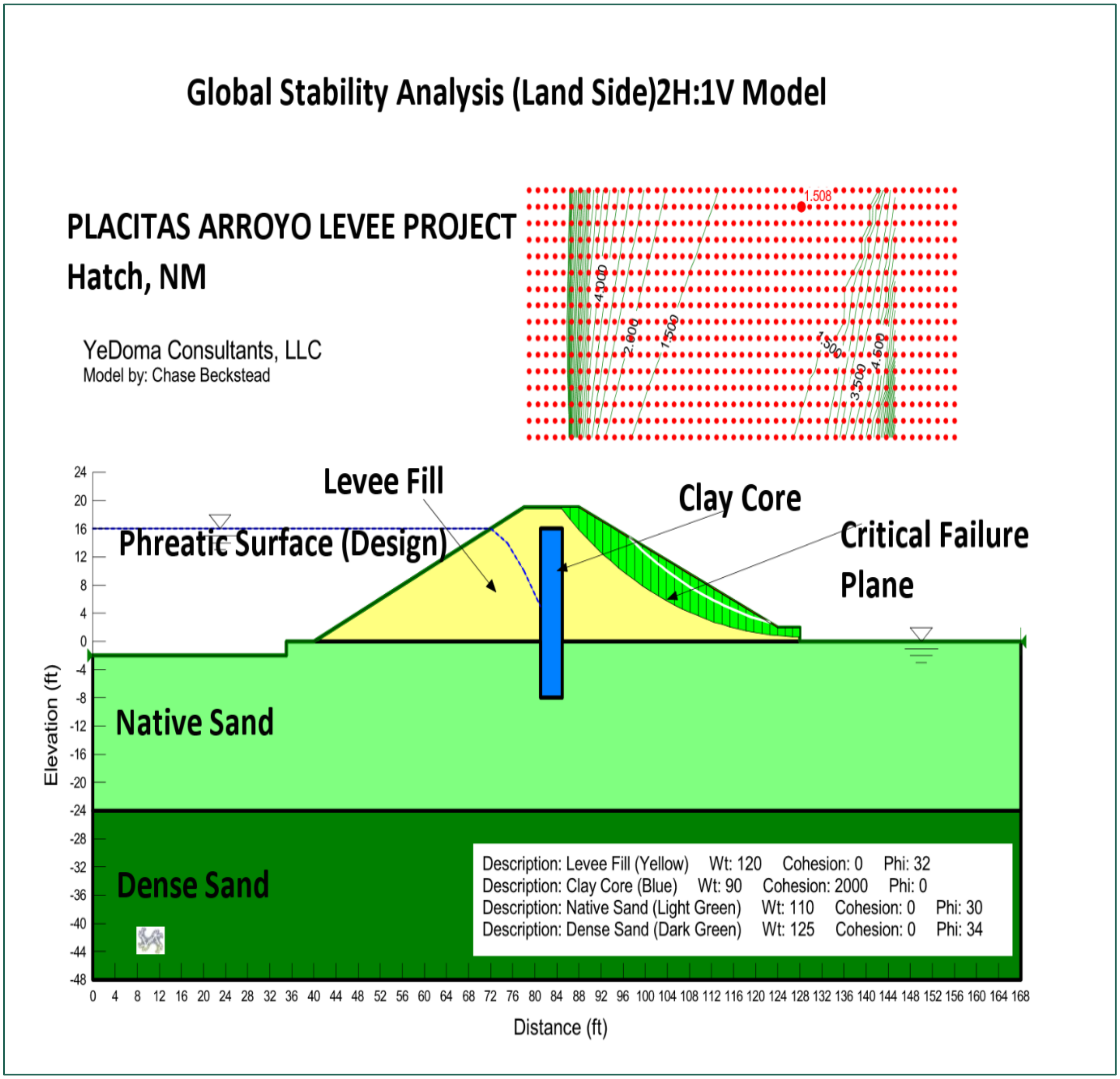


Figure 20: Failure Plane for Land Side of Proposed Levee

G. Seepage Analysis

It was calculated that the flow path of the water from the stream side face of the levee to the land side of the levee would be about 106 feet if the land side slopes were constructed at 3H:1V. The seepage of water through the levee was calculated to be 95 ft^2/day per foot of levee. **Figure 21** presents the flow nets created to perform the seepage analysis. The land side may be constructed at a 2H:1V side slope however, an additional berm structure that is 10 ft X 2 ft is recommended to counteract any excessive uplift water pressure created due to seepage. These geotechnical recommendations will have to be refined based on final design of the channel and levee.

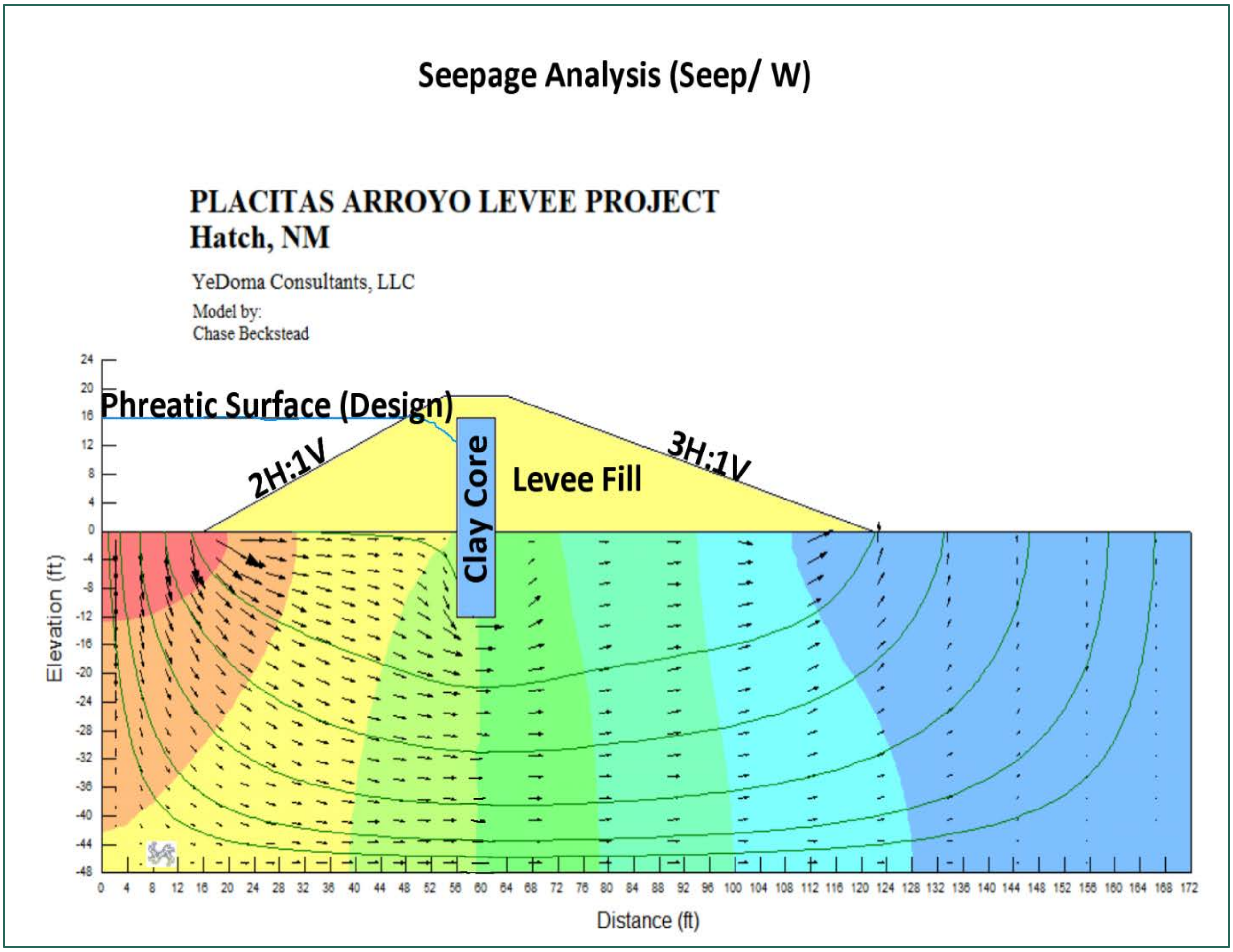


Figure 21: Flow Net for Seepage Analysis

H. Sediment Control

Sediment control was also part of the scope of this analysis. Inline sacrificial sediment control pits were considered as the part of the channel design for just downstream of the NM 26, where the proposed section will be widened. If the NM 26 Bridge is reconstructed to pass design flows, the area upstream of NM 26, just by the cemetery, will also be a prime location for sediment control. Due to the high sediment loads, sediment control may only be feasible up to the 2-year storm which will be approximately 44 ac-ft. **Figure 22** shows a 3D rendition of what the in-line sediment traps would look like. The biggest issue with a sediment control facility is maintenance. The highest level of sediment transport occurs in what is termed as the "dominant discharge (Q_d)". Based on the Sediment and Erosion Guide (Mussetter Engineering, Inc., 2008), Q_d is typically assumed to be 20% of the Q_{100}. Since the Q_{100} for the Placitas Arroyo is 17,594 cfs, the dominant discharge will be approximately 3,519 cfs which correlates to the 2-yr-24-hr storm. The sediment volume for this event is approximately 1,156 ac-ft of which 4% or 46 ac-ft will be the sediment load. The sediment traps shown conceptually will have to be sized accordingly. A rigorous maintenance schedule will have to be developed and maintained between the VOH, DACFC, and EBID to ensure that these structures are cleaned out regularly for them to be effective. Proposed HEC-RAS models and output can be found in **Appendix G.**

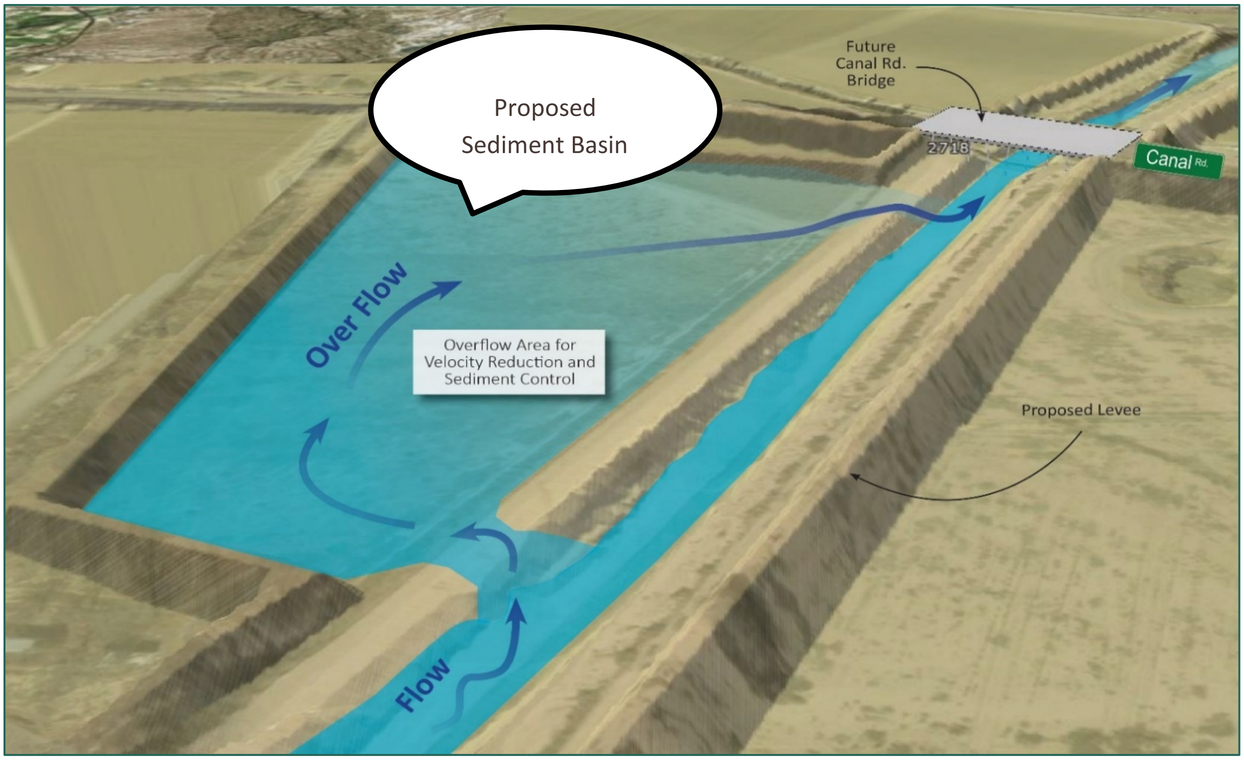


Figure 22: In-Line Sediment Control Structure Upstream of Canal Road

A secondary sediment trap was considered upstream of NM 26 near the cemetery as shown in **Figure 23**. However, the topography is such that this location doesn't provide adequate storage volume unless an embankment is built.

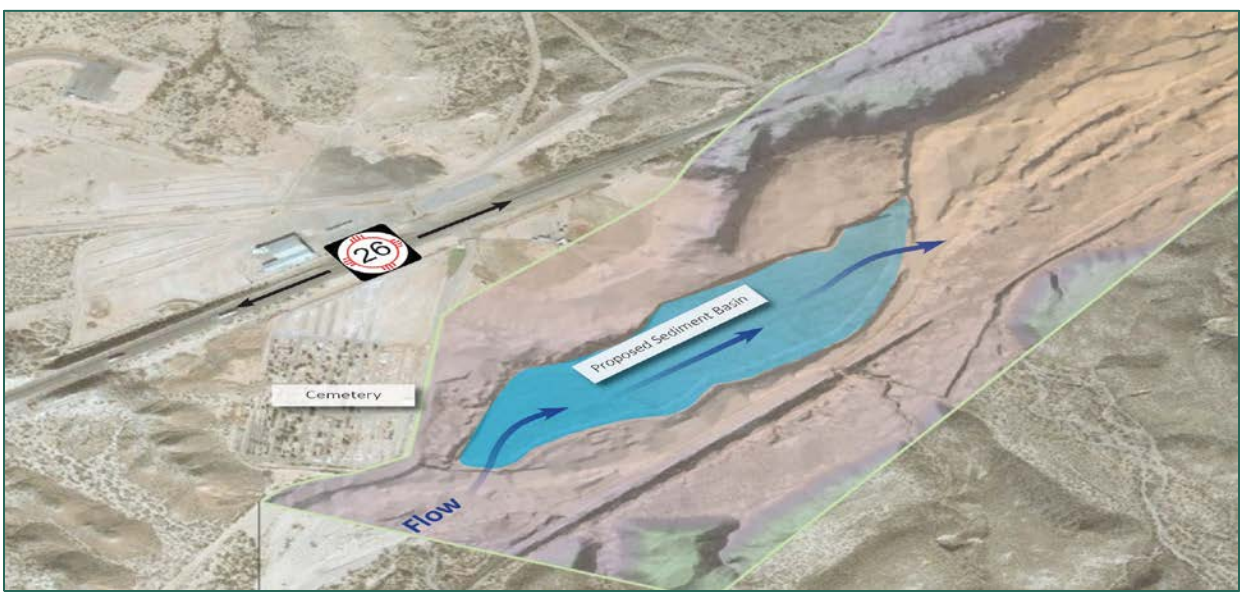


Figure 23: Sediment Trap Upstream of NM 26

An alternative sediment basin could be constructed just before the outlet of the Placitas Arroyo as shown on **Figure 24.**

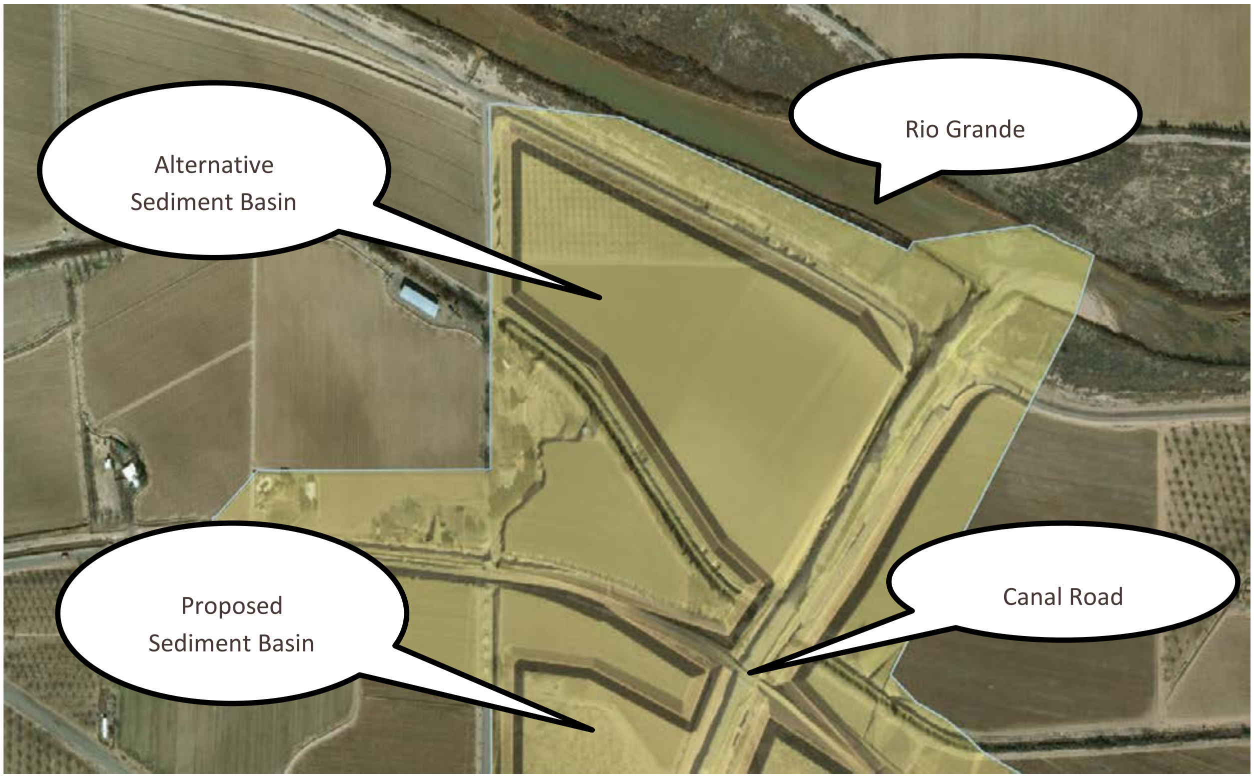


Figure 24: Alternative Sediment Basin

I. Canal Road Crossing

The existing Canal Road Bridge crossing needs to be removed due to the severe capacity limitations of the existing box culverts. Several options were proposed at the 60% level. The pros and cons of the various options are presented in **Table 7.** Smith and the DACFC conducted a special meeting with the Board of Trustees and the Mayor to review these options and answer questions or concerns. The options selection would have to be a collective decision made by the VOH Board of Trustees. After much deliberation, the VOH voted to select the following options. The existing box culverts will be removed, and a temporary dip crossing will be constructed. In the long run, the temporary dip crossing will be replaced with a bridge. The dip crossing will maintain access for farm vehicles, but significant efforts must be made to ensure that traffic does not cross the dip crossing in the event of a storm.

Table 7: Pros and Cons of Proposed Options

OPTION	PROS	CONS
LOW FLOW CROSSING	EXISTING ACCESS	EXTENSIVE ROADWAY IMPROVEMENTS TO MEET CRITERIA I.E HIGH COST
		FEMA FLOODPLAIN REMOVAL WILL NOT BE POSSIBLE
		HAZARDOUS DURING STORMS
NEW BRIDGE CROSSING	EXISTING ACCESS	VERY EXPENSIVE
	FEMA FLOODPLAIN REMOVAL ENTIRELY POSSIBLE	
ABANDON CROSSING	FEMA FLOODPLAIN REMOVAL ENTIRELY POSSIBLE	LONGER TRAVEL TIME +/- 5 MINUTES
	COST EFFECTIVE	

Dip crossings pose a significant hazard to overconfident motorists who often ignore warning signs and circumvent traffic barriers to cross a flooded dip crossing often resulting in fatalities. Some of the options available for traffic barricading are automated gates that are activated by a pressure transducer, or manually operated gates. All these options will be costly, and none are fail safe. Furthermore, in the event of a large storm, flows will escape at this location and damage the EBID canal on the north side of Canal Road and the agricultural fields as well. The future bridge crossing will be approximately the same height and span as the NM 187.

This is not the option that Smith Engineering recommends and the VOH must assume full liability and risk from any flood damage or loss of life that may occur after this dip crossing is constructed.

SECTION 5. PRIORITIZATION OF ALTERNATIVES

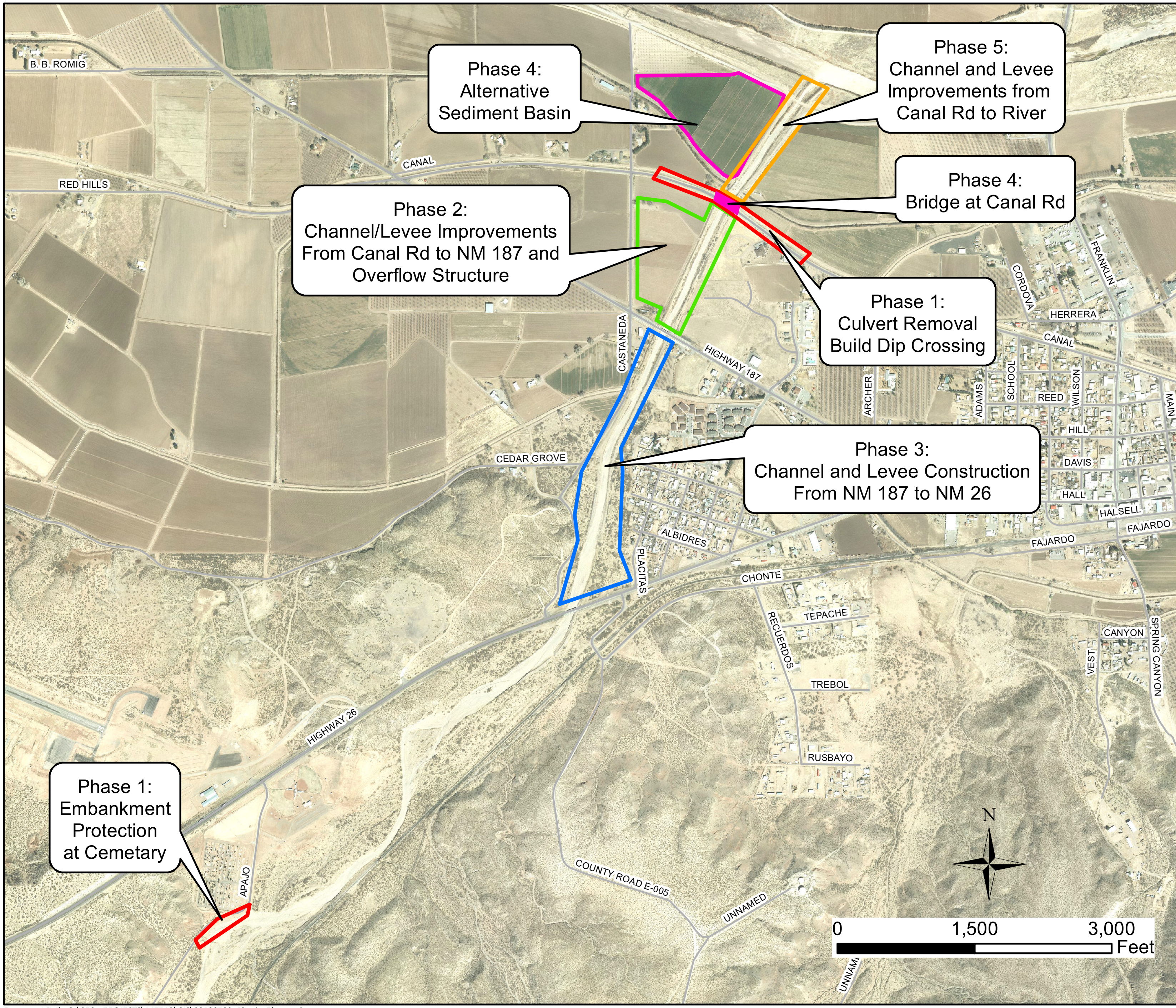
5.1 VIABLE ALTERNATIVES

The most viable option for flood control is the levee and channel system that will convey the 100-year flows through the VOH limits. The two alternatives considered for the levee channel design will be a function of which permitting process will be pursued. Alternative 1 assumes that a Nationwide Permit will be pursued. Alternative 2 assumes that an Individual Permit will be pursued. A cost comparison was developed for both alternatives between using riprap vs. shotcrete as channel lining. The scope of the overall project in either alternative will be extensive and extremely expensive. Therefore, the overall project will have to be designed in logical and manageable phases. Breaking the project into manageable phases will also allow the VOH and DACFC to obtain the funding necessary. Due to a high number of unknowns, a 35% contingency was used. This number can be reduced once the project is in preliminary and final design. Bridge costs for Canal Road were based on the average bid price for NM 187. The dollar value from 2007 was adjusted to 2018 dollars, and contingency was added. This assumption is reasonable since both bridges will have similar spans and heights. NMDOT standard serials for wire enclosed riprap lining were used to estimate quantities. The unit cost for shotcrete is based on an existing project that Smith is designing in Rio Rancho. The unit cost may be subject to change if this scenario is chosen as the mix design will have to be further refined based on geotechnical considerations, availability of materials and proximity to project site.

NM 26 bridge replacement costs were excluded from the list of proposed projects as that is part of NMDOT's future capital improvement project schedule over a 10-year horizon.

To assist in funding the total project was divided into 5 phases. **Figure 25** shows the locations for the 5 phases. Smith prepared a preliminary Engineers Opinions of Probable Cost for the 5 phases for both alternatives.

Table 8 summarizes the total cost for the five phases in the preferred order for **Alternative 1 Scenario 1**. Detailed cost estimates are included in **Appendix I** for further reference.



Legend

Phase

- Phase 1
- Phase 2
- Phase 3
- Phase 4
- Phase 4
- Phase 5
- Roads

Placitas Arroyo Drainage Master Plan

Prepared for

HATCH NEW MEXICO CHILE CAPITAL OF THE WORLD

Prepared by

SMITH ENGINEERING

Figure 25
Phasing Plan

July 2018

Document Path: Q:\SEC---PROJECTS\117112\GIS\20180529_PhasingPlan.mxd

Table 8: Total Costs for Alternative 1, Scenario 1 all Phases

ALTERNATIVE 1 - GRADING 11 - ENGINEER'S OPINION OF PROBABLE COST (EOPC) FOR RIPRAP LINED CHANNEL	
Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing & Embankment Protection at Cemetery	$643,000
Phase 2 - Channel & Levee Construction from Canal Road to NM 187	$8,136,000
Phase 3 - Channel & Levee Construction from NM 187 to NM 26	$10,093,000
Phase 4 - Canal Road Bridge	$4,324,000
Phase 5 - Channel and Levee Construction from Canal Road to Rio Grande	$4,477,000
Total Without Alternate Sediment Facility at Outlet of Placitas Arroyo	$27,673,000
Alternate Sediment Storage Pond at Outfall of Placitas Arroyo	$9,144,000
Total with Alternate Sediment Facility at Outlet of Placitas Arroyo	$36,817,000

These phases will take several years to fund and construct. As such, a time value analysis was performed to evaluate how the cost of the projects will increase over a 20-year period at 2-year intervals with a constant inflation rate of 3.5%.

Figure 26: Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing

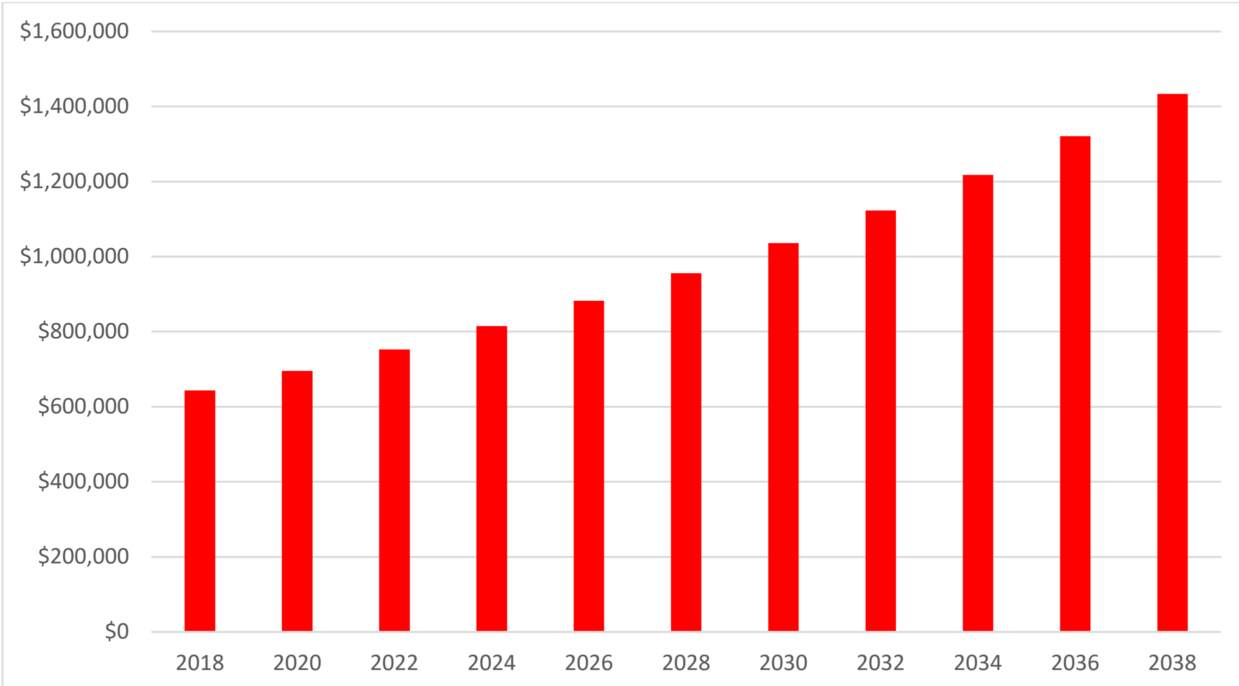


Figure 27: Phase 2 - Channel & Levee Construction from Canal Road to NM 187

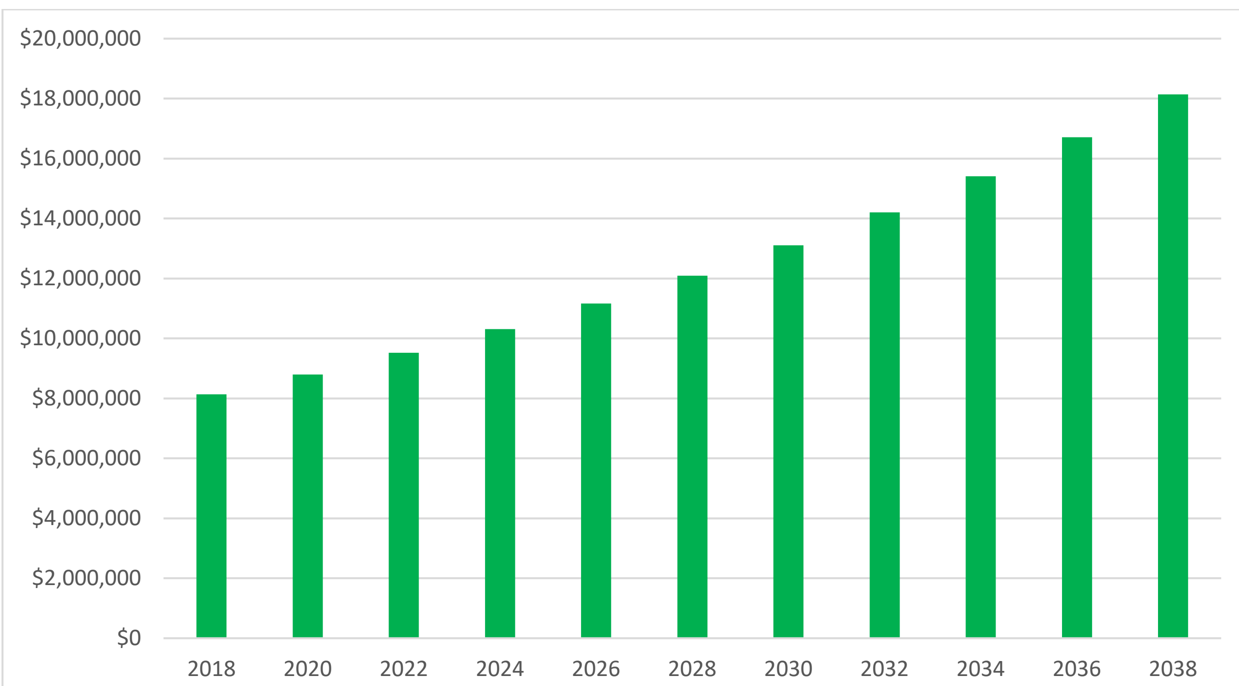


Figure 28: Phase 3 - Channel & Levee Construction from NM 187 to NM 26

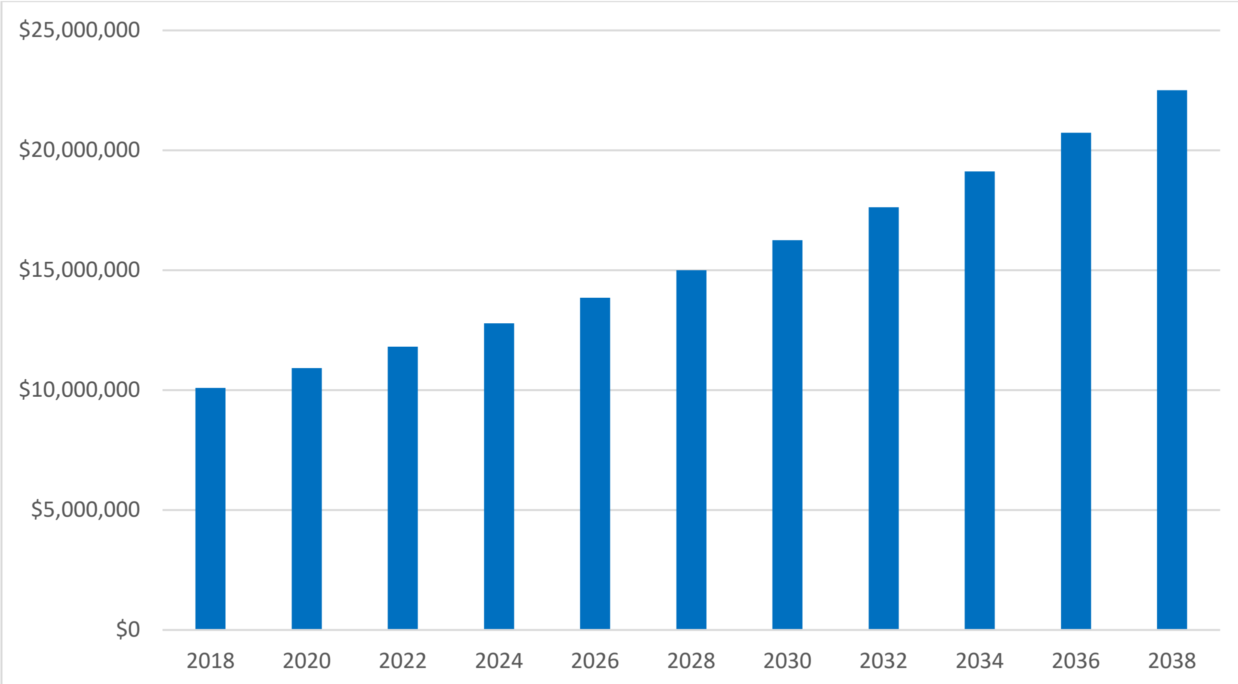


Figure 29: Phase 4 - Canal Road Bridge

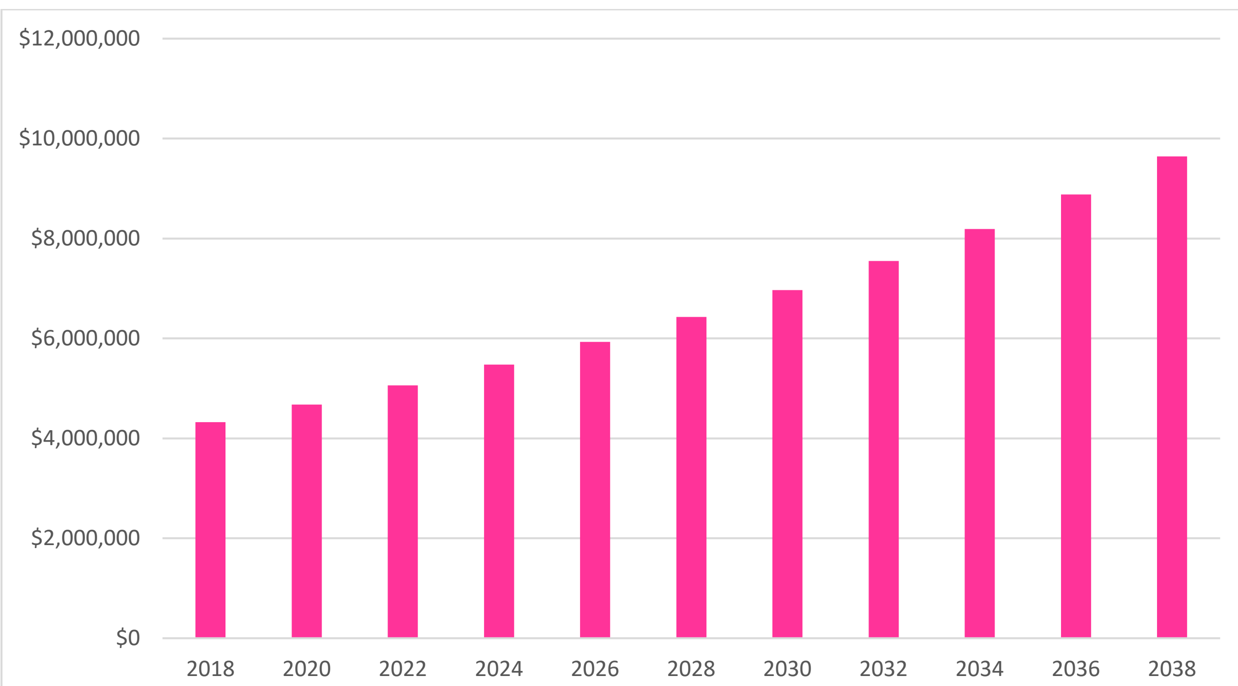


Figure 30: Phase 5 - Channel & Levee Construction from Canal Road to the Rio Grande

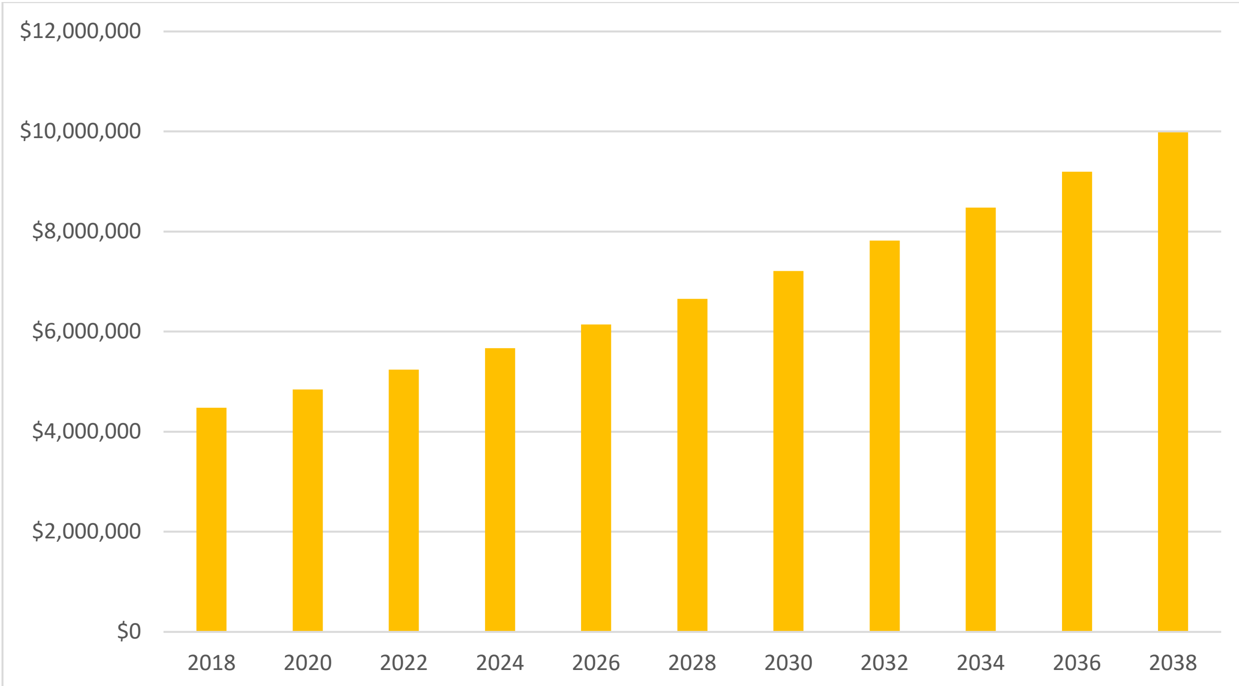
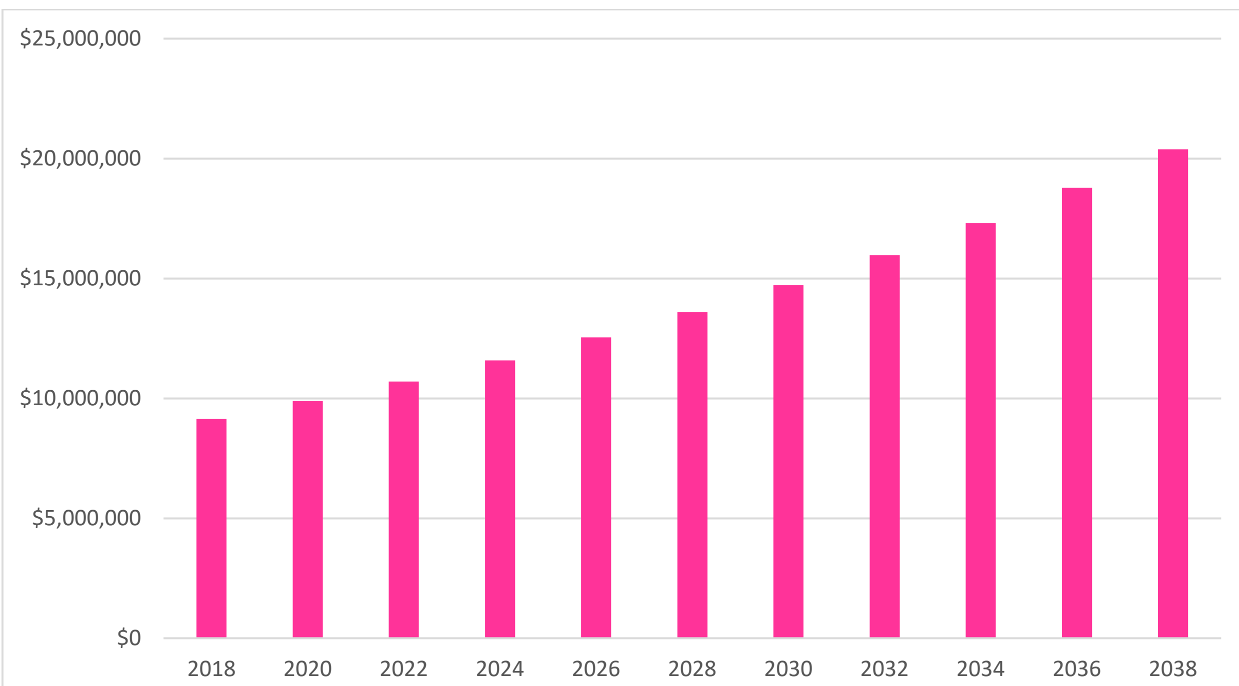


Figure 31: Alternate Sediment Storage Pond at Outfall of Placitas Arroyo



The alternate sediment basin is an additional feature that may be incorporated into Phase 5 as it will provide additional sediment control and velocity reduction.

Alternative 1 Scenario 2 assumes that the channel and levee will be a shotcrete lined facility. **Table 9** summarizes the costs of the 5 phases, the successive figures present the amortization of the phases over a 20-year period at an inflation rate of 3.5%.

Table 9: Total Costs for Alternative 1, Scenario 2 all Phases

ALTERNATIVE 1 - GRADING 11 - ENGINEER'S OPINION OF PROBABLE COST (EOPC) FOR SHOTCRETE LINED CHANNEL	
Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing & Embankment Protection at Cemetery	$643,000
Phase 2 - Channel & Levee Construction from Canal Road to NM 187	$6,725,000
Phase 3 - Channel & Levee Construction from NM 187 to NM 26	$7,965,000
Phase 4 - Canal Road Bridge	$4,324,000
Phase 5 - Channel and Levee Construction from Canal Road to the Rio Grande	$3,585,000
Total	$23,242,000
Alternate Sediment Storage Pond at Outfall of Placitas Arroyo	$7,211,000
Total with Alternate	$30,453,000

Figure 32: Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing

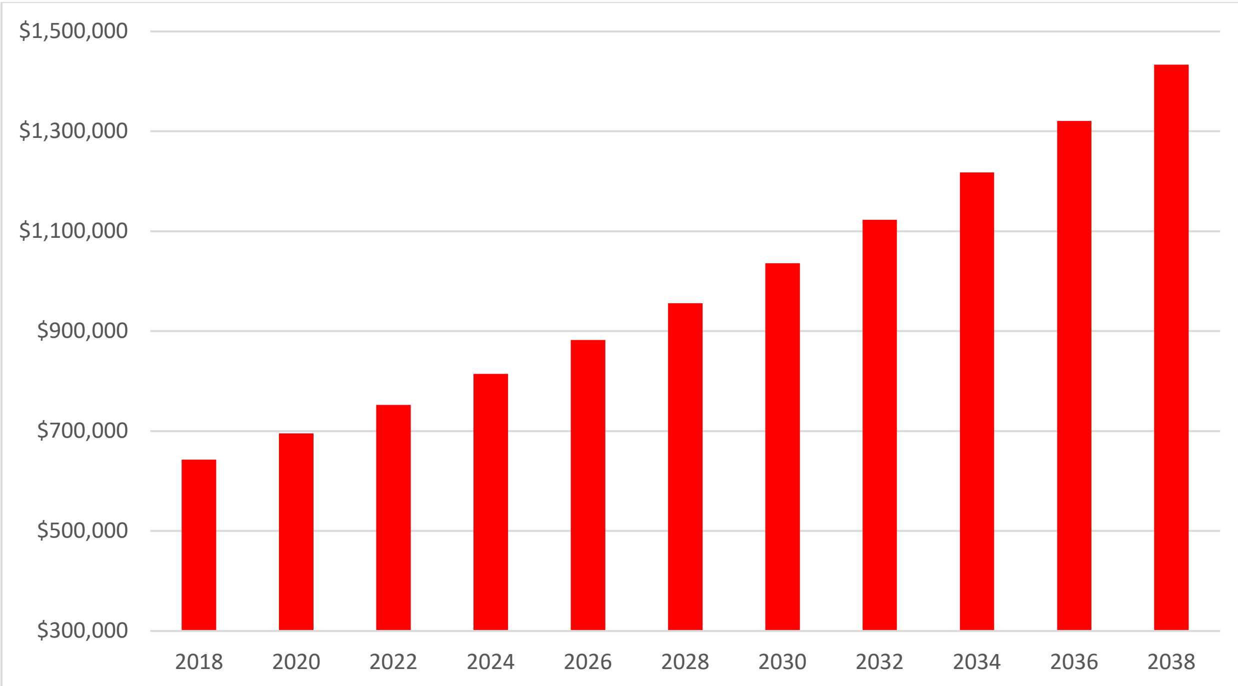


Figure 33: Phase 2 - Channel & Levee Construction from Canal Road to NM 187

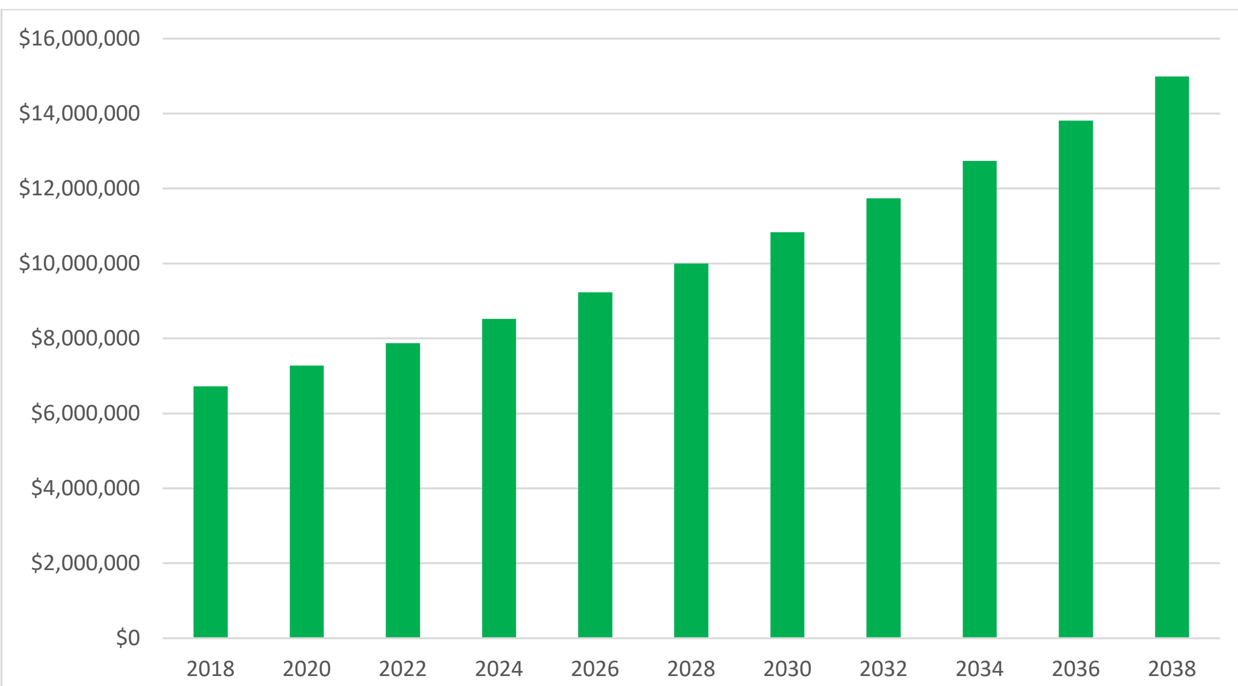


Figure 34: Phase 3 - Channel & Levee Construction from NM 187 to NM 26

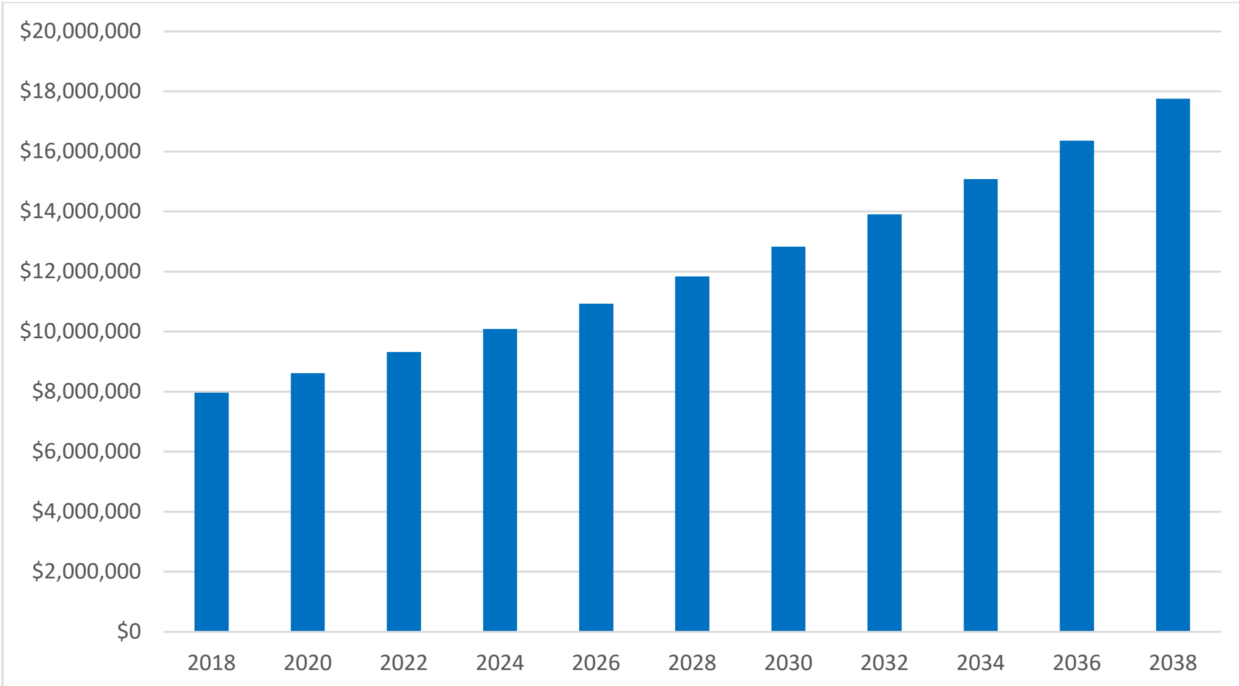


Figure 35: Phase 4 - Canal Road Bridge

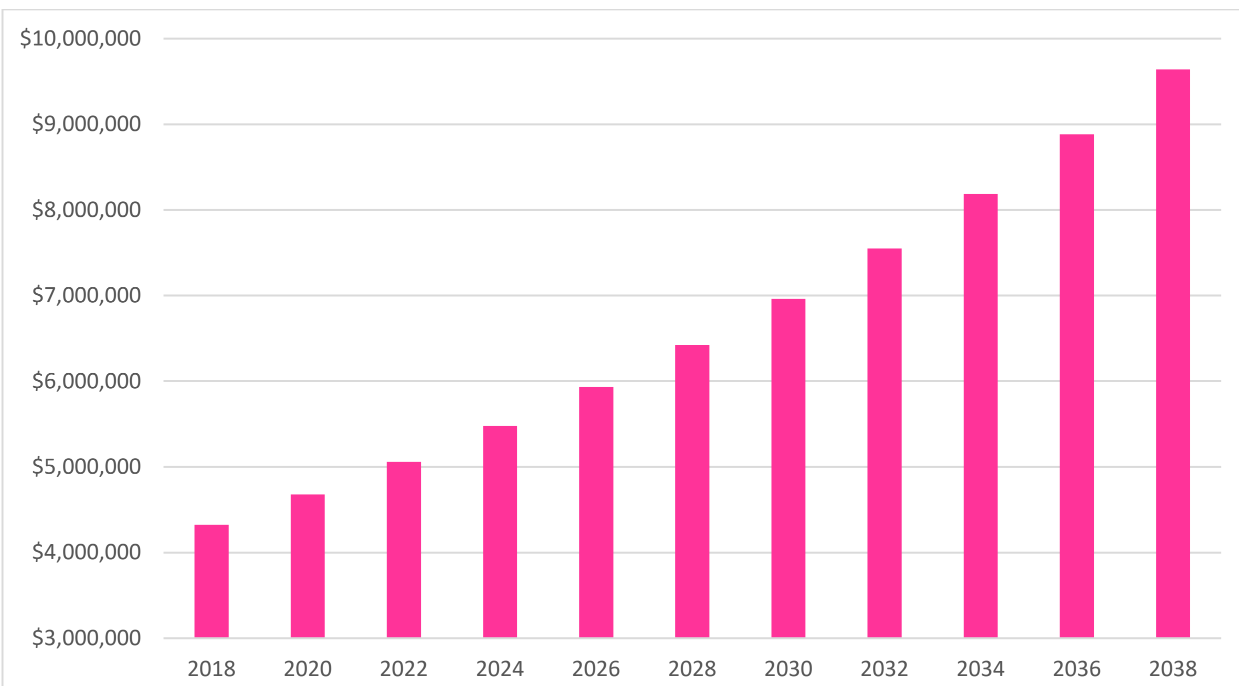


Figure 36: Phase 5 - Channel and Levee Construction from Canal Road to the Rio Grande

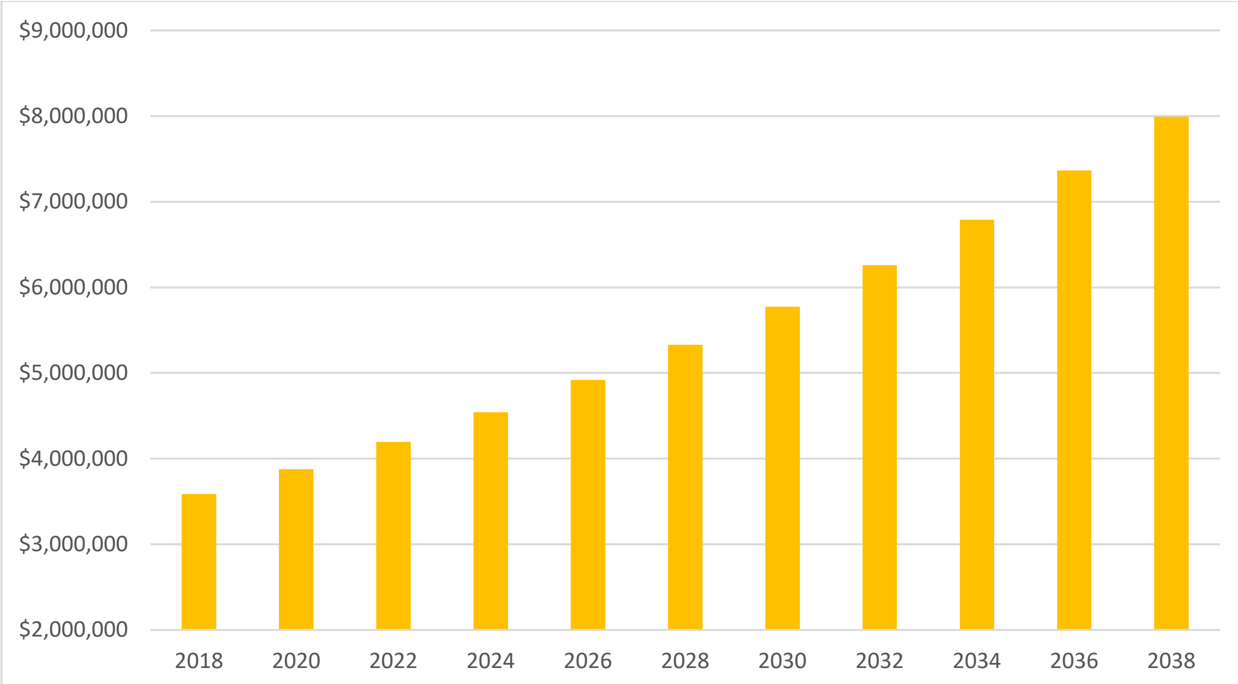
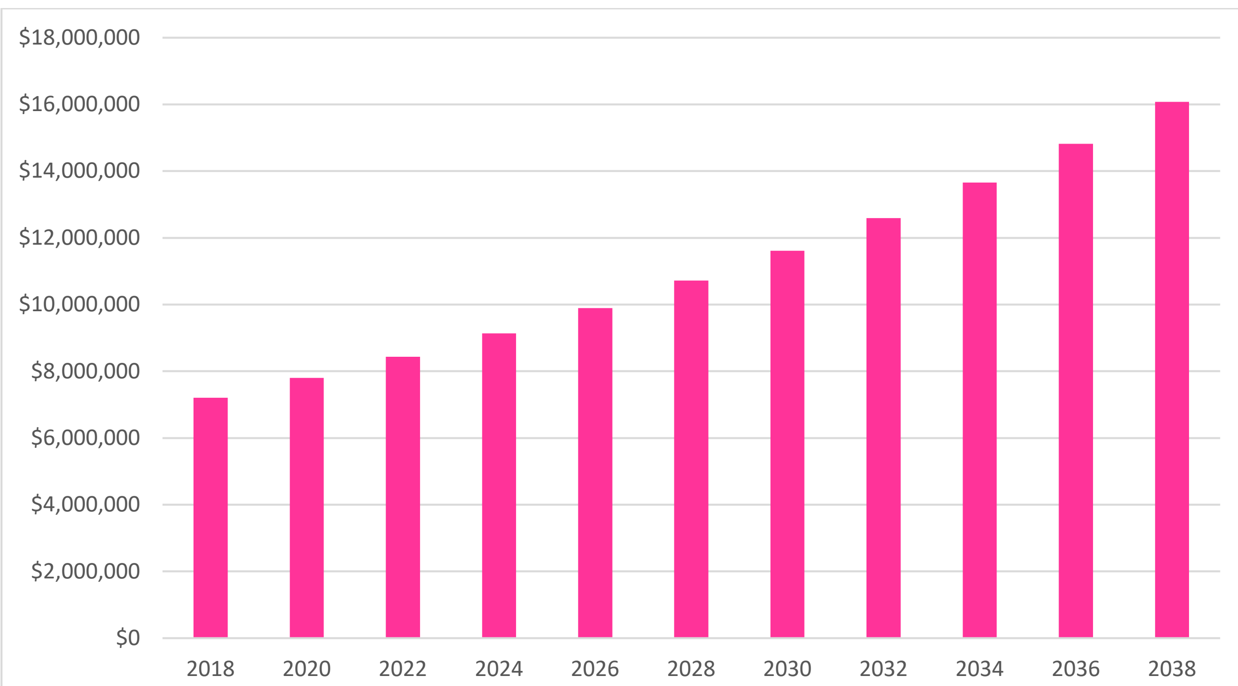


Figure 37: Alternate Sediment Storage Pond at Outfall of Placitas Arroyo



The alternate sediment basin is an additional feature that may be incorporated into Phase 5 as it will provide additional sediment control and velocity reduction.

Alternative 2 assumes full channel construction under the Individual Permit. Once again, two scenarios were considered. Scenario 1 considers a riprap lined channel whereas Scenario 2 considers a shotcrete lined channel. **Table 10** presents the cost summary for Scenario 1 and the succeeding figures preset the amortization of the projects over 20 years. Alternative 2 also includes $2 million for the Individual Permit.

Table 10: Total Costs for Alternative 2, Scenario 1 all Phases

ALTERNATIVE 2 - GRADING 12 - ENGINEER'S OPINION OF PROBABLE COST (EOPC) FOR RIPRAP LINED CHANNEL	
Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing & Embankment Protection at Cemetery	$643,000
Phase 2 - Channel & Levee Construction from Canal Road to NM 187	$7,847,000
Phase 3 - Channel & Levee Construction from NM 187 to NM 26	$8,985,000
Phase 4 - Canal Road Bridge	$4,324,000
Phase 5 - Channel and Levee Construction from Canal Road to Rio Grande	$3,983,000
Individual Permit	$2,000,000
Total Without Alternate Sediment Facility at Outlet of Placitas Arroyo	$27,782,000
Alternate Sediment Storage Pond at Outfall of Placitas Arroyo	$9,155,000
Total with Alternate Sediment Facility at Outlet of Placitas Arroyo	$36,937,000

Figure 38: Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing

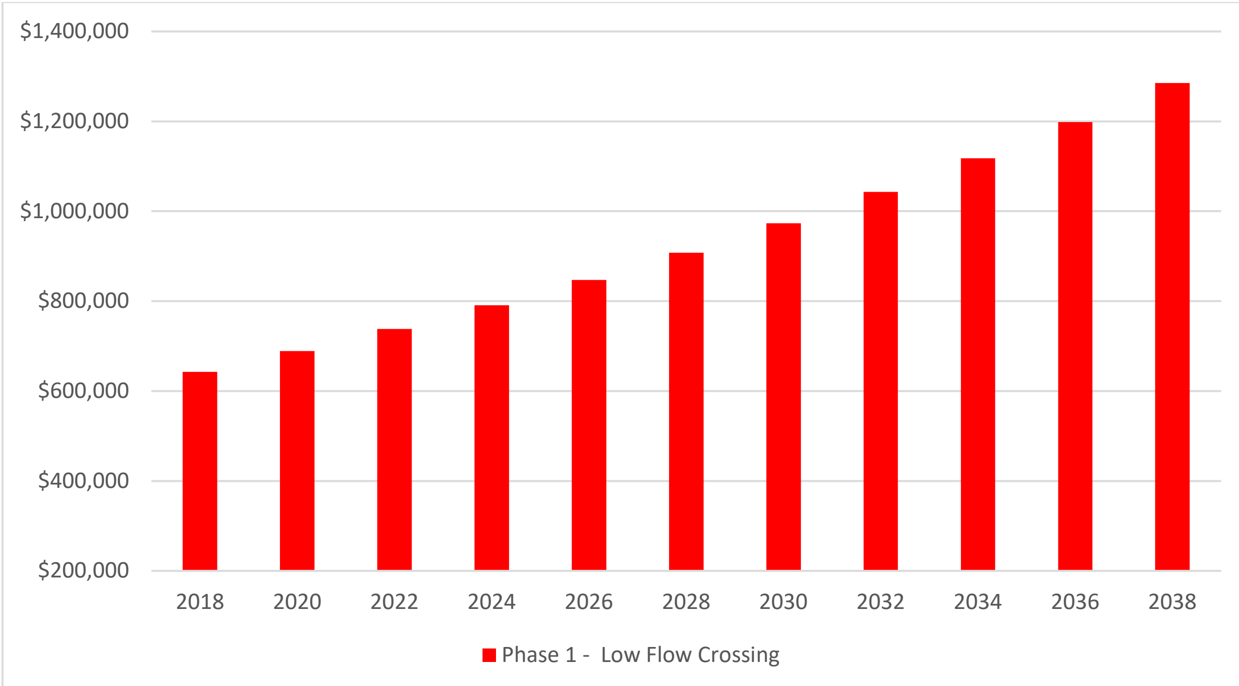


Figure 39: Phase 2 - Channel & Levee Construction from Canal Road to NM 187

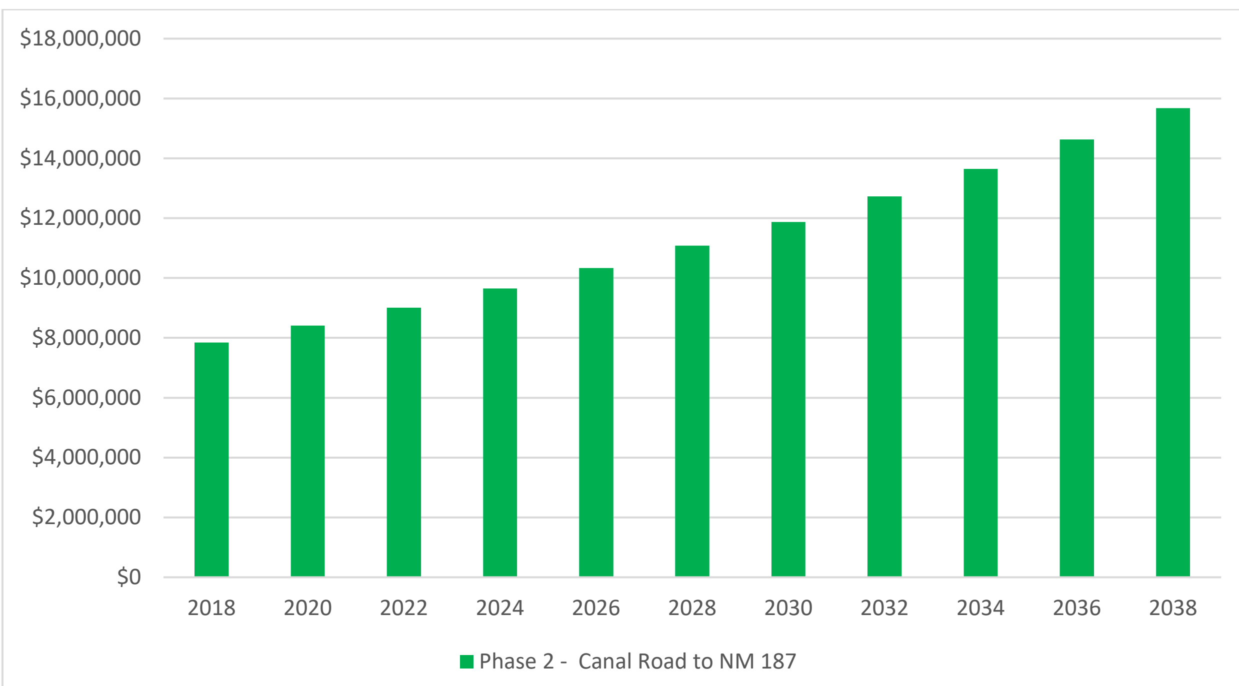


Figure 40: Phase 3 - Channel & Levee Construction from NM 187 to NM 26

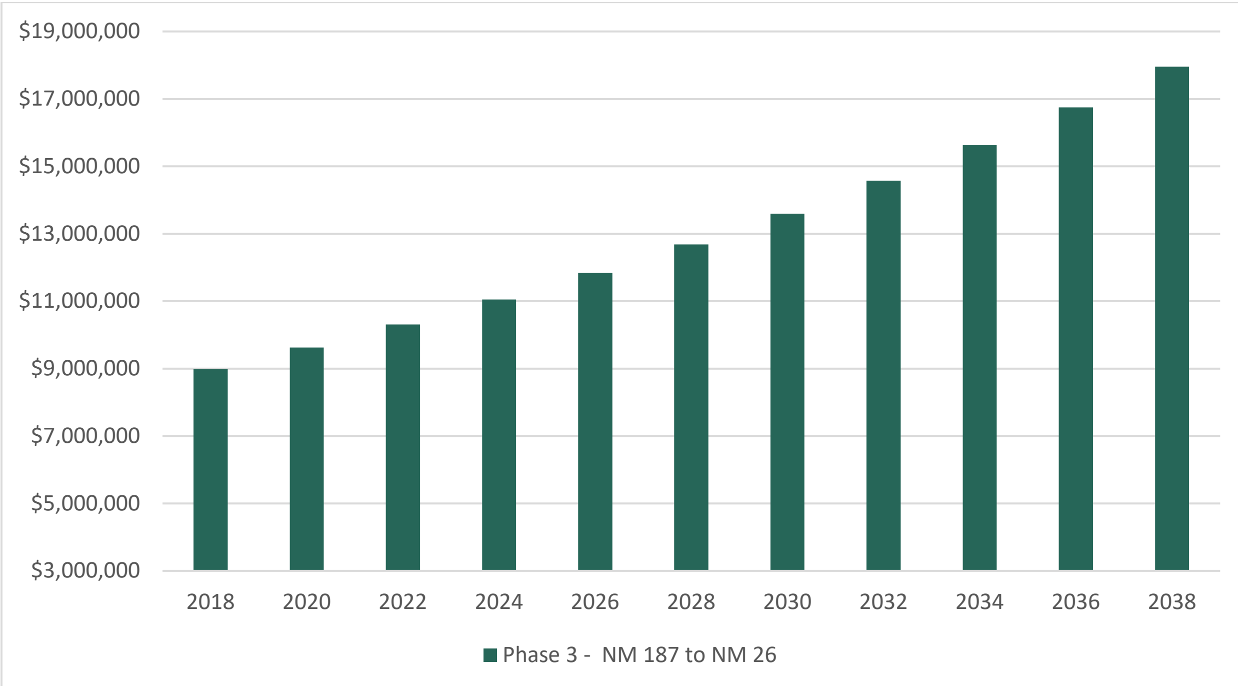


Figure 41: Phase 4 - Canal Road Bridge

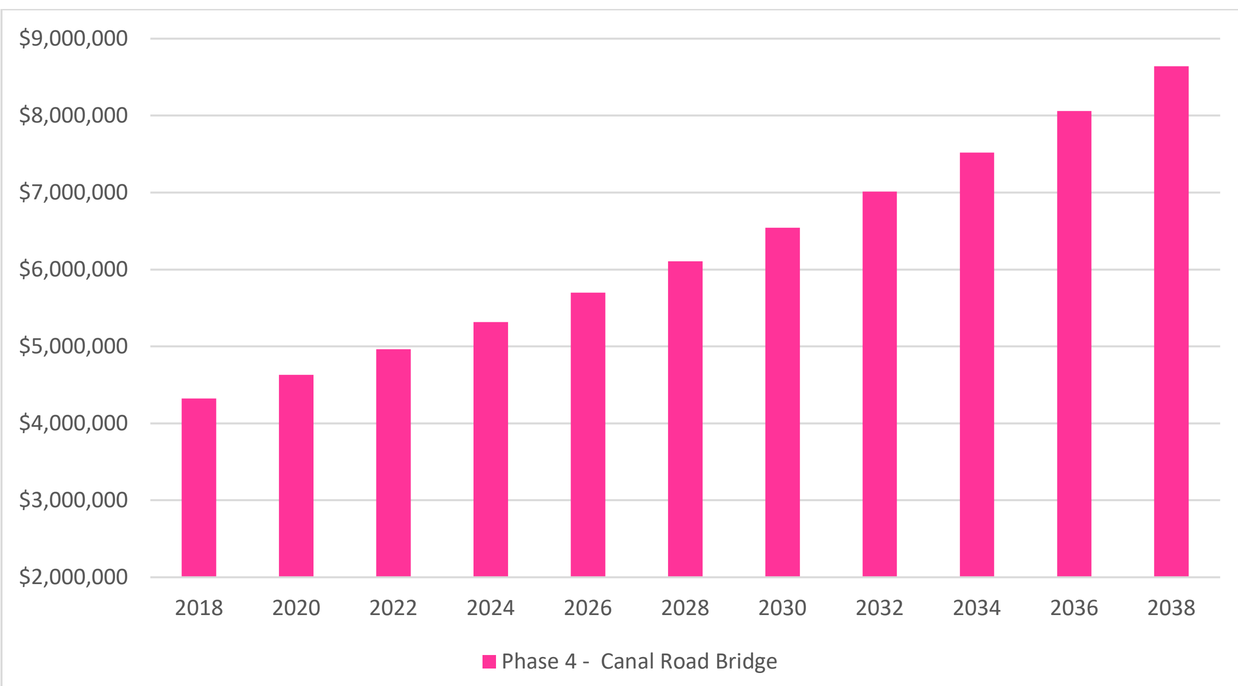


Figure 42: Phase 5 - Channel & Levee Construction from Canal Road to the Rio Grande

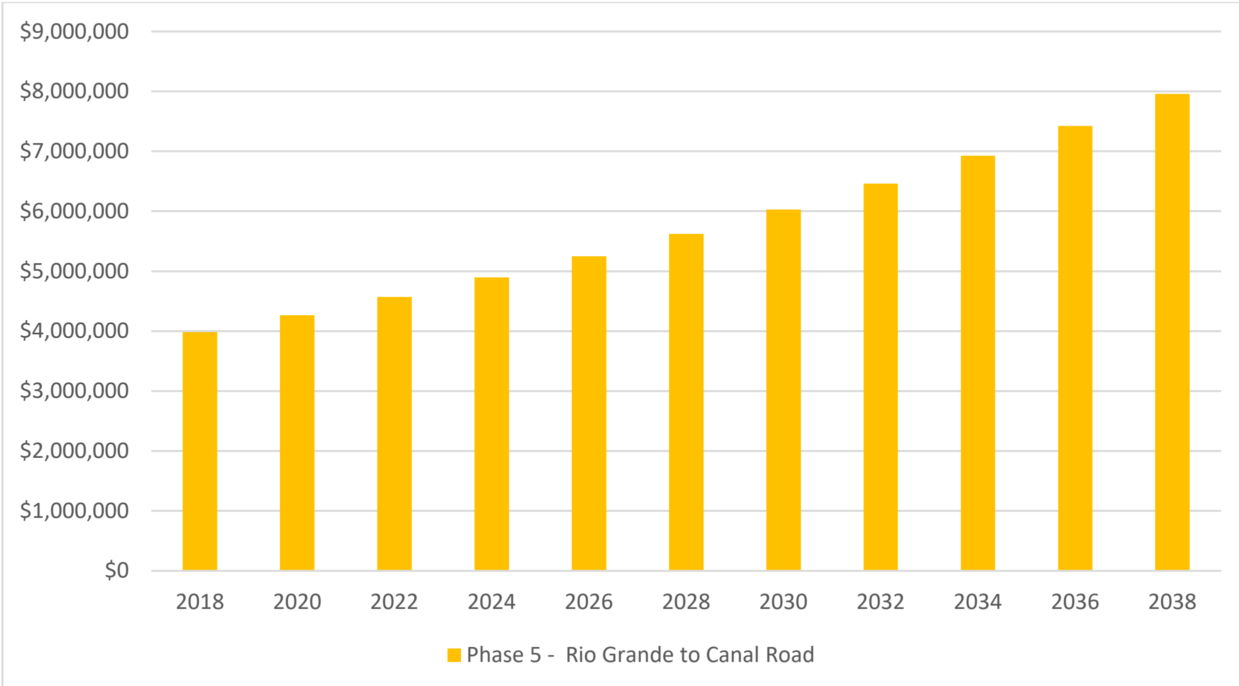
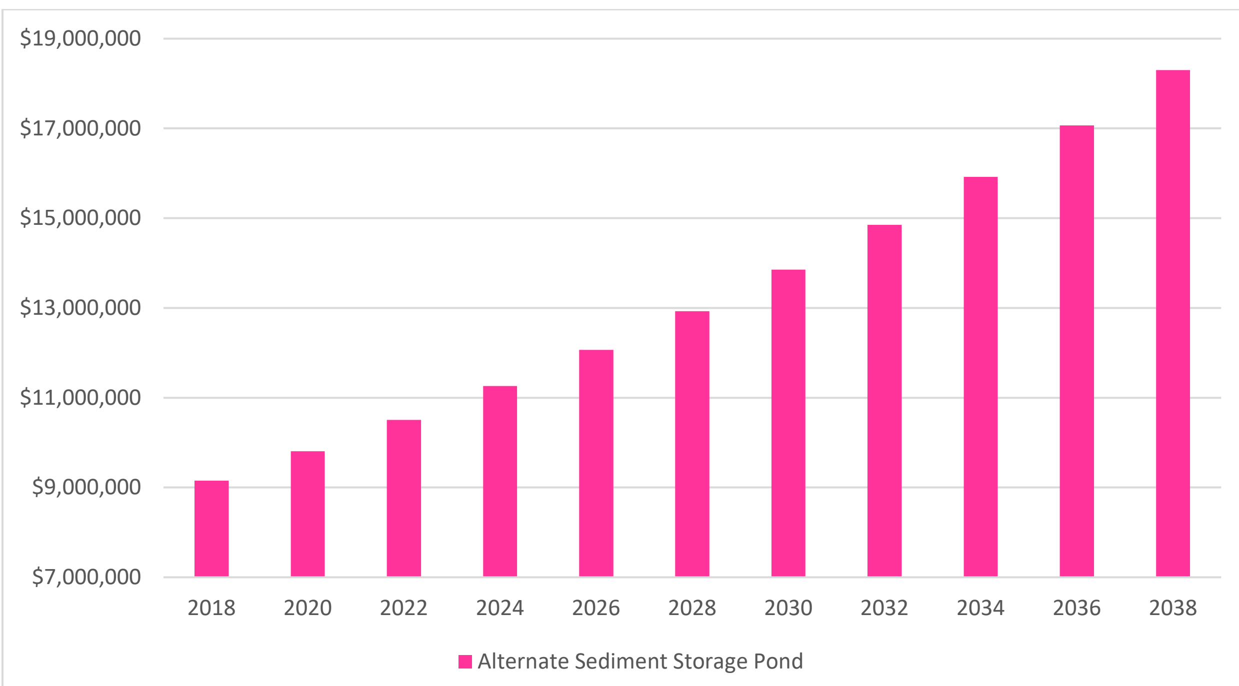


Figure 43: Alternate Sediment Storage Pond at Outfall of Placitas Arroyo



The alternate sediment basin is an additional feature that may be incorporated into Phase 5 as it will provide additional sediment control and velocity reduction.

Alternative 2 Scenario 2 assumes that the channel and levee will be a shotcrete lined facility. **Table 11** summarizes the costs of the 5 phases; the successive figures present the amortization of the phases over a 20-year period at an inflation rate of 3.5%.

Table 11: Total Costs for Alternative 2, Scenario 2 all Phases

ALTERNATIVE 2 - GRADING 12 - ENGINEER'S OPINION OF PROBABLE COST (EOPC) FOR SHOTCRETE LINED CHANNEL	
Phase 1 - Removal of Canal Rd Box Culverts and Build Low Flow Crossing & Embankment Protection at Cemetery	$643,000
Phase 2 - Channel & Levee Construction from Canal Road to NM 187	$6,398,000
Phase 3 - Channel & Levee Construction from NM 187 to NM 26	$6,859,000
Phase 4 - Canal Road Bridge	$4,324,000
Phase 5 - Channel and Levee Construction from Canal Road to Rio Grande	$3,039,000
Individual Permit	$2,000,000
Total Without Alternate Sediment Facility at Outlet of Placitas Arroyo	**$23,263,000**
Alternate Sediment Storage Pond at Outfall of Placitas Arroyo	$7,222,000
Total With Alternate Sediment Facility at Outlet of Placitas Arroyo	**$30,485,000**

Figure 44: Phase 1 - Removal of Canal Road Box Culverts and Build Low Flow Crossing

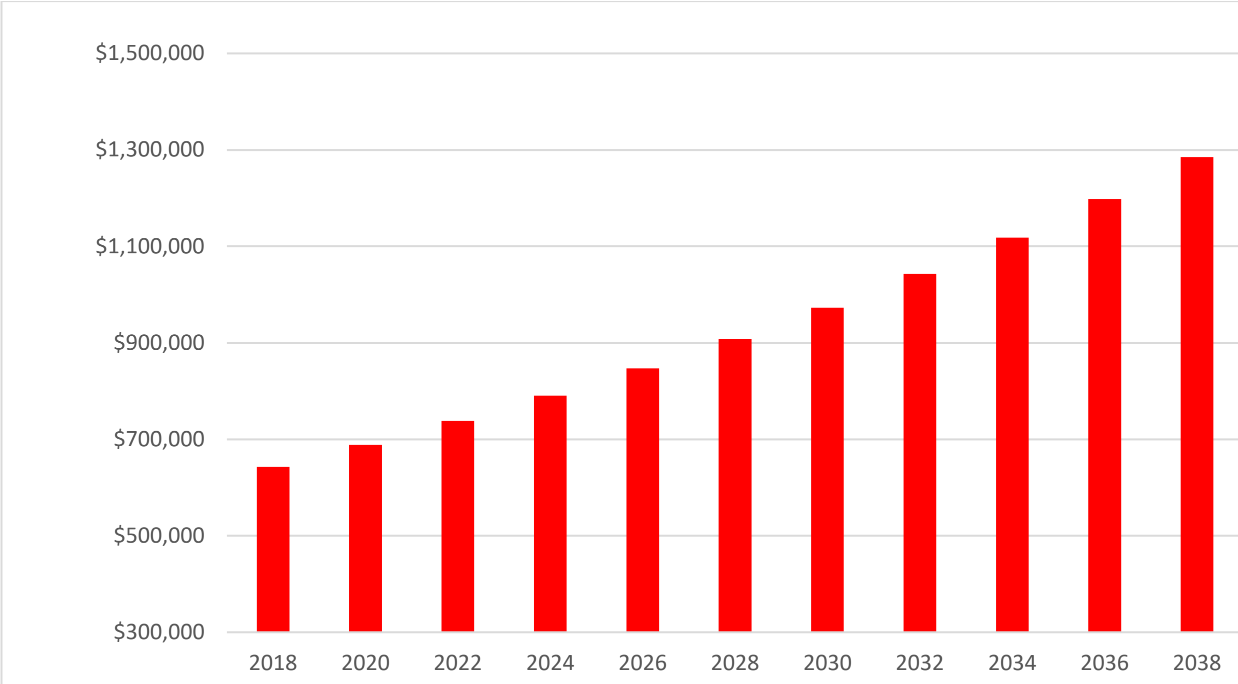


Figure 45: Phase 2 - Channel & Levee Construction from Canal Road to NM 187

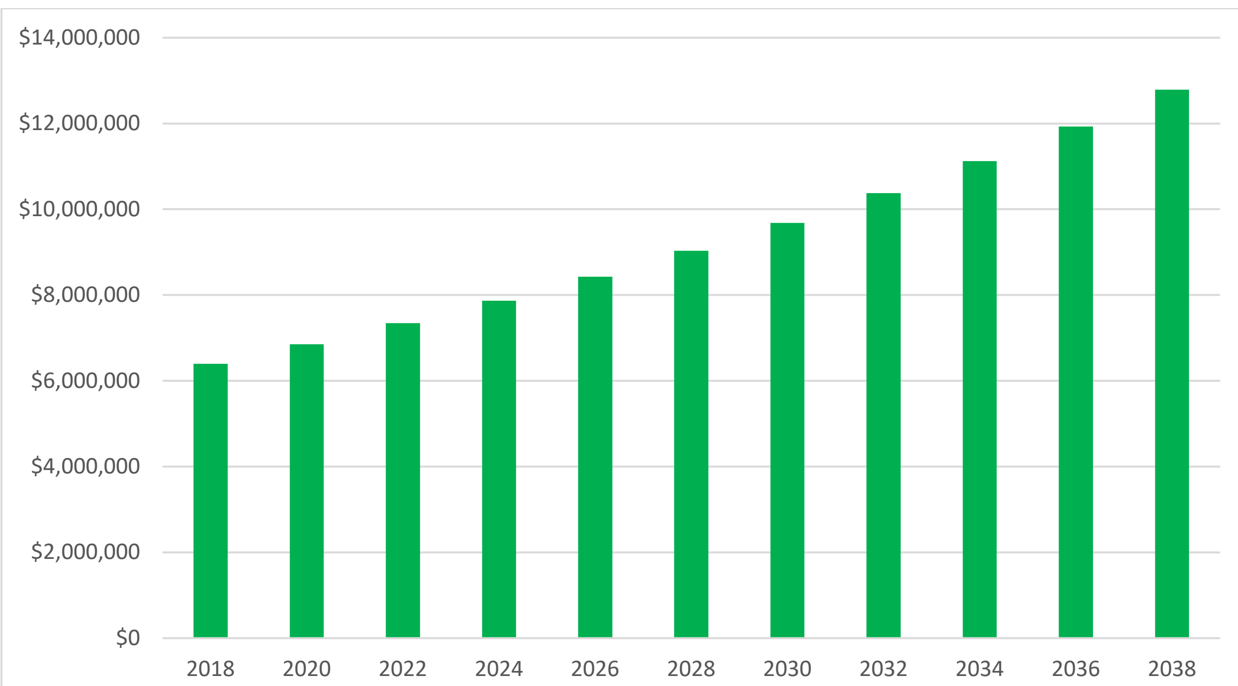


Figure 46: Phase 3 - Channel & Levee Construction from NM 187 to NM 26

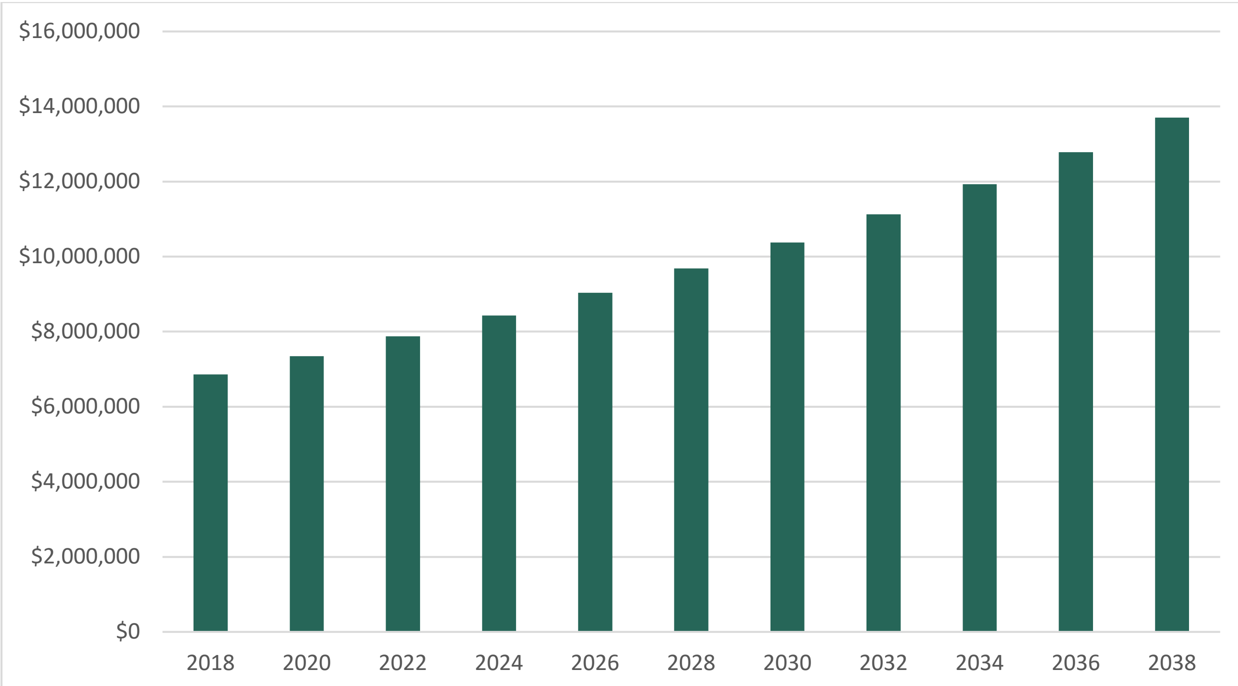


Figure 47: Phase 4 - Canal Road Bridge

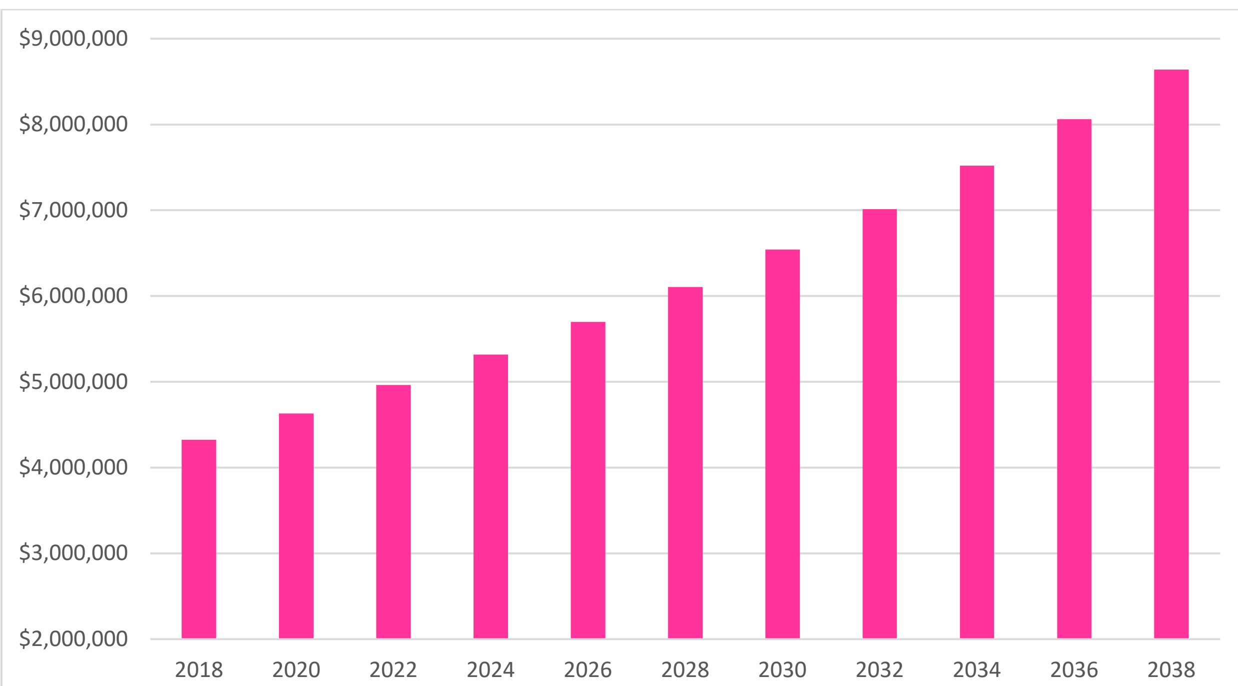


Figure 48: Phase 5 - Channel & Levee Construction from Canal Road to the Rio Grande

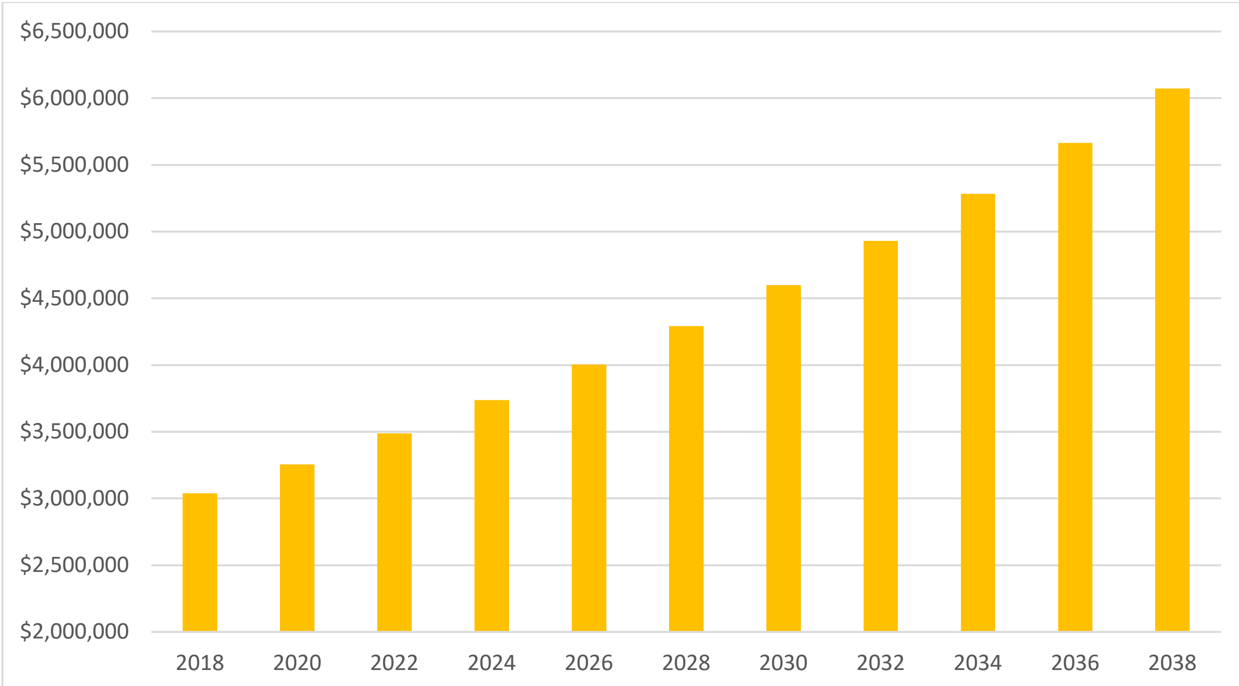
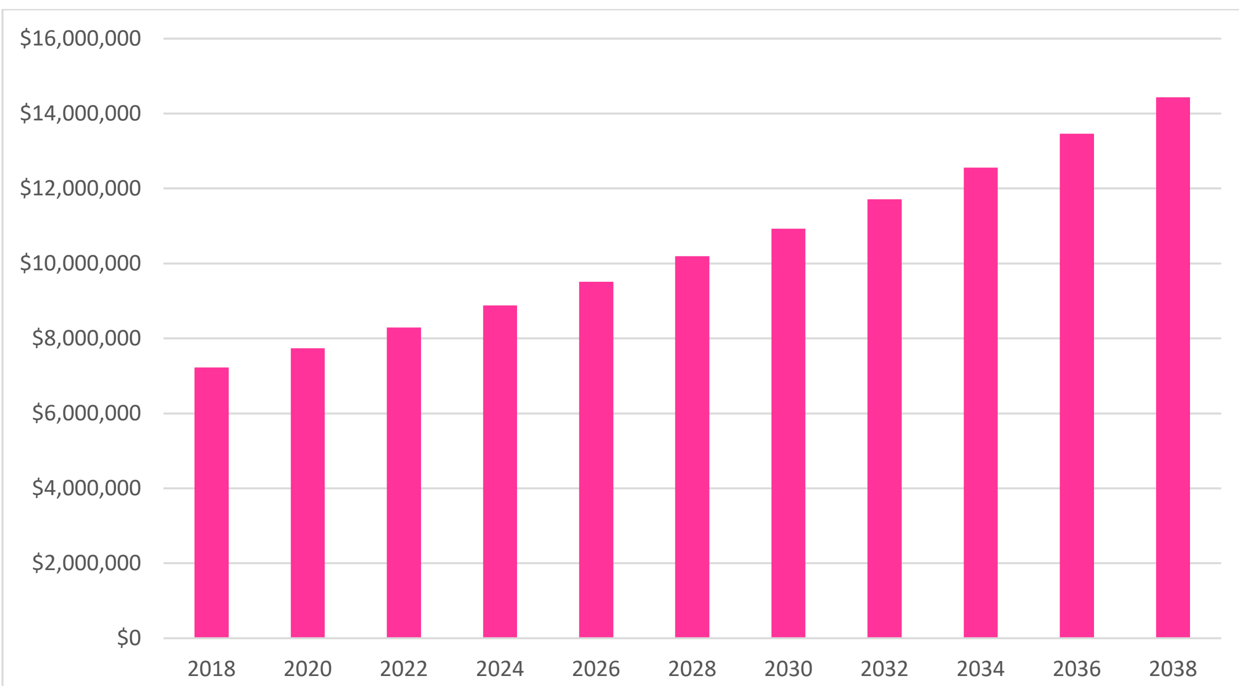


Figure 49: Alternate Sediment Storage Pond at Outfall of Placitas Arroyo



5.2 CONCLUSIONS AND RECOMMENDATIONS

The conveyance and sediment control facilities presented will provide significant flood mitigation for the design storm. The total project cost is significant with many aspects that will require further consideration and direction from the VOH and DACFC. The table below summarizes the total costs for the two alternatives and two scenarios.

Alternative Description	Scenario 1: Riprap Lined Channel Including Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 1: Riprap Lined Channel Excluding Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 2: Shotcrete Lined Channel Including Alternate Sediment Facility at Outlet of Placitas Arroyo	Scenario 2: Shotcrete Lined Channel Excluding Alternate Sediment Facility at Outlet of Placitas Arroyo
Alternative 1: Benched Channel Design Assuming Nationwide Permit	$36,817,000	$27,673,000	$30,453,000	$23,242,000
Alternative 2: Trapezoidal Channel Design Assuming Individual Permit	$36,937,000	$27,782,000	$30,485,000	$23,263,000

The difference in cost between Alternative 1 and 2 is marginal. However, the time component between the two alternatives as far as when the phases for design and construction can begin could be at a minimum of two years for Alternative 2 due to the stringent individual permit requirements. This offset in time will increase the overall costs of the all successive phases based on the time-value analysis presented earlier. It will also delay the flood protection component for the VOH leaving the area more susceptible to flooding while the permitting process is being resolved. There are significant hydraulic advantages in using a shotcrete lined channel as the depth of flow in the overall channel will be lower providing more freeboard at the bridges. This means that the overall design for the levee height may be reduced which in turn will save construction costs. The cost of the shotcrete lined channel is also significantly lower than the riprap lined channel. However, the mix design for the shotcrete must be further refined based on final design parameters and this may cause the unit cost to increase. The shotcrete lined surface may also be vulnerable to impact damage from the large boulders transported in the channel. This could cause localized failure which if unchecked may compromise the overall levee structure. Further energy dissipation alternatives will have to be considered at the outlet of the channel as the flow velocities will be higher at the outlet and this will increase the cost.

The wire enclosed riprap lining has a significantly higher cost as it is a very labor-intensive process to build riprap lining. It will, however, be more impervious to impact damage. The hydraulics are not as efficient due to the high energy losses from the extremely rough surface. It will however mitigate the need for outlet energy dissipation as channel velocities will be lower.

Canal Road options have significant financial disadvantages. The VOH and DACFC will have to acquire approximately $643,000 to abandon the box culverts, build the low flow crossing and install early warning traffic control devices. Later the low flow crossing will have to be reconstructed into a full span bridge at a cost of approximately $4.3 million making the overall cost of these phases at least $4.97 million. The early warning traffic control device costing around $115,000 will be unnecessary at that point. Earnest consideration must be given to the financial and economic viability and sustainability of the Canal Road crossing option.

The alternate sediment basin is an expensive item and at this stage considered secondary. The VOH and DACFC may want to consider it in collaboration with IBWC in terms of funding. It will also require acquiring property which is agricultural land, actively farmed.

The final design option will be a function of which permit is pursued. The VOH and DACFC will further discuss the most feasible permit in the near future.

SECTION 6. REFERENCES

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Legend

- Analysis Points

Tc Flowpaths

Type of Flow

- Sheet Flow
- Shallow Concentrated Flow
- Open Channel Flow
- 10ft Contours
- Railroads
- Roads
- Placitas Arroyo
- Subbasin Boundary
- 22 0.14 Subbasin ID & Area In Sq Miles

10 FT CONTOUR DATA EXTRACTED FROM 2014 DIGITAL ELEVATION MODELS (DEM). 2014 DEM AND ORTHOPHOTOGRAPHY WAS PROVIDED BY DONA ANA COUNTY FLOOD COMMISSION

Placitas Arroyo Drainage Master Plan

PREPARED FOR

HATCH NEW MEXICO CHILE CAPITAL OF THE WORLD

PREPARED BY

SMITH ENGINEERING

Figure A Drainage Basin Map

July 2018

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