CITY OF SANTA FE NEW MEXICO PUBLIC WORKS DEPARTMENT

ARROYO DE LOS CHAMISOS CROSSING PROJECT

DRAFT FINAL DRAINAGE REPORT

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CITY OF SANTA FE NEW MEXICO
PUBLIC WORKS DEPARTMENT

PROJECT NO.: LP50039
DATE: JUNE 2024

WSP
2440 LOUISIANA BLVD NE
SUITE 400
ALBUQUERQUE, NM

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1 EXECUTIVE SUMMARY

1.1 SUMMARY

City of Santa Fe has contracted WSP to develop 30 percent construction plans to connect Richards Avenue from Cerrillos Rd to Rodeo Road. The analysis in this study begins at the high point in Richards Ave at station 19+50 and ends at the intersection of Richards Avenue and Cerrillos Rd on the north side at station 58+00. As part of the design project, a drainage analysis for the area was completed. The analysis evaluated offsite and onsite runoff affecting the proposed area. Additionally, the project will require traversing the existing Arroyo de Los Pinos (ADLP) and Arroyo de Los Chamisos (ADLC). ADLP has an existing arch culvert while ADLC is an at grade, low water crossing. Both arroyos are identified as FEMA flood hazard zones categorized as Zone AE.

The study identified three major offsite tributary areas to the proposed project in Richards Avenue:

- Cerrillos Road
- Mark Brandt Park
- Rodeo De Santa Fe

A detailed watershed analysis was completed to determine the total inflow from all three basins. Onsite hydrology based on the proposed roadway design was also completed. The cumulative analysis was then used to develop roadway drainage concepts that became the basis of design for the 30% construction plans.

An SRH-2D hydraulic model was developed using FEMA FIS flows to understand the channel hydraulics of ADLP and ADLC. Based on the results, recommendations were made for bridge alternatives that will span the arroyos including low chord elevations so that the proposed bridges will provide 2 ft of freeboard during the design storm.

The major design components for Richards Avenue based on this analysis will include:

- Conveyance system of inlets and storm drains consisting of pipe diameters ranging from 24-inches to 66 inches
- Modification of the inlet structure from Mark Brandt Park from a 48-inch CMP to a 66-inch RCP
- Realignment of the outlet pipe from Mark Brandt Park so that it is completely in City right of way
- Installation of various erosion control outlet structures at the location of new outfalls
- Installation of green storm water applications throughout the project limits
- A new 220-ft two span bridge across Arroyo de Los Chamisos
- A new 45-ft clear span bridge across Arroyo de Los Pinos
- A grade control structure downstream of the new bridge at Arroyo de Los Pinos

1.2 DESCRIPTION AND PURPOSE OF PROJECT

This draft final drainage report was developed in support of the proposed roadway design for Richards Avenue connection from Cerrillos Rd to Rodeo Rd. Refer to **Figure 1** for the project location map. The report presents the findings of an area-wide drainage analysis and provides proposed roadway drainage structures and solutions to address area flooding issues.

The project will connect the north and south segments of Richards Ave with a new roadway and two new bridges over the Arroyo de Los Pinos (ADLP) and the Arroyo de Los Chamisos (ADLC). The existing portions of Richards Ave will be repaved, and a new intersection will be designed at the intersection of Richards and Camino Del Prado. New storm drain has been proposed to drain the existing and proposed areas of Richards Ave. As part of the analysis, the drainage flowing to Marc Brandt Park was also analyzed. The park currently serves as a multi-use facility that has a tributary area of approximately 0.46 square miles. There have been incidents of flooding at the southern end of the park and the City of Santa Fe (City) requested that the area be analyzed, capacity issues identified, and conceptual recommendations be developed.

1.3 FIELD OBSERVATION

WSP employees visited the site multiple times to observe the current conditions, verify drainage patterns, basin boundaries, do a field assessment of the ADLP and ADLC, and Marc Brandt Park. The condition of several critical structures was observed as well. The existing outlet from Marc Brandt Park is a 48-inch corrugated metal pipe (CMP). The downstream side of the pipe is in poor condition and is partially obstructed as shown in **Photo 1**. The downstream channel has a rocky bed as shown in **Photo 2**.

Photo 1: Partially Obstructed 48-inch outlet Pipe

Photo 2: Rocky Streambed downstream of Marc Brandt Pipe Outfall

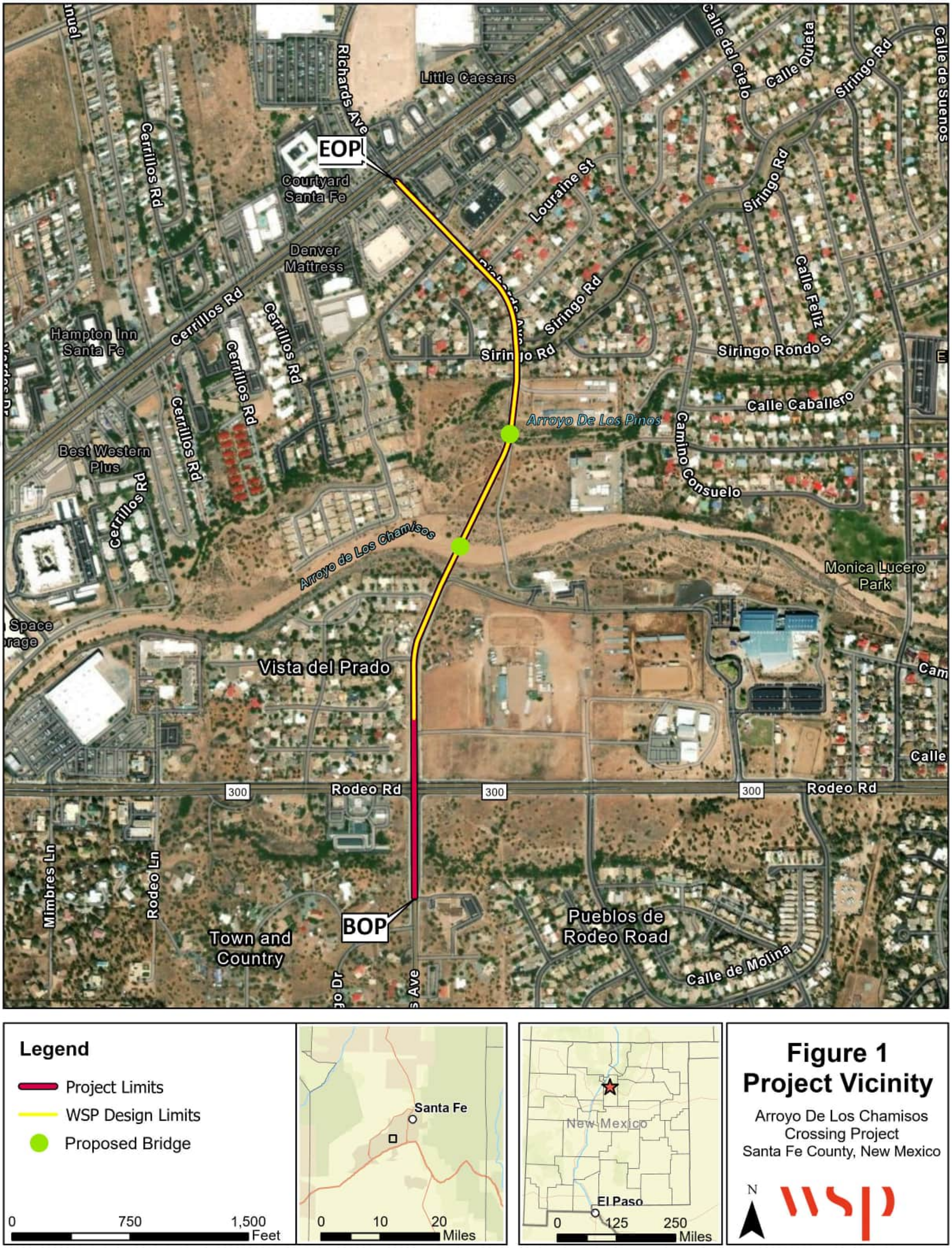

Figure 1
Project Vicinity

Arroyo De Los Chamisos
Crossing Project
Santa Fe County, New Mexico

J:\30900483 CoSanta Fe On-Call Design Services\03 Task Orders\Task 04 - ADLC IC and ID\04 Engineering - CN LP50039 (See ProjectWise)\Proposed\Drainage\Preliminary\GIS\Exhibits\maps\Fig1ProjectArea

2 EXISTING CONDITIONS

2.1 EXISTING FLOOD CONTROL STRUCTURES

Many of the commercial areas along Cerrillos Rd have retention facilities that were constructed as part of the development plan; this includes the detention pond at Franklin Miles Park at the headwaters of this watershed at the intersection of Camino Carlos Ray and Siringo Rd, however, Marc Brandt Park is the only major detention facility in the study area that can act as a regional detention facility. The park has a sidewalk and lighting facilities along the bottom of the pond. Some the foundations for the lighting poles are starting to get exposed due to erosion caused by the runoff from smaller storms as shown on **Photo 3**.

The walk area also gets saturated during the monsoon season often making the trails inaccessible. Runoff from the tributary area primarily drains to the park via overland flow and there are no major storm drain systems upstream of the park. The existing park pond is a long linear system that is connected through a series of 48-inch pipes and a bridge crossing at Calle Vianson. Siringo Rd forms a one-way loop around the park and has several inlets that capture and divert surface runoff via inlets and 24-inch pipes as shown in **Photo 4**.

Photo 3: Exposed Footing for Existing Light Pole.

Photo 4: Pipe Outlet from Existing Inlet on Siringo Rd.

2.2 EXISTING DRAINAGE INFRASTRUCTURE

2.2.1 EXISTING STORM DRAIN

There is an existing storm drain system in Cerillos Rd that collects roadway drainage through a system of curb drop inlets. The roadway south of Arroyo de Los Chamisos has one set of inlets near the intersection of Cam Del Prado that outlets into a pond adjacent to property at 2310 Cam Del Prado. This storm drain collects flow from Richards Ave to the south and flows from the Rodeo De Santa Fe (RDSF) property to the east. Sediment and debris from RDSF overflows into the roadway clogging the inlets.

2.2.2 EXISTING CROSSING CULVERTS

There are three existing crossing culverts on the project all flowing east to west. From north to south, these are as follows:

- A 48” CMP flows from Marc Brandt Park upstream of Richards to the Arroyo de Los Pinos. The downstream end of the culvert has large amounts of debris covering it that appears to have been deposited by erosion from high water events. This culvert also appears to cross under private property.
- A 24” CMP just south of the existing paved roadway drains water from the east side of Richards Ave to the unnamed arroyo. Field inspection and survey data show this culvert to have sedimentation at the inlet and erosion issues at the outfall. The culvert also has very minimal cover under the existing service road.
- There is a 128” span X 83” rise CMPA at the service road crossing the Arroyo de Los Pinos. This culvert has headwalls at both the upstream and downstream ends. 3-ft of head cut exists at the outlet of the culvert.

Photo 5: Existing Arch Pipe Crossing at Arroyo de Los Pinos

2.3 WATERSHED DESCRIPTION

The project limits have tributary drainage from Cerrillos Rd, Marc Brandt Park and the two major arroyos in the ADLP and ADLC. On the south side of ADLC, there are offsite flows that drain to Richards Ave from the RDSF. An overall HEC-HMS model was developed for Cerrillos Rd and Marc Brandt Park, however, flows from the FEMA Flood Insurance Study for ADLP and ADLC were adopted to model the open channel hydraulics for the two arroyos and to size the bridge crossings at ADLP and ADLC. **Figure 2** shows the overall drainage area that was modeled.

2.3.1 CERRILLOS ROAD

Flow along the south side of Cerrillos road travels west along the roadway and directed along Cerrillos with the use of valley gutters in the intersections until Richards Ave and the turnout just east of Richards Ave. Flow reaches Richards Rd from as far east as the outfall of Ashbaugh park outfall roughly two miles east.

The existing storm drain system from Richards Ave to St. Michaels Dr. along Cerrillos Rd. was designed with the 10-yr storm as the design storm and allows excess flow to follow existing drainage patterns as described in Addendum #1 to "Final Drainage Report Cerrillos Road Reconstruction Project" prepared by Bohannan Huston Inc (BHI) in June 2000.

Cerrillos road between St. Michaels Dr and St. Francis Dr is currently in design by WSP with the drainage design being performed by BHI. The storm drain system in this project is being designed for the 10-yr storm with the 25-yr storm as the check storm as described in the Preliminary Drainage Report for NM 14 (Cerrillos Rd) prepared by BHI in June 2022.

The overall watershed is heavily urbanized with commercial and business areas as the primary land use. The subbasins draining to Cerrillos Rd were delineated and incorporated into the overall watershed model. An inlet capacity analysis was done to confirm the potential offsite flows that would drain into Richards Ave. **Figure 3** shows the roadway basins delineated and the flow accounting performed based on the capture capacity of the inlets along Cerrillos.

Special Conditions for LR_28A and LR_28B

Basins LR_28A and LR_28B currently drain to Richards Avenue. These two basins have a highly impervious development and generate higher peak flows and would have a significant effect on the design and location of inlets and the storm drain size. A closer evaluation was conducted as part of the research, grading and drainage plans obtained from the city. The age of the plans and lack of visual clarity on the files, made it difficult to read exact values for the drainage calculations, however, it was clear from the grading plan that the sites were designed with retention ponds to retain the runoff onsite. To verify this, the current LiDAR was used to establish the footprint of the existing retention ponds in the basin. Stage storage data was developed to compute the available volumes relative to the total discharge volume from HEC-HMS. The pond volumes demonstrated sufficient retention volume however, some of the inlet structures onsite may not effectively capture all the runoff. Therefore, 20% of the runoff was allowed to drain to Richards Avenue and these flows were integrated into the conveyance design for the project. The computed stage storage data and pond limits are included in **Appendix C.6**.

2.3.2 SIRINGO ROAD AND MARC BRANDT PARK

The tributary basin area that drains to Marc Brandt Part is approximately 0.46 square miles. Within in this drainage area there are two ponding areas. A detention pond located at Franklin Miles Park at the headwaters for this watershed drains into a small detention pond at the cul de sac of Escondida Ct, which drains to Siringo Rd through a rectangular concrete channel. The unnamed pond at Escondida Ct was not simulated in the watershed model due to its small footprint, however both Franklin Miles Park and Marc Brandt Park were simulated in the

HEC-HMS model to determine the total inflow and total outflow from the parks. The watershed is primarily a mix of medium density residential areas, business areas, some commercial development, and parks. Figure 4 shows the basins delineated for the Marc Brandt Park. There are no major storm drains in this portion of the watershed and surface drainage is the primary means of conveyance through existing roads.

2.3.3 SOUTH OF ARROYO DE LOS CHAMISOS/ RODEO DE SANTA FE

This segment of Richards Ave receives offsite flows from the Rodeo De Santa Fe property from several locations on the east side of Richards Ave. RDSF property is mostly undeveloped with pockets of buildings and impervious areas. The area also contains corrals and ungraded parking areas combined with vegetated open space. The runoff from the rodeo deposits sediment in the road along with the stormwater runoff. Subbasins were delineated for this location and added to HEC-HMS model. The basins drain in westerly direction with well-defined outlet points that drain onto Richards Avenue.

Figure 2
Overall Basin Map

Arroyo De Los Chamisos
Crossing Project
Santa Fe County, New Mexico

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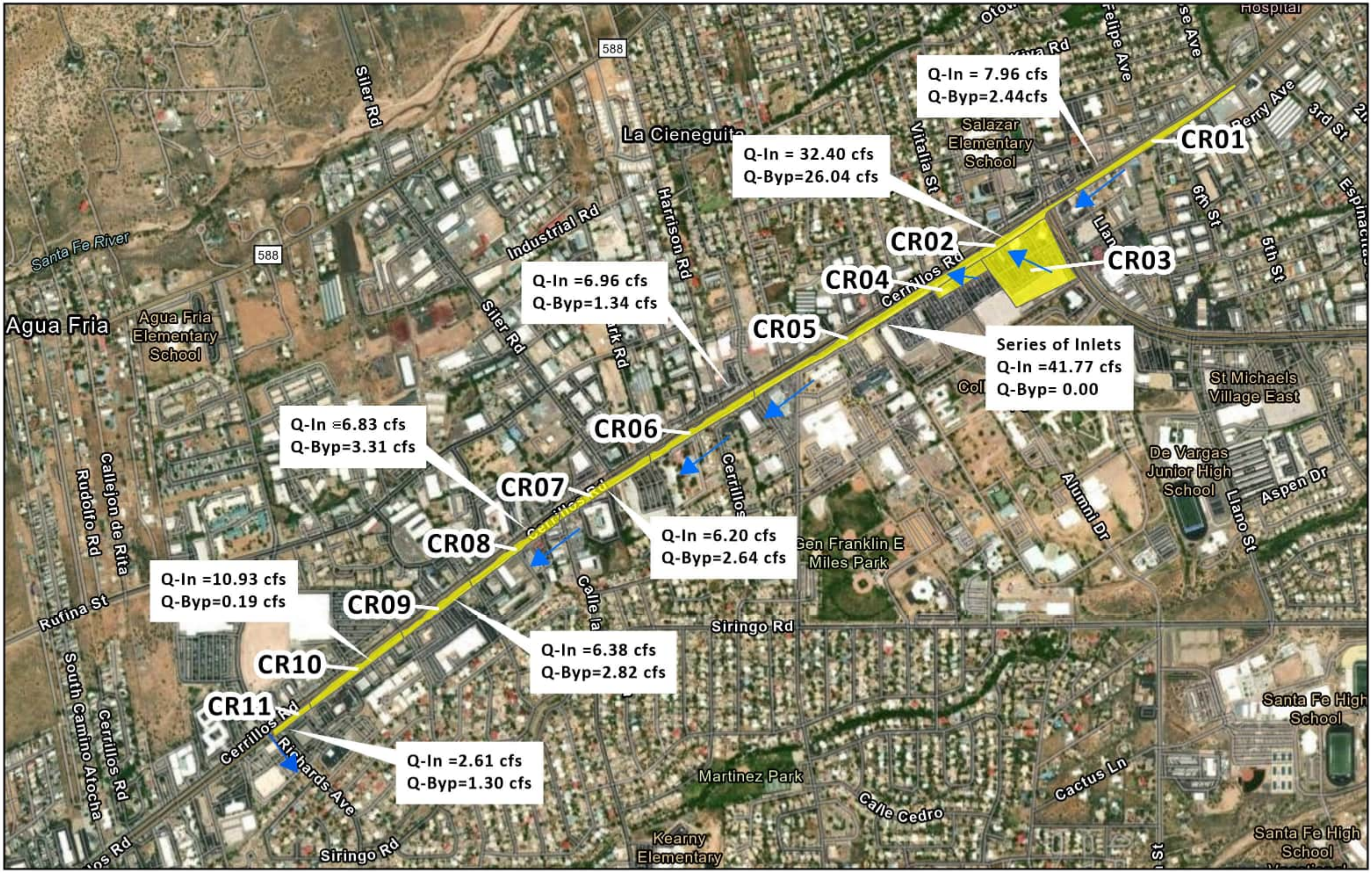

Legend

Roadway Sub-Basins

Q-In Intercepted Flow

Q-Byp Bypass Flow

0 1,000 2,000 Feet

Figure 3
Runoff on Cerrillos Road

Arroyo De Los Chamisos Crossing Project

Santa Fe County, New Mexico

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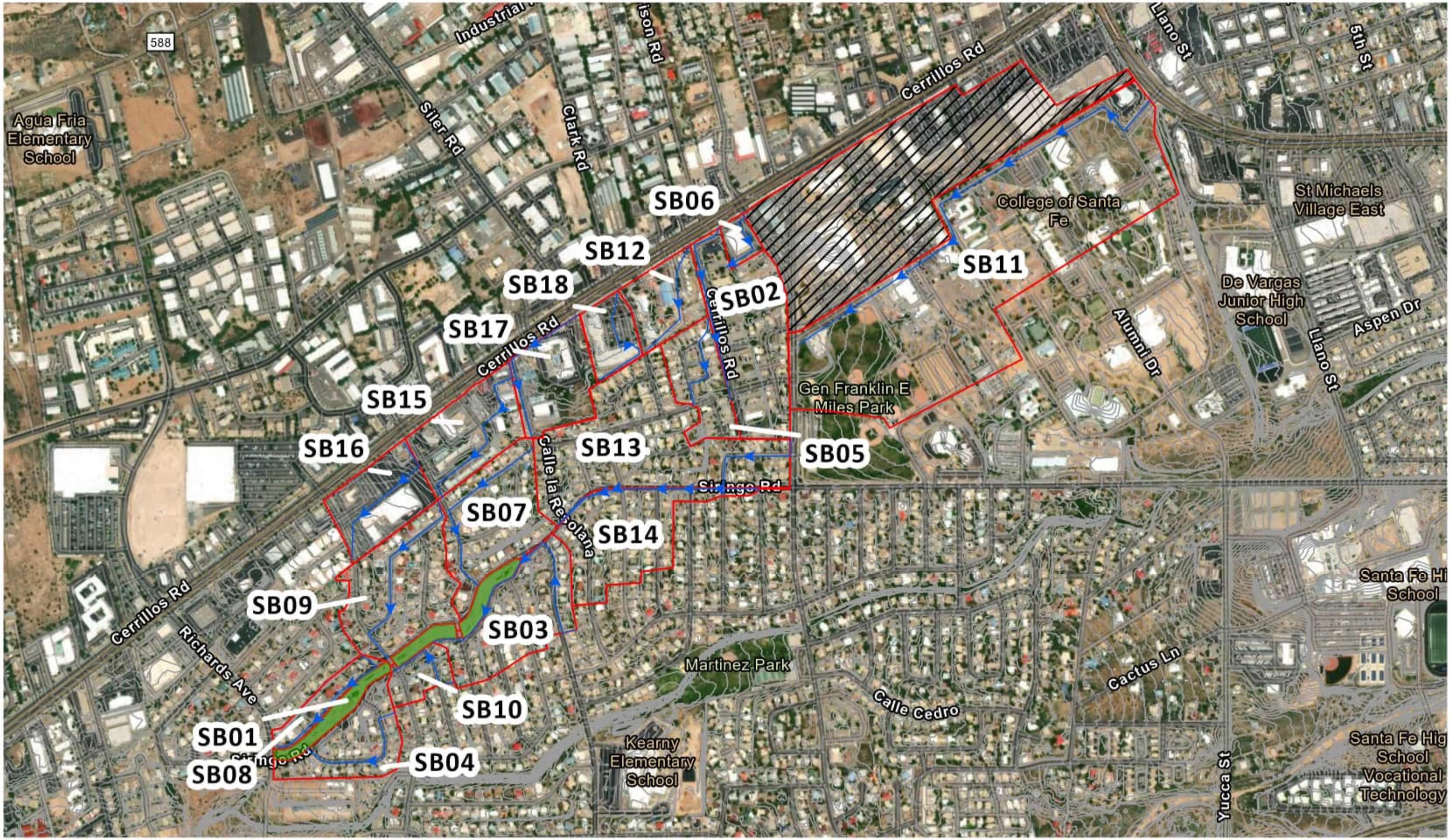

Legend

- 2ft Contour
- Non Contributing Area
- Marc Brandt Pond Basins
- Marc Brandt Park
- Longest Flowpath

0 1,000 2,000 Feet

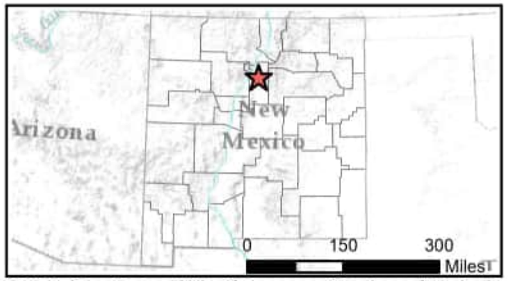

Figure 4
Marc Brandt Park Basin Map

Arroyo De Los Chamisos Crossing Project
Santa Fe County, New Mexico

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3 HYDROLOGY

3.1 DRAINAGE BASIN DELINIATION AND ANALYSIS CRITERIA FOR ON SITE AND OFFSITE BASINS

The project follows the Drainage Design Criteria for NMDOT Projects (July 2018). The 50-year recurrence interval storm is the design flood for the bridge, curb inlets, storm drain trunk lines, and the check flood is the 100-year recurrence interval storm. **Table 1** provides a list of design criteria that must be satisfied when designing the drainage infrastructure. HEC-HMS V4.11 was used for watershed modeling following the NMDOT guidelines.

Table 1: Drainage Design Criteria

	Criteria	Value	Reference
Floodplain	100-Year Flood Peak Flow	3750 cfs (Chamisos) / 1550 cfs (Pinos)	FEMA
CBC Wingwall Skew	Design Criteria	45° (maximum)	NMDOT Std Plan 511-67-1/2
Bridge Scour	Design Storm Check Storm	100-yr Flood scour depth - coordinate w/ bridge designer to set bridge piers/abutments 500-yr event - coordinate w/ bridge designer to protect bridge structure	NMDOT DDM
Existing Culverts	Design Storm Check Storm	50-yr 24-hr spread limited to edge of driving lane 100-yr 24-hr spread limited to ½ of a driving lane	NMDOT DDM Table 203-1, Table 204-1
New Culverts	Design Storm Check Storm Diameter Velocity	50-yr 24-hr spread limited to edge of driving lane 100-yr 24-hr spread limited to ½ of a driving lane 24-in minimum 4 ft/sec minimum	NMDOT DDM Table 203-1, Table 204-1
Bridge Deck Drains	Design Storm Chack Storm	50-yr Limit water spread to edge of driving lane 100-yr Limit water spread to one half of a driving lane	NMDOT DDM Table 203-1, Table 204-1

	Criteria	Value	Reference
Roadside Ditches	Design Storm Check Storm	50-yr 24-hr spread is limited to edge of shoulder 100-yr 24-hr spread limited to ½ of a driving lane	NMDOT DDM Table 203-1, Table 204-1
Curb Drop Inlets	Design Storm Check Storm Grate Clogging	50-yr 24-hr spread is limited to ½ of a driving lane for 2-lane road 100-yr 24-hr depth is limited to top of curb 25% on grade; 50% in sag	NMDOT DDM Table 203-1, Table 204-1
Storm Drain	Diameter -trunk and laterals Slope Velocity	24-in (minimum) 0.3% (minimum) 2.5 ft/s (minimum)	NMDOT DDM Table 206-1,
Green Infrastructure/ Water Quality Design	80th percentile rainfall event 90th percentile rainfall event	0.50 inches 0.68 inches	EPA MS4 Permit, section V.7.B
Retention Pond Sizing	Infiltration rate	0.5 in/hr. (minimum)	EPA (Ref. 7)

High-resolution elevation data (2-foot DEM) was obtained from the USGS LiDAR database. This data was used in ESRI ArcGIS Pro (version 3.1.2) to define basin boundaries. Two-foot contour data was then extracted from the DEM to develop the basin boundaries.

Extensive field work was done to verify the basin boundaries. Several areas shown in **Figure 4** were identified as noncontributing basins both the Cerrillos basins and Mark Brandt Park basins due to the presence of retention ponds and or walls that prevent flow from draining downstream.

3.2 RAINFALL DATA

Rainfall data was obtained from the “Precipitation-Frequency Atlas of the United States” NOAA Atlas 14, Volume1, and Version 5 using the NOAA Atlas 14 at the centroid of the basins.

Table 2 provides the 24hr rainfall depth data needed for the hydrological analysis. Precipitation frequency depth estimate data sheets for the project area are provided in **Appendix C.1**.

Table 2: NOAA Atlas 14 Data

Point Precipitation frequency estimates (in inches)						
Storm	2-year	5-year	10-year	25-year	50-year	100-year
24-hr	1.51	1.88	2.17	2.56	2.87	3.18

3.3 SUBBASIN PARAMETERS

3.3.1 SOILS DATA AND RUNOFF CURVE NUMBER (CN)

The NRCS Runoff Curve Number (CN) method was employed to calculate the initial abstraction Loss for determining excess precipitation (direct runoff).

The CN values for the subbasins are determined by a combination of Hydrologic Soil Groups (HSG), vegetation, and cover. These groups, ranging from HSG Type A to HSG Type D, categorize soils based on infiltration rates. HSG C and D, characterized by slow infiltration rates, exhibit a higher potential for runoff, while HSG A and B encourage greater infiltration. Soil characteristics for the watersheds were sourced from the Natural Resources Conservation Service (NRCS) Web Soil Surveys, cross-referenced with Santa Fe Zoning Maps and on-site observations of watershed conditions and vegetative cover.

Within the study area, hydrologic soil groups A, B, and C are present, with type C being predominant. CN values corresponding to each HSG type were obtained from Table 2-2d of the NMDOT Drainage Design Manual and were calculated based on the weighted average percentage of each HSG type within the project area.

Appendix C.2 shows the HSG distribution (A, B, C, and D) within the Arroyo de Los Chamisos Watershed. **Appendix C.3** provides a summary of Curve Numbers.

3.3.2 TIME OF CONCENTRATION

The Velocity Method was used to determine the time of concentration (Tc) for sub-basins that drain into the Mark Brandt Park. Tc represents the time it takes for water to travel from the farthest point in a sub-basin (most hydraulically remote) to the outlet or area of interest.

To identify the longest flow path within each sub-basin, contour data is analyzed. Water can travel through the sub-basins as sheet flow, shallow concentrated flow, gutter flow, or a combination of these. Sheet flow is limited to a maximum length of 300 feet unless if the upper basin slope is below 0.5%.

The Tc for each sub-basin is calculated by summing the travel times for each identified flow type. Lag time, equal to 0.6 times Tc, is factored into the calculations. Manning's Roughness Coefficients are chosen based on the NMDOT Drainage Manual Table 502-5.

The minimum allowable Tc is 12 minutes, and a minimum Tc of 12 minutes is applied to both the Richard Avenue and Cerillos Road basins. To use the SCS unit hydrograph transformation method in HEC-HMS, lag time is the required input. NRCS categorizes the relationship between Tc and the lag time (T_L) as:

$$T_L = 0.6\ Tc$$

A detailed output of Tc calculations is provided in **Appendix C.4**.

3.3.3 ROUTING REACHES

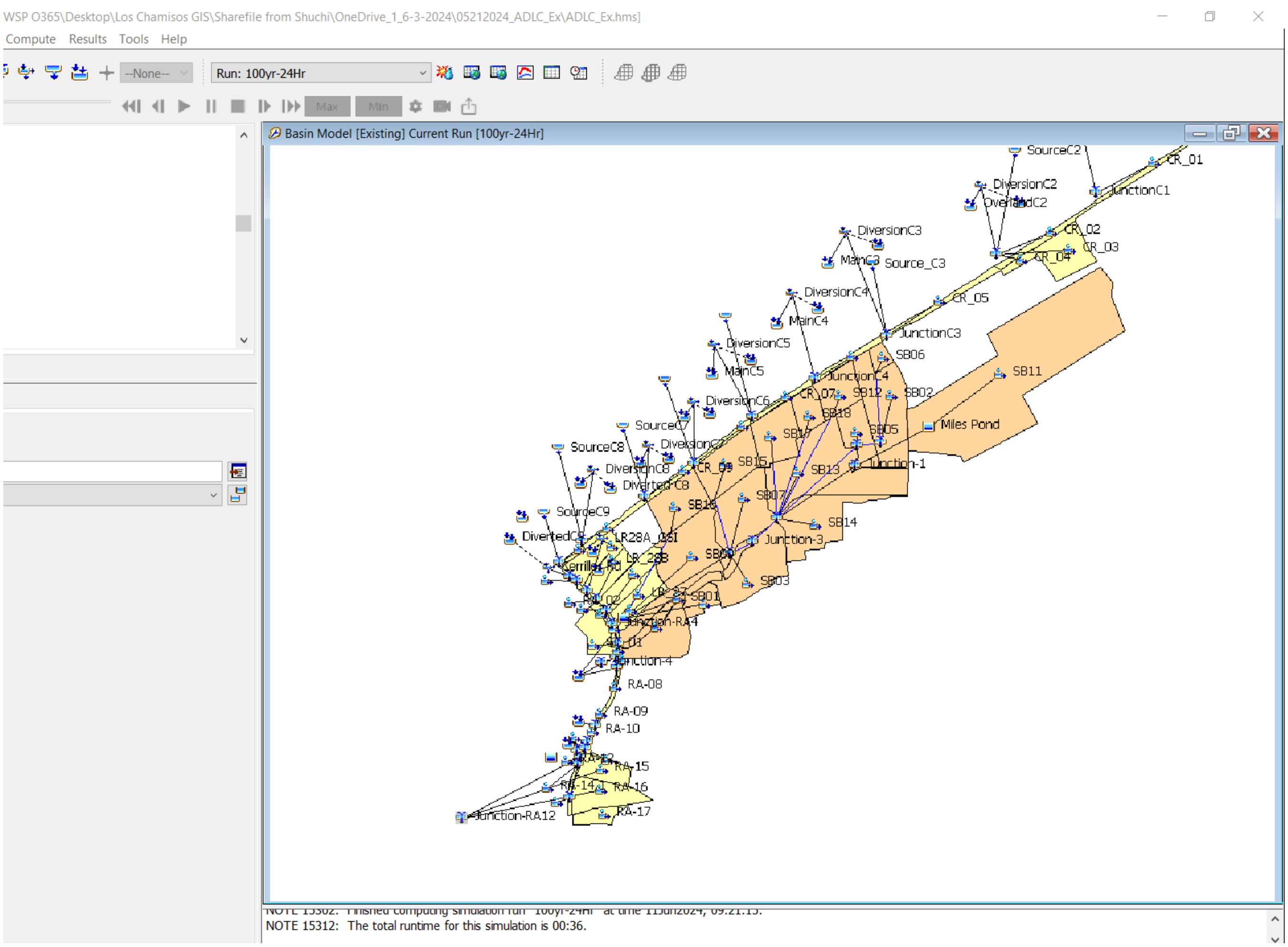
A routing reach simulates hydrographs travel through a downstream basin accounting for the effects of time and space. The Muskingum-Cunge method was used. Parameters for routing reaches were determined from LiDAR, and aerial imagery.

3.3.4 COMPUTATION TIME INCREMENT FOR HMS MODEL.

For basins with short lag times, the recommended time increment should not exceed 5 minutes otherwise the peak flows may not get captured. For this model, a time step of 5 minutes was used.

A schematic view of the HEC-HMS model is provided in **Figure 5**.

Figure 5: HEC-HMS Schematic



3.4 SUMMARY OF HYDROLOGIC RESULTS

3.4.1 RUNOFF FROM CERRILLOS ROAD

Flow along the south side of Cerrillos road travels west along the roadway and directed along Cerrillos with the use of valley gutters in the intersections until Richards Ave and the turnout just east of Richards Ave.

A total of 22 single grate combination inlets exists in this section of roadway. The capacity of each inlet was calculated in FHWA Hydraulic Toolbox determine the total flow draining to each inlet, the amount of flow captured, and the bypass flows to the downstream inlets. **Figure 3** shows the schematic layout of this analysis including the flows. At the intersection of Richards Avenue and Cerrillos Rd, a total bypass of 1.3 cfs drains into Richards Ave during the 50-yr storm. This flow was incorporated into the system design for the proposed project.

Peak discharges contributing to the Mark Brandt Park / Pond were modeled within HEC-HMS. The 2-, 5-, 10-, 25-, 50-, and 100-year storms were computed and peak inflows at the pond are provided in **Table 3**.

Table 3: HEC-HMS Output Summary Table

INFLOW PEAK DISCHARGES (CFS)						
Storm Event	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Mark Brandt Park / Pond	104	176	238	334	420	514

4 PROPOSED CONDITIONS DRAINAGE ANALYSIS

4.1 CONCEPTUAL ASSESMENT OF MARC BRANDT PARK

Based on existing Lidar data, elevation area, volume tables were developed, which indicated that cumulatively the pond has approximately 2.4-acre feet of storage volume. Elevation area data is provided in **Appendix C.5**. For a drainage of 0.47 square miles, this is inadequate for any meaningful attenuation. However, this data was incorporated into the HMS model to perform a reservoir routing analysis to determine actual function of the pond. Since the outlet structure is an unregulated 48-inch CMP, the built-in outlet tools in HMS were utilized which uses the culvert as the principal outfall structure. The intersection of Siringo Rd and Richards Ave was set as the emergency spillway. This is where overtopping would occur if the capacity of the pond was exceeded. The elevation for the road was based on elevations from LiDAR. The pond routing routine in HMS utilizes the elevation area data in conjunction with culvert parameters to compute internal storage volume and discharge rating curve for the ponds function. HMS uses the same culvert nomographs as HY-8.

A reservoir routing summary is provided in **Table 4**. The model also assumes clean conditions for the outlet culvert and so no clogging is applied. The results show that at the 50-year storm there is overtopping.

Table 4: Marc Brandt Park Routing (50-yr)

Pond Name	Drainage Area (mi^2)	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Inflow Volume (acre-ft)	Peak Discharge Volume (acre-ft)	Design Storage Volume (acre-ft)	50-yr Peak Volume (acre-ft)	Max Water Surface Elevation (ft)	Pond Invert (ft)	Top of Pond Elevation (ft)	Freeboard (ft)
Marc Brandt	0.47	420	418	31.4	31.4	2.4	1.3	6631.97	6624	6631.5	-0.47

The pond will overtop in the 50-year event as the storage volume is severely inadequate to provide any meaningful attenuation. Any proposed pond would have to provide in the region of 22–25-acre feet of storage volume to provide meaningful attenuation in the system.

The results were presented to the city, and it was noted that to provide the necessary volume in the current park footprint, severe impacts to the footprint and aesthetic value of the park would occur. The side slopes would have to be steepened and this would cause the loss of most of the existing trees and grass in the park. The city recommended that this was a sufficient high level of analysis to allow them to plan for future modifications and funding acquisition. The final layout of proposed improvements would be determined at that time.

As part of the roadway project, the sidewalk at the terminus of the park will be relocated so that it transitions out of the park near the access steps in the south end. For 30% Plans, this will be demonstrated as a concept sketch as the area will require additional survey.

4.2 ANALYSIS FOR ROADWAY DRAINAGE

The proposed roadway improvements will need to accommodate minor offsite runoff from Cerrillos Rd, and offsite flows from the eastern residential areas and some developed areas at the intersection of Cerrillos and Richards Ave. Additionally, localized roadway flows will need to be captured at specified intervals to meet the spread criteria. Roadway basins were delineated as well to capture the vertical design profile of the road, proposed bridges, and the roundabout at the intersection of Camino Del Prado of Bentley Flowmaster was used to analyze the capacity of the inlets. It utilizes the procedures outlined in HEC-22 to compute capture and bypass flows. Proposed storm drain alignments were developed. Vertical data for the existing water, sewer and dry utilities will be obtained at the next phase of design and conflicts will likely require changes to the storm drain design. For this submittal, utilities were considered in the horizontal direction only with the assumption that water and dry utilities will be relocated.

For inlets at grade, the computations assume a 25% clogging factor. For inlets at a sag, a 50% clogging factor was applied. All inlets designed at sag locations will have flanking inlets to provide redundancy incase the primary inlets get 100% clogged.

For each inlet, the captured flow was added into the storm drain network, and bypass flows were carried downstream. For Marc Brandt Park, the routed outflow from the HEC HMS model was added as a point discharge to the hydraulic network. This is a conservative assumption as the flows from the pond will have some upstream ponding which will regulate the rate at which flow enters the system. The culvert at Mark Brand Park is inlet controlled so the inlet conditions will govern inflow into the storm drainage system. Junction head Losses were based on the number of laterals connecting to each manhole. The design intent is to utilize wye fittings for the smaller diameter pipes to save costs on manholes. Head losses for fittings and manholes are computed using different computation methods, Conservation of Momentum for bends and fittings, and Conservation of Energy for manholes, therefore, to be consistent, an absolute head loss table from Bentley was used to account for head losses throughout the system. The table is included in the appendix.

The analysis is summarized on a schematic plot, Figure 5, that shows the roadway basins, storm drain network, location of inlets, captured flows and bypass to the next basin. The proposed storm drain diameters will range from 24-inch to 66-inch diameter pipes for the section of storm drain from Mark Brandt Park to the outfall. The overall system satisfies the HGL criteria for the design storm. Schematic profiles of the trunklines are also provided in **Appendix D.2**.

Summary of Major Design Components

- Collector laterals will be required to capture flows from the offsite basins at Lorraine Ct and at Siringo Rd to meet spread criteria.
- The existing 48-inch pipe that is in the residential side yard will be abandoned in place.
- The 48-inch pipe will be replaced with a 66-inch pipe from the park, with proper headwalls. The outfall will connect with the main northern trunkline and discharge to the existing discharge point. The outfall will have a different alignment to make the bends in the system hydraulically efficient.
- The outlet velocity from the northern trunkline will be in approximately 23 feet per second and will require an engineered energy dissipation structure.
- A new outfall will be required at Station 32+50 on the north approach to the proposed bridge.
- Runoff on the south side of the roundabout will be captured and diverted to the existing pond. The pond can also be modified to have an outfall as well if the city desires it. It will serve as an infiltration area.
- There are two major offsite basins, RA-15 and 16, that will drain to the back of the proposed wall on the east side of Richards Ave south of the roundabout. These flows will be conveyed to the arroyo, where it drains to currently. The total flow is approximately 26 cfs that will need to be conveyed to the southern trunkline.

- A rectangular concrete channel that is 4' wide by 14 inches tall will be sufficient to convey a maximum of 35 cfs with a normal depth of 13.1 inches. The 100 year-24 hr. storm's peak flow is approximately 32 cfs. The anticipated channel velocity is around 8 feet per second which should keep sediment suspended. This will minimize maintenance issues. The channel will drain to a pair of median drop inlets with a rural type of grate that has sufficiently large openings to mitigate debris accumulation. The inlets will be designed to operate in a sump condition with 2 feet of head.

4.3 OUTFALL DESIGN FOR PROPOSED STRUCTURES

4.3.1 NORTH RICHARD TRUNKLINE OUTFALL

The proposed improvements will require modifications to an existing outlet. For the northern trunkline a high system velocity requires an engineered dissipation structure at the outlet. The vertical profile of the storm drain will very likely change once the utility conflicts are identified; therefore, the sizing calculations were performed at a high level. The HEC-14 Manual for Hydraulic Design for Energy Dissipators for Culverts and Channels was used a guide for energy dissipater design. Based on the Froude Number of 1.8, and an outlet velocity of 23 feet per second, two scour applications were conceptually evaluated.

Option 1: Scour Hole Geometry

This method requires gradation data for D84 and D16 which was not available at the time of this report and therefore the calculations were not performed. However, the premise of this option is that the scour geometry is computed based on equations in Section 5 of HEC-14. The scour hole is then filled in with appropriately sized rip rap. This can be customized to incorporate vegetation at the end of the dissipation basin as flow transitions to the natural channel with velocities more akin to natural channels. If this method makes it through the screening and approval process with the City, gradation data will be obtained, and a more detailed computation performed.

Option 2 : Rip Rap Basin

This method is the use of a rip rap stilling basin using a low tailwater application. The design intent is to have a flared end section installed at the outlet of the 66-inch pipe which will reduce the brink depth. The reduction in brink depth in conjunction with a low tailwater basin will allow an efficient transition to natural velocities. **Figure 6** and **Figure 7** shows a schematic view of the basin. Calculations were done to determine the dimensions of the basin as shown in **Table 5**. The primary parameter is the dissipator pool depth, h_s, which is based on Equation 10.1 in HEC-14 and shown below.

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{g y_e}}\right) - C_o$$

where,

h_s = dissipator pool depth, m (ft)

y_e = equivalent brink (outlet) depth, m (ft)

D_{50} = median rock size by weight, m (ft)

C_o = tailwater parameter

The critical dimensions shown in the **Figure 6** are a function of h_s as shown in **Table 5**: Riprap Basin Dimensions. The designer has the choice of picking the computed or the minimum dimensions that will satisfy the dissipation requirements. If this method is screened and approved, minimum dimensions along with vegetative applications will be used.

Supporting calculations are included in Appendix .

Figure 6: Profile of Riprap Basin

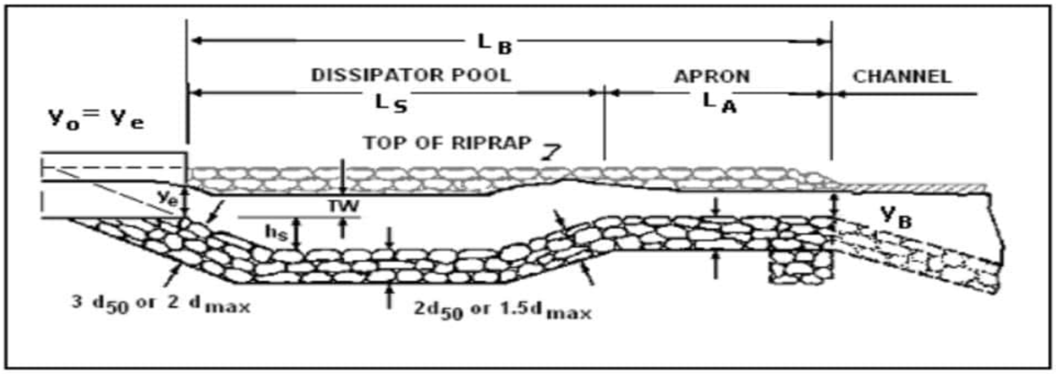
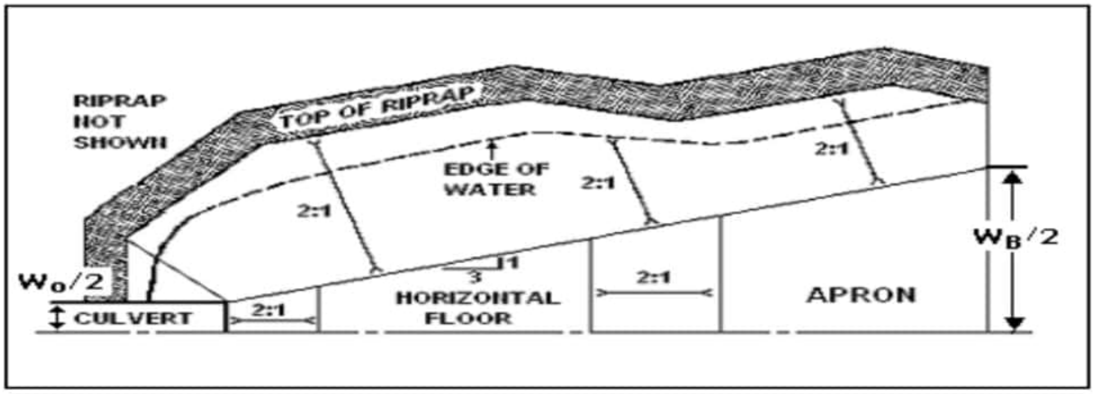


Figure 7: Half Plan of Riprap Basin



Based on the outlet velocity, the required D50 would be 9 inches and the maximum and minimum acceptable dimensions are:

Table 5: Riprap Basin Dimensions

Basin Dimensional Parameters	Max Dimensions (ft)	Max Dimensions (ft)
Ls = 10*hs	24	24
Lb = 15*hs	36	36
La = Lb-Ls	12	12
Wb/2 = (Lb*3)/2	54	54

4.3.2 OTHER OUTFALL LOCATIONS

A new outfall will be required at approximately Station 32+50. A Zuni bowl has been proposed since the flows and velocities are not egregious.

On the Fire station Rd, at station 11+00, curb cuts are recommended to divert minor flows off the road and into infiltration areas.

The existing pond at the southwest corner of the roundabout will be utilized to capture roadway runoff from the south side of the roundabout to the high point in the road at approximately Station 19+75. The pond will act as a rain garden to promote infiltration.

4.4 GREEN STORMWATER INFRASTRUCTURE APPLICATIONS

The City of Santa Fe places a huge emphasis on implementing green stormwater infrastructure where possible. The ROW throughout the project reach is limited, however several locations that could be utilized for GSI applications are identified on the included roll plot at the end of the report. Based on "A Guide to Incorporating Green Infrastructure into Roadway Projects in Santa Fe", GSI applications should target runoff from the first 0.6 inches of runoff. The following locations and applications are recommended for the proposed Richards Corridor. These options are located at strategic locations and will greatly help with water quality and infiltration in the more frequent small storms.

Table 6: Proposed GSI Locations

Approx Start Station	Approx End Station	Application
52+00	54+00	Tree Trench
46+50	46+60	Tree Trench/Bio Retention Pond
42+50	-	Supplementary Vegetative Erosion Control at Outlet of 66-inch Pipe
32+75	-	Zuni Bowl
Fire Station Rd Sta 11+00	-	Bio Swale
28+00	29+00	Infiltration Pond
South Trunkline Outlet	-	Bio Swales

5 SRH-2D MODELING

5.1 METHODOLOGY

5.1.1 BRIDGE HYDRAULIC MODELS

In addition to criteria listed in **Table 1**, general industry guidelines for the proposed bridge improvements include: (1) no adverse impacts on adjacent properties, (2) no significant changes to channel velocity, aggradation or degradation, scour, head cutting, and conveyance, and (3) compliance with USACE, the NM Environment Department, FEMA, and other agencies' requirements.

The Bureau of Reclamation's Sediment and River Hydraulics, Two-Dimensional Model (SRH-2D) version 13.3.6, was used for the hydraulic analysis of the existing and proposed bridges. SRH-2D solves the two-dimensional depth averaged dynamic wave equations using the finite volume numerical method. Solved variables include the water surface elevation, water depth, and depth averaged velocity. Additional output variables include the Froude Number, bed shear stress, and critical sediment diameter.

A steady flow analysis was performed in SRH-2D to develop water surface profiles for the 50-, 100-, and 500-year recurrence interval storm events. The basis for the hydraulic modeling is a ground and bathymetry survey that was performed by Bohannan Huston, Inc in March 2021. Limits for this survey extended approximately 1000-ft upstream and 500-ft downstream of the proposed Arroyo de Los Chamisos bridge location, and 700-ft upstream and 300-ft downstream of the Arroyo de Los Pinos crossing. For areas where elevation data is needed and was not included in the survey, USGS Lidar Point Cloud data was used.

A scatter data set was imported from the survey and USGS data and a mesh was created to model the existing crossing on the Arroyo de Los Pinos and the Arroyo de Los Chamisos main channel. The scatter data was adjusted at the pier locations and a mesh was created from proposed surfaces to model the proposed bridge configurations. Manning's "n" values, monitoring points and lines, and boundary conditions were associated with each mesh. The boundary conditions consisted of upstream inlet boundaries (Inlet-Q) with a steady state peak flow discharge and subcritical flow regime and a downstream exit boundary (Exit- H) with a constant water surface elevation and subcritical flow regime at the downstream end of each Arroyo.

Manning's "n" values for the channel and floodplains were established based on *"Open-Channel Hydraulics*" by Ven Te Chow, visual observation, and photographs. Values were added to the model using a material properties shapefile. The assumed natural channel and floodplain "n" values for the model ranged from 0.035 to 0.075 and an "n" value of 0.016 was used for the roadway pavement. Refer to **Appendix E** for the SRH-2D hydraulic analysis.

Flow data for the channels in the Arroyo de Los Chamisos and Arroyo de Los Pinos SRH-2D models uses the FEMA flow rates from the Santa Fe County, New Mexico Flood Insurance Study Volume 1 of 3. An Excerpt from the Flood Insurance Study can be seen in **Appendix B**.

5.2 FEMA ANALYSIS

The bridges are located within an AE flood zone with accompanying Zone X floodplain delineated for the 500-year storm as shown on the Flood Insurance Rate Maps (FIRM) 35049C0394D and 35049C0413E. Development within a Zone AE Floodplain requires that any development will not increase the water surface elevation of the base flood more that 1-ft at any point throughout the community as described in CFR-2011-Title 44-vol 1

Section 60.3. The basis for the analysis came from the latest Flood Insurance Study for Santa Fe County completed in 2008 and revised in 2012, which used a HEC-RAS one-dimensional model to determine base flood elevations. There are no cross sections located within the project area, so base flood elevations were used for comparison. For the purposes of this analysis the HEC-RAS model will serve as original effective model. To complete the analysis, a Corrected Effective Model was developed using Aquaveo SMS version 13.3. The corrected effective model uses flows from the FIS for the Arroyo de Los Chamisos and the Arroyo de Los Pinos. Flows for the outfall from Marc Brandt Park and Richards Ave that drain into the Arroyo de Los Chamisos were taken from the hydrology section of this report. The corrected effective model was updated with the proposed bridge configuration and all proposed roadway grading to create a Proposed Conditions Model. Water surface elevations used for the FEMA analysis are pulled from the SRH-2D model via monitor lines.

5.2.1 ARROYO DE LOS CHAMISOS BRIDGE HYDRAULIC MODEL RESULTS

The hydraulic analysis of the existing and proposed conditions was analyzed to ensure that all FEMA and City of Santa Fe requirements are met. Within the Zone AE floodplain any flooding from the development is not to exceed 1-ft anywhere in the community. The largest local water surface rise was near the bridge and is 0.31-ft. The existing and proposed both converge upstream and downstream of the bridge within the model limits. **Table 7** shows the comparison between the HEC-RAS Effective model, the SRH-2D Corrected Effective Model, and the SRH-2D Proposed Conditions Model.

Table 7: Arroyo De Los Chamisos FEMA Analysis

Location Relative to Bridge	HEC-RAS Effective Model WSE (ft)	SRH-2D Corrected Effective Model WSE (ft)	Difference (Corrected - Effective)	SRH-2D Proposed Conditions Model WSE (ft)	Rise (ft)
475’ Downstream	6606	6,606.20	0.20	6606.20	0.00
100’ Downstream	6611	6,611.13	0.13	6611.17	0.04
160’ Upstream	6616	6,616.00	0.00	6616.31	0.31
410’ Upstream	6621	6,621.15	0.15	6621.15	0.00
750’ Upstream	6626	6,626.19	0.19	6626.19	0.00
1075’ Upstream	6631	6,630.96	-0.04	6630.95	-0.01

The comparison of the existing and proposed SRH-2D models revealed that the proposed 220’ two span bridge with a centrally located pier creates minor a local rise in water surface elevation. This is attributed to the reduction of the flow area within the channel caused by locations of the abutments and roadway fill. Refer to **Table 8** to for the water surface elevations at the upstream face of the bridge. Refer to **Appendix E.3** for water surface elevation cross sections. In addition, the main channel velocities for the existing and proposed conditions were also evaluated. The proposed velocities are slightly higher due to the constrained flow area in the proposed condition. The results of the SRH-2D hydraulic model show the proposed bridge meets the NMDOT’s drainage design criteria for freeboard of 2-ft for the 50-yr storm and below the low chord for the 100-yr storm.

Table 8: Arroyo De Los Chamisos SRH-2D Results

Bridge	Storm Event	WSE at Upstream Face of Proposed Bridge (ft)	Proposed Low-Chord Elevation (ft.)	Available Free Board (ft.)	Channel Velocity (ft/s)
Existing Conditions (no bridge)	50-Year	6616.40	-	-	10.68
	100-Year	6616.66	-	-	11.81
	500-Year	6618.16	-	-	15.58
Proposed Bridge	50-Year	6615.94	6618.26	2.32	10.92
	100-Year	6615.84	6618.26	2.42	12.53
	500-Year	6617.06	6618.26	1.20	15.84

5.2.2 ARROYO DE LOS PINOS BRIDGE HYDRAULIC MODEL RESULTS

The existing crossing across Arroyo de Los Pinos is a 128" span x 83" rise corrugated metal pipe arch (CMPA) with vertical headwalls. The existing culvert is undersized and there is significant overtopping of the roadway. The downstream edge of the culvert has about 3-ft of head cut. A grade control structure will be designed to stop the head cut from migrating upstream and is reflected in the proposed grading. The proposed bridge is a 45-ft clear span bridge with vertical abutments. The existing condition of the crossing allows for weir flow over the road, so the proposed structure needs the able to pass both the flow through the existing culvert and all the flow that is flowing over the roadway. The proposed clear span bridge has been chosen because it has the capacity to pass the flow without adversely affecting the nearby properties with increased flood inundation limits in each design storm. Refer to **Table 9** below for the FEMA model comparison.

Table 9 Arroyo de Los Pinos FEMA Analysis

Location Relative to Bridge	HEC-RAS Effective Model WSE (ft)	SRH-2D Corrected Effective Model WSE (ft)	Difference (Corrected - Effective)	SRH-2D Proposed Conditions Model WSE (ft)	Rise (ft)
550' Downstream	6615	6615.51	0.51	6615.50	0.00
200' Downstream	6620	6619.98	-0.02	6619.79	-0.19
At Bridge	*6626	*6626.33	0.33	6621.45	-4.87
460' Upstream	6631	6630.96	-0.04	6630.96	0.00
*Flow is overtopping the roadway					

The comparison of the existing and proposed SRH-2D models revealed that the proposed bridge design creates a local drop in water surface elevation. This drop is due to the overtopping of the roadway in the existing conditions. The proposed clear span bridge is wide enough to pass all flow while maintaining the required NMDOT freeboard requirements. Refer to **Table 10** to for the water surface elevations at the upstream face of the bridge. In addition, the main channel velocities for the existing and proposed conditions were also evaluated and the proposed velocities are much higher. This is because the existing condition was in an overtopping condition and in proposed all the flow stays within the channel. Velocities are increased 300-ft upstream and downstream of the proposed bridge.

Table 10 Arroyo de Los Pinos SRH-2D Results

Bridge	Storm Event	WSE at Upstream Face of Proposed Bridge (ft)	Low-Chord Elevation (ft.)	Available Free Board (ft.)	Channel Velocity (ft/s)
Existing Conditions (no bridge)	50-Year	6625.72	-	-	5.01
	100-Year	6626.37	-	-	6.16
	500-Year	6628.09	-	-	7.51
Proposed Bridge	50-Year	6620.55	6628.00	7.45	11.08
	100-Year	6621.60	6628.00	6.40	12.08
	500-Year	6625.07	6628.00	2.93	12.88

5.3 SCOUR ANALYSIS ARROYO DE LOS CHAMISOS

Upon satisfactory completion of the hydraulic analysis, a scour analysis was performed to estimate the scour potential at the proposed bridge. As per the Department's drainage design criteria, the 100-year and 500-year storm events were used for the scour analysis. The following FHWA publications were consulted for bridge scour analyses and countermeasure recommendations:

- Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18), April 2013
- Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20 (HEC-20), April 2012
- Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance, Hydraulic Engineering Circular No. 23 (HEC-23), September 2009

As recommended by HEC-18, channel horizontal stability, long term stream degradation, contraction scour, and local (abutment and pier) scour were analyzed for the bridge. The following sections provide the basis for the data used in the scour analysis and the corresponding scour results.

5.3.1 SITE GEOLOGY

Soil samples were taken at multiple locations along the channel and the material was classified as loamy sand (SM), coarse sand (SM) and gravelly coarse sand (SW-SM) according to the USCS classification. Sample location BR03 (35-ft depth) from the geotechnical report prepared by YeDoma Consultants, LLC was used to classify the particle sizes for the scour at Arroyo de Los Chamisos. A sample depth of 35-ft was chosen to match the calculated scour depth. Refer to **Appendix E.6** for the sieve analysis results and gradation curves.

5.3.2 CHANNEL HORIZONTAL AND VERTICAL STABILITY

Like most sandy channels, the ADLC appears to be dynamic. Based on Google Earth imagery, prior to 2015, there were several dunes and islands just west of the proposed road crossing while the channel displayed braiding. In 2017, major channel grading appears to have occurred. The channel, along with the natural braiding was graded and the toe armoring along the north bank, along the residential was installed.

At some point, bend way weirs were also installed along the south bank to retrain the thalweg from encroaching further south. These bend way weirs were observed in during the field work, and there has been significant sediment accumulation and revegetation that has occurred along the south bank indicating that the bend way weirs have been successfully stabilizing the south bank. From site visits, the channel bottom appears to be stable with no large indications of head cutting through the design reach.

Once the bridge selections are approved, and grading refined around the design of the bridge abutments and piers, a more quantitative analysis will be provided as part of the final drainage report as far as vertical channel stability is concerned.

5.3.3 CONTRACTION AND ABUTMENT SCOUR

A bridge scour coverage was created in SRH-2D by defining bridge scour arcs at specific locations along the channel and at the bridge. The approach section arc was created 340-ft upstream of the bridge and the contracted section arc was created at the bridge. Using the scour arcs, SRH-2D developed the input data required in the scour equations. The scour equations were solved using the FHWA Hydraulic Toolbox version 5.1. Refer to **Appendix E.4** for the bridge scour input values, and the Hydraulic Toolbox scour analysis.

5.3.4 PIER SCOUR

The proposed bridge is a 2-span structure with vertical abutments and a 4-ft wide round nose wall pier. To compute pier, scour a rectangular hole 4-ft wide was made in the mesh within SMS to simulate the pier. **Table 11** summarizes the results of the scour analysis for the 100- and 500-year storms. Hydraulic Toolbox scour printouts are shown in **Appendix E.4**.

Table 11: Arroyo De Los Chamisos Scour Summary

FLOOD FREQUENCY (YEAR)	Q (CFS)	LONG TERM CHANNEL DEGRADATION	CONTRACTION SCOUR (FT)	MAX PIER SCOUR (FT)	TOTAL PIER SCOUR (PIER + CONTRACTION) (FT)	ABUTMENT SCOUR (FT)	
						NORTH	SOUTH
100	3750	0	0.23	31.07	31.30	0.34	0
500	10600	0	0.15	37.62	37.77	1.11	0

5.3.5 SCOUR COUNTERMEASURE RECOMMENDATIONS

Scour countermeasure will be analyzed for the Final Drainage Report.

5.4 SCOUR ANALYSIS ARROYO DE LOS PINOS

5.4.1 SITE GEOLOGY

Soil samples were taken at multiple locations along the channel and the material was classified as well-graded sand with clay and gravel (or silty clay and gravel), SW-SC, according to the USCS classification. Sample location GR-09 from the geotechnical report prepared by YeDoma Consultants, LLC was used to classify the particle sizes for the scour at Arroyo de Los Pinos. Refer to **Appendix E.6** for soil gradation curves.

5.4.2 CHANNEL HORIZONTAL AND VERTICAL STABILITY

Historical imagery was reviewed to observe the channel migration for the ADLC and the ADLP. Based on field observation, the ADLP appears to be degradational as evidenced by a 3-foot head cut at the existing arch culvert structure. A grade control structure is recommended to prevent this head cut from migrating upstream.

5.4.3 CONTRACTION AND ABUTMENT SCOUR

A bridge scour coverage was created in SRH-2D by defining bridge scour arcs at specific locations along the channel and at the bridge. The approach section arc was created 325-ft upstream of the bridge and the contracted section arc was created at the bridge. Using the scour arcs, SRH-2D developed the input data required in the scour equations. The scour equations were solved using the FHWA Hydraulic Toolbox version 5.1. Refer to **Appendix E.5** for the bridge scour input values, and the Hydraulic Toolbox scour analysis. **Table 12** shows a summary of scour values for the Arroyo de Los Pinos bridge.

Table 12: Arroyo de Los Pinos Scour Summary

FLOOD FREQUENCY (YEAR)	Q (CFS)	CHANNEL DEGRADATION	CONTRACTION SCOUR (FT)	MAX PIER SCOUR (FT)	TOTAL PIER SCOUR (PIER + CONTRACTION) (FT)	ABUTMENT SCOUR (FT)	
						NORTH	SOUTH
100	980	0	1.57	-	1.57	1.66	1.22
500	1550	0	3.50	-	3.50	3.61	3.15

NOTE THAT THIS IS A CLEAR SPAN BRIDGE THEREFORE PIER SCOUR IS NOT APPLICAPLE .

6 CONCLUSION

This drainage report draws the following conclusions:

There are minor offsite flows from Cerrillos Rd that will affect the design of the infrastructure in Richards Ave.

The offsite analysis of Marc Brandt Park Pond indicates that the pond has insufficient volume to handle the 50-year storm and there would be overtopping at Richards Ave due to the lack of capacity in the existing outlet pipe. Future considerations on optimizing the pond volume should be made. These considerations should include a means to preserve the existing ambience of the park's trees and vegetation.

A storm drain system will be required in Richards Ave to safely convey onsite and offsite drainage while meeting spread and flow depth criteria. The system will include pipes ranging from 24-inch diameter to 66-inch diameter in size.

Offsite basins form the Rodeo de Santa Fe will have to be conveyed in a rectangular channel and added into the storm drain system at the southeast corner of proposed roundabout at Richards Ave and the Fire Station Rd.

Various green infrastructure applications are proposed. These will be discussed further with the City to determine if they are desirable before finalizing the design approach.

The SRH-2D modeling analyzed floodplain limits in existing and proposed conditions, and bridge scour in the proposed conditions. The analysis of the floodplain found that we are within the limits provided by CFR-2011-Title 44-vol 1 Section 60.3, and that a CLOMR/LOMR may be required as the floodplain is changing.

The bridge scour analysis found a pier scour depth of 37-ft in the 500-yr storm for the proposed Arroyo de Los Chamisos bridge. As design moves forward this information will guide decision making for armoring and positioning of the pier to reduce the scour.

The recommendations made in this drainage report satisfy the requirements set forth in the NMDOT Drainage Design Manual for the drainage analysis and design for infrastructure for a two-lane road, new storm drainage infrastructure, and new bridges.

ARROYO DE LOS CHAMISOS DESIGN CRITERIA

CLASSIFICATION: LOW-SPEED URBAN
POSTED SPEED: 25 MPH
DESIGN SPEED: 30 MPH
MINIMUM RADIUS: 333' (2% NORMAL CROWN, NO SUPERELEVATION)

LEGEND

- *PROPOSED CURB AND GUTTER*
- *PROPOSED CONCRETE SIDEWALK*
- *PROPOSED MULTI-USE TRAIL*
- EXISTING R.O.W.
- *PROPOSED R.O.W.*
- *PROPOSED C.M.E.*

CITY OF SANTA FE

ARROYO DE LOS CHAMISOS

ROADWAY LAYOUT - OVERALL CORRIDOR

DATE: 07/25/2023

DESIGNED BY: WSP USA

DRAWING SCALE: 1" = 50'

APPENDIX

A ANNOTATED PHOTOS

Arroyo de Los Chamisos Annotated Photographs

Photo 1: Looking southwest at the 48-inch CMP outlet pipe at Mark Brandt Park

Photo 2: Steps providing entry to the pond bottom at Mark Brandt Park

Photo 3: Looking southwest at Marc Brandt Park. Note the sidewalk at the pond bottom.

Photo 4: Existing 128" x 83" Arch Pipe at Arroyo de Los Pinos

Photo 5: Approximately 3 ft difference between pipe invert and channel bottom

Photo 6: Looking upstream at Arroyo de Los Pinos

Photo 7: Existing Vegetation at Arroyo de Los Chamisos Low Water Crossing

Photo 8: Boulder Retaining Wall on the north bank of Arroyo de Los Chamisos for embankment armoring

APPENDIX

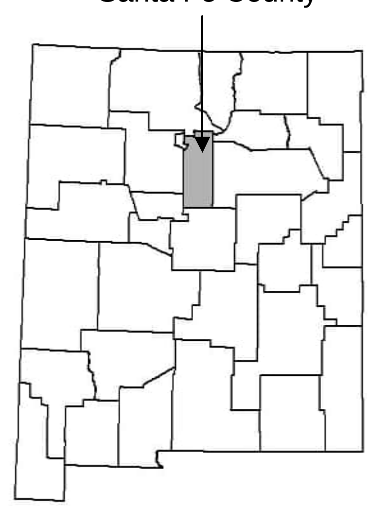
B FEMA FIS PANELS AND MAP DATA

FLOOD INSURANCE STUDY

VOLUME 1 OF 3

SANTA FE COUNTY, NEW MEXICO

AND INCORPORATED AREAS



COMMUNITY NAME	COMMUNITY NUMBER
COCHITI, PUEBLO OF	350152
EDGEWOOD, TOWN OF	350018
ESPANOLA, CITY OF	350052
NAMBE, PUEBLO OF	350124
POJOAQUE, PUEBLO OF	350153
SAN ILDEFONSO, PUEBLO OF	350092
SANTA CLARA, PUEBLO OF	350151
SANTA FE, CITY OF	350070
SANTE FE COUNTY UNINCORPORATED AREAS	350069
SANTO DOMINGO, PUEBLO OF	350081
TESUQUE, PUEBLO OF	350137

REVISED:
DECEMBER 4, 2012

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
35049CV001B

have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term, average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood that equals or exceeds the 1-percent-annual-chance in any 50-year period is approximately 40 percent (4 in 10); for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for each flooding source studied by detailed or enhanced approximate Type 1 methods affecting the community.

Peak discharge-drainage area relationships for Santa Fe County are shown in Table 7, "Summary of Discharges."

Table 7 – Summary of Discharges

		PEAK DISCHARGES Cubic feet per second (cfs)			
Flooding Source and Location	DRAINAGE AREA (mi^2)	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
ADMIN ARROYO					
Confluence with Rio Tesuque	2.3	*	*	1,730	*
ARROYO BARRANCA					
Confluence with Arroyo Mascaras	1.1	*	*	560	*
ARROYO DE LA PAZ					
Confluence with Arroyo de Los Antores	0.6	*	*	540	*
ARROYO DE LA PIEDRA					
Confluence of East Arroyo de La Piedra	0.3	*	*	230	*
Confluence with Arroyo Mascaras	1.9	*	*	1,876	*
ARROYO DE LOS AMIGOS					
Confluence with Arroyo de Los Chamisos	0.5	*	*	600	*
ARROYO DE LOS ANTORES					
Confluence of Arroyo de La Paz	0.3	*	*	380	*
Confluence with Arroyo de Los Chamisos	1.0	*	*	940	*
ARROYO DE LOS CHAMISOS					
Cross Section BT	1.5	185	670	1,080	3,100
Cross Section BL	1.6	265	980	1,580	4,500

*Data not computed

Table 7 – Summary of Discharges (continued)

Flooding Source and Location	DRAINAGE AREA (in mi²)	PEAK DISCHARGES Cubic feet per second (cfs) 10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
ARROYO DE LOS CHAMISOS (continued)					
Cross Section AU	3.6	315	1,175	1,900	5,400
Cross Section AQ	5.4	410	1,510	2,400	6,900
Cross Section AF	7.2	480	1,800	2,850	8,200
Cross Section P	10.1	600	2,250	3,600	10,250
Cross Section F	11.0	640	2,375	3,750	10,600
Cross Section A	14.4	760	2,800	4,400	12,500
From Rodeo Road to State Route 85	19.4	2,060	4,590	6,050	10,510
From State Route 85 to confluence of Arroyo Hondo	26.0	2,470	5,410	7,080	12,060
ARROYO DE LOS CHAMISOS (NORTH FORK)					
Cross Section AP	2.1	230	760	1,380	3,900
Cross Section H	2.6	260	980	1,550	4,400
Cross Section B	3.3	310	1,140	1,800	5,200
ARROYO DEL ROSARIO					
Just Upstream of Rio Grande Ave	0.3	*	*	512	*
ARROYO EN MEDIO					
Upstream of Old Pecos Trail	0.7	*	*	684	*
Confluence with Arroyo de los Chamisos	3.4	*	*	1,800	*
ARROYO HONDO					
0.25 mile upstream of BNSF Railroad	10.8	*	*	1,650	*
Confluence with Canada del Rancho	16.3	810	1,910	2,490	4,040
Confluence with Arroyo de Los Chamisos	23.9	1,150	2,630	3,400	5,460
ARROYO MASCARAS					
Cross Section Z	1.5	185	670	1,080	3,100
Cross Section Y	1.2	265	980	1,580	4,500
Cross Section X	3.4	315	1,175	1,900	5,400
Cross Section C	5.4	410	1,510	2,400	6,900
Cross Section 1300	7.2	480	1,800	2,850	8,200
ARROYO RANCHITO					
Confluence with Arroyo Mascaras	0.7	*	*	300	*

*Data not computed

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the summary of Stillwater Elevations table in the Flood Insurance Study Report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction, and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures in this jurisdiction.

The **projection** used in the preparation of this map was New Mexico State Plane, Central zone. The **horizontal datum** was NAD 83, GRS80 spheroid. Differences in datum, spheroid, projection or State Plane zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

Spatial Reference System Division
National Geodetic Survey, NOAA
Silver Spring Metro Center
1315 East-West Highway
Silver Spring, Maryland 20910
(301) 713-3191

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713-3242**, or visit their website at www.ngs.noaa.gov.

Base map information shown on this FIRM was provided by the U.S. Geological Survey and Bohannan - Huston Incorporated (BHI). Digital Orthophoto Quadrangles were produced at a scale of 1:12,000 from photography dated 1997. BHI orthophotos were produced at a scale of 1:12,000 from photography dated 2001.

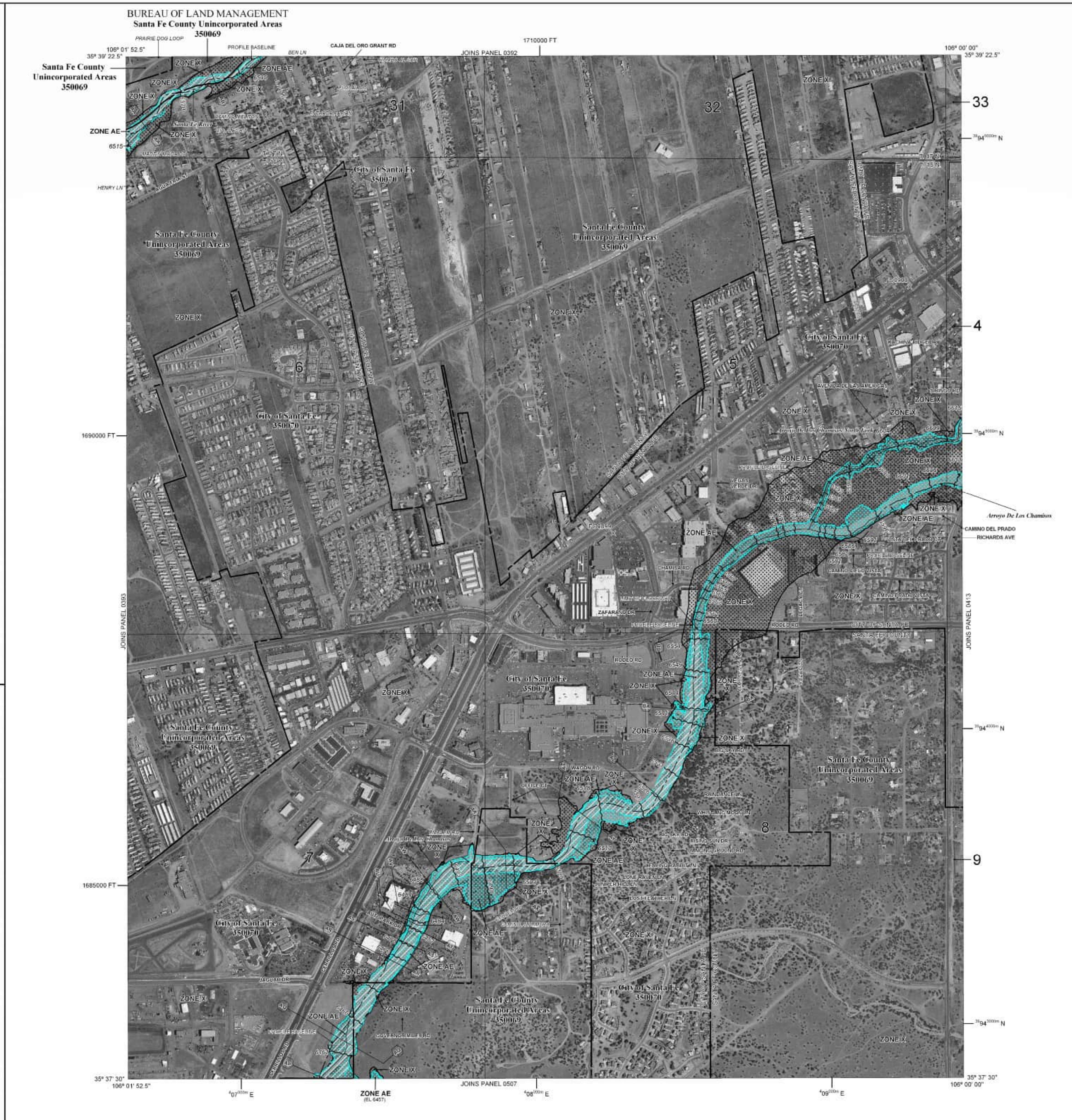
Based on updated topographic information, this map reflects more detailed and up-to-date **stream channel configurations and floodplain delineations** than those shown on the previous FIRM for this jurisdiction. As a result, the Flood Profiles and Floodway Data tables for the Flood Insurance Study report may reflect stream channel distances that differ from what is shown on the map. Also, the road to floodplain relationships for unrevised streams may differ from what is shown on previous maps.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and their website at www.msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1- 877- FEMA MAP** (1-877-336-2627) or visit the FEMA website at www.fema.gov.



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X — Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D — Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary
0.2% annual chance floodplain boundary
Floodway boundary
Zone D Boundary
CBRS and OPA Boundary
Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

A — A	Cross section line
23 - - - - 23	Transect line
97° 07' 30", 32° 22' 30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
$^{42}76^{000m}$E	1000-meter Universal Transverse Mercator grid values, zone 13
600000 FT	5000-foot grid ticks: New Mexico State Plane coordinate system, Central zone (FIPSZONE 3002), Transverse Mercator
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL

JUNE 17, 2008

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

250 0 500 1000 FEET
150 0 150 300 METERS

NFIP
NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0394D

FIRM
FLOOD INSURANCE RATE MAP
SANTA FE COUNTY, NEW MEXICO AND INCORPORATED AREAS

PANEL 394 OF 1100

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SANTA FE, CITY OF	350070	0394	D
SANTA FE COUNTY (UNINCORPORATED AREAS)	350069	0394	D

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

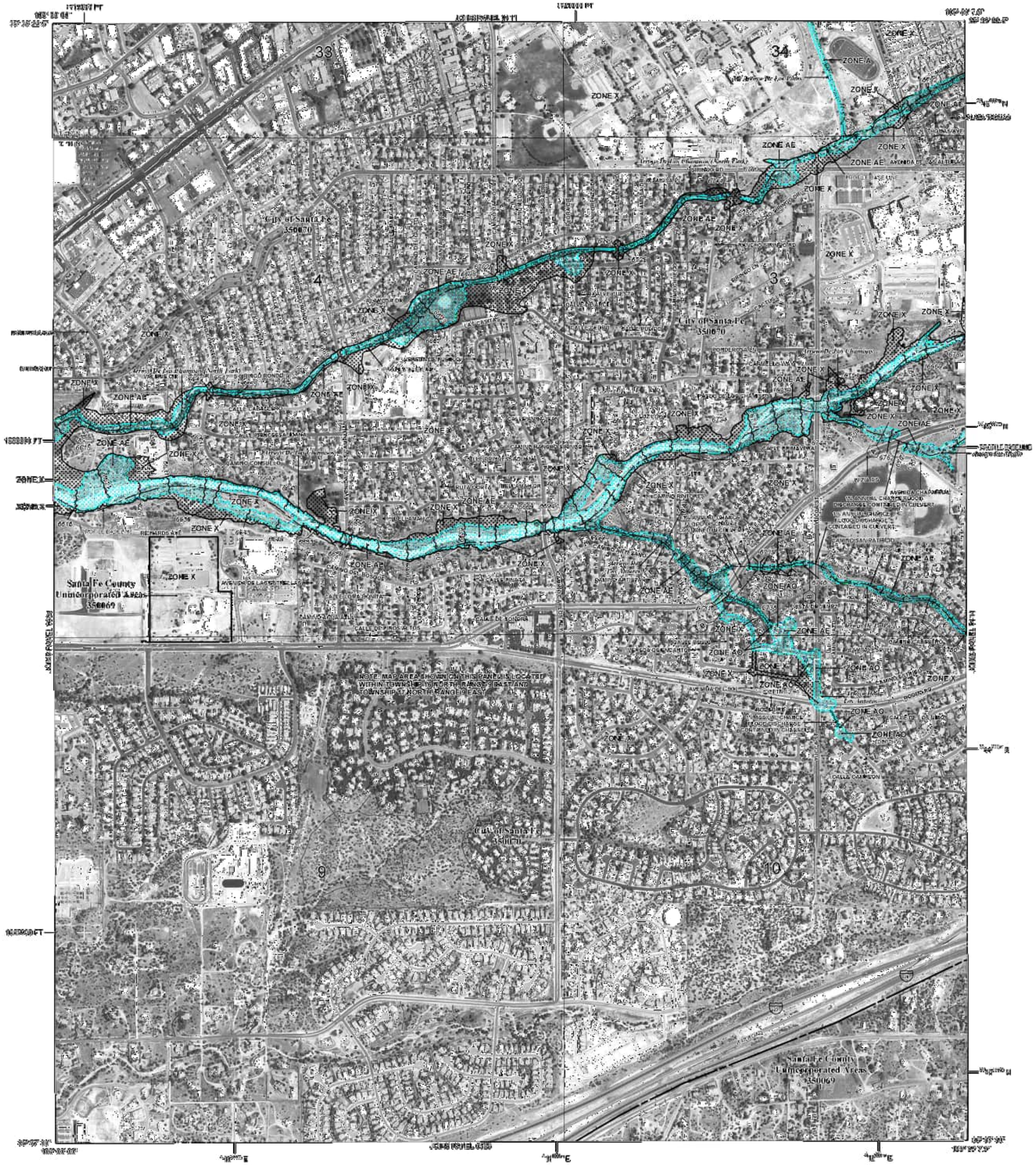
MAP NUMBER 35049C0394D

EFFECTIVE DATE JUNE 17, 2008

Federal Emergency Management Agency

NOTES TO USERS

LEGEND



MAP SCALE 1" = 500'

PANEL 0413E

FIRM

FLOOD INSURANCE RATE MAP

SANTA FE COUNTY, NEW MEXICO

AND INCORPORATED AREAS

PANEL 413 OF 1100

MAP NUMBER

MAP REVISED

DECEMBER 4, 2012

Federal Emergency Management Agency

APPENDIX

C HYDROLOGY

C.1. PRECIPITATION DATA

C.2. HYDROLOGIC SOIL GROUP

C.3. CURVE NUMBER CALCULATIONS

C.4. TIME OF CONCENTRATION CALCULATIONS

C.5. EXISTING STAGE STORAGE DATA FOR MARC BRANDT PARK

C.6. SPECIAL CONDITIONS FOR LR_28A AND LR_28B

C.7. HEC-HMS

C.1. Precipitation Data

NOAA Atlas 14, Volume 1, Version 5
Location name: Santa Fe, New Mexico, USA*
Latitude: 35.6474°, Longitude: -105.9964°
Elevation: 6639 ft**

* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.197** (0.171-0.226)	**0.255** (0.221-0.294)	**0.340** (0.294-0.390)	**0.404** (0.349-0.464)	**0.493** (0.424-0.564)	**0.561** (0.480-0.643)	**0.634** (0.539-0.725)	**0.708** (0.598-0.810)	**0.810** (0.677-0.927)	**0.889** (0.738-1.02)
10-min	**0.299** (0.260-0.344)	**0.388** (0.337-0.448)	**0.517** (0.447-0.594)	**0.615** (0.532-0.707)	**0.751** (0.645-0.859)	**0.854** (0.732-0.978)	**0.965** (0.821-1.10)	**1.08** (0.910-1.23)	**1.23** (1.03-1.41)	**1.35** (1.12-1.55)
15-min	**0.371** (0.323-0.426)	**0.481** (0.418-0.555)	**0.642** (0.555-0.737)	**0.762** (0.660-0.876)	**0.931** (0.800-1.06)	**1.06** (0.907-1.21)	**1.20** (1.02-1.37)	**1.34** (1.13-1.53)	**1.53** (1.28-1.75)	**1.68** (1.39-1.92)
30-min	**0.499** (0.435-0.574)	**0.648** (0.562-0.747)	**0.864** (0.747-0.992)	**1.03** (0.889-1.18)	**1.25** (1.08-1.43)	**1.43** (1.22-1.63)	**1.61** (1.37-1.84)	**1.80** (1.52-2.06)	**2.06** (1.72-2.35)	**2.26** (1.88-2.59)
60-min	**0.618** (0.538-0.710)	**0.802** (0.696-0.925)	**1.07** (0.925-1.23)	**1.27** (1.10-1.46)	**1.55** (1.33-1.77)	**1.76** (1.51-2.02)	**1.99** (1.70-2.28)	**2.23** (1.88-2.55)	**2.55** (2.13-2.91)	**2.80** (2.32-3.21)
2-hr	**0.740** (0.637-0.870)	**0.942** (0.811-1.11)	**1.24** (1.07-1.47)	**1.48** (1.26-1.74)	**1.80** (1.53-2.12)	**2.06** (1.74-2.41)	**2.34** (1.96-2.72)	**2.63** (2.18-3.06)	**3.02** (2.48-3.52)	**3.34** (2.71-3.89)
3-hr	**0.789** (0.690-0.924)	**1.00** (0.870-1.17)	**1.30** (1.13-1.52)	**1.54** (1.33-1.80)	**1.88** (1.61-2.18)	**2.14** (1.83-2.48)	**2.42** (2.05-2.80)	**2.72** (2.28-3.14)	**3.12** (2.59-3.62)	**3.45** (2.84-3.99)
6-hr	**0.909** (0.798-1.04)	**1.14** (1.00-1.31)	**1.46** (1.27-1.68)	**1.70** (1.48-1.95)	**2.05** (1.77-2.34)	**2.32** (1.99-2.65)	**2.59** (2.22-2.97)	**2.88** (2.45-3.30)	**3.28** (2.75-3.75)	**3.59** (2.99-4.12)
12-hr	**1.04** (0.925-1.19)	**1.31** (1.16-1.50)	**1.65** (1.46-1.88)	**1.91** (1.68-2.18)	**2.28** (2.00-2.59)	**2.56** (2.23-2.91)	**2.84** (2.46-3.24)	**3.14** (2.71-3.58)	**3.54** (3.02-4.02)	**3.85** (3.26-4.38)
24-hr	**1.21** (1.11-1.32)	**1.51** (1.39-1.65)	**1.88** (1.72-2.05)	**2.17** (2.00-2.37)	**2.56** (2.35-2.80)	**2.87** (2.62-3.13)	**3.18** (2.90-3.47)	**3.50** (3.17-3.81)	**3.93** (3.53-4.28)	**4.26** (3.81-4.64)
2-day	**1.36** (1.25-1.49)	**1.70** (1.56-1.86)	**2.11** (1.93-2.30)	**2.44** (2.23-2.65)	**2.87** (2.63-3.12)	**3.21** (2.92-3.49)	**3.55** (3.23-3.86)	**3.90** (3.53-4.24)	**4.35** (3.91-4.74)	**4.71** (4.21-5.14)
3-day	**1.48** (1.36-1.61)	**1.84** (1.69-2.01)	**2.28** (2.09-2.49)	**2.63** (2.41-2.86)	**3.10** (2.83-3.36)	**3.46** (3.15-3.76)	**3.82** (3.47-4.15)	**4.19** (3.79-4.55)	**4.68** (4.21-5.09)	**5.06** (4.52-5.51)
4-day	**1.59** (1.46-1.74)	**1.99** (1.82-2.16)	**2.45** (2.25-2.67)	**2.82** (2.58-3.07)	**3.32** (3.03-3.60)	**3.70** (3.37-4.02)	**4.09** (3.71-4.44)	**4.48** (4.05-4.86)	**5.00** (4.50-5.43)	**5.40** (4.84-5.87)
7-day	**1.86** (1.71-2.01)	**2.31** (2.13-2.51)	**2.83** (2.61-3.07)	**3.24** (2.98-3.50)	**3.78** (3.47-4.08)	**4.18** (3.84-4.52)	**4.59** (4.20-4.95)	**4.99** (4.56-5.40)	**5.52** (5.02-5.98)	**5.92** (5.36-6.42)
10-day	**2.09** (1.93-2.28)	**2.60** (2.40-2.84)	**3.19** (2.95-3.48)	**3.66** (3.38-3.98)	**4.28** (3.94-4.66)	**4.75** (4.36-5.18)	**5.23** (4.79-5.70)	**5.70** (5.21-6.22)	**6.32** (5.74-6.90)	**6.80** (6.15-7.43)
20-day	**2.77** (2.56-3.03)	**3.44** (3.18-3.77)	**4.19** (3.86-4.58)	**4.76** (4.38-5.20)	**5.48** (5.05-5.99)	**6.02** (5.53-6.58)	**6.54** (6.00-7.16)	**7.05** (6.45-7.72)	**7.68** (7.01-8.42)	**8.15** (7.41-8.94)
30-day	**3.38** (3.14-3.65)	**4.20** (3.91-4.53)	**5.08** (4.72-5.47)	**5.72** (5.32-6.16)	**6.54** (6.08-7.04)	**7.13** (6.61-7.67)	**7.70** (7.13-8.30)	**8.24** (7.61-8.88)	**8.90** (8.19-9.61)	**9.37** (8.60-10.1)
45-day	**4.21** (3.94-4.50)	**5.21** (4.88-5.58)	**6.23** (5.83-6.66)	**6.97** (6.53-7.44)	**7.88** (7.37-8.40)	**8.51** (7.96-9.07)	**9.09** (8.50-9.70)	**9.63** (8.99-10.3)	**10.3** (9.57-11.0)	**10.7** (9.97-11.4)
60-day	**4.86** (4.56-5.19)	**6.03** (5.65-6.44)	**7.20** (6.76-7.70)	**8.05** (7.55-8.60)	**9.08** (8.52-9.70)	**9.79** (9.18-10.5)	**10.5** (9.80-11.2)	**11.1** (10.4-11.8)	**11.8** (11.0-12.6)	**12.3** (11.5-13.2)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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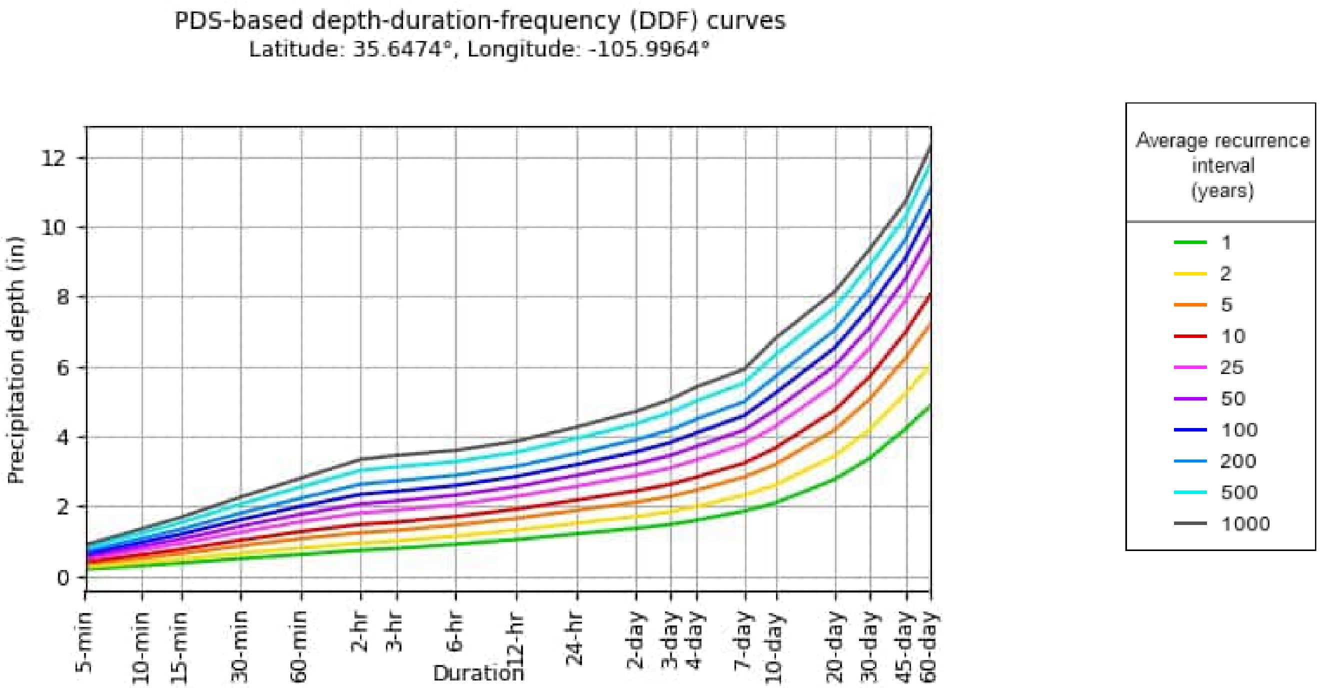
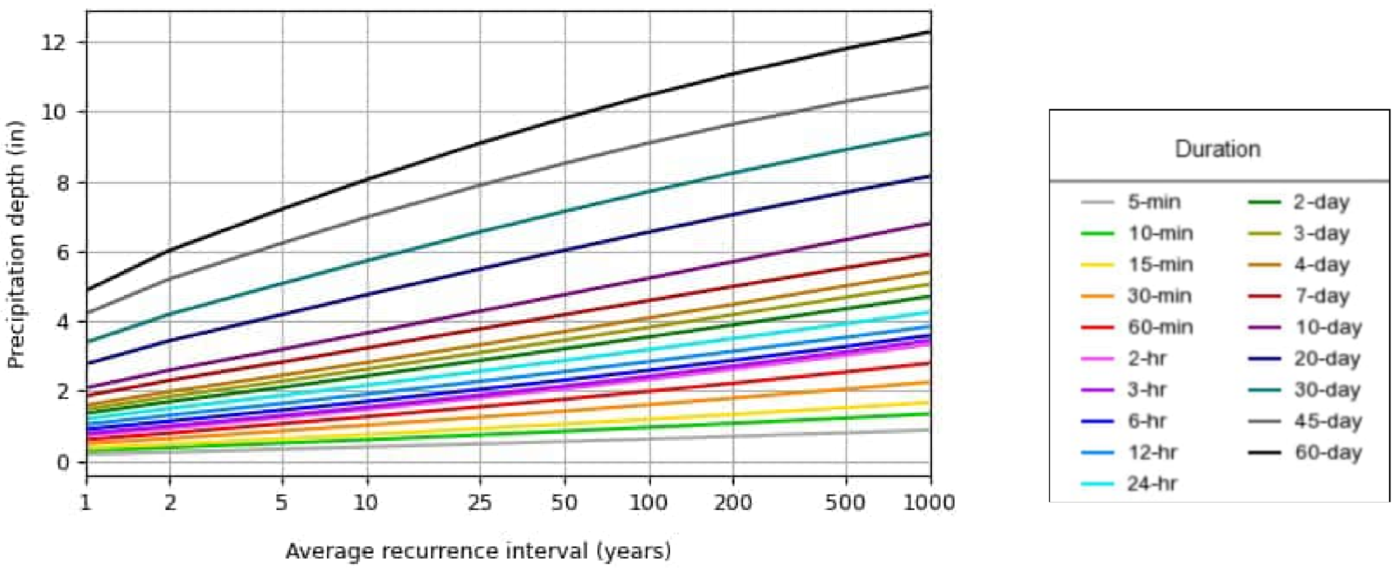
PF graphical





NOAA Atlas 14, Volume 1, Version 5 Created (GMT): Wed Apr 10 15:04:06 2024

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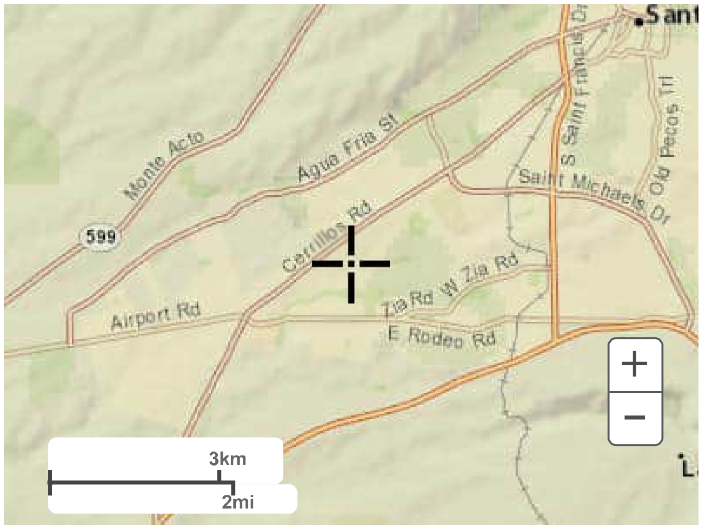
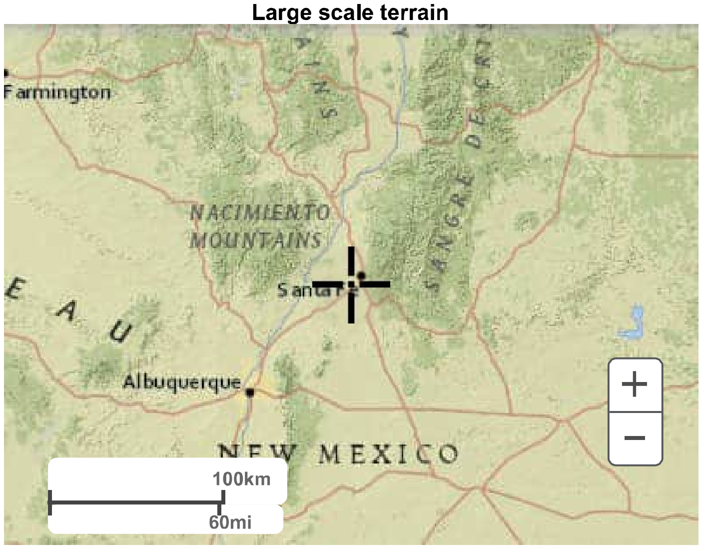
Maps & aerials

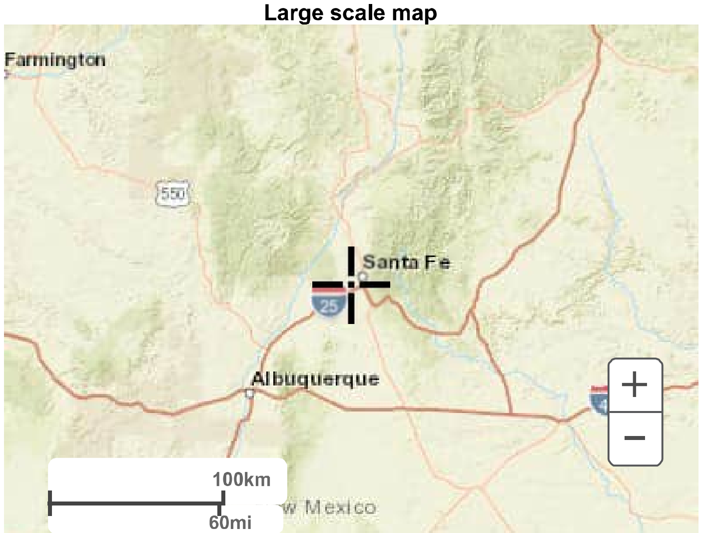
Small scale terrain



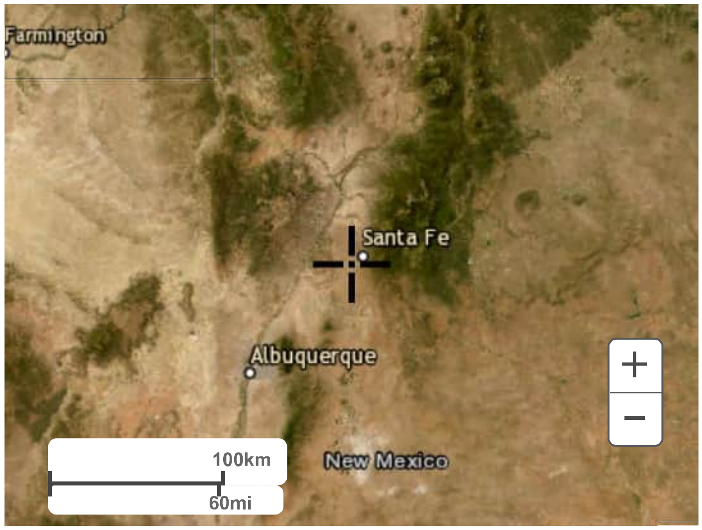
Large scale terrain



Large scale map



Large scale aerial



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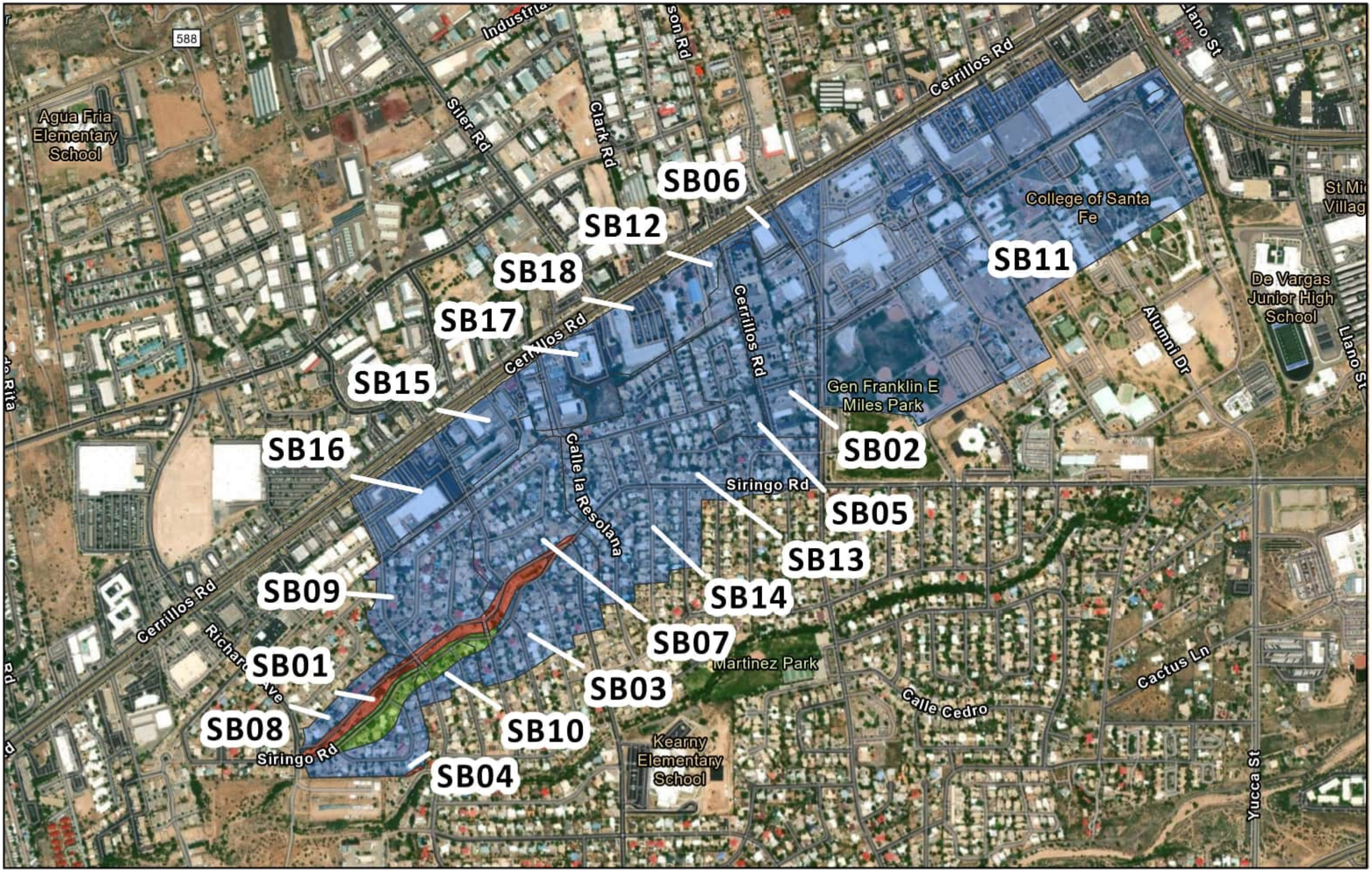
US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

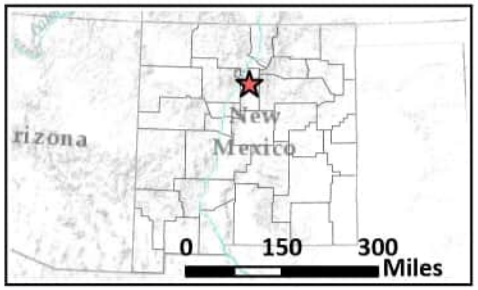
C.2. Hydrologic Soil Group



HSG

- A
- B
- C

0 1,000 2,000 Feet



Hydrologic Soil Group

Arroyo De Los Chamisos
Crossing Project
Santa Fe County, New Mexico

N

wsp

J:\30900483 CoSanta Fe On-Call Design Services\03 Task Orders\Task 04 - ADLC IC and ID\04 Engineering - CN LP50039 (See ProjectWise)\Proposed\Drainage\Preliminary\GIS\Exhibits\maps\Figure3 Soil Map

C.3. Curve Number Calculations

CURVE NUMBER CALCULATION (1 of 2)							
Subbasin	**Soil FID**	**Land Use**	**Hydrologic Soil Group**	**CN**	**Area (Acres)**	**Total Area**	**Weighted CN**
RA01		ROW		98	0.39	0.39	**98**
RA02		ROW		98	0.3	0.3	**98**
RA03		ROW		98	0.2	0.2	**98**
RA04		ROW		98	0.25	0.25	**98**
RA05		ROW		98	0.29	0.29	**98**
RA06		ROW		98	0.23	0.23	**98**
RA07		ROW		98	0.3	0.3	**98**
RA08		ROW		98	0.57	0.57	**98**
RA09		ROW		98	0.38	0.38	**98**
RA10		ROW		98	0.35	0.35	**98**
RA11		ROW		98	0.056	0.056	**98**
RA12		ROW		98	0.68	0.68	**98**
RA13		ROW		98	0.21	0.21	**98**
RA14		ROW		98	0.87	0.87	**98**
RA15	201	C2	A	79	4.58		
	208	C2	C	79	0.2	4.78	**79**
RA16	201	C2	A	79	0.2		
	208	C2	C	79	7.2	7.4	**79**
RA17	208	C2	C	79	2.9	2.9	**79**
SC01	208	R5	B	75	0.1		
	204	R5	C	83	3.84		
		ROW		98	0.46	4.4	**84**
SC02		R5	C	98	1.04	1.04	**98**
CR01		ROW		98	2.3	2.3	**98**
CR02		ROW		98	2.47	2.47	**98**
CR03		C2	C	94	8.6	8.6	**94**
CR04		C2	C	94	1.3	1.3	**94**
CR05		ROW		98	3.14	3.14	**98**
CR06		ROW		98	1.84	1.84	**98**
CR07		ROW		98	1.66	1.66	**98**
CR08		ROW		98	1.66	1.66	**98**
CR09		ROW		98	1.3	1.3	**98**
CR10		ROW		98	1.83	1.83	**98**
CR11		ROW		98	0.6	0.6	**98**

CURVE NUMBER CALCULATION (2 of 2)							
Subbasin	**Soil FID**	**Land Use**	**Hydrologic Soil Group**	**CN**	**Area (Acres)**	**Total Area**	**Weighted CN**
SB01	204	R-5	B	69	4.1		
	208	R-5	C	79	0.4		
	211	R-5	A	49	1.1	5.6	**61**
SB02	208	R21PUD	C	83	11.7		
	208	C-2	C	94	3.42		
	208	ROW	C	98	2.68	17.8	**87**
SB03	204	R-5	B	75	0.3		
	208	R-5	C	83	11.22		
	211	R-5	A	61	0.26		
		ROW	C	98	1.72	13.5	**84**
SB04	204	R-5	B	75	0.1		
	208	R-5	C	83	4.67		
	211	R-5	A	61	2.6		
		ROW	C	98	1.53	8.9	**79**
SB05	208	R12PUD	C	83	0.4		
	208	R5	C	83	0.06		
	208	ROW	C	83	0.89		
	208	R7	C	83	6.15	7.5	**83**
SB06	207	C-2	C	94	1.6		
	208	C-2	C	94	0.2	1.8	**94**
SB07	204	R-5	B	75	0.7		
	208	R-5	C	83	11.89		
		ROW	C	98	2.01	14.6	**84**
SB08	204	R-5	B	75	0.7		
	208	R-5	C	83	2.55		
		ROW	C	98	0.35	3.6	**82**
SB09	204	R-5	B	75	0.17		
	208	R-5	C	83	13.53		
		ROW	C	98	2.2	15.9	**84**
SB10	204	R-5	B	75	0.0081		
	208	R-5	C	83	1.86		
	211	R-5	A	61	1.07		
		ROW		98	0.47	3.4081	**78**
SB11	207	R5	C	83	72.46	72.5	**78**
SB12	207	C-2	C	94	5.6		
	208	C-2	C	94	1.7	7.3	**90**
SB13	208	R12PUD	C	83	9.87		
	ROW		C	98	2.65		
	208	R5	C	83	15.68	28.2	**84**
SB14	208	R5	C	83	14.77		
	ROW		C	98	1.68	14.8	**84**
SB15	207	C-2	C	94	9.61		
		ROW	C	98	1.49	11.1	**94**
SB16	207	SC-1	C	94	8.69		
	207	C-2	C	94	0.73	9.4	**94**
SB17	207	C-2	C	94	6.99		
	208	C-2	C	94	2.32		
	ROW	C-2	C	98	0.29	9.6	**94**

C.4. Time of Concentration Calculations

Arroyo de Los Chamisos Inlet (Tc) Calculations (Velocity Method)

Manning's n				
Sheet Flow	Grass	0.150	**Smooth Surface**	0.011
Gutter Flow	Concrete gutter with asphalt pavemen (smooth)	0.013		

See Table 402-7 and 502-5 of Drainage Manual for more values

P_2 = 1.51 in. 2-year, 24-hour rainfall; City of Santa fe

← Input | Calculated →

	SHEET FLOW (<= 300 FEET)				SHALLOW CONCENTRATED FLOW			GUTTER FLOW						SHEET FLOW			SHALLOW CONCENTRATED FLOW			GUTTER FLOW						TOTAL		
DRAINAGE BASIN	US ELEV. (FT)	DS ELEV. (FT)	LENGTH (FT)	MANNING'S N VALUE	DS ELEV. (FT)	LENGTH (FT)	k	LENGTH (FT)	DEPTH (FT)	US ELEV. (FT)	DS ELEV. (FT)	CROSS-SLOPE (SX)	Ku	SLOPE (FT/FT)	T_{T1} (HR)	TT1 (HR) Taken	SLOPE (FT/FT)	VELOCITY (FT/SEC)	T_{T2} (HR)	SLOPE FT/FT	WIDTH OF FLOW (T) (FT)	FLOW AREA SQFT	DISCHARGE CFS	VELOCITY FT/S	T_{T3} (HR)	TIME OF CONCENTRATION $T_C=T_{T1}+T_{T2+}T_{T3}$ (HR)	(MIN)	Tc Taken
SB_02	6742.0	6740.0	80.0	0.150	6716.0	1440.0	20.328							0.025	0.182	0.182	0.017	2.624	0.152				0.000			0.334	20.058	20.1
SB_03	6696.0	6692.0	110.0	0.150				1970	0.30	6692	6660	0.020	0.560	0.036	0.202	0.202				0.016	15.000	2.250	11.027	4.901	0.1117	0.314	18.82	18.8
SB_04	6664.0	6662.0	130.0	0.150				935	0.20	6662	6634	0.020	0.560	0.015	0.326	0.326				0.030	10.000	1.000	5.072	5.072	0.0512	0.377	22.61	22.6
SB_05	6730.0	6724.0	100.0	0.150				705	0.20	6724	6714	0.020	0.560	0.060	0.153	0.153				0.014	10.000	1.000	3.490	3.490	0.0561	0.209	12.56	12.6
SB_06	6746.0	6744.0	65.0	0.011	6738.0	465.0	20.328							0.031	0.018	0.083	0.013	2.309	0.056							0.139	8.36	12.0
SB_07	6706.0	6704.0	68.0	0.150				1382	0.25	6700	6672	0.020	0.560	0.029	0.150	0.150				0.020	12.500	1.563	7.569	4.844	0.0792	0.229	13.73	13.7
SB_08	6656.0	6654.0	150.0	0.150				1085	0.25	6654	6632	0.020	0.560	0.013	0.387	0.333				0.020	12.500	1.563	7.572	4.846	0.0622	0.395	23.71	23.7
SB_09	6690.0	6688.0	130.0	0.150				1280	0.20	6688	6654	0.020	0.560	0.015	0.326	0.326				0.027	10.000	1.000	4.776	4.776	0.0744	0.400	24.01	24.0
SB_10	6674.0	6672.0	110.0	0.150				570.0	0.20	6672.0	6656.0	0.020	0.560	0.018	0.267	0.267				0.028	10.000	1.000	4.910	4.910	0.0322	0.299	17.93	17.9
SB_11								3560.0	0.20	6806.0	6732	0.020	0.560							0.021	10.000	1.000	4.225	4.225	0.2340	0.234	14.04	14.0
SB_12	6740	6736.0	170.0	0.150	6722.0	760.0	20.328					0.024	0.341	0.024	0.341	0.333	0.018	2.759	0.077				0.000			0.410	24.57	24.6
SB_13	6732.0	6730.0	130.0	0.150				2090	0.20	6730	6682	0.020	0.560	0.015	0.326	0.326				0.023	10.000	1.000	4.441	4.441	0.1307	0.456	27.38	27.4
SB_14								1650.0	0.20	6724.0	6682.0	0.020	0.560							0.025	10.000	1.000	4.676	4.676	0.0980	0.098	5.88	12.0
SB_15	6718.0	6716.0	150.0	0.011	6706.0	1260.0	20.328	1263.0	0.20	6706.0	6690.0	0.020	0.560	0.013	0.048	0.083	0.008	1.811	0.193				0.000			0.277	16.60	16.6
SB_16	6702	6698	140	0.011	6678	1030	20.328							0.029	0.033	0.083	0.019	2.833	0.101				0.000			0.184	11.06	12.0
SB_17	6730.0	6728.0	110.0	0.011	6720.0	1240.0	20.328	620.00	0.30	6720.00	6708.00	0.02	0.560	0.018	0.033	0.083	0.006	1.633	0.211	0.019	15.000	2.250	12.037	5.350	0.0322	0.326	19.59	19.6
SB_18	6732.0	6728.0	100.0	0.011	6716.0	950.0	20.328							0.040	0.022	0.083	0.013	2.285	0.116				0.000			0.199	11.93	12.0

C.5. Existing Stage Storage Data for Marc Brandt Park and Franklin Miles Park

Stage Storage Data			
Mark Brandt Park		Franklin Miles park	
Elevation (ft)	Area (ft^2)	Elevation (ft)	Area (ft^2)
6626	73.86	6723	439.39
6627	467.1	6724	3634.6
6628	3844.81	6725	12554.7
6629	6338.61	6726	18494.74
6630	10177.92	6727	23868.55
6631	16695.22	6728	29065.77
6632	39003.53	6729	33819.37

C.6. Special Conditions for LR_28A and LR_28B

Pond Stage Storage at the Shopping Center on the Corner of Richards and Cerrillos

POND 1				POND 2				POND 3			
Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)	Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)	Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)
6662	310	N/A	N/A	6656	270	N/A	0	6654	1264	N/A	0
6663	696	503	503	6657	626	448	448	6656	3767	5031	**5031**
6664	1493	1095	1598	6658	1273	950	1398				
6665	2506	2000	3598	6659	2279	1776	3174				
6666	3759	3133	6730	6660	3609	2944	**6118**				
6667	4376	4068	10798								
6668	5189	4783	**15581**								
POND 4				POND 5				POND 5			
Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)	Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)	Contour Elevation (ft)	Contour Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)
6654	448	N/A	0	6652	1637	N/A	0	6658	276	N/A	0
6655	953	701	701	6653	3167	2402	2402	6659	838	557	557
6656	1554	1254	**1954**	6654	4787	3977	6379	6660	1378	1108	1665
				6655	6263	5525	**11904**	6661	1980	1679	3344
								6662	2715	2348	**5692**
Total Ponding Volume		**46280 ft^2**	**1.1 acre-feet**								

C.7. HEC-HMS

50 Yr-24 Hr Output for Existing Conditions				
Hydrologic Element	Draiange Area (sq.mi^2)	Peak Discharge	Time of Peak	Volume (acre-feet)
Arroyo Reach	0.233	261.2	7 May 2024, 06:20	0.557
Arroyo Trail Pond	0.467	417.8	7 May 2024, 06:20	0.640
Bikepath	0.0203	29.4	7 May 2024, 06:10	0.023
Cerrillos Rd		1.3	7 May 2024, 06:10	0.023
CR_01	0.0036	9.6	7 May 2024, 06:10	0.020
CR_02	0.004	10.6	7 May 2024, 06:10	0.064
CR_03	0.0134	35.4	7 May 2024, 06:10	0.107
CR_04	0.002	5.3	7 May 2024, 06:10	0.673
CR_05	0.0049	13.0	7 May 2024, 06:10	0.106
CR_06	0.0029	7.6	7 May 2024, 06:10	0.886
CR_07	0.0026	6.9	7 May 2024, 06:10	0.742
CR_09	0.002	5.4	7 May 2024, 06:10	0.135
CR_10	0.0029	7.6	7 May 2024, 06:10	1.763
CR_11	0.0009	2.4	7 May 2024, 06:10	0.519
CR08	0.0026	6.9	7 May 2024, 06:10	0.166
DiversionC1	0.0036	1.6	7 May 2024, 06:10	2.448
DiversionC2		21.4	7 May 2024, 06:10	2.448
DiversionC3		0.0	6 May 2024, 24:00	6.738
DiversionC4	0.0029	0.7	7 May 2024, 06:10	6.743
DiversionC5		2.1	7 May 2024, 06:10	6.743
DiversionC6		2.7	7 May 2024, 06:10	3.309
DiversionC7		2.3	7 May 2024, 06:10	2.428
DiversionC8		10.4	7 May 2024, 06:10	0.356
DivertedC3	0	39.0	7 May 2024, 06:10	0.358
DivertedC4	0	7.0	7 May 2024, 06:10	2.787

DivertedC5	0	6.2	7 May 2024, 06:05	2.791
DivertedC6	0	6.8	7 May 2024, 06:05	0.859
DivertedC7	0	6.4	7 May 2024, 06:05	3.650
DivertedC9	0	1.3	7 May 2024, 06:05	3.654
Diverted C8	0	0.0	6 May 2024, 24:00	1.730
GSI Facility	0.0001	0.3	7 May 2024, 06:10	1.780
Inlet 1		3.8	7 May 2024, 06:10	1.780
Inlet 2	0.0089	2.4	7 May 2024, 06:10	0.831
Inlet 3	0.0172	5.7	7 May 2024, 06:10	1.094
JRA-14	0.0008	2.0	7 May 2024, 06:10	1.096
JRA-14.1	0.0017	4.4	7 May 2024, 06:10	0.832
JunctionC1	0.0036	9.6	7 May 2024, 06:10	19.144
JunctionC2		53.8	7 May 2024, 06:10	19.163
JunctionC3		39.0	7 May 2024, 06:10	19.163
JunctionC4	0.0029	7.6	7 May 2024, 06:10	19.172
JunctionC5		8.3	7 May 2024, 06:10	1.730
JunctionC6		9.5	7 May 2024, 06:10	1.504
JunctionC7		8.7	7 May 2024, 06:10	2.017
JunctionC8		10.4	7 May 2024, 06:10	2.018
JunctionC9		2.6	7 May 2024, 06:10	24.423
Junction-RA-5	0.0458	49.9	7 May 2024, 06:10	1.844
Junction-RA09	0.0006	1.6	7 May 2024, 06:10	1.880
Junction-RA1		1.3	7 May 2024, 06:10	1.780
Junction-RA10	0.0005	1.4	7 May 2024, 06:10	1.781

Junction-RA11	0.0203	29.4	7 May 2024, 06:10	0.817
Junction-RA12	0.019	26.0	7 May 2024, 06:10	0.409
Junction-RA3	0.0273	20.2	7 May 2024, 06:10	0.277
Junction-RA4	0.0377	36.9	7 May 2024, 06:10	31.531
Junction-RA6	0.0458	49.9	7 May 2024, 06:10	2.448
Junction-RA7	0.0017	4.4	7 May 2024, 06:10	0.127
Junction-1	0.09	92.6	7 May 2024, 06:15	0.065
Junction-2	0.233	270.6	7 May 2024, 06:15	0.042
Junction-3	0.233	265.6	7 May 2024, 06:15	0.234
Junction-4	0.0458	49.9	7 May 2024, 06:10	34.213
Junction-5	0.0198	28.0	7 May 2024, 06:10	1.884
Junction-7	0.043	58.0	7 May 2024, 06:15	0.563
Junction-8	0.031	44.5	7 May 2024, 06:15	0.282
Junction-9	0.293	335.5	7 May 2024, 06:15	34.711
Los Chamisos Outlet	0.0006	1.6	7 May 2024, 06:10	37.439
Los Pinos Outlet	0.5144	447.2	7 May 2024, 06:20	0.290
LR_27	0.0094	14.2	7 May 2024, 06:10	0.076
LR_28A	0.0089	12.0	7 May 2024, 06:10	0.998
LR_28B	0.0078	10.4	7 May 2024, 06:10	0.473
LR_39	0.0094	12.6	7 May 2024, 06:10	1.471
LR28A	0.0089	2.4	7 May 2024, 06:10	0.108
LR28A_GSI Sink	0	9.6	7 May 2024, 06:10	1.579
LR28B	0.0078	2.1	7 May 2024, 06:10	0.076
LR28B_GSI_Sink	0	8.3	7 May 2024, 06:10	1.655

MainC3		0.0	6 May 2024, 24:00	1.655
MainC4	0.0029	0.7	7 May 2024, 06:10	0.290
MainC5		2.1	7 May 2024, 06:10	0.690
MainC6		2.7	7 May 2024, 06:10	361.547
MainC7		2.3	7 May 2024, 06:10	362.237
MainC8		10.4	7 May 2024, 06:10	0.000
MainC9	0	0.0	6 May 2024, 24:00	0.000
Miles Pond	0.09	92.6	7 May 2024, 06:15	0.369
OverlandC2		21.4	7 May 2024, 06:10	0.369
OverlandFlowC1	0.0036	1.6	7 May 2024, 06:10	0.509
Pond	0.0017	4.4	7 May 2024, 06:10	0.509
Pond 1	0.0017	0.0	7 May 2024, 07:10	0.011
RA-04	0.001	2.5	7 May 2024, 06:10	0.011
RA-05	0.0012	3.1	7 May 2024, 06:10	0.005
RA-06	0.0003	0.8	7 May 2024, 06:10	0.405
RA-07	0.0005	1.2	7 May 2024, 06:10	0.405
RA-08	0.0009	2.4	7 May 2024, 06:10	0.005
RA-09	0.0006	1.6	7 May 2024, 06:10	0.403
RA-10	0.0005	1.4	7 May 2024, 06:10	39.209
RA-12	0.0008	2.0	7 May 2024, 06:10	39.612
RA-13	0.0001	0.3	7 May 2024, 06:10	39.612
RA-14	0.0008	2.0	7 May 2024, 06:10	39.612
RA-14.1	0.0009	2.4	7 May 2024, 06:10	0.366
RA-15	0.0075	8.4	7 May 2024, 06:10	19.438
RA-16	0.0115	17.6	7 May 2024, 06:10	19.804
RA-17	0.0045	6.5	7 May 2024, 06:10	0.022
RA_01	0.001	2.6	7 May 2024, 06:10	0.022
RA_02	0.0005	1.2	7 May 2024, 06:10	0.365
RA_03	0.0008	2.0	7 May 2024, 06:10	36.655
Reach-1	0.003	6.9	7 May 2024, 06:15	37.019
Reach-10	0.233	265.6	7 May 2024, 06:15	0.035
Reach-11	0.017	33.3	7 May 2024, 06:15	0.035
Reach-12	0.015	32.4	7 May 2024, 06:10	0.286
Reach-2	0.031	43.6	7 May 2024, 06:15	45.929
Reach-3	0.043	57.5	7 May 2024, 06:15	46.215
Reach-4	0.011	15.2	7 May 2024, 06:20	0.032
Reach-5	0.007	16.1	7 May 2024, 06:15	0.032
Reach-8	0.015	27.2	7 May 2024, 06:15	0.129
SB01	0.109	14.4	7 May 2024, 06:25	2.638
SB02	0.028	37.6	7 May 2024, 06:15	2.767
SB03	0.02	23.5	7 May 2024, 06:15	0.020

SB04	0.014	10.9	7 May 2024, 06:20	0.137
SB05	0.012	15.7	7 May 2024, 06:10	0.020
SB06	0.003	7.0	7 May 2024, 06:10	0.157
SB07	0.023	30.1	7 May 2024, 06:10	0.128
SB09	0.025	25.8	7 May 2024, 06:20	0.106
SB10	0.005	4.1	7 May 2024, 06:15	0.106
SB11	0.09	107.2	7 May 2024, 06:10	0.234
SB12	0.011	15.4	7 May 2024, 06:20	0.234
SB13	0.044	42.3	7 May 2024, 06:20	0.000
SB14	0.023	32.7	7 May 2024, 06:10	0.084
SB15	0.017	33.7	7 May 2024, 06:15	0.084
SB16	0.015	35.0	7 May 2024, 06:10	0.084
SB17	0.015	28.4	7 May 2024, 06:15	0.016
SB18	0.007	16.3	7 May 2024, 06:10	0.016
SB8	0.006	5.5	7 May 2024, 06:20	0.537
SC_01	0.0069	9.8	7 May 2024, 06:10	0.617
SourceC2		2.5	6 May 2024, 24:00	0.000
SourceC5		1.4	6 May 2024, 24:00	37.150
SourceC6		2.6	6 May 2024, 24:00	362.237
SourceC7		3.3	6 May 2024, 24:00	0.498
SourceC8		2.8	6 May 2024, 24:00	2.747
SourceC9		0.2	6 May 2024, 24:00	19.782
Source_C3		26.0	6 May 2024, 24:00	36.985
StormSewersC1	0	8.0	7 May 2024, 06:10	46.184
StormSewersC2	0	32.4	7 May 2024, 06:05	0.000
				0.401

APPENDIX

D HYDRAULICS

D.1 FLOWMASTER INLET TABLES

D.2. GUTTER FLOW OUTPUT

D.3. STORMCAD PROFILE

D.4. SCOUR CALCULATIONS

D.1. Flowmaster Inlet Tables

Flowmaster Outputs

Combination Inlets on Grade Physical Characteristics (typ)														
Gutter Depression (in)	Total Depression (in)	Grate Flow Option	Calculation Option	Clogging (%)	Curb Opening Length (ft)	Grate Length (ft)	Grate Type	Grate Width (ft)	Gutter Cross Slope (ft/ft)	Gutter Width (ft)	Local Depression (in)	Local Depression Width (in)	Road Cross Slope (ft/ft)	Roughness Coefficient
0	2	Exclude None	Use Both	25	10.7	5.5	P-50 mm (P-1-7/8")	2	0.02	2	2	24	0.02	0.017

Combination Inlets on Grade (results)																	
Label	Discharge (cfs)	Slope (ft/ft)	Efficiency (%)	Intercepted Flow (cfs)	Bypass Flow (cfs)	Spread (ft)	Depth (in)	Flow Area (ft^2)	Velocity (ft/s)	Splash Over Velocity (ft/s)	Frontal Flow Factor	Side Flow Factor	Grate Flow Ratio	Equivalent Cross Slope (ft/ft)	Active Grate Length (ft)	Length Factor	Total Interception Length (ft)
Inlet 1	2.56	0.006	96.01	2.46	0.1	11.5	2.8	1.3	1.92	11.7	1	0.517	0.398	0.053	4.1	0.509	12.9
Inlet 2	2.46	0.006	96.38	2.37	0.09	11.4	2.7	1.3	1.9	11.7	1	0.522	0.403	0.054	4.1	0.52	12.6
Inlet 3	2.79	0.006	95.16	2.65	0.14	11.9	2.9	1.4	1.96	11.7	1	0.508	0.388	0.052	4.1	0.486	13.5
Inlet 4	6.3	0.03	71.92	4.53	1.77	12	2.9	1.4	4.4	11.7	1	0.194	0.386	0.052	4.1	0.213	30.8
Inlet 5.1	4.29	0.033	78.91	3.39	0.9	10.2	2.4	1	4.14	11.7	1	0.212	0.442	0.057	4.1	0.256	25.6
Inlet 5.2	5.06	0.043	74.54	3.77	1.29	10.3	2.5	1.1	4.77	11.7	1	0.173	0.438	0.056	4.1	0.219	29.8
Inlet 6	7.3	0.03	69.03	5.04	2.26	12.6	3	1.6	4.56	11.7	1	0.184	0.368	0.051	4.1	0.196	33.3
Inlet 7.1	1.29	0.01	99.32	1.28	0.01	8.1	1.9	0.7	1.96	11.7	1	0.508	0.53	0.064	4.1	0.652	10
Inlet 7.2	1.56	0.005	99.51	1.55	0.01	9.9	2.4	1	1.58	11.7	1	0.602	0.452	0.058	4.1	0.694	9.4
Inlet 8	4.91	0.01	83.96	4.12	0.79	13.4	3.2	1.8	2.74	11.7	1	0.362	0.351	0.049	4.1	0.317	20.7
Inlet 9	0.78	0.02	99.96	0.78	0	5.9	1.4	0.3	2.24	11.7	1	0.448	0.669	0.076	4.1	0.722	9.1
Inlet 10	0.72	0.02	100	0.72	0	5.7	1.4	0.3	2.2	11.7	1	0.457	0.683	0.077	4.1	0.753	8.7
Inlet 11	0.72	0.02	100	0.72	0	5.7	1.4	0.3	2.2	11.7	1	0.457	0.683	0.077	4.1	0.753	8.7
Inlet 12	2.19	0.006	97.35	2.13	0.06	10.9	2.6	1.2	1.85	11.7	1	0.535	0.418	0.055	4.1	0.553	11.8

Combination Inlet in Sag																			
Discharge (cfs)	Spread (ft)	Gutter Width (ft)	Gutter Cross Slope (ft/ft)	Road Cross Slope (ft/ft)	Local Depression (in)	Local Depression Width (in)	Grade Width (ft)	Grate Length (ft)	Grate Type	Clogging (%)	Curb Opening Length (ft)	Opening Height (ft)	Curb Throat Type	Throat Incline Angle	Depth (in)	Gutter Depression (in)	Total Depression (in)	Open Grate Area (ft)	Active Grate Weir Length (ft)
3.46	12.5	2	0.02	0.02	2	24	2	3.5	P-50 mm (P-1-7/8")	50	3.5	5	Horizontal	90	3	0	2	3.1	5.5

Cerrillos Road Inlet Capacities (50-yr Storm)											
Subbasin	**Long Slope**	**Cross-Slope**	**Manning's Roughness**	**Gutter Width**	**Design Flow**	**Curb Depth**	**Intercepted Flow**	**Clogging Factor**	**No. of Inlets**	**Intercepted Flow**	**Bypass Flow**
	ft/ft	**ft/ft**		**ft**	**cfs**	**ft**	**cfs**	**20%**		**cfs**	**cfs**
CR01	0.02	0.02	0.013	1.7	10.40	0.28	4.976	3.9808	2	7.9616	2.4
CR02	0.02	0.02	0.013	1.7	58.44	0.53	13.5	10.8	3	32.4	26.0
CR05	0.02	0.02	0.013	1.7	40.24	0.46	10.443	8.3544	5	41.772	0.0
CR06	0.02	0.02	0.013	1.7	8.30	0.28	4.347	3.4776	2	6.9552	1.3
CR07	0.02	0.02	0.013	1.7	8.84	0.27	3.878	3.1024	2	6.2048	2.6
CR08	0.02	0.02	0.013	1.7	10.14	0.28	4.27	3.416	2	6.832	3.3
CR09	0.02	0.02	0.013	1.7	9.21	0.27	3.99	3.192	2	6.384	2.8
CR10	0.02	0.02	0.013	1.7	11.12	0.29	4.556	3.6448	3	10.9344	0.2
CR11	0.02	0.02	0.013	1.7	2.79	0.17	1.665	1.332	1	1.332	1.5

D.2. Gutter Flow Output

Gutter (InletCapacityCalcs_REV.fm8)

Label	Channel Slope (ft/ft)	Discharge (cfs)	Gutter Width (ft)	Spread (ft)	Manning Coefficient	Flow Area (ft^2)	Depth (in)	Velocity (ft/s)
RA14	0.013	0.99	1.5	7.0	0.017	0.5	1.7	2.02
RA14.1	0.013	2.19	1.5	9.4	0.017	0.9	2.3	2.47
RA_01SB	0.006	1.28	1.5	8.9	0.017	0.8	2.1	1.62
RA_02SB	0.006	1.89	1.5	10.3	0.017	1.1	2.5	1.78
RA_03SB	0.012	3.28	1.5	11.1	0.017	1.2	2.7	2.65
RA_06	0.010	0.40	1.5	5.2	0.017	0.3	1.3	1.46
RA_07	0.005	1.10	1.5	8.7	0.017	0.8	2.1	1.45
RA_08	0.017	1.19	1.5	7.1	0.017	0.5	1.7	2.34

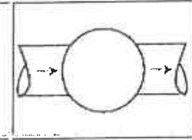
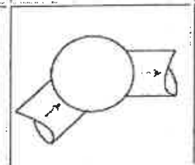
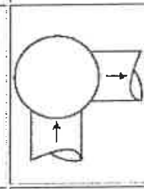
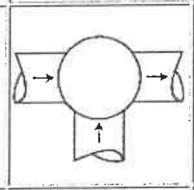
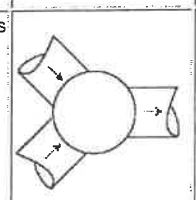
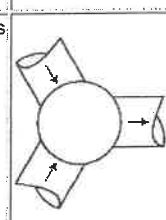
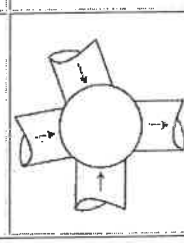
D.3. StormCAD Profiles

Headloss Coefficients for Junctions

< >

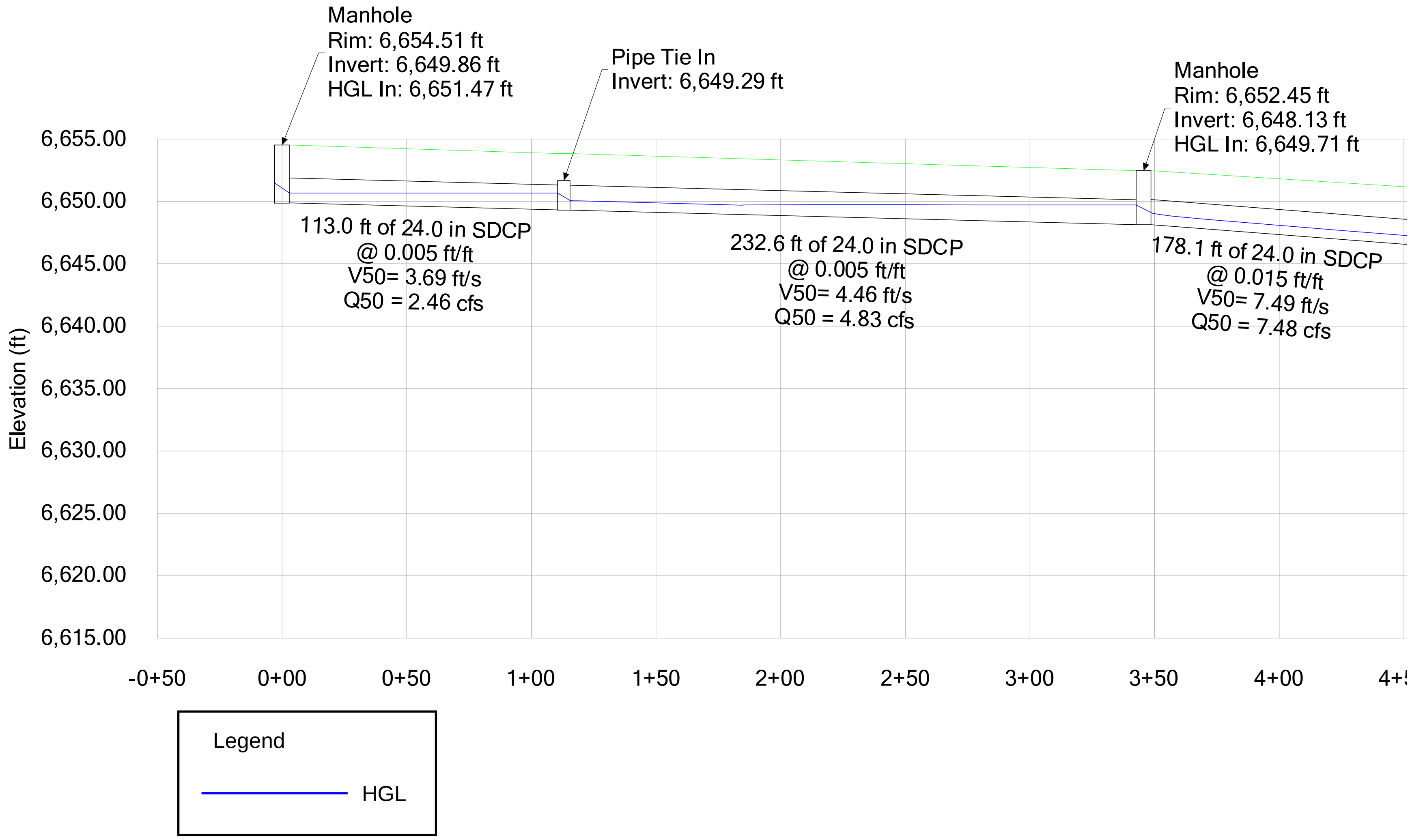
These are typical headloss coefficients used in the standard method for estimating headloss through manholes and junctions.

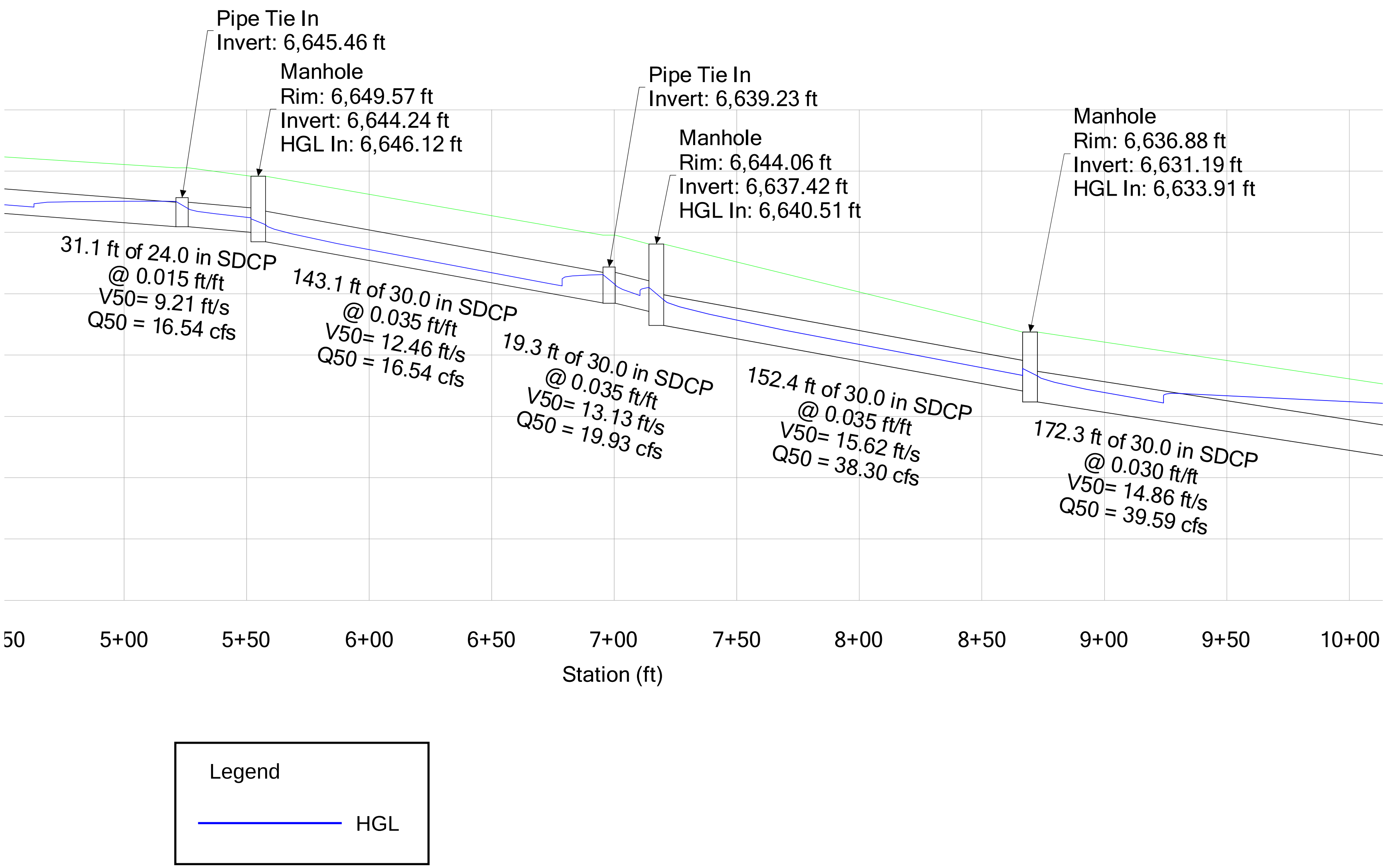
Table B-3: Typical Headloss Coefficients

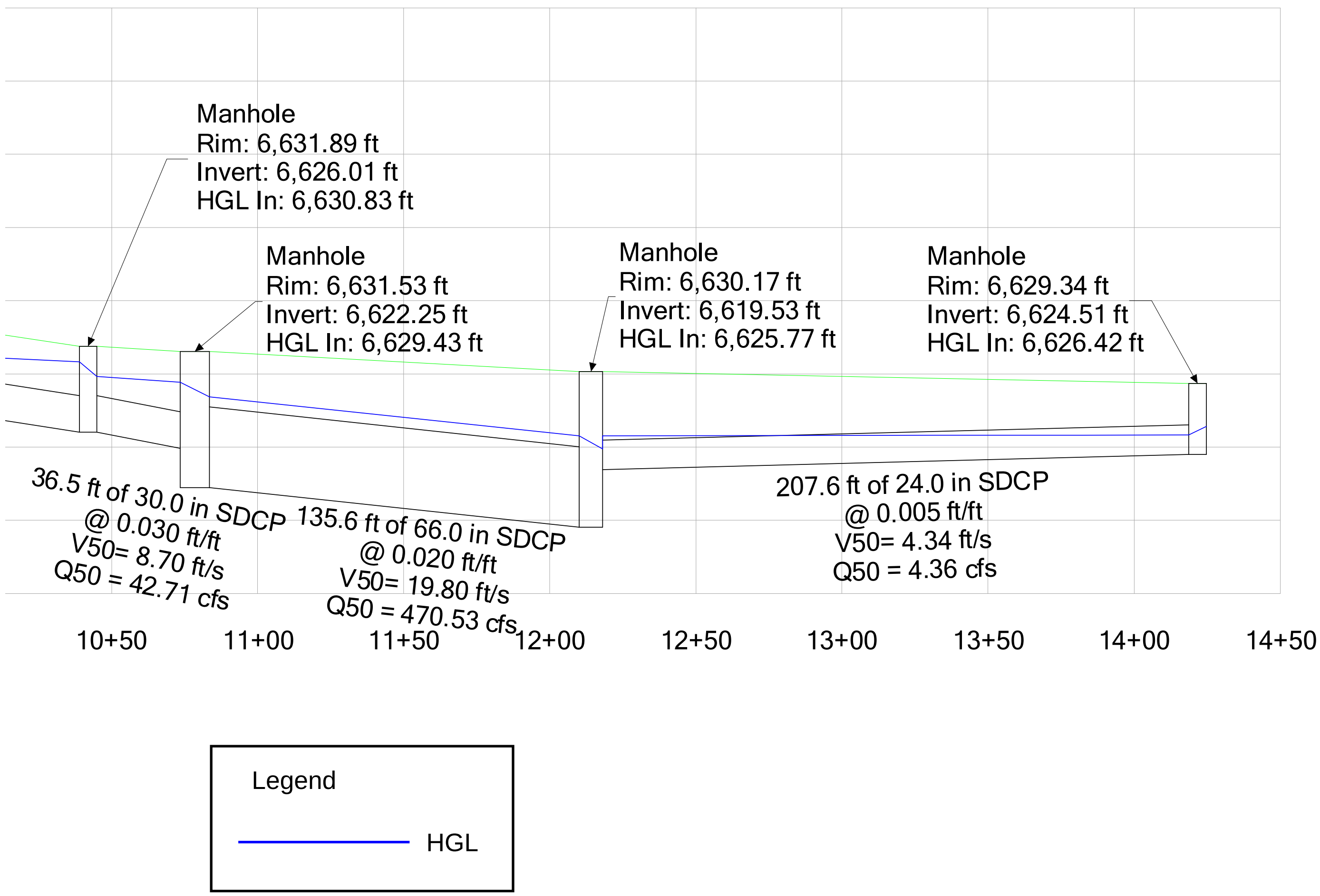
Type of Manhole	Diagram	Headloss Coefficient
Trunkline only with no bend at the junction		0.5 A
Trunkline only with 45° bend at the junction		0.6 B
Trunkline only with 90° bend at the junction		0.8 C
Trunkline with one lateral		Small 0.6 Large 0.7 D
Two roughly equivalent entrance lines with angle < 90° between lines		0.8 E ✓
Two roughly equivalent entrance lines with angle > 90° between lines		0.9 F
Three or more entrance lines		1.0 G

Profile Report

Engineering Profile - North Richards (30%_StormCAD.stsw)

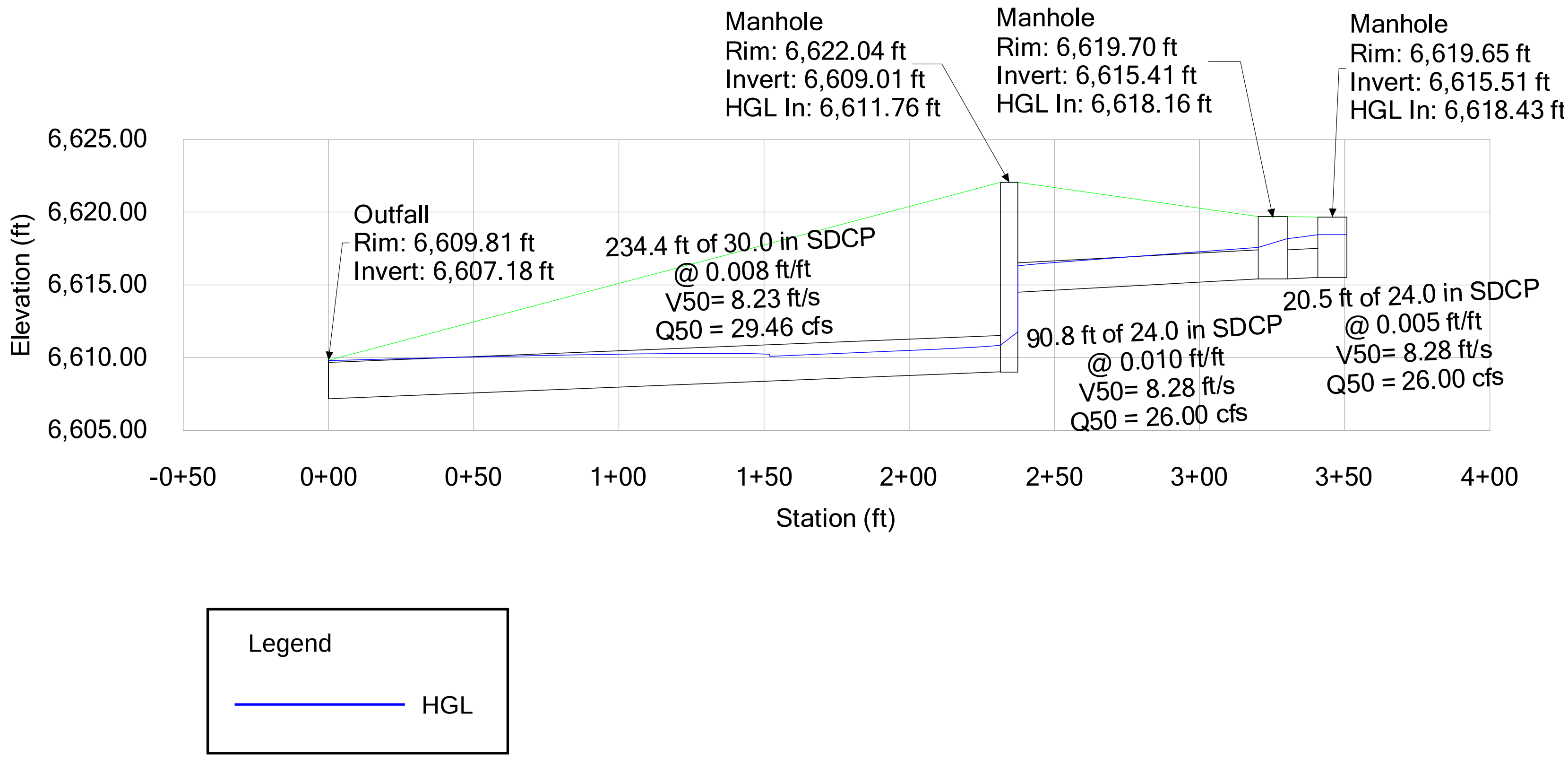



Pipe Tie In
Invert: 6,645.46 ft
Manhole
Rim: 6,649.57 ft
Invert: 6,644.24 ft
HGL In: 6,646.12 ft
Pipe Tie In
Invert: 6,639.23 ft
Manhole
Rim: 6,644.06 ft
Invert: 6,637.42 ft
HGL In: 6,640.51 ft
Manhole
Rim: 6,636.88 ft
Invert: 6,631.19 ft
HGL In: 6,633.91 ft
31.1 ft of 24.0 in SDCP
@ 0.015 ft/ft
V50= 9.21 ft/s
Q50 = 16.54 cfs
143.1 ft of 30.0 in SDCP
@ 0.035 ft/ft
V50= 12.46 ft/s
Q50 = 16.54 cfs
19.3 ft of 30.0 in SDCP
@ 0.035 ft/ft
V50= 13.13 ft/s
Q50 = 19.93 cfs
152.4 ft of 30.0 in SDCP
@ 0.035 ft/ft
V50= 15.62 ft/s
Q50 = 38.30 cfs
172.3 ft of 30.0 in SDCP
@ 0.030 ft/ft
V50= 14.86 ft/s
Q50 = 39.59 cfs
50
5+00
5+50
6+00
6+50
7+00
7+50
8+00
8+50
9+00
9+50
10+00
Station (ft)
Legend
HGL


Manhole
Rim: 6,631.89 ft
Invert: 6,626.01 ft
HGL In: 6,630.83 ft
Manhole
Rim: 6,631.53 ft
Invert: 6,622.25 ft
HGL In: 6,629.43 ft
Manhole
Rim: 6,630.17 ft
Invert: 6,619.53 ft
HGL In: 6,625.77 ft
Manhole
Rim: 6,629.34 ft
Invert: 6,624.51 ft
HGL In: 6,626.42 ft
36.5 ft of 30.0 in SDCP
@ 0.030 ft/ft
V50= 8.70 ft/s
Q50 = 42.71 cfs
135.6 ft of 66.0 in SDCP
@ 0.020 ft/ft
V50= 19.80 ft/s
Q50 = 470.53 cfs
207.6 ft of 24.0 in SDCP
@ 0.005 ft/ft
V50= 4.34 ft/s
Q50 = 4.36 cfs
10+50
11+00
11+50
12+00
12+50
13+00
13+50
14+00
14+50
Legend
HGL

Profile Report

Engineering Profile - South Side Trunkline (30%_StormCAD.stsw)



APPENDIX

E SRH-2D

E.1.SMS MATERIALS COVERAGE

E.2.WATER SURFACE ELEVATION PLOTS

E.3.WATER SURFACE ELECATION CROSS SECTIONS

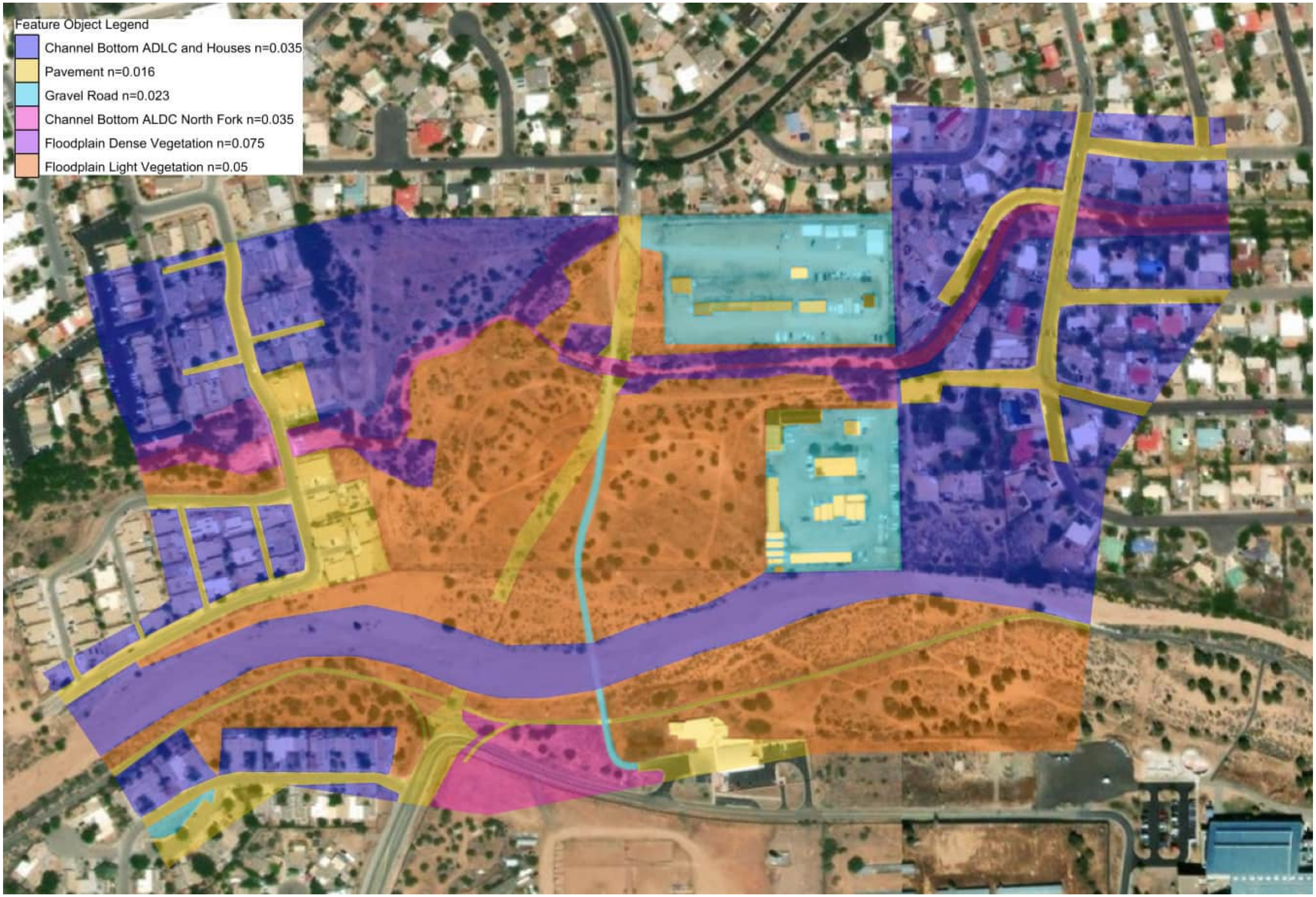
E.4.ARROYO DE LOS CHAMISOS SCOUR

E.5.ARROYO DE LOS PINOS SCOUR

E.6.EXCEPRTS FROM GEOTECHNICAL REPORT PREPARED BY YEDOME CONSULTANTS LLC

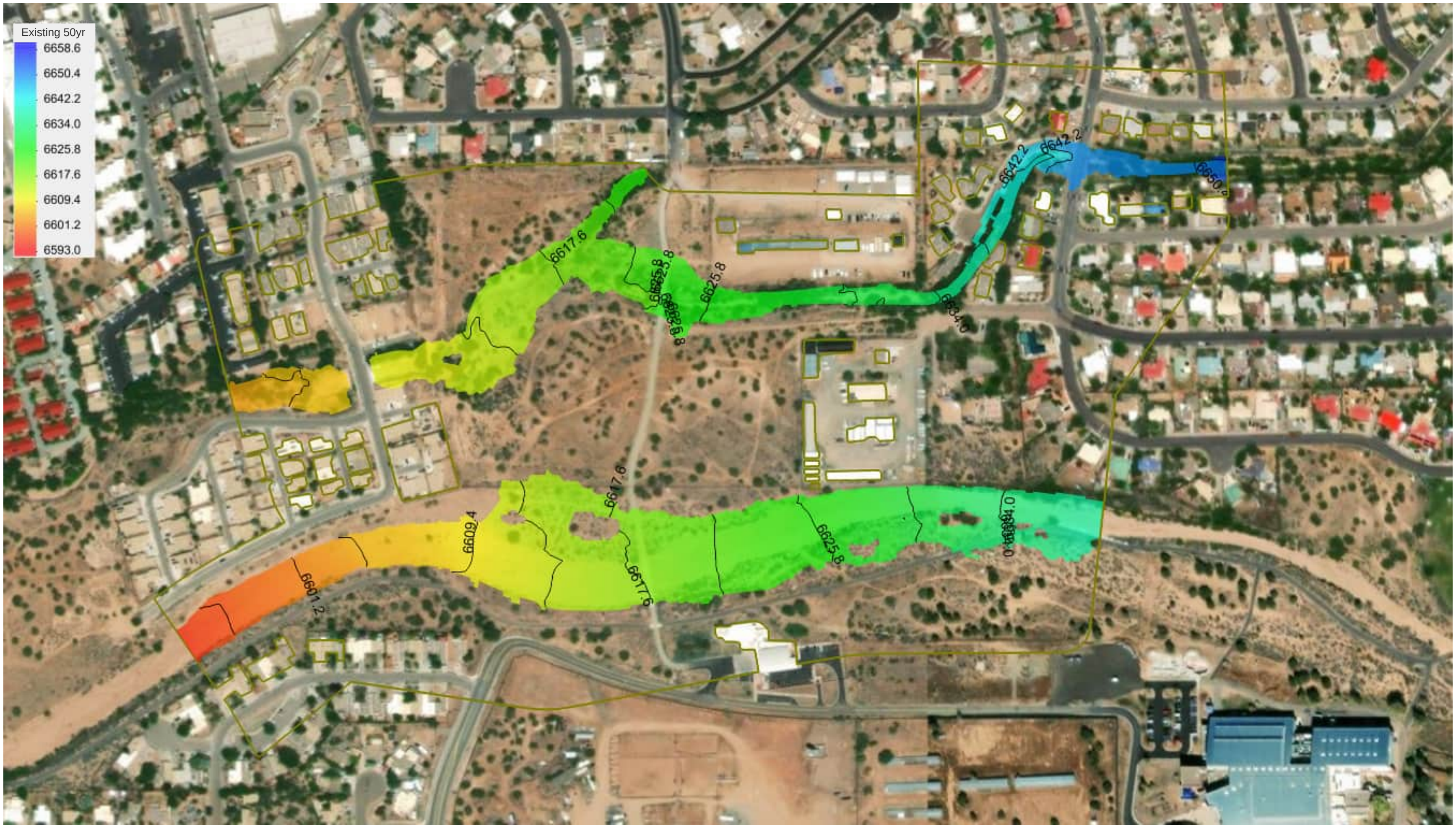
E.1. SMS Materials Coverage

Proposed Materials Coverage

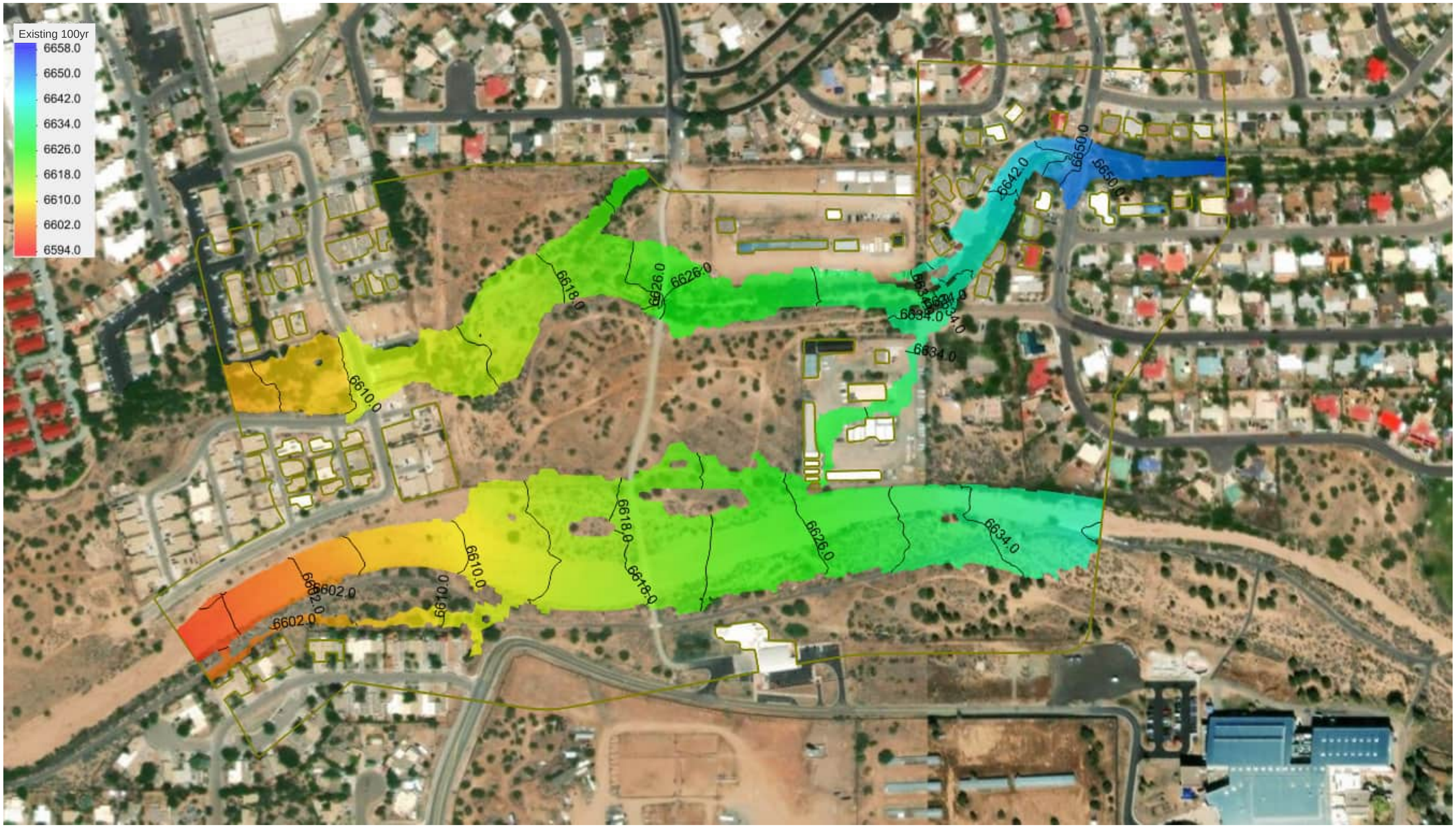


E.2. Water Surface Elevation Plots

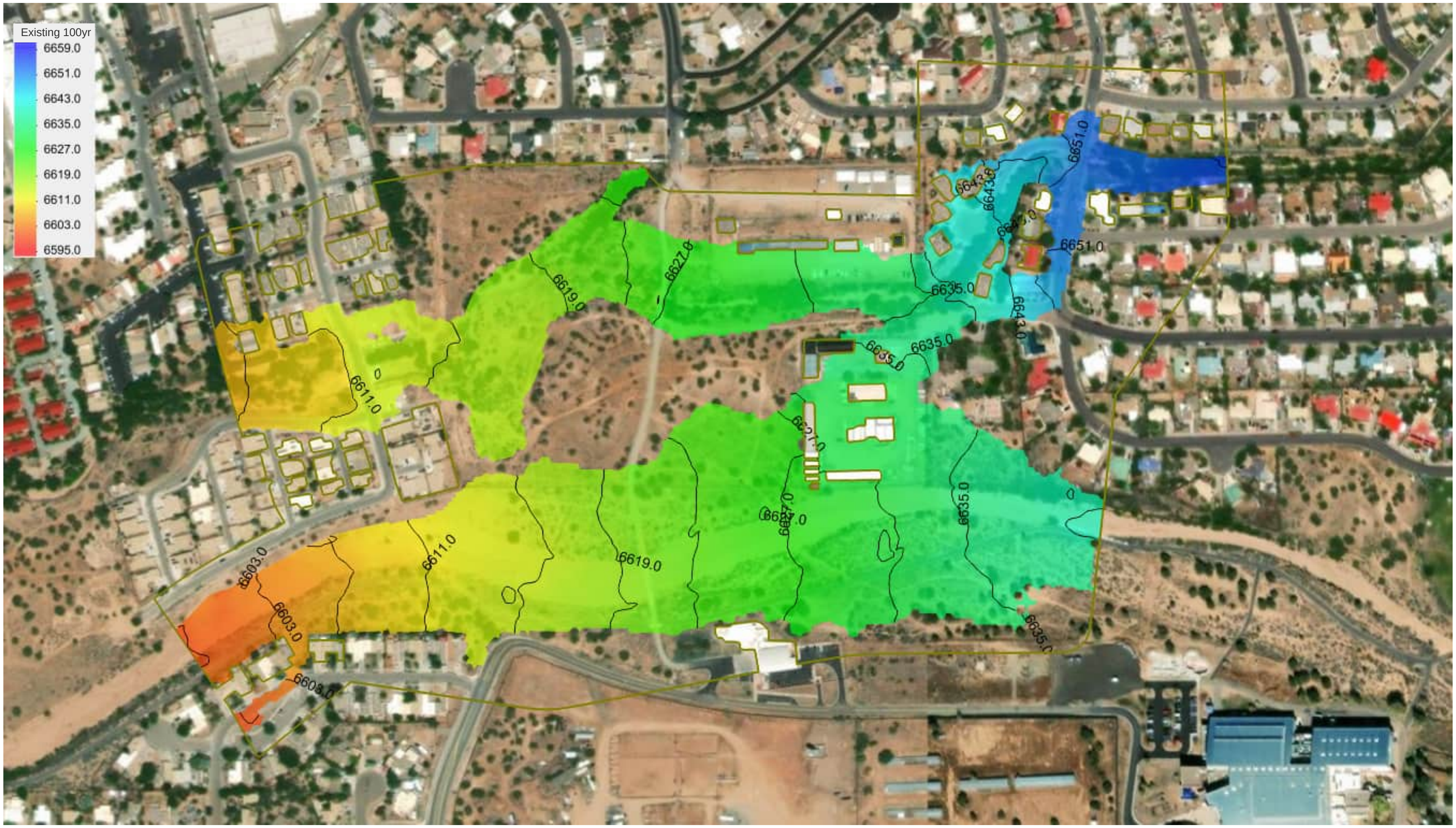
Existing 50-yr WSE



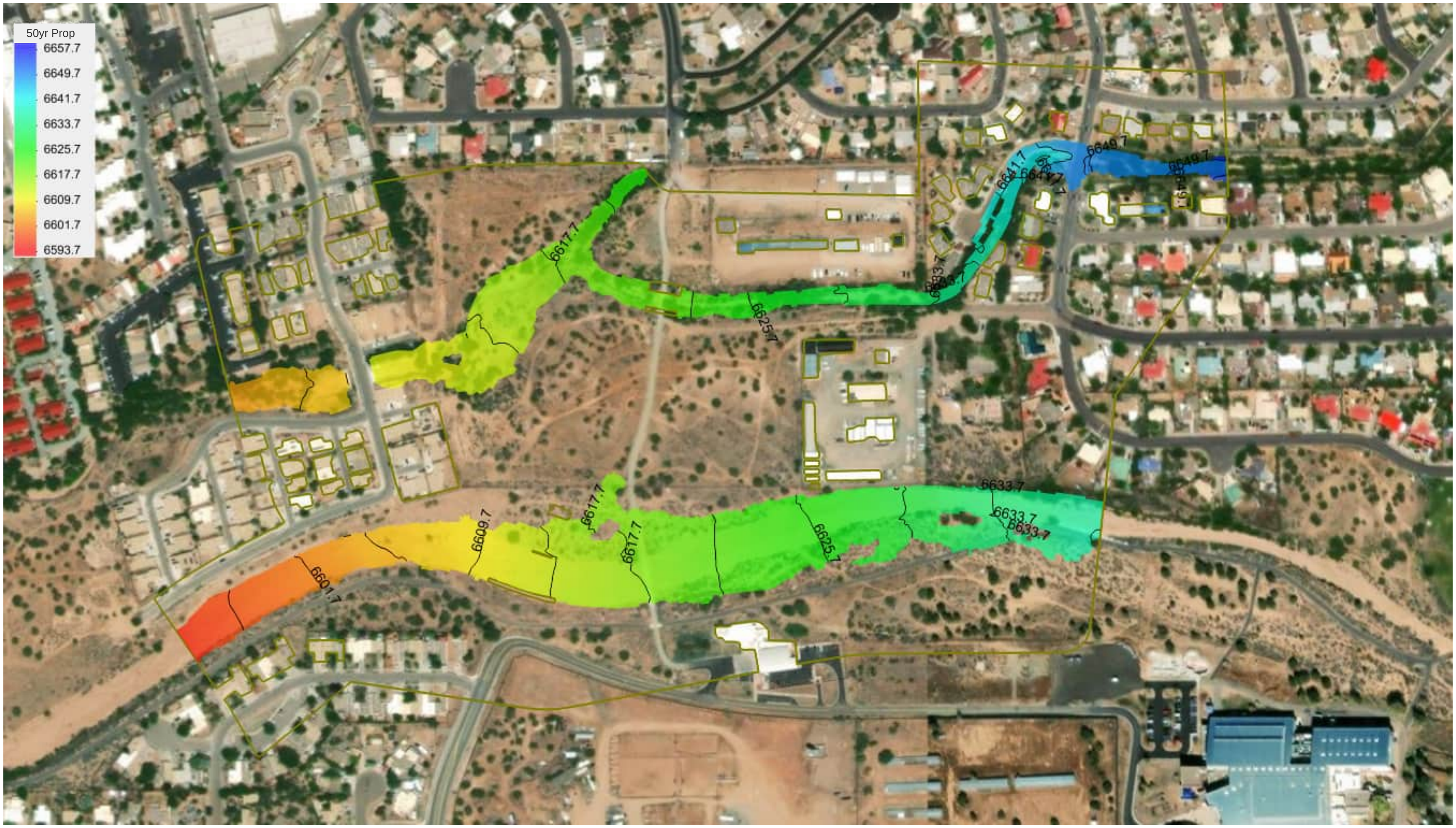
Existing 100-yr WSE



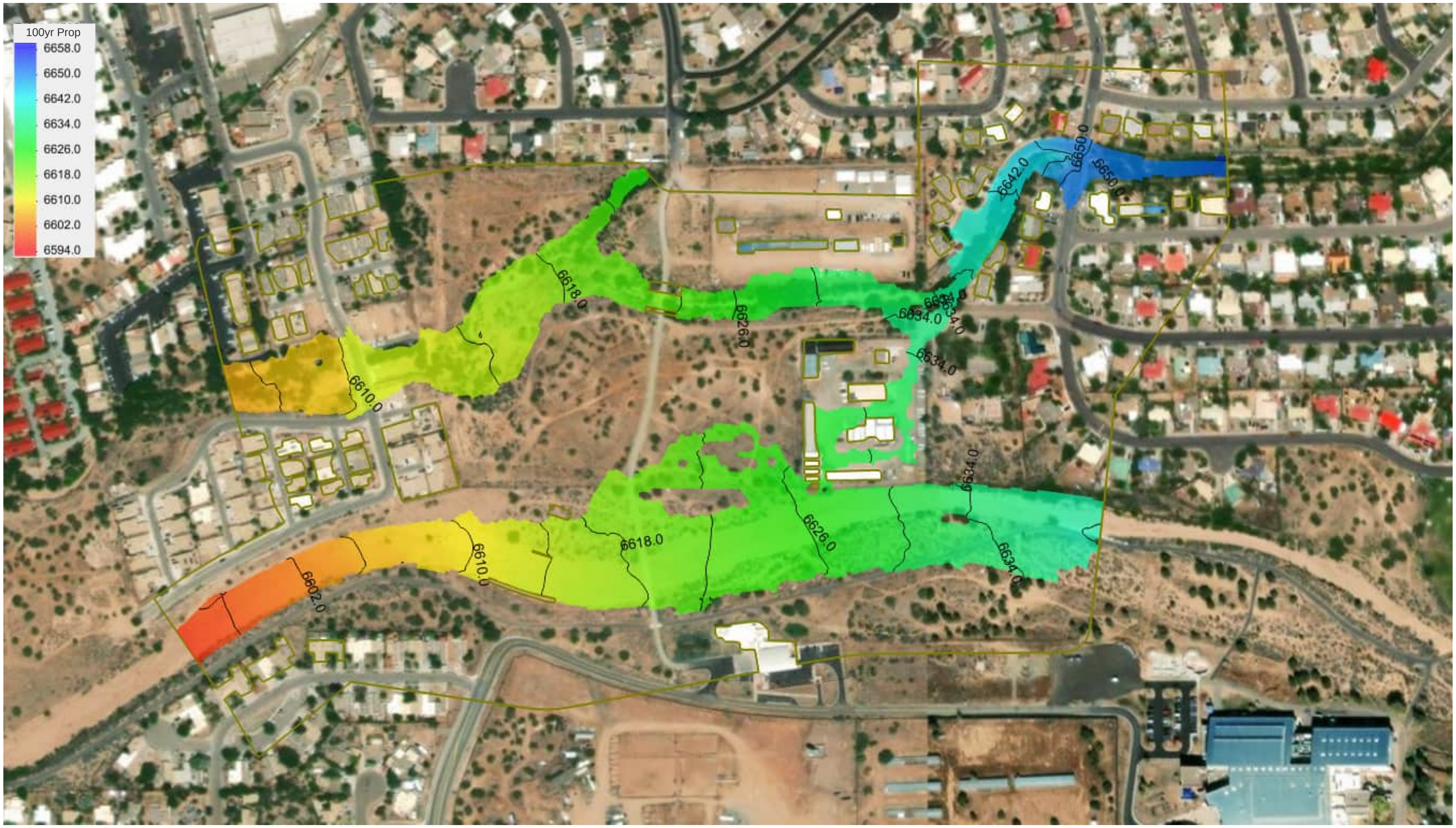
Existing 500-yr WSE



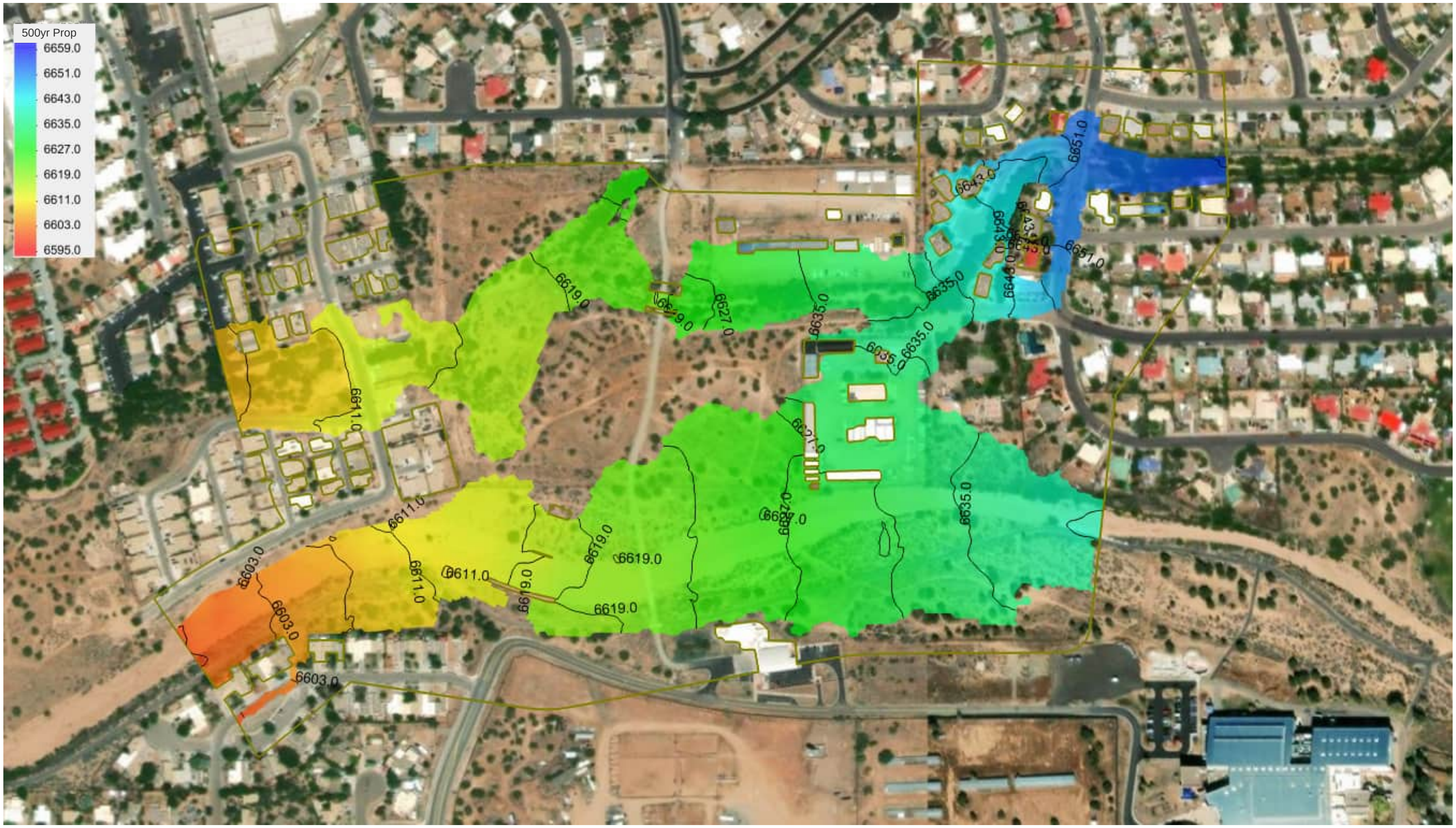
Proposed 50-yr WSE



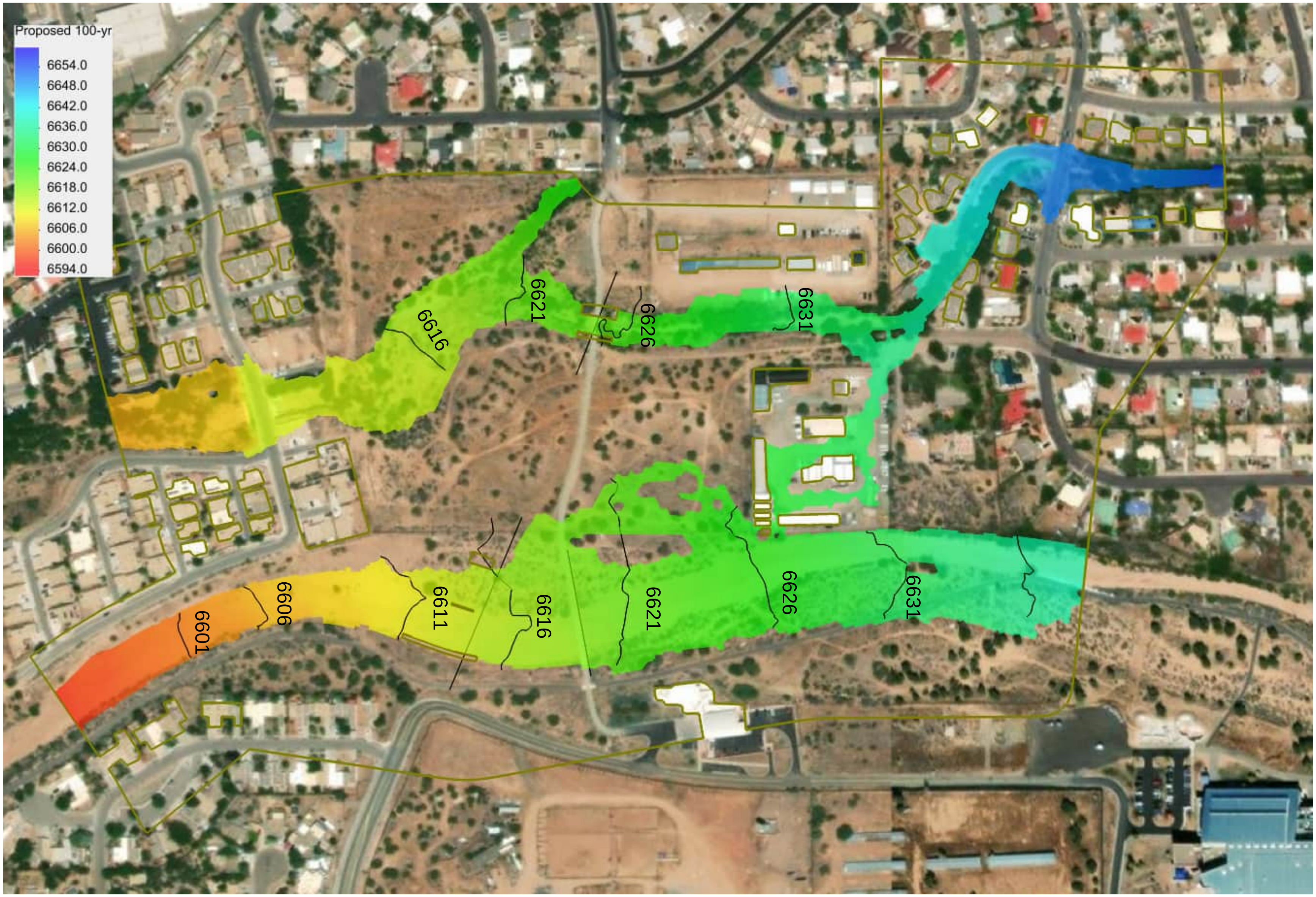
Proposed 100-yr WSE



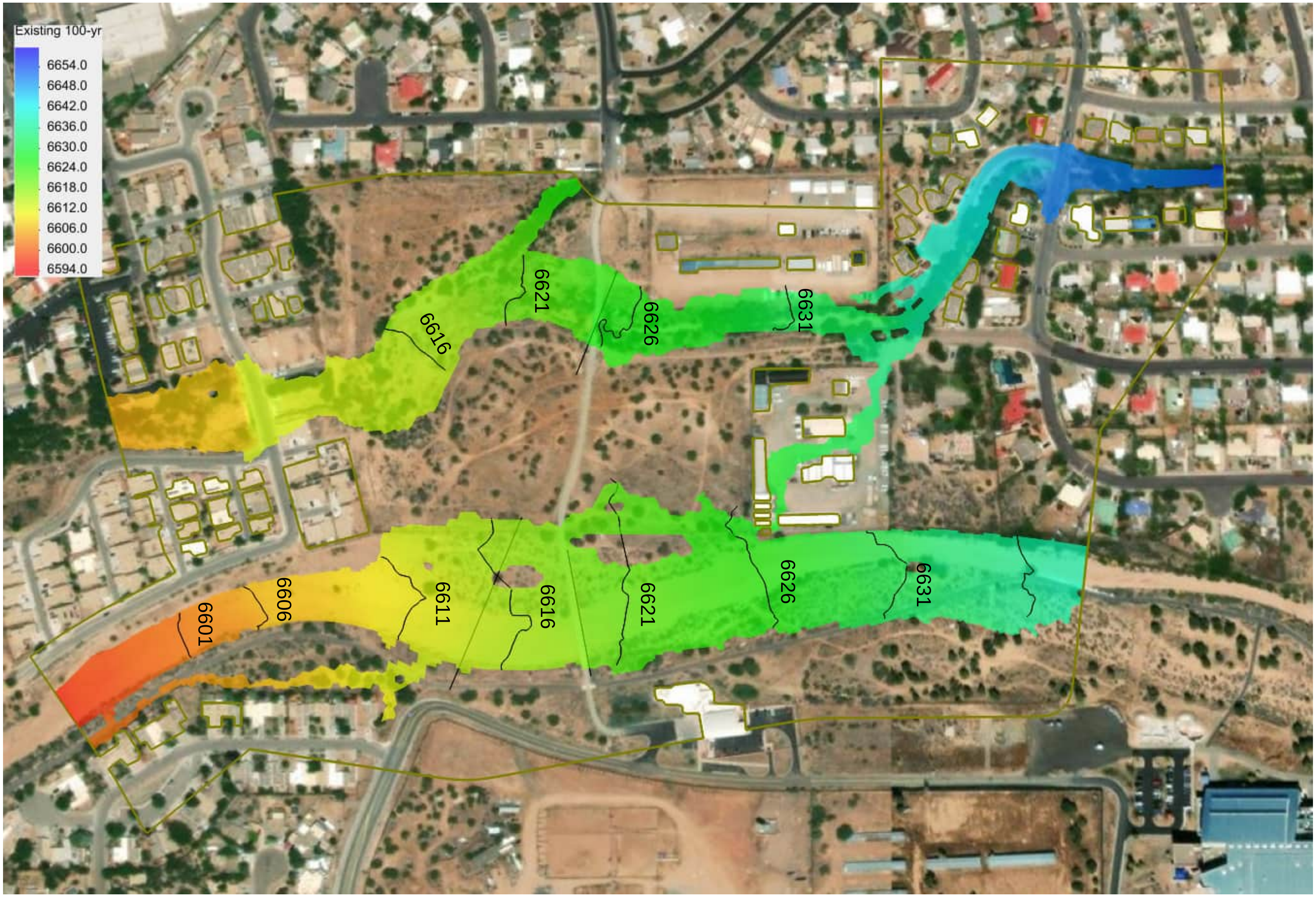
Proposed 500-yr WSE



Proposed 100-yr WSE and FEMA BFE



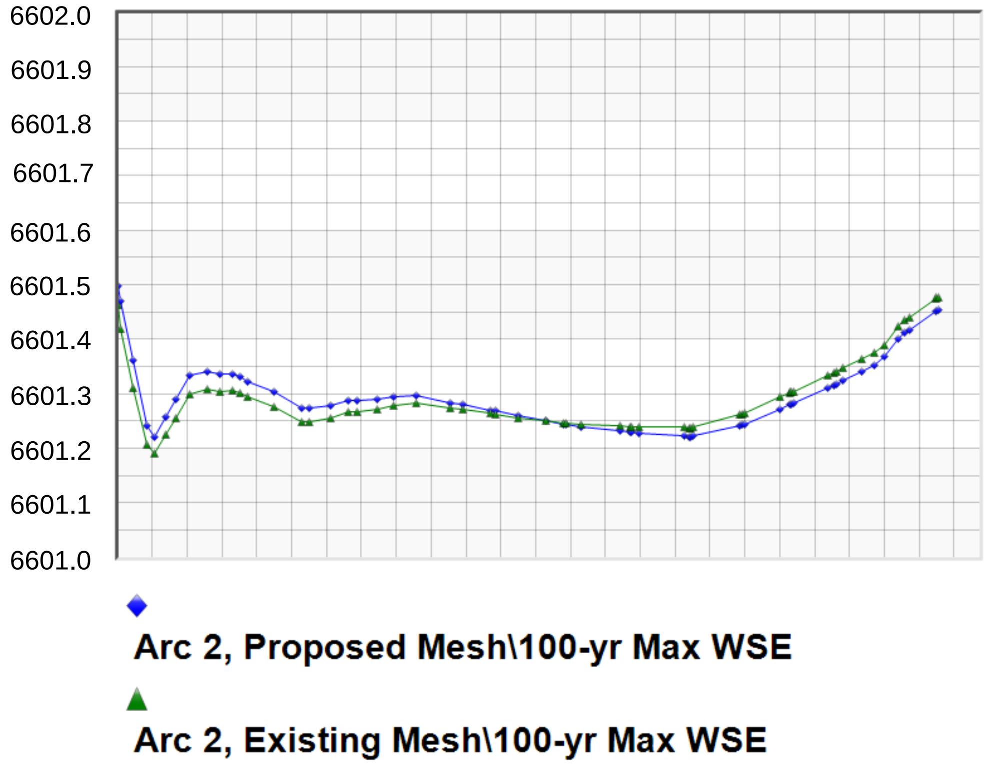
Existing 100-yr WSE and FEMA BFE



E.3. Water Surface Elevation Cross Sections

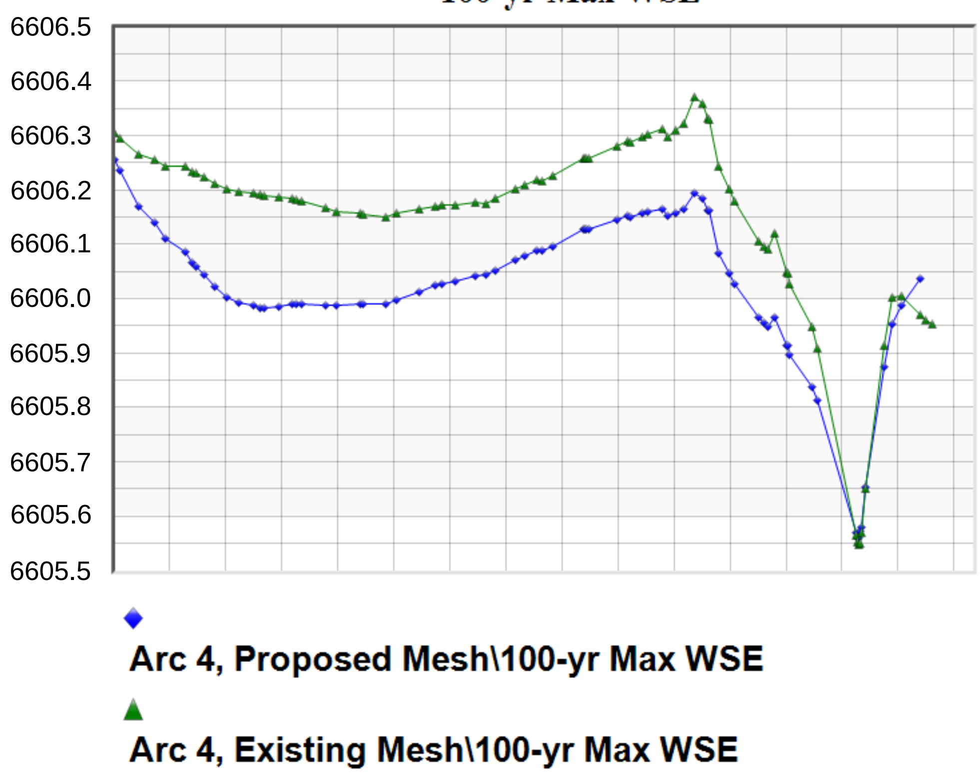
Arroyo De Los Chamisos BFE 6601

100-yr Max WSE



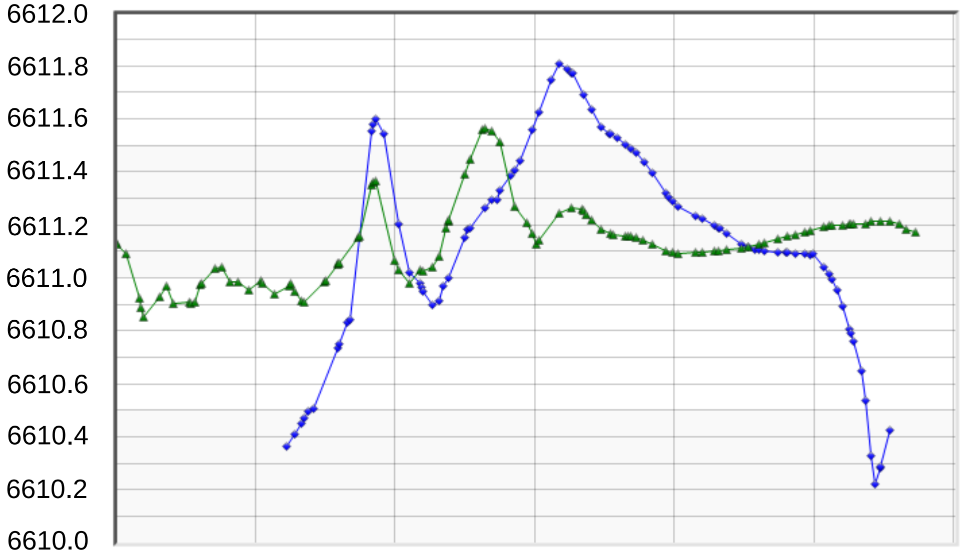
Arroyo De Los Chamisos BFE 6606

100-yr Max WSE



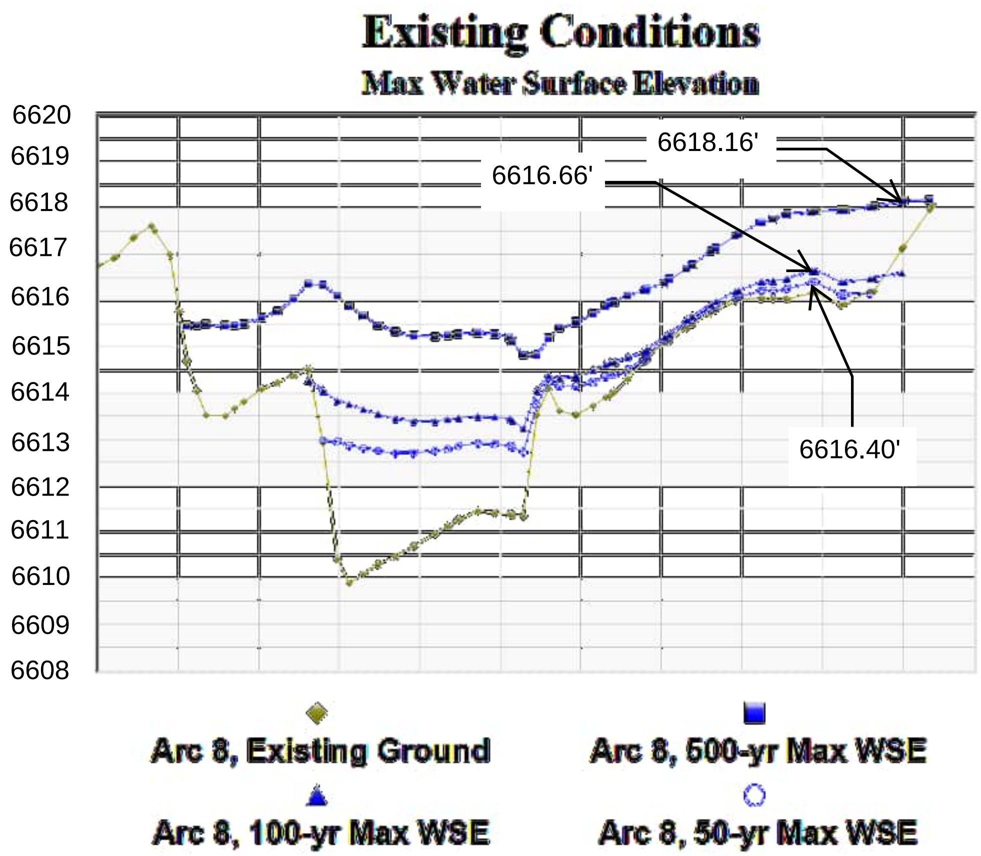
Arroyo De Los Chamisos BFE 6611

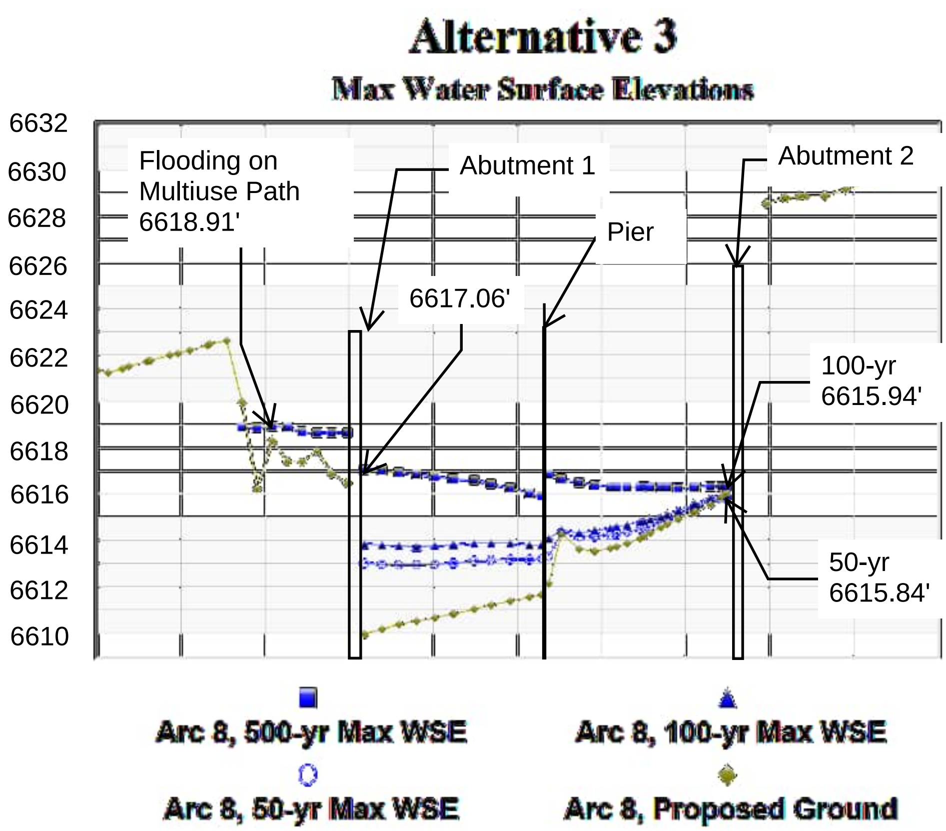
100-yr Max WSE

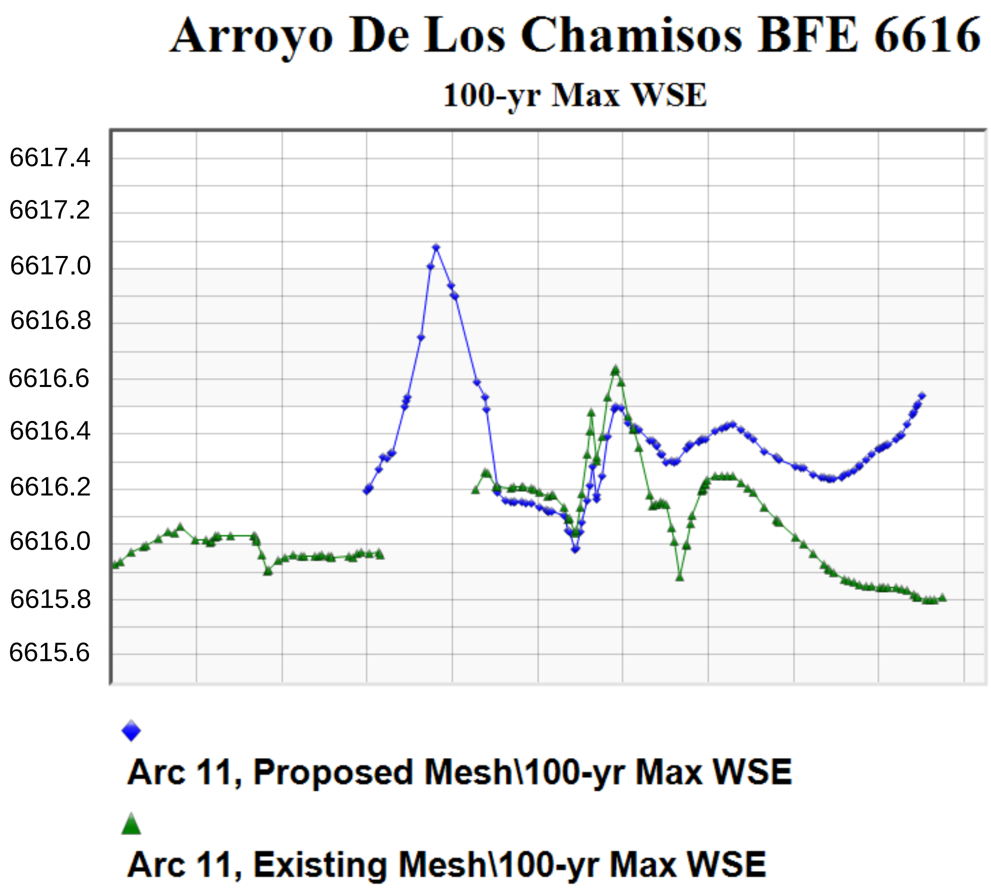


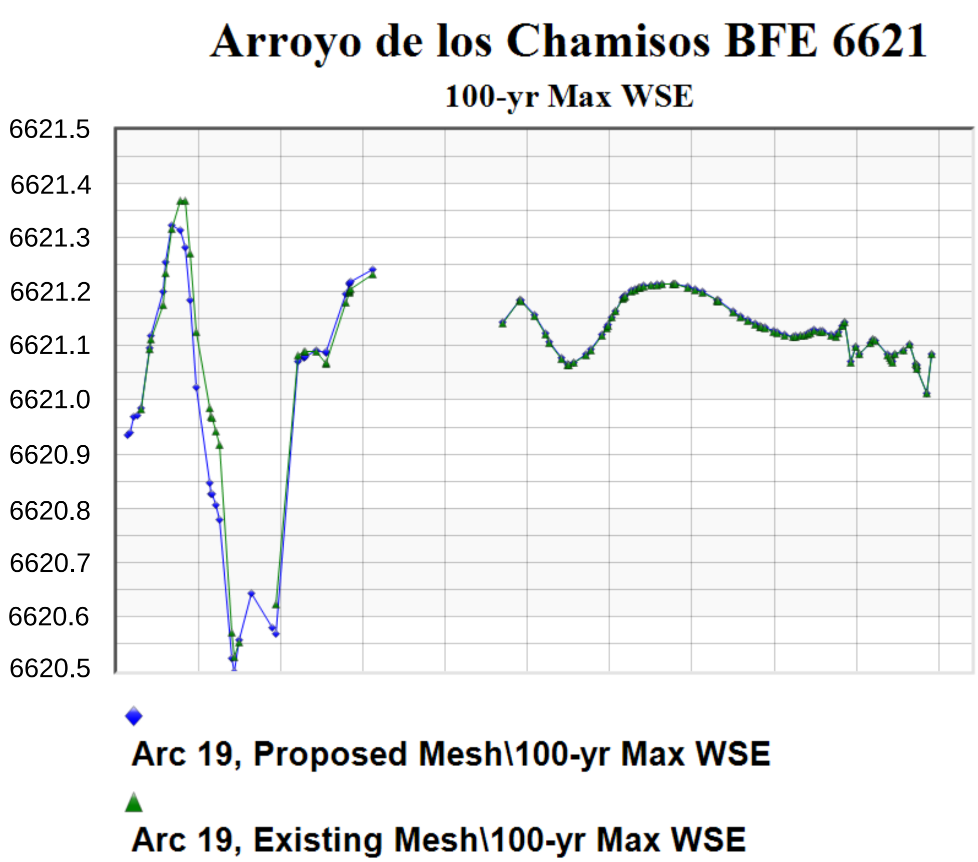
Arc 8, Proposed Mesh\100-yr Max WSE

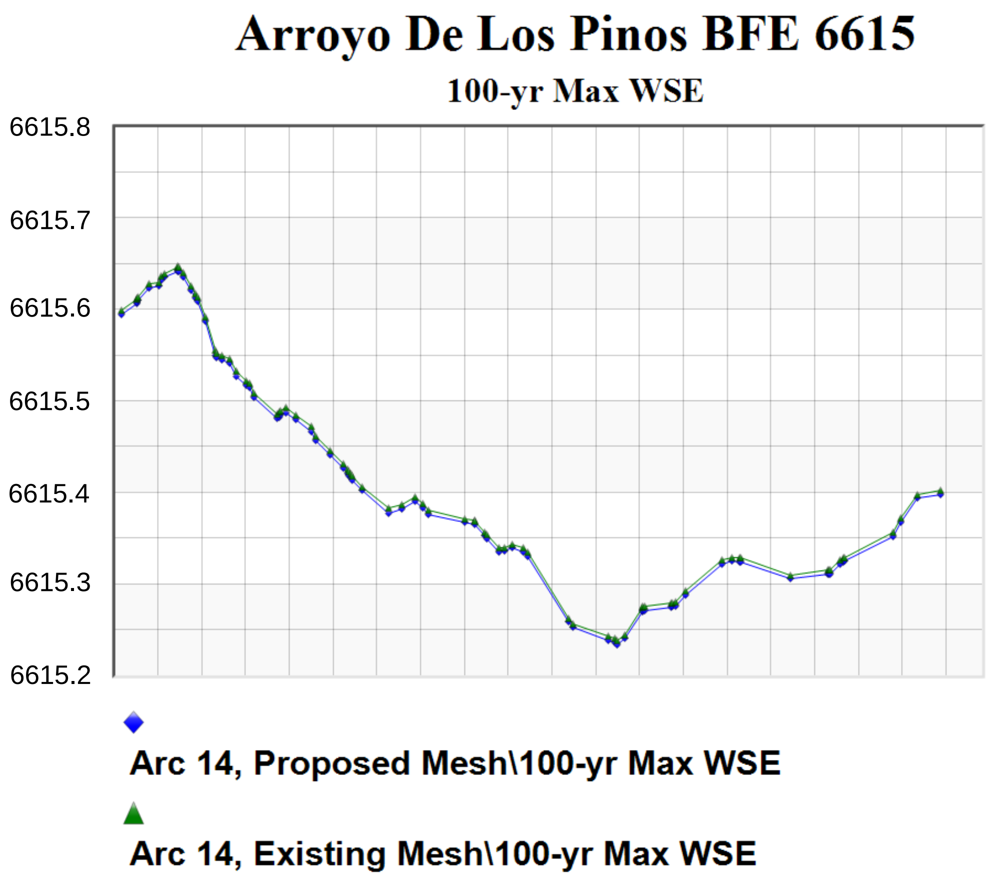
Arc 8, Existing Mesh\100-yr Max WSE

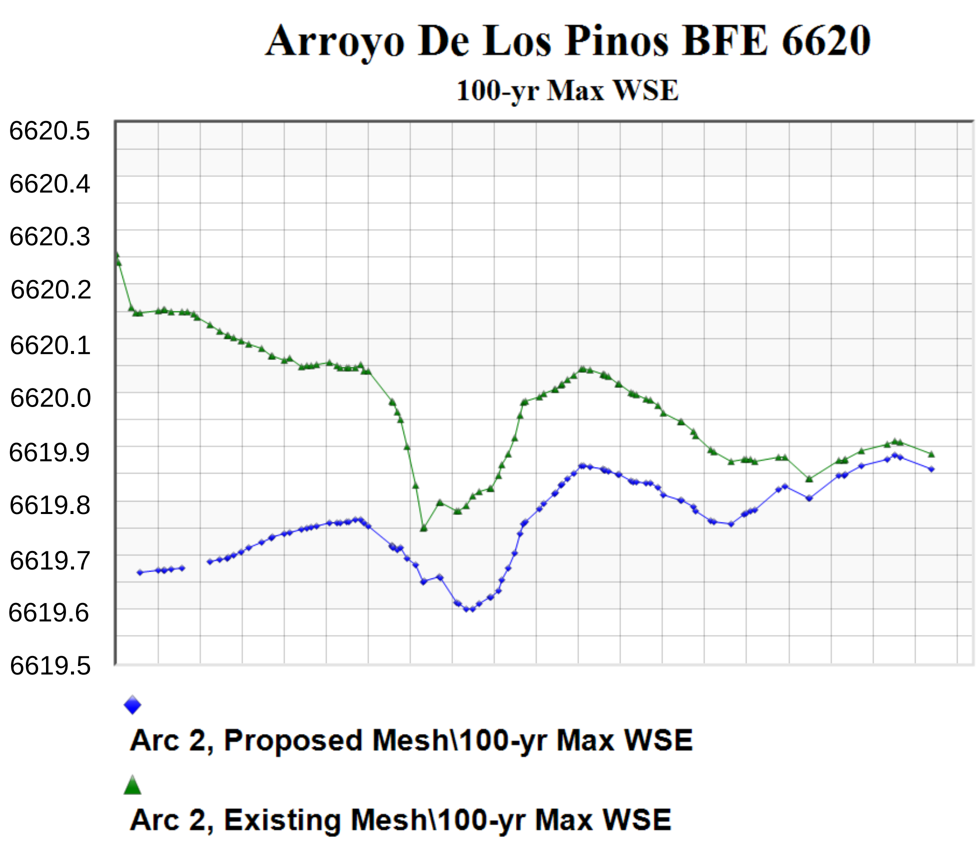

Existing Conditions
Max Water Surface Elevation
6620
6619
6618
6617
6616
6615
6614
6613
6612
6611
6610
6609
6608
6618.16'
6616.66'
6616.40'
Arc 8, Existing Ground
Arc 8, 500-yr Max WSE
Arc 8, 100-yr Max WSE
Arc 8, 50-yr Max WSE

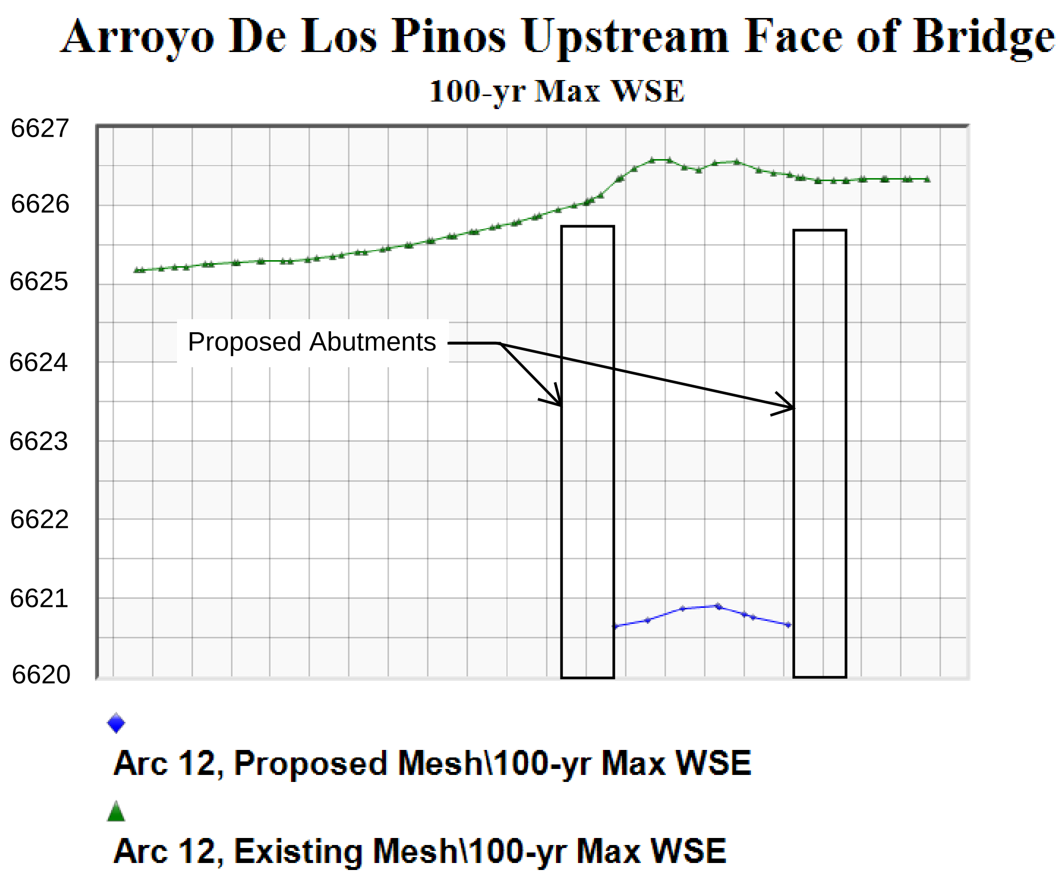

Alternative 3
Max Water Surface Elevations
6632
6630
6628
6626
6624
6622
6620
6618
6616
6614
6612
6610
Flooding on
Multiuse Path
6618.91'
Abutment 1
Abutment 2
Pier
6617.06'
100-yr
6615.94'
50-yr
6615.84'
Arc 8, 500-yr Max WSE
Arc 8, 100-yr Max WSE
Arc 8, 50-yr Max WSE
Arc 8, Proposed Ground

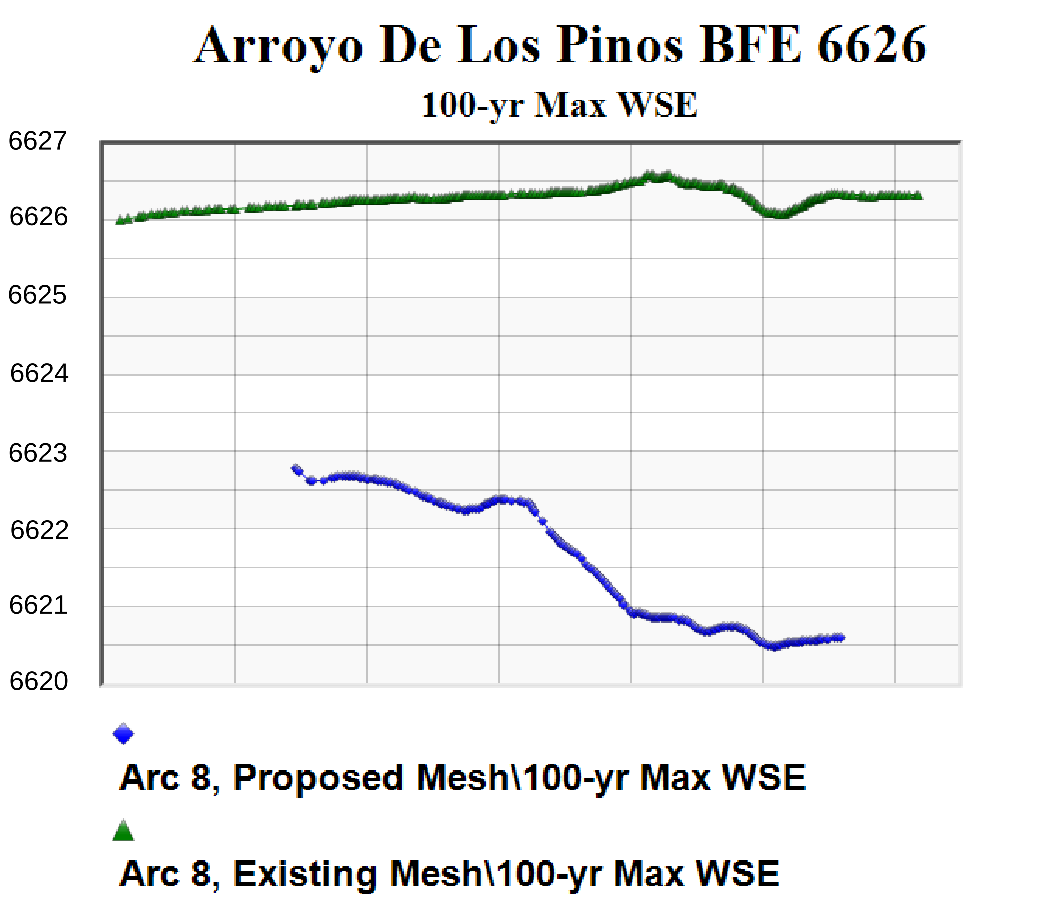

Arroyo De Los Chamisos BFE 6616
100-yr Max WSE
6617.4
6617.2
6617.0
6616.8
6616.6
6616.4
6616.2
6616.0
6615.8
6615.6
Arc 11, Proposed Mesh\100-yr Max WSE
Arc 11, Existing Mesh\100-yr Max WSE


Arroyo de los Chamisos BFE 6621
100-yr Max WSE
6621.5
6621.4
6621.3
6621.2
6621.1
6621.0
6620.9
6620.8
6620.7
6620.6
6620.5
Arc 19, Proposed Mesh\100-yr Max WSE
Arc 19, Existing Mesh\100-yr Max WSE


Arroyo De Los Pinos BFE 6615
100-yr Max WSE
6615.8
6615.7
6615.6
6615.5
6615.4
6615.3
6615.2
Arc 14, Proposed Mesh\100-yr Max WSE
Arc 14, Existing Mesh\100-yr Max WSE

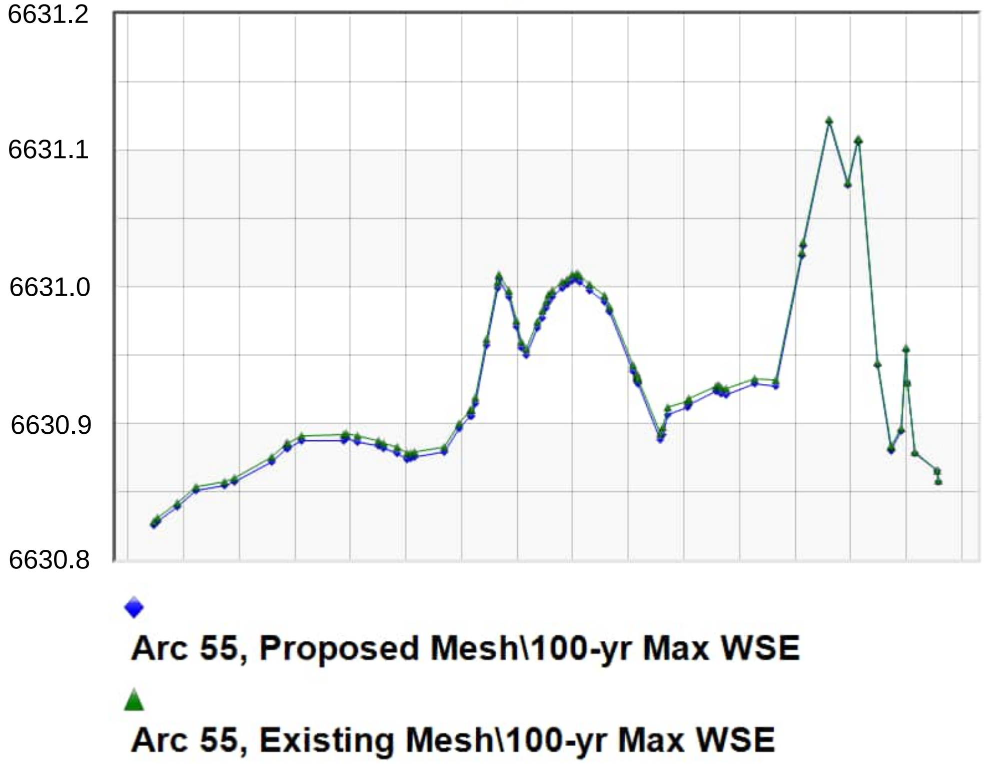

Arroyo De Los Pinos BFE 6620
100-yr Max WSE
6620.5
6620.4
6620.3
6620.2
6620.1
6620.0
6619.9
6619.8
6619.7
6619.6
6619.5
Arc 2, Proposed Mesh\100-yr Max WSE
Arc 2, Existing Mesh\100-yr Max WSE


Arroyo De Los Pinos Upstream Face of Bridge
100-yr Max WSE
6627
6626
6625
6624
6623
6622
6621
6620
Proposed Abutments
Arc 12, Proposed Mesh\100-yr Max WSE
Arc 12, Existing Mesh\100-yr Max WSE


Arroyo De Los Pinos BFE 6626
100-yr Max WSE
6627
6626
6625
6624
6623
6622
6621
6620
Arc 8, Proposed Mesh\100-yr Max WSE
Arc 8, Existing Mesh\100-yr Max WSE

Arroyo De Los Pinos BFE 6631

100-yr Max WSE



E.4. Arroyo de Los Chamisos Scour

Arroyo De Los Chamisos Bridge Scour Summary				
Scenario	500-yr	100-yr		
Bridge Geometry				
Bridge Cross-Section				
WSE				
Contraction Scour				
Applied Contraction Scour Depth	0.07	**0.23**	ft	
Live Bed Contraction Scour Depth	0.07	**0.23**	ft	
Applied Contraction Scour Elevation with LTD	**6609.29**	6609.29	ft	
Approach Cross-Section				
Local Scour at Piers				
Plot Pier Scour				
Piers				
Pier Name	Pier 1	Pier 1		
Pier Scour Depth	**37.62**	31.07	ft	Computation Method: HEC-18
Total Scour at Pier	**37.62**	31.07	ft	
Total Scour Elevation at Pier	**6571.66**	6578.05	ft	
Local Scour at Abutments				
Left Abutment				
Plot Left Abutment Scour				
Abutment Scour Depth	-2.22	-1.51	ft	NCHRP Method: Scour Condition A (includes LTD)
Total Scour at Abutment	**0**	0	ft	
Total Scour Elevation at Abutment	6612.91	**6611.97**	ft	
Right Abutment				
Abutment Scour Depth	**1.11**	-1.5	ft	NCHRP Method: Scour Condition A (includes LTD)
Max Flow Depth including Abutment Scour	1.61	2.42	ft	Including the long-term scour depth
Total Scour at Abutment	**1.11**	0	ft	
Local Streambed Elevation at Abutment	6615.5	6615.5	ft	
Total Scour Elevation at Abutment	**6614.39**	6617	ft	

Arroyo De Los Chamisos 100-yr Scour Parameters

Contraction Scour

Computation Method: Clear-Water or Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	2.77	ft	
D50	0.418490	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream	10.31	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of siz...	1.47	ft/s	
Contraction Scour Condition	Live Bed		
Live Bed Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.013536	ft/ft	
Discharge in Contracted Section	3455.15	cfs	
Discharge Upstream that is Transporting Sediment	2619.00	cfs	
Width in Contracted Section	175.44	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment	91.74	ft	
Depth Prior to Scour in Contracted Section	2.02	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results			
k1	0.690000		
Shear Velocity	1.10	ft/s	
Fall Velocity	0.18	ft/s	
Average Depth in Contracted Section after Scour	2.24	ft	
Scour Depth	0.23	ft	Negative values imply 'zero' scour depth
Scour may be limited by armoring. Comput...			

Pier Scour

Computation Method: HEC-18

Parameter	Value	Units	Notes
Input Parameters			
Pier Shape	Round Nose		
Bed Condition	Clear-Water Scour		Dune Height is N/A
Depth Upstream of Pier	3.31	ft	
Velocity Upstream of Pier	12.31	ft/s	
Width of Pier	4.00	ft	width for the zero skew condition
Length of Pier	57.43	ft	
Angle of Attack	29.98	Degrees	
Results			
Froude Number Upstream	1.19		
Correction Factor for Pier Nose Shape (K1)	1.00		
Correction Factor of Angle of Attack (K2)	3.50		
Pier Length to Pier Width (L/a)	12.00		If L/a > 12, use 12
Correction Factor for Bed Condition (K3)	1.10		
Scour Depth	31.07	ft	
Scour Hole			
Angle of Repose	44.00	degrees	
Use the Pier Width as the Bottom Width of Scour ...	☑		
Scour Hole Bottom Width	4.00	ft	
Scour Hole Top Width	63.25	ft	

Left Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Ch...		
Abutment Type (k1)	Vertical-wall abu...		
Unit Discharge, Upstream in Main Channel (q1)	28.55	cfs/ft	
Unit Discharge in Constricted Area (q2)	19.69	cfs/ft	
D50 (D50)	0.418628	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	2.77	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	3.92	ft	Depth at Abutment Toe
Results			
q2 / q1	0.69		
Average Velocity Upstream (V)	10.31	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	1.47	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	2.01	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	2.42	ft	Including the long-term scour depth
Scour Hole Depth (ys)	-1.51	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth

Right Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type (k1)	Spill-through abutment		
Unit Discharge, Upstream in Main Channel (q1)	28.55	cfs/ft	
Unit Discharge in Constricted Area (q2)	19.69	cfs/ft	
D50 (D50)	0.418628	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	2.77	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	3.92	ft	Depth at Abutment Toe
Results			
q2 / q1	0.69		
Average Velocity Upstream (V)	10.31	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	1.47	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	2.01	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	2.42	ft	Including the long-term scour depth
Scour Hole Depth (ys)	-1.50	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth

Arroyo De Los Chamisos 500-yr Scour Parameters

Contraction Scour

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Pressure Scour Method			
Compute Pressure Scour (Vertical Contraction Scour)	☐		
Input Parameters			
Input Parameters for Scour Condition			
Average Depth Upstream of Contraction (y1)	4.41	ft	(hu) in pressure scour
D50 (D50)	0.418628	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream (V)	14.02	ft/s	
Computed Contraction Scour Condition	Live Bed		Vc < V
Input Parameters for Live Bed			
Temperature of Water	60.00	ºF	Used to determine the fall velocity
Slope of Energy Grade Line at Approach Section (S1)	0.012763	ft/ft	
Discharge in Contracted Section (Q2)	9739.11	cfs	
Discharge Upstream that is Transporting Sediment (Q1)	5673.18	cfs	
Bottom Width in Contracted Section (W2)	178.86	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment (W1)	91.74	ft	
Depth Prior to Scour in Contracted Section (y0)	4.36	ft	
Unit Weight of Water (gamma w)	62.40	lb/ft^3	
Unit Weight of Sediment (gamma s)	165.00	lb/ft^3	
Results			
Results of Scour Condition			
Critical velocity above which bed material of size D and smaller will be transported (Vc)	1.59	ft/s	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the bed material (Dm)	0.523284	mm	
Average Depth in Contracted Section after Scour (y2)	23.57	ft	
Scour Depth (ys)	19.21	ft	Negative values imply 'zero' scour depth
Results of Live Bed Method			
k1 (k1)	0.690000		
Shear Velocity (V*)	1.35	ft/s	
Fall Velocity (T)	0.18	ft/s	
Average Depth in Contracted Section after Scour (y2)	4.42	ft	
Scour Depth (ys)	0.07	ft	Negative values imply 'zero' scour depth
Shear Applied to Bed by Live-Bed Scour (theta 0)	0.9037	lb/ft^2	
Shear Required for Movement of D50 Particle (Tau c)	0.0055	lb/ft^2	
Recommendations			
Recommended Scour Condition	Live Bed		Determined by comparing scour depths (including long-term degradation
Recommended Scour Depth	0.07	ft	Negative values imply 'zero' scour depth
Override Recommended Scour Condition	☐		

Pier Scour

Computation Method: HEC-18

Parameter	Value	Units	Notes
Input Parameters			
Pier Shape (K1)	Round Nose		
Bed Condition (K3)	Clear-Water Scour		Dune Height is N/A
Depth Upstream of Pier (y1)	6.60	ft	
Velocity Upstream of Pier (V1)	14.60	ft/s	
Width of Pier (a)	4.00	ft	width for the zero skew condition
Length of Pier (L)	57.43	ft	
Angle of Attack (K2)	31.56	Degr...	
Results			
Froude Number Upstream (Fr1)	1.00		
Correction Factor for Pier Nose Shape (K1)	1.00		
Correction Factor of Angle of Attack (K2)	3.59		
Pier Length to Pier Width (L/a)	12.00		If L/a > 12, use 12
Correction Factor for Bed Condition (K3)	1.10		
Scour Depth (ys)	37.62	ft	
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Use the Pier Width as the Bottom Width of Scour Hole	☑		
Scour Hole Bottom Width (K)	4.00	ft	
Scour Hole Top Width (W)	76.58	ft	

Left Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Ch...		
Abutment Type (k1)	Vertical-wall abu...		
Unit Discharge, Upstream in Main Channel (q1)	61.84	cfs/ft	
Unit Discharge in Constricted Area (q2)	54.45	cfs/ft	
D50 (D50)	0.418628	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	4.41	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	6.96	ft	Depth at Abutment Toe
Results			
q2 / q1	0.88		
Average Velocity Upstream (V)	14.02	ft/s	
Critical Velocity above which Bed Materal of Size ...	1.59	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	3.95	ft	
Scour depth from Long-Term Degradation calcul...	0.00	ft	
Maximum Flow Depth including Abutment Scour (...	4.75	ft	Including the long-term scour depth
Scour Hole Depth (ys)	-2.22	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Ratio of Bottom Width of Scour Hole to Scour Hol...	0.00		1.0 means the bottom width will be equal to scour hole depth

Right Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Ch...		
Abutment Type (k1)	Spill-through ab...		
Unit Discharge, Upstream in Main Channel (q1)	61.84	cfs/ft	
Unit Discharge in Constricted Area (q2)	54.45	cfs/ft	
D50 (D50)	0.418628	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	1.50	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	0.50	ft	Depth at Abutment Toe
Results			
q2 / q1	0.88		
Average Velocity Upstream (V)	41.23	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	1.33	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	1.34	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	1.61	ft	Including the long-term scour depth
Scour Hole Depth (ys)	1.11	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth
Scour Hole Bottom Width (K)	0.00	ft	
Scour Hole Top Width (W)	1.15	ft	

E.5. Arroyo de Los Pinos Scour

Arroyo De Los Pinos Bridge Scour Summary				
Scenario				
Bridge Geometry				
Bridge Cross-Section				
WSE				
Contraction Scour				
Selected Contraction Computation Method	**Clear-Water and Live-Bed Scour**	Clear-Water and Live-Bed Scour		
Applied Contraction Scour Depth	2.49	1	ft	Live Bed
Clear Water Contraction Scour Depth	17.99	8.53	ft	Item bolded is the governing contraction scour for scenario
Live Bed Contraction Scour Depth	**2.49**	**1**	ft	Item bolded is the governing contraction scour for scenario
Streambed Thalweg Elevation	6616.4	6616.4	ft	
Applied Contraction Scour Elevation with LTD	**6615.13**	6616.4	ft	
Approach Cross-Section				
Local Scour at Abutments				
Left Abutment				
Plot Left Abutment Scour				
Abutment Scour Depth	**7.74**	1.46	ft	NCHRP Method: Scour Condition A (includes LTD)
Max Flow Depth including Abutment Scour	12.6	4.49	ft	Including the long-term scour depth
Total Scour at Abutment	**7.74**	1.46	ft	
Local Streambed Elevation at Abutment	6617.29	6617.29	ft	
Total Scour Elevation at Abutment	**6609.56**	6616.15	ft	
Right Abutment				
Plot Right Abutment Scour				
Abutment Scour Depth	**8.19**	1.91	ft	NCHRP Method: Scour Condition A (includes LTD)
Max Flow Depth including Abutment Scour	12.6	4.49	ft	Including the long-term scour depth
Total Scour at Abutment	**8.19**	1.91	ft	
Local Streambed Elevation at Abutment	6617.7	6617.7	ft	
Total Scour Elevation at Abutment	**6609.56**	6616.15	ft	

Arroyo De Los Pinos 100-yr Scour Parameters

Contraction Scour

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Pressure Scour Method			
Compute Pressure Scour (Vertical Contraction Scour)	☐		
Input Parameters			
Input Parameters for Scour Condition			
Average Depth Upstream of Contraction (y1)	4.24	ft	(hu) in pressure scour
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream (V)	9.01	ft/s	
Computed Contraction Scour Condition	Live Bed		Vc < V
Input Parameters for Live Bed			
Temperature of Water	60.00	ºF	Used to determine the fall velocity
Slope of Energy Grade Line at Approach Section (S1)	0.021453	ft/ft	
Discharge in Contracted Section (Q2)	995.83	cfs	
Discharge Upstream that is Transporting Sediment (Q1)	1480.06	cfs	
Bottom Width in Contracted Section (W2)	30.19	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment (W1)	38.76	ft	
Depth Prior to Scour in Contracted Section (y0)	2.58	ft	
Unit Weight of Water (gamma w)	62.40	lb/ft^3	
Unit Weight of Sediment (gamma s)	165.00	lb/ft^3	
Results			
Results of Scour Condition			
Critical velocity above which bed material of size D and smaller will be transported (Vc)	2.30	ft/s	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the bed material (Dm)	1.616164	mm	
Average Depth in Contracted Section after Scour (y2)	11.11	ft	
Scour Depth (ys)	8.53	ft	Negative values imply 'zero' scour depth
Results of Live Bed Method			
k1 (k1)	0.690000		
Shear Velocity (V*)	1.71	ft/s	
Fall Velocity (T)	0.49	ft/s	
Average Depth in Contracted Section after Scour (y2)	3.59	ft	
Scour Depth (ys)	1.00	ft	Negative values imply 'zero' scour depth
Shear Applied to Bed by Live-Bed Scour (theta 0)	0.6844	lb/ft^2	
Shear Required for Movement of D50 Particle (Tau c)	0.0170	lb/ft^2	
Recommendations			
Recommended Scour Condition	Live Bed		Determined by comparing scour depths (including long-term degradation
Recommended Scour Depth	1.00	ft	Negative values imply 'zero' scour depth
Override Recommended Scour Condition	☐		

Left Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Ch...		
Abutment Type (k1)	Vertical-wall abu...		
Unit Discharge, Upstream in Main Channel (q1)	38.19	cfs/ft	
Unit Discharge in Constricted Area (q2)	32.98	cfs/ft	
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	4.24	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	3.02	ft	Depth at Abutment Toe
Results			
q2 / q1	0.86		
Average Velocity Upstream (V)	9.01	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	2.30	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	3.74	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	4.49	ft	Including the long-term scour depth
Scour Hole Depth (ys)	1.46	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth
Scour Hole Bottom Width (K)	0.00	ft	
Scour Hole Top Width (W)	1.52	ft	

Right Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type (k1)	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	38.19	cfs/ft	
Unit Discharge in Constricted Area (q2)	32.98	cfs/ft	
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	4.24	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	2.58	ft	Depth at Abutment Toe
Results			
q2 / q1	0.86		
Average Velocity Upstream (V)	9.01	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	2.30	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.20		
Flow Depth including Contraction Scour (yc)	3.74	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	4.49	ft	Including the long-term scour depth
Scour Hole Depth (ys)	1.91	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth
Scour Hole Bottom Width (K)	0.00	ft	
Scour Hole Top Width (W)	1.97	ft	

Arroyo De Los Pinos 500-yr Scour Parameters

Contraction Scour

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Pressure Scour Method			
Compute Pressure Scour (Vertical Contraction Scour)	☐		
Input Parameters			
Input Parameters for Scour Condition			
Average Depth Upstream of Contraction (y1)	5.76	ft	(hu) in pressure scour
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream (V)	10.04	ft/s	
Computed Contraction Scour Condition	Live Bed		Vc < V
Input Parameters for Live Bed			
Temperature of Water	60.00	ºF	Used to determine the fall velocity
Slope of Energy Grade Line at Approach Section (S1)	0.019794	ft/ft	
Discharge in Contracted Section (Q2)	2252.93	cfs	
Discharge Upstream that is Transporting Sediment (Q1)	2240.90	cfs	
Bottom Width in Contracted Section (W2)	30.18	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment (W1)	38.76	ft	
Depth Prior to Scour in Contracted Section (y0)	4.39	ft	
Unit Weight of Water (gamma w)	62.40	lb/ft^3	
Unit Weight of Sediment (gamma s)	165.00	lb/ft^3	
Results			
Results of Scour Condition			
Critical velocity above which bed material of size D and smaller will be transported (Vc)	2.42	ft/s	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the bed material (Dm)	1.616164	mm	
Average Depth in Contracted Section after Scour (y2)	22.38	ft	
Scour Depth (ys)	17.99	ft	Negative values imply 'zero' scour depth
Results of Live Bed Method			
k1 (k1)	0.690000		
Shear Velocity (V*)	1.92	ft/s	
Fall Velocity (T)	0.49	ft/s	
Average Depth in Contracted Section after Scour (y2)	6.88	ft	
Scour Depth (ys)	2.49	ft	Negative values imply 'zero' scour depth
Shear Applied to Bed by Live-Bed Scour (theta 0)	1.1849	lb/ft^2	
Shear Required for Movement of D50 Particle (Tau c)	0.0170	lb/ft^2	
Recommendations			
Recommended Scour Condition	Live Bed		Determined by comparing scour depths (including long-term degradation
Recommended Scour Depth	2.49	ft	Negative values imply 'zero' scour depth
Override Recommended Scour Condition	☐		

Left Abutment

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Ch...		
Abutment Type (k1)	Vertical-wall abu...		
Unit Discharge, Upstream in Main Channel (q1)	57.82	cfs/ft	
Unit Discharge in Constricted Area (q2)	74.66	cfs/ft	
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	5.76	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	4.86	ft	Depth at Abutment Toe
Results			
q2 / q1	1.29		
Average Velocity Upstream (V)	10.04	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	2.42	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.76		
Flow Depth including Contraction Scour (yc)	7.17	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	12.60	ft	Including the long-term scour depth
Scour Hole Depth (ys)	7.74	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth
Scour Hole Bottom Width (K)	0.00	ft	
Scour Hole Top Width (W)	8.01	ft	

Right Abutment

Abutment Scour

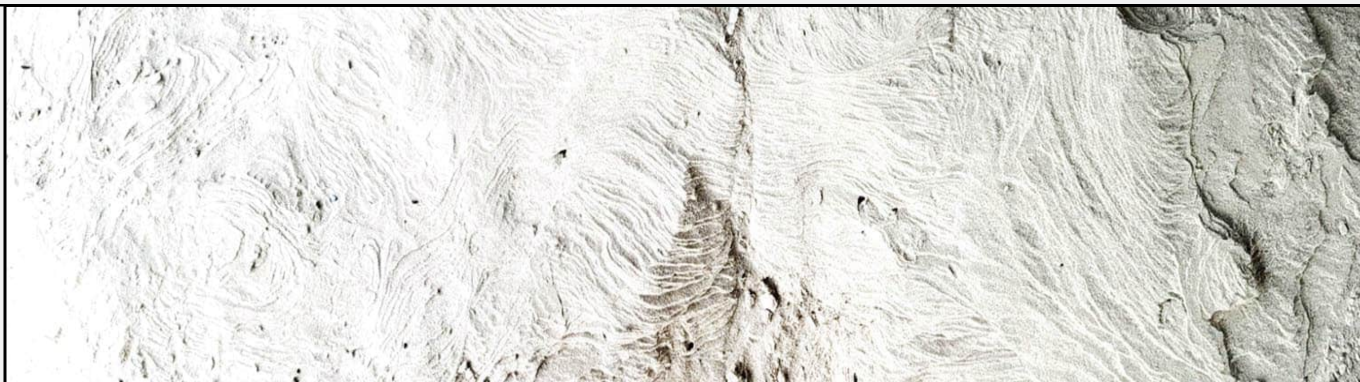
Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type (k1)	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	57.82	cfs/ft	
Unit Discharge in Constricted Area (q2)	74.66	cfs/ft	
D50 (D50)	1.292931	mm	0.2 mm is the lower limit for cohesive material
Upstream Flow Depth (y1)	5.76	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour (y0)	4.40	ft	Depth at Abutment Toe
Results			
q2 / q1	1.29		
Average Velocity Upstream (V)	10.04	ft/s	
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported (Dm)	2.42	ft/s	Used in contraction scour calculations
Scour Condition	Live Bed		
Scour Condition	a (Main Channel)		
Amplification Factor (alpha A or alpha B)	1.76		
Flow Depth including Contraction Scour (yc)	7.17	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour (ymax)	12.60	ft	Including the long-term scour depth
Scour Hole Depth (ys)	8.19	ft	Negative values imply 'zero' scour depth
Scour Hole			
Angle of Repose (theta)	44.00	degr...	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will be equal to scour hole depth
Scour Hole Bottom Width (K)	0.00	ft	
Scour Hole Top Width (W)	8.48	ft	

E.6. Excerpts from Geotechnical Report Prepared by YeDoma Consultants LLC

ARROYO DE LOS CHAMISOS CROSSING
PRELIMINARY GEOTECHNICAL REPORT
SANTA FE COUNTY, NEW MEXICO

Publish Date: 7/17/2023



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523 Louisiana Blvd SE
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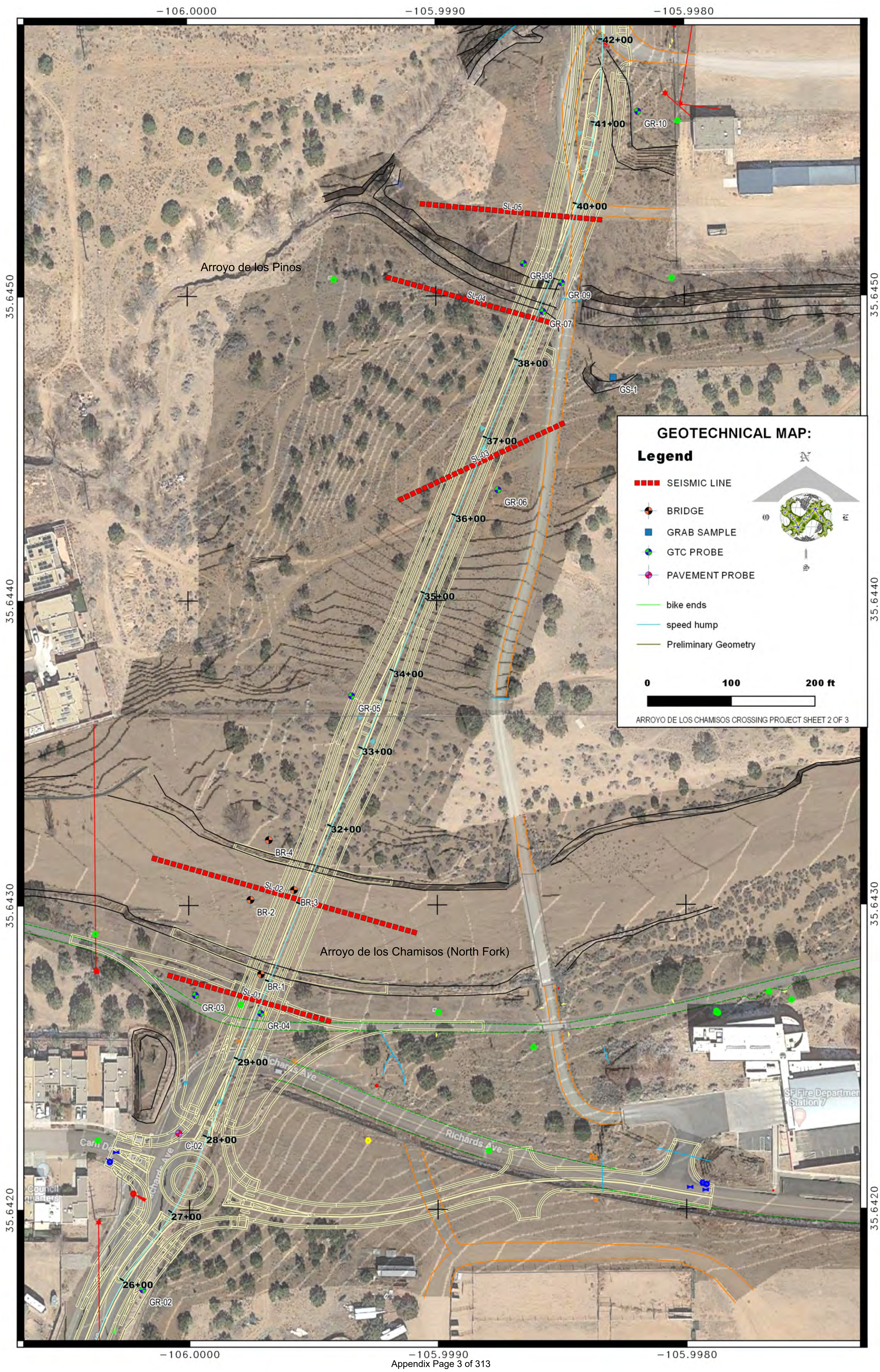


GEOTECHNICAL MAP:
Legend
SEISMIC LINE
BRIDGE
GRAB SAMPLE
GTC PROBE
PAVEMENT PROBE
bike ends
speed hump
Preliminary Geometry
0
100
200 ft
ARROYO DE LOS CHAMISOS CROSSING PROJECT SHEET 2 OF 3
−106.0000
−105.9990
−105.9980
35.6450
35.6440
35.6430
35.6420
Arroyo de los Pinos
Arroyo de los Chamisos (North Fork)
Richards Ave
SF Fire Department Station 7
42+00
41+00
40+00
38+00
37+00
36+00
35+00
34+00
33+00
32+00
29+00
28+00
27+00
26+00
SL-01
SL-02
SL-03
SL-04
SL-05
GR-02
GR-03
GR-04
GR-05
GR-06
GR-07
GR-08
GR-09
GR-10
GS-1
BR-1
BR-2
BR-3
BR-4
C-02

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

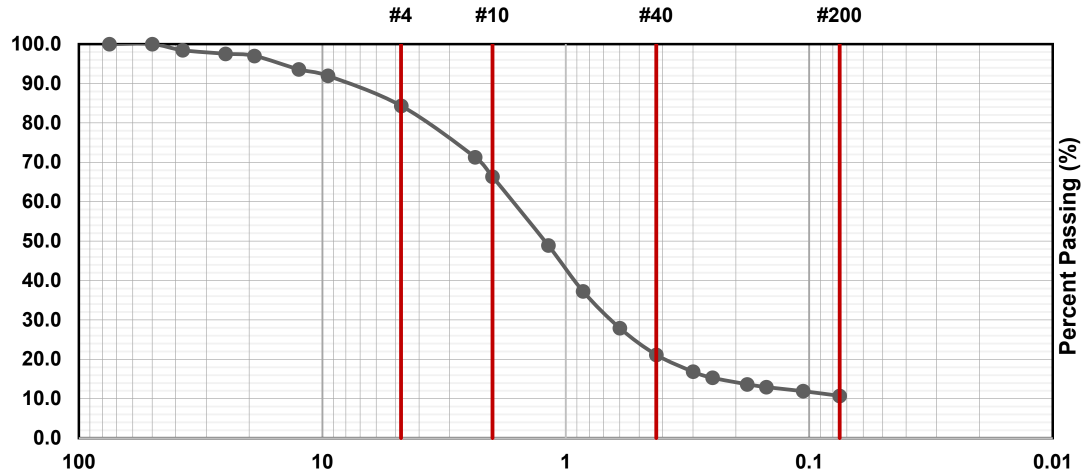
Report Date:	5/22/2023	**Sample Date:**	3/7 - 3/8/2023	**Material Type:**	Bulk
Report No.:	621194-SieveReport-19	**Sample By:**	YeDoma	**Station/Depth:**	0' - 1'
Project Name:	ADLC	**Test Date:**	5/18/2023	**Sample Location:**	GR-09-GS-01
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Sample Received:**	3/8/2023
Project No.:	621194	**Client:**	WSP		

Client Address: 2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110

Test Method: **ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**

Sample: 194-65 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried

Soil Gradation



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:

Group Name: Well-graded sand with clay and gravel (or silty clay and gravel)

Group Symbol: SW-SC

AASHTO Classification - ASTM D3282-15:

Group Name: A-2-6

Atterberg Limits: ASTM D4318-17

Liquid Limit: 26

Plastic Limit: 14

Plasticity Index: 12

Shape Parameters:

Fineness Modulus: 3.393

C_U: 26.67

C_C: 2.40

D_{60}: 2.000

D_{30}: 0.600

D_{10}: 0.075

Sieve Size	% Passing
3"	-
2"	100
1 1/2"	99
1"	98
3/4"	97
1/2"	94
3/8"	92
#4	84
#8	71
#10	66
#16	49
#20	37
#30	28
#40	21
#50	17
#60	15
#80	14
#100	13
#140	12
#200	11

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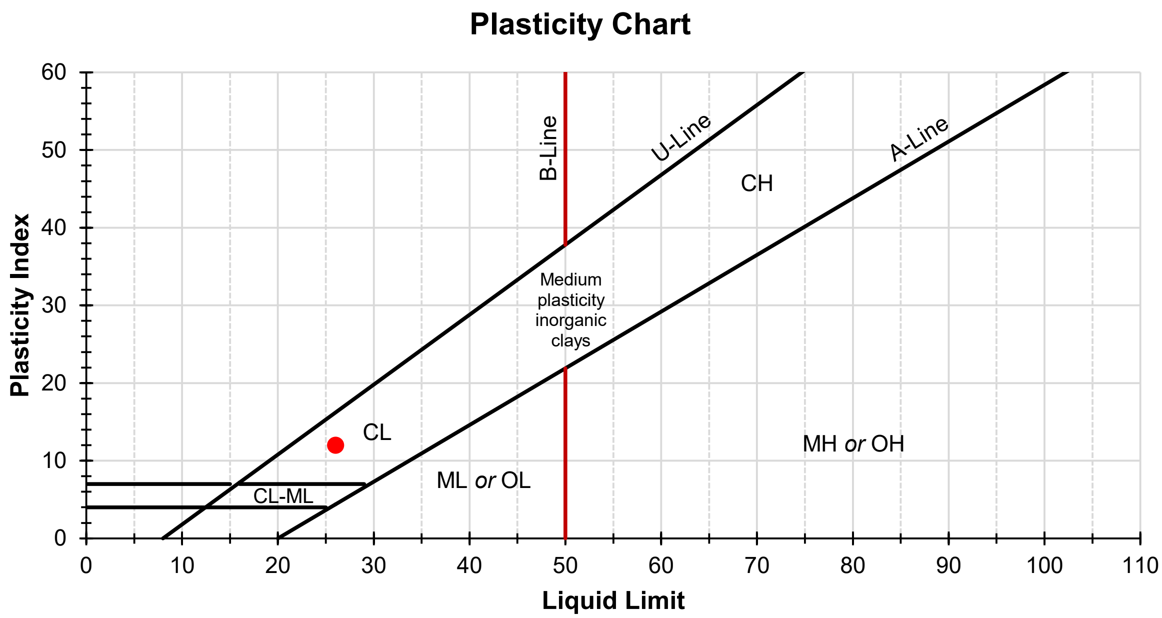
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/22/2023	**Sample Date:**	3/7 - 3/8/2023	**Material Type:**	Bulk
Report No.:	621194-SieveReport-19	**Sample By:**	YeDoma	**Station/Depth:**	0' - 1'
Project Name:	ADLC	**Test Date:**	5/18/2023	**Sample Location:**	GR-09-GS-01
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Sample Received:**	3/8/2023
Project No.:	621194	**Client:**	WSP		
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

Version 2, 10/27/2021

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Albuquerque, NM 87108

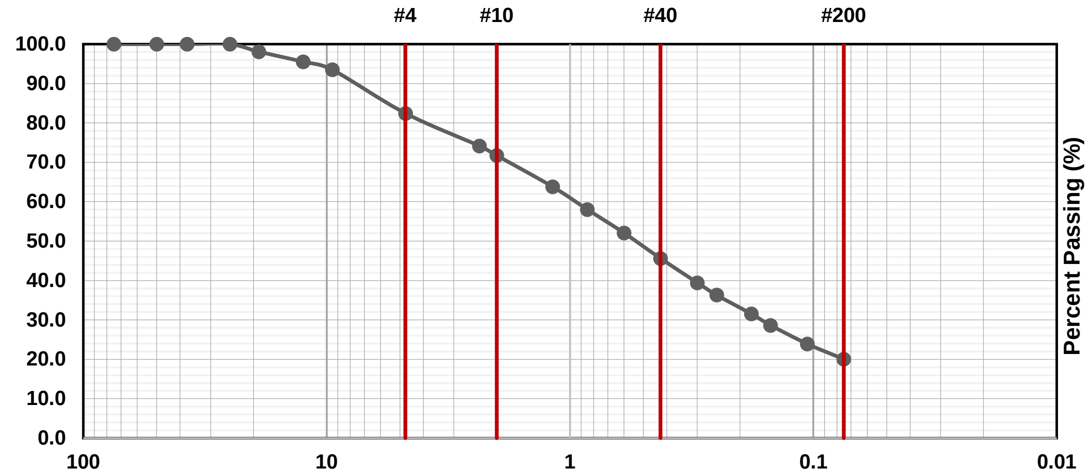
Sheet 1/2

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-34	**Sample By:**	YeDoma	**Station/Depth:**	20'-21.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				

Sample: BR3-20 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried

Soil Gradation



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:
Group Name: Silty sand with gravel

Group Symbol: SM

AASHTO Classification - ASTM D3282-15:
Group Name: A-1-b

Atterberg Limits: ASTM D4318-17
Liquid Limit: NV
Plastic Limit: NP
Plasticity Index: SNP

Shape Parameters:
Fineness Modulus: 2.441
C_U: 11.33
C_C: 0.35

D_{60}: 0.850
D_{30}: 0.150
D_{10}: 0.075

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	-
1"	100
3/4"	98
1/2"	96
3/8"	94
#4	82
#8	74
#10	72
#16	64
#20	58
#30	52
#40	46
#50	39
#60	36
#80	32
#100	29
#140	24
#200	20

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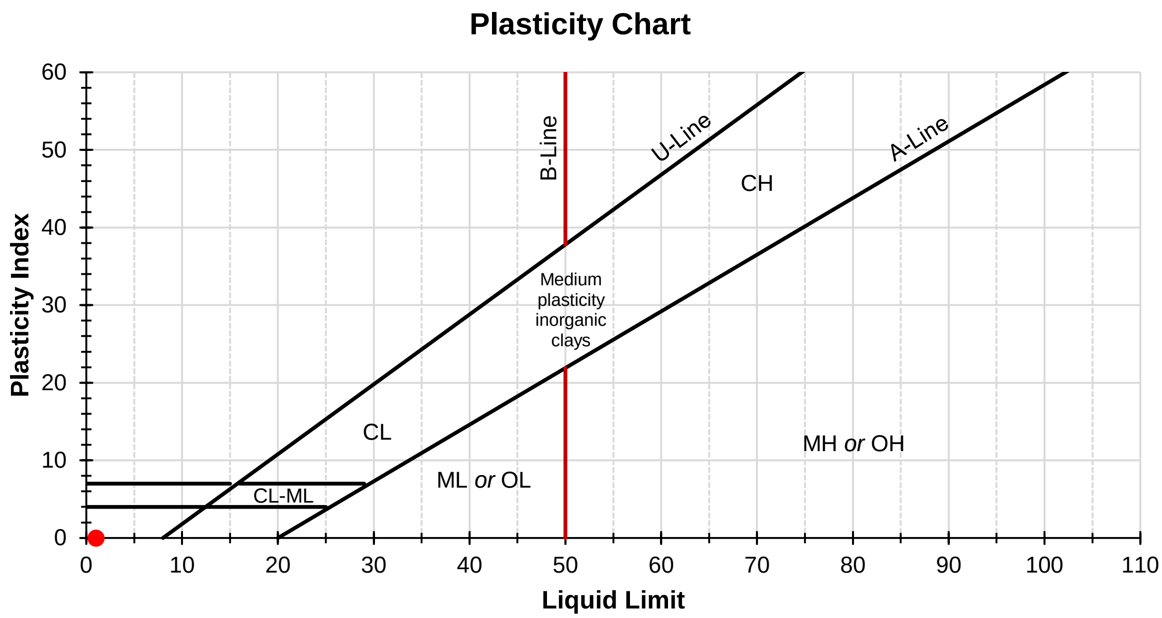
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-34	**Sample By:**	YeDoma	**Station/Depth:**	20'-21.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

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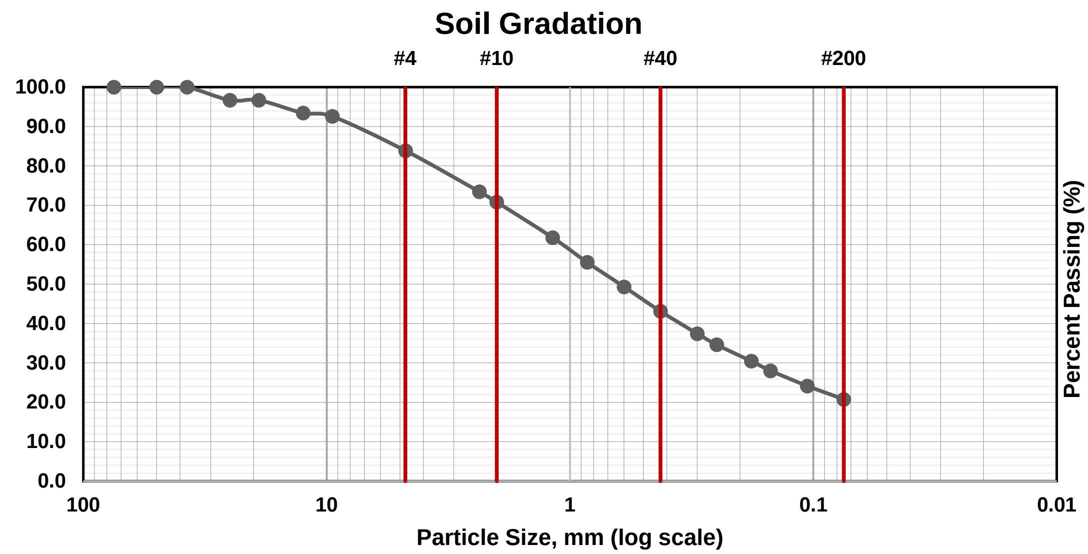
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-35	**Sample By:**	YeDoma	**Station/Depth:**	30'-31.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				

Sample: BR3-30 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:
Group Name: Silty sand with gravel

Group Symbol: SM

AASHTO Classification - ASTM D3282-15:
Group Name: A-1-b

Atterberg Limits: ASTM D4318-17
Liquid Limit: NV
Plastic Limit: NP
Plasticity Index: SNP

Shape Parameters:
Fineness Modulus: 2.501
C_U: 15.73
C_C: 0.37

D_{60}: 1.180
D_{30}: 0.180
D_{10}: 0.075

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	100
1"	97
3/4"	97
1/2"	93
3/8"	93
#4	84
#8	73
#10	71
#16	62
#20	56
#30	49
#40	43
#50	37
#60	35
#80	30
#100	28
#140	24
#200	21

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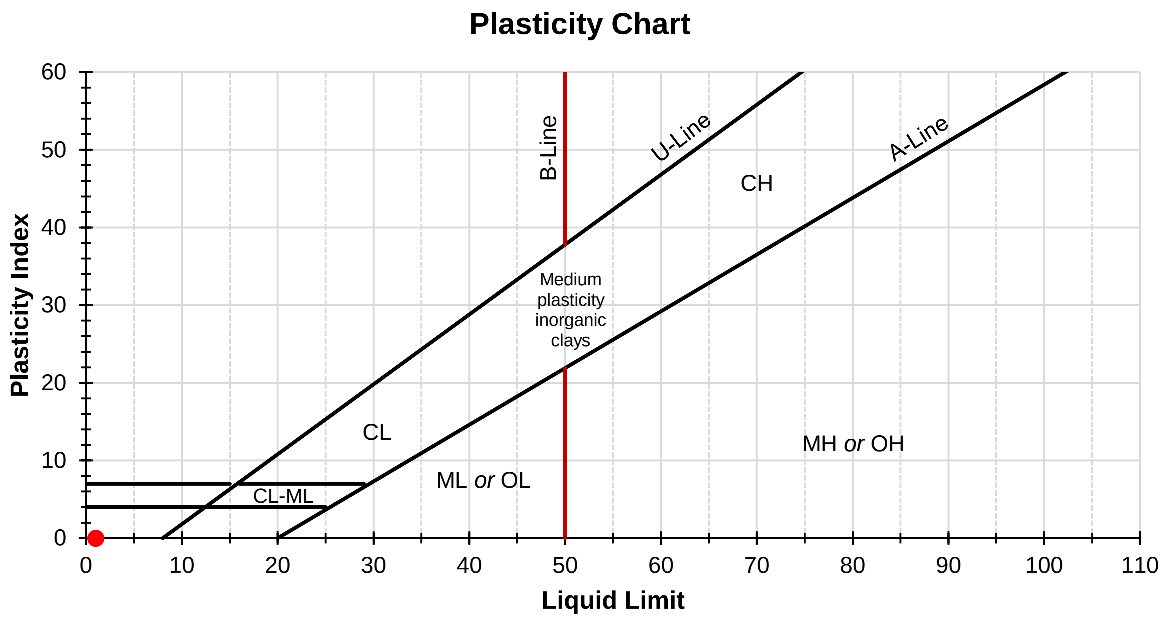
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-35	**Sample By:**	YeDoma	**Station/Depth:**	30'-31.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

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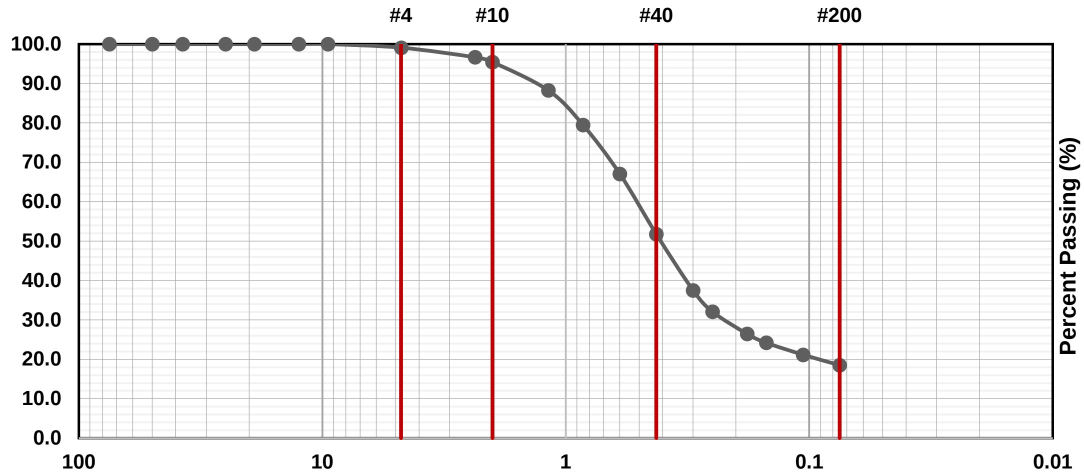
Sheet 1/2

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-36	**Sample By:**	YeDoma	**Station/Depth:**	35'-36.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				

Sample: BR3-35 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried

Soil Gradation



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:
Group Name: Clayey sand

Group Symbol: SC

AASHTO Classification - ASTM D3282-15:
Group Name: A-2-4

Shape Parameters:
Fineness Modulus: 1.872
C_U: 8.00
C_C: 1.39

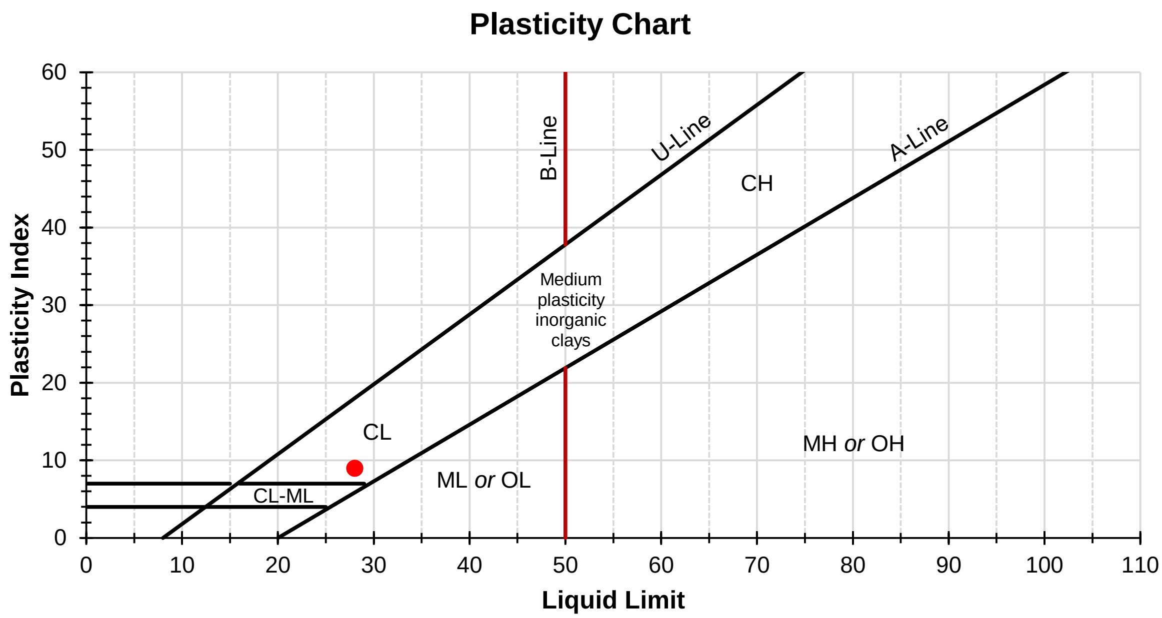
D_{60}: 0.600
D_{30}: 0.250
D_{10}: 0.075

Atterberg Limits: ASTM D4318-17
Liquid Limit: 28
Plastic Limit: 19
Plasticity Index: 9

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	-
1"	-
3/4"	-
1/2"	-
3/8"	100
#4	99
#8	97
#10	95
#16	88
#20	80
#30	67
#40	52
#50	38
#60	32
#80	26
#100	24
#140	21
#200	19

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-36	**Sample By:**	YeDoma	**Station/Depth:**	35'-36.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

Sheet 1/2

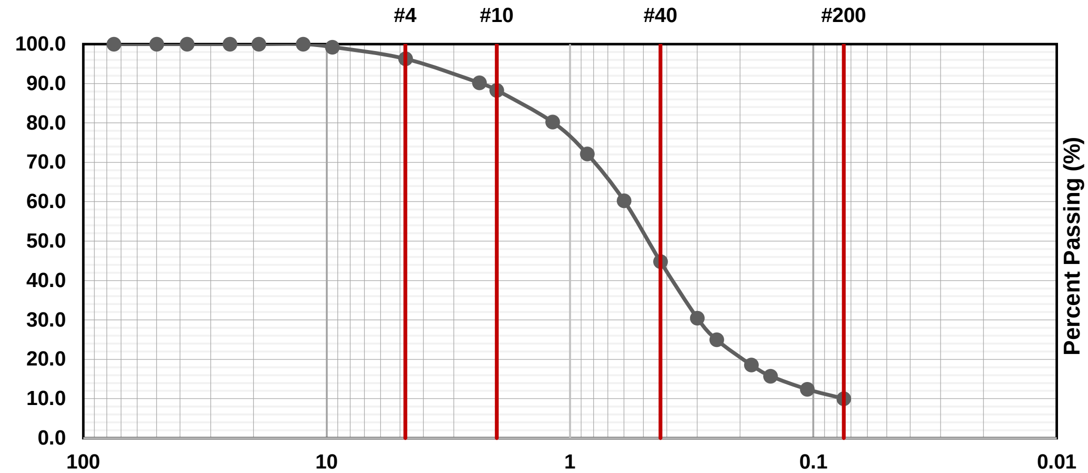
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-37	**Sample By:**	YeDoma	**Station/Depth:**	50'-51.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		

Test Method: **ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**

Sample: BR3-50 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried

Soil Gradation



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:

Group Name: Well-graded sand with silt

Group Symbol: SW-SM

AASHTO Classification - ASTM D3282-15:

Group Name: A-1-b

Atterberg Limits: ASTM D4318-17

Liquid Limit: NV

Plastic Limit: NP

Plasticity Index: SNP

Shape Parameters:

Fineness Modulus: 2.275

C_U: 8.00

C_C: 2.00

D_{60}: 0.600

D_{30}: 0.300

D_{10}: 0.075

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	-
1"	-
3/4"	-
1/2"	100
3/8"	99
#4	96
#8	90
#10	88
#16	80
#20	72
#30	60
#40	45
#50	30
#60	25
#80	19
#100	16
#140	12
#200	10

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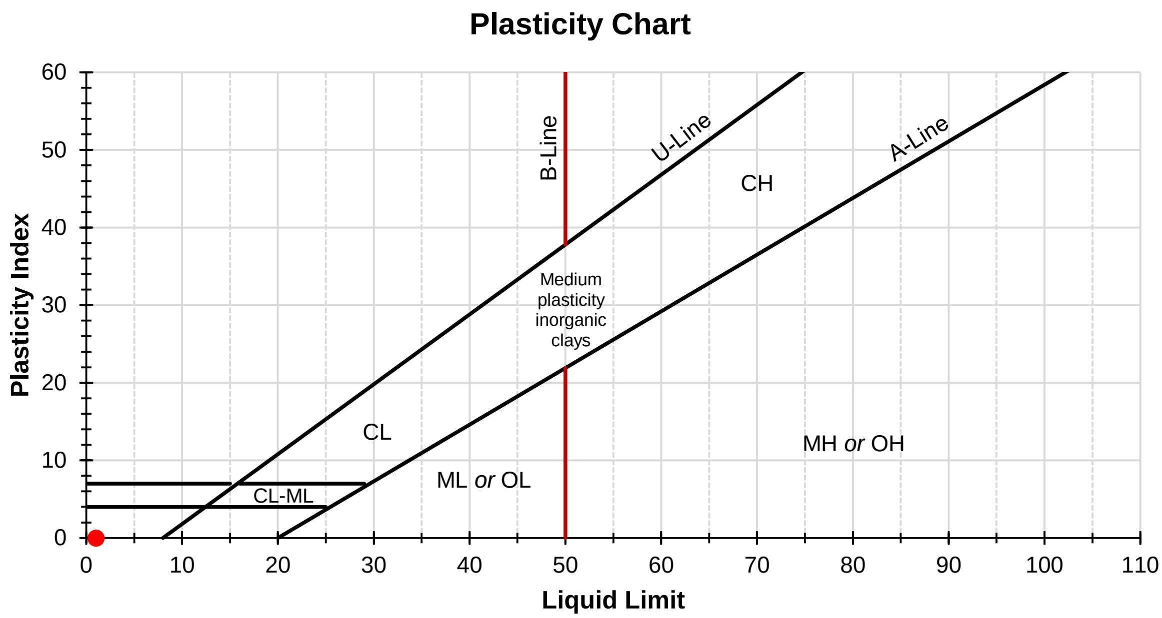
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-37	**Sample By:**	YeDoma	**Station/Depth:**	50'-51.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

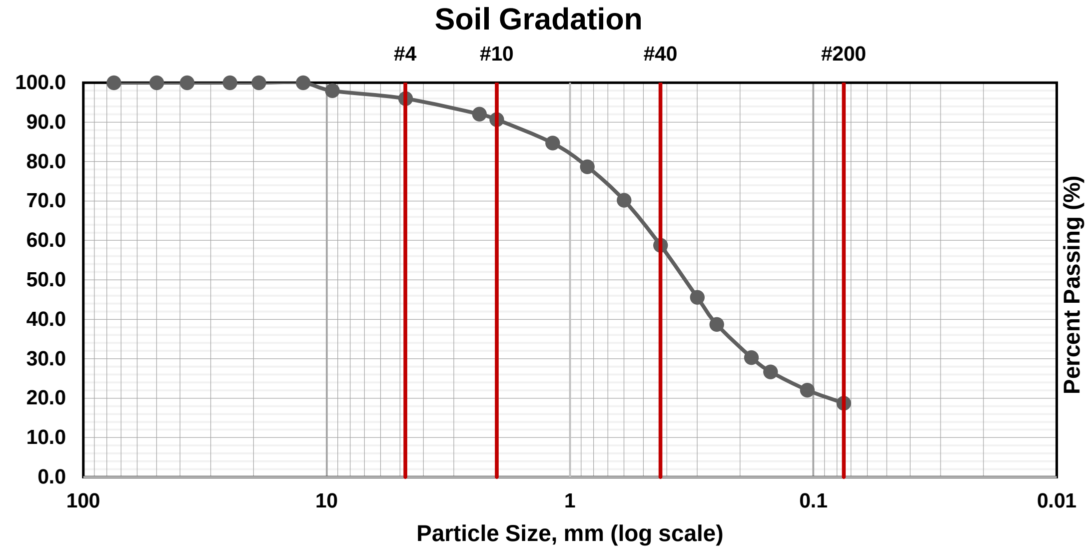
Sheet 1/2

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-38	**Sample By:**	YeDoma	**Station/Depth:**	55'-56.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		

Test Method: **ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**

Sample: BR3-55 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:
Group Name: Clayey sand

Group Symbol: SC

AASHTO Classification - ASTM D3282-15:
Group Name: A-2-4

Shape Parameters:
Fineness Modulus: 1.784
C_U: 5.67
C_C: 1.02

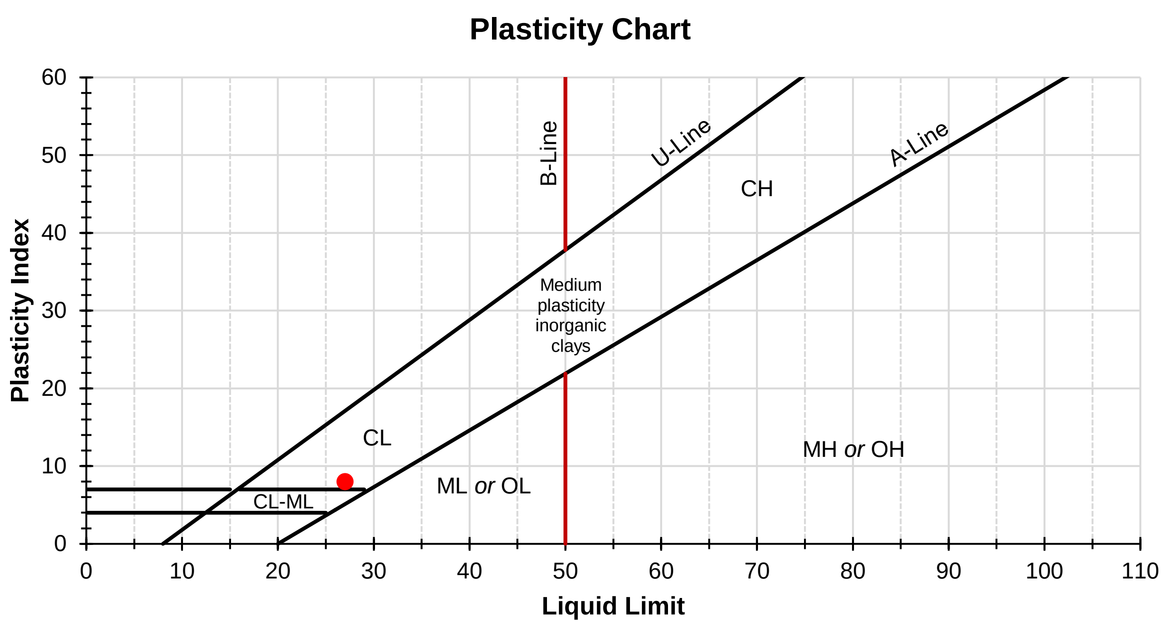
D_{60}: 0.425
D_{30}: 0.180
D_{10}: 0.075

Atterberg Limits: ASTM D4318-17
Liquid Limit: 27
Plastic Limit: 19
Plasticity Index: 8

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	-
1"	-
3/4"	-
1/2"	100
3/8"	98
#4	96
#8	92
#10	91
#16	85
#20	79
#30	70
#40	59
#50	46
#60	39
#80	30
#100	27
#140	22
#200	19

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/24/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-38	**Sample By:**	YeDoma	**Station/Depth:**	55'-56.5'
Project Name:	ADLC	**Test Date:**	5/24/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

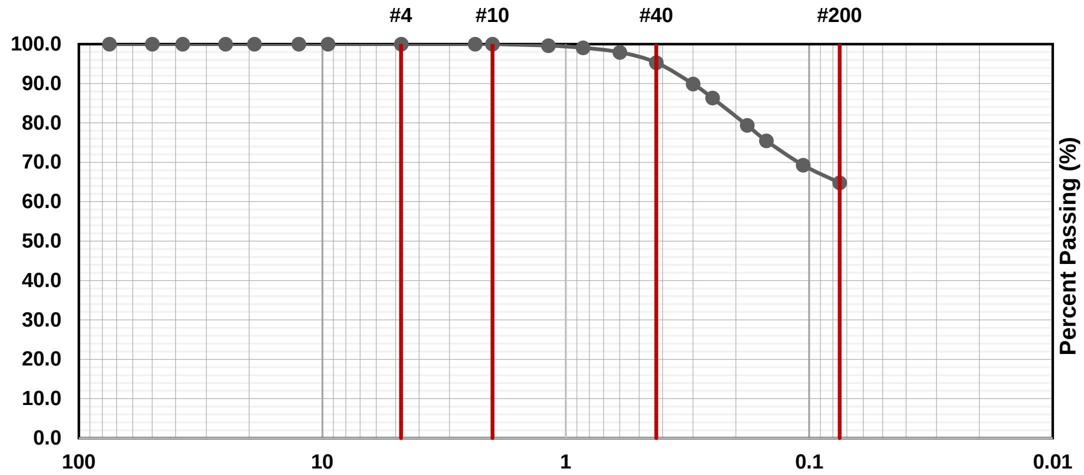
Reviewed By: Technical Manager
Jesse Reinikainen, PE

Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/26/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-39	**Sample By:**	YeDoma	**Station/Depth:**	61.2'-61.5'
Project Name:	ADLC	**Test Date:**	5/26/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				

Sample: BR3-60 **Soak Time:** 2 Hours **Sample Prep Method:** Oven-dried

Soil Gradation



Gradation

Dispersion Process: ☐ Ultrasonic bath ☐ Shaking Apparatus ☒ None

ASTM D2487 Classification:

Group Name: Sandy fat clay

Group Symbol: CH

AASHTO Classification - ASTM D3282-15:

Group Name: A-7-6

Atterberg Limits: ASTM D4318-17

Liquid Limit: 65

Plastic Limit: 23

Plasticity Index: 42

Shape Parameters:

Fineness Modulus: 0.371

C_U: 1.00

C_C: 1.00

D_{60}: 0.075

D_{30}: 0.075

D_{10}: 0.075

Sieve Size	% Passing
3"	-
2"	-
1 1/2"	-
1"	-
3/4"	-
1/2"	-
3/8"	-
#4	-
#8	-
#10	100
#16	100
#20	99
#30	98
#40	95
#50	90
#60	86
#80	79
#100	75
#140	69
#200	65

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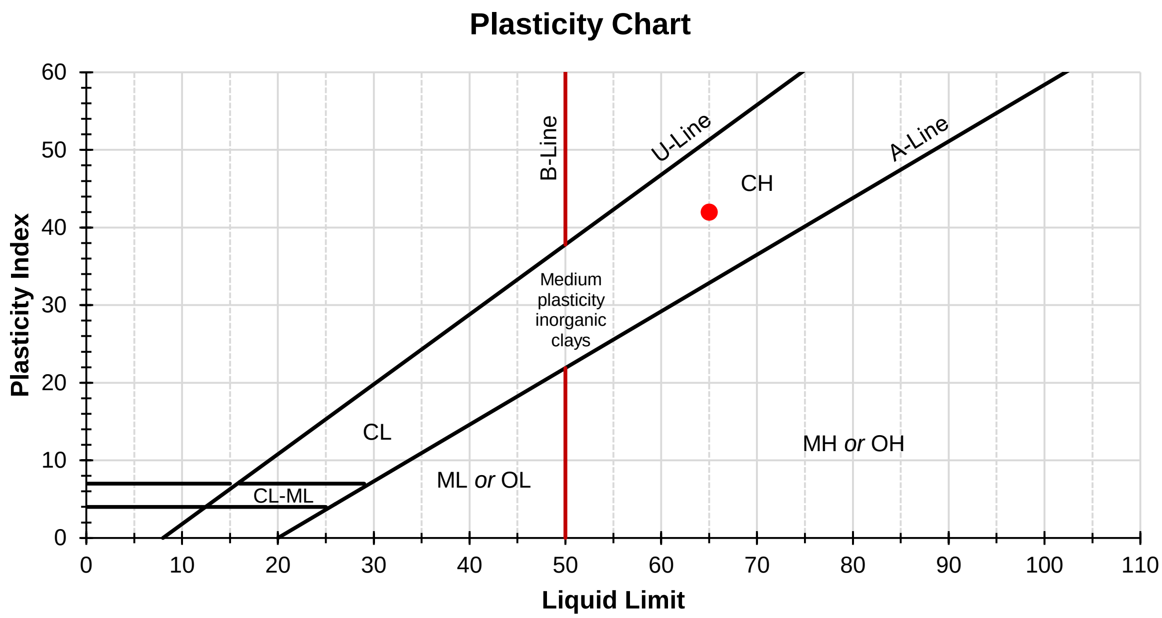
Standard Test Methods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis

Report Date:	5/26/2023	**Sample Date:**	4/7 - 4/8/2023	**Material Type:**	Downhole
Report No.:	621194-SieveReport-39	**Sample By:**	YeDoma	**Station/Depth:**	61.2'-61.5'
Project Name:	ADLC	**Test Date:**	5/26/2023	**Location:**	BR-03
Project Location:	Santa Fe, NM	**Tested By:**	DS	**Boring No.:**	BR-03
Project No.:	621194	**Client:**	WSP	**Sample Received:**	4/8/2023
		Client Address:	2440 Louisiana Blvd NE Suite 400, Albuquerque, NM 87110		
Test Method:	**ASTM D6913-17, ASTM D2487-17, ASTM D3282-15, ASTM D4318-17**				



Notes/Comments/Deviations from Test Standard:

Reviewed By: Technical Manager
Jesse Reinikainen, PE

Version 2, 10/27/2021

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