United States Section of the International Boundary and Water Commission

Design for the Construction of Channel Maintenance Alternatives within the Rio Grande Canalization Project Doña Ana County, New Mexico

Contract No. IBM15D0003
Order No. IBM16T0018

Design Summary Report
Final Submittal

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Table of Contents

List of Tables

List of Figures

1.0 PROJECT DESCRIPTION

1.1 Historical Background

The Rio Grande Canalization Project (RGCP), which extends 105.4 miles from Percha Dam in Sierra County, New Mexico to American Dam in El Paso, Texas, was constructed between 1938 and 1943 as authorized by the Act of Congress approved June 4, 1936 (Public Law 648, 49 Stat 1463) to: "facilitate compliance with the convention between the United States and Mexico concluded May 21, 1906, providing for the equitable division of the waters of the Rio Grande, and to properly regulate and control, to the fullest extent possible, the water supply for use in the two countries as provided by treaty." The Act authorizes the United States International Boundary and Water Commission (USIBWC) to construct, operate and maintain the RGCP in accordance with the plan in the Engineering Record of December 14, 1935. The USIBWC objectives for the RGCP can be summarized by: Flood Conveyance and Flood Protection, Channel Conveyance Reliability, Delivery Efficiency, Compliance with U.S. Regulations, and Minimizing Costs.

1.2 Project Background

There is ongoing sediment inflow from the tributary arroyos, resulting in sediment deposition forming sediment plugs at arroyo confluences along sections of the Rio Grande. Sediment inflow also results in island formations and raising of river beds. Sediment accumulation prevents draining of irrigation return flow to the Rio Grande and may result in increases in water surface elevations, which could impact levee freeboard and increase the flooding risk to adjoining communities. A study entitled *Channel Maintenance Alternatives and Sediment Transport Studies for the RGCP Final Report* was completed in 2015 by Tetra Tech, Inc. (hereafter "Tetra Tech 2015"). The report identified nine (9) representative problem locations experiencing sediment accumulation along the 105.4 miles of the RGCP that were evaluated in the study. The report then evaluated, scored, and ranked various Channel Maintenance Alternatives (CMAs) for each of the nine (9) problem locations. The report presented a conceptual sediment trap as one of the CMAs, and because of the high benefit-to-cost consequence of the sediment trap as determined in the report, it was recommended as an alternative to be used at all of the problem locations.

USIBWC contracted URS Group, Inc. (URS) to perform design of one or more of the CMAs at each of two (2) selected locations within USIBWC's ROW. The two selected sites are referred to as "Thurman I Arroyo" and "Thurman II Arroyo" and are located within Problem Location 2, which extends a distance of approximately 3.3 miles from the Salem Bridge at NM Highway 391 downstream to the confluence with Placitas Arroyo (see Figure 1-1). The Tetra Tech 2015 study found that during recent monsoon season tributary flow events (i.e., 2006 and 2013), Thurman I and II Arroyos each delivered significant quantities of sediment to the RGCP, and appear to have delivered additional sediment since that time. After the 2006 events, USIBWC removed sediment from the river, reconstructed the opposite bank and excavated the mouth of the Thurman I Arroyo. Evidence of bank protection along the right bank opposite Thurman II Arroyo suggests that similar activities were undertaken at this tributary. Islands have formed along the downstream portions of both of the Thurman Arroyo fans, along with numerous other islands and vegetated bars along the reach.

The intent of the design project is such that the constructed alternatives will improve conveyance efficiency, hydraulic capacity, drainage return flows, and levee infrastructure, and will decrease flood risk and reduce the overall operations and maintenance at each site.

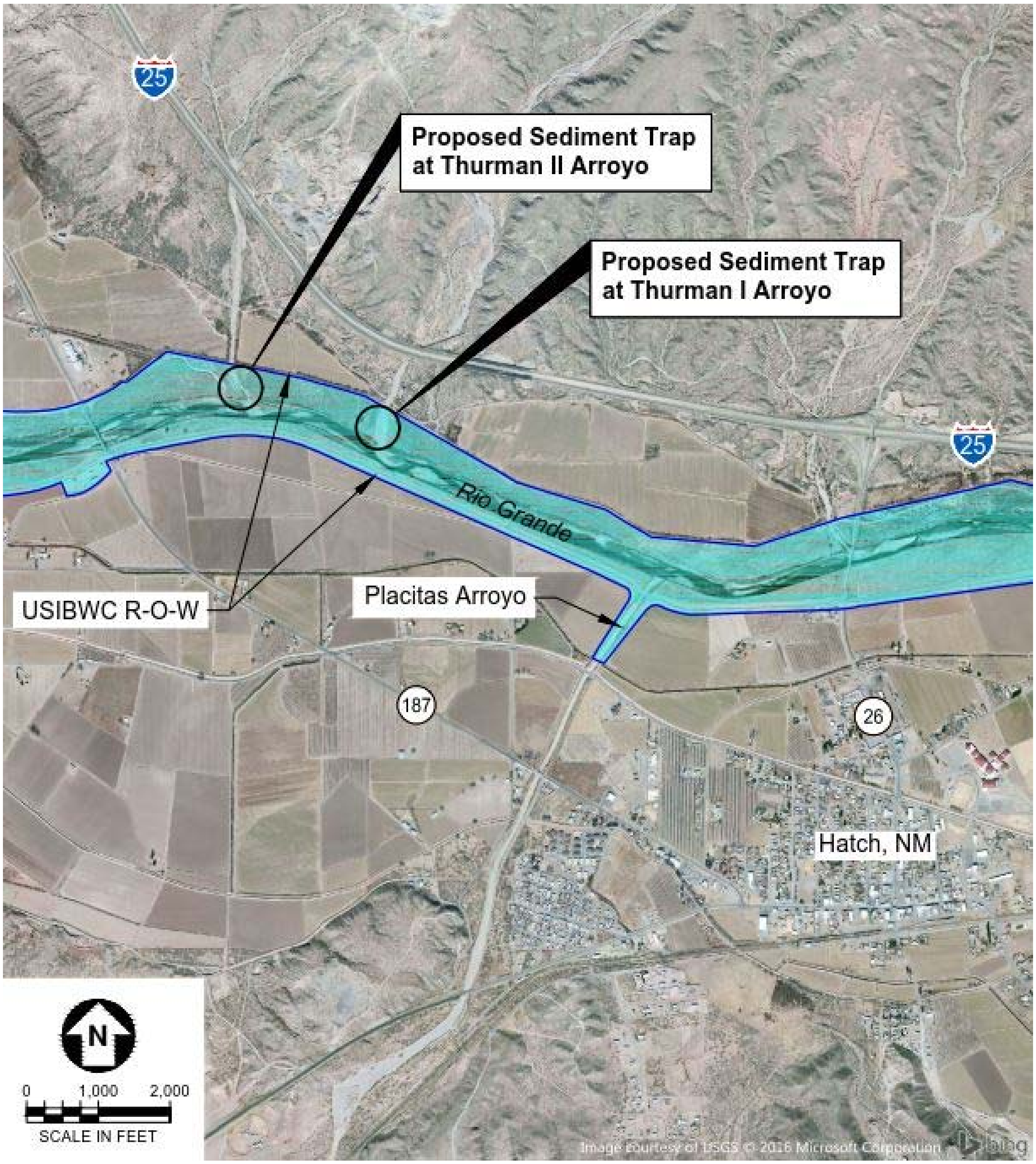


Figure 1-1. Project Location Map

1.3 Objectives

The design objectives for both sites addressed under this scope of work are as follows:

1. Provide design for a solution that permits removal of localized sediment within the main channel to ensure adequate hydraulic capacity for each design and allow for routine maintenance within the sediment trap(s).
2. Coordinate all work with impacted stakeholders.
3. Analyze potential drainage impacts along to adjacent properties so such drainage improvements are incorporated into the design.
4. Design any modifications to affected structures that may be necessary due to the proposed modifications.
5. Provide any alternative recommendations in lieu of the sediment traps for effectively trapping/collecting the sediment prior to reaching the Rio Grande confluence.

2.0 REVIEW OF PREVIOUS REPORTS AND DRAWINGS

2.1 Previous Reports and Drawings Obtained

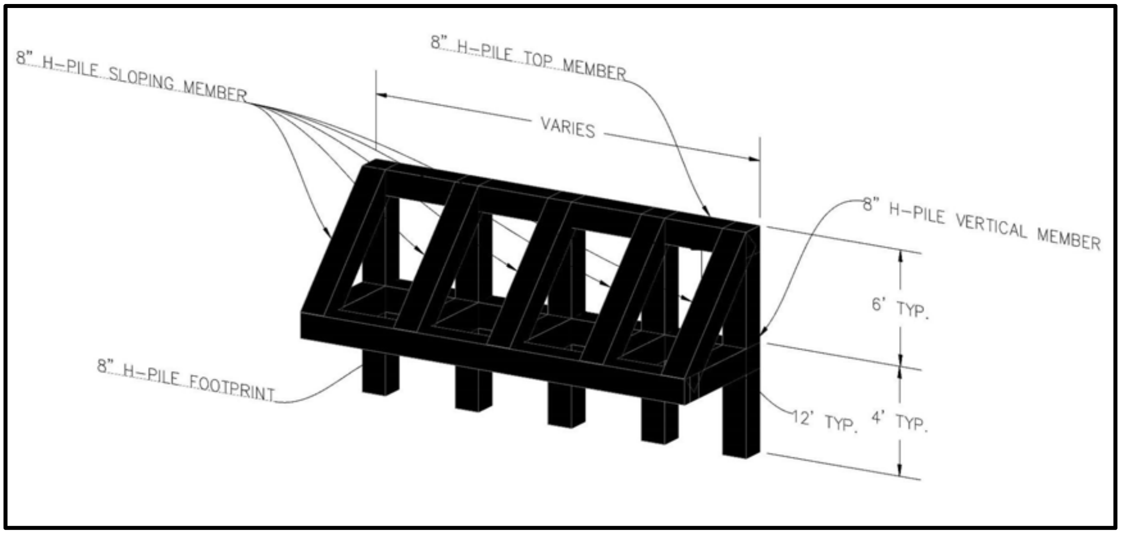
The following documents were provided by USIBWC and reviewed by URS for information:

1. Tetra Tech, Inc. *Channel Maintenance Alternatives (CMA) and Sediment Transport Studies for the Rio Grande Canalization Project: Final Report*. Tetra Tech, Inc.; October 20, 2015.
2. USIBWC. *Canalization River Management Plan – Part 4 – Channel Maintenance Plan, Draft*. USIBWC; August 2016.

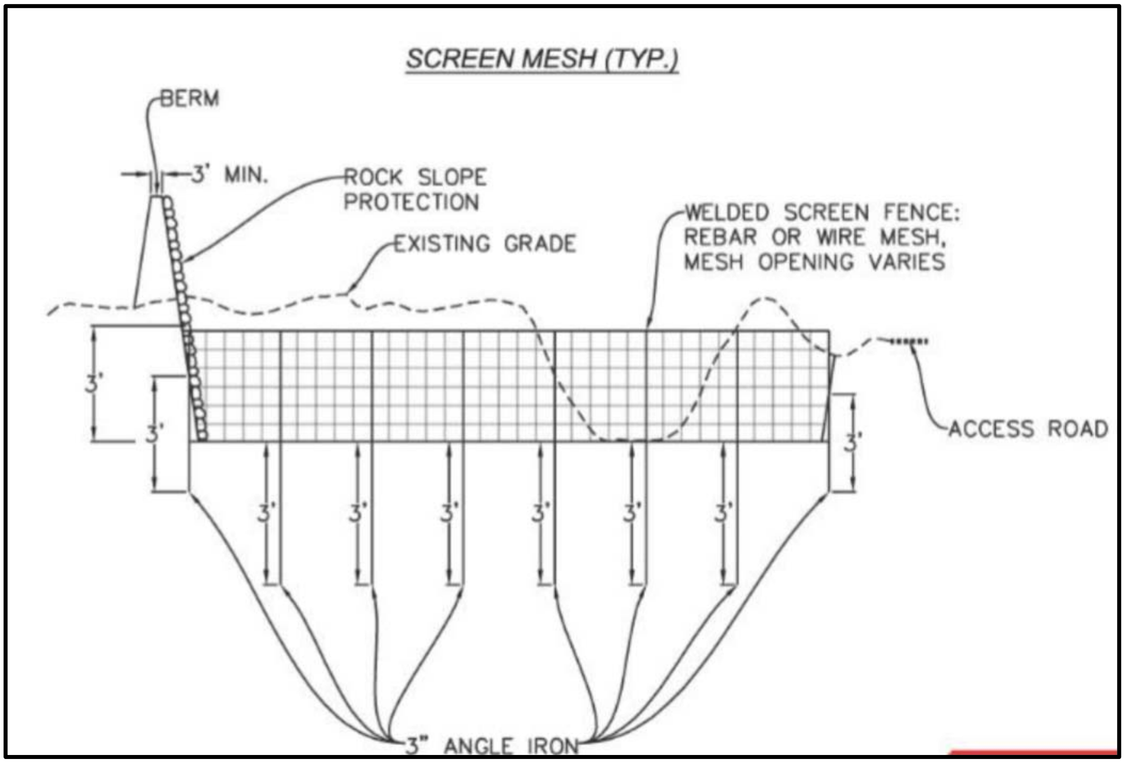
2.2 Sediment Trap System Design Considerations

The recommended channel maintenance alternative for Thurman I and II Arroyos, as presented in Tetra Tech 2015, was a screen-based sediment trap system. The sediment trap system functions by filtering (capturing) sediment from incoming arroyo flows to prevent sediment accumulation in the Rio Grande. Large particles are captured at the upstream end of the channel, and progressively finer sediments are captured at intermediate locations as flows travel downstream. Based on conceptual design drawings presented in Tetra Tech 2015, each arroyo sediment trap system is a collective of several independent structures ("trapping features") as described below.

1. Upstream Debris Rack. The debris rack captures large particles (cobbles, gravels, etc.) and other debris near the beginning of the channel that could detrimentally affect downstream trapping features. The debris rack consists of structural steel sections connected to and supported by deep foundations (e.g., driven piles or drilled piers). A conceptual drawing of the debris rack is provided in Graphic 2-1.
2. Intermediate Sediment Traps. Intermediate sediment traps are intended to capture sand, silt, and clay size fractions of the incoming flows, and typically consist of either rock check structures or mesh screens. For this project, only mesh screen options were considered in design. In this case, the sediment trap consists of a mesh screen of rebar or welded wire fabric (WWF) connected to and supported by vertical angle sections driven into the subgrade. Preliminary information indicated five intermediate sediment traps would be required at each Thurman I Arroyo and Thurman II Arroyo. A conceptual drawing of the sediment trap is provided in Graphic 2-2.
3. Downstream Embayment. The downstream embayment would be constructed at the confluence of the arroyo and the Rio Grande. The purpose of the embayment is to provide habitat benefits as a lower velocity, off-channel refuge area with vegetative cover off the Rio Grande.



Graphic 2-1. Conceptual Drawing of Upstream Debris Rack (Tetra Tech 2015).



Graphic 2-2. Conceptual Drawing of Sediment Trap Screen (Tetra Tech 2015).

As part of this project, URS performed engineering evaluation and comparison of different sediment trap systems for the Thurman I and II Arroyos, including a sediment basin design. Detailed discussion of the findings and conclusions are provided in Section 5, Hydrologic and Hydraulic Analyses.

3.0 TOPOGRAPHIC SURVEY

Topographic surveys of the two arroyo sites were not performed for this design effort. To perform the design, create the necessary grading, and develop construction drawings of the sediment basins, URS used LiDAR data provided by USIBWC. The LiDAR data were created in 2011 by Watershed Sciences, Inc. for Tetra Tech, Inc., the contractor for USIBWC.

USIBWC also provided surveyed cross-section data of the two arroyos. Nine cross-sections were provided at Thurman I Arroyo – three cross-sections between the Rio Grande and the north USIBWC right-of-way boundary; five cross-sections between the north right-of-way and Interstate Highway 25; and one cross-section on the north side of IH25. Nineteen cross-sections were provided at Thurman II Arroyo – six cross-sections between the Rio Grande and north USIBWC right-of-way boundary; eleven cross-sections between the north right-of-way and Interstate Highway 25; and two cross-section on the north side of IH25. The cross-section data were collected by USIBWC personnel in September 2016.

3.1 Survey Control

Horizontal and vertical datums used for the design drawings are based on:

Horizontal Control:
New Mexico State Plane Coordinate System, Central Zone, North American Datum of 1983 (NAD 83)

Vertical Control:
North American Vertical Datum of 1998 (NAVD 88)

4.0 UTILITIES STUDY

4.1 Overview

URS conducted a utility study encompassing the area of the proposed sediment traps to determine if existing utility conflicts were present. A site visit was made by URS employees to both arroyos on October 20, 2016. During the site visit, observations were made to determine if there were any visible signs of utilities, either overhead or underground. There were no visible indications that utilities are present at the two sites. After the initial site visit, the New Mexico 811 Damage Prevention Center (NM811) was contacted to facilitate location of utilities in the project area. NM811 generated a design conference ticket that notified utility companies which have utilities in the area. The purpose of the design conference ticket was to open a line of communication between the utility owners and URS to identify if any conflicts exist. Utility owners of the area include: CenturyLink, El Paso Electric, and The Village of Hatch.

CenturyLink, the telecommunications utility and internet service provider of the area, was the first utility owner to contact URS. CenturyLink informed URS that none of their utilities exist in the area. After discussion with CenturyLink, a technician from United States Infrastructure Corporation (USIC), a large underground utility locator, contacted URS. URS provided the project location map to the technician, and the technician stated that no utilities exist in the area except for the possibility of a small telephone line. After informing the technician that CenturyLink cleared the area of their utilities, this concern was relieved.

To identify water utilities, The Village of Hatch Public Works Director (PWD) was contacted. The design conference ticket listed Garfield Mutual Domestic Water Consumers Association (Garfield MDWCA) as the water utility owner of the area, but upon contacting Garfield MDWCA, URS was instructed to speak with The Village of Hatch Public Works. Initial conversation with the PWD revealed that Hatch does not have water utilities extending to the site, but the project location map was sent to reaffirm, and the PWD did confirm this.

El Paso Electric utility locations were determined by conversation with USIC. URS was informed that all electric utilities in the area are aerial, so no buried conflicts would be encountered.

To determine the presence of gas lines, the National Pipeline Mapping System (NPMS) Public Map Viewer was used and confirmed that gas lines do not exist near the project area (see Figure 4-1). According to the NPMS, the Public Map Viewer is a web-based mapping application designed to assist the general public with displaying and querying data related to gas transmission and hazardous liquid pipelines, liquefied natural gas plants, and breakout tanks that are under Department of Transportation (DOT) Pipeline and Hazardous Materials Safety Administration (PHMSA) jurisdiction. The NPMS also notes that the application does not contain distribution or gas-gathering pipelines.

Although no evidence of utilities was found at either of the two sites, this does not guarantee that no utilities are present. An extensive Subsurface Utility Engineering (SUE) investigation was not part of this contract and was not performed at the two sites. The Construction Contractor will need to contact the utility providers at least 48 hours prior to the commencement of any construction work. It is the responsibility of each individual utility owner to remove, relocate, and protect their respective utility.

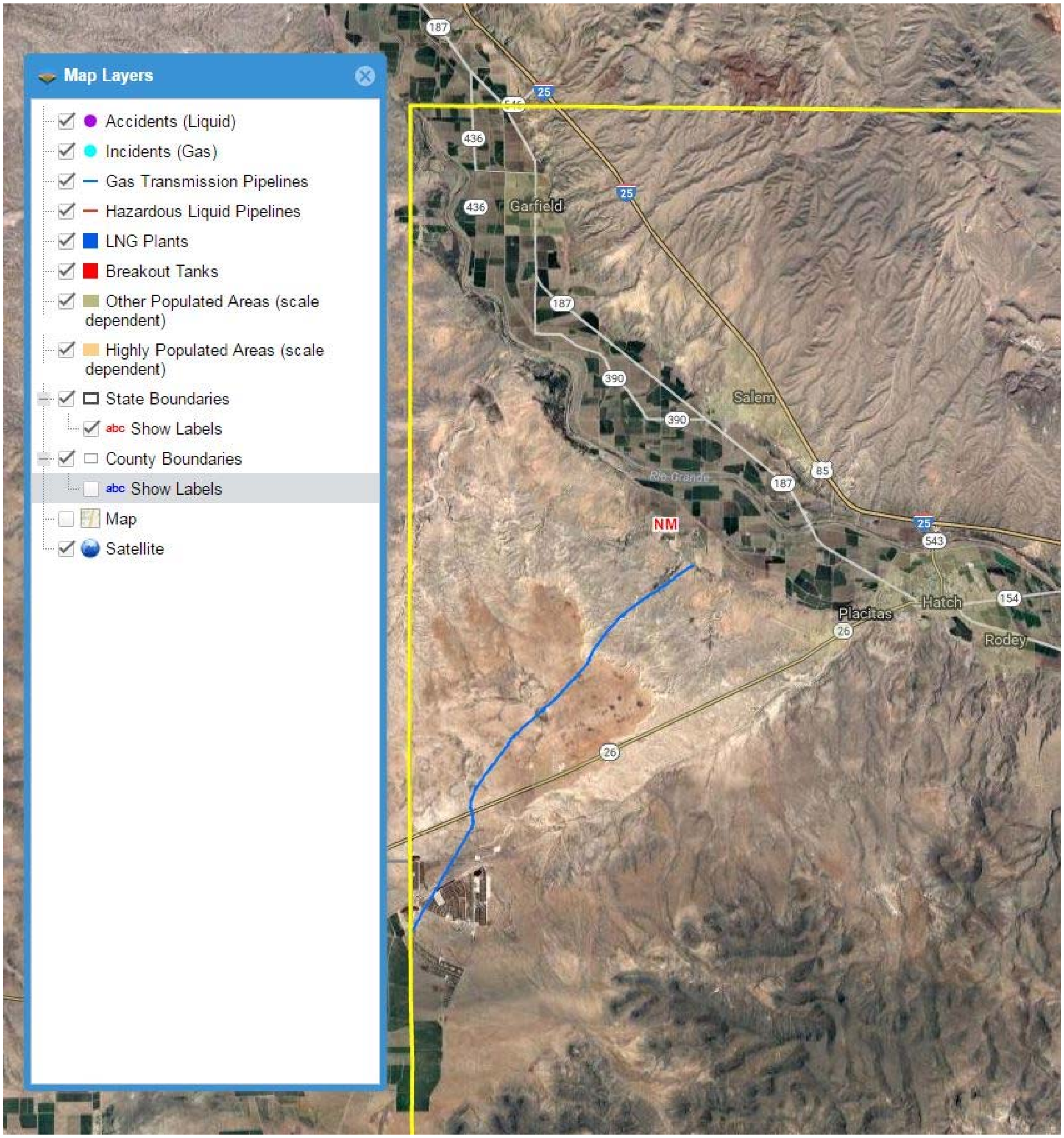


Figure 4-1. NPMS Public Map Viewer Near Project Site

5.0 HYDROLOGIC AND HYDRAULIC ANALYSES

Extensive hydrologic and hydraulic (H&H) analyses were performed as part of Task Order IBM16T0018, Channel Maintenance Alternatives and are documented in a separate report entitled *Hydrologic & Hydraulic Sediment Trap-Basin Analysis*. The H&H report is included in Appendix A of this report.

In summary, the H&H report concluded that a sediment trap system consisting of a basin-based trap would likely have superior performance to that of a mesh-based trap. This conclusion was based on predicted sediment trapping efficiency, structural design considerations, ease of maintenance, and scour potential. Based on H&H findings, the URS team determined that a sediment trap basin is the preferred channel maintenance alternative for both Thurman I and II Arroyos.

The sediment basin system consists of deepening the arroyo channel to construct a basin for sediment collection. The basin is designed to provide sufficient time for sediment to settle out of arroyo flows and to be deposited at the bottom of the channel. The basin is sloped towards the river so that the larger size sediments are deposited at the beginning of the basin, and progressively finer particles are deposited further downstream. The sedimentation basin side-slopes are laid back at a minimum of 3H:1V to maintain long-term stable slopes. The downstream termination of the basin is accomplished by constructing a reinforced-concrete retaining wall ("basin end wall") near the confluence with the Rio Grande. The end wall will be approximately 4.4 to 5.7 feet above the basin finish elevation to provide freeboard for the 100-year storm flows in the arroyo. The end wall serves as an overflow weir when higher flows occur, or when significant volumes of sediment have already been collected in the basin that permit less stormwater storage. Scour protection would be provided on both the upstream and downstream sides of the end wall.

URS proposed the change to a sediment basin system to USIBWC at the 60% design submittal, and USIBWC concurred with the design change. Consequently, the sediment trap basin system is the selected channel maintenance alternative for both arroyos on this project. Details of the sediment trap basin designs are provided in the subsequent sections of this report.

6.0 GEOTECHNICAL DESIGN

6.1 Overview

URS did not perform a geotechnical field investigation under this scope of work. In completing the geotechnical design of this project, URS used existing geotechnical information prepared by others and furnished by USIBWC, and has relied on this information as accurate and complete. URS applied engineering judgment in the review of the data based on our experience in the region and with similar geologic materials, but URS does not warrant the accuracy or completeness of information provided by others. The following sections summarize the geotechnical design approach and conclusions for this project.

6.2 Summary of Existing Information

6.2.1 Previous Geotechnical Studies

Existing geotechnical information used in the design of this project included the following previous geotechnical studies:

- S&B Infrastructure, Ltd. in association with Raba-Kistner Consultants, Inc. (RKCI 2008), *Final Technical Memorandum, Geotechnical Explorations of Levee System within the Rio Grande Canalization Project, Hatch-Tonuco Levee Segment*. Volumes I-IIE, August 4, 2008.
- Kleinfelder West, Inc. (Kleinfelder 2010), *Final Geotechnical Report, NMDOT D1310, Hatch Pavement Remediation, Hatch, New Mexico*, April 23, 2010.

A total of 18 test borings from the two previous studies were examined as potentially relevant to this project. Borings are summarized in Table 6-1. The approximate boring locations are depicted on Figure 6-1. Additional discussion on the relevancy of the geotechnical borings is provided in subsequent sections.

6.2.2 Groundwater Information

Available groundwater data are primarily historical information, including: (1) water levels encountered during drilling by RKCI 2008 and Kleinfelder 2010; (2) piezometers installed by RKCI 2008; (3) USIBWC well data; and (4) U.S. Geological Survey (USGS) well data.

Note that the majority of existing wells are located at a great distance from the site, particularly the USIBWC wells located along the Rio Grande approximately 8,800 to 9,900 feet downstream (east) and approximately 21,800 to 40,800 feet upstream (west) of the site. Due to the great distance from the site, groundwater elevations at the USIBWC wells are not expected to reflect actual groundwater elevations at the site. However, URS expects the trends in groundwater levels exhibited at each well to be indicative of the groundwater trends at the site and along the Rio Grande as a whole. Additional discussion on the relevancy of the well data is provided in subsequent sections. Approximate locations of USIBWC and USGS groundwater wells are shown in Figure 6-2.

For the purposes of this report, maximum ground water elevation at both Thurman I and II Arroyos was assumed to be at El. 4058.0.

6.2.3 Groundwater Conditions

6.2.3.1 River Stage Data

In the Rio Grande Valley, the flow regimes of the Rio Grande are generally differentiated by irrigation season and non-irrigation season. During the irrigation season, which typically runs from March 1 through October 31 (but may vary during dry years with a decrease in water supply), the U.S. Bureau of Reclamation releases flows from the upstream flood control structures to provide irrigation water for crops downstream. This ultimately results in higher river flows and river stage elevations during the irrigation versus the non-irrigation seasons. The Contractor should be fully aware of the irrigation season schedule prior to beginning any construction activity.

Due to the proximity of the site to the Rio Grande, river stage data were examined as part of the groundwater evaluation to develop an understanding of river fluctuation and degree of communication between river levels and groundwater levels. The nearest upstream river gage is below Caballo Dam ("CAAN5") and the nearest downstream gage is at Hayners Bridge near Rincon ("RHB5'). Gages CAAN5 and RHB5 were located approximately 20.5 and 11.5 miles from the project site, respectively. Approximate locations of the river gages are shown in Figure 6-3. River stage data from the nearest gaging stations are summarized in Table 6-2. Plots of the data are presented in Figures 6-4 and 6-5.

URS examined U.S. Army Corps of Engineers (USACE) 1996 data (*Fixed-Bend Cross Sections HEC-2, Rio Grande Canalization Improvement Project, Percha Diversion Dam, New Mexico, to American Diversion Dam, Texas*, July 1996) and Tetra Tech 2015 data to estimate channel geometry at the river gage locations. This information included minimum riverbed elevation (thalweg), riverbank elevation, and both low flow and 100-yr water surface elevations. Similarly, these sources were used to estimate Rio Grande channel geometry at Thurman I and II Arroyos. This information was used to provide a reference point for the river gage data, and to permit interpolation of river data at the project site. Interpretations were limited by the fact that the USACE 1996 and Tetra Tech 2015 data did not extend fully to the upstream river gage (located approximately 6,000 ft north). Another limitation was lack of definitive river gage datum; several online resources had differing datum elevation or no datum elevation. URS made a best estimate from available data in selecting datum elevation. The graphical data are presented in Appendix B.

In general, river water surface elevations appear to hold relatively constant throughout the non-irrigation season, with "static" readings of approximately Elev. 4142.5 and 4009.5 feet for gages CAAN5 and RHB5, respectively. While elevations vary considerably during the irrigation season, a recurring "static" peak can be observed at approximately Elev. 4149 and 4012 feet for CAAN5 and RHB5, respectively. Corresponding irrigation season levels are approximately 6.5 and 3.5 feet higher than the non-irrigation season levels at each gage, respectively.

6.2.3.2 Short-Term Data from Borings

Complete details of groundwater conditions encountered during drilling of borings associated with previous studies are provided in Table 6-3. In summary, groundwater was observed in all RKCI 2008 borings at the time of drilling. The depth varied from approximately 6.5 feet to 13.5 feet below ground surface (bgs). Maximum and minimum groundwater elevation encountered in these borings was Elev. 4054.7 and 4051.6 feet above mean sea level (ft MSL), respectively. These borings were performed in March 2008, which is during the irrigation season when the Rio Grande levels – and by association groundwater levels – would be expected to be higher than yearly average.

Groundwater was not observed in Kleinfelder 2010 borings. This is likely due to both the distance of these borings from the Rio Grande, and termination elevations of the borings being approximately 10 feet higher than where groundwater was encountered in the RKCI 2008 borings.

Note these observations represent groundwater conditions at the time of the field exploration and may not be indicative of other times or at other locations. Groundwater conditions can change with varying seasonal and weather conditions and other factors.

6.2.3.3 Long-Term Data from Monitoring Wells

Monitoring well data dating back to as early as 1994 and 2013 for USGS and USIBWC wells, respectively, were examined and are plotted over time in Figures 6-4 and 6-5. Data plots of wells considered to be most relevant to the project are shown as solid lines, while other wells examined are shown as dotted lines. Ground surface elevation and initial reading (or reading at time of drilling) for each boring location and well are shown as point values. River stage data are also presented. Note that water level during drilling was lower than initial well reading in some cases for USIBWC wells; URS believes this may be attributed to rapid rise of river levels (and hence groundwater levels) shortly after installation, and/or delayed water entry associated with lower-permeability soils.

USGS wells considered to be most relevant to the project include wells "USGS-H-13" and "USGS-.434" (abbreviated to the last digits of the USGS identification number). USGS-H-13 is located approximately 6,600 ft upstream of Thurman II Arroyo, while USGS-.434 is located approximately 3,100 ft downstream of Thurman I Arroyo. Both are located within 200 feet of the Rio Grande banks. The USIBWC wells considered to be most relevant to the project include wells RS-MW-6 and RS-MW-7, which are both located downstream of Thurman I Arroyo by approximately 8,800 and 9,900 feet, respectively, and within about 400 feet of the Rio Grande banks.

Examination of the long-term data indicates that, overall, the amplitude of groundwater level fluctuations generally does not exceed about 10 feet. The amplitude of fluctuation is closer to about 5 feet for USGS wells. Nearly all of the wells exhibited cyclic water levels that correlate very strongly with timing of spikes in river stage data, confirming that groundwater is strongly influenced by water levels in the Rio Grande. The delay in communication with river and groundwater was difficult to assess due to infrequency of readings in some piezometers, but URS expects the response to be fairly rapid based on expected high permeability of subsurface soils, and could range from as little as days to weeks based on the data. One exception was USGS-.434, which did not exhibit a strong cyclic response, and may be due to the presence of more clayey (lower permeability) subsurface soils at this location.

6.3 Site Surface Conditions

The Thurman I and II Arroyos sites are located south of the foothills of Redhouse Mountain, south of U.S. Hwy 85 and north of the Rio Grande. The area is generally undeveloped floodplain area and farmland. A USIBWC levee access road, part of the Hatch-Tonuco levee reach, crosses the arroyo alignments at a nearly perpendicular orientation, and is located approximately 400 to 500 feet north of the Rio Grande.

The topography of the site is relatively flat and gently slopes towards the Rio Grande. Ground surface elevations range from about Elev. 4060 to 4065 feet on the upstream end of Thurman I and II Arroyos at the northern limit of the USIBWC right-of-way (ROW). At the confluence of the arroyos and the Rio Grande, located approximately 500 to 600 feet south of the ROW, ground elevations range from approximately Elev. 4060 to 4062 feet.

The ground surface in Thurman I and II Arroyos is sparsely vegetated and mostly barren. The ground surface is relatively rocky in the arroyos, particularly at the confluence of the Rio Grande where sediment has accumulated. Visible particles generally consist of rounded coarse gravels and cobbles carried downslope from the mountains and foothills. Particle sizes generally grade larger at the confluence of the Rio Grande, where a relatively high occurrence of boulder-size particles was observed by URS.

6.4 Subsurface Characterization

6.4.1 <u>Geology</u>

According to published geologic mapping by the New Mexico Bureau of Geology and Mineral Resources (2003), the site is primarily underlain by the Quaternary-aged alluvium (Qa). Alluvium consists of floodplain deposits of the Rio Grande and contains varying proportions of sand, silt, clay, and gravel.

The specific project setting is located along the levee of the Rio Grande which was channelized in the 1930's as part of a valley-wide flood control and irrigation project. According to RKCI 2008 (after Dena, 2000 and Doser et al., 2001), the Rio Grande Floodplain deposits represent the most recent incision of the Rio Grande into the Camp Rice formation during the late Pleistocene through the Holocene Epochs. The Rio Grande Floodplain deposits are reported to be up to 200 feet thick and are comprised of distant and local source sediments ranging from gravel to silt size. Crevasse splays are the typical deposition pattern encountered on the Rio Grande floodplains, which are comprised of thin fans of sand and silt spread across the floodplain during a break in a natural or artificial levee caused by a flood event. The splay grows uniformly progressively coarser and sandier until channels are cut into the top of the splay. These channels are then filled with coarse sediment that becomes progressively finer-grained upward, opposite the majority of the splay deposition. These channels are discontinuous, often become commingled, and are difficult to correlate across short distances.

6.4.2 Near-Surface Soil Mapping

Near-surface soil maps published by NRCS Web Soil Survey indicate several soil units are present at each site. "Near-surface" refers to soils in the upper 80 inches (6.7 feet) bgs. Mapping was originally performed in the 1950's to 1970's with periodic updates by NRCS (formerly SCS), but changes to near-surface conditions resulting from recent human activity or on-going environmental/geologic processes may not always be captured by these maps. The mapped soil units along the arroyo alignment for each site are listed below. Note that other soil units are mapped in the project area, but are not discussed herein.

- Thurman I Arroyo:
  - Brazito loamy fine sand, 0 to 1% slopes MLRA 42.2 (Br).
  - Riverwash (RE).
- Thurman II Arroyo:
  - Belen loam (Be).
  - Brazito very fine sandy loam, thick surface (Bs).
  - Riverwash (RE).

Published properties of these soil units are provided in Tables 6-4 and 6-5. The intent of presenting this information is to identify potential variations between the relatively widely spaced existing borings.

6.4.3 Properties of Arroyo Sediments

Tetra Tech 2015 collected a bulk sample to represent the sediment gradation delivered by Thurman I Arroyo to the Rio Grande confluence. The sample was designated as Pebble Count PC3, and was taken from the surface of the Thurman I Arroyo fan. The sample included cobble- and gravel-size particles, with the approximate grain size distribution presented in Table 6-6.

Photographs of the sediment surface taken by URS staff during a site visit on October 20, 2016 provide confirmation of the coarse-grained nature of accumulated sediments in both Thurman I and II Arroyos. However, the URS photographs indicate even larger particles are present in the fans of both arroyos, in some cases appearing to be upwards of 12 to 18 inches in diameter or larger (boulder-size particles). These areas are presented in Photos 6-1 and 6-2.

6.4.4 Subsurface Stratigraphy

Generalized subsurface stratigraphy is based on the nearest borings to each specific arroyo, which included only RKCI 2008 borings. Conditions in the Kleinfelder 2010 borings were found to be appreciably different and are not believed to be representative of conditions in the portions of arroyos located within the USIBWC ROW. In the event that this evaluation needs to consider conditions in the arroyos outside USIBWC ROW and nearer to U.S. Hwy 85, the Kleinfelder 2010 data should be revisited.

Photo 6-1. Example of deposited sediment in Thurman I Arroyo.

Photo 6-2. Example of deposited sediment in Thurman I Arroyo.

The generalized subsurface stratigraphy and measured soil properties within each stratum area are shown in Tables 6-7 through 6-10. Borings on the opposite side of the Rio Grande from the arroyos (HT-162, HT-163, and HT-164) were assumed to be representative of conditions on the downstream end of the arroyos near the Rio Grande, while the other nearby borings (HT-74, HT-75, HT-76, and HT-77) were assumed to be representative of conditions on the upstream end of the arroyos. In general, similar soil layering was observed at all locations. However, it is unknown whether theses borings are representative of the actual subsurface conditions at the arroyos due to the distance from actual arroyo alignments (i.e., 200- to 600-foot offsets). Also, the "upstream" and "downstream" borings are spaced approximately 800 to 900 feet apart, and it is unknown whether subsurface conditions near the middle portions of the arroyo alignments are similar. Consequently, URS suggests that additional pre-construction geotechnical data (borings, field testing, etc.) be conducted to validate the assumptions contained herein.

The subsurface stratigraphy presented below does not account for the arroyo sediments, which includes many large cobble- and boulder-size particles. These materials were not sampled in any of the borings, and the depth/thickness of these materials are not known. Therefore, the generalized subsurface stratigraphy should be considered as representative of conditions "outside" the arroyo channels. For the purposes of this report, URS has assumed the upper 5 feet of soil "inside" the arroyo channels (bottom and slopes) consists of arroyo sediments, believed to be a mixture of gravel- to boulder-sized particles in a silty sand matrix. Further discussion and recommendations related to stratigraphy are presented in subsequent sections.

6.5 Seismic Design Considerations

6.5.1 Site Seismic Classification

Site soils can be classified as Site Class E ("Soft Soil") according to the 2012 International Building Code (IBC) - Section 1613.3.3 and ASCE 7-10, Chapter 20, Table 20.3-1. Site Class E is for soft/loose soil profiles with an average SPT N-value less than 15 averaged over the upper 100 feet bgs. While the maximum exploration depth of the considered borings was only 20 feet, URS conservatively assumed conditions below 20-foot depth were similar. Additional geotechnical explorations could potentially justify use of improved Site Class designation.

6.5.2 Site Coefficients

In accordance with Section 1613.3 of the IBC, the spectral response accelerations for the "Risk-Targeted Maximum Considered Earthquake" (MCER) were obtained from the USGS website. These design parameters are summarized in Table 6-11.

Based on the mapped acceleration parameters, the site is classified under the Seismic Design Category C, which indicates a Low to Moderate Seismic Risk Level per Section 11.4 of ASCE 7-10. The mapped acceleration parameters mentioned above are for information purposes only and will need to be confirmed by the Project Structural Engineer-of-Record for final design.

6.5.3 Liquefaction Potential

As required per ASCE 7-10, liquefaction potential was evaluated for the 2% 50-year event (2,475-year mean return period). A de-aggregation of seismic hazard was performed using the USGS' online 2008 Interactive De-aggregation tool. The analysis provides the moment Magnitude (M) and Peak Horizontal Ground Acceleration (PGA) for the NEHRP "BC-rock" boundary, which corresponds to a site underlain by rock. The analysis used the default value of 760 m/s (2,493 ft/s) for the average shear wave velocity over the upper 30 meters of the site subsurface (Vs30). Due to the proximity of the sites, the parameters were very similar and considered equivalent, and are shown below.

- M (mean) = 6.01; and
- PGA = 0.109 g.

In accordance with the NEHRP 2009 procedures, the BC-rock PGA value was site-adjusted to account for actual types underlying the site. The adjustment was based on a Site Class "E" and Risk Category "I/II/III" (non-essential facilities), and performed using the USGS' online Seismic Design Mapping tool. The resulting amplification factor of 2.43 yields a site-adjusted PGA_{design} = 0.261.

A liquefaction triggering analysis was performed based on SPT N-value according to methods published Idriss and Boulanger (2008). The triggering analysis was performed for worst-case conditions at boring HT-75, which included loose sands below the groundwater table with low fines content. Blow counts ranged from 2 to 8 bpf.

The analysis indicates that the factor of safety (FOSL) against liquefaction is greater than the minimum value of 1.2 at most depths, with the exception being a relatively thin zone (3.5-ft thick) between about Elev. 4050 and 4054 feet where the FOSL is less than 1.0. This zone was very loose with N = 2 bpf. Estimated liquefaction-induced lateral displacement and settlement within this zone are 2 to 3 feet and approximately 2 inches, respectively.

The analysis results suggest the site has zones that may be susceptible to liquefaction. However, the likely extent of liquefaction is limited, and the consequences of occurrence are relatively low due to the nature of proposed construction (i.e., slopes and walls less than 10 feet high without habitable structures). In the event of an earthquake causing liquefaction, URS believes the cost of repairs would be significantly less than the cost of installing pre-earthquake liquefaction mitigation measures. Therefore, URS believes that liquefaction mitigation measures are not warranted for this site.

6.6 Arroyo Channel Design Considerations

6.6.1 General Description

Current URS design drawings indicate the existing arroyo channels will be overexcavated to provide increased flow capacity and remove accumulated sediments. The channel bottom for Thurman I Arroyo will be excavated to approximately 1 to 2.5 feet below existing grade, and for Thurman II Arroyo, to approximately 2 to 4 feet below existing grade. The cross-section of the channel will be widened to a maximum bottom width of 150 feet for each arroyo, with permanent excavation slopes inclined at 3H:1V. Maximum channel slope heights are approximately 9.5 and 7 feet for Thurman I and II Arroyos, respectively. Excavated channel lengths of Thurman I and II Arroyos will be 375 and 400 feet, respectively.

6.6.2 Stability of Permanent Slopes

URS performed slope stability analyses to confirm the stability of the proposed channel cross-section geometry. URS adopted USACE design criteria for various loading conditions and minimum factors of safety (FOS):

- End of Construction: FOS ≥ 1.3;
- Steady-State Seepage: FOS ≥ 1.5; and
- Sudden Drawdown: FOS ≥ 1.0 to 1.2.

The slope stability analyses were performed for both Thurman I and II Arroyos using geometry indicated in the 90% Design Drawings. Cross-sections selected for analyses were located at the downstream end of the channels where groundwater was estimated to be the most shallow. Stability analyses were conducted using the computer program Slope/W (2016) developed by Geo-Slope International, Ltd. The program calculates the FOS against slope failure using limit equilibrium procedures and assuming two-dimensional, plane strain conditions. Potential failure surfaces were analyzed using Spencer's method because it simultaneously solves equations of force equilibrium and moment equilibrium.

Subsurface conditions for both sections were conservatively modeled as a loose sand with a total unit weight of 120 pcf, an effective friction angle of 28 degrees, and an effective cohesion of 0 psf. Modeled groundwater elevations were varied based on the specified loading conditions analyzed. Steady-state conditions considered two cases: 1) horizontal phreatic surface coincident with full flow capacity of the channel; and 2) horizontal phreatic surface coincident with the bottom of the channel with no flow. Due to the high-permeability subsurface soils, end-of-construction conditions were considered equivalent to steady-state conditions.

The calculated FOS from the slope stability analyses all met minimum design criteria and are presented in Table 6-12.

6.6.3 Erosion Protection

Channel slopes and bottom should be protected from erosion from both conveyance flows and surface water sheet flow related to precipitation events. Typical categories of erosion protection include vegetated earthen cover or armoring. Armoring options may include riprap, shotcrete, concrete lining, geosynthetic layers, or other proprietary technologies.

The type and extent of required erosion protection is generally dependent upon flow velocity and slope inclination. Erosion protection requirements are established in the Design Drawings and Technical Specifications. Due to cost considerations and design flow velocities, URS recommends riprap as the most feasible erosion protection alternative for this project. If site conditions present potentially unfavorable conditions, an overview of design considerations for erosion protection are presented below.

6.6.3.1 Vegetated Earthen Cover

In general, vegetated earthen cover is limited to cases where flow velocity is less than about 3 feet per second (fps) and slope inclination is 3H:1V or flatter.

Given the arid environment of the site and limited existing vegetation, long-term vegetation establishment and maintenance may not be a feasible option.

6.6.3.2 Chemical Amendment

In areas where sufficiently low-flow velocity is anticipated and vegetated cover is not feasible, chemical amendment may be considered to provide adequate erosion protection. This would generally consist of a minimum 1.5-foot thick surficial layer of chemically-amended on-site fill materials. Chemically-amended fill material could include cement-treatment or lime-treatment of on-site soils. However, site-specific bench-scale treatability studies would be required to determine suitability for chemical amendment and minimum application rates. The resulting mixture should be non-dispersive with a pinhole dispersion rating of ND1 or ND2 (ASTM D4647) and crumb test Grade 1 or Grade 2 (ASTM D6572).

6.6.3.3 Armoring

Armoring should be considered in high flow velocity areas, and other areas of concentrated flow and on sloping areas where sheet flow may cause erosion. Armoring may also be considered for general erosion protection if vegetated earthen cover is deemed not feasible at the site.

Appropriate consideration must be given to provide internal drainage for "impermeable" armoring options, such as shotcrete and concrete lining, to prevent build-up of hydrostatic pressure behind the lining on slopes which can cause failure. Geocomposite drainage layers discharging into weep holes protruding through the armoring near the base of the slopes.

Impermeable armoring on the channel bottom may be subject to buoyant uplift pressures, in which case a drainage relief system should be incorporated. This may include a granular bedding layer as an underdrain system with uniformly-spaced vertical relief vents protruding through the armoring.

Further, armoring options such as riprap and geosynthetics must be designed for particle-size compatibility to prevent internal erosion and soil loss from behind the armoring layers. All armoring options should be extended sufficiently deep below proposed grade to prevent scour and undermining. An anchor trench at the top of the slope should be provided for geosynthetic armoring.

6.6.4 Excavation and Temporary Slopes

Temporary excavations are the sole responsibility of the Contractor. All temporary excavations should comply with OSHA guidelines. Excavations that cannot be sloped to a stable configuration will require shoring. All shoring designs, and any excavations deeper than 20 feet, should be designed by a Professional Civil Engineer licensed in the State of New Mexico.

For planning purposes, URS expects much of the on-site soils to be classified as OSHA Type C soils. These soils require temporary excavation slopes no steeper than 1.5H:1V. Due to the sandy nature of on-site soils with generally minimal fines content (i.e., lack of "binder"), temporary excavations as flat as 2H:1V may be needed to maintain localized stability and minimize raveling, and potentially flatter if groundwater seeps are encountered.

6.6.5 Dewatering

The need for extensive dewatering is not anticipated for much of the channel reconstruction, provided that the construction schedule can be controlled to occur during the non-irrigation season. However, some degree of dewatering should be anticipated to permit construction. A detailed discussion of dewatering design considerations is provided in Section 6.9.

6.6.6 Perimeter Berms

If perimeter berms are required alongside the channel, they should be constructed of earthfill classified as CL or SC in accordance with USCS, have a plasticity index between 8 and 20, fines content of at least 30% passing the No. 200 sieve, and be non-dispersive with a pinhole dispersion rating of ND1 or ND2 (ASTM D4647) and crumb test Grade 1 or Grade 2 (ASTM D6572). Alternatively, berms consisting of lower-plasticity sandy soils (SM, SP, SP-SM) may be used in conjunction with slope erosion protection armoring per Section 6.6.4. Compaction of berm materials should be in accordance with Section 6.9.

Perimeter berms will be subject to the same erosion-protection requirements and permanent slope requirements as channel slopes.

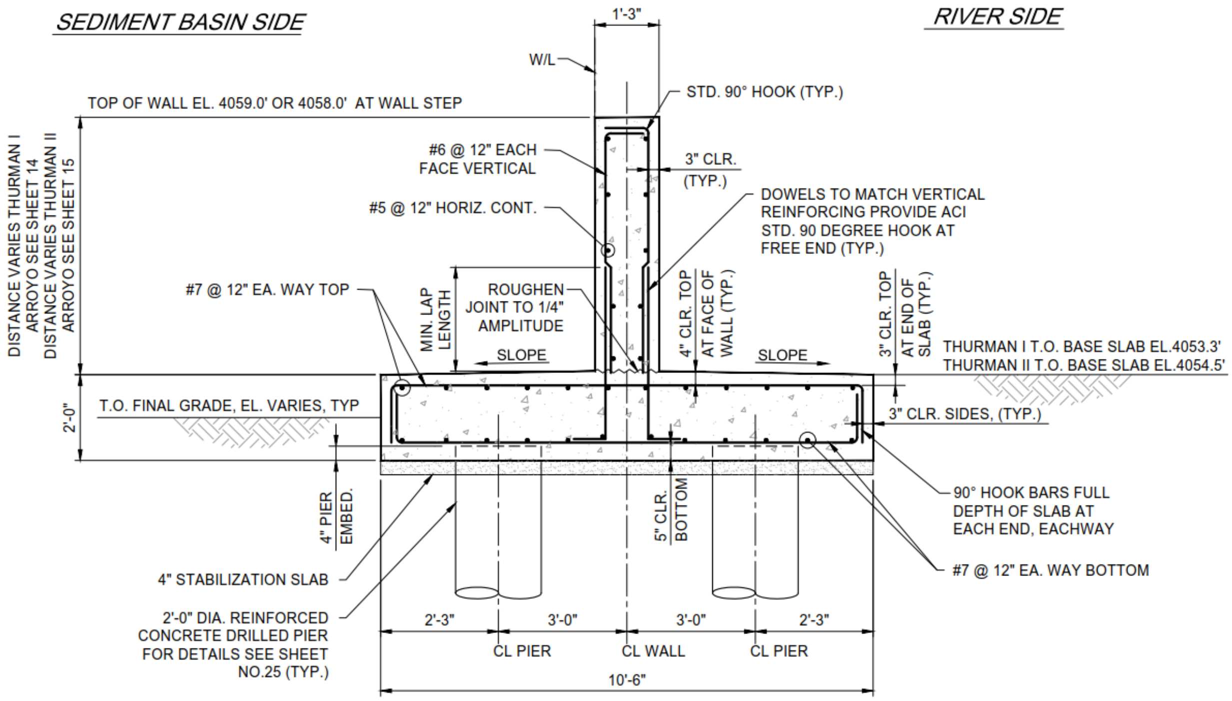
It is anticipated that the perimeter berms will be constructed with on-site excavated soils. The recommended engineering parameters for fill/on-site excavated soils are presented in Section 6.10.

6.7 Sediment Basin Design Considerations

6.7.1 General Description

Current URS design drawings show proposed sediment basin end walls located at the end of each arroyo channel, oriented perpendicular to the channel alignment. The purpose of the end walls is to trap sediment in the channel basin before it can reach the Rio Grande. Both of the two basin end walls are proposed as reinforced-concrete (RC) "T-walls" with waterstops. The top of the middle section of the wall is slightly lower elevation to serve as a weir during high-flow events in the arroyos to permit controlled overflow. The maximum exposed wall heights are 5.7 and 4.5 feet for Thurman I and II Arroyos, respectively.

Basin end walls will be supported on deep foundations due to the risk of scouring during 100-year storm flows in the adjacent Rio Grande. The bases of the end walls are set a minimum of 2 feet below lowest surrounding finish grade, roughly Elev. 4051.3 and 4053.5 ft MSL for Thurman I and II Arroyos, respectively. An example of the end wall is shown in Graphic 6-1.



Graphic 6-1. Drawing of End Wall for Sediment Basin.

6.7.2 <u>Foundation Support Type Considerations</u>

Due to the potential for channel scour/undermining and the desire to minimize excavation depths due to shallow groundwater elevations, shallow foundations were not preferred for this project. Therefore, only deep foundations were considered as suitable support type. Suitable deep foundation types may include driven piles or drilled piers, discussed in further detail below.

6.7.2.1 <u>Driven Piles</u>

The presence of cobble- to boulder-size particles within the upper layers of the arroyo channels may present installation difficulty for driven piles and may possibly result in pile damage and/or early refusal. Consequently, driven pile foundations could require pre-drilling or manual excavation through the cobble/boulder zones to reach proposed tip depths. In such case, the annular space around the pile would be backfilled with controlled low-strength material (CLSM). Cobble or boulder layers may also be present at depth, which may require pre-drilling or driving shoes to minimize pile damage during driving.

6.7.2.2 <u>Drilled Piers</u>

In lieu of driven piles, drilled pier foundations may be used to support the end wall structures. Drilled pier installation will be affected by the presence of dry to saturated, loose coarse-grained soils with minimal fines content, which may result in unstable/caving borehole conditions. Consequently, temporary casing and/or slurry methods of construction may be required to provide stable borehole for drilled piers.

Due to the potential driving difficulties at this site, URS recommends the end wall foundation support system consist of drilled pier foundations.

6.7.3 Foundation Design Recommendations

6.7.3.1 Discussion and Limitations

The existing geotechnical borings available for review only extend slightly below the bottom elevation of the proposed basin end walls. Consequently, unknown subsurface conditions exist below the proposed end wall (i.e., the zone from which drilled pier foundations will derive resistance to loading). Standard engineering practice is to limit foundation design depths to within elevations where sufficient geotechnical information exists. This practice serves to minimize professional liability risk and reduce the risk of construction-related claims. However, based on project criteria, foundations will need to extend below the limits of previous explorations.

Due to the lack of site-specific information, URS has made conservative assumptions of subsurface conditions to develop preliminary recommendations for drilled pier foundations. URS has conservatively assumed the subsurface conditions below the proposed basin end walls consist of loose, saturated sand with a unit weight of 100 pcf and effective friction angle of 28 degrees.

URS recommends these assumptions be validated prior to construction with additional deep geotechnical borings at each basin end wall location to: 1) ascertain if soil strengths are equal to or greater than those assumed for design; and 2) identify any challenging construction conditions that could result in construction claims. Such challenges could include underground obstructions (boulders/cobbles), conditions leading to borehole instability such as excessive drilling fluid loss in permeable sands/gravels or dense layers that preclude temporary casing advancement, and presence of soils requiring different drilling methods (e.g., very soft versus very hard material).

6.7.3.2 Axial Load Resistance

Foundation axial load resistance will be developed primarily by skin friction only due to anticipated small pier diameter and light loads that are not conducive to developing end-bearing resistance. Therefore, deep foundation axial resistance is a function of both pile surface area and embedment depth.

Ultimate axial compression resistance for an individual drilled pier is calculated as follows:

$$Q_{ult} = A_S \times \sum \left(L_i \times q_{s,i} \right)$$

Where: A_s = pier surface area per unit length;
L_i = length of pier penetrating a soil layer "i"; and
$q_{s,i}$ = ultimate unit skin friction of soil layer "i".

The ultimate unit skin friction (q_s) is determined as follows:

$$\text{Cohesive Soils:} \quad q_s = \alpha \times S_u$$

$$\text{Cohesionless Soils:} \quad q_s = K \times \sigma'_v \times \tan\delta$$

Where: α = adhesion factor;
S_u = undrained shear strength;
K = lateral earth pressure coefficient;
σ'_v = effective overburden pressure; and
δ = interface friction between soil and pile.

Based on Allowable Stress Design (ASD) procedures, a factor of safety (FOS) must be applied to the calculated ultimate resistance to obtain an allowable foundation resistance. The allowable resistance must be greater than the maximum unfactored foundation load. Minimum required design FOS values per USACE EM 1110-1-1905 are presented as follows:

- No Pier Load Test: FOS = 3.0.
- Dynamic Load Test: FOS = 2.5.
- With Static Pier Load Test: FOS = 2.0.

Ultimate axial tension ("uplift") resistance can be taken as 70% of the calculated allowable compression resistance. Design allowable axial load resistances are presented in Table 6-13. Note that the allowable axial load resistance values presented in Table 6-13 assume that No Pier Load Test will be performed on this project.

6.7.3.3 Foundation Settlement

Settlement of deep foundations under compression loads is expected to be less than 1% of the pile diameter. Based on preliminary estimates of expected foundation loads and pile sizes, settlement is expected to be between 1/4 inch and 1/2 inch.

6.7.3.4 Foundation Lateral Resistance and Deflection

Lateral responses of driven pile foundations were computed using the program LPILE Version 7 developed by Ensoft, Inc. The program performs soil-structure interaction calculations employing the P-Y curve methodology to provide estimates of lateral deflection and internal shear and bending moment demand. LPILE parameters for design are provided in Table 6-14. The strength contribution of the upper 2 feet was neglected from lateral resistance (represented by a nominal undrained shear strength of 1 psf).

6.7.3.5 Group Effects

Drilled pier foundations will be spaced a minimum of three diameters center-to-center to preclude the need for reducing the capacity of individual foundations related to group effects.

6.7.4 Lateral Earth Pressures

Retaining walls and other buried structures may be subjected to lateral earth pressures resulting from finished-grade elevation differentials acting from one side of the structure to the other.

This includes the end walls that may periodically retain flow-deposited sediments on one side that reach an elevation higher than that of ground surface on the other side.

In general, retaining structures may be classified as either "yielding" or "non-yielding" walls. "Yielding" walls are those which are relatively flexible and free to rotate enough such that the retained soil reaches an active state of limit equilibrium ("active" condition). "Nonyielding" walls are those that do not move or move very little ("at-rest" condition), usually because of restraint at the top so that they are not free to tilt or because of their size, geometry, and rigidity.

The values presented in Table 6-15 are recommended for the design of below-grade and retaining walls, assuming the drainage conditions noted and a horizontal backfill condition. If sloping ground is planned behind retaining walls, earth pressure coefficients should be re-assessed.

6.7.5 External Surcharge Loads

Retaining structures subjected to adjacent external uniform surcharge pressures (e.g., traffic, foundations, etc.) must be designed for lateral surcharge pressures. Lateral surcharge is typically applied as a uniform rectangular pressure distribution extending from the top of the wall to the footing base, and is generally taken as the vertical surcharge multiplied by the appropriate earth-pressure coefficient provided above. Retaining walls subjected to adjacent external point loads or strip loads must be designed to resist lateral forces associated with these loads.

6.7.6 Seismic Loads

Retaining structures may be analyzed for dynamic lateral earth pressures using the Mononobe-Okabe method. The procedure requires application of a horizontal acceleration coefficient equal to half the design PHGA.

6.7.7 Hydrodynamic Loads

Predicted channel velocities are sufficiently low such that hydrodynamic loads are not anticipated.

6.7.8 Uplift and Buoyancy

The proposed end walls may be subject to uplift loads under the wall base and related buoyancy effects. Static uplift pressures can be calculated according to the following equation:

$$\mu = \gamma_w \times z$$

Where: μ = vertical uplift pressure (psf);
γ_w = unit weight of water (62.4 pcf); and
z = elevation differential between bottom of footing and groundwater level (feet).

The end walls on this project are supported by deep foundations as discussed in Section 6.7.3. URS judges that uplift loads and buoyancy are considered part of the deep foundation design and do not pose a significant hazard to the structural integrity of the end wall.

6.8 Dewatering Design Considerations

6.8.1 Overview

Dewatering is only needed if construction-in-the-dry is required. Construction-in-the-wet may be permitted for certain aspects of the proposed construction to limit costs and time associated with dewatering. For example, channel excavation and riprap placement may be permissible in-the-wet provided that slope stability and design grades can be maintained within specified tolerances. This option can be further explored at the request of USIBWC. For the purposes of this report, URS has assumed construction-in-the-dry is required by USIBWC.

6.8.2 General Design Criteria

URS understands that construction-in-the-dry is preferred for the proposed construction, and dewatering may be needed to achieve this. Criteria for the design of dewatering systems is summarized as follows:

- Max. allowable groundwater level during construction: 2 feet below interim grade.
- Minimum factor of safety against heave/blowout: $FOS_{heave} \geq 2.0$.
- Maximum exit gradient: $i_{exit} \leq 0.5$.

Based on these criteria, approximate existing and proposed ground surface elevations and guidelines for allowable groundwater elevations during construction are summarized in Table 6-16.

It is recognized that the Rio Grande generally serves as a recharge feature for adjacent groundwater. Consequently, groundwater levels vary seasonally with river level. The highest levels occur during the irrigation season when upstream control gates are opened to allow water to flow through the Rio Grande. However, in the non-irrigation season, minimal flows occur in the Rio Grande, and groundwater levels are relatively low. Irrigation and non-irrigation seasons are generally defined as follows, but may vary year to year:

- Irrigation Season: March 1 to October 31; and
- Non-Irrigation Season: November 1 to February 28.

The following sections provide technical criteria and recommendations for design and implementation of dewatering systems for use by the Contractor.

6.8.3 Design Groundwater Levels

6.8.3.1 Overview

Existing design river stage and corresponding groundwater elevations at the project site for both irrigation and non-irrigation seasons are presented in Table 6-17. URS developed this criteria based on river stage and groundwater information presented in Section 6.4. Specifications governing the design of dewatering systems will be based on these design water levels.

6.8.3.2 Non-Irrigation Season

If proposed construction occurs during the non-irrigation season, URS expects that dewatering controls may be needed along approximately the downstream 1/3 of the channel alignment. Groundwater levels during the non-irrigation season are approximately 3 to 10 feet deeper than during the irrigation season. Based on URS' judgement, non-irrigation season groundwater levels are generally deeper than the anticipated lowest interim excavation grade of the arroyo channel bed, the exception being at the arroyo / Rio Grande confluence. In some cases where the ground surface is at or slightly above the groundwater table, some groundwater drawdown may be required to provide trafficability. Therefore, dewatering requirements would likely be limited to sumping and pumping to maintain dry excavation surface over the majority of the arroyo alignments, with possible localized need for seepage cutoffs (i.e., sheepile cofferdams) and/or well-point system at the Rio Grande confluence.

6.8.3.3 Irrigation Season

If construction is attempted during the irrigation season, URS expects substantial dewatering controls will be required. Based on river stage information presented in Section 6.4, much of the arroyo alignments will be inundated with water, and groundwater levels will be significantly elevated above the anticipated lowest interim excavation grade of the channel bottom (up to 9 feet). Consequently, URS anticipates that temporary flood control cofferdams, seepage cutoffs, and well-point dewatering systems would be required to provide access to the work area. Groundwater drawdown of up to 2 feet would be required to provide access to much of the areas even outside the arroyo channel banks. Based on the relatively high permeability of site soils and safety considerations, URS believes that construction during the irrigation season may not be economically feasible.

6.8.3.4 Recommendations

Based on comments received from USIBWC for the 60% design, it is our understanding that the Environmental Management Division (EMD) recommends construction not be done during irrigation season. EMD judges that doing so would not only increase construction costs, but would require USIBWC to obtain different water quality certifications and comply with additional environmental regulations, including the Migratory Bird Treaty Act. If construction occurs during the non-irrigation season, dewatering efforts should be relatively minor, except at the confluence of the arroyos and the Rio Grande. However, if the anticipated construction duration exceeds the non-irrigation season or other factors preclude this option, dewatering requirements must be established in the contract. URS expects this would incur significant costs.

Recommended groundwater levels for design of dewatering systems are provided in Table 6-17 for both irrigation and non-irrigation seasons.

6.8.4 Design Hydraulic Conductivity

6.8.4.1 Estimates of Hydraulic Conductivity
Hydraulic conductivity was estimated from correlations with grain-size distribution using several published relationships. All correlations incorporated the RCKI 2008 data exclusively. A summary of the results is provided below.

- The Massmann (2003) procedure is based on the d10, d60, d90, and fines fraction of the soil. The correlations yielded ksat ranging from 2.9E-01 cm/s to 9.9E-02 cm/s, with a logarithmic average of 1.9E-01 cm/s.
- The Hazen (1892) procedure is based on the d10 of the soil exclusively. The correlations yielded ksat ranging from 6.9E-02 cm/s to 2.5E-05 cm/s, with an arithmetic average of 2.4E-02 cm/s.
- The Kozeny-Carman (1956) procedure is based on the void ratio (e), effective particle diameter, and particle shape factor of the soil. The correlations yielded ksat ranging from 8.7E-01 to 1.8E-03 cm/s, with an arithmetic average of 2.1E-01 cm/s.
- Shepherd (1989) presents several published correlations for alluvial soils from earlier researchers. Each correlation relates the d10 of the soil to permeability. The correlations yielded ksat ranging from 4.1E-06 to 6.4E-02 cm/s, with an arithmetic average of 7.5E-03 cm/s.
- One laboratory test on a reconstituted sample (HT-75, 7.5-9 feet depth) of SP material, compacted to dry density of 90.1 pcf at moisture content of 7.2%, yielded a ksat = 8.1E-03 cm/s.

Based on published NRCS soil data (presented earlier), hydraulic conductivity ranges in the upper 5 to 6 feet below existing grade are reported as follows:

- Thurman I Arroyo soil types: ksat = 1E-04 to 4E-03 cm/s
- Thurman II Arroyo soil types: ksat = 1E-06 to 1E-02 cm/s

6.8.4.2 Recommended Hydraulic Conductivity for Design
Based on the foregoing, URS recommends the design hydraulic conductivity values shown in Table 6-18.

6.8.5 Permitting Requirements

The Contractor will be responsible for identifying and securing necessary permits for any dewatering activities.

6.8.6 Dewatering Plan

Existing geotechnical data for the Thurman I and II Arroyos is limited to a few feet (6 to 10 feet) below excavation grades. Assumed soil hydraulic conductivity properties presented in Table 6-18 suggest that a well-point system with a spacing as small as 5 feet will be required to draw down the groundwater level to a depth of 2 feet below excavation grade. This kind of well spacing is likely cost prohibitive and may not be warranted for this project. Consequently, URS recommends that a traditional open sump-and pump-system be considered. Open sumping may be feasible from carefully constructed sumps if the soils underlying the channel and traps consist of clean, well-graded sand and gravel. Based on the limited geotechnical data available, it is assumed that open sumping may be the best dewatering alternative.

The open sumping method typically requires strategically located sumps and drainage ditches on both sides of the excavation and grading the channel such that all the ground and surface water is diverted to the ditches and sumps. Submersible or engine-driven trash pumps will be used to pump the water out of the sumps continuously to have a relatively dry excavation around the project site. The proposed depths of excavation are in the range of 2.5 to 3 feet below existing grade, and the arroyo channels are between 100 and 200 feet wide. Depending on the grading tolerances and the depth to groundwater at the time of construction, it is possible that water could be tolerated in the channel excavation during the construction activities.

6.8.7 Dewatering Pilot Study

URS assumes the typical dewatering depth to be at 2 feet below the proposed excavation grade. Part of the pilot test program could include excavating a 20-foot-wide channel to the planned channel bottom elevation with measurement of flow into the ditch using appropriate measuring techniques. Measurement can also be done using a submersible pump with an attached Electro-magnetic flowmeter. URS judges that a pumping capacity between 200 and 500 gallons per minute (gpm) may be required during the Dewatering Pilot Study for an approximate excavation area measuring 20 feet wide x 20 feet long x 3 feet below the groundwater table.

Based on the measured flow rates, the typical construction details for a Dewatering Plan can be developed during the construction phase of this project.

6.9 Construction Considerations

6.9.1 Site Preparation

The initial step in site preparation should be the removal of all existing structures and large rocks and cobbles from the excavation area extending laterally to a distance at least 5 feet beyond the proposed construction limits. Prior to fill placement, the site should be cleared of the existing vegetation and trees. Any surficial debris, organics, or other undesirable materials should also be removed from the excavation area. Site preparation requirements will be incorporated into the Project Specifications.

6.9.2 Excavations

Temporary excavations are the sole responsibility of the Contractor. Temporary excavations should conform to Occupational Health and Safety Administration (OSHA) requirements contained in 29 CFR Part 1926, Subpart P. For planning purposes, site soils may be classified as Type C per OSHA criteria with a maximum excavation slope of 1H:1V. If insufficient right-of-way is available to provide safe excavation slopes, temporary shoring will be required.

Temporary shoring should be designed by a Professional Engineer licensed in the State of New Mexico. Based on encountered soil types, excavation to the proposed grades generally should be possible using conventional modern earth-moving equipment. Rock excavation or blasting is not anticipated. Excavation requirements will be incorporated into the Project Specifications.

6.9.3 Subgrade Preparation

Subgrade preparation is applicable to the areas of: 1) the channel embankments, and 2) the end wall alignments. The subgrade should be clean and free of unsuitable materials (trash, organics, wood, and other degradable or deleterious materials). Before the start of construction of structures or placement of fill, the subgrade should be proof-rolled with a minimum of six complete passes using a minimum 15-ton (static) vibratory roller or equivalent. If pockets of unsuitable materials encountered in this process cannot be satisfactorily compacted at the subgrade, these soils should be removed and replaced with a minimum thickness of 2 feet of embankment fill or other material approved by the geotechnical engineer. The geotechnical engineer or his designated representative should observe the excavated subgrade for these areas to verify that potentially compressible or weak soils are not present. The geotechnical engineer will determine the extent of loose, soft, or compressible material and recommend appropriate remedial measures. Subsurface stratigraphy obtained from the available borings shows that the top 4 feet are predominantly poorly graded silty sands (SM) with gravel and/or cobbles with varying fines content as presented in Tables 6-7 through 6-10. Such soils can be highly erosion prone and need to be thoroughly compacted per the recommendations of this section to perform as a competent soil mass.

The exposed subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned, and recompacted to at least 95% maximum dry density as determined by Standard Proctor testing (ASTM D 698) at a moisture content within ±2% of optimum moisture to promote bonding with proposed fill. If adequate compaction of fill cannot be achieved, the fill may have to be removed and subgrade stabilization performed at the direction of the geotechnical engineer. Subgrade preparation requirements will be incorporated into the Project Specifications.

6.9.4 Embankment Fill

Embankment fill shall consist of a clean, select, non-expansive fill, free from excess silt, clay balls, or other deleterious matter, and having a plasticity index between 6 and 15, a maximum liquid limit of 30, and a fines content of at least 20% passing the U.S. No. 200 sieve by weight. Generally, soils meeting these plasticity requirements and classified as CL, CL-ML, SM, or SC by the Unified Soil Classification System exhibit the characteristics of a desirable structural fill. URS recommends the excavated site soils along the channel alignment be used as embankment fill, assuming they satisfy all of the engineering properties requirements stated above.

Fill materials should be placed in loose lifts of 6 to 8 inches thick. Each lift should be compacted to a minimum of 95% of the maximum dry density at a moisture content within 2% of optimum in accordance with ASTM D-698. Each lift should be tested and approved prior to placement of the next lift. Compaction tests should be performed at regular intervals to determine the effectiveness of the compaction operations (typically the maximum of one test per lift or every

5,000 square feet of fill surface). Care should be taken to ensure that adjacent structures are not negatively affected by the excavation and/or compaction operations.

6.9.5 Drainage Fill

URS recommends that a 6-inch thick, clean sand backfill be placed under the end wall alignment to act as a drainage layer as well as a relief zone during increased hydrostatic head conditions. Well Graded Sand (SW), free from organic matter and containing less than 5% fines, is recommended as the drainage fill material. Exisiting site soils may satisfy the drainage fill requirements. Sufficient engineering testing shall be performed prior to using existing site soils as drainage fill material.

6.9.6 Structure Backfill

Structure backfill can be considered to be the same as embankment fill for this project.

6.9.7 Quality Control / Quality Assurance

Earthwork: All Earthwork must satisfy the minimum requirements for Quality Control/Quality Assurance as outlined in the Division 31 specifications of the construction bid package.

Concrete and Steel: All concrete must be tested for Quality Control/Quality Assurance as outlined in the Division 03 Concrete specifications and the Division 31 Drilled Piers specification of the construction bid package.

Structural Steel: Structural steel must satisfy the minimum requirements for Quality Control/Quality Assurance as outlined in the Division 03 specifications for Reinforcing Steel and the Division 31 Drilled Piers specification of the construction bid package.

Table 6-1. Summary of Available Groundwater Information by Others

Boring No. (1) (2) (3)	Date Drilled	Latitude (4)	Longitude	Approx. Arroyo Offset (ft) (5)	Approx. Distance from River Edge (ft)	Ground Surface Elev. (ft MSL)	Total Depth bgs (ft)	Initial GWT Depth (ft)	Initial GWT Elev. (ft MSL)
HT-74	3/20/08	32.684009	107.17539	300 E (I)	600 N	4065.1	20	13.5	4051.6
HT-75	3/20/08	32.685004	107.17837	600 W (I)	500 N	4059.7	20	6	4053.7
HT-76	3/20/08	32.685606	107.18154	600 E (II)	500 N	4060.2	20	6.5	4053.7
HT-77	3/21/08	32.686130	107.18473	400 W (II)	600 N	4061.2	20	6.5	4054.7
HT-162	3/25/08	32.683245	107.18173	200 E (II)	200 S	4067.4	20	13	4054.4
HT-163	3/25/08	32.682543	107.17864	500 W (I)	200 S	4066.2	20	13	4053.2
HT-164	3/25/08	32.681497	107.17572	400 E (I)	100 S	4064.9	20	13	4051.9
BH-3	*10/21/08*	*~32.6855*	*~107.1713*	*1,200 E (I)*	*1,500 N*	*~4087*	*15.2*	*Dry to 15.2*	*Dry to ~4071.8*
BH-5	*10/22/08*	*~32.6861*	*~107.1718*	*1,000 E (I)*	*1,700 N*	*~4095*	*15.3*	*Dry to 15.3*	*Dry to ~4079.7*
BH-7	*10/21/08*	*~32.6856*	*~107.1720*	*1,000 E (I)*	*1,500 N*	*~4087*	*15.4*	*Dry to 15.4*	*Dry to ~4071.6*
K-1	*9/1/09*	*32.685527*	*107.17061*	*1,500 E (I)*	*1,600 N*	*4086.6*	*11.0*	*Dry to 11.0*	*Dry to 4075.6*
K-2	*9/1/09*	*32.685555*	*107.17111*	*1,300 E (I)*	*1,550 N*	*4086.7*	*26.5*	*Dry to 26.5*	*Dry to 4060.2*
K-3	*9/1/09*	*32.685694*	*107.17227*	*1,000 E (I)*	*1,400 N*	*4088.5*	*26.5*	*Dry to 26.5*	*Dry to 4062.0*
K-4	*9/2/09*	*32.685722*	*107.17102*	*1,300 E (I)*	*1,600 N*	*4086.8*	*26.5*	*Dry to 26.5*	*Dry to 4060.3*
K-5	*9/2/09*	*32.685805*	*107.17163*	*1,100 E (I)*	*1,600 N*	*4087.4*	*26.5*	*Dry to 26.5*	*Dry to 4060.9*
K-6	*9/1/09*	*32.685972*	*107.17080*	*1,400 E (I)*	*1,700 N*	*4092.6*	*26.5*	*Dry to 26.5*	*Dry to 4066.1*
K-7	*9/2/09*	*32.6860833*	*107.171361*	*1,200 E (I)*	*1,700 N*	*4095.1*	*26.5*	*Dry to 26.5*	*Dry to 4068.6*
K-8	*9/2/09*	*32.6862222*	*107.172222*	*1,000 E (I)*	*1,600 N*	*4095.6*	*26.5*	*Dry to 26.5*	*Dry to 4069.1*

1. 'BH' series borings = NMDOT 2008 as presented in Kleinfelder 2010.
2. 'K' series borings = Kleinfelder 2010.
3. 'HT' series borings = RKCI 2008.
4. '~' = value estimated by URS.
5. bgs = below ground surface
6. MSL = mean sea level
7. Offset relative to Thurman I Arroyo denoted by "(I)" and offset relative to Thurman II Arroyo denoted by "(II)".
8. *Italicized text* indicates borings that were determined to be less relevant to the project site.

Table 6-2. Summary of Nearest River Gage Station Data and Water Level Estimates for Project Site

					Elevations (ft MSL)					Water Surface Elev. Range (ft) (3) (4)			
Name	**Approx. River STA.**	**Latitude**	**Longitude**	**Approx. Distance (miles) (1)**	**River Gage Datum (2)**	**Riverbed**	**River-bank GS (2)**	**Low Flow WSE**	**100-yr WSE**	**Min.**	**Max.**	**Static Low / High**	**Avg.**
River Gage CAAN5: Rio Grande at Below Caballo Dam	5567+00 (4)	---	---	---	---	4139	4146	4139	4140	---	---	---	---
	Actual (estimated) (4)	32.885	107.292	20.5 U/S (II)	4140.9	~4140	~4150	~4143	~4145	4140.6	4149.8	4142.5 / 4149	4144.3
Thurman II Arroyo (at Rio Grande)	4545+63	---	---	---	n/a	4051	4060	4055	4063	---	---	---	---
Thurman I Arroyo (at Rio Grande)	4526+12	---	---	---	n/a	4051	4058	4054	4062	----	---	---	---
River Gage RHB5: Rio Grande at Hayners Bridge Near Rincon	3902+50	32.613	107.019	11.5 D/S (I)	4006.1	4003	4010	4007	4019	4002.2	4019.4	4009.5 / 4012	4009.5

1. (I) = distance from Thurman I Arroyo; (II) = distance from Thurman II Arroyo; D/S = Downstream; U/S = Upstream; GS = Ground Surface.
2. Data source:
   a. CAAN5: Obtained from USGS website which shows datum at Elev. 4140.90 feet (http://waterdata.usgs.gov/nwis/inventory/?site_no=08362500&agency_cd=USGS&).
   b. RHB5: Reported value is average of values from Iowa State University of Science and Technology (1222 m = 4009.4 ft) (https://mesonet.agron.iastate.edu/sites/site.php?station=RHBN5&network=NM_DCP) and Gladstone Family Weather (1220 m = 4002.8 ft) (http://weather.gladstonefamily.net/site/RHBN5).
3. Apparent erroneous readings from the following dates were omitted from calculated values:
   a. CAAN5: 10/28/2011 (14:45 UTC) to 3/16/2012 (18:15 UTC).
   b. RHBN5: 5/30/2012 (12:15 UTC) to 5/31/2012 (15:45 UTC).
4. CAAN5 is located approximately 6,000 feet north of the study area boundary (STA 5567+00). Values estimated from available data.
5. WSE = Water Surface Elevation.

Table 6-3. Summary of Available Groundwater Information by Others

Well No.	Date Drilled	Latitude	Longitude	Approx. Arroyo Offset (ft) (1)	Approx. Distance from River Edge (ft)	Ground Surface Elev. (ft MSL)	Total Depth (ft)	Initial GWT Depth (ft)	Initial GWT Elev. (ft MSL)	Observed GWT Elev. Range (ft)	
										Min.	Max.
USGS Wells											
USGS 324041107100001 19S.03W.05.434	3/15/94	32.678436	107.16730	3,300 E (I)	200 S	4057.35	16.06	4.97	4052.38	4046.01	4054.25
USGS 324122107120802 19S.04W.01.214 H-13	1/24/06	32.690141	107.20293	6,100 W (II)	200 N	4071.04	28.10	15.03	4056.01	4053.16	4060.54
USGS 324021107114301 19S.03W.07.131A	*5/25/04*	*32.673058*	*107.196205*	*5,700 SW (II)*	*3,200 S*	*4069.05*	*16.98*	*15.95*	*4053.1*	*4052.81*	*4059.96*
USGS 324007107095501 19S.03W.08.423 HD-1	*12/13/00*	*32.668638*	*107.16533*	*6,400 SE (I)*	*3,300 S*	*4058*	*60*	*8.0*	*4050.0*	*4042.58*	*4050.0*
USGS 324059107122301 19S.04W.01.3234	*3/19/94*	*32.683105*	*107.20712*	*7,100 W (II)*	*2,000 S*	*4069.05*	*19.89*	*8.61*	*4060.44*	*4051.65*	*4062.08*
USIBWC Wells											
CCA-MW-1	*6/2/13*	*32.71388*	*107.25313*	*23,600 NW (II)*	*400 N*	*4083.29*	*16*	*12.51*	*4070.78*	*NA*	*NA*
CCA-MW-2	*6/2/13*	*32.75191*	*107.25300*	*24,800 NW (II)*	*1,400 N*	*4083.67*	*16*	*11.8*	*4071.87*	*NA*	*NA*
CCA-MW-3	*6/2/13*	*32.72229*	*107.25743*	*26,300 NW (II)*	*800 N*	*4085.2*	*16*	*10.29*	*4074.91*	*NA*	*NA*
CCB-MW-1	*6/2/13*	*32.70332*	*107.25009*	*21,400 W (II)*	*300 N*	*4079.22*	*12*	*13.12*	*4066.1*	*NA*	*NA*
CCB-MW-2	*6/2/13*	*32.70658*	*107.25499*	*23,200 W (II)*	*200 N*	*4081.43*	*16*	*6.78*	*4074.65*	*NA*	*NA*

Well No.	Date Drilled	Latitude	Longitude	Approx. Arroyo Offset (ft) [1]	Approx. Distance from River Edge (ft)	Ground Surface Elev. (ft MSL)	Total Depth (ft)	Initial GWT Depth (ft)	Initial GWT Elev. (ft MSL)	Observed GWT Elev. Range (ft)	
										Min.	Max.
USIBWC Wells (continued)											
CCB-MW-3	6/2/13	32.70068	107.24448	19,500 W (II)	100 N	4070.92	16	7.89	4063.03	4060.27	4065.67
JAR-MW-1	*6/1/13*	*32.74698*	*107.28342*	*37,700 NW (II)*	*200 N*	*4093.43*	*16*	*10.21*	*4083.22*	*4081.29*	*4088.6*
JAR-MW-2	*6/1/13*	*32.74924*	*107.28442*	*38,500 NW (II)*	*200 N*	*4094.32*	*12*	*10.64*	*4083.68*	*4083.31*	*4089.21*
JAR-MW-3	*6/1/13*	*32.74785*	*107.28341*	*37,900 NW (II)*	*300 N*	*4093.04*	*12*	*10.81*	*4082.23*	*4082.26*	*4088.86*
RS-MW-1	6/24/13	32.67586	107.13010	14,600 E (I)	250 S	4048.02	16	2.75	4045.27	4038.16	4045.27
RS-MW-2	6/24/13	32.67510	107.12796	15,300 E (I)	400 S	4051.89	16	2.73	4049.16	4040.24	4049.16
RS-MW-4	5/31/13	32.67458	107.13096	14,450 E (I)	100 N	4045.13	16	2.81	4042.32	4034.88	4043.89
RS-MW-5	6/24/13	32.67166	107.12328	17,000 E (I)	250 S	4043.14	16	1.76	4041.38	4033.78	4041.58
RS-MW-6	5/31/13	32.67889	107.14799	8,800 E (I)	400 S	4048.94	12	8.96	4039.98	4039.39	4045.09
RS-MW-7	6/24/13	32.68116	107.14383	9,900 E (I)	400 N	4050.87	16	6.24	4044.63	4036.68	4050.18
YE-MW-1	*6/1/13*	*32.73695*	*107.27731*	*34,200 NW (II)*	*400 N*	*4090.86*	*16*	8.35	*4082.51*	*4080.53*	*4085.18*
YE-MW-2	*6/1/13*	*32.73448*	*107.27420*	*32,800 NW (II)*	*150 N*	*4090.68*	*16*	*8.19*	*4082.49*	*4079.48*	*4087.48*
YE-MW-3	*6/1/13*	*32.73603*	*107.27448*	*33,300 NW (II)*	*500 N*	*4090.13*	*16*	*8.46*	*4081.67*	*4081.00*	*4084.85*

1. Offset relative to Thurman I Arroyo denoted by "(I)" and offset relative to Thurman II Arroyo denoted by "(II)".
2. GWT = Groundwater table.
3. NA = Not provided to URS.
4. *Italicized text* indicates well data that were determined to be less relevant to the project site.

Table 6-4. Summary of Published Engineering Properties of Near-Surface Soil Units near Thurman I Arroyo
(after NRCS Web Soil Survey)

Symbol	Parent Material	Morphology	Depth Range (ft)	USCS	Pass #200 (%)	Fragments >3 inches (%)	LL	PI	Ksat (in/hr)	Shrink/ Swell Potential	Risk of Corrosion to Steel	Risk of Corrosion to Concrete
Br	Mixed Sandy Alluvium	Floodplains, River Valleys	0-1	SC-SM	25-34 (31)	0	17-24 (21)	2-7 (5)	1E-03 to 4E-03	Low	Moderate	Moderate
			1-5	SM	11-20 (16)	0	0-21 (18)	NP-6 (3)	1E-03 to 4E-03			
RE	Mixed Sand	River Channel	0–1.5	CL,CL-ML	65-90 (78)	0	0-37 (19)	NP-17 (2)	4E-04 to 1E-03	Variable	Variable	Variable
			1.5–5	CL-ML	95-100 (98)	0	20-25 (23)	5-10 (8)	4E-04 to 1E-03			

1. Typical ranges with generally representative values in parentheses.

Table 6-5. Summary of Published Engineering Properties of Near-Surface Soil Units near Thurman II Arroyo
(after NRCS Web Soil Survey)

Symbol	Parent Material	Morphology	Depth Range (ft)	USCS	Pass #200 (%)	Fragments >3 inches (%)	LL	PI	Ksat (cm/sec)	Shrink/ Swell Potential	Risk of Corrosion to Steel	Risk of Corrosion to Concrete
Be	Mixed Clayey Alluvium over Loamy Alluvium	Floodplains	0-1	CL-ML,CL	50-75 (65)	0	0-25 (13)	NP-10 (5)	4E-04 to 1E-03	Low	High	Moderate
			1-2	CH	70-85 (80)	0	50-65 (58)	25-40 (33)	1E-06 to 4E-05			
			2-5	CL-ML,CL	50-100 (75)	0	0-30 (15)	NP-10 (5)	4E-04 to 1E-03			
Bs	Mixed Sandy Alluvium	Floodplains.	0-1.5	CL	65-80 (75)	0	25-30 (28)	9	4E-04 to 1E-03	Low	Moderate	Moderate
			1.5-5	SM,SC-SM,SP	0-40 (20)	0	0-20 (10)	NP-5 (3)	4E-03 to 1E-02			
RE	Mixed Sand	River Channel	0–1.5	CL,CL-ML	65-90 (78)	0	0-37 (19)	NP-17 (2)	4E-04 to 1E-03	Variable	Variable	Variable
			1.5–5	CL-ML	95-100 (98)	0	20-25 (23)	5-10 (8)	4E-04 to 1E-03			

1. Typical ranges with generally representative values in parentheses.

Table 6-6. Approximate Gradation of Arroyo Sediments
(after Tetra Tech 2015)

Percent Finer	Particle Size	
	mm	inches
100	250	9.8
90	140	5.5
60	84	3.3
50	79	3.1
30	63	2.4
10	32	1.3
5	26	1.0

Table 6-7. Subsurface Stratigraphy and Measured Properties near Thurman I Arroyo (Upstream)

Description	Thickness (feet)	Top Elev. (ft)	USCS	LL	PI	Minus #200	Water Content (%)	SPT N-Value (bpf)
Med. Dense Silty Sand	2 to 7	4065.1 to 4059.7	SM	n/a	NP	18	4	9-35 (22)
Loose Poorly-Graded Sand	11 to 18	4058.1 to 4057.7	SP-SM, SP	n/a	NP	3-10 (6)	3-12 (6)	2-14 (9)
Dense Gravel	NE to 2+	4045.1 to NE	GP	n/a	NP	1	9	46

1. Based on borings HT-74 and HT-75.
2. NE = Not encountered.
3. n/a = not measured.
4. Average value in parentheses.

Table 6-8. Subsurface Stratigraphy and Measured Properties near Thurman I Arroyo (Downstream)

Description	Thickness (feet)	Top Elev. (ft)	USCS	LL	PI	Minus #200	Water Content (%)	SPT N-Value (bpf)
Med. Dense Clayey Sand / Sandy Clay	2	4066.2 to 4064.9	SC, CL	25-28 (27)	9-15 (12)	50	5-6	12-16 (14)
Loose Silty Sand	4.5 to 5	4064.2 to 4062.9	SM, SP-SM	n/a	NP	12-25 (18)	1-6 (3)	2-11 (6)
Loose Poorly-Graded Sand	5+ to 13+	4059.2 to 4058.4	SP, SP-SM	n/a	NP	0-5 (2)	2-17 (7)	2-11 (5)

1. Based on borings HT-163 and HT-164.
2. n/a = not measured.
3. Average value in parentheses.

Table 6-9. Subsurface Stratigraphy and Measured Properties near Thurman II Arroyo (Upstream)

Description	Thickness (feet)	Top Elev. (ft)	USCS	LL	PI	Minus #200	Water Content (%)	SPT N-Value (bpf)
Med. Dense Silty Sand	2 to 6.5	4060.2 to 4061.2	SM	n/a	NP	17-43 (31)	6-26 (13)	6-17 (10)
Loose Poorly-Graded Silty Sand	6.5 to 8	4058.2 to 4054.7	SP-SM	n/a	NP	5-9 (6)	3-25 (14)	5-11 (8)
Loose Poorly-Graded Sand	7+ to 10+	4050.2 to 4048.2	SP	n/a	NP	1-4 (3)	11-23 (17)	4-9 (6)

1. Based on borings HT-76 and HT-77.
2. n/a = not measured.
3. Average value in parentheses.

Table 6-10. Subsurface Stratigraphy and Measured Properties near Thurman II Arroyo (Downstream)

Description	Thickness (feet)	Top Elev. (ft)	USCS	LL	PI	Minus #200	Water Content (%)	SPT N-Value (bpf)
Med. Dense Clayey Sand (Fill)	2	4067.4	SC	30	11	40	3	14
Loose Poorly-Graded Silty Sand	5	4065.4	SP-SM	n/a	NP	10	2	3-13 (8)
Loose Poorly-Graded Silty Sand	13+	4060.4	SP-SM	n/a	NP	5	2-6 (4)	4-6 (5)

1. Based on boring HT-162.
2. n/a = not measured.
3. Average value in parentheses.

Table 6-11. Seismic Design Coefficients (Site Class E)

F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}	S_S	S_1
2.471	3.500	0.640 g	0.282 g	0.427 g	0.188 g	0.259 g	0.081 g

1. 'g' denotes the value representing a fraction of gravitational acceleration.

Table 6-12. Calculated Slope Stability Factors of Safety (FOS)

Boundary Condition	Analysis Type	Minimum FOS	Calculated FOS	
			Thurman I	Thurman II
Groundwater Level at El. 4058.0 ft	Steady-State	1.5	1.51	1.75
Groundwater Level at Excavated Channel Bottom (Elevation Varies) (1)	Steady-State	1.5	1.65	1.80
Rapid Drawdown Condition	Rapid Drawdown	1.2	1.25	1.57
End-of-Construction (2)	Steady-State	1.3	1.65	1.80

1. Channel bottom elevations vary between Thurman I (El. 4052.75 ft) and Thurman II (El. 4054.50 ft).
2. Site soil conditions at both Thurman I and Thurman II are assumed to be granular (sandy) in nature, and changes in groundwater level are fairly rapid. Due to the relatively small differences in assumed boundary conditions, URS judges that the end-of-construction case is similar to the long-term, steady-state condition with goundwater level at the bottom of the excavated channel.

Table 6-13. Allowable Axial Capacities for Drilled Piers

Pier Tip Depths, feet (1)	Drilled Pier Axial Capacity, kips (2)		
	24-inch	30-inch	36-inch
45	34	42	50
50	40	50	57
55	46	57	65
60	53	64	75

1. Depth below finish grade of retaining wall base. Finish grade elevations varies between Thurman I (El. 4052.75) to Thurman II (El. 4054.50). Pier tip elevations varies along the retaining wall monoliths as shown on Drawing No. 15 of the 90% Design Submittal Plan Sheets.
2. Scour depths of 5 feet below finish grade has been assumed in all pier capacity calculations.

Table 6-14. Lateral Resistance Parameters for Deep Foundations

Layer Description	Depth (ft)	Thickness (ft)	P-Y Curve Material Model	Eff. Unit Wt. (pcf)	Su (psf)	ε50 (in/in)	φ (deg)	K (pci)
Disregard (1)	0 – 2	2	Soft Clay	110	1	0.02	---	---
Med. Dense Silty Sand (N>10) above GWT	2 – 5	3	Sand (Reese)	110	---	---	30	60
Loose Silty Sand (N<10) above GWT	5 – 7	2	Sand (Reese)	100	---	---	28	20
Loose Silty Sand (N<10) below GWT	7 – 18	11	Sand (Reese)	37.6	---	---	28	25

1. If scour analyses indicate scour depths greater than 2 feet, the depth of the disregard layer should be extended to the scour depth.

Table 6-15. Lateral Earth Pressure Recommendations

Soil Type	Depth Interval (feet)	Total Unit Wt. (pcf) (3)	Friction Angle (deg)	At-Rest Coefficient, K_0	Active Coefficient, K_A	Passive Coefficient, K_P (4)
Flow-Deposited Sediment	---	95	25	0.58	0.41	2.4
Compacted On-Site Soils (1)	---	125	32	0.47	0.31	3.2
Granular Backfill (2)	---	130	35	0.43	0.27	3.6
Native Med. Dense Silty Sand (N>10)	0 to 5	110	30	0.50	0.33	3.0
Native Loose Silty Sand (N<10)	5 to 20	100	28	0.53	0.36	2.7

1. On-site native soils meeting classifications of SM, SP, SP-SM, SC, or CL recompacted to the requirements in this report.
2. Sand or gravel conforming to ASTM C-33 gradations with less than 5% passing the No. 200 sieve and compacted to the requirements in this report.
3. When soils are below the groundwater table, submerged unit weights should be applied and hydrostatic pressure should be added to total lateral pressures.
4. Passive resistance should only be relied upon for stability when scour and/or erosion cannot occur. Passive resistance should be neglected in the upper 2 feet minimum below finished grade.

Table 6-16. Construction Elevations

Location	Thurman I Arroyo		Thurman II Arroyo	
	U/S End	**D/S End**	**U/S End**	**D/S End**
Existing Grade of Channel Banks	±4065	±4062	±4063	±4060
Existing Grade of Channel Bottom	±4057	±4055	±4060	±4056
Proposed Final Grade of Channel Bottom	±4055	±4052	±4056	±4054
Lowest Anticipated Interim Excavation	±4055	±4051	±4056	±4053
Allowable Groundwater Level During Construction to Provide Minimum 2-Foot Separation	<4053	<4049	<4054	<4051

Notes:
U/S = Upstream
D/S = Downstream
Elevations are reported as feet above mean sea level.

Table 6-17. Design Water Levels for Thurman I and II Arroyos

Season	Location	Design Rio Grande River WSE Elevation (ft)	Design GWT Elevation (ft) (1)	Arroyo Channel Bed		Arroyo Channel Banks	
				DD Elev. (ft)	DD Depth (ft)	DD Elev. (ft)	DD Depth (ft)
Non-Irrigation	U/S End (STA. 1+50)	4052	4052	4053	n/a	4061	n/a
	D/S End (STA. 6+50)	4052	4052	4049	3	4058	n/a
Irrigation	U/S End (STA. 1+50)	4058	4058	4053	5	4061	n/a
	D/S End (STA. 6+50)	4058	4058	4049	9	4058	n/a
100-Year Flood	U/S End (STA. 1+50)	4063	4063	(2)	(2)	(2)	(2)
	D/S End (STA. 6+50)	4063	4063	(2)	(2)	(2)	(2)

1. Lower design GWT may be justified if demonstrated by two-dimensional finite-element seepage analysis performed for steady-state conditions in accordance with USACE criteria.
2. Construction activities should be temporarily suspended for flood events due to safety concerns. Dewatering not recommended during flood conditions.

U/S = Upstream
D/S = Downstream
WSE = Water Surface Elevation
GS = Ground Surface
GWT = Groundwater Table
DD = Drawdown

Table 6-18. Recommended Hydraulic Conductivity Design Parameters

Stratum	Description	Elevation Range (ft)	Computed Range of Ksat (cm/s)	Design Kv (cm/s)	Design Kh/Kv	Design Kh (cm/s)
I	Med. Dense Silty Sand	Above 4060	2.1E-04 to 1.1E-01	1.5E-02	2	3.0E-02
II	Loose Poorly-Graded Sand	4060 – 4050	4.1E-06 to 1.3E+00	1.0E-01	1	1.0E-01
III	Loose Poorly-Graded Sand / Gravel	Below 4050	9.5E-04 to 1.3E+00	1.5E-01	1	1.5E-01

1. Additional pre-construction geotechnical exploration (borings, field testing, etc.) is recommended to validate these preliminary design recommendations.

7.0 SEDIMENT BASIN END WALL STRUCTURAL DESIGN

7.1 General

The structural portion of this project consists of design and construction of two reaches of 200'-0" long, drilled-pier supported, reinforced concrete end walls at each Thurman Arroyo I and II sediment basins (400'-0" total linear feet of wall construction). The structural criteria outlined in this section are the basis for analysis and design of these sediment retention basin and drainage improvements. These criteria describe the structural systems and constituent components, list material properties, identify the governing design codes and technical references with which the designs shall comply and present the design load cases for the structural and foundation analyses and designs. Analysis, design, and detailing of these structures generally follow the USACE Manuals and recommendations for hydraulic systems.

7.2 Concrete Design Criteria

Reinforced concrete design is performed in accordance with the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete (ACI 318-11) and USACE Engineering Manual, EM 1110-2-2104.

The design of reinforced concrete structures requires that factored design strength equal or exceed the factored loads imparted on structures per USACE EM 1110-2-2104. The strength reduction and load factors used in the design of conventionally reinforced concrete structures are in accordance with ACI 318-11, except for the strength reduction factor (ϕ) for shear. Strength reduction factor values for design are taken as 0.85 for shear and 0.90 for bending to match the USACE EM 1110-2-2104 uniform load factor method. Load factor values contained in the ACI Code are modified by the strength requirements for reinforced concrete hydraulic structures listed in USACE EM 110-2-2104 (including Change 1). The Single Load Factor Method is used, which includes a load factor of 1.7 for both shear and moment design. Due to the likelihood of water head at the concrete end wall structures, a hydraulic load factor of 1.3 is also applied in the design of these members. Use of the hydraulic factor is intended to reduce stress in concrete reinforcement under service loads and minimize the potential for cracking of the concrete exposed to the environment. Allowable overstress values for individual load combinations are applied to the factored loads for design where applicable. In general, the specified 28-day compressive strength for concrete (f'c) is 4,000 psi for all structures.

7.2.1 <u>Concrete Reinforcing Steel</u>

Reinforcing steel for concrete conforms to ASTM A 615/A 615 M, Gr. 60, f_y = 60 ksi. Per USACE EM 1110-2-2104, the maximum flexural reinforcing requirement is 0.375 ρ_b. Minimum flexural requirements for reinforcing are based on ACI 318-11. Temperature and shrinkage requirements are per USACE EM 1110-2-2104, which specifies the total area of reinforcement in both faces must be equal to or greater than the product of 0.0028 times the gross cross-sectional area of the section. Clear concrete cover is specified as per ACI 318-11 and USACE EM 1110-2-2104.

7.3 Steel Design Criteria

Steel design shall utilize USACE EM 1110-2-2105, the allowable strength design portion of the *American Institute of Steel Construction Manual*, 13th edition and the *Allowable Strength Design Construction Manual*. Load combinations shall be in accordance with USACE EM 1110-2-2105. Commonly specified steels are as follows, unless otherwise noted:

- Plates: ASTM A572, Grade 50;
- Bolts and nuts: ASTM A325, min. 3/4 inch; ASTM A325;
- Anchor Bolts: F1554, Grade 55;
- Structural Tubing and HSS: ASTM A500, Grade B;
- Channels and Angles: ASTM A36; and
- Wide Flange Structural Sections: ASTM A992.

Generally, components that will be exposed to the elements shall be hot-dipped galvanized where specified. Where steel components are fully embedded into concrete, the material shall be uncoated.

7.4 Design Loads

7.4.1 <u>General Loading</u>

Dead loads were determined in accordance with applicable engineering manuals and American Society of Civil Engineers (ASCE) 7-10 and include the self-weight of all permanent construction components, including foundations and walls, overburden pressures, and all permanent non-removable stationary construction. Table 7-1 lists the unit weights used for these materials.

Table 7-1. Unit Weights

Item	Weight [pcf]
Water (Fresh)	62.4
Flow-Deposited Sediment	95
Compacted On-Site Soils	125
Granular Backfill	130
Native Medium Dense Silty Sand	110
Native Loose Silty Sand	100
Riprap	130
Reinforced Concrete (Normal Weight)	150
Steel	490
Gravel	135

A live load surcharge of 250 psf was applied at the sediment basin side of the wall to account for construction equipment which may be required for excavation of impounded sediment in accordance with the operation and maintenance procedures of the sediment basin. See Section 7.6.2 for descriptions of Design Load Conditions that include surcharge.

Structures were designed for lateral and vertical pressures from the surrounding water and soil. Lateral soil pressures have been computed using the single wedge method as described in USACE EM 1110-2-2502, Chapter 3. Hydrostatic loads used for design refer to the vertical and horizontal loads induced by a static water head and buoyant pressures. The end wall monoliths are designed for uplift loading using a pervious uplift condition as no seepage cut-off wall is included.

7.4.2 Wind Loading

Wind loads for the site were calculated using ASCE 7-10 provisions. The wind pressures were calculated with an ultimate level, basic wind speed of 115 mph (3-second peak gust) for a risk category II structure as defined by ASCE 7-10. The exposure category is taken as "C" for the site. By inspection, it was determined that wind loading of the wall will not control the design since the effective pressures of both hydrostatic water load and lateral surcharge due to soil and/or sediment backfill will exert a larger load effect on the wall than the wind load design pressures.

7.4.3 Seismic Loading

The design earthquake load condition for the end wall structures is the Maximum Design Earthquake (MDE), as specified in USACE EM 1110-2-6053. The MDE ground motion has a 10% chance of exceedance in a 100 year period and is specified to have a return period of 950 years. Seismic forces associated with the MDE are considered extreme loads. Earthquake loads are combined with other loads that are expected to be present during routine operations. Seismic design coefficients are given at http://earthquake.usgs.gov/hazards.

Seismic coefficients using Peak Ground Accelerations and the USGS design response spectra based on ASCE 7-10, Site Class "E" values are as noted below for the 2% in 50-year level event which corresponds to a return period of approximately 2,500 years. These values are reduced in accordance with USACE EM 1110-2-6053 for the MDE level seismic event with a 10% chance of exceedance in 100 years which corresponds to a return period of 950 years.

2% in 50 Year	10% in 100 Year (MDE Design)
S_S = 0.259 g	S_S = 0.153 g
S_{DS} = 0.427 g	S_{DS} = 0.254 g
S_1 = 0.081 g	S_1 = 0.041 g
S_{D1} = 0.188 g	S_{D1} = 0.097 g
PGA = 0.107	PGA = 0.061
PGA_M = 0.261	PGA_M = 0.152

Seismic Design Parameters

- Fa = 2.471;
- Fv = 3.500;
- R = 4, Ordinary reinforced concrete shear walls;
- I = 1.0; and
- Cs = 0.064.

7.5 Design Load Parameters

7.5.1 Vertical Loads

The unit weight of the soil is used to calculate the vertical loads exerted by the soil. For soil above the water table, the saturated unit weight is used. When the soil is submerged below the water table, the submerged buoyant weight of the soil is used.

7.5.2 Horizontal Loads

Soil pressure coefficients were used for calculating the lateral loads in accordance with Table 6-14. Horizontal loads are determined using equivalent fluid pressures determined by the combination of soil unit density times the respective pressure coefficient. The "at-rest" pressure coefficient (K_o) is used to determine the forces acting on the wall for the design of the drilled pier foundations due to the minimal amount of movement anticipated at the wall stem. Movements are assumed to be sufficiently small such that active and passive pressures will not develop in the soil.

7.5.3 Seismic Pressure Coefficients

Seismic soil pressures were calculated according to the Mononobe-Okabe formula using:

$$K_h = 0.228 \text{ for soil with } \phi = 28°$$

Seismic lateral loads from the soil are applied to the wall per the Mononobe-Okabe design procedure. Seismic design procedures outlined in ASCE 7-10 (Equivalent Lateral Force Method) were used to calculate the load effects of the seismic acceleration of the concrete wall structure.

For the MDE, the earthquake load condition (E1 Load Case) does not govern the design of any wall segments when considering allowable overstress factors per USACE methodology. Liquefaction potential is not specifically accounted for in the structural design. If liquefaction occurs, it is expected that some instability could occur at the drilled piers due to the increase in unbraced length caused by liquefied surrounding soils.

7.5.4 Erosion Control Protection

Erosion control protection at the pier-supported walls is required in some locations due to the proposed final grade, slopes and scour potential of the hydraulic flows in the arroyo channels. To minimize the possibility of scour at the floodwall foundations, protective rip-rap armoring will be used at the upstream and downstream sides of the end wall in some locations, depending on soil conditions and levee slope. Required scour protection is shown on the Civil Drawings.

7.6 Design of End Walls

7.6.1 Wall Sections

The structural wall design for the project includes two reaches of reinforced concrete, inverted T-shaped wall segments. Each end wall monolith reach, one at each of the two Arroyos, consists of 30'-0" and 40'-0" long monoliths at each segment for a total overall length of 200'-0" at both Thurman I Arroyo and Thurman II Arroyo. The proposed wall segments all have deep foundations consisting of 24" diameter reinforced concrete drilled piers. The drilled piers vary in length between 45'-0" and 60'-0" embedment based on required load conditions including axial and lateral forces. No seepage cut-off wall is required per hydraulic and geotechnical analyses results.

The individual T-wall segments are composed of concrete monoliths separated by 1/2-inch expansion joints with embedded 3-bulb continuous water-stops. The T-wall stem heights vary from approximately 4'-0" to 6'-0" and are all 1'-3" thick. The center portion of the end wall along each reach is depressed 12" lower than the adjacent end walls at the channel sloped banks which creates an overflow weir for base stormwater flows from the arroyos to spill over the end wall.

The concrete base slab is 2'-0" thick and 10'-6" wide at all monoliths. Minimum center to center spacing of the drilled piers is 3 times the pile diameter to preclude group effect reductions which equates to 6'-0" on center minimum spacing. Minimum edge distance to the drilled pier centerline is 2'-3" at all sides. The proposed wall section is shown in Figure 7-1 for the typical end wall and foundation. The finished grades vary at the upstream and downstream sides of the end wall as per the Civil Drawings.

7.6.2 Load Cases

Table 6-15 shows the hydraulic conditions at both Thurman I and II Arroyos. Since the end walls are not intended to be flood protection structures; rather, it is intended for flood-stage water to overtop the wall and the walls to act as a weir, the design hydraulic condition for all walls was taken as the maximum water to top of wall condition. Due to the likelihood of water reaching the top of wall condition, no additional allowable overstress was considered in the design (0% allowable overstress). Since the foundation and backfill soils are assumed to be pervious without seepage cut-off measures, a uniform distribution of uplift pressures which vary linearly from the upstream to downstream sides of the wall was considered.

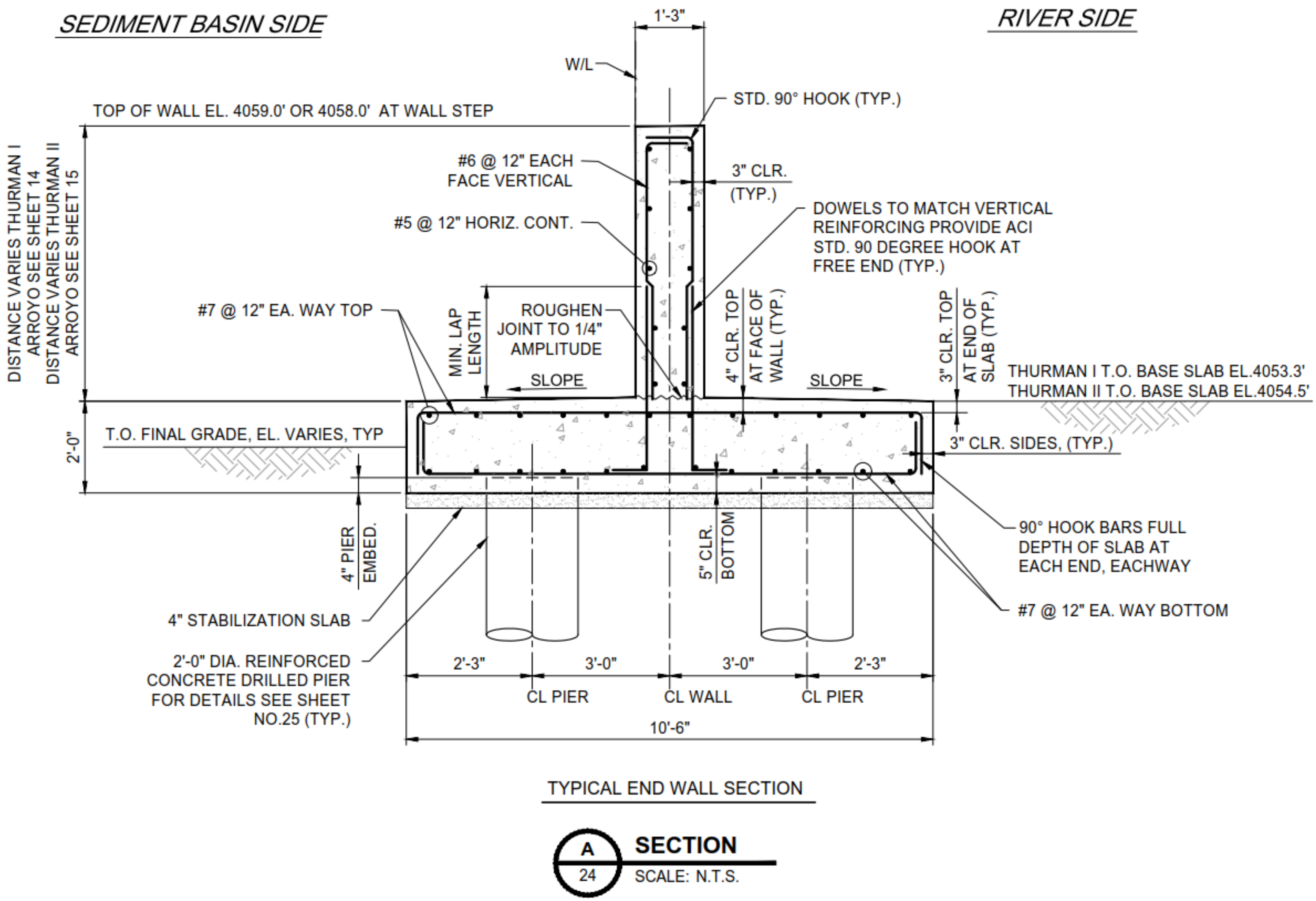


Figure 7-1. Proposed Pier-Supported Floodwall

Retained soil heights at the upstream and downstream side of the end wall vary along the length as the end wall crosses the channel arroyo and sediment basin. Also, due to the varying height of sediment captured over time at the upstream side of the wall, several load conditions were investigated to determine the envelope of maximum load effect on the structures and foundations. The following load conditions were investigated and are summarized in the following table.

Case W1, Water to Top of Wall with Sediment Basin Empty

Case W1 considers water to top of wall at the upstream side of the sediment basin. In this case, soil heights are assumed to be lowest at approximately top of footing elevation at both the upstream and downstream sides of the wall. This case will occur after initial construction and before sediment is trapped behind the wall at the upstream side. Due to the likelihood of water reaching the top of wall condition, no additional allowable overstress is considered in case W1 (0% allowable overstress).

Case R1, Retaining Condition with Sediment Basin Full

Case R1 considers a fully drained condition with the upstream side of the wall filled with sediment to maximum top of wall height. In this case, it is assumed that the sediment basin has become fully impounded with sediment material at the upstream side and the grade at the downstream side of the wall is at top of footing elevation. A surcharge load is also included at the upstream side to account for possible construction equipment required to excavate the sediment from the basin. Including surcharge at the upstream side of the wall creates the largest load effect; therefore, surcharge was not considered at the downstream side of the wall. Since the surcharge loading will be a temporary construction case, an allowable overstress of 16.67% is considered for Case R1. This load case scenario is unlikely to occur assuming that operation and maintenance procedures are followed which require the basin to be excavated of sediment to limit the fill level to 75% of its maximum capacity.

Case R2, Water to Top of Wall with Sediment Basin Full

Case R2 considers water to top of wall at the upstream side of the sediment basin. Additionally, the upstream side of the wall is taken to be fully filled with sediment to maximum top of wall height. In this case, it is assumed that the sediment basin has become fully impounded with sediment material during a flood event in which water will also reach the top of wall elevation. As described in Case R1, this load case scenario is unlikely as maximum sediment fill heights should be limited to 75% of the maximum basin capacity through proper operation and maintenance. Although unlikely, this load case scenario could occur during a flooding event in which the basin is both completely filled with sediment and water reaches top of wall height. No additional allowable overstress is considered for Case R2 (0% allowable overstress).

Case R3, Maximum Retaining Wall Condition at Channel Banks

Case R3 considers the portion of the end wall where the maximum soil heights are retained at both the upstream and downstream side of the wall simultaneously. Near the channel banks, the side slopes increase in height gradually up to a point where the end wall is fully embedded into the bank by 5'-0" minimum at each end. The 5'-0" embedment is used to minimize the potential for hydraulic scour along the bank slopes at the ends of the wall. With soil backfill to top of wall at both sides of the wall stem concurrently, the maximum compression force in the deep-foundations is investigated. This scenario occurs at both Arroyos I and II and therefore no allowable overstress is considered for Case R3 (0% allowable overstress).

Case E1, Earthquake Loading at Full Sediment Basin

Case E1 considers earthquake loading conditions on the wall caused by the backfill soil or sediment fill acting at the upstream side of the wall. In this case, it is assumed that the sediment basin has become fully impounded with sediment material at the upstream side and the grade at the downstream side of the wall is at top of footing elevation. Case E1 is classified as an extreme load condition for the minimum safety factors per USACE EM 1110-2-2502 and is based on the MDE. An allowable overstress of 33.33% is used due to the extreme nature and short duration of the MDE seismic event for Case E1.

Table 7-2. Load Combinations for Structural End / Retaining Wall Design

Load Case	Load Combination	Dead	Hydrostatic	Uplift	Soil	Wind	Surcharge	Seismic	Factored Load	Pile Design Allowable Overstress
W1	Water to Top of Wall with Sediment Basin Empty	X	X	X	X				2.21	0%
R1	Retaining Wall Condition with Sediment Basin Full + Surcharge	X			X		X		1.89	16.67%
R2	Water to Top of Wall with Sediment Basin Full	X	X	X	X				2.21	0%
R3	Maximum Retaining wall Condition at Channel Slope Banks	X			X				2.21	0%
E1	Earthquake at Sediment Basin Side with Sediment Basin Full	X			X			X	1.66	33%

Note 1: Concrete design is performed according to the single load factor method specified in Section 3-3 of USACE EM 1110-2-2104, Change 1, "Strength Design for Reinforced Concrete Hydraulic Structures." See Subsection 3-3.d., for load factors for the Earthquake Condition. For load conditions classified as unusual or extreme, the factored load Uh is reduced by a reduction factor, which is 0.75 for unusual and extreme cases. The reduction factors are incorporated in the single load factors listed in the chart.

8.0 RIPRAP DESIGN

8.1 Data Collection

8.1.1 Site Visit

URS and USIBWC staff visited the area of the junction of each of the two arroyos with the Rio Grande on October 20, 2016. Observations on this site visit pertinent to riprap design included:

- The largest rocks conveyed into the Rio Grande from Thurman I Arroyo and deposited in the fan downstream of the junction appeared to be isolated stones about 18 to 22 inches in mean dimension.
- The largest rocks conveyed into the Rio Grande from Thurman II Arroyo and deposited in the fan downstream of the junction appeared to be much more numerous stones, about 9 to 12 inches in mean dimension.
- The rocks were all rounded, i.e., if used as basin riprap, the roundness of the stone should be considered in design.

8.1.2 Pebble Count

As part of the 2015 study of channel maintenance alternatives within the river reach (Tetratech, 2015), a series of pebble counts were performed. The pebble count for the Thurman Arroyo area is reproduced in Figure 10 of the *Hydrology and Hydraulics Report*. The pertinent information from this count included:

- The mean pebble size was about 7 cm or 2.8 inches;
- The maximum pebble size noted was 300 mm or about 12 inches; and
- The D_{90} was 100 mm or about 4 inches.

8.1.3 Relation to River Flood

The proposed riprap basin is entirely within the left overbank of the Rio Grande. Per the current hydraulic model of the Rio Grande, the left overbank average velocities for either arroyo for the 1% annual chance flood are estimated to be less than 2 ft/sec; much less than the velocities estimated for the flood of the same probability on each arroyo (see the following section). Therefore, the governing hydraulics for riprap design are those associated with flows from the watershed of each individual arroyo.

8.2 Riprap Design

8.2.1 Stone Size

Several steps were taken to calculate a recommended riprap size for each arroyo downstream of the sediment basin. First, a cross section was cut across the channel downstream of each sediment basin. Second, using the results of the 2D modeling discussed in the *Hydrology and*

Hydraulics Report, a maximum flow rate through each cross section was then estimated, and an average slope was estimated through the arroyo channel. These inputs along with cross-section coordinates were used to estimate normal depth and other hydraulic parameters in the cross sections by use of the Alpha method (USACE, 1994, Appendix C), shown in Table 8-1. Finally, these hydraulic parameters were then used to estimate required riprap diameters/gradation based on the methods presented in Table 8-2. These methods follow from various theoretical and empirical approaches and provide a range of potential riprap sizes.

Table 8-1. Modeled Hydraulic Parameters in Thurman I and Thurman II Arroyos Downstream of Proposed Sediment Basins

Parameter	Thurman I Value	Thurman II Value
Average Depth (ft)	3.0	2.9
Max Depth (ft)	3.9	3.6
Average Shear (psf)	2.0	2.0
Max Shear (psf)	2.8	2.6
Average Velocity (ft/s)	7.5	7.6
Max Velocity (ft/s)	9.9	9.3
Bottom Width (ft)	52.3	51.0

Table 8-2. Riprap Sizing Analysis Results

Method	Reference	Required Hydraulic Parameters	Calculated D_{50} (inches)	
			Thurman I	Thurman II
Isbash Method	NEH, eq. TS14C-1 (Technical Supplement 14C)	Velocity	8.51	7.5
NCHRP Report 108 Method	NEH, eq. TS14C-4 (Technical Supplement 14C)	Hydraulic Radius, Slope	8.41	7.7
U.S. Bureau of Reclamation Method	NEH, eq. TS14C-9 (Technical Supplement 14C)	Velocity	16.4	14.4
USGS Method	NEH, eq. TS14C-10 (Technical Supplement 14C)	Velocity	32.1	27.6
Tillatoba Model Study	NEH, eq. TS14C-11 (Technical Supplement 14C)	Depth, Velocity	67.98	59.1
USACE Steep Slope Method	NEH, eq. TS14C-12 (Technical Supplement 14C); USACE, EM 1110-2-1601	Flow, Bottom Width, Slope	9.56	9.1
USACE Method (1970)	Blodgett and McConaughy, 1986	Shear	9.1	8.1
HEC-11 Sizing	Brown, S. A., and Eric S. Clyde, USACE, 1989	Depth, Velocity	10.09	14.5
USACE method (1991) [(1)]	NEH, Technical Supplement 14C; USACE, EM 1110-2-1601	Depth, Velocity	11.4 – 14.3	15.4 – 19.3

1. Two values for each arroyo were calculated using this method based upon stability factors; the larger D50 value corresponds to a rounded rock stability factor (0.375) and the smaller to an angular rock stability factor.

Observations and data from the site visit, pebble count, and riprap sizing analysis were taken into account to recommend an appropriate riprap size for the Thurman I and Thurman II Arroyos.

8.2.1.1 Thurman I Arroyo

According to the riprap sizing analysis presented above, the USACE 1991 method presents a reasonable range of D_{50} values (11.4 to 14.3 inches) after large and small values calculated by use of other methods are excluded. This D_{50} for rounded riprap (14.3 inches) corresponds to a D_{100} of 24 inches. Therefore, because the largest rocks noted on site are within this range (according to the site visit), on-site riprap with a D_{50} of 14 inches is recommended for placement downstream of the sediment basin on the Thurman I Arroyo.

8.2.1.2 Thurman II Arroyo

The riprap sizing analysis for the Thurman II Arroyo resulted in sizes ranging from 7.5 to 59.1 inches. After excluding very large and small values, again the USACE 1991 method suggests a reasonable range of D_{50} values, with 15.4 inches recommended for angular riprap and 19.3 inches for rounded rock. A D_{50} of 19.3 inches corresponds to a D_{100} of 33 inches. This size appears conservative for the following reasons:

- From the site visit and pebble counts, the largest readily identifiable on-site (in the river fan downstream of the junction) stones appear to be approximately 12 inches in diameter;
- There were major storms in 2008 (100-year return period for the storm at the Caballo Dam rain gage, see Table 3 of the *Hydrology and Hydraulics Report*) and 2013 (40-year return period) in the region of the arroyos; and
- It can be assumed that the lack of larger sizes in the river fan are indicative of a relatively low risk of arroyo flows moving stones much larger than D_{50} 12 inches.

Therefore, angular riprap from off site with a D_{50} of 15 inches (slightly smaller than the estimated 15.4 inches) is recommended to protect the Thurman II Arroyo downstream of the sediment basin. For rounded riprap, a D_{50} of 18 inches (slightly smaller than the estimated 19.4 inches) is recommended.

8.2.2 Riprap Placement

According to USACE guidance (1994), riprap should be placed with a layer thickness of no less than either $1.5*D_{50}$ or D_{100}, whichever is greater. D_{100} is estimated per standard gradations in Table 3-1 of the USACE guidance. Table 3 presents the estimated D_{50}, $1.5*D_{50}$ and D_{100} for both arroyos, according to whether angular or rounded riprap is used. Based on these results, it is recommended that the riprap layers for the Thurman I and Thurman II Arroyos have a thickness (T) of 24 inches and 27 inches, respectively.

Table 8-3. D_{50}, 1.5*D_{50} and D_{100} Values for Thurman I and Thurman II Arroyos

Thurman I	1.5*D_{50} (ft)	Round	21.5
		Angular	17.1
	D_{50} (ft)	Round	14.3
		Angular	11.4
	D_{100} (ft)	Round	24.0
		Angular	18.0
Thurman II	1.5*D_{50} (ft)	Round	29.0
		Angular	23.1
	D_{50} (ft)	Round	18.0
		Angular	15.0
	D_{100} (ft)	Round	33.0
		Angular	27

In terms of configuration, the riprap should be placed according to Method C from Plate B-41, (USACE, 1994). The riprap section should begin immediately downstream of the concrete scour protection and extend for nine times the riprap thickness downstream of the sediment basin end walls. A filter layer should be placed underneath the riprap in this section as required.

8.3 Limitations

Quantitative evaluations of hydrological and hydraulic studies are approximate and difficult to determine with complete accuracy. URS has endeavored to apply judgement for this evaluation to the degree practical, while utilizing acceptable design methods and guidelines for this study.

9.0 SEDIMENT REMOVAL

9.1 Overview

The Channel Maintenance Report (Tetra Tech, 2015) stated that during the 2006 and 2013 monsoon season tributary flow events, Thurman I Arroyo and Thurman II Arroyo delivered significant quantities of sediment to the Rio Grande, and islands have formed along the downstream portions of both Thurman Arroyo fans, along with numerous other islands and vegetated bars along the reach. Tetra Tech also reports that additional sediment appears to have been delivered since that time.

A portion of Section C.4 of the Scope of Work for Task Order IBM16T0018 states that URS is to design and provide construction specifications for the removal of localized sediment within the Rio Grande main channel to insure adequate hydraulic capacity. However, USIBWC later requested that the sediment removal within the Rio Grande be expanded to include the Channel Excavation Long for Problem Location 2 as presented in the Tetra Tech report, and to remove as much as 150,000 cubic yards of sediment. The following section describes the method and data used to prepare the sediment removal design.

9.2 Sediment Removal Plan Development

The Rio Grande alignment and stationing as shown on the sediment removal plans is the same as that used by Tetra Tech in the Channel Maintenance Report, and was obtained electronically by URS from the USIBWC. The Channel Maintenance Report stated that the alignment and stationing was based on the base model station line for the RGCP that was prepared as part of the USACE (2007) study. The limits of the Channel Excavation Long extend from upstream River Sta. 4554+50 to downstream River Sta. 4459+00, a length of 9,550 feet. The Tetra Tech report listed the length of Channel Excavation Long in Problem Location 2 as 9,500 feet.

URS used the same 2011 LiDAR data provided by USIBWC to prepare the sediment removal plans. URS also used the parameters established in the Channel Maintenance Report, which stated that the excavated channel would span the entire width of the channel, and the excavation profile would need to have a down-gradient slope and tie into the downstream existing bed profile to avoid creation of a pool/sediment. Additionally, the proposed excavation thalweg profile and cross-section figures in Appendix G of the Channel Maintenance Report were studied to estimate the proposed channel bottom excavation elevations. All cross-sections provided indicated that the proposed excavation bottom profile would be below the existing thalweg profile. However, the 2011 LiDAR data defines the existing thalweg channel very poorly, if at all, since the channel most likely had water in it at the time of LiDAR collection. To estimate a more accurate thalweg profile, URS used the RGCP CMA Study Survey Sections (2014) that were collected by Tetra Tech in preparation of the Channel Maintenance Report. The data for the Tetra Tech surveyed sections were presented as Appendix A – Topographic and bathymetric survey data collected by Tetra Tech in Appendix C – Del Sur Surveying Surveyor's Report of the Channel Maintenance Report.

There are a total of 12 surveyed sections that fall within the extents of the Channel Excavation Long in Problem Location 2. A profile connecting the thalweg elevations at each cross-section location was superimposed onto the design profile of the plans and was then used to compare the proposed bottom of excavation profile to the existing thalweg profile. Each of the surveyed sections was also projected onto the closest cross-section on the cross-section sheets in the plan set to verify the efficacy of the design cross-sections.

Table 9-1 reproduces a portion of Table 8 from the Channel Maintenance Report and shows the length, width, depth, and estimated sediment volume to be removed at Problem Location 2 using the Channel Excavation Long. For comparison, Table 9-2 summarizes the same information as Table 9-1 for the proposed Channel Excavation Long sediment removal as designed.

Table 9-1. Tetra Tech's Excavation Parameters for the Channel Excavation Long Sediment Removal Alternative at Problem Location 2

Excavated Length (ft)	Avg. Excavated Depth (ft)	Avg. Excavated Width (ft)	Excavated Volume (CY)
9,500	3.0	110	126,890

Table 9-2. URS Designed Excavation Parameters for the Channel Excavation Long Sediment Removal Alternative at Problem Location 2

Excavated Length (ft)	Avg. Excavated Depth (ft)	Avg. Excavated Width (ft)	Excavated Volume (CY)
9,550	2.0	185	131,000

[1]Average excavated depth is calculated using the excavated volume divided by the excavated length and average excavated width.

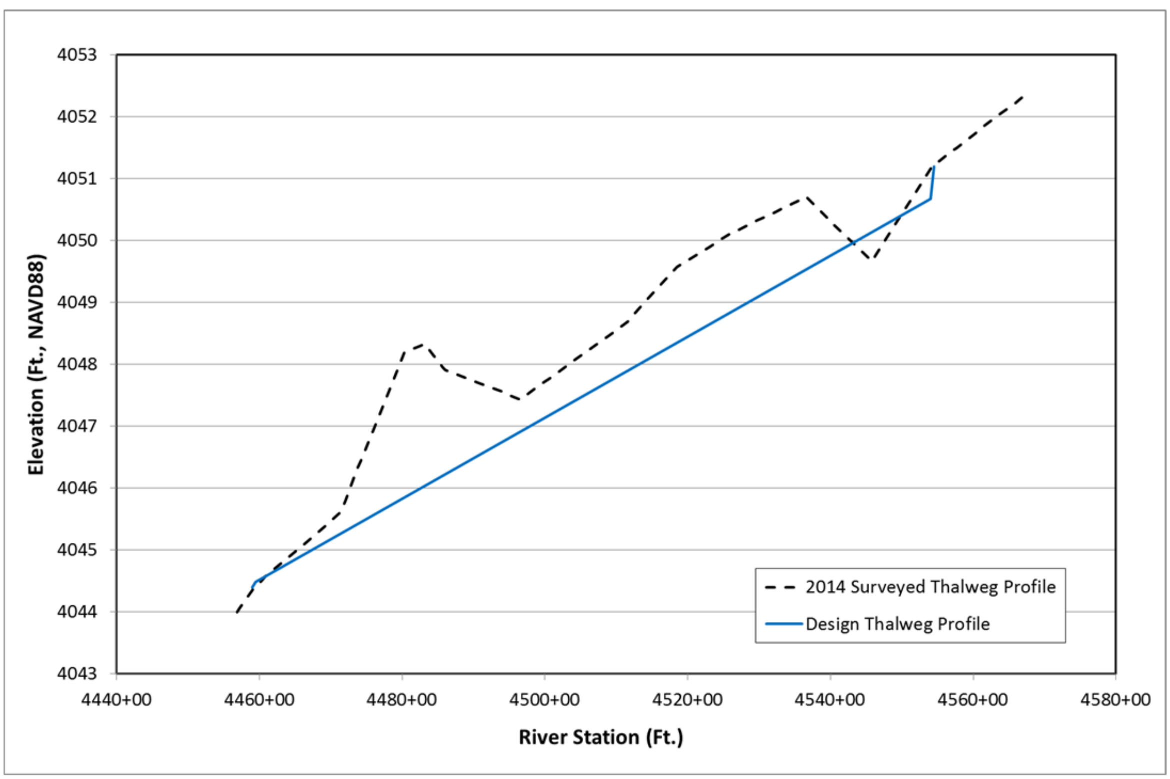


Figure 9-1. Pre-Sediment Removal Thalweg vs. Post-Sediment Removal Thalweg

10.0 RIGHT-OF-WAY/EASEMENTS

It is a requirement of this Task Order that all design work and future construction activities of the sediment basins must be confined to USIBWC right-of-way. Site access for both Thurman I and Thurman II can obtained by using public roads (Hwy 187 to the west and Hwy 26 to the east), and therefore, no access easements or temporary construction easements will be required. Right-of-way data for the sites were obtained in the form of a GIS shapefile from USIBWC.

11.0 CONSIDERATIONS AND ENGINEERING DECISIONS

11.1 Summary

The H&H report in Appendix A presents the detailed studies, modeling, and analysis that were performed under this Task Order and includes the following elements:

- Data Collection;
- Background Studies;
- Soil and Water Assessment Tool (SWAT) Modeling to Estimate Frequency of Larger Diameter Particle Movement;
- Concept Design:
    - Estimation of Arroyo Daily Flow Regime;
    - Estimation of Sediment Characteristics;
    - Estimating Arroyo Sediment Discharges;
    - Evaluation of Mesh-Based Traps;
    - Evaluation of Basin-Based Traps; and
- Recommendations.

Evaluation was performed on the mesh-based sediment traps presented in the Tetra Tech 2015 report and on basin-based traps. The two design alternatives were evaluated in terms of the sediment sizes trapped, the sediment volume trapped, maintenance requirements, the amount of bypassing sediments, and the mobility of the bypassed sediments in the Rio Grande. The analysis indicates that the basin-based trap will likely have superior performance relative to the mesh-based trap designs for most of the design options.

Some of the disadvantages of the mesh-based traps include:

- Potential scour at piling locations during flood events;
- Potential damage to screens from debris impact;
- Screens limiting the flow capacity in arroyos; and
- Maintenance difficulties in cleaning the screens with large equipment.

The potential for scour for the basin-based trap is much less and can be reduced with standard methods such as riprap and armoring. Therefore, the recommendation is for the basin-based traps to be constructed. Table 11-1 presents additional comparisons between the sediment traps and the basin-based traps.

11.2 Sediment Basin Sizing and Maintenance Interval

The proposed sediment basins at Thurman I Arroyo and Thurman II Arroyo have been laid out based on constraints of existing topography from the 2011 LiDAR data; available space between the USIBWC northern right-of-way and the Rio Grande; and allowing for an embayment between the basin and the Rio Grande at each location. As shown on the current design drawings, the total storage volumes of the Thurman I and Thurman II Arroyo sediment basins are 5.31 acre-feet and 5.43 acre-feet, respectively. Based on mean annual sediment yields of 1.12 acre-feet for Thurman I Arroyo and 1.98 acre-feet for Thurman II Arroyo, and only allowing the basins to fill to 75% capacity before cleaning, the maintenance interval for the Thurman I Arroyo basin is estimated to be 3.5 years, and 2.0 years for the Thurman II Arroyo basin.

11.3 Compensatory Mitigation Considerations

USIBWC anticipates applying for an individual permit under the Clean Water Act Section 401/404 from the U.S. Army Corps of Engineers for the construction of the sediment basins (USIBWC, 2017). The permit would include a compensatory mitigation plan and would propose to use the following three types of mitigation:

1. Establish onsite riparian areas along each new sediment basin banks by planting native willows.

2. Enhance existing riparian habitat along the embayment and river banks by removing nonnative vegetation such as saltcedar and planting native willows and cottonwoods.

3. Protect the embayment created after the endwall in constructed as an aquatic habitat pool on the riverside of the endwall.

URS recommends that no woody vegetation be allowed to grow within the sediment basins, within approximately 10-feet of the end walls, or within the limits of the rock riprap downstream of the end walls. Any woody vegetation that does take root within these areas should be removed as soon as possible.

Table 11-1. Sediment Trap vs. Basin Comparison

Criteria	Alternative 1 Sediment Trap Design with Multiple Metal Screens	Alternative 2 Sediment Basin Design with Concrete End Walls	Comments
Is disturbance to Waters of the U.S. less than 0.5 acre to qualify for NWP?	No	No	Both alternatives are excavated sediment traps/basins and the areas of disturbance are similar for the two alternatives.
Does a practicable alternative exist that is less damaging to the aquatic environment?	No	No	The two alternatives are similarly damaging during the construction phase, but then provide water quality benefits after construction.
Would the nation's waters be significantly degraded?	No	No	The two alternatives would ultimately improve the water quality by trapping sediment that would otherwise reach the Rio Grande. But Alternative 2 would be more effective at removing smaller grained sediment than Alternative 1.
Fulfills Need and Purpose of Project?	Maybe	Yes	Modeling indicates that both alternatives would trap sediment, but to the extent that Alternative 1 would perform as intended is unknown. Concern would be that the upstream screen(s) would trap the sediment and become clogged and would require frequent cleaning and maintenance. There is more certainty that Alternative 2 would perform as needed to fulfil the purpose and need of the project.
Has the required 30-year design life?	No	Yes	The screen design of Alternative 1 could be easily damaged by cobbles or boulders being transported down the arroyos. The concrete end walls of Alternative 2 are greatly more substantial and would last the 30-year design life.
Subject to scour and erosion?	Yes	Yes	Alternative 1 would be subject to scour around all structural columns and buttresses supporting the screens. The end wall of Alternative 2 is also subject to scour and will require mitigation.
Ease of maintenance and removal of sediment?	No	Yes	The design of Alternative 1 with its many compartments and screens would be difficult to clean and maintain. It would be difficult for maintenance equipment to navigate around the screens. The design of Alternative 2 would easily allow maintenance equipment to navigate around the basins for cleaning.

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Appendix A
H&H Sediment Trap-Basin Analysis

United States Section of the International Boundary and Water Commission

Design for the Construction of Channel Maintenance Alternatives within the Rio Grande Canalization Project Doña Ana County, New Mexico

Contract No. IBM15D0003
Order No. IBM16T0018

Appendix A

Hydrologic and Hydraulic
Sediment Trap-Basin Analysis
Final Submittal

May 31, 2018

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Table of Contents

List of Tables

List of Figures

A.1 EXECUTIVE SUMMARY

This report documents the hydrologic and hydraulic (H&H) analyses performed as part of Task Order (TO) IBM16T0018, Channel Maintenance Alternatives. The purpose of this TO was to design sediment traps at the discharge locations of two arroyos, Thurman I and Thurman II.

The approach for the trap design was the application of flow and sediment transport models. A historic time series of discharges from the arroyos was developed using historic hourly rainfall. The HEC-HMS model (U.S. Army Corps of Engineers [USACE], 2016) was calibrated to peak flow return period data from the U.S. Geological Survey (USGS) and then used with the historic rainfall to develop the arroyo discharge time series.

The Soil and Water Assessment Tool (SWAT) was first used to estimate daily variations in sediment sizes and volumes from each arroyo into the Rio Grande, through the period of the historic rainfall record. A historic time series of sediment loads was then developed using the historic flow time series and applying a simple transport formula with the sediment grain size distribution. The historic time series of loads was calibrated to the published annual average loads (i.e., 1.12 and 1.98 acre-feet/year from Thurman I and II arroyos, respectively).

The two types of designs were compared in terms of trap efficiency and likely construction costs: 1) mesh-based traps; and 2) basin-based traps. The design alternatives were evaluated in terms of the sediment sizes trapped, the sediment volume trapped, maintenance requirements, the amount of bypassing sediments, and the mobility of the bypassed sediment in the Rio Grande. The analysis indicated that the basin-based trap will likely have superior performance relative to the mesh-based trap designs for most of the design options. One of the primary deficiencies in the mesh-based traps is that they allow bypass of larger particles sizes, some of which will not likely be transported by the river. Thus, there is a larger potential for shoaling to occur in the Rio Grande in the vicinity of the traps. In addition, structural considerations and the potential for scour favor the basin-based traps.

There were 24 different basin-based trap alternative designs considered for the Thurman I Arroyo and 24 for the Thurman II Arroyo. For the Thurman I Arroyo, the design volumes ranged from 1.13 to 4.21 acre-feet. For the range of historic flows, 24 of the alternatives would retain within the basin sediment all sizes that could not be transported by the Rio Grande and therefore should minimize shoaling in the river. These basins were estimated to require maintenance (clean-out) on average every 1 to 3 years, depending on the trap volume selected, but the historic pattern showed this interval would vary widely within a 30-year period. Any volumes greater than 4.21 acre-feet available within the site constraints would have better sediment retention performance and reduced frequency of cleanout.

For the Thurman II trap, the design volumes ranged from 0.9 to 2.6 acre-feet. For the range of historic flows, 21 alternatives would retain within the basin sediment all sizes that could not be transported by the Rio Grande and therefore should minimize shoaling in the river. The basins were estimated to require maintenance (clean-out) on average every 0.5 to 1.3 years, depending on the trap volume selected, but the historic pattern showed this interval to vary widely within a

30-year period. Any volumes greater than 2.6 acre-feet available within the site constraints would have better sediment retention performance and reduced frequency of cleanout.

Changes to the flooding potential along the arroyos due to the proposed sediment traps were evaluated. A 2D methodology using HEC-RAS 5.0.3 (USACE, 2016) was selected to best estimate flooding in the relatively flat agricultural area adjacent to the Rio Grande. A major assumption of this modeling approach was that flooding of the Rio Grande was non-coincident with flooding of the Thurman I and Thurman II arroyos.

For Thurman I, the 100-year flood from the arroyo watershed was largely contained within the existing arroyo flow area. The construction of the sediment basin wall forces an increase in water surface elevation upstream for the full extent of the basin, but the water surface elevation is contained within the basin.

For Thurman II, in the existing condition, the arroyo opening through the raised terrace access road was estimated to constrict 100-year flood flows from the arroyo watershed, causing shallow ponding on the upstream side of the road for a width of over 3,800 feet. The road overtops in very shallow flow for a width of about 2500 feet and proceeds in shallow flow to the Rio Grande. The flow to the west within the river terrace is blocked by an irrigation feature. The construction of the sediment basin wall forces flow into the west overbank for a distance of less than 200 feet upstream of the wall. The overtopping of the full width of the wall increases the depth of flow immediately downstream of the wall. The maximum flow depth for flows within the flow area with increased depth is approximately 1 foot (an increase of about 0.5 foot over the existing flow depth in the same area). The associated velocities are not estimated to be erosive.

A.2 INTRODUCTION

This report documents the hydrologic and hydraulic (H&H) analyses performed as part of Task Order (TO) IBM16T0018, Channel Maintenance Alternatives. The purpose of this TO was to design sediment traps at the discharge locations of two arroyos, Thurman I and Thurman II, into the Rio Grande (see Figure A-1). The *Channel Maintenance Alternatives and Sediment Transport Studies for the Rio Grande Canalization Final Report* (Channel Maintenance Report; Tetra Tech, 2015) found that there is ongoing sediment inflow from these arroyos, resulting in sediment deposition forming sediment plugs in the Rio Grande. This sediment inflow results in island formations and rising of the river bed. The river bed rise inhibits draining of irrigation return flow to the Rio Grande and may eventually, if not controlled, result in increases in water surface elevations that could impact levee freeboard and increase the flooding risk to adjoining communities. The H&H studies and analyses covered in this report include:

- Data Collection;
- Background Studies;
- SWAT Modeling to Estimate Frequency of Larger Diameter Particle Movement;

- Concept Design:
  - Estimation of Arroyo Daily Flow Regime;
  - Estimation of Sediment Characteristics;
  - Estimating Arroyo Sediment Discharges;
  - Evaluation of Mesh-Based Traps;
  - Evaluation of Basin-Based Traps; and
- Recommendations.

A.3 DATA COLLECTION

Data collected for the H&H modeling of this project included the following.

A.3.1 Topographic Data

Topographic data for the project area were available in three forms:

- U.S. Geological Survey (USGS) 30m Digital Elevation Model (DEM) data, available from the USGS on-line data portal. These data were obtained and used to delineate watershed boundaries, ascertain streamline bedslopes, and estimate approximate major flood hydraulic cross-sections within the watershed.
- USIBWC 1m LiDAR. These data were available within and slightly outside of the USIBWC right-of-way. These data were used in pond volume estimates.
- Ground survey of Thurman I and II Arroyos cross-sections, provided by USIBWC. These data were used to estimate refined hydraulic cross-sections at the arroyo outlets.

A.3.2 Land Use Data

Spatial landuse data were obtained from the USGS data portal. There was very little variation in estimated land use across the arroyo watersheds, which was classified as Range-brush (RNGB) shrub/scrub. This homogeneity was confirmed by aerial photo review.

A.3.3 Soils Data

Spatial County Soils Survey data (SSURGO) were obtained from the U.S. Department of Agriculture (USDA). These data included information on soil series, depth and characteristics of surficial soil layers, and hydrologic soil group (HSG). These data were used as inputs to study hydrologic models: as direct inputs to SWAT, and as base data input for derivation of curve numbers for HEC-HMS.

A.3.4 Rainfall Data

USIBWC provided historic data for two hourly rainfall gages in the vicinity of the project area: Caballo Dam (COOP 291286) and Jornadat Experimental Range (COOP 29426). Thiessen polygon analysis demonstrated that the Caballo Dam gage alone provided the most representative data for the project area.

A.3.5 Rio Grande Flow Data

Daily flow data for the Rio Grande Caballo Dam flow gage (IBWC 08-3625) for the historic record (1938 to 2013) were obtained from the USGS data portal. Daily flow data for the Rio Grande El Paso gage (IBWC 08-3640, 1889 to October 2016) were obtained from USIBWC.

A.3.6 Sediment Data

Sediment data from the project area came from three sources: the Channel Maintenance Report, USDA spatial soils data (Soil Survey Geographic Database [SSURGO]), and a project site visit on October 20, 2016.

- Channel Maintenance Report. This report included sampling and grain size distribution analysis of the smaller material within Problem Location 2 (which includes Thurman I and II Arroyos). A grain size distribution of larger sizes was obtained by pebble count in the fan of Thurman I Arroyo.
- The SSURGO dataset includes rough estimates of grain size by soil series within the watershed. These data were used with limited success within SWAT sediment transport modeling.
- The project visit on October 20, 2016 included visits to the outlets of both arroyos from north of Interstate 25 to the Rio Grande. A summary of the observations from this trip is provided in Attachment A, October 20, 2016 Site Visit (to be provided with the 100% submittal).

A.4 BACKGROUND STUDIES

A.4.1 Local Rainfall Statistics

Hourly rain data for the Caballo Dam precipitation gage were obtained for the period from 1947 to 2013. The period from 1983 to 2013 was selected as most representative of current climate patterns and most relevant for comparison to recent USIBWC maintenance experience. The data for this period were analyzed to identify the peak rain depth for periods of continuous rainfall ranging from 1 hour to 24 hours. A summary of analysis results is presented in Table A-1.

The *City of El Paso Drainage Design Manual* provides a depth-duration-frequency table for the El Paso area, based upon a statistical analysis of hourly data from the long duration record at the El Paso Airport. This table is reproduced in Table A-2.

The largest storms within the recent historic record of the Caballo Dam gage occurred in July 2008. Modeling of these storms within SWAT (see Section 3.5) estimated hydrographs at the arroyo outlets shown in Figure A-2 for Thurman I and Figure A-3 for Thurman II. These hydrographs show a lag time between rainfall and estimated flood peak of 1 to 2 hours, making storms of this duration the most critical in terms of generating flow peaks at these two arroyos.

The data in Table A-2 were compared to the data in Table A-1 to estimate the return period of storms from 1983 to 2013. This comparison is shown in Table A-3. Table A-3 shows that the most extreme storms of the critical 1- to 2-hour duration likely occurred in 1989 and 1999 (this assumes the localized storm over the Caballo Dam gage also occurred over the arroyo watersheds), which had an estimated 25-year return frequency. More recently, in 2010, a storm with roughly a 10-year return frequency occurred. These storms, assuming general equivalent antecedent watershed soil moisture, would be expected to have generated the highest instantaneous flood peaks and moved the largest diameter sediment through the mechanism of exceedance of threshold shear.

The most extreme statistical storm with a 1-day duration within the period studied was in 2008, when the depth of 24-hour precipitation rose to the level of a 1% annual chance (100-year return period) storm. In this circumstance of an extreme longer duration rainfall, there is a strong chance that historic sediments deposited long term within the arroyo were mobilized. Large volumes of stormwater fill voids within the deposited sediments in the arroyo bed and liquefy the sediments en masse (as opposed to mobilization by exceedance of threshold shear). The mass of sediment flows as a viscous debris flow down the arroyo and is ultimately deposited in the Rio Grande. Debris flows occurred on arroyos in the El Paso area (Fairbanks Drive) during and following the August 2006 sequence of storms in that area. This mechanism may be the source for the large volume of larger material seen in the river fans of both project arroyos.

A.4.2 Estimation of Extreme Arroyo Flowrates Using Regression

The magnitude of extreme floods for each of the two arroyos was estimated using regression equations developed by the USGS for New Mexico in SRI 2008-5119 "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in new Mexico and Surrounding Areas" (USGS, 2008). These regressions were developed from gaging of peak floods on watersheds of similar size within the general region of New Mexico. The equations are based upon watershed area only. Table A-4 provides a summary of the estimated extreme flows for each arroyo.

A.4.3 Statistical Analysis of Rio Grande Flows in Reach

The existing flood study for the Rio Grande in the project reach provides an estimate of the 100-year flood as 15,150 cfs. The origin study for this value has not been reviewed, but the flow estimate is likely based upon a statistical extreme storm over the full Rio Grande watershed that raises flood levels in main stem dams well above levels seen in the historic record. To estimate the likely range of extreme flows given historic main stem dam levels, a statistical analysis using the Log Pearson Type III statistical distribution was performed for the two Rio Grande gages upstream and downstream of the project site.

Flood flows within the Rio Grande in the project reach are strongly controlled by large main stem dams (Elephant Butte, Caballo) upstream. Since the construction of Caballo Dam in 1938, the peak mean daily flood discharged from the dam has been 7,650 cfs (1942), with the next highest flood considerably smaller (4,646 cfs in 1987). The next gage downstream from Caballo Dam is located at El Paso and measures the increase due to inflow from the intervening watershed, which includes inflow from the two arroyos. The measured peak flows in the Rio Grande since the construction are summarized in Table A-5, sorted by flowrate (from highest to lowest) measured at El Paso.

Table A-6 shows the results of the Log Pearson Type III analysis using PEAKFQ USGS software, the annual peak flows for the El Paso gage from Table A-4, and the selection of a regional skew coefficient using a USGS regional reference (USGS WRIR 96-4117, 1996). This analysis estimates the 1% annual chance (100-year return period) flood as 5,626 cfs, again assuming floodpool levels in the upstream dams remain within historic levels.

A.4.4 Development of Hydrologic Modeling Spatial Datasets

A.4.4.1 Basin Delineation

Watershed and subwatershed boundaries were developed using GIS software applied to USGS spatial topography. Initial subwatershed delineation was performed at a fine scale (see Figure A-4) to identify locations within the watershed of highest streambed boundary shear stress. Shear stress is a function of flow depth (determined by flowrate, channel shape, slope and roughness) and energy slope (largely determined by bed slope). These parameters vary within the watersheds.

A.4.4.2 Spatial Soils Definition

Spatial soils data were differentiated by HSG and applied to the subwatersheds. Figure A-5 depicts the variation across the two watersheds in HSG.

A.4.4.3 Spatial Slope Definition

Variation in slope across the watersheds was estimated using GIS methods. Figure A-6 depicts the variation in topographic slope across the two watersheds.

A.4.5 SWAT Modeling to Estimate Frequency of Larger Diameter Particle Movement

Per the field visit, the existing arroyo fans that extend into the Rio Grande maintain their character due to armoring with larger-diameter cobbles that resist transport by the flow in the river. Watershed modeling was performed using SWAT to ascertain whether source regions for these cobbles could be identified and whether frequency of mobilization of these cobbles could be estimated.

A.4.5.1 Description of SWAT

SWAT (USDA-ARS and Texas A&M AgriLife Research) has proven to be an effective tool for assessing water resource problems for a wide range of scales and environmental conditions across the globe. The development of SWAT is a continuation of nearly 30 years of modeling efforts conducted by the USDA Agricultural Research Service (ARS). SWAT is a basin-scale, continuous-time model that operates on a daily time step and is designed to predict the impact of management on water, sediment, and agricultural chemical yields in ungauged watersheds. In

SWAT, a watershed is divided into multiple watersheds (subbasins), which are further subdivided into hydrologic response units (HRUs) that consist of homogenous land use, soil type, and slope layers.

SWAT was selected for this study primarily because of its full integration with the USDA SSURGO soils dataset. The model inputs and uses the full metadata from SSURGO, which greatly facilitates the coding of soils moisture storage capacities within a watershed.

All analyses with SWAT were performed using the hourly rainfall dataset from Caballo Dam from 1983 to 2013.

A.4.5.2 SWAT Model Calibration

Per the analyses summarized in Table A-3, the peak storm of critical duration (1 to 2 hours) that occurred within the 1983 to 2013 study period was on the order of an 8-year to 25-year return period. Loss parameters were adjusted in SWAT to approximate the peak flow values estimated in Table A-3 for this range of flood. The calibrated SWAT model had a peak flow of 950 cfs for Thurman I and 967 cfs for Thurman II.

A.4.5.3 Identification of Watershed Locations of Maximum Shear Stress

The subwatersheds shown in Figure A-4 were compared to the soils map in Figure A-5 and the slope map in Figure A-6, and locations of comparison were identified for estimation of peak threshold shear. These locations are shown in Figure A-7.

For each of these locations, the following analyses were performed.

- Hydraulic cross-sections were developed from best available topography: USGS DEM in upstream subwatersheds and LiDAR or ground survey at arroyo outlet locations.
- Rating curves using a normal depth assumption were developed for each of these locations.
- Daily flowrates using SWAT were developed using each of these locations.
- Daily flowrates at each location were converted to a threshold shear with associated particle diameter using the Shield relation (Simons Li, 1996):

$$d = \frac{(RS}{(\gamma_S - \gamma)(0.047)}$$

where: d = movable grain size (feet);
(= unit weight of water (62.4 pcf);
$(_S$ = unit weight of sediment (pcf);
R = hydraulic depth (feet); and
S = bed slope.

- Daily flowrates at each location were converted to an alternate threshold shear with associated particle diameter using a van Rijn relation (van Rijn, 1998):

$$\text{Threshold Shear } (\tau) = 0.9067d^{0.65}$$

where: d = movable grain size (feet).

The end result of this analysis was to demonstrate that the locations of peak threshold shear were at the outlet to Subbasin 19 in Thurman I and the outlet of Subbasin 52 for Thurman II.

A.4.5.4 Estimation of Larger Diameter Particle Movement

Tables 7 and 8 provide a summary of results of larger particle movement by year using the Shield relation (Table A-7) and van Rijn relation (Table A-8). These estimates show transported particle sizes much smaller than evidenced by the material in the fans in the arroyo outlets, documented in the grain size distributions in the Channel Maintenance Report. Attempts to adjust the model to estimate larger transported sediment sizes were unsuccessful. Likely sources of the inability to adjust the model are:

- Lack of definition of the low-flow channel shape in the upper watershed. A more representative confined channel shape would materially increase estimated boundary shear.
- Hourly time step. Since the watersheds have a lag time of 1 to 2 hours, the SWAT model limitation of an hourly step (i.e., the model cannot handle shorter time steps) results in a likely underestimation of peak flow.

A.4.5.5 Conclusions

The primary results of interest from the SWAT modeling are:

- Peak threshold shear stresses likely occur at the base of the watershed prior to the flattening of the slope into the river terrace; and
- The model predicts, even in higher subwatersheds, threshold velocities well under the velocities needed to move the larger particles armoring the fan material at the arroyo outlets. There is a likelihood that the motion of these larger cobbles and boulders is associated with longer duration heavy rainfall (1 to 3 days) and resulting debris flow. Such an event may have occurred in 2008, per rainfall patterns in July 2008.

A.5 CONCEPT DESIGN

A.5.1 Approach

The concept design approach follows the outline provided below.

- Estimation of Arroyo Daily Flow Regime. A historic time series of discharges from the arroyos was developed using historic hourly rainfall. The HEC-HMS model (U.S.

Army Corps of Engineers [USACE], 2016) was used with the historic rainfall to develop the time series. It was calibrated to peak flow return period data from the USGS. The output was provided in 5-minute increments.

- Estimation of Sediment Characteristics. The sediment grain size distribution of the sediment load from the arroyos was developed by merging the bulk sample and pebble count data.
- Estimation of Arroyo Sediment Discharges. A historic time series of sediment loads was then developed using the historic flow time series and applying a simple transport formula with the sediment grain size distribution. The historic time series of loads was calibrated to the published annual average loads (i.e., 1.12 and 1.98 acre-feet/year from Thurman I and II Arroyos, respectively).
- Evaluation of a Mesh-Based Trap. The performance of the mesh-based trap was completed. The historic flow and load time series were not essential to this analysis, and only the grain size data and published annual loads were used. The results of the analysis provide an estimate of the annual average sediment trapped and bypassed for different assumptions and mesh-based trap parameters, typical maintenance requirements, and the characteristics of the bypassed sediments.
- Evaluation of a Basin-Based Trap. A series of basin-based trap designs was developed, and their performance was evaluated using the historic time series of flows and loads. The results of the analysis provide an estimate of the annual average sediment trapped and bypassed for different assumptions and trap parameters, typical maintenance requirements, and the characteristics of the bypassed sediments.
- The two types of designs were compared in terms of trap efficiency and likely construction costs.

The analysis was completed for each arroyo (Thurman I and Thurman II) separately.

A.5.2 Establishing Arroyo Discharges

A historic time series of flows was established with the HEC-HMS model. The SWAT modeling demonstrated that the critical flows for sediment movement were located just upstream of the arroyo outlet, and the estimation of these flows would be better estimated by a model with a shorter time step. Therefore, the 31-year selected historical hourly rainfall from the Cabello gage was used as input to the HEC-HMS model. The models were relatively simple, run at a 5-minute time step using a single basin element to represent the entire basin (5.78 and 7.67 sq mi for Thurman I and Thurman II Arroyos, respectively). The basin discharge was modeled using only a constant loss and unit hydrograph.

The constant loss and lag time values were obtained by calibration to the USGS peak flow return period data (USGS, 2008). The values obtained using the USGS regression equation for Thurman I and II are shown in Table A-4 in cfs and converted to m^3/s in Tables 9 and 10.

For Thurman I, the calibration values were 6 mm/hour for the loss and a lag time of 150 minutes. For Thurman II, they were 6 mm/hour and 150 minutes. The HEC-HMS model results were used in a return-period analysis using a log-Pearson Type III distribution. (This is the approach used by the USGS in developing the regression equation.) The results for the two arroyos are shown in Tables 9 and 10.

Plots comparing the USGS data and the model simulation results are shown in Figures 8 and 9.

The simulated time series data obtained from the calibrated HEC-HMS models are used in the subsequent sediment basin analysis.

A.5.3 Sediment Characteristics

There are two gradation curves available for estimating the average sediment gradation. One is based on a bulk sample, and the other a pebble count of the sediments in the depositional fan at the base of Thurman I Arroyo. The grain size curves were digitized for further analysis and are shown in Figure A-10.

The actual average gradation curve is likely somewhere between the two curves. It was indicated in the Channel Maintenance Report that the "true" average gradation can be obtained as a mix of 45% of the pebble count gradation and 55% of the bulk sediment gradation. This is based on data analysis at a site where there was a bulk sample and pebble count from a nearby area of the deposit that was not reworked by the river. The 45/55 ratio has been adopted in this analysis. The curve shown in Figure A-11 was obtained when the 45/55 ratio was applied to the bulk sample and pebble count data.

The estimated average sediment gradation has a d_{50} of 25 mm and a d_{84} of 80 mm. In all subsequent analyses, it was assumed that this was the gradation of the sediment loads from Thurman I and II Arroyos.

A.5.4 Establishing Arroyo Sediment Discharges

A historic time series of sediment loads was developed using the historic time series of flows described previously. The approach for developing the sediment loads consisted of assuming a channel cross-section, applying the normal flow approximation, then calculating the sediment transport based on the normal flow velocity and height. A standard transport formula was applied to estimate the load. This procedure is described in more detail in Attachment B.

The key parameters for the analysis include the arroyo average slope, the channel width, and the grain size. The slope for the arroyos was estimated from the region DEM. It is 0.02 for Thurman I and 0.016 for Thurman II. The grain size parameters for the analysis were taken from the grain size curve established in Section A.4.3. The bottom roughness used in calculating the bed stress was based upon d_{84}, which is 80 mm (Figure A-11), and the critical stress for erosion is based on the mean grain diameter of 25 mm.

The transport simulation was calibrated to the annual average loads established by USACE in a 2007 report (USACE, 2007). Using the normal flow approximation and standard transport formula, the sediment load for each flow record in the historic time series (31 years at 5-minute intervals) was estimated. The annual average load was then computed and compared to the reported annual average loads of 1.12 and 1.98 acre-feet from Thurman I and II Arroyos, respectively. The assumed channel width was adjusted in a series of simulations until the computed annual average loads matched the reported values.

The annual loads for each year in the simulations are shown in Figure A-12 for Thurman I and Figure A-13 for Thurman II.

These calibrated time series of flows and loads were used to estimate the performance of the settling basin traps.

A.5.5 Evaluation of Proposed Mesh-Based Traps

It is possible to estimate the mesh-based trap performance based solely on the average gradation of the sediments emanating from the arroyos. This is because the mesh-based traps do not rely on sediment settling and consequently are not dependent on residence time. Therefore, the flowrates are not inherent in the analysis, and the performance can be estimated using only the sediment data and mesh sizes.

The smallest proposed mesh size is 2 inches. Therefore, for estimating the trap efficiency, we can assume that any particles greater than 2 inches (50.8 mm) will be retained by the trap. This results in about 37% of the sediment being removed by the trap, based on the grain size distribution estimated for the arroyo loads. Particle sizes of 50 mm and smaller will pass through the trap.

However, the trap efficiency is slightly greater than 37% because it is likely that the void space in the sediment trapped by each mesh will be filled with smaller sediment sizes (i.e., smaller sizes will be trapped in the voids between larger sizes). We do not know exactly how much void space will be occupied, but assuming that the available void space is 0.35, we can estimate the additional trapping capacity for a range of filling. The results for a range of void filling ratios are shown in Table A-11.

We have adopted a judgement-based total trapping efficiency of 44.4% as our best estimate, which is equivalent to assuming 20% of the void space is occupied by trapped sediments.

The gradation of the sediment that passes through the trap can also be estimated knowing the smallest mesh opening (i.e., 2 inches). The grain size curve obtained after removing the larger particles from the grain size curve and renormalizing is shown in Figure A-14.

The passed sediment will have a d_{50} of about 7 mm, with the largest size being just under 50 mm.

The rate at which the mesh-based traps will fill, and consequently, the average maintenance interval, can be estimated from the annual sediment yields and the mesh-based trap volume. The sediment yields for Thurman I and II Arroyos have been estimated by USACE (USACE, 2007) as 1.12 and 1.98 acre-feet, respectively.

Establishing the maximum mesh-based trapping volume requires careful consideration of a number of factors. The Channel Maintenance Report indicates that the volume of the Thurman I and Thurman II traps are 4.1 and 2.9 acre-feet, respectively. However, these values appear to be based on an area that is larger than what may actually be available for trapping sediment.

The mesh-based trap volumes reported in the Channel Maintenance Report are calculated from estimated areas of 1.4 and 1.0 acres and a 3-foot depth. However, the trap schematics shown in Appendix H of the Channel Maintenance Report indicate that not all of the area is used for the trap. The area is divided into six sections, and only the upper five sections are used for trapping sediment. Also, it is indicated in the Channel Maintenance Report that the mesh heights will be 4 feet. The berms that currently line the edges of the tap are approximately 3 to 4 feet in height, based on the 2007 LIDAR data, and it is assumed the lower berms will be raised to 4 feet. The following calculations were made to arrive at the final maximum trap volumes:

4.54 (acre-feet)= (5/6)**(4/3)*4.1 (acre-feet)

3.23 (acre-feet)= (5/6)**(4/3)*2.9 (acre-feet)

The traps cannot be allowed to fill to their full depth, as the banks of the channel forming the trap will overflow, and sediment will overtop the banks and spread into unintended areas.

The flow and load analysis previously completed indicates that the flow depths for higher events will likely be relatively shallow, probably less than 1 foot. Therefore, it will be necessary to limit the height of the trapped sediment to 2 to 2.5 feet to ensure that there is capacity to retain the arroyo discharges within the banks of the trap. We have considered both 2- and 2.5-foot limits on the fill heights in the analysis.

Another factor that requires consideration is the filling template. It is likely that the sediment will start filling on the upstream side of each mesh. A schematic depicting this filling pattern is shown in Figure A-15. We have estimated the trap performance for a range of infilling angles, from 0 to 10 degrees (0 degrees is equivalent to uniform infilling).

The assumptions, estimated trapping volumes, and maintenance intervals for each scenario are summarized in Table A-12 for Thurman I and in Table A-13 for Thurman II.

The analysis reported above also assumed that each segment of the trap fills at the same rate. The grain size curves indicate that the sediments composing the load may not be equally divided among the mesh sizes, and therefore, one segment of the trap may fill before the others. Since the filling of any one segment can cause the trap to fail, the filled segment will need to be cleaned even if the other segments are not at capacity. Ultimately, this effect will likely decrease the maintenance interval, requiring maintenance more often.

The annual average maintenance interval is a simplified parameter for long-term planning performance. The actual maintenance intervals will likely be highly variable due to the large variability in event occurrence and intensity. To estimate this variability, the time series of sediment loads was used to simulate the meshed-based traps filling. The analysis consisted of simulating the filling of the trap with the simulated loads until the trap reached the Maximum Trapping Volume. Once the maximum trapping volume was reached, a maintenance event was recorded, the trapped volume reset to zero, and the simulation continued. This was done for three of the scenarios indicated in Tables 5 and 6. The results of the analysis are shown in Tables 14 and 15.

The results indicate that the time between maintenance intervals can be 1 day to many years. Cases where the interval is recorded as 1 or a few days indicate situations in which the trap was overwhelmed by a large event. In reality, 1-day intervals of maintenance during severe event conditions is highly unlikely, and when these situation occur, the trap will overflow, and some of the larger grain sizes will likely deposit in the Rio Grande. The likelihood of this happening appears to decrease with increased maximum trapping volumes. However, the maximum trapping volumes are limited by right-of-way concerns, so it is not possible to avoid the possibility of occasional overtopping.

The fate of the sediment passing through the mesh-based trap depends on the mobility of sediment in the Rio Grande. To assess the mobility, the bottom stresses associated with typical flow events in the Rio Grande were compared to the critical stress for movement for the grain sizes estimated to pass through the trap. A series of Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer models representing conditions over a range of typical flows was provided by USIBWC. These models were developed and applied as reported in the Channel Maintenance Report. For typical flows in the range of 2,000 to 4,000 cfs, the bottom shear stresses ranged from 0.09 to 0.14 lbf/ft^2.

To estimate the largest sediment sizes that could be mobilized under these flow conditions, the impact of hiding and exposure on the critical stress for erosion needs to be considered. For sediments with varying grain sizes, the smaller grains will "hide" behind the larger grains or be hidden in troughs formed by the larger grains. When this happens, the stress required to mobilize the smaller grains increases compared to the case of a uniform bed of smaller grains. The larger grains protrude higher into the flow than they would if they were in a bed of similar sized grains, and therefore, the stress needed to mobilize the larger grains decreases relative to that for a bed of similar grain sizes. These effects are depicted in Figure A-16.

The Wu method (Wu et al., 2000) was applied to account for the hiding and exposure, and the revised critical stresses were used to assess the fate of the bypassed sediments. The results of the analysis are summarized in Table A-16, where Tc is the uniform grain size critical stress and Tc* represents the effect of the hiding and exposure on the critical stress for the bypassing distribution.

These results indicate that the largest grain size that can be mobilized is approximately 13 mm. Thus, it is likely that larger grain sizes, in the range of 14 to 50 mm, will remain in the vicinity of the trap discharge area.

A.5.6 Evaluation of Basin-Based Sediment Traps

The basin-based trap consists of excavating a basin between the upland right-of-way and the Rio Grande, installing a weir or sill at the downstream end (just before the river), and directing the arroyo flow into the basin. As the flow enters the basin, the flow speed will decrease, consequently decreasing the transport capacity and causing sediment to be retained in the basin.

A schematic of the basin parameters for Thurman I Arroyo are shown in Figures 17 and 18. Figure A-19 shows a cross-section. The existing slope of the arroyo channel in the vicinity of the trap is shown, based on three transects with elevation data extracted from the 2007 LIDAR data. The basin would consist of a sill upstream of the river and relatively flat base with a slope from the base to the upper extent of the right-of-way.

Figure A-18 shows a map view of the basin-based trap concept. The basin would widen from the upper extent near the right-of-way to the sill near the river.

A.5.7 Comparison of Mesh-Based and Basin-Based Concept Designs

A number of the basin parameters were varied to provide a range of basin sizes. The effective basin length is the orange part of the base shown in Figure A-17. The flow from the arroyo will flow down the ramp portion, and the actual settling basin water elevation will be (slightly) higher than the height of the sill. The flow pattern and water elevation are shown in Figure A-19. The volume of the basin available to trap sediment is based on the length of the base, the height between the sill elevation, and the effective width. The effective width is the average of the top and bottom widths.

A model of the sediment transport through the basin was developed to determine the performance of the basin. The model consisted of tracking the sediment for each 5-minute interval of the 31-year historic time series of flow and loads. The model assumed that all sediment traveling as bedload would be trapped and determined the amount of suspended load trapped using the residence time, settling speed, and basin height. During the simulation, the basin filled, the basin effective volume decreased, and the reduction of flow speed was less and consequently less sediment was trapped. When the trapped sediment reached a critical volume in the basin, the model assumed the basin was dredged (i.e., a maintenance event) and restored to its original configuration, and the simulation then continued. The number of times the critical height was reached was recorded to estimate the maintenance interval.

The critical volume is the volume of trapped sediment after which no more sediment can be trapped or the flow starts to overflow the sides of the basin. We assumed that the critical volumes were 50% and 75% of the design volume for the simulations. The other parameters that were varied are:

- Bottom width: 50, 100, 150, and 200 feet; and
- Height: 2, 3, and 4 feet.

The top width was set at 32.8 feet (10 m) for all simulations, and the length of the base was 400 feet (for Thurman I). A summary of the basin-based trap performance is shown in Table A-17.

For Thurman II, it was assumed that the excavated area would follow the existing arroyo thalwag. This configuration is shown in Figure A-20. The effective base length for this alignment was approximately 400 feet.

The same parameter variations for the Thurman I analysis were used for the Thurman II analysis. The results are summarized in Table A-18.

The fate of the bypassed sediment can be estimated by using the same considerations of hiding and exposure discussed previously. With a river shear stress of 0.14 lbf/ft^2, the maximum grain size that can be moved is approximately 13 mm. For most of the scenarios considered and summarized in Tables A-17 and A-18 and considering the maximum bypassed sediment size (second column from right), all of the bypassed sediment can be moved, and therefore, shoaling in the vicinity of the basin discharge is unlikely. However, for a few scenarios, larger sediment sizes on the order of 16 to 32 mm may remain and yield some shoaling.

A.6 CURRENT VERSUS POST-PROJECT 100-YEAR FLOODPLAIN

A.6.1 General Approach

This section describes 2D modeling carried out to assess the impact of the proposed project on the 100-year floodplain. There are two floodplains to consider: the floodplain within the Rio Grande associated with releases from Caballo Dam and the aggregate flow from the watershed below Caballo Dam, and the floodplain within each arroyo associated with flood flows from each arroyo watershed.

The walls of the proposed basins block flow area of the arroyo, which is incised through the overbank of the river. The walls do not block active flow area of the river in the direction of river flow. The construction to the north of the walls also do not involve fill into the active flow area of the river. The effect (and purpose) of the project will be to reduce discharge of sediment into the active flow area of the river, which will have the effect of preventing a gradual rise in base flood elevation, and extending the period of effectiveness of channel maintenance. Therefore the projects are not expected to increase base flood levels of the river.

Therefore this section addresses the effect of the projects on the floodplains of the individual arroyos. The walls of the proposed basins do intercept flow area of the arroyos. A 2D methodology using HEC-RAS 5.0.3 (U.S. Army Corps of Engineers, 2016) was selected to best estimate flooding in the relatively flat agricultural area adjacent to the Rio Grande. A major assumption of this modeling approach is that flooding of the Rio Grande is non-coincident with flooding of the arroyos Thurman I and Thurman II. The following sections provide details on model setup and simulation for both existing and post-project conditions, and also compare the model results to highlight changes brought about by the project.

A.6.2 Develop Existing Condition Model

A.6.2.1 <u>Source Data</u>

A.6.2.1.1 <u>Topography</u>

Topographic data for the project area were obtained from IBWC and used to develop the 2D terrain. A 1m LiDAR-derived DEM was imported into HEC-RAS 5.0.3. Based on the site visit, the terrain just downstream of the gravel road across Thurman I was flattened remove the steep slope shown in the DEM in this area. No other edits were made to the terrain.

A.6.2.1.2 Roughness

Spatial landuse data were obtained from the National Land Classification Dataset, NLCD (U.S. Geological Survey, 2011). Manning's 'n' roughness values were assigned to the various land classes in the NLCD.

A.6.2.1.3 Flows

Flows in each arroyo were estimated from the arroyo HMS models described in the Hydrology and Hydraulics Report. A point precipitation frequency estimate value corresponding to the 100-yr event was obtained NOAA Atlas 14 (Volume 1, Version 5) for Hatch, NM. No other HMS model inputs were changed. Output hydrographs (10-min increments) were adjusted at each time step so that the peak flow of each matched the peak flow predicted by USGS regression equations appropriate to New Mexico (U.S. Geological Survey, 2008). This increased the peak flow in Thurman I from 1352.6 cfs to 2070 cfs, and from 1511.5 cfs to 2388 cfs in Thurman II.

A.6.2.2 Model Assembly

A.6.2.2.1 Development of 2D Flow Area

A 2D flow area was developed in HEC-RAS 5.0.3 to encompass both arroyos. The mesh generated had an average cell size of 100 square feet. This resulted in the generation of more than 100,000 cells, allowing for detailed hydraulic analysis outside of the arroyo channels. Break lines were added to add further definition to the 2D area at the access roads potentially affected by flooding. The sediment basin walls were added as 2D area connections and modeled as weirs with a weir coefficient of 2.8.

To further refine roughness within the 2D model area, the arroyos were hand delineated and manually assigned manning's 'n' values of 0.04 within HEC-RAS. These values were set to take preference over pre-assigned NLCD-derived roughness values (U.S. Army Corps of Engineers, Application Guide, 2016).

A.6.2.2.2 Boundary Conditions

Boundary conditions were generated upstream and downstream of the proposed project area. For upstream conditions, a flow hydrograph was input along with a bed slope parallel to the direction of flow. For downstream conditions, a normal depth boundary condition was set, with perpendicular bed slopes between 0.1% and 3.5% assigned based on local underlying terrain.

A.6.2.2.3 Duration and Time Step

The flood event was simulated for 24 hours at a one second interval. Full momentum equations were used as the method of model computation, with a maximum number of iterations set at 20.

A.6.2.3 Model Results

Figure 21 shows the maximum flooding depth and extent estimated during the existing condition model simulation.

A.6.3 Develop Post-Project Model

A.6.3.1 Source Data

With the exception of the topographic data, the source data were not altered from the existing condition.

A.6.3.1.1 Topographic Changes

The following changes were made to the terrain in the post-project model:

- The proposed sediment trap basins were added to the existing terrain.
- The access road upstream of the Thurman II sediment basin was edited to moreaccurately represent post-project conditions.

A.6.3.2 Model Assembly

Apart from the edits to the terrain described above, no changes were made to the model inputs or simulation parameters.

A.6.3.3 Model Results

Figure A-22 shows the maximum flooding depth and extent estimated during the post-project condition model simulation.

A.6.4 Comparison

This section compares pre-project to post-project flood plains associated with 100-year return period flood from the arroyo watershed. Again, the Rio Grande is assumed in this comparison to be at non-flood stage.

Figures A-21 and A-22 show maximum flooding depth and extent estimated during the pre-project and post-project condition model simulations respectively. Figure A-23 shows locations where the project increases flood depth. The following changes are noted:

- For Thurman I, the 100-year flood is largely contained within the existing arroyo flow area. The construction of the sediment basin wall forces an increase in water surface elevation upstream for the full extent of the basin, but the water surface elevation is contained within the basin. The flow profile is shown in Figure A-24.
- For Thurman II, in the existing condition the arroyo opening through the raised terrace access road is estimated to constrict 100-year flood flows, causing shallow

ponding on the upstream side of the road for a width of over 3,800 feet. The road overtops in very shallow flow for a width of about 2500 feet, and proceeds in shallow flow to the Rio Grande. The flow to the west within the river terrace is blocked by an irrigation feature. The construction of the sediment basin wall forces flow into the west overbank for a distance of less than 200 feet upstream of the wall. The overtopping of the full width of the wall increases the depth of flow immediately downstream of the wall. The maximum flow depth for flows within the flow area with increased depth is approximately 1 foot (an increase of about 0.5 foot over the existing flow depth in the same area). The associated velocities are not estimated to be erosive. The flow profile is shown in Figure A-25.

A.7 RECOMMENDATIONS

The sediment trap and associated H&H analyses provide quantitative assessments of the trapping efficiencies for a range of design parameters for both the mesh-based and basin-based traps. The design parameters were constrained geometrically by right-of-way limits, feasible excavation depths, and water levels. However, the parameters were varied within the extent of those limits.

The design alternatives were evaluated in terms of the sediment sizes trapped, the sediment volume trapped, maintenance requirements, the amount of bypassing sediments, and the mobility of the bypassed sediment in the Rio Grande.

The analysis indicates that the basin-based trap will likely have superior performance relative to the mesh-based trap designs for most of the design options.

In addition, structural considerations and the potential for scour favor the basin-based traps. The mesh-based traps will include pilings that can be subjected to scour during flood events, as the 100-year and lesser floods are estimated to extend into the broader river overbank where the sediment basin is to be located. Also, the loads of the flow and sediment on the meshes may be significant and yield infeasible designs. For instance, the forces of 2-inch cobbles hitting the 2-inch mesh when driven by 900 cfs peak flows will be significant. The mesh wire diameter will need to be large to withstand these forces, causing the screen to be dense and limiting the flow capacity. It is possible that the flow will back up and then overtop the sides of the trap causing sediment to reach unintended areas.

The potential for scour for the basin-based trap is much less and can be reduced with standard methods such as riprap and armoring. The main potential for structural impact is large grains raveling as bedload hits the downstream sill. This can also be prevented with standard riprap.

Therefore, the recommendation is for the basin-based trap to be constructed. Guidance on sizes versus performance for each of the two sites is provided below.

- There were 24 different sediment basin alternative designs considered for the Thurman I Arroyo trap and 24 for the Thurman II Arroyo Trap. For the Thurman I trap, 23 of the designs provided feasible alternatives. The design volumes ranged from 1.13 to 4.21 acre-feet. For the range of historic flows, these volumes would retain within the basin sediment all sizes above 16 mm. The Rio Grande is estimated in a normal annual peak flow period to be able to transport sediment sizes of 14 mm and larger, so it is expected that the accumulation of sediments in the Rio Grande would be minimized. These basins are estimated to require maintenance (clean-out) on average every 1 to 3 years, depending on the trap volume selected, but the historic pattern shows this interval to vary widely within a 30-year period. Any volumes greater than 4.21 acre-feet that are available within the site constraints would have better sediment retention performance and reduced frequency of cleanout.

- For the Thurman II trap, 21 of the designs provided feasible alternatives. The design volumes ranged from 0.9 to 2.6 acre-feet. For the range of historic flows, these volumes would retain within the basin sediment all sizes above 16 mm. The Rio Grande is estimated in a normal annual peak flow period to be able to transport sediment sizes of 14 mm and larger, so it is expected that the accumulation of sediments in the Rio Grande would be minimized. The basins are estimated to require maintenance (clean-out) on average every 0.5 to 1.3 years, depending on the trap volume selected, but the historic pattern shows this interval to vary widely within a 30-year period. Any volumes greater than 2.6 acre-feet that are available within the site constraints would have better sediment retention performance and reduced frequency of cleanout.

A.8 PROCESS SUMMARY - APPROACH FOR SEDIMENT TRAP PERFORMANCE EVALUATION

A.8.1 Arroyo Discharges

A long-term time series of arroyo discharges for Thurman I and II was developed using HEC-HMS Version 4.2. The input rainfall time series was based on the 31-year historical hourly rainfall from the Cabello gage (1983 – 2013). The SCS Unit Hydrograph method for transformation was used for the HEC-HMS model. The model output arroyo discharged at a 5-minute time step.

The HEC-HMS model was calibrated to the USGS Peak Discharge Rates (USGS, 2008). The calibration consisted of simulating the long-term record (31 years), conducting a return period analysis on the daily peak flows from the simulation using a Log Pearson Type III distribution, and then comparing the results to the USGS peak flows developed for the southwest U.S. The lag time coefficient in the SCS Unit Hydrograph was varied in a series of simulations until a good match between the simulated and USGS return period peak flows was obtained.

Long periods of no rain were removed from the rainfall record for the calibration effort to decrease the simulation time of the HEC-HMS (Version 4.2) model, and this rainfall record is herein referred to as the compressed rainfall time series. However, the compressed time series still represented the total rainfall during the 31-year period.

The calibrated model was then applied with the compressed rainfall time series to develop a compressed time series of arroyo discharges. All subsequent simulations of sediment loads and basin performance were based on the compressed time series.

A.8.2 Arroyo Sediment Loads

Soil and Water Assessment Tool (SWAT) computer software (Version SWAT 2012 rev. 664, with ArcSWAT 2012.10.19) was first used to estimate daily variations in sediment sizes and volumes from each arroyo into the Rio Grande, through the period of the historic rainfall record. SWAT was selected for this study primarily because of its full integration with the USDA SSURGO soils dataset, which was the best dataset available for sediment within the studied watersheds. The model demonstrated that larger particles were likely to be moved only rarely during short, widely separated periods during the historic record. The model predicted threshold velocities well under the velocities needed to move the larger particles armoring the fan material at the arroyo outlets. This result was estimated to be partially due to the limited detail available for the topography of the upper arroyos, and the inability to estimate dimensions of smaller, confined, low-flow channels where hydraulic stresses might be raised above those predicted by the model.

There is a likelihood that the motion of larger cobbles and boulders seen in the Rio Grande channel are associated with longer duration heavy rainfall (1 to 3 days) and resulting debris flow. Such an event may have occurred in 2008, per rainfall patterns in July 2008.

The arroyo discharge time series was used with a sediment transport formula to estimate the sediment load. The results of the SWAT model analysis were used to guide the transport formula parameters. For each time increment in the arroyo discharge record, the sediment load was estimated using the Peter-Meyers formula. The formula was coded in an Excel Workbook.

The application of the Peter-Meyers total load formula required inputs for bottom stress and grain size diameter. The specification of these inputs is provided in Attachment B, Calculation Method for Sediment Loads.

The simulated sediment load was calibrated to the average annual load based on the USACE data (USACE, 2007). The representative arroyo channel width was used as the calibration parameter and was varied in a series of simulations until the simulated sediment matched the USACE-based annual average load.

The final calibrated results provided a time series of arroyo flows and sediment loads.

A.8.3 Sediment Basin Performance Analysis

A sediment transport model for evaluating the sediment basin performance was coded in FORTRAN and used to evaluate each of the alternative basin designs. The model simulated the time series of sedimentation in the basin using the time series of arroyo discharges and sediment loads. The sediment was characterized with 14 grain size classes. The simulations represented 31 years of historic record.

The model determined the flow speed in the basin based on the instantaneous flow rate from the arroyo discharge, the basin geometry, and water depth in the basin. The flow speed was used with the Rouse parameter to determine the fraction of sediment load that was transported through the basin as bed load and suspended load. The model assumed that all bedload was trapped within the basin. For grain size classes transported as suspended load, the transport capacity was calculated. If the sediment concentration was higher than the capacity, sediment settled out into the basin at the prescribed settling speed. If the suspended sediment concentration was lower than the capacity, then sediment was eroded from the basin bed.

During the simulation, the available basin water storage volume would decrease as sediment accumulated in the basin. When the basin filled with sediment, it was recorded as a maintenance event, and the basin was restored to its initial volume.

For each alternative basin design, the basin performance model was applied for a 31-year simulation. The program recorded the amount of sediment that was trapped in the basin and the amount that exited the basin, and tracked these parameters for each grain size class. The number of maintenance events and their data were also recorded.

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Table A-1. Maximum Rainfall Depth by Duration Caballo Dam Gage 1983-2013

	Maximum Rainfall (inches)			
	1-Hour	6-Hour	12-Hour	24-Hour
1983	0.4	0.7	0.7	0.7
1984	0.3	1	1.2	2.2
1985	0.6	0.7	0.7	0.8
1986	1	1.9	2	2.2
1987	0.5	0.7	0.9	1.5
1988	0.8	0.9	0.9	0.9
1989	1.4	1.8	2	2
1990	0.7	1.2	1.5	1.9
1991	0.5	0.7	0.8	0.9
1992	0.4	0.6	0.7	0.8
1993	0.4	0.7	0.7	0.7
1994	0.3	1	1.2	2.2
1995	0.6	0.7	0.7	0.8
1996	1	1.9	2	2.2
1997	0.5	0.7	0.9	1.5
1998	0.8	0.9	0.9	0.9
1999	1.4	1.8	2	2
2000	0.7	1.2	1.5	1.9
2001	0.5	0.7	0.8	0.9
2002	0.4	0.6	0.7	0.8
2003	0.2	0.6	0.7	0.8
2004	0.3	0.6	0.7	1.2
2005	0.6	1	1	1.1
2006	0.8	1	1.2	1.7
2007	0.8	0.9	0.9	0.9
2008	0.8	1.8	2.1	3.3
2009	0.8	1.2	1.2	1.4
2010	1.1	1.3	1.6	1.6
2011	0.7	0.8	0.9	1.1
2012	0.2	0.7	0.7	0.7
2013	0.7	1.4	1.9	2.8

Table A-2. Depth-Duration-Frequency Data for El Paso Airport (from City of El Paso Drainage Design Manual)

Return Frequency (years)	Annual Chance (%)	Total Rainfall Depth (inches) by Duration				
		1 Hour	2 Hours	6 Hours	12 Hours	24 Hours
1	95%	0.41	0.52	0.66	0.72	0.8
2	50%	0.7	0.88	1.07	1.18	1.35
5	20%	0.97	1.22	1.46	1.61	1.83
10	10%	1.15	1.45	1.73	1.91	2.16
25	4%	1.41	1.79	2.11	2.33	2.6
50	2%	1.61	2.06	2.43	2.68	2.96
100	1%	1.84	2.36	2.78	3.06	3.34
250	0.4%	2.18	2.82	3.3	3.63	3.89
500	0.2%	2.47	3.21	3.74	4.12	4.35

Table A-3. Estimated Return Period for Peak Rainfall Depths at Caballo Dam Gage 1983-2013

	Return Period (years)				
	Hour	2-Hour	6-Hour	12-Hour	24-Hour
1983	1.0	0.8	1.1	1.0	0.9
1984	0.7	1.0	1.8	2.1	11.4
1985	1.7	1.5	1.1	1.0	1.0
1986	5.8	3.1	16.7	13.2	11.4
1987	1.3	1.5	1.1	1.4	2.9
1988	3.1	1.8	1.6	1.4	1.2
1989	24.4	8.9	12.8	13.2	7.6
1990	2.0	1.8	3.0	4.2	6.1
1991	1.3	1.0	1.1	1.2	1.2
1992	1.0	0.8	0.9	1.0	1.0
1993	1.0	0.8	1.1	1.0	0.9
1994	0.7	1.0	1.8	2.1	11.4
1995	1.7	1.5	1.1	1.0	1.0
1996	5.8	3.1	16.7	13.2	11.4
1997	1.3	1.5	1.1	1.4	2.9
1998	3.1	1.8	1.6	1.4	1.2
1999	24.4	8.9	12.8	13.2	7.6
2000	2.0	1.8	3.0	4.2	6.1
2001	1.3	1.0	1.1	1.2	1.2
2002	1.0	0.8	0.9	1.0	1.0
2003	0.5	0.8	0.9	1.0	1.0
2004	0.7	1.0	0.9	1.0	1.7
2005	1.7	3.1	1.8	1.6	1.5
2006	3.1	2.2	1.8	2.1	4.2
2007	3.1	2.2	1.6	1.4	1.2
2008	3.1	3.1	12.8	16.8	94.7
2009	3.1	1.8	3.0	2.1	2.3
2010	8.6	4.8	3.8	4.9	3.6
2011	2.0	1.8	1.3	1.4	1.5
2012	0.5	0.8	1.1	1.0	0.9
2013	2.0	2.2	4.5	9.8	38.9

Table A-4. Estimated Extreme Flows, Thurman I and II Arroyos per USGS Regression

Return Frequency (years)	Annual Chance (%)	Flowrate (cfs)	
		Thurman I (Area 5.8 sq mi)	Thurman II (Area 7.7 sq mi)
2	50%	325	369
5	20%	631	721
10	10%	896	1,025
25	4%	1,304	1,497
50	2%	1,664	1,915
100	1%	2,070	2,388
500	0.2%	3,226	3,746

Table A-5. Annual Peak Mean Daily Flow for Rio Grande Gages at El Paso and Caballo Dam, 1940-2013

Annual Peak Mean Daily Flow (cfs)					
Year	**Caballo Dam**	**El Paso**	**Year**	**Caballo Dam**	**El Paso**
1942	7,650	6,997	1969	2,930	1,659
1958	3,020	5,528	1968	2,470	1,629
1941	2,360	4,228	1974	2,683	1,619
1938	2,428	4,078	1953	2,820	1,579
1987	4,646	3,898	1990	2,534	1,571
1995	4,539	3,883	1973	2,810	1,559
1999	2,552	3,406	1985	2,293	1,549
1944	3,000	3,279	1946	2,730	1,539
1986	3,640	3,209	1993	2,529	1,539
1943	2,920	3,189	1978	1,992	1,489
1962	2,910	3,169	1960	3,130	1,479
1957	2,540	3,019	1992	2,317	1,444
2006	1,941	2,993	1981	2,265	1,439
2008	2,406	2,852	2001	2,597	1,430
1975	2,224	2,769	1989	2,816	1,419
1939	2,490	2,709	1980	2,485	1,409
1979	2,424	2,539	2005	2,437	1,387
1947	2,810	2,509	2009	2,617	1,345
1994	3,566	2,499	1976	2,350	1,329
1950	3,080	2,459	1965	2,840	1,319
2000	2,469	2,400	1982	2,242	1,309
1940	2,550	2,199	2004	2,065	1,271
2010	2,506	2,143	2007	2,216	1,264
1961	2,820	2,139	1963	3,050	1,249
1988	3,292	2,139	2015	2,716	1,211
2002	2,603	2,111	1955	2,240	1,200
1984	2,511	1,939	2011	2,128	1,161
1948	3,030	1,929	1951	2,307	1,150
1949	2,830	1,909	1983	2,217	1,090
1966	3,410	1,869	1977	2,044	1,076
1967	2,430	1,869	2014	2,613	1,063
1998	2,889	1,839	1971	2,211	1,060
1945	2,680	1,819	1954	1,840	971
1959	2,786	1,819	2003	1,884	957
1970	2,693	1,809	1972	2,112	862
1952	2,836	1,749	2012	2,139	808
1991	2,250	1,733	2013	2,412	770
1996	2,385	1,705	1956	2,240	728
1997	2,635	1,680	1964	1,350	406

Table A-6. Log Pearson Type III Analysis of El Paso Rio Grande Gage, 1940-2013

Annual Exceedance Probability (Annual Chance %)	Return Period (years)	BULL.17B Estimate of Flowrate (cfs)	95% Confidence Limit Lower	95% Confidence Limit Upper
95%	1.1	879	765	986
90%	1.1	1,007	889	1,117
80%	1.3	1,196	1,075	1,313
67%	1.5	1,418	1,291	1,546
50%	2	1,711	1,571	1,863
43%	2	1,855	1,705	2,023
20%	5	2,542	2,318	2,825
10%	10	3,175	2,855	3,609
4%	25	4,074	3,587	4,770
2%	50	4,817	4,175	5,765
1%	100	5,626	4,802	6,876
0.50%	200	6,510	5,473	8,120
0.20%	500	7,807	6,438	9,992

Table A-7. Estimated Threshold Sediment Diameters Moved, 1983-2013, Using the Shield Relation

	Number of Hours D_{50} Size Moved					
	Gravel				Cobbles	
	Threshold D_{50} (inches)					
	0.07	**0.2**	**1**	**2**	**3**	**4**
Subbasin 19 Thurman I						
1988	25	6	0	0	0	0
1989	205	72	7	2	0	0
1990	160	40	4	2	2	0
1991	26	17	0	0	0	0
1992	32	11	0	0	0	0
1993	49	29	0	0	0	0
1994	161	27	0	0	0	0
1995	48	32	3	2	1	0
1996	116	26	3	2	1	0
1997	103	40	1	0	0	0
1998	23	5	0	0	0	0
1999	175	72	3	0	0	0
2000	157	36	5	4	1	0
2001	25	15	0	0	0	0
2002	28	12	0	0	0	0
2003	41	8	0	0	0	0
2004	59	31	4	0	0	0
2005	91	23	0	0	0	0
2006	204	72	3	2	1	0
2007	116	15	0	0	0	0
2008	103	37	10	5	2	0
2009	56	27	0	0	0	0
2010	119	38	0	0	0	0
2011	41	16	0	0	0	0
2012	21	5	0	0	0	0
2013	324	99	21	7	3	0

	Number of Hours D_{50} Size Moved					
	Gravel				Cobbles	
	Threshold D_{50} (inches)					
	0.07	**0.2**	**1**	**2**	**3**	**4**
Subbasin 52 Thurman II						
1988	25	6	0	0	0	0
1989	205	72	0	2	0	0
1990	17	4	2	0	2	0
1991	3	3	0	0	0	0
1992	12	1	0	0	0	0
1993	49	29	0	0	0	0
1994	12	0	0	0	0	0
1995	12	3	0	0	0	0
1996	4203	4	3	2	1	0
1997	3510	40	1	0	0	0
1998	23	5	0	0	0	0
1999	175	72	3	0	0	0
2000	5460	10	6	0	1	0
2001	5	15	0	0	0	0
2002	3	1	0	0	0	0
2003	0	8	0	0	0	0
2004	10	31	0	0	0	0
2005	91	12	0	0	0	0
2006	204	72	3	2	1	0
2007	116	0	0	0	0	0
2008	3851	37	10	3	0	0
2009	56	27	0	0	0	0
2010	119	38	0	0	0	0
2011	2194	0	0	0	0	0
2012	21	5	0	0	0	0
2013	117	99	21	7	3	0

Table A-8. Estimated Threshold Sediment Diameters Moved, 1983-2013, Using the van Rijn Relation

	Number of Hours D_{50} Size Moved						
	Gravel				Cobbles		
	Threshold D_{50} (inches)						
	0.07	**0.2**	**1**	**2**	**3**	**4**	**6**
Subbasin 19 Thurman I							
1988	127	64	0	0	0	0	0
1989	64	28	4	2	1	0	0
1990	32	10	3	2	2	1	1
1991	13	3	0	0	0	0	0
1992	10	1	0	0	0	0	0
1993	25	2	0	0	0	0	0
1994	23	0	0	0	0	0	0
1995	22	7	3	2	1	1	0
1996	21	8	3	2	2	1	0
1997	33	5	0	0	0	0	0
1998	2	1	0	0	0	0	0
1999	62	31	2	0	0	0	0
2000	34	16	5	4	3	1	0
2001	12	2	0	0	0	0	0
2002	10	0	0	0	0	0	0
2003	7	0	0	0	0	0	0
2004	28	6	2	0	0	0	0
2005	19	6	0	0	0	0	0
2006	71	17	3	2	2	1	0
2007	14	3	0	0	0	0	0
2008	37	21	9	6	3	2	1
2009	24	7	0	0	0	0	0
2010	35	8	0	0	0	0	0
2011	15	2	0	0	0	0	0
2012	4	0	0	0	0	0	0
2013	74	39	18	9	6	3	1

	Number of Hours D_{50} Size Moved						
	Gravel				Cobbles		
	Threshold D_{50} (inches)						
	0.07	**0.2**	**1**	**2**	**3**	**4**	**6**
Subbasin 52 Thurman II							
1988	1	0	0	0	0	0	0
1989	64	10	2	0	0	0	0
1990	24	4	3	3	3	2	1
1991	9	0	0	0	0	0	0
1992	5	0	0	0	0	0	0
1993	14	0	0	0	0	0	0
1994	38	0	0	0	0	0	0
1995	13	3	2	0	0	0	0
1996	17	5	3	2	2	1	0
1997	20	0	0	0	0	0	0
1998	1	0	0	0	0	0	0
1999	61	8	2	1	0	0	0
2000	28	10	6	3	3	2	1
2001	9	0	0	0	0	0	0
2002	4	0	0	0	0	0	0
2003	5	0	0	0	0	0	0
2004	23	0	0	0	0	0	0
2005	18	0	0	0	0	0	0
2006	56	6	3	3	2	1	0
2007	8	0	0	0	0	0	0
2008	29	10	5	3	3	2	1
2009	16	0	0	0	0	0	0
2010	23	1	0	0	0	0	0
2011	12	0	0	0	0	0	0
2012	0	0	0	0	0	0	0
2013	2199	23	10	7	5	3	1

Table A-9. Comparison of USGS Regression-Based Discharges and the Model Simulations for Thurman I Arroyo

Return Period (yrs)	Thurman I	
	USGS (m^3/s)	HEC-HMS (m^3/s)
2	9.2	11.3
5	17.9	21.6
10	25.4	29.3
25	36.9	39.5
50	47.2	47.3
100	58.7	55.0

Table A-10. Comparison of USGS Regression-Based Discharges and the Model Simulations for Thurman II Arroyo

Return Period (yrs)	Thurman II	
	USGS (m^3/s)	HEC-HMS (m^3/s)
2	10.5	12.7
5	20.4	24.5
10	29.0	33.3
25	42.4	45.1
50	54.3	54.1
100	67.7	62.9

Table A-11. Percentage of Sediment Load Trapped for Different Void Space Filling Ratios

Assumed Fraction of Void Space Occupied (max = 0.35)	Percent Trapped by Mesh	Addional Percent Trapped inVoids	Final Trapped Percent
0.1	37	3.7	40.7
0.2	37	7.4	44.4
0.3	37	11.1	48.1

Table A-12. Mesh-Based Trap Performance for Thurman I Arroyo

Maximum Filling Height (ft)	Angle of Fill (degrees)	Maximum Trapping Volume (af)	Annual Load (af/yr)	Annual Load Trapped* (af/yr)	Maintenance Interval (yrs)
2	0	2.22	1.12	0.50	4.46 (Scenario A, Table 14)
2	2	1.44	1.12	0.50	2.90 (Scenario B, Table 14)
2	5	0.58	1.12	0.50	1.16
2	10	0.29	1.12	0.50	0.57
2.5	0	2.77	1.12	0.50	5.57
2.5	2	2.25	1.12	0.50	4.53
2.5	5	0.90	1.12	0.50	1.81 (Scenario C, Table 14)
2.5	10	0.45	1.12	0.50	0.90

* Assumed 44.4 % of annual load is trapped.

Table A-13. Mesh-Based Trap Performance for Thurman II Arroyo

Maximum Filling Height (ft)	Angle of Fill (degrees)	Maximum Trapping Volume (af)	Annual Load (af/yr)	Annual Load Trapped* (af/yr)	Maintenance Interval (yrs)
2	0	1.58	1.98	0.88	1.79 (Scenario A, Table 15)
2	2	1.03	1.98	0.88	1.17 (Scenario B, Table 15)
2	5	0.41	1.98	0.88	0.47
2	10	0.20	1.98	0.88	0.23
2.5	0	1.97	1.98	0.88	2.24
2.5	2	1.60	1.98	0.88	1.82
2.5	5	0.64	1.98	0.88	0.73 (Scenario C, Table 15)
2.5	10	0.32	1.98	0.88	0.36

* Assumed 44.4 % of annual load is trapped.

Table A-14. Simulated Maintenance Events for the Mesh-Based Trap for Thurman I Arroyo

Scenario A from Table 12			Scenario B from Table 12			Scenario C from Table 12		
Maintenance Date	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**
7/24/1988	2023	**ave**	6/29/1986	1267	**ave**	6/29/1986	1267	**ave**
8/4/1989	376	1306	6/22/1989	1089	838	5/17/1987	322	561
7/2/1995	2158	**min**	8/4/1989	43	**min**	6/22/1989	767	**min**
7/24/1998	1118	376	6/28/1990	328	43	6/27/1989	5	5
8/4/1999	376	**max**	6/28/1996	2192	**max**	8/4/1989	38	**max**
8/6/2006	2559	2559	7/24/1998	756	2192	3/24/1990	232	1796
7/9/2008	703		7/6/1999	347		9/15/1991	540	
8/19/2011	1136		3/23/2000	261		6/28/1996	1748	
			5/27/2005	1891		9/12/1996	76	
			7/22/2007	786		6/22/1999	1013	
			7/27/2008	371		6/27/1999	5	
			7/24/2010	727		8/4/1999	38	
			9/13/2013	1147		3/23/2000	232	
						9/14/2001	540	
						8/15/2006	1796	
						8/2/2007	352	
						7/9/2008	342	
						6/29/2009	355	
						7/24/2010	390	
						9/11/2013	1145	
						9/17/2013	6	

Table A-15. Simulated Maintenance Events for the Mesh-Based Trap for Thurman II Arroyo

Scenario A from Table 13			Scenario B from Table 13			Scenario C from Table 13		
Maintenance Date	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**
6/29/1986	1267	ave	6/29/1986	1	ave	8/12/1985	946	ave
8/4/1987	401	560	6/30/1986	321	331	6/29/1986	321	234
6/22/1989	688	min	5/17/1987	434	min	6/30/1986	1	min
7/5/1989	13	13	7/24/1988	333	1	7/1/1986	1	1
8/4/1989	30	max	6/22/1989	5	max	8/4/1987	399	max
6/28/1990	328	2230	6/27/1989	38	1830	7/24/1988	355	1386
7/2/1995	1830		8/4/1989	1		6/22/1989	333	
6/28/1996	362		8/5/1989	231		6/27/1989	5	
7/24/1998	756		3/24/1990	96		6/28/1989	1	
6/22/1999	333		6/28/1990	1830		7/5/1989	7	

Scenario A from Table 13			Scenario B from Table 13			Scenario C from Table 13		
Maintenance Date	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**	**Maintenance Date**	**Days Since Last Maintenance**	**Stats (days)**
8/4/1999	43		7/2/1995	362		8/4/1989	30	
8/30/1999	26		6/28/1996	1		8/5/1989	1	
6/28/2000	303		6/29/1996	755		8/30/1989	25	
8/6/2006	2230		7/24/1998	333		6/19/1990	293	
8/22/2006	16		6/22/1999	5		6/28/1990	9	
7/9/2008	687		6/27/1999	9		9/15/1991	444	
8/5/2008	27		7/6/1999	29		7/2/1995	1386	
7/24/2010	718		8/4/1999	26		6/28/1996	362	
8/19/2011	391		8/30/1999	302		6/29/1996	1	
9/13/2013	756		6/27/2000	1795		6/30/1996	1	
			5/27/2005	446		8/3/1997	399	
			8/16/2006	340		7/24/1998	355	
			7/22/2007	11		6/22/1999	333	
			8/2/2007	342		6/27/1999	5	
			7/9/2008	27		6/28/1999	1	
			8/5/2008	374		7/6/1999	8	
			8/14/2009	344		7/7/1999	1	
			7/24/2010	391		8/4/1999	28	
			8/19/2011	756		8/5/1999	1	
			9/13/2013	4		3/23/2000	231	
						6/27/2000	96	
						9/14/2001	444	
						5/27/2005	1351	
						8/15/2006	445	
						8/22/2006	7	
						8/2/2007	345	
						8/4/2007	2	
						7/9/2008	340	
						7/27/2008	18	
						8/5/2008	9	
						6/29/2009	328	
						7/24/2010	390	
						7/25/2010	1	
						7/2/2011	342	
						8/19/2011	48	
						9/11/2013	754	
						9/13/2013	2	
						9/17/2013	4	

Table A-16. Effect of Hiding and Exposure on the Critical Stress for the Bypassed Sediment Distribution

Grain Size (mm)	Tc (lbf/ft^2)	Tc*(lbf/ft^2)
0.125	0.003	0.038
0.25	0.004	0.030
0.5	0.006	0.026
1	0.010	0.030
2	0.027	0.057
4	0.056	0.081
8	0.119	0.115
16	0.255	0.160
32	0.541	0.208
50.8	0.925	0.248

Table A-17. Basin-Based Trap Performance for Thurman I Arroyo

Critical Volume (fraction)	Bottom Width (ft)	Height (ft)	Max Vol (af)	Maintenance Interval (yrs)	Trapped (af/yr)	Bypassed (af/yr)	Percent Passed	Bypassed Sediment	
								Max (mm)	Med (mm)
0.75	50.0	4.0	1.50	1.19	0.95	0.17	15.1	8.00	0.50
0.75	50.0	3.0	1.13	1.11	0.80	0.32	28.9	16.00	1.00
0.75	50.0	2.0	0.76	1.00	0.58	0.54	48.1	32.00	4.00
0.75	100.0	4.0	2.40	1.72	1.09	0.03	3.0	4.00	0.50
0.75	100.0	3.0	1.81	1.41	0.99	0.13	11.5	4.00	0.50
0.75	100.0	2.0	1.22	1.19	0.80	0.32	28.7	16.00	1.00
0.75	150.0	4.0	3.30	2.38	1.11	0.01	0.5	4.00	0.50
0.75	150.0	3.0	2.49	1.82	1.08	0.04	3.5	4.00	0.50
0.75	150.0	2.0	1.67	1.48	0.88	0.24	21.5	8.00	0.50
0.75	200.0	4.0	4.21	3.10	1.12	0.00	0.0	1.00	0.50
0.75	200.0	3.0	3.17	2.21	1.09	0.03	2.9	4.00	0.50
0.75	200.0	2.0	2.13	1.63	0.98	0.14	12.1	4.00	0.50
0.50	50.0	4.0	1.50	0.70	1.11	0.01	1.2	4.00	0.50
0.50	50.0	3.0	1.13	0.57	1.02	0.10	9.0	8.00	0.50
0.50	50.0	2.0	0.76	0.50	0.78	0.34	30.3	16.00	1.00
0.50	100.0	4.0	2.40	1.11	1.12	0.00	0.1	2.00	0.50
0.50	100.0	3.0	1.81	0.84	1.11	0.01	0.8	4.00	0.50
0.50	100.0	2.0	1.22	0.63	1.00	0.13	11.2	8.00	0.50
0.50	150.0	4.0	3.30	1.55	1.12	0.00	0.0	1.00	0.50
0.50	150.0	3.0	2.49	1.15	1.12	0.00	0.2	1.00	0.50
0.50	150.0	2.0	1.67	0.79	1.08	0.04	3.3	4.00	0.50
0.50	200.0	4.0	4.21	1.94	1.12	0.00	0.0	-	-
0.50	200.0	3.0	3.17	1.48	1.12	0.00	0.0	1.00	0.50
0.50	200.0	2.0	2.13	1.00	1.11	0.01	1.0	2.00	0.50

Table A-18. Basin-Based Trap Performance for Thurman II Arroyo

Critical Volume (fraction)	Bottom Width (ft)	Height (ft)	Max Vol (af)	Maintenance Interval (yrs)	Trapped (af/yr)	Bypassed (af/yr)	Percent Passed	Bypassed Sediment	
								Max (mm)	Med (mm)
0.75	50.0	4.0	1.2	0.58	1.6	0.4	21.0	8.00	1.00
0.75	50.0	3.0	0.9	0.53	1.3	0.7	34.3	16.0	2.00
0.75	50.0	2.0	0.6	0.49	0.9	1.0	52.6	50.0	4.00
0.75	100.0	4.0	1.9	0.78	1.9	0.1	5.3	8.00	0.50
0.75	100.0	3.0	1.4	0.69	1.6	0.4	19.6	8.00	0.50
0.75	100.0	2.0	1.0	0.61	1.2	0.8	38.7	32.0	1.00
0.75	150.0	4.0	2.6	1.03	2.0	0.0	0.8	4.00	0.50
0.75	150.0	3.0	2.0	0.84	1.8	0.2	9.3	8.00	0.50
0.75	150.0	2.0	1.3	0.74	1.4	0.6	30.8	16.0	1.00
0.75	200.0	4.0	3.3	1.29	2.0	0.0	0.2	2.00	0.25
0.75	200.0	3.0	2.5	1.00	1.9	0.1	3.0	4.00	0.50
0.75	200.0	2.0	1.7	0.84	1.6	0.4	21.5	8.00	0.50
0.50	50.0	4.0	1.2	0.32	1.9	0.0	1.7	4.00	0.50
0.50	50.0	3.0	0.9	0.26	1.8	0.2	10.4	8.00	0.50
0.50	50.0	2.0	0.6	0.23	1.3	0.7	32.9	32.0	0.50
0.50	100.0	4.0	1.9	0.50	2.0	0.0	0.3	2.00	0.12
0.50	100.0	3.0	1.4	0.38	2.0	0.0	1.2	4.00	0.25
0.50	100.0	2.0	1.0	0.30	1.7	0.3	15.1	8.00	0.50
0.50	150.0	4.0	2.6	0.67	2.0	0.0	0.1	1.00	0.12
0.50	150.0	3.0	2.0	0.52	2.0	0.0	0.3	2.00	0.25
0.50	150.0	2.0	1.3	0.36	1.9	0.1	4.0	4.00	0.50
0.50	200.0	4.0	3.3	0.86	2.0	0.0	0.0	1.00	0.12
0.50	200.0	3.0	2.5	0.65	2.0	0.0	0.1	1.00	0.25
0.50	200.0	2.0	1.7	0.44	2.0	0.0	1.2	4.00	0.25

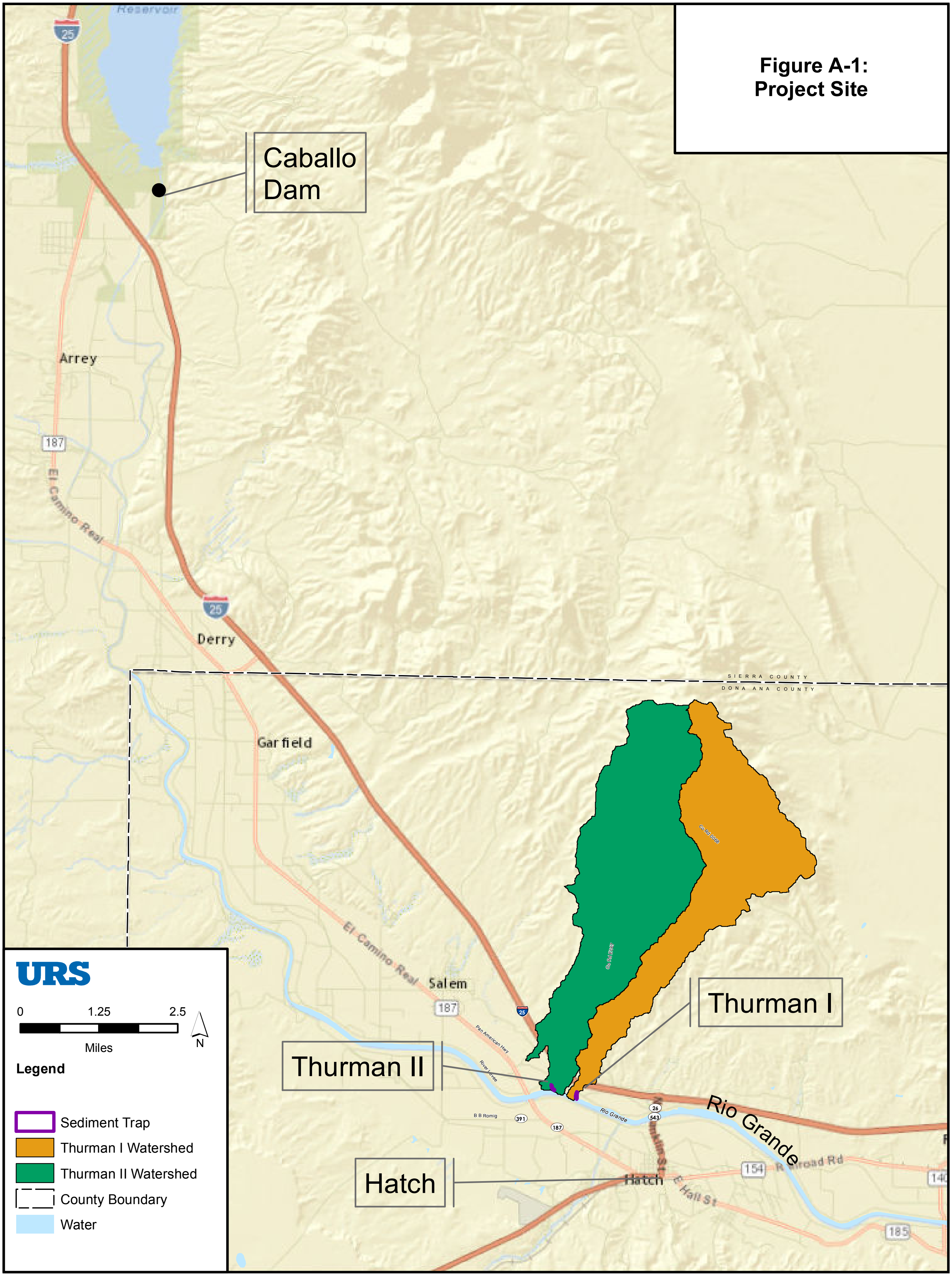


Figure A-1: Project Site

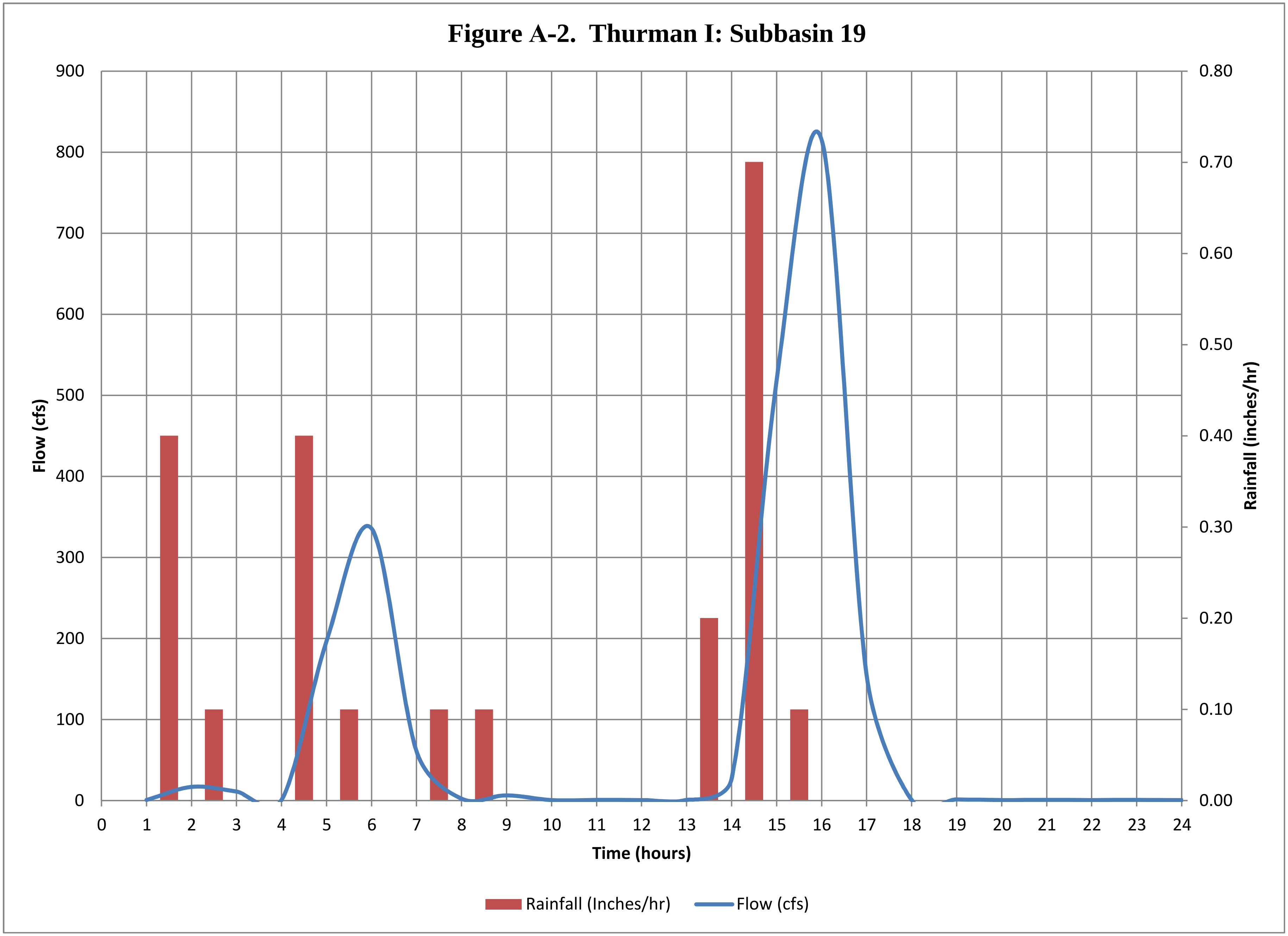


Figure A-2. Thurman I: Subbasin 19
Flow (cfs)
Rainfall (inches/hr)
Time (hours)
0
100
200
300
400
500
600
700
800
900
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Rainfall (Inches/hr)
Flow (cfs)

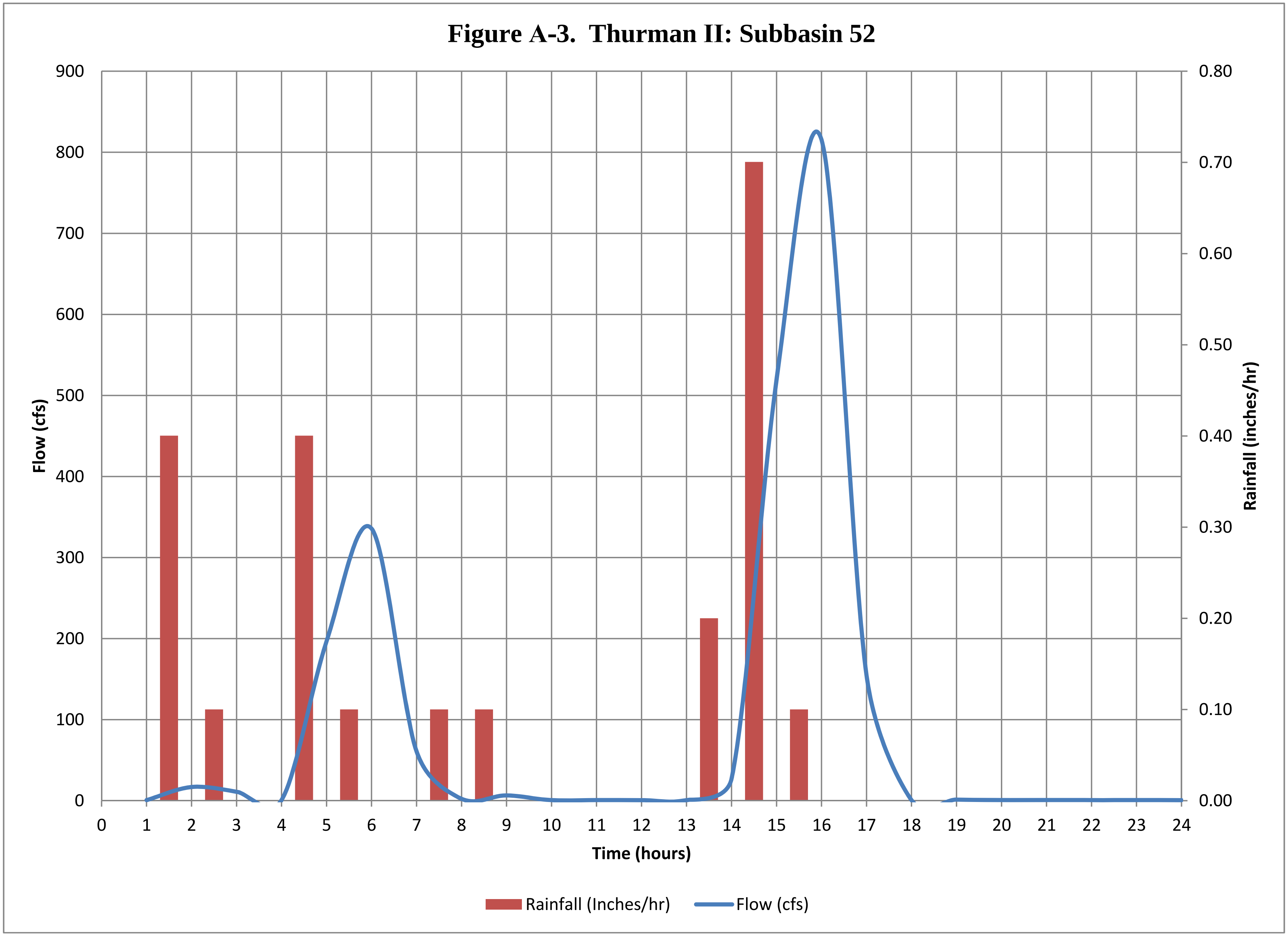


Figure A-3. Thurman II: Subbasin 52
Flow (cfs)
Rainfall (inches/hr)
Time (hours)
0
100
200
300
400
500
600
700
800
900
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Rainfall (Inches/hr)
Flow (cfs)

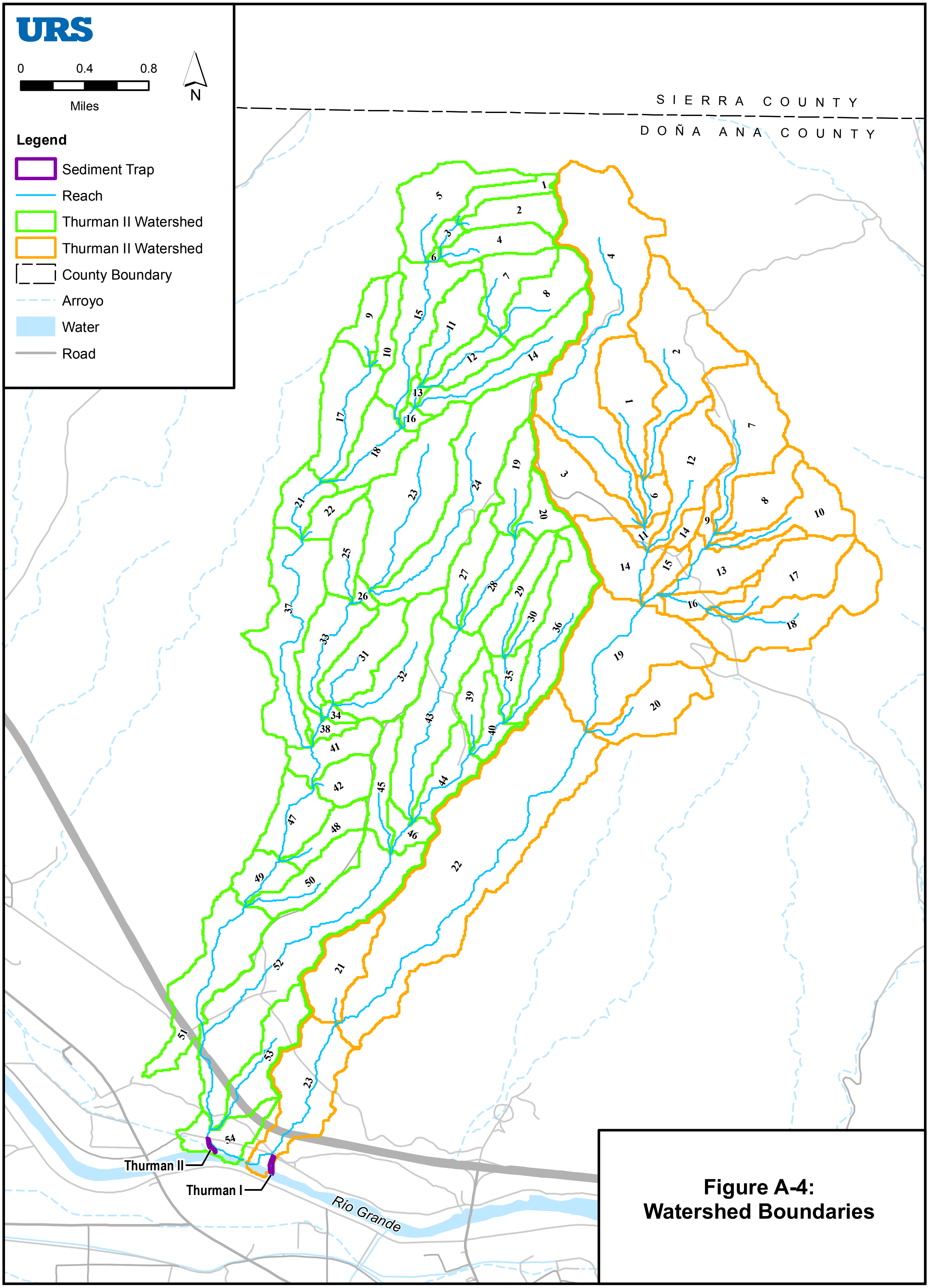


Figure A-4: Watershed Boundaries

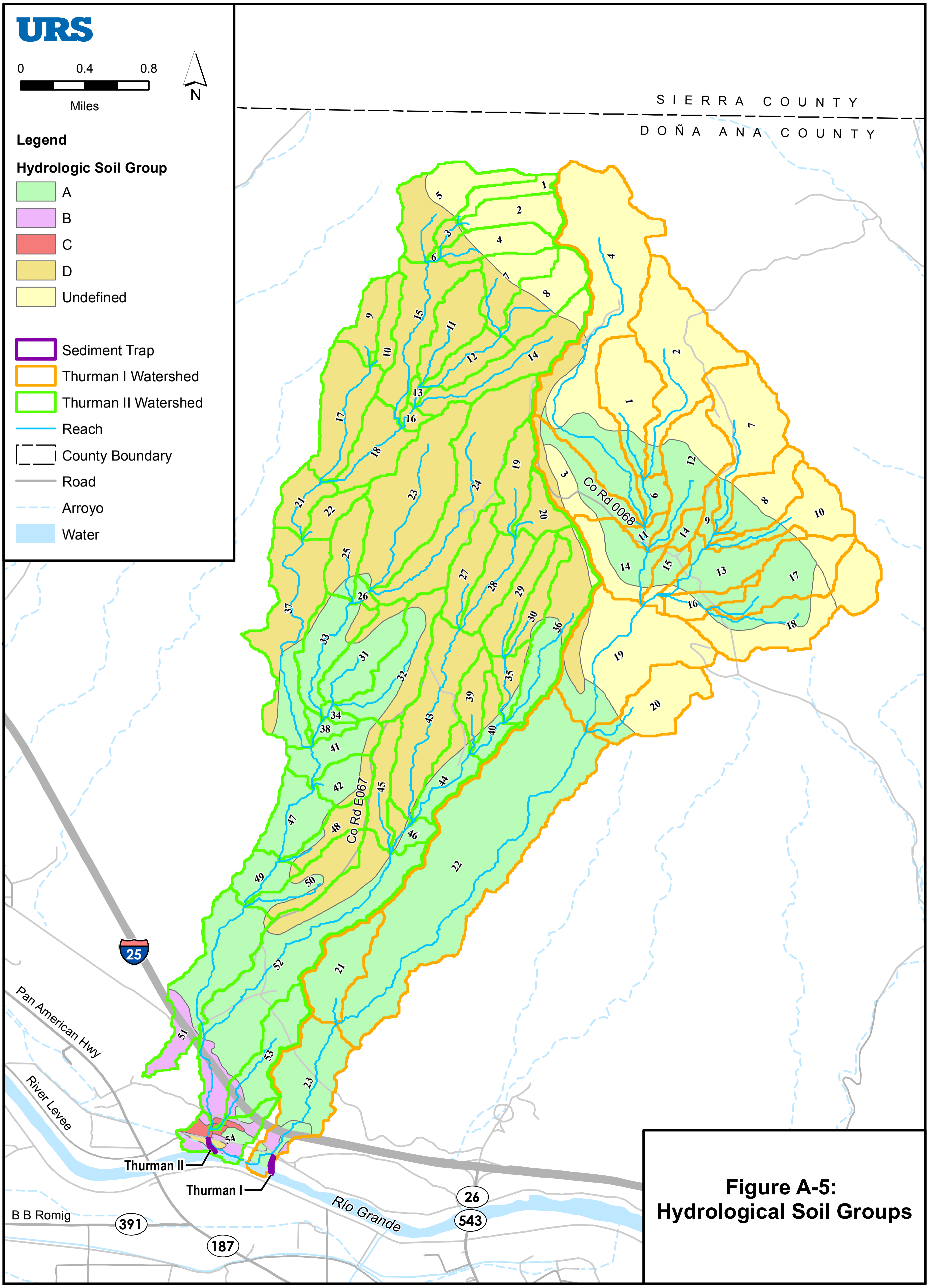


Figure A-5: Hydrological Soil Groups

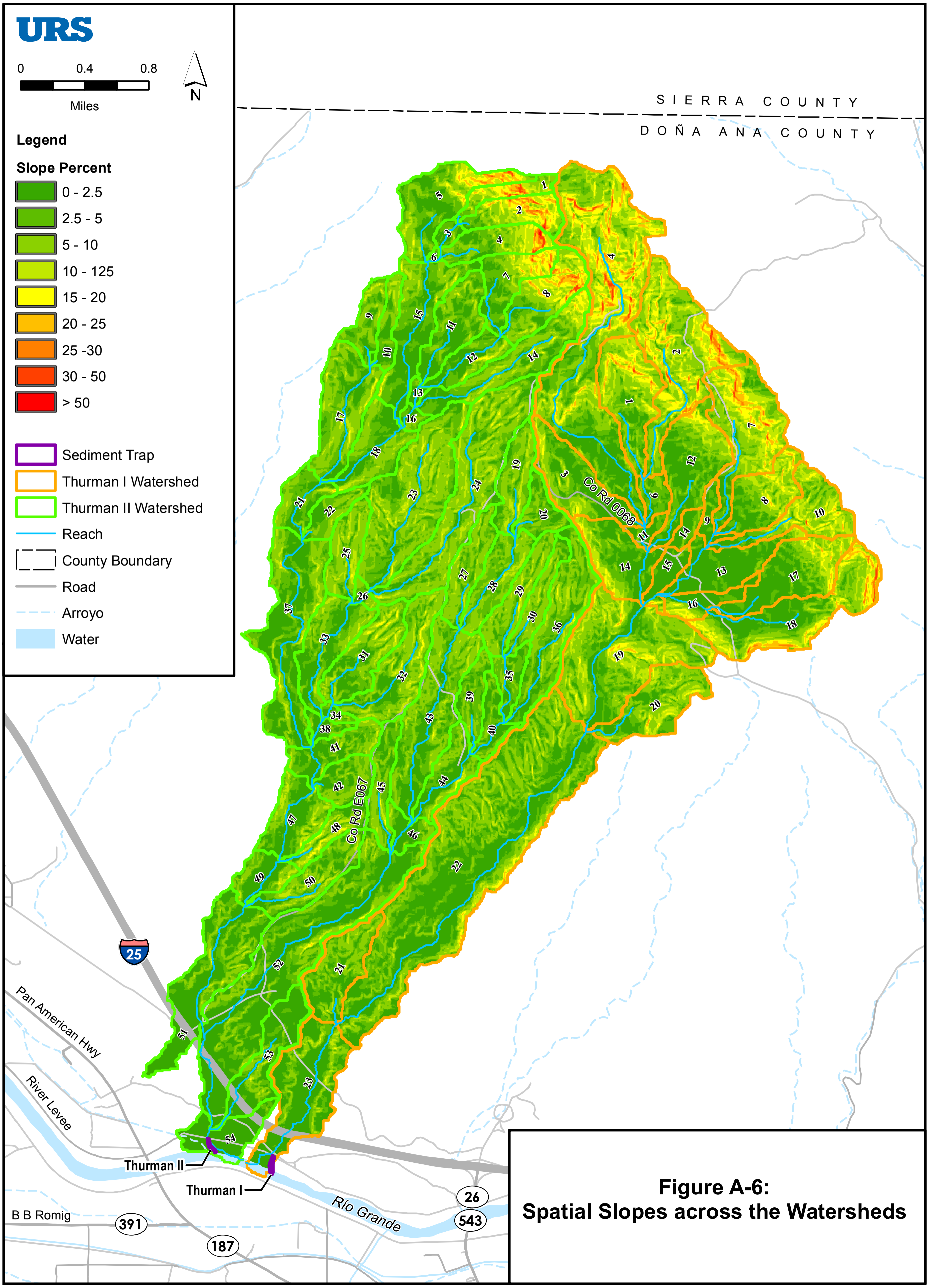
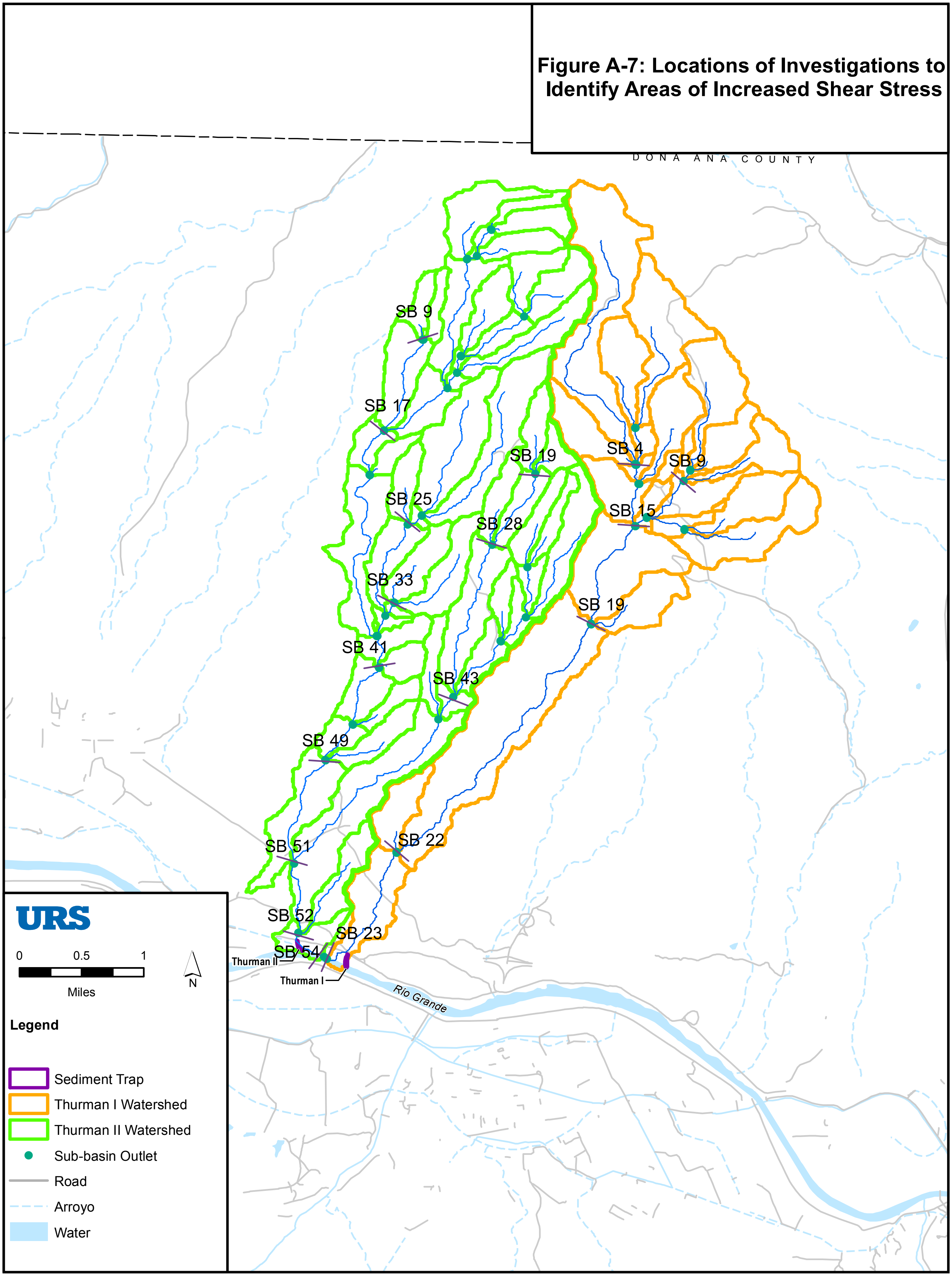


Figure A-6: Spatial Slopes across the Watersheds

Figure A-7: Locations of Investigations to Identify Areas of Increased Shear Stress



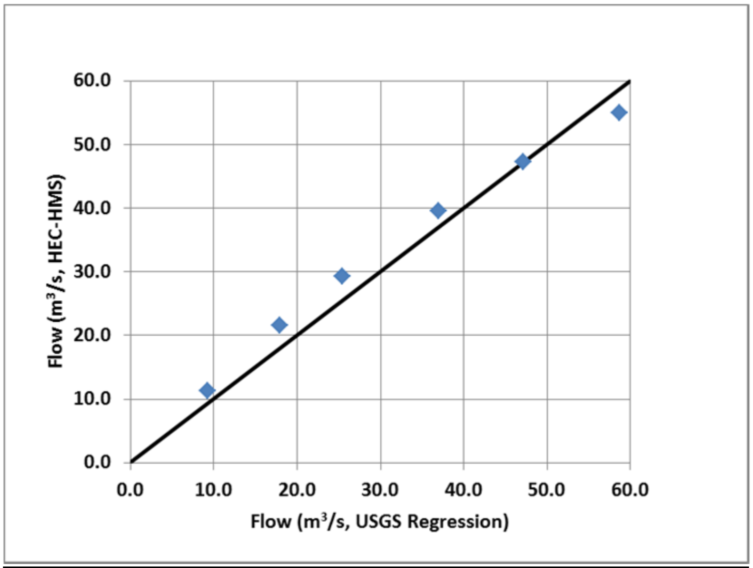


Figure A-8. Comparison of USGS Regression-Based Discharges and Results of Return Period Analysis of Model Output for Thurman I Arroyo

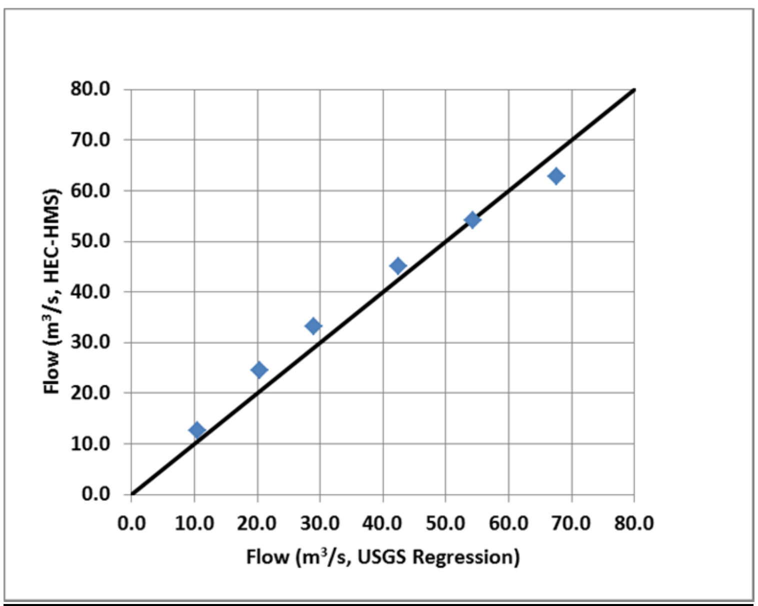


Figure A-9. Comparison of USGS Regression-Based Discharges and Results of Return Period Analysis of Model Output for Thurman II Arroyo

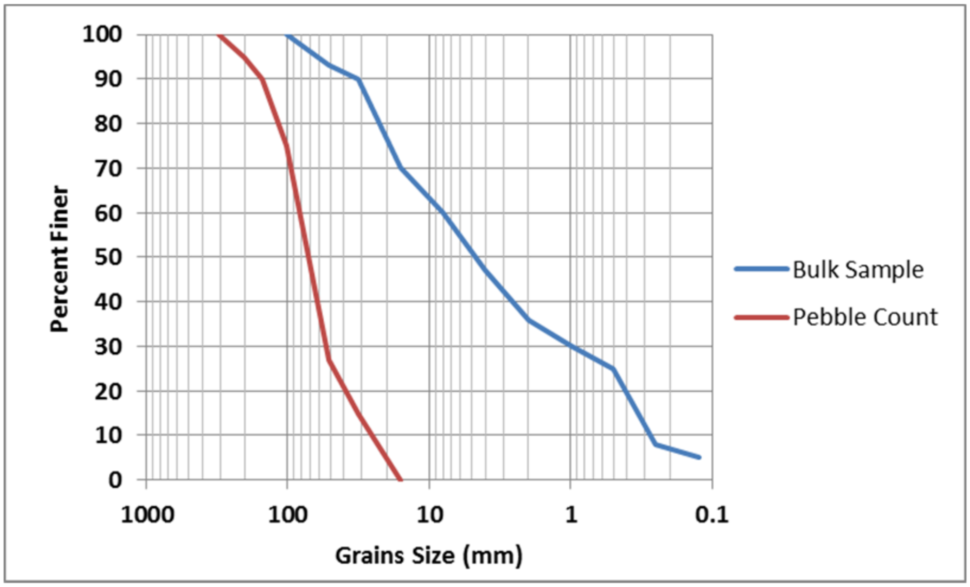


Figure A-10. Digitized Grain Size Curves for the Bulk Sample and Pebble Count Data Collected from Deposits in Problem Area 2

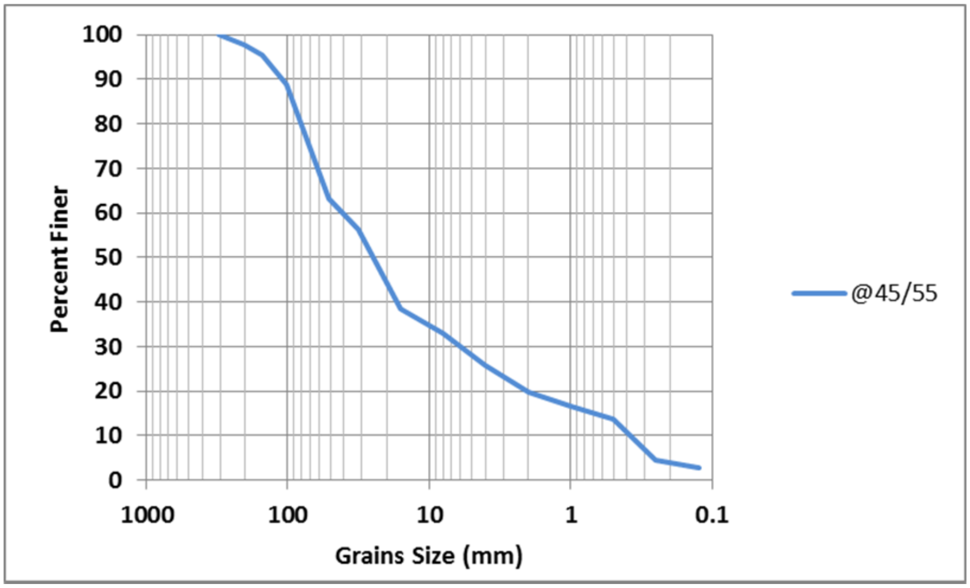


Figure A-11. Grain Size Curve after Merging the Bulk Sample and Pebble Count Data Collected from Deposits in Problem Area 2 Using the 45/55 Ratio

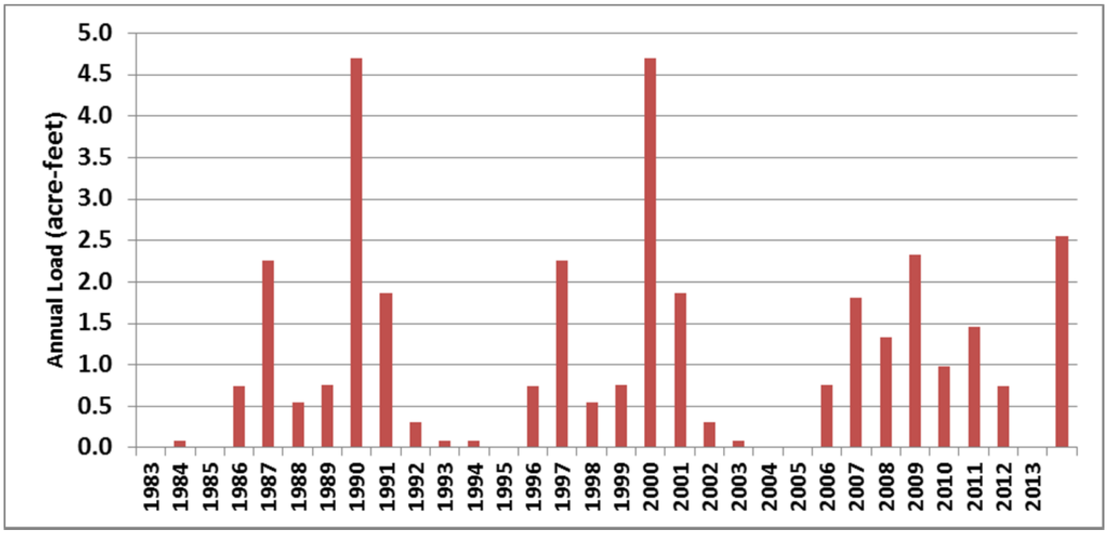


Figure A-12. Simulated Annual Sediment Loads from Thurman I Arroyo

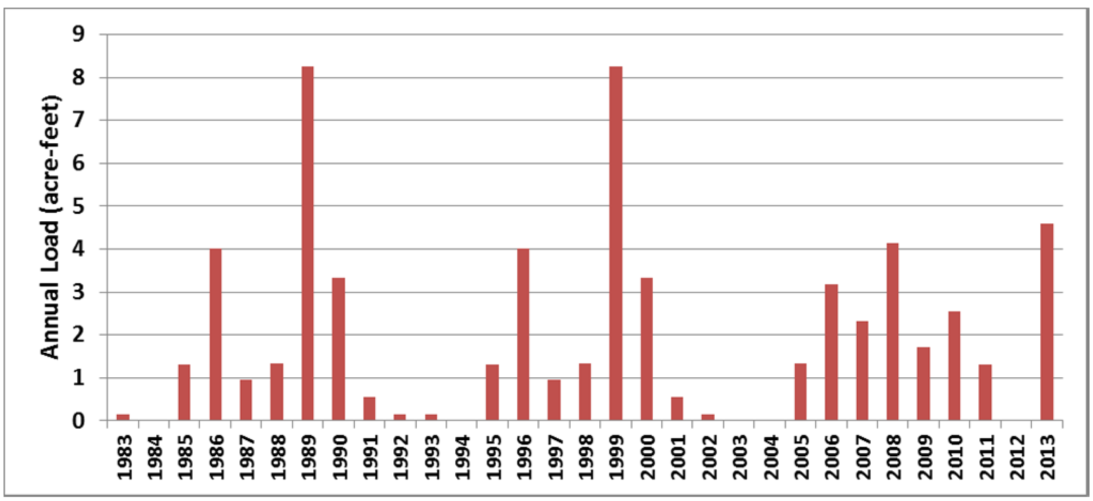


Figure A-13. Simulated Annual Sediment Loads from Thurman II Arroyo

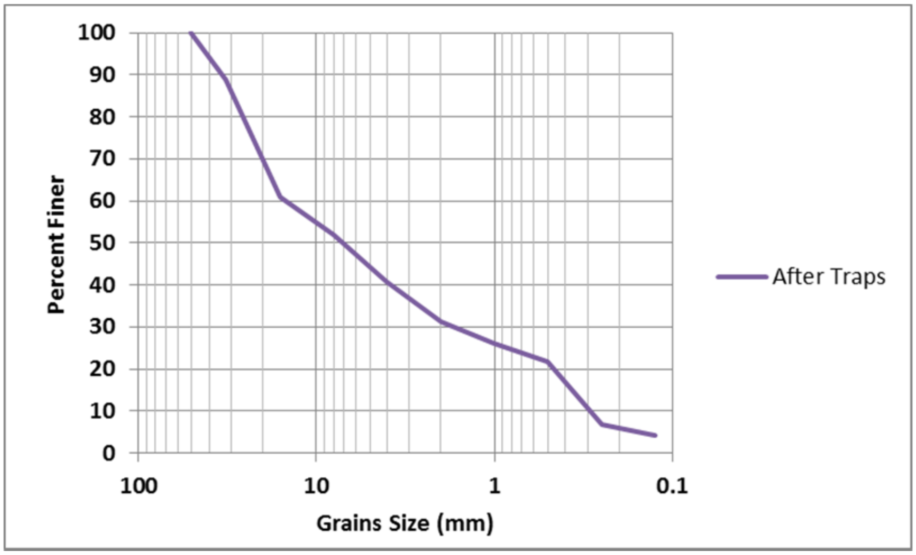


Figure A-14. Grain Size Curve of Sediment Estimated to Pass Through the Mesh-Based Trap

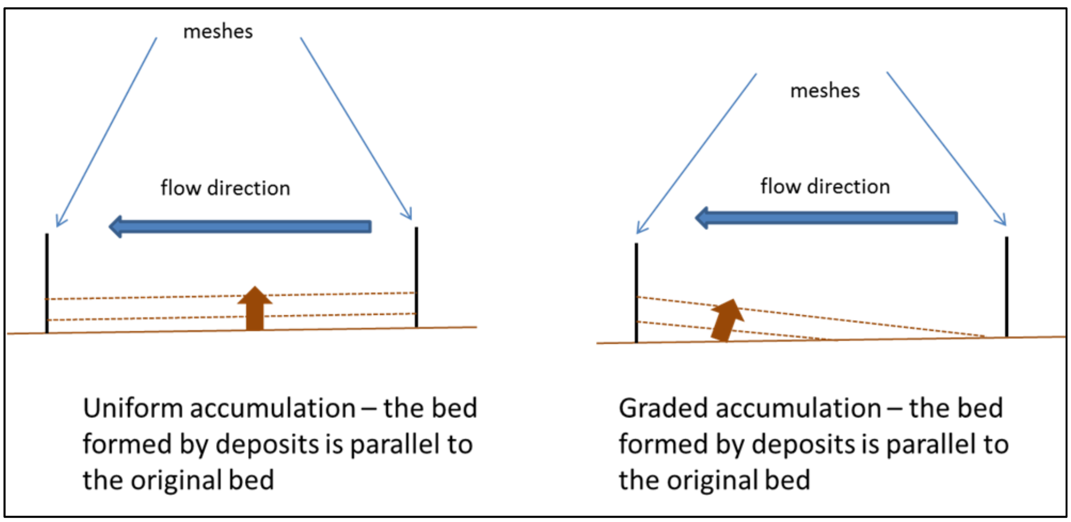


Figure A-15. Uniform and Sloped Filling Patterns.

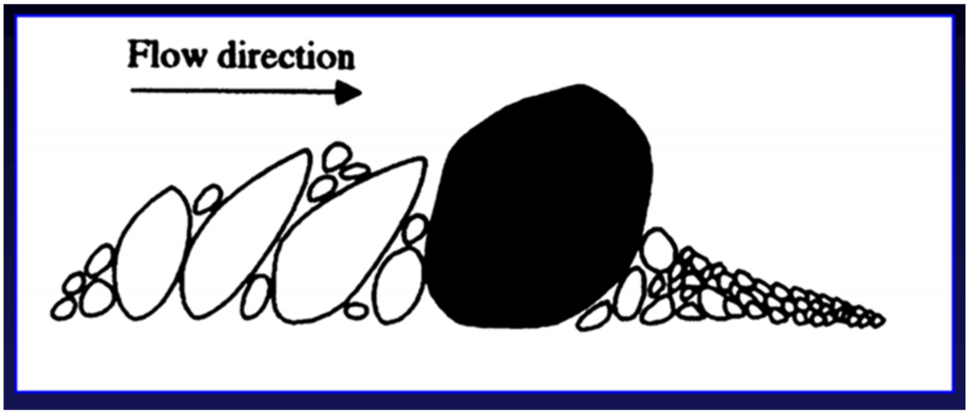


Figure A-16. Schematic Showing Hiding and Exposure of Variable-Sized Grains

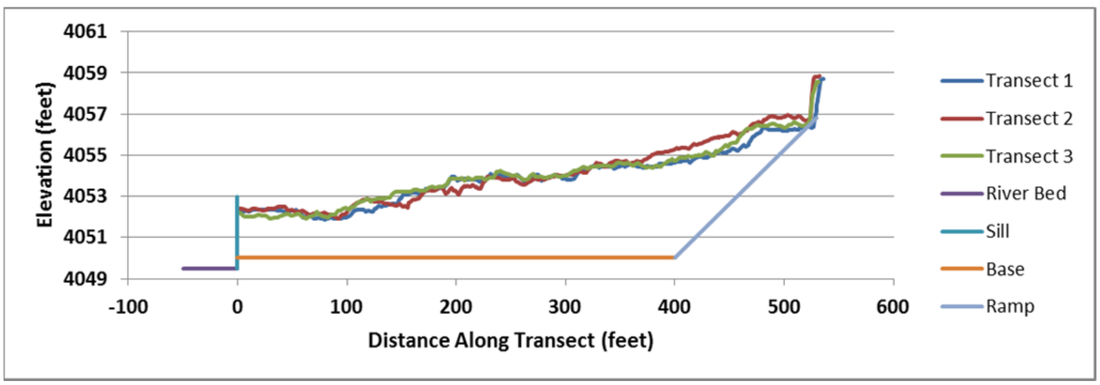


Figure A-17. Cross-Section of Basin-Based Trap and Geometric Parameters (Sill, Base, Ramp)

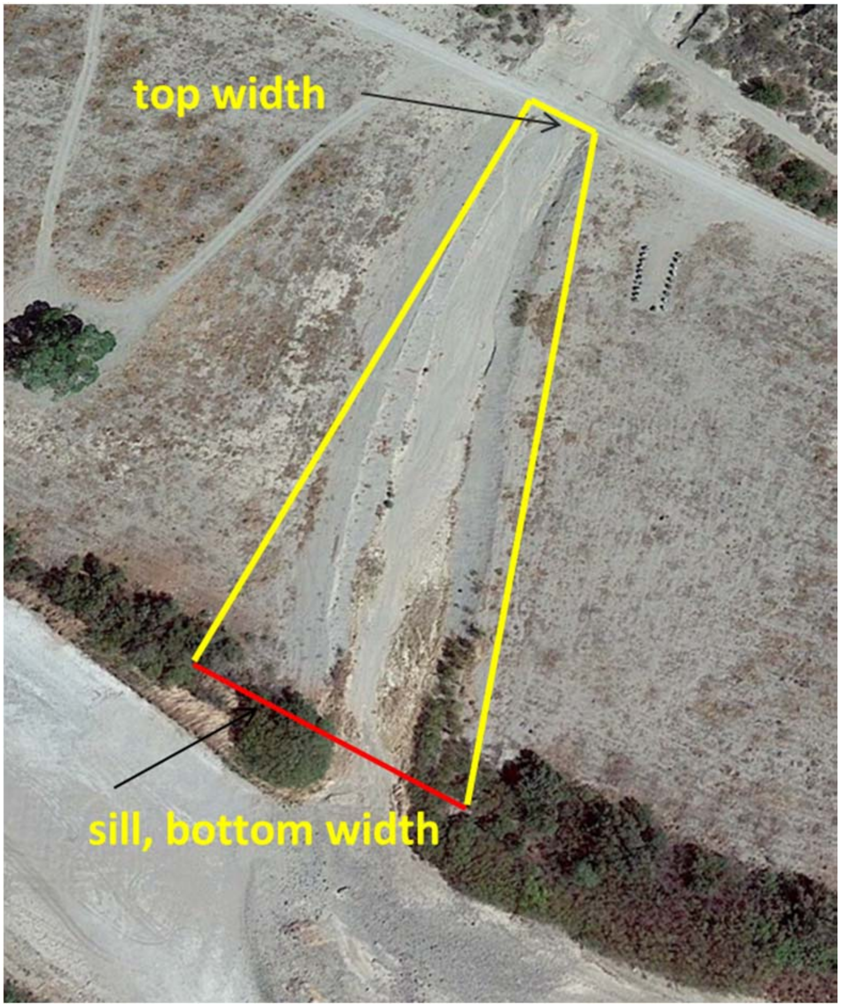


Figure A-18. Map View of Basin-Based Trap and Geometric Parameters (Sill, Top and Bottom Widths)

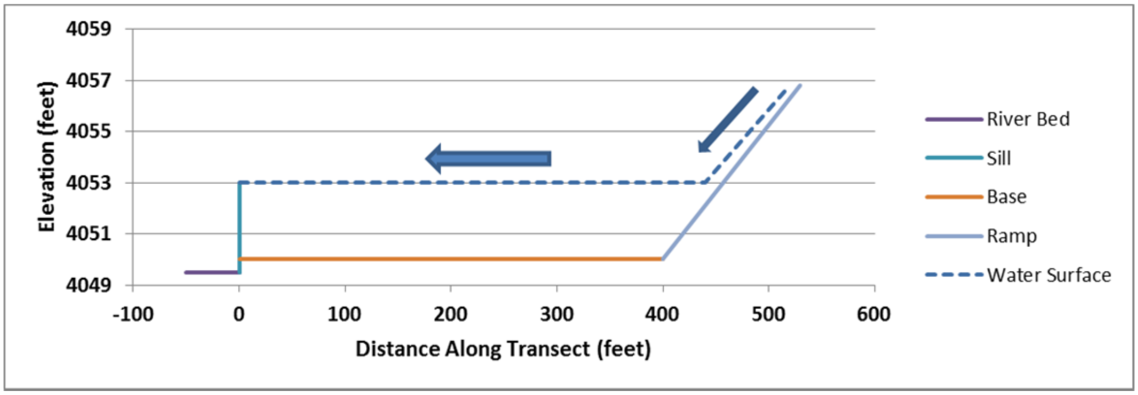


Figure A-19. Water Elevation and Flow Paths in Basin

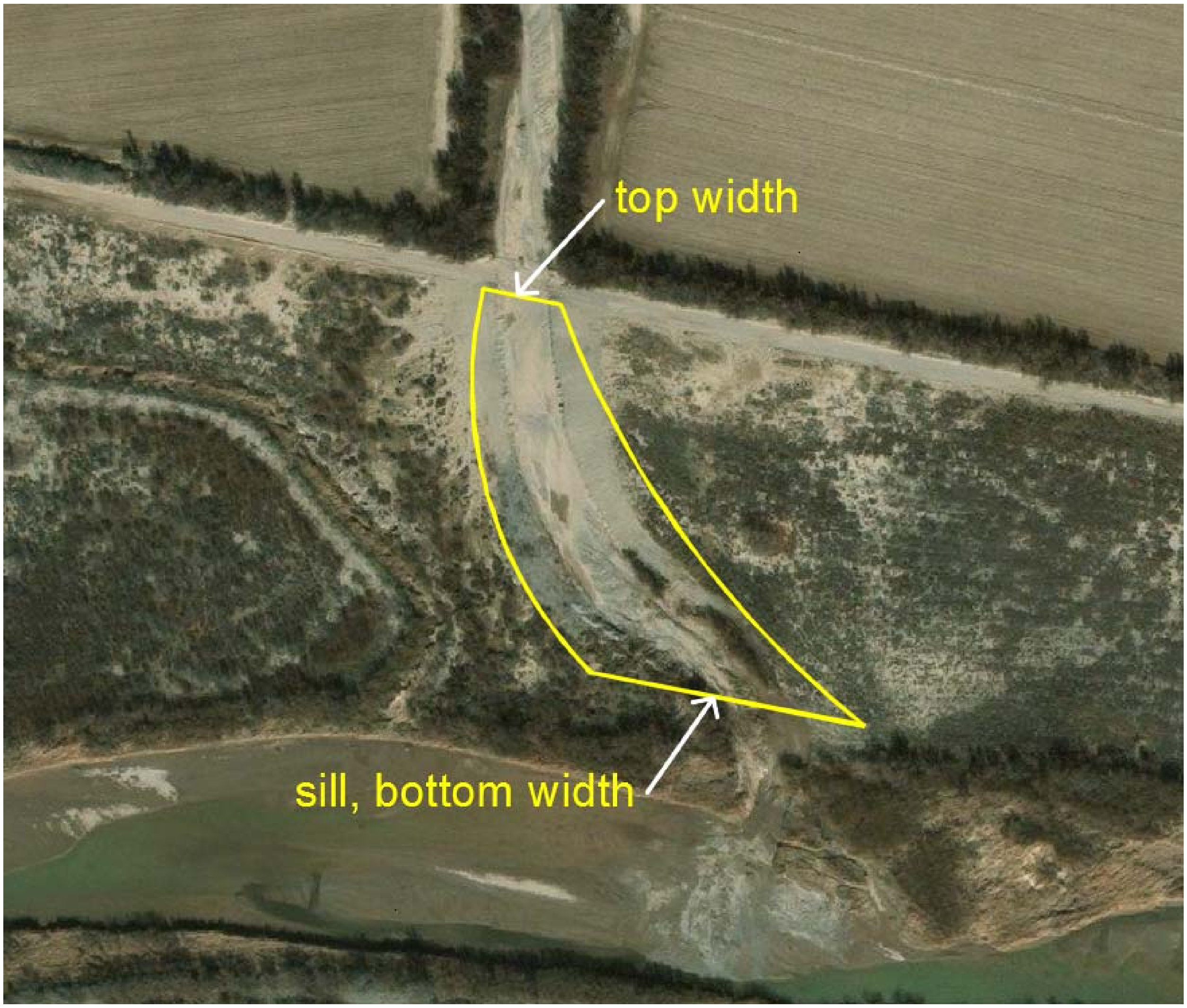


Figure A-20. Alignment for Thurman II Basin-Based Trap

Figure A-21:
THURMAN ARROYO I & II
100-yr Flood
Existing Conditions
Inundation Extent

Legend

Arroyo

Road

Inundation Depth (ft)

0 - 1

1 - 2

2 - 3

3 - 5

5 - 10

N

URS

0 250 500

Feet

Figure A-22: THURMAN ARROYO I & II 100-yr Flood Post-Project Conditions Inundation Extent

Legend

- Arroyo
- Road

Inundation Depth (ft)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 5
- 5 - 10

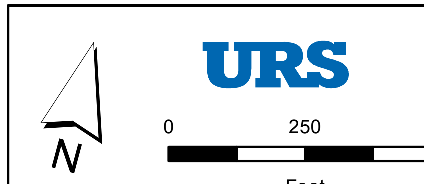


Figure A-23: THURMAN ARROYO I & II Comparison between 100-yr Flood Existing & Post-Project Conditions Inundation Extents

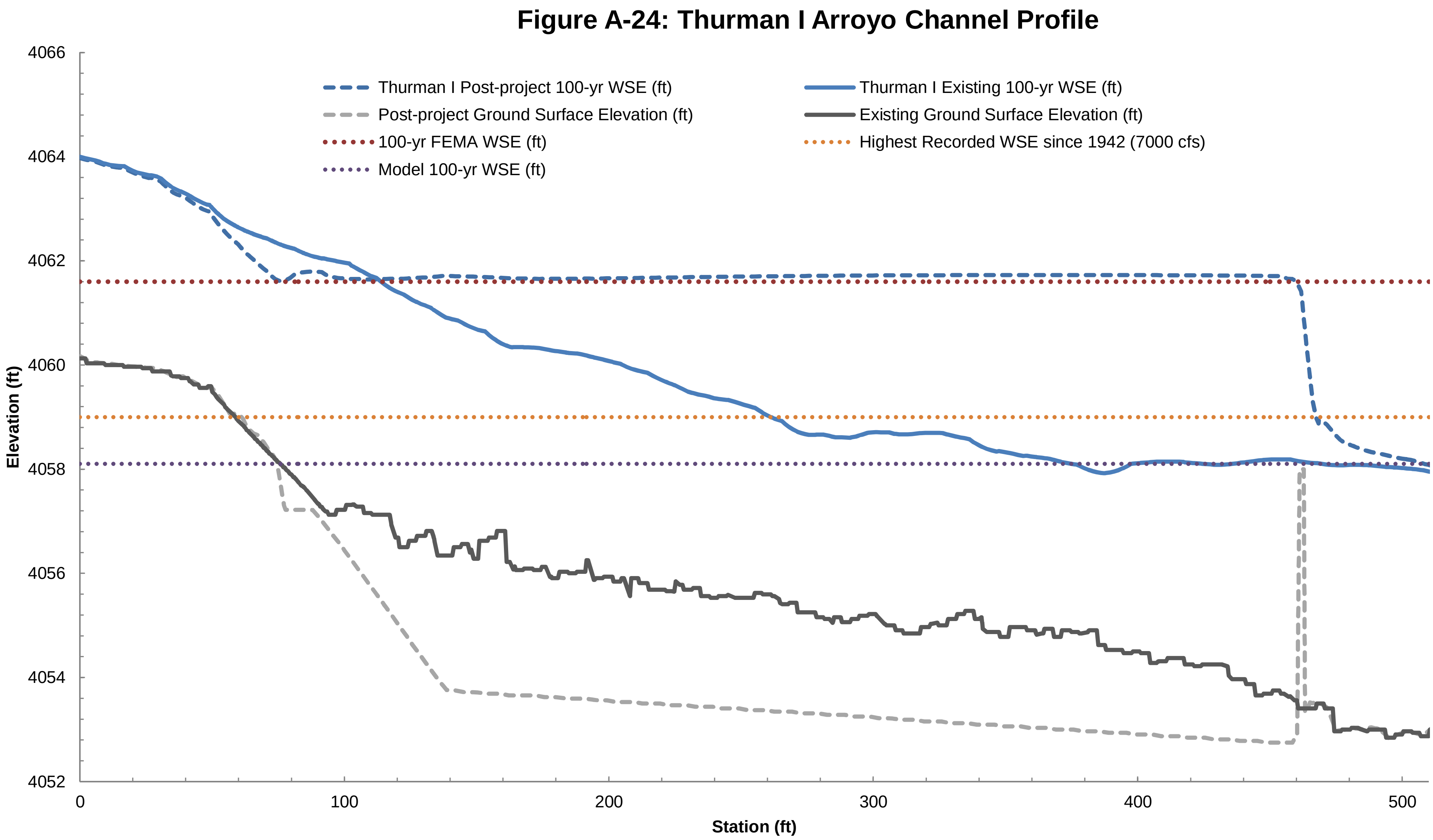
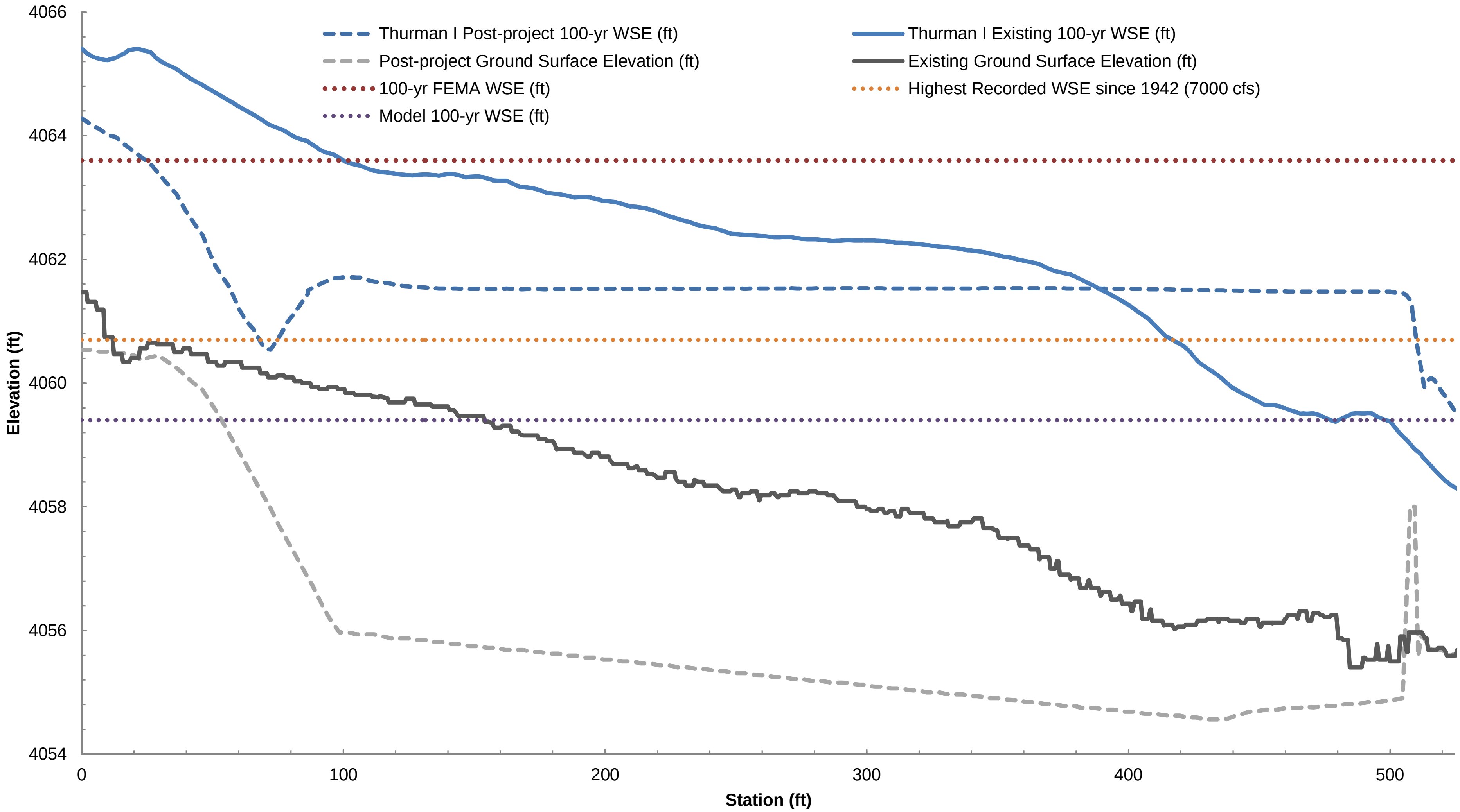


Figure A-24: Thurman I Arroyo Channel Profile
Thurman I Post-project 100-yr WSE (ft)
Thurman I Existing 100-yr WSE (ft)
Post-project Ground Surface Elevation (ft)
Existing Ground Surface Elevation (ft)
100-yr FEMA WSE (ft)
Highest Recorded WSE since 1942 (7000 cfs)
Model 100-yr WSE (ft)
Elevation (ft)
4066
4064
4062
4060
4058
4056
4054
4052
0
100
200
300
400
500
Station (ft)

Figure A-25: Thurman II Arroyo Channel Profile



Attachment A

Report of Site Visit
October 20, 2016

Prepared by:

URS
9400 Amberglen Blvd.
Austin, Texas 78729

REPORT ON OCTOBER 20, 2016 VISIT TO PROJECT SITES

Areas Visited

On October 20, 2016, the sediment basin design project team (both URS and IBWC staff) made a field visit to the Thurman I Arroyo and Thurman II Arroyo project area. Figure 1 shows the area of the site visit. The visit included a visit to the mouth of Thurman I (Area A) and upstream to the location of the Thurman I discharge under IH10 (Area C, D). The visit also included a visit to the mouth of Thurman II (Area E) and upstream to the location of the Thurman II discharge under IH10 (Area G, H). Selected photos taken during the site visit are provided with this report.

Thurman Arroyo I

Figure 1 shows Areas A (river), B (County Road to river), C (Interstate Highway culverts), D (Upstream of IH10).

Area A: The photos provided with this report show the area just downstream of the arroyo point into the river. The bar formed is armored by large cobbles and boulders. The river has an apparent ability, with the routine high flows during the irrigation season, to carry away most of the fines, leaving what is seen in the photos: large, cobble-sized material. This large material impinging on the river from the north bank of the river forces the river to the south, where it erodes the bank.

Area B: There is near complete lack of the very large cobbles and boulders present at the mouth of the arroyo in the river.

Area C: There is a constructed berm that trains the arroyo to flow into the IH10 culvert. The aggregate width of the culvert openings is much smaller than the width of the arroyo. The culverts are scoured clean, except for some minor deposition right at the entrance.

Area D: the area upstream of the IH has much larger cobbles and boulders than are visible in Area B, presumably deposited by the slower velocities in the backwater of the IH culverts. The Area between C and B has material to similar size as B.

Is sum, the physical condition appears to indicate the big material drops upstream of the IH, is largely absent between the IH and the river, and present in significant volume in the river, but only visible in the river primarily at the arroyo outlet and a short distance upstream.

The unexplained phenomenon is how cobbles dominate the arroyo bed in Areas D and A, but largely absent from Area C through Area B. This may be due to periodic maintenance within this reach of the arroyo.

Thurman Arroyo II

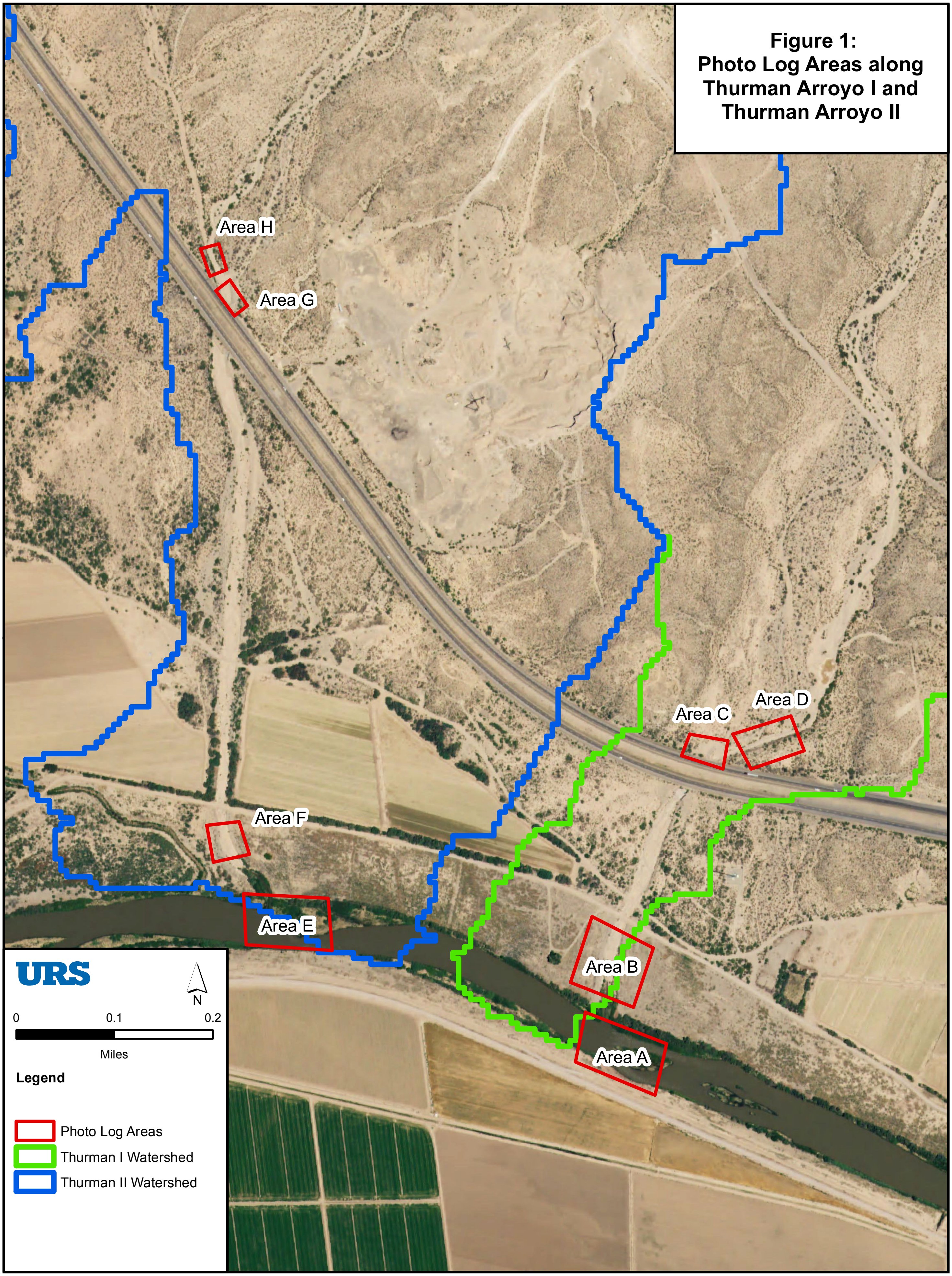
The conditions in the analogous reach of Thurman II were similar to those in Thurman I, with some changes noted below.

The size of rock armoring the channel at the arroyo outlet (Area E) is noticeably smaller than is the case at Area A in Thurman Arroyo I. The river at the Thurman Arroyo II outlet does not appear wider than at the Thurman Arroyo I outlet. Materia in the arroyo bed and banks in Area F is much smaller in particle than in Area E.

The IH10 crossing for Thurman II has a culvert system of much wider aggregate width of opening, roughly matching the width of the arroyo upstream. Deposition of large material occurs in the culverts, decreasing in D50 as you proceed down the culvert length.

Large material appears evenly present in the channel upstream of the IH10.

Figure 1: Photo Log Areas along Thurman Arroyo I and Thurman Arroyo II



URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2159.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area A	
View Direction: West	
Description: Confluence of Thurman Arroyo I with Rio Grande	

Filename: IMG_2160.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area A	
View Direction: NA	
Description: Confluence of Thurman Arroyo I with Rio Grande showing size of deposited sediment in bed	

URS	**PHOTOGRAPHIC RECORD**	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2162.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo I, Area A **View Direction:** NA **Description:** Confluence of Thurman Arroyo I with Rio Grande showing size of deposited sediment in bed	
Filename: IMG_2155.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo I, Area B **View Direction:** South **Description:** Upstream of confluence of Thurman Arroyo I with Rio Grande, downstream of levee	

URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2164.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area C	
View Direction: Southwest	
Description: Looking downstream through culvert under IH25 on Thurman Arroyo I	

Filename: IMG_2165.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area C	
View Direction: Southwest	
Description: Looking downstream through culvert under IH25 on Thurman Arroyo I, highlighting size of sediment deposition	

URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2167.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area D	
View Direction: North	
Description: In Thurman Arroyo I channel upstream of IH25 crossing	

Filename: IMG_2168.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo I, Area D	
View Direction: NA	
Description: In Thurman Arroyo I channel upstream of IH25 crossing, highlighting size of sediment deposition	

URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2169.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo I, Area D **View Direction:** NA **Description:** In Thurman Arroyo I channel upstream of IH25 crossing, highlighting size of sediment deposition	

Filename: IMG_2170.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo I, Area D **View Direction:** NA **Description:** In Thurman Arroyo I channel upstream of IH25 crossing, highlighting size of sediment deposition	

URS	**PHOTOGRAPHIC RECORD**

Project:	**Site:**	**Contract No.**
Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM	Thurman Arroyo I and Thurman Arroyo II	IBM15D0003 **Order No.** IBM16T0018

Filename:
IMG_2178.jpg

Date of Photo:
10/20/2016

Site Location:
Thurman Arroyo II, Area E

View Direction:
East

Description:
Confluence of Thurman Arroyo II with Rio Grande

Filename:
IMG_2179.jpg

Date of Photo:
10/20/2016

Site Location:
Thurman Arroyo II, Area E

View Direction:
NA

Description:
Confluence of Thurman Arroyo II with Rio Grande, highlighting size of sediment deposition

URS	**PHOTOGRAPHIC RECORD**	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2180.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo II, Area E	
View Direction: East	
Description: Confluence of Thurman Arroyo II with Rio Grande	

Filename: IMG_2171.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo II, Area F	
View Direction: NA	
Description: Upstream of confluence of Thurman Arroyo II with Rio Grande, downstream of levee	

URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2173.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area F **View Direction:** NA **Description:** Upstream of confluence of Thurman Arroyo II with Rio Grande, downstream of levee	

Filename: IMG_2164.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area G **View Direction:** South **Description:** Looking downstream through culvert under IH25 on Thurman Arroyo II	

URS	PHOTOGRAPHIC RECORD	
Project: **Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM**	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2165.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area G **View Direction:** South **Description:** Looking downstream through culvert under IH25 on Thurman Arroyo II, highlighting size of sediment deposition	

Filename: IMG_2191.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area H **View Direction:** South **Description:** Thurman Arroyo II channel upstream of IH25 crossing	

Filename: IMG_2194.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area H **View Direction:** Unknown **Description:** Thurman Arroyo II channel upstream of IH25 crossing	
Filename: IMG_2196.jpg **Date of Photo:** 10/20/2016 **Site Location:** Thurman Arroyo II, Area H **View Direction:** NA **Description:** Thurman Arroyo II channel upstream of IH25 crossing; highlighting size of sediment deposition	

Filename: IMG_2198.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo II, Area H	
View Direction: NA	
Description: Thurman Arroyo II channel upstream of IH25 crossing; highlighting size of sediment deposition	

Filename: IMG_2199.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo II, Area H	
View Direction: NA	
Description: Thurman Arroyo II channel upstream of IH25 crossing; highlighting size of sediment deposition	

URS	**PHOTOGRAPHIC RECORD**

Project: Design for the Construction of CMA within the Rio Grande Canalization Project, Doña Ana County, NM	**Site:** Thurman Arroyo I and Thurman Arroyo II	**Contract No.** IBM15D0003 **Order No.** IBM16T0018

Filename: IMG_2200.jpg	
Date of Photo: 10/20/2016	
Site Location: Thurman Arroyo II, Area H	
View Direction: NA	
Description: Thurman Arroyo II channel upstream of IH25 crossing; highlighting size of sediment deposition	

ATTACHMENT B

Calculation Method for Sediment Loads

Attachment B

Method for Calculating Sediment Transport

A historic time series of sediment loads was developed using the historic time series of flows described previously. The sediment load for each flow record in the historic time series (31 years at 5-minute intervals) was estimated. The method for estimating the sediment load is based on the Peter-Meyers formula. The required inputs for the formula were obtained from the sediment characteristics and arroyo geometry.

The Peter-Meyer formula in dimensional form is:

$$q = \frac{8\, d\, (\tau - \tau_c)\sqrt{gds'}}{1-\sigma} \qquad \text{Eq. 1}$$

where q is the sediment transport (m^3/s), τ is the bottom stress in Pascals (Pa) imparted on the sediment by the flow, τ_c is the critical stress for erosion (Pa), d is the sediment grain diameter (m), g is the acceleration due to gravity (m/s^2), σ is the sediment porosity and

$$s' = \frac{\rho_s - \rho_w}{\rho_w} \qquad \text{Eq. 2}$$

Where ρ_s and ρ_w are the sediment and water densities (Kg/m^3).

The values assumed for the sediment and water densities were 2600 kg/m^3 and 1000 kg/m^3, the acceleration due to gravity was 9.8 m/s^2, and the sediment porosity was assumed to be 0.3.

The grain diameter was determined from the grain size curve established in Section 3.0 of the H&H Report. The median grain diameter (d_{50}) was 25 millimeters (mm) and was the value used in the analysis for determining the critical stress for erosion. The critical stress for erosion was obtained from the shields curve, and when using a sediment d_{50} of 25 mm, yielded a critical stress of 15 (Pa).

The bottom stress was determined from the flow speed and height using the log-flow assumption:

$$\tau = \rho_w\, u_*^2 \qquad \text{Eq. 3}$$

where

$$u_* = \frac{\kappa\, u}{Log(^h/_{z_o})} \qquad \text{Eq. 4}$$

Where κ is an empirically determined coefficient with a value of 0.4, u is the depth-averaged velocity (m/s), h is the water depth (m), and z_o is the bottom roughness (m) defined for turbulent flows as:

$$z_o = \frac{d_{84}}{30} \qquad \text{Eq 5}$$

where d_{84} is the 84th percentile sediment grain size diameter. Using the sediment grain distribution from the H&H report, the value of d_{84} is approximately 80 mm (0.008 m).

The flow depth h and depth-averaged speed were obtained for a given flow using the normal flow approximation. The normal flow approximation assumes that the flow is steady and that the water surface is parallel to the land surface. The result of this approximation is that the gravitational force on the water is balanced by the bottom friction. Expressing this mathematically yields:

$$\frac{k \frac{Q}{wh}}{Log(\frac{h}{z_o} - 1)} = h^2\, g\, S \qquad \text{Eq 6}$$

Where Q is the flow rate (m^3/s), w is the flow width, and S is the arroyo slope. Note that Q/(wh) is the depth-averaged velocity. This equation can be solved for h once the slope and width are specified. The slope for the arroyos was estimated from the regional DEM. The average slope along the Thurman I arroyo is approximately 0.02 and for the Thurman II arroyo is 0.016.

The flow width was found to vary with the flow rate. Figure B-1 shows an image of the Thurman II arroyo where it appears that there are various flow channels of different widths. It is likely that the flow width varies with the flow rate, with larger widths occurring for the higher flows.

Figure B-1. Varying Channel Widths within the Arroyos

To represent this characteristic of the flows, the cross-section of the flow was assumed to be triangular, as shown in Figure B-2.

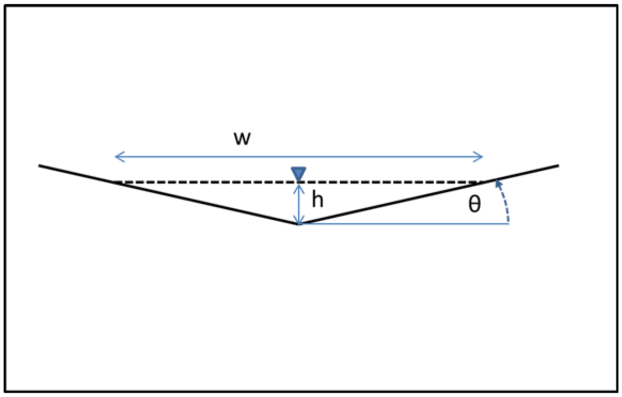


Figure B-2. Approach for Relating Flow Width w to Flow Height h

The width is related to the flow height h according to:

$$w = \frac{2h}{\tan\theta} \qquad \text{Eq. 7}$$

where the parameter θ controls the rate at which the width increases with flow depth. This parameter was determined via calibration to the annual sediment loads.

Equation 7 can be inserted into Equation 6 to form a non-linear equation for the flow depth h. It is solved using the bisection method. For each flow rate Q, the historic time series, the depth is determined from Equations 6 and 7, and then Equations 1 through 5 are used to calculate the sediment transport rate.

The transport simulation based on Equations 1 through 7 was calibrated to the annual average loads established and reported in the Tetra-Tech 2015 report. The representative channel width was varied using the parameter θ in a series of simulations until the computed annual average loads matched the reported values of 1.12 and 1.98 acre feet.

The annual load for each year in the 31 simulations is shown in Figure B-3 for Thurman I and Figure B-4 for Thurman II.

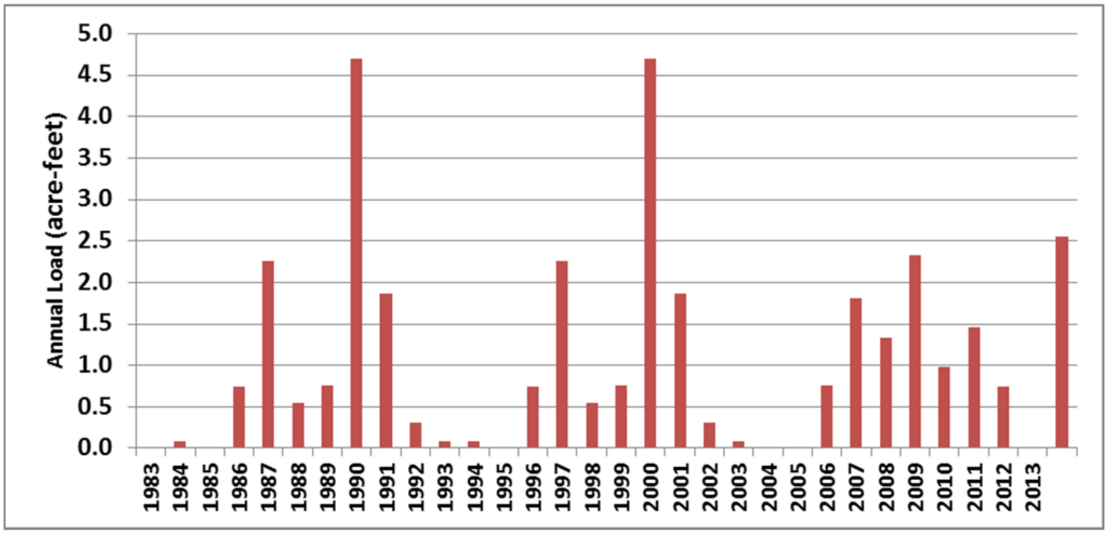


Figure B-3. Simulated Annual Sediment Loads from Thurman I Arroyo

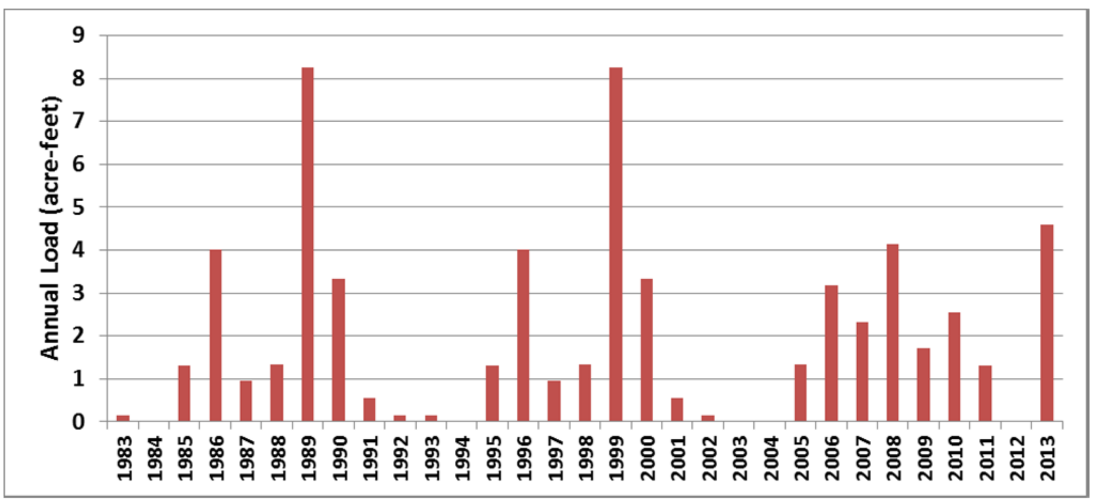


Figure B-4. Simulated Annual Sediment Loads from Thurman II Arroyo

Appendix B
Geotechnical Information

B1. River Level and Groundwater Data

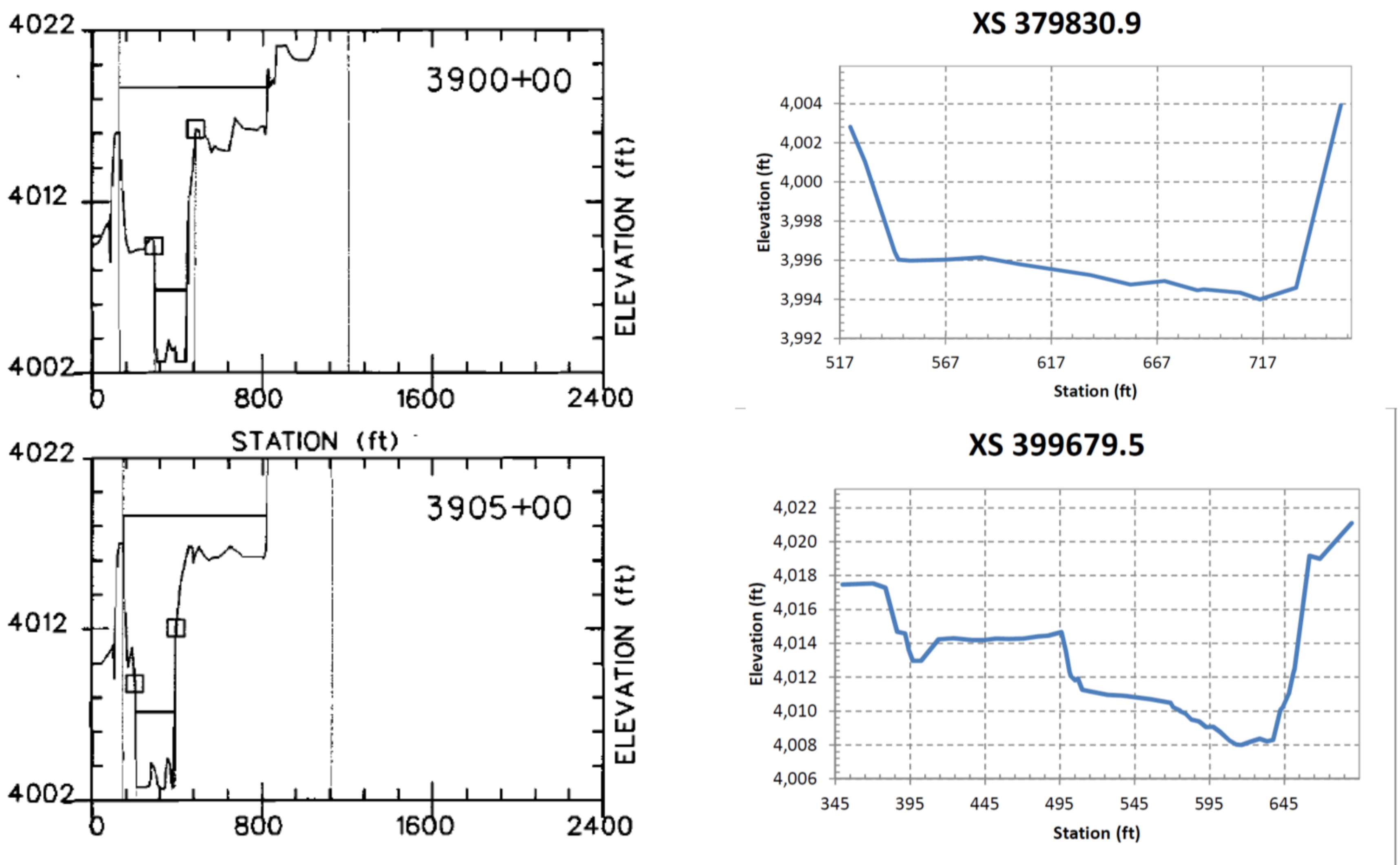
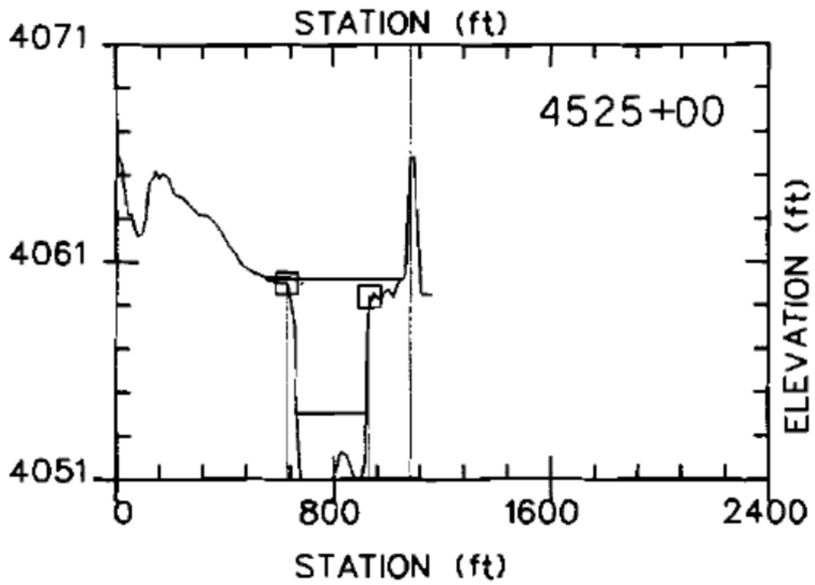


Figure B.1. Rio Grande Cross-Sections near Rio Grande AT - Hayners Bridge Near Rincon River Gage.
(after USACE 1996 and Tetra Tech 2015)



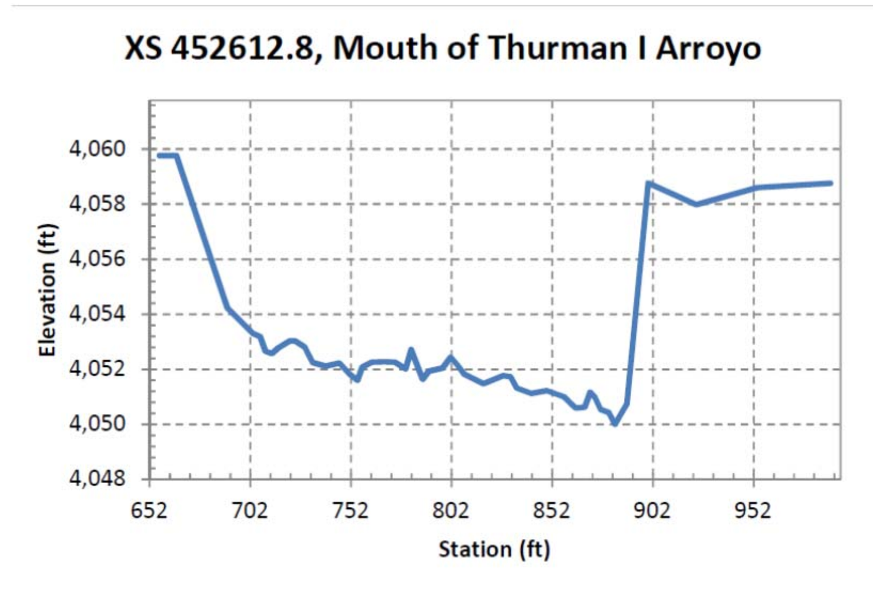


Figure B.2. Rio Grande Cross-Sections near Thurman I Arroyo
(after USACE 1996 and Tetra Tech 2015)

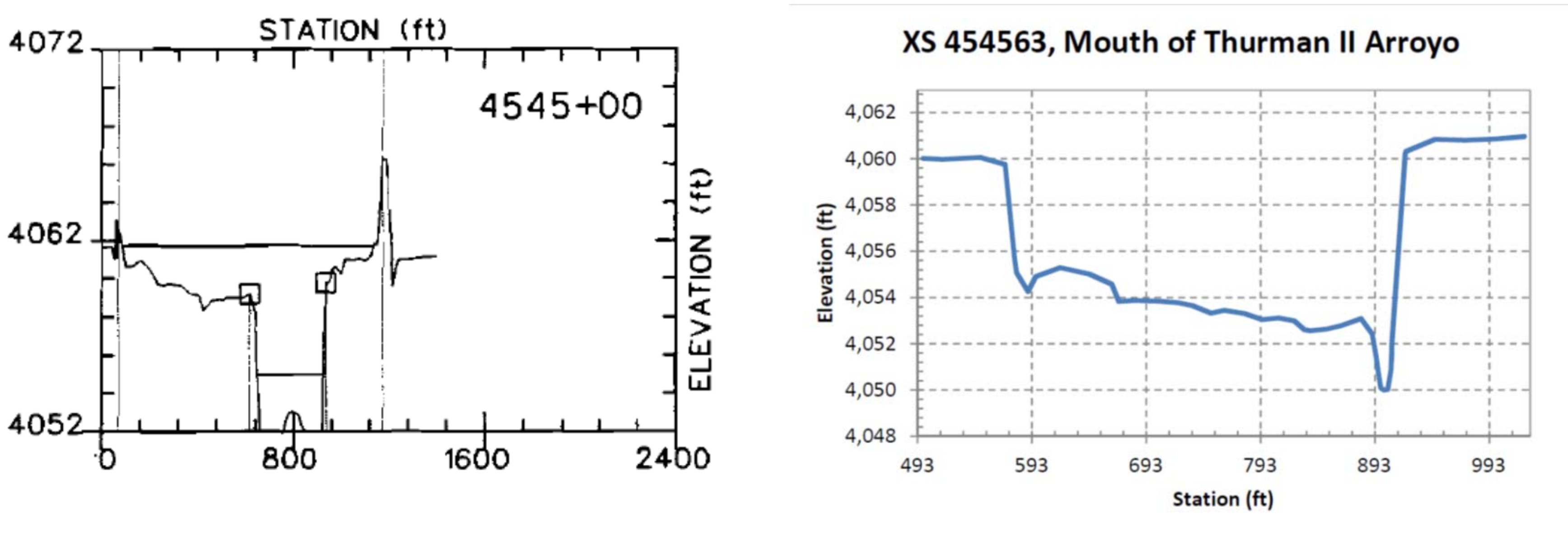


Figure B.3. Rio Grande Cross-Sections near Thurman II Arroyo
(after USACE 1996 and Tetra Tech 2015)

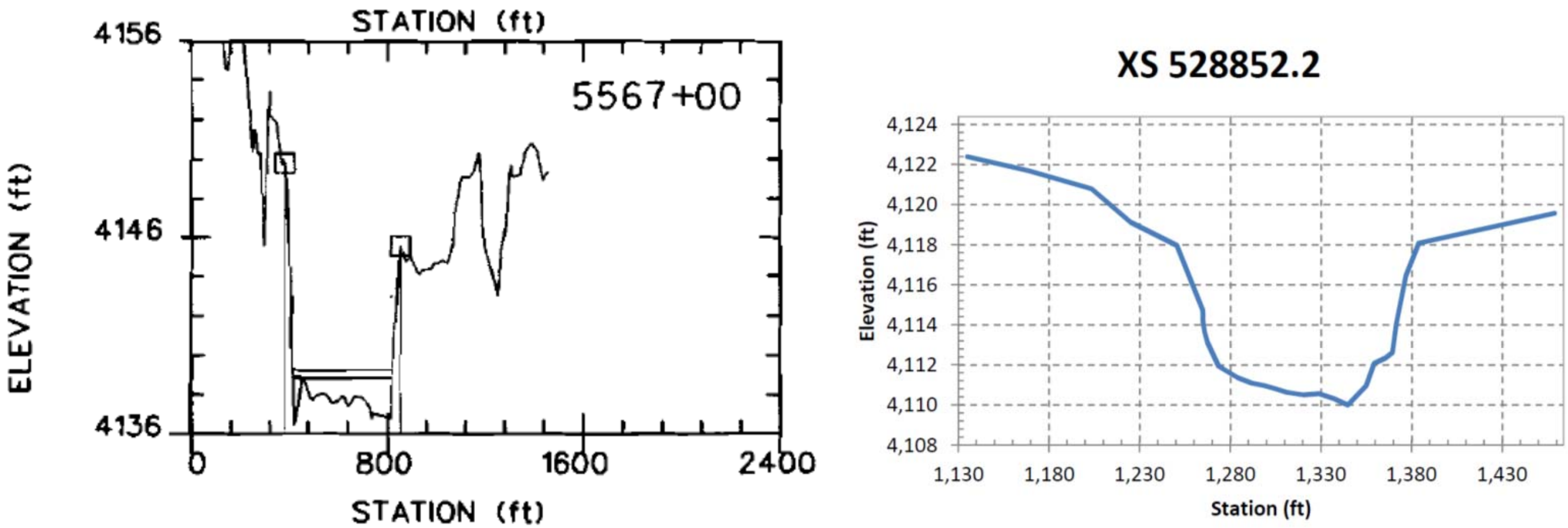


Figure B.5. Rio Grande Cross-Sections near Rio Grande AT Below Caballo Dam River Gage.
(after USACE 1996 and Tetra Tech 2015)

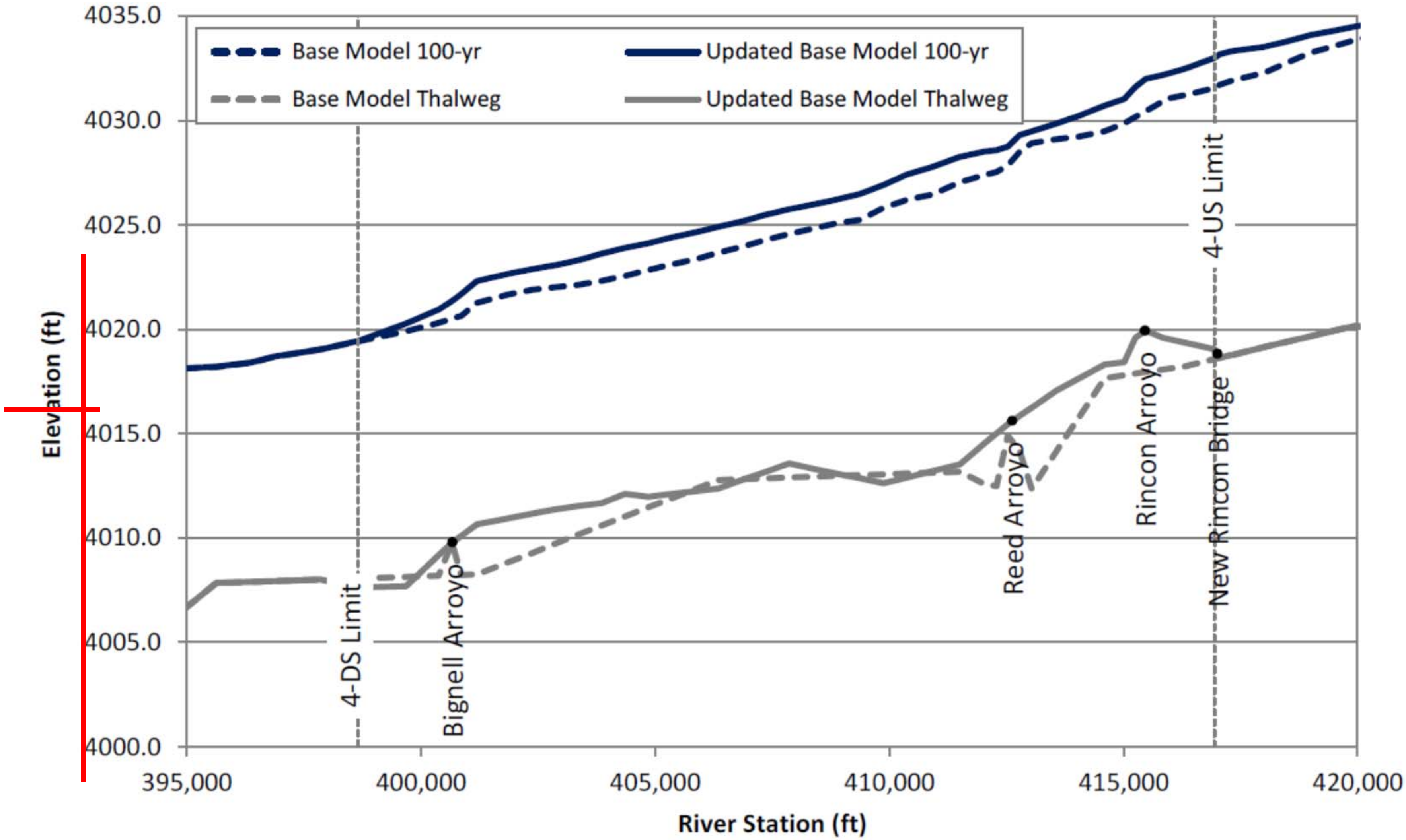


Figure B.4. Predicted water surface profiles from STA 3950+00 to 4200+00
(from Tetra Tech 2015)

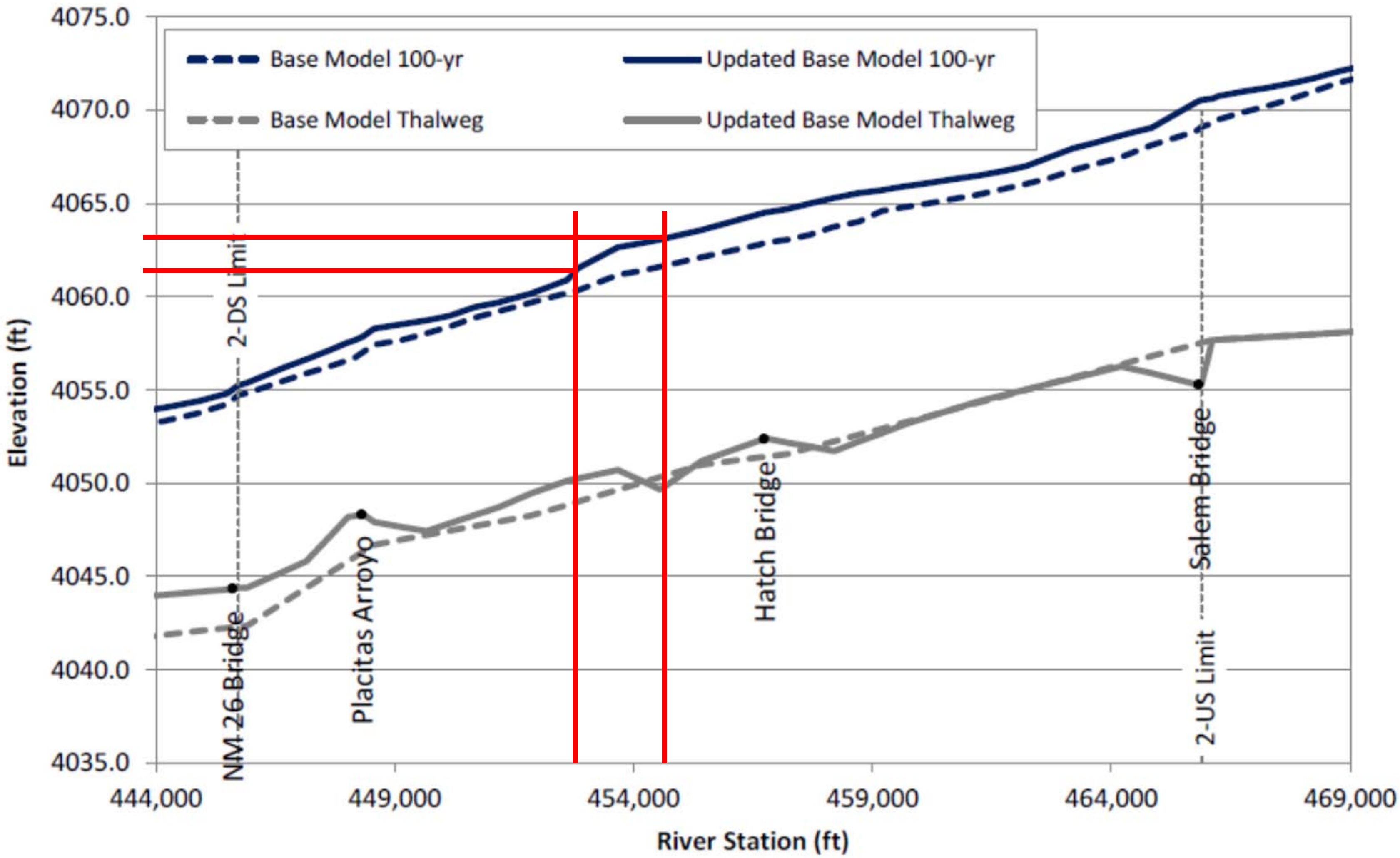


Figure B.6. Predicted water surface profiles from STA 4440+00 to 4690+00
(from Tetra Tech 2015)

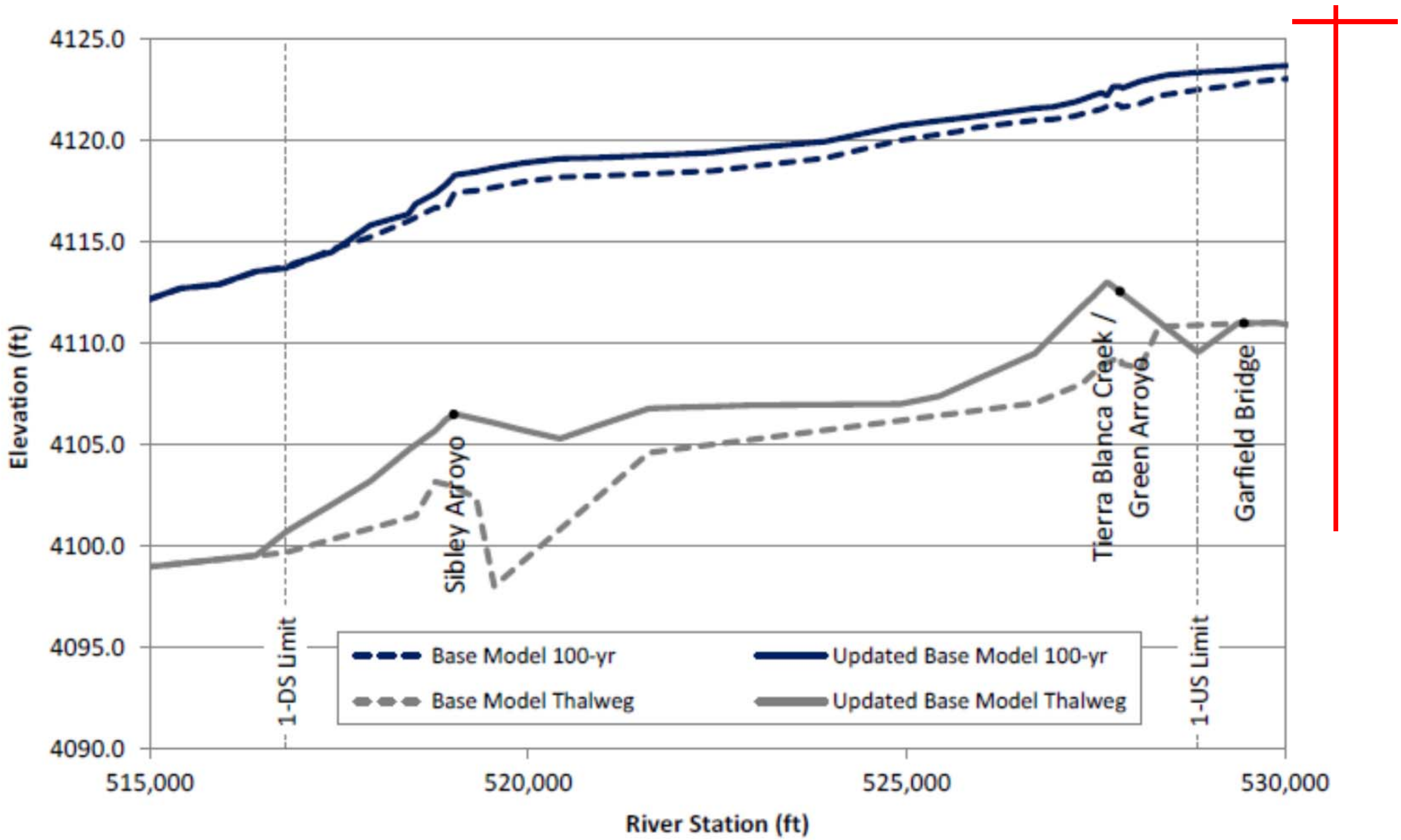


Figure B.7. Predicted water surface profiles from STA 5150+00 to 5300+00
(from Tetra Tech 2015)

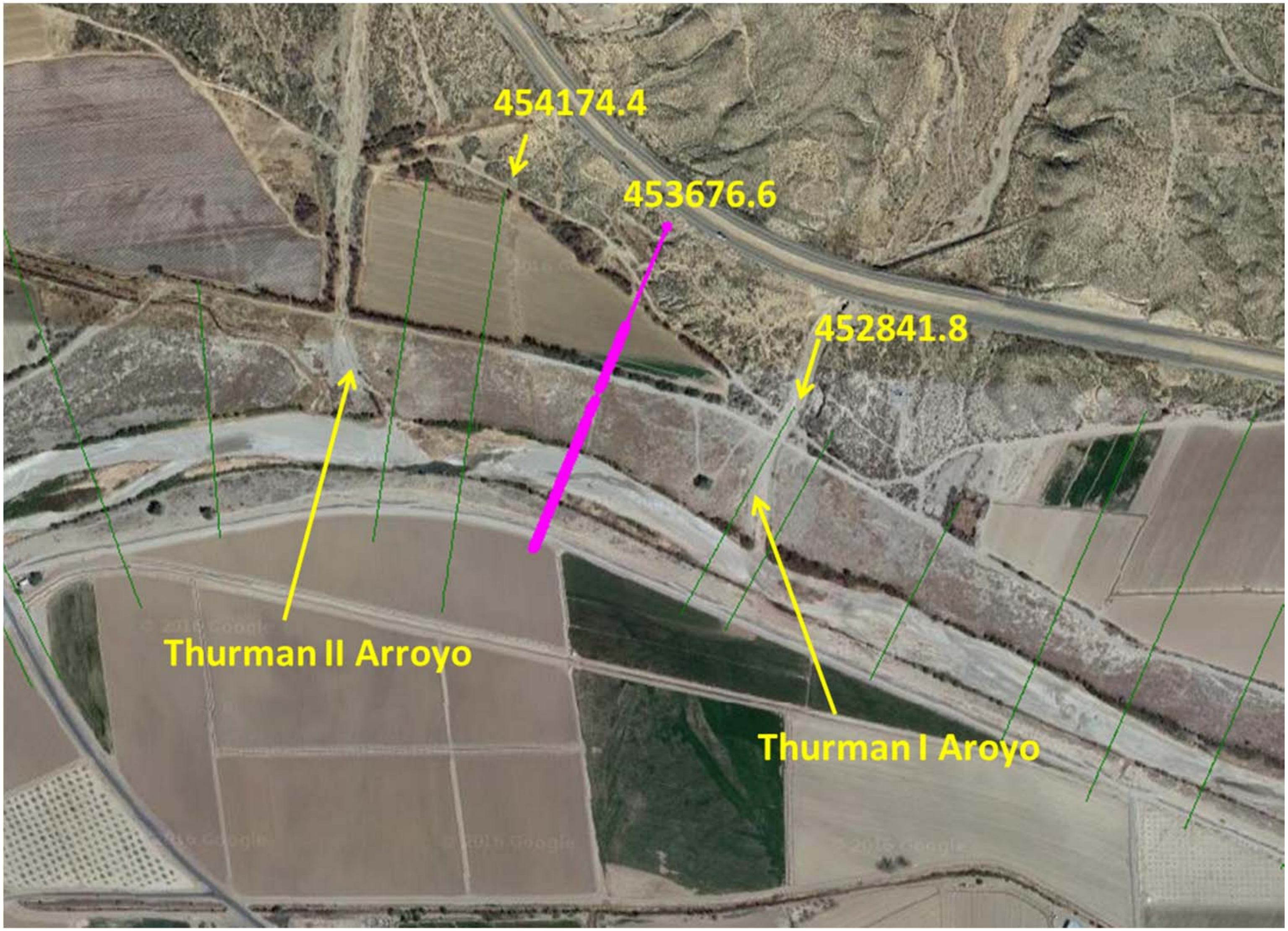


Figure B.8. Plan of Analysis Sections for Rio Grande Flows
(from URS 2016 preliminary H&H analyses)

Steady flow rate

Water elevation

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)
PL2	454174.4	500	500.00	4050.12	4053.93	4052.03	4053.95	0.000283	1.27	392.62
PL2	454174.4	1000	1000.00	4050.12	4054.84	4052.54	4054.90	0.000406	1.81	552.48
PL2	454174.4	1400	1400.00	4050.12	4055.40	4052.89	4055.47	0.000468	2.10	666.54
PL2	454174.4	2000	2000.00	4050.12	4056.11	4053.16	4056.20	0.000528	2.45	817.35
PL2	454174.4	2350	2350.00	4050.12	4056.48	4053.36	4056.58	0.000553	2.62	897.71
PL2	454174.4	3000	3000.00	4050.12	4057.10	4053.70	4057.23	0.000587	2.89	1038.60
PL2	454174.4	3500	3500.00	4050.12	4057.55	4053.99	4057.70	0.000607	3.06	1142.98
PL2	454174.4	4000	4000.00	4050.12	4057.96	4054.22	4058.12	0.000627	3.22	1242.20
PL2	454174.4	4500	4500.00	4050.12	4058.34	4054.42	4058.52	0.000643	3.35	1342.19
PL2	454174.4	5000	5000.00	4050.12	4058.70	4054.66	4058.89	0.000657	3.47	1443.79
PL2	454174.4	5500	5500.00	4050.12	4059.04	4054.92	4059.24	0.000671	3.59	1548.75
PL2	454174.4	6000	6000.00	4050.12	4059.35	4055.15	4059.56	0.000685	3.70	1687.82
PL2	454174.4	100-yr Routed Q	15150.00	4050.12	4062.91	4057.84	4063.14	0.000601	4.35	5546.33
PL2	453676.6	500	500.00	4050.71	4053.80	4052.18	4053.82	0.000265	1.14	440.41
PL2	453676.6	1000	1000.00	4050.71	4054.67	4052.54	4054.71	0.000342	1.59	627.54
PL2	453676.6	1400	1400.00	4050.71	4055.20	4052.78	4055.25	0.000391	1.85	755.70
PL2	453676.6	2000	2000.00	4050.71	4055.89	4053.10	4055.96	0.000438	2.14	932.58
PL2	453676.6	2350	2350.00	4050.71	4056.24	4053.23	4056.32	0.000458	2.29	1027.01
PL2	453676.6	3000	3000.00	4050.71	4056.86	4053.54	4056.96	0.000483	2.52	1192.16
PL2	453676.6	3500	3500.00	4050.71	4057.30	4053.74	4057.41	0.000496	2.67	1312.67
PL2	453676.6	4000	4000.00	4050.71	4057.71	4053.95	4057.83	0.000504	2.81	1430.90
PL2	453676.6	4500	4500.00	4050.71	4058.09	4054.15	4058.22	0.000509	2.94	1564.09
PL2	453676.6	5000	5000.00	4050.71	4058.45	4054.32	4058.59	0.000509	3.04	1764.50
PL2	453676.6	5500	5500.00	4050.71	4058.79	4054.53	4058.94	0.000501	3.11	1986.29
PL2	453676.6	6000	6000.00	4050.71	4059.10	4054.72	4059.25	0.000495	3.18	2205.34
PL2	453676.6	100-yr Routed Q	15150.00	4050.71	4062.71	4057.07	4062.87	0.000412	3.76	6169.03
PL2	452841.8	500	500.00	4050.25	4053.38	4052.03	4053.43	0.000962	1.79	279.61
PL2	452841.8	1000	1000.00	4050.25	4054.14	4052.60	4054.23	0.001118	2.34	427.05
PL2	452841.8	1400	1400.00	4050.25	4054.62	4053.03	4054.73	0.001144	2.65	527.33
PL2	452841.8	2000	2000.00	4050.25	4055.25	4053.40	4055.39	0.001127	2.99	668.91
PL2	452841.8	2350	2350.00	4050.25	4055.60	4053.61	4055.75	0.001107	3.15	746.34
PL2	452841.8	3000	3000.00	4050.25	4056.19	4053.92	4056.37	0.001079	3.40	883.25
PL2	452841.8	3500	3500.00	4050.25	4056.62	4054.15	4056.82	0.001067	3.55	984.89
PL2	452841.8	4000	4000.00	4050.25	4057.01	4054.37	4057.23	0.001067	3.71	1079.19
PL2	452841.8	4500	4500.00	4050.25	4057.39	4054.56	4057.62	0.001057	3.84	1172.03
PL2	452841.8	5000	5000.00	4050.25	4057.75	4054.75	4057.99	0.001042	3.96	1261.26
PL2	452841.8	5500	5500.00	4050.25	4058.08	4054.92	4058.34	0.001034	4.09	1345.83
PL2	452841.8	6000	6000.00	4050.25	4058.39	4055.08	4058.66	0.001038	4.22	1422.80
PL2	452841.8	100-yr Routed Q	15150.00	4050.25	4061.60	4057.56	4062.23	0.001446	6.48	2516.31

upstream

center

downstream

Table B.1. Rio Grande Water Surface Elevation for Various Flow Rates (corresponds to Figure B.8)
(from URS 2016 preliminary H&H analyses)

B2. Borings from Previous Geotechnical Studies

(Raba Kistner, 2008)

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SHARLOTTE LYNNE TEAGUE, P.E.
TEXAS REGISTRATION 62160
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ENGINEERING ◆ MANAGEMENT ◆ TECHNOLOGY

TECHNICAL MEMORANDUM FOR

Geotechnical Exploration of Levee System Within the Lower Rio Grande Canalization Project

BOREHOLE LOCATION MAP
HATCH-TONUCO LEVEE

UNITED STATES SECTION, INTERNATIONAL BOUNDARY & WATER COMMISSION

CONTRACT NO. IBM05D0001

TASK ORDER NO. IBM08T0009

Hatch–Tonuco Levee
Sheet 4 of 6
Bore Hole 38–77 East Side

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TECHNICAL MEMORANDUM FOR

Geotechnical Exploration of Levee System Within the Lower Rio Grande Canalization Project

BOREHOLE LOCATION MAP
HATCH-TONUCO LEVEE

UNITED STATES SECTION, INTERNATIONAL BOUNDARY & WATER COMMISSION

TASK ORDER NO. IBM08T0009

CONTRACT NO. IBM05D0001

Hatch–Tonuco Levee
Sheet 4a of 6
Bore Hole 162–200 West Side

Hole No. HT-74

DRILLING LOG	DIVISION: IBWC	INSTALLATION: Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-3/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1355526.617; E 613840.081	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-74	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Manuel Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4051.606
7. THICKNESS OF OVERBURDEN: N/A	16. DATE HOLE: STARTED 3/20/2008; COMPLETED 3/20/2008
8. DEPTH DRILLED INTO ROCK: N/A	17. ELEVATION TOP OF HOLE: 4065.106
9. TOTAL DEPTH OF HOLE: 20	18. TOTAL CORE RECOVERY FOR BORING: %
	19. GEOLOGIST: Ben Natera, E.I.T.

ELEVATION (a)	DEPTH (b)	LEGEND (c)	CLASSIFICATION OF MATERIALS (Description) (d)	BLOW COUNT (e)	% RECOVERY (f)	SAMPLE INTERVAL (g)	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* (h)
+4065.1	0.0		**SILTY SAND (SM)** fine, poorly graded, dense to medium dense, dry, brown, non-plastic	35	67	0.0 1.5	
				26	64	2.5 4.0	
			- loose 5 to 6.5 feet	9	81	5.0 6.5	
+4058.1	7.0		**SAND (SP)** fine, poorly graded, medium dense, dry to wet, brown, non-plastic	14	0	7.5 9.0	
				10	94	10.0 11.5	
							Free water oberserved at 13.5 ft. Drilling mud introduced at 13.5 ft.
				14	44	15.0 16.5	
+4047.1	18.0		**SANDY GRAVEL (GP)** poorly graded, very dense, wet, dark brown, non-plastic	46	106	18.5 20.0	
+4045.1	20.0						

NOTES:
1. Free water was observed at 13.5 ft.
2. Mud-rotary methods used below 13.5 ft due to caving soils.
3. Cement-bentonite grout used as backfill.

Hole No. HT-75

DRILLING LOG	DIVISION IBWC	INSTALLATION Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-3/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1354611.367; E 614209.979	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-75	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Manuel Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4053.719
7. THICKNESS OF OVERBURDEN: N/A	16. DATE HOLE: STARTED 3/20/2008; COMPLETED 3/20/2008
8. DEPTH DRILLED INTO ROCK: N/A	17. ELEVATION TOP OF HOLE: 4059.719
9. TOTAL DEPTH OF HOLE: 20	18. TOTAL CORE RECOVERY FOR BORING: %
	19. GEOLOGIST: Ben Natera, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+4059.7	0.0		**SILTY SAND (SM)** fine, poorly graded, medium dense, brown, non-plastic, with clay seams	20	58	0.0 1.5	
+4057.7	2.0		**SAND with SILT (SP-SM)** fine, poorly graded, medium dense to loose, dry to wet, dark brown, non-plastic	11	86	2.5 4.0	
				8	86	5.0 6.5	Free water oberserved at 6 ft. Drilling mud introduced at 6 ft.
			- dark brown below 7.5 feet - very loose 7.5 to 9 feet	2	72	7.5 9.0	
				8	94	10.0 11.5	
			- with clay pockets 15 to 16.5 feet	8	56	15.0 16.5	
+4039.7	20.0			9	31	18.5 20.0	

NOTES:
1. Free water was observed at 6 ft.
2. Mud-rotary methods used below 6 ft due to caving soils.
3. Cement-bentonite grout used as backfill.

Hole No. HT-76

DRILLING LOG	DIVISION: IBWC	INSTALLATION: Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-3/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1353637.977; E 614437.732	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-76	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Manuel Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4053.708
	16. DATE HOLE: STARTED 3/20/2008; COMPLETED 3/20/2008
7. THICKNESS OF OVERBURDEN: N/A	17. ELEVATION TOP OF HOLE: 4060.208
8. DEPTH DRILLED INTO ROCK: N/A	18. TOTAL CORE RECOVERY FOR BORING: %
9. TOTAL DEPTH OF HOLE: 20	19. GEOLOGIST: Ben Natera, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+4060.2	0.0		**SILTY SAND (SM)** fine, poorly graded, medium dense, dry, brown, non-plastic	10	61	0.0 1.5	
+4058.2	2.0		**SAND with SILT (SP-SM)** fine, poorly graded, medium dense to loose, dry to wet, brown, non-plastic, with gravel	11	83	2.5 4.0	
				10	81	5.0 6.5	
			- dark brown below 7.5 feet	7	83	7.5 9.0	Free water oberserved at 6.5 ft. Drilling mud introduced at 6.5 ft.
+4050.2	10.0		**SAND (SP)** fine, poorly graded, loose, wet, dark brown, non-plastic, with trace gravel	6	83	10.0 11.5	
				5	36	15.0 16.5	
				9	78	18.5 20.0	
+4040.2	20.0						

NOTES:
1. Free water was observed at 6.5 ft.
2. Mud-rotary methods used below 6.5 ft due to caving soils.
3. Cement-bentonite grout used as backfill.

Hole No. HT-77

DRILLING LOG	DIVISION IBWC	INSTALLATION Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-3/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1352659.824; E 614636.812	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-77	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Manuel Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4054.723
	16. DATE HOLE: STARTED 3/21/2008; COMPLETED 3/21/2008
7. THICKNESS OF OVERBURDEN: N/A	17. ELEVATION TOP OF HOLE: 4061.223
8. DEPTH DRILLED INTO ROCK: N/A	18. TOTAL CORE RECOVERY FOR BORING: %
9. TOTAL DEPTH OF HOLE: 20	19. GEOLOGIST: Ben Natera, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* h
+4061.2	0.0		**SILTY SAND (SM)** firm, poorly graded, medium dense to loose, moist to wet, brown, non-plastic	17	64	0.0 1.5	
				6	86	2.5 4.0	
				8	86	5.0 6.5	
+4054.7	6.5		**SAND with SILT (SP-SM)** fine, poorly graded, loose, wet, brown, non-plastic				Free water oberserved at 6.5 ft. Drilling mud introduced at 6.5 ft.
				5	83	7.5 9.0	
				10	78	10.0 11.5	
+4048.2	13.0		**SAND (SP)** fine, poorly graded, loose, wet, gray, non-plastic				
				4	94	15.0 16.5	
				9	72	18.5 20.0	
+4041.2	20.0						

NOTES:
1. Free water was observed at 6.5 ft.
2. Mud-rotary methods used below 6.5 ft due to caving soils.
3. Cement-bentonite grout used as backfill.

Hole No. HT-162

DRILLING LOG	DIVISION: IBWC	INSTALLATION: Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-1/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1353572.983; E 613579.124	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-162	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Derek Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4054.439
7. THICKNESS OF OVERBURDEN: N/A	16. DATE HOLE: STARTED 3/25/2008; COMPLETED 3/25/2008
8. DEPTH DRILLED INTO ROCK: N/A	17. ELEVATION TOP OF HOLE: 4067.439
9. TOTAL DEPTH OF HOLE: 20	18. TOTAL CORE RECOVERY FOR BORING: %
	19. GEOLOGIST: Isaac Puentes, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+4067.4	0.0		**FILL: CLAYEY SAND (SC)** fine, poorly graded, dry, medium dense, brown, moderately plastic, with trace gravel	14	61	0.0 1.5	
+4065.4	2.0		**FILL: SAND with SILT (SP-SM)** fine, poorly graded, medium dense to very loose, dry, brown, non-plastic, with trace gravel	13	83	2.5 4.0	
				3	89	5.0 6.5	
+4060.4	7.0		**SAND with SILT (SP-SM)** fine, poorly graded, loose, dry, brown, non-plastic	6	89	7.5 9.0	
				6	89	10.0 11.5	
				5	89	15.0 16.5	
+4047.4	20.0			4		18.5 20.0	

NOTES:
1. Free water was observed at 13 ft.
2. Cement-bentonite grout used as backfill.

Hole No. HT-163

DRILLING LOG	DIVISION: IBWC	INSTALLATION: Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-1/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1354522.869; E 613315.547	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-163	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Derek Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4053.228
7. THICKNESS OF OVERBURDEN: N/A	16. DATE HOLE: STARTED 3/25/2008; COMPLETED 3/25/2008
8. DEPTH DRILLED INTO ROCK: N/A	17. ELEVATION TOP OF HOLE: 4066.228
9. TOTAL DEPTH OF HOLE: 20	18. TOTAL CORE RECOVERY FOR BORING: %
	19. GEOLOGIST: Isaac Puentes, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+4066.2	0.0		**FILL: CLAYEY SAND (SC)** fine, poorly graded, medium dense, dry, brown, low plasticity, with trace gravel	12	72	0.0 1.5	
+4064.2	2.0		**FILL: SILTY SAND (SM)** fine, poorly graded, loose, dry, brown, with trace gravel, non-plastic	5	72	2.5 4.0	
				4	89	5.0 6.5	
+4059.2	7.0		**SAND (SP)** fine, poorly graded, medium dense to loose, dry, brown, non-plastic	11	72	7.5 9.0	
				8	89	10.0 11.5	
							Free water observed at 13 ft. Drilling mud introduced at 13 ft.
+4051.2	15.0		**SAND (SP)** fine, poorly graded, very loose, wet, brown, non-plastic	3	89	15.0 16.5	
				2	83	18.5 20.0	
+4046.2	20.0						

NOTES:
1. Free water was observed at 13 ft.
2. Cement-bentonite grout used as backfill.

Hole No. HT-164

DRILLING LOG	DIVISION IBWC	INSTALLATION Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT: The Rio Grande Canalization Project	10. SIZE AND TYPE OF BIT: 3-1/4 Inch ID Hollow Stem Auger
2. LOCATION (Coordinates or Station): N 1355415.615; E 612927.012	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: MSL
3. DRILLING AGENCY: Raba-Kistner Consultants, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL: CME-75
4. HOLE NO. *(As shown on drawing title and file number)*: HT-164	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED N/A
5. NAME OF DRILLER: Derek Duenez	14. TOTAL NUMBER CORE BOXES: N/A
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER: 4051.862
7. THICKNESS OF OVERBURDEN: N/A	16. DATE HOLE: STARTED 3/25/2008; COMPLETED 3/25/2008
8. DEPTH DRILLED INTO ROCK: N/A	17. ELEVATION TOP OF HOLE: 4064.862
9. TOTAL DEPTH OF HOLE: 20	18. TOTAL CORE RECOVERY FOR BORING: %
	19. GEOLOGIST: Isaac Puentes, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+4064.9	0.0		**FILL: CLAYEY SAND (SC)** fine, poorly graded, medium dense, dry, brown, low plasticity	16	83	0.0 1.5	
+4062.9	2.0		**FILL: SILTY SAND (SM)** fine, poorly graded, loose to very loose, dry, brown, non-plastic	5	83	2.5 4.0	
				2	89	5.0 6.5	
+4058.4	6.5		**SAND with SILT (SP-SM)** fine, poorly graded, medium dense to loose, dry, brown, non-plastic	11	89	7.5 9.0	
				8	89	10.0 11.5	
							Free water observed at 13 ft. Drilling mud introduced at 13 ft.
+4049.9	15.0		**SAND (SP)** fine, poorly graded, very loose to loose, wet, brown, non-plastic	3	83	15.0 16.5	
				4	89	18.5 20.0	
+4044.9	20.0						

NOTES:
1. Free water was observed at 13 ft.
2. Cement-bentonite grout used as backfill.

B3. Laboratory Tests from Previous Geotechnical Studies

(Raba Kistner, 2008)

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: The Rio Grande Canalization Project
Hatch/Tonuco Segment

FILE NAME: AEA08-020-02.GPJ 4/26/2008

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
HT-73	7.5 to 9.0	12	4			NP					
HT-73	10.0 to 11.5	7	7			NP	SP-SM		5		
HT-73	15.0 to 16.5	10									
HT-73	18.5 to 20.0	7	18			NP	SP		2		
HT-74	0.0 to 1.5	35	4			NP					
HT-74	2.5 to 4.0	26	4			NP	SM		18		
HT-74	5.0 to 6.5	9				NP					
HT-74	7.5 to 9.0	14									
HT-74	10.0 to 11.5	10	4			NP					
HT-74	15.0 to 16.5	14	12			NP	SP		3		
HT-74	18.5 to 20.0	46	9			NP			1		
HT-75	0.0 to 1.5	20									
HT-75	2.5 to 4.0	11	5			NP	SP		5		
HT-75	5.0 to 6.5	8	3			NP					
HT-75	7.5 to 9.0	2	21			NP	SP-SM		7		
HT-75	7.6		9						10		
HT-75	10.0 to 11.5	8	20			NP					
HT-75	15.0 to 16.5	8	28			NP			8		
HT-75	18.5 to 20.0	9	18			NP					
HT-76	0.0 to 1.5	10	6			NP	SM		34		
HT-76	2.5 to 4.0	11	3			NP					
HT-76	5.0 to 6.5	10	3			NP	SP-SM		5		
HT-76	7.5 to 9.0	7	19			NP	SP-SM		5		
HT-76	10.0 to 11.5	6	16			NP	SP		1		
HT-76	15.0 to 16.5	5	11			NP			4		
HT-76	18.5 to 20.0	9				NP					
HT-77	0.0 to 1.5	17									
HT-77	2.5 to 4.0	6	8			NP	SM		43		
HT-77	5.0 to 6.5	8	26			NP	SM		17		
HT-77	7.5 to 9.0	5	25			NP	SP-SM		9		
HT-77	10.0 to 11.5	10	20						5		
HT-77	15.0 to 16.5	4	23			NP	SP		4		
HT-77	18.5 to 20.0	9	17								
HT-78	0.0 to 1.5	11	6			NP	SM		36		
HT-78	2.5 to 4.0	11	3						5		
HT-78	5.0 to 6.5	9	8			NP	SP		3		
HT-78	7.5 to 9.0	2	19			NP	SP		2		
HT-78	10.0 to 11.5	2	18								
HT-78	15.0 to 16.5	6									

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial FV = Field Vane

PROJECT NO. AEA08-020-02

Raba-Kistner

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: The Rio Grande Canalization Project
Hatch/Tonuco Segment

FILE NAME: AEA08-020-02.GPJ 4/26/2008

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
HT-159	15.0 to 16.5	6	20			NP					
HT-159	18.5 to 20.0	5	16			NP	SP		4		
HT-160	0.0 to 1.5	13	6	30	16	14					
HT-160	2.5 to 4.0	3	6			NP	SM		43		
HT-160	5.0 to 6.5	4	6			NP	SM		35		
HT-160	7.5 to 9.0	8	6			NP					
HT-160	10.0 to 11.5	6	3			NP					
HT-160	10.1		3						18		
HT-160	15.0 to 16.5	5	18			NP					
HT-160	18.5 to 20.0	4	17			NP	SP		3		
HT-161	0.0 to 1.5	15	8	35	20	15	CL		73		
HT-161	2.5 to 4.0	4	1			NP	SP		1		
HT-161	5.0 to 6.5	4	2			NP	SP		2		
HT-161	7.5 to 9.0	5	2			NP					
HT-161	10.0 to 11.5	2	3			NP			2		
HT-161	15.0 to 16.5	9	19			NP	SM		39		
HT-161	18.5 to 20.0	2	16			NP					
HT-162	0.0 to 1.5	14	3	30	19	11	SC		40		
HT-162	2.5 to 4.0	13	2			NP					
HT-162	5.0 to 6.5	3	2			NP	SP-SM		10		
HT-162	7.5 to 9.0	6	6			NP					
HT-162	10.0 to 11.5	6	2			NP	SP		5		
HT-162	15.0 to 16.5	5				NP					
HT-162	18.5 to 20.0	4				NP					
HT-163	0.0 to 1.5	12	5	28	19	9	CL		50		
HT-163	2.5 to 4.0	5	2			NP			12		
HT-163	5.0 to 6.5	4	1			NP					
HT-163	7.5 to 9.0	11	2			NP					
HT-163	10.0 to 11.5	8	2			NP	SP		0		
HT-163	15.0 to 16.5	3				NP					
HT-163	18.5 to 20.0	2	9			NP	SP		3		
HT-164	0.0 to 1.5	16	6	25	15	10	SC		50		
HT-164	2.5 to 4.0	5	6			NP	SM		25		
HT-164	5.0 to 6.5	2	5			NP					
HT-164	7.5 to 9.0	11	2			NP	SP-SM		5		
HT-164	10.0 to 11.5	8	4			NP					
HT-164	15.0 to 16.5	3	16			NP	SP		3		
HT-164	18.5 to 20.0	4	17			NP	SP		1		
HT-165	0.0 to 1.5	17	5			NP	ML		62		

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial FV = Field Vane

PROJECT NO. AEA08-020-02

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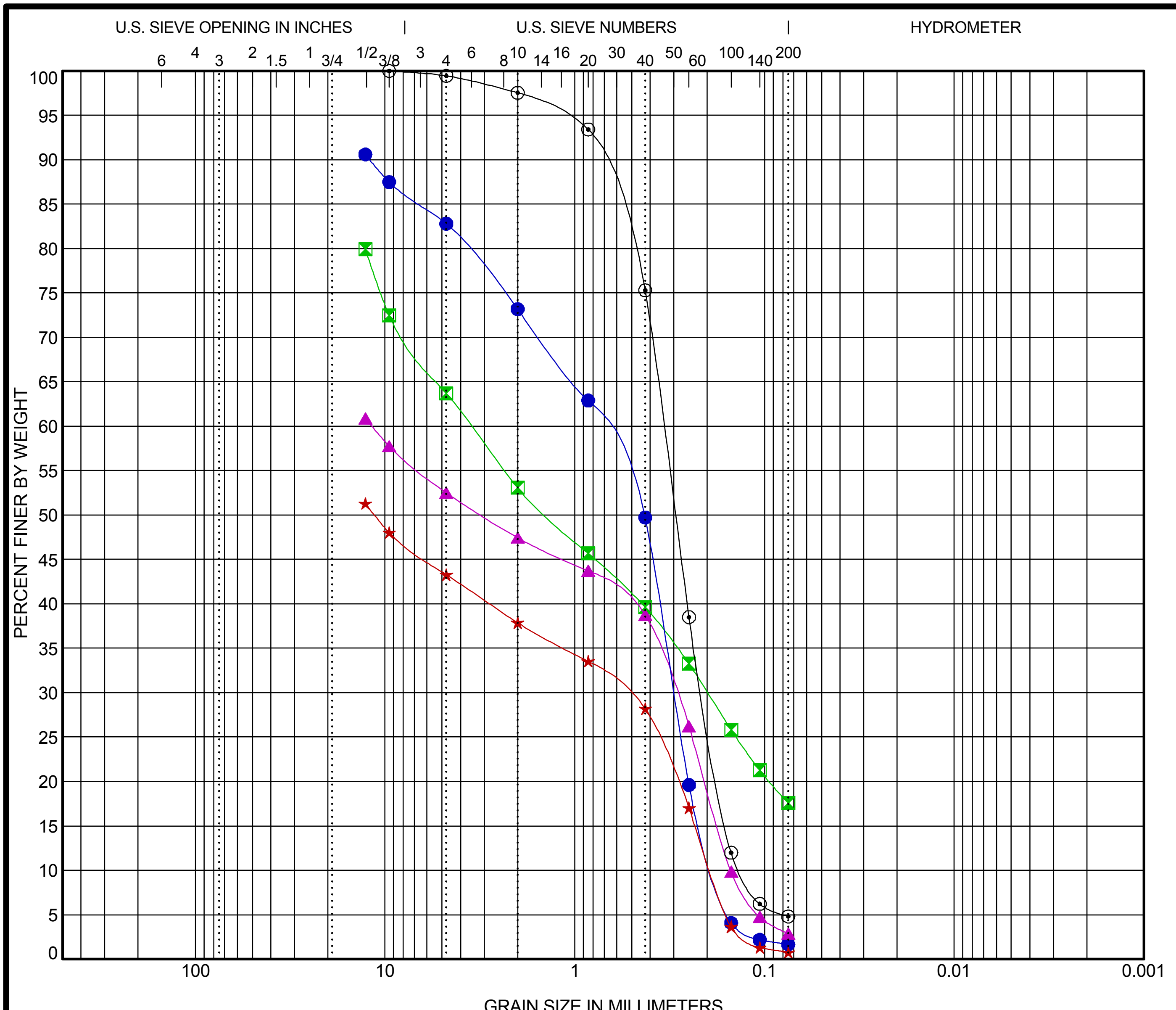
FIGURE 219?



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● HT-73	18.5	**POORLY GRADED SAND with GRAVEL (SP)**			NP	0.68	4.00
⊠ HT-74	2.5	**SILTY SAND with GRAVEL (SM)**			NP		
▲ HT-74	15.0	**POORLY GRADED SAND with GRAVEL (SP)**			NP	0.05	77.54
★ HT-74	18.5				NP		
⊙ HT-75	2.5	**POORLY GRADED SAND (SP)**			NP	0.99	2.56

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● HT-73	18.5	12.7	0.73	0.3	0.182	7.8	81.1	1.7	
⊠ HT-74	2.5	12.7	3.515	0.2		16.2	46.1	17.6	
▲ HT-74	15.0	12.7	11.693	0.293	0.151	8.4	49.6	2.9	
★ HT-74	18.5	12.7		0.537	0.191	8.0	42.5	0.8	
⊙ HT-75	2.5	9.5	0.341	0.212	0.133	0.6	94.6	4.8	

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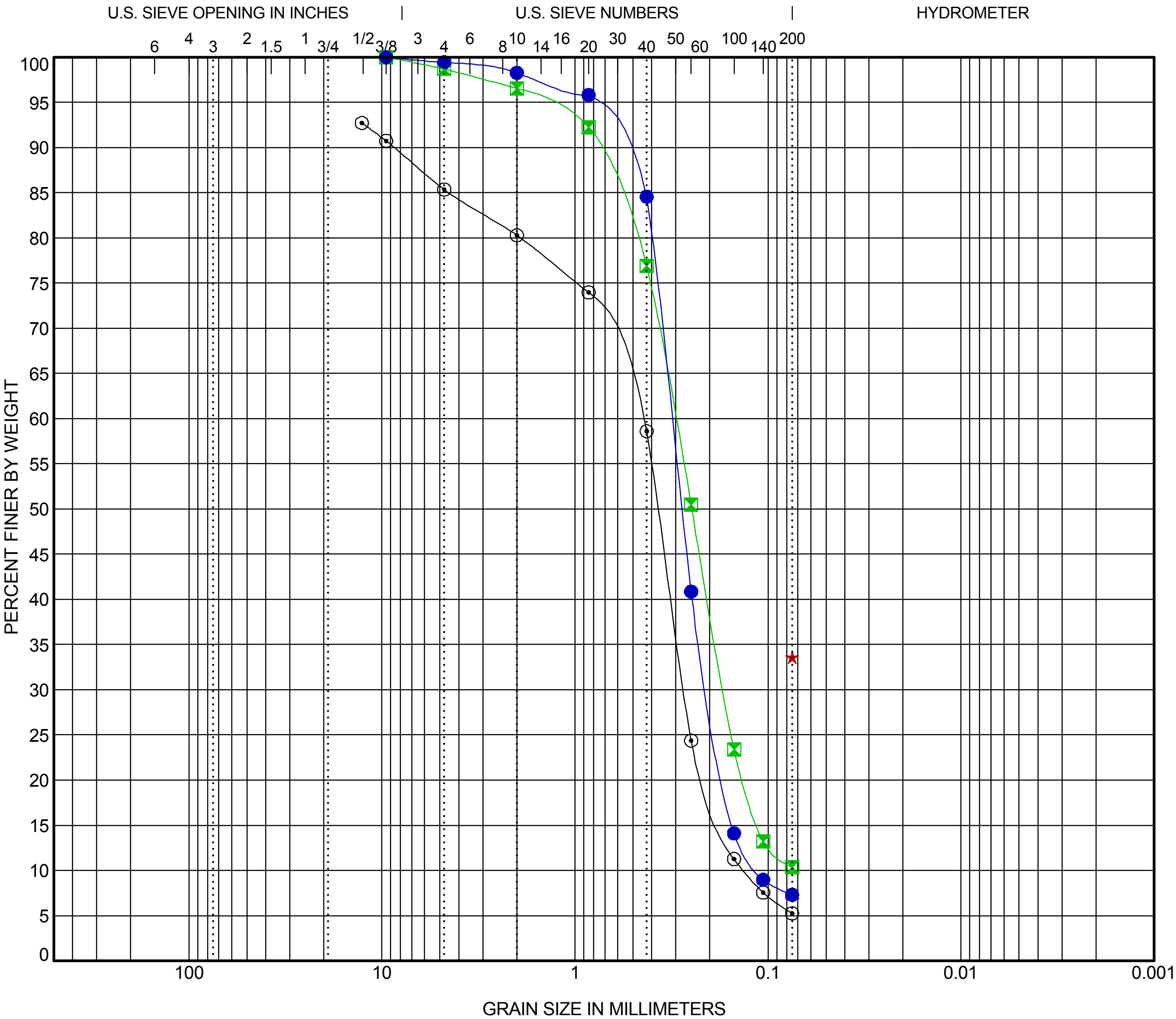
GRAIN SIZE DISTRIBUTION

The Rio Grande Canalization Project
Hatch/Tonuco Segment



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	HT-75	7.5	POORLY GRADED SAND with SILT (SP-SM)			NP	1.15	2.77
⊠	HT-75	7.6					1.33	4.23
▲	HT-75	15.0				NP		
★	HT-76	0.0	SILTY SAND (SM)			NP		
⊙	HT-76	5.0	POORLY GRADED SAND with SILT (SP-SM)			NP	1.23	3.39

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HT-75	7.5	9.5	0.315	0.203	0.114	0.6	92.1	7.3	
⊠	HT-75	7.6	9.5	0.303	0.17		1.3	88.3	10.4	
▲	HT-75	15.0	0.075				0.0	0.0	7.5	
★	HT-76	0.0	0.075				0.0	0.0	33.6	
⊙	HT-76	5.0	12.7	0.453	0.273	0.133	7.4	80.1	5.2	

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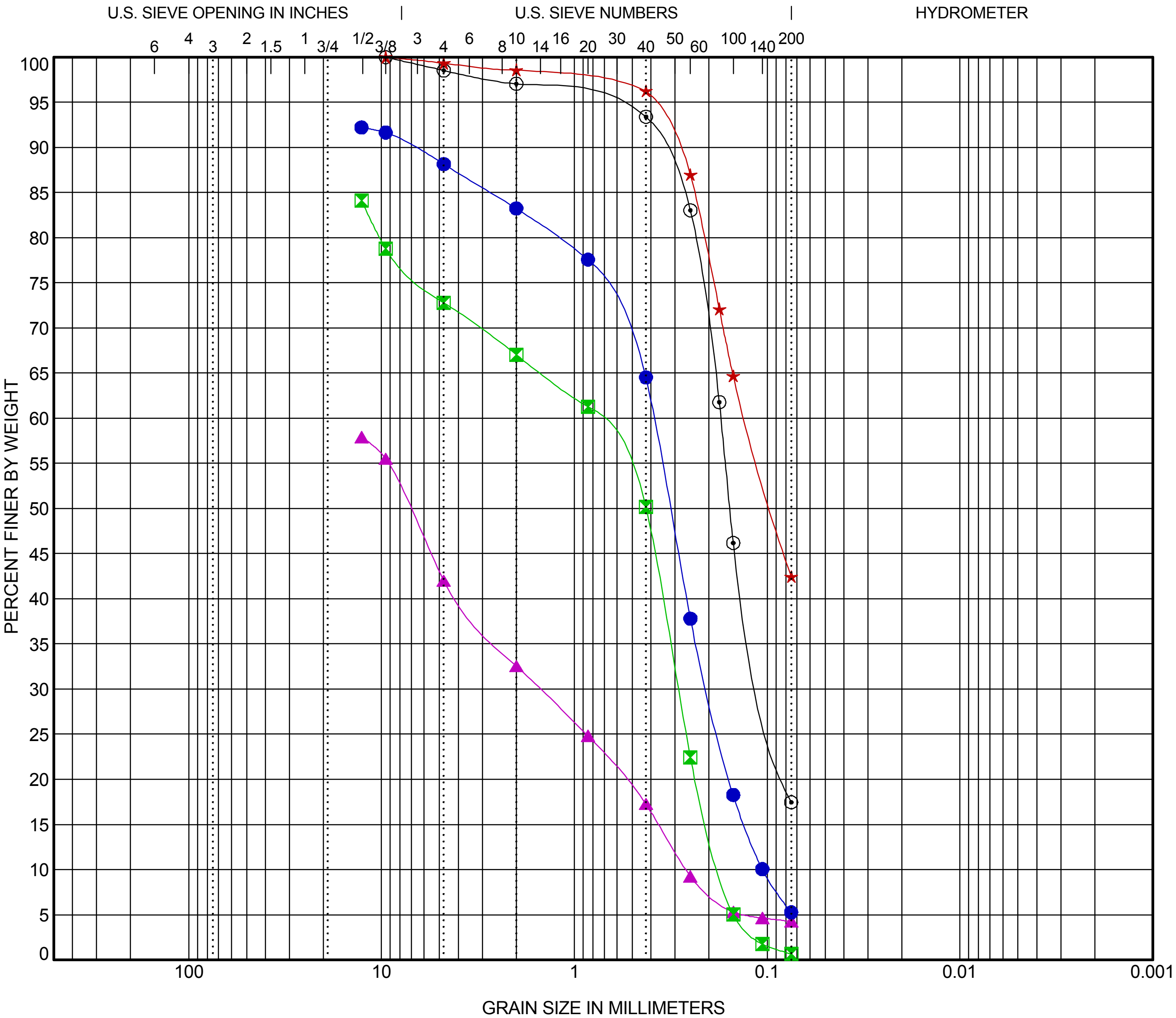

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● HT-76	7.5	**POORLY GRADED SAND with SILT (SP-SM)**			**NP**	**1.01**	**3.68**
⊠ HT-76	10.0	**POORLY GRADED SAND with GRAVEL (SP)**			**NP**	**0.61**	**4.52**
▲ HT-76	15.0				**NP**		
★ HT-77	2.5	**SILTY SAND (SM)**			**NP**		
⊙ HT-77	5.0	**SILTY SAND (SM)**			**NP**		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● HT-76	7.5	12.7	0.388	0.204	0.106	4.1	82.9	5.3	
⊠ HT-76	10.0	12.7	0.784	0.289	0.174	11.3	72.2	0.6	
▲ HT-76	15.0	12.7		1.509	0.263	15.9	37.7	4.3	
★ HT-77	2.5	25.7	0.13			0.7	56.9	42.5	
⊙ HT-77	5.0	25.7	0.174	0.102		1.5	81.1	17.4	

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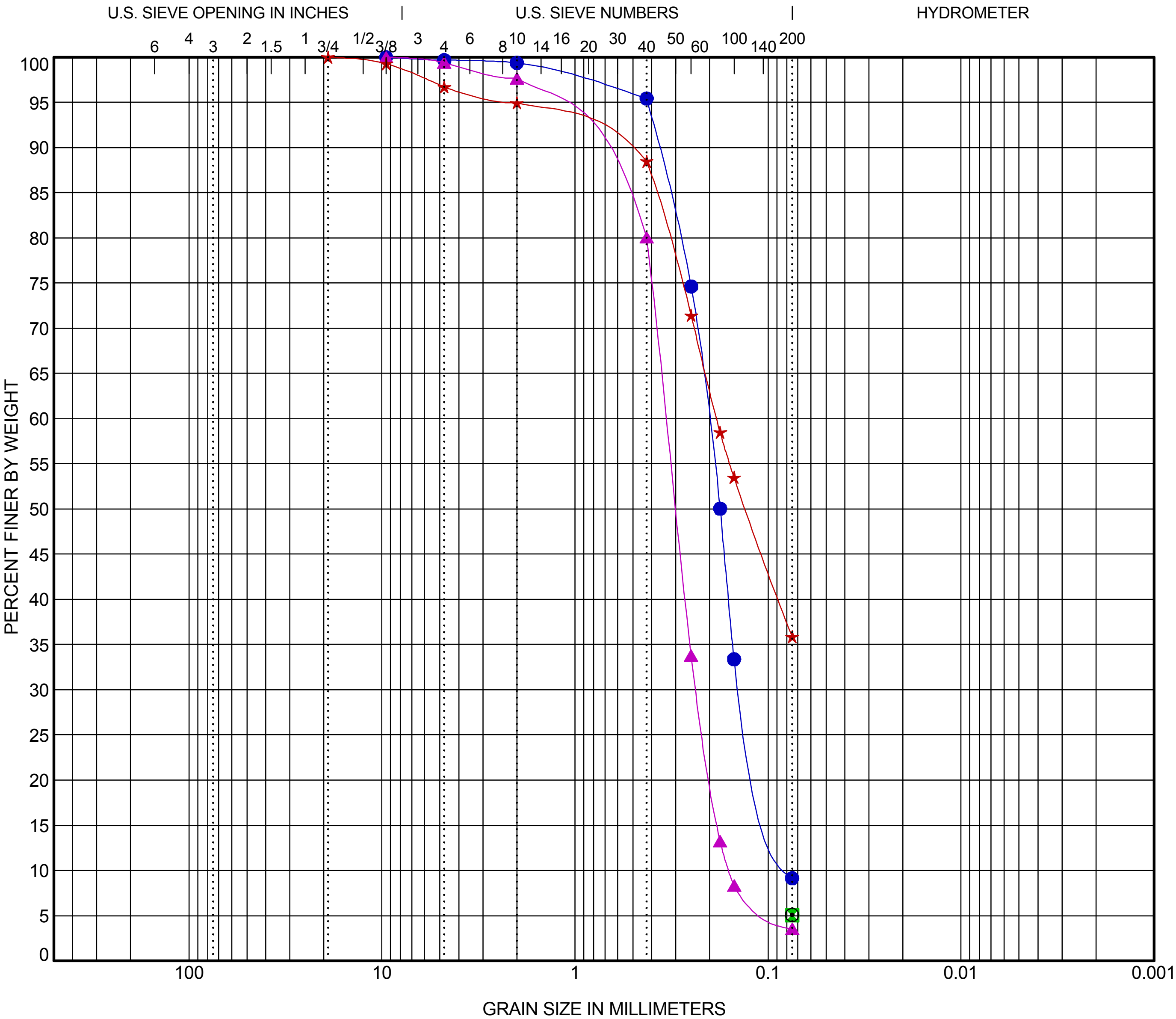

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**HT-77**	**7.5**	**POORLY GRADED SAND with SILT (SP-SM)**			**NP**	**1.19**	**2.65**
⊠	**HT-77**	**10.0**						
▲	**HT-77**	**15.0**	**POORLY GRADED SAND (SP)**			**NP**	**1.03**	**2.13**
★	**HT-78**	**0.0**	**SILTY SAND (SM)**			**NP**		
⊙	**HT-78**	**2.5**						

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**HT-77**	**7.5**	**25.7**	**0.204**	**0.136**	**0.077**	**0.3**	**90.6**	**9.1**	
⊠	**HT-77**	**10.0**	**0.075**				**0.0**	**0.0**	**5.1**	
▲	**HT-77**	**15.0**	**25.7**	**0.338**	**0.235**	**0.159**	**0.6**	**95.9**	**3.5**	
★	**HT-78**	**0.0**	**25.7**	**0.184**			**3.3**	**60.9**	**35.9**	
⊙	**HT-78**	**2.5**	**0.075**				**0.0**	**0.0**	**5.1**	

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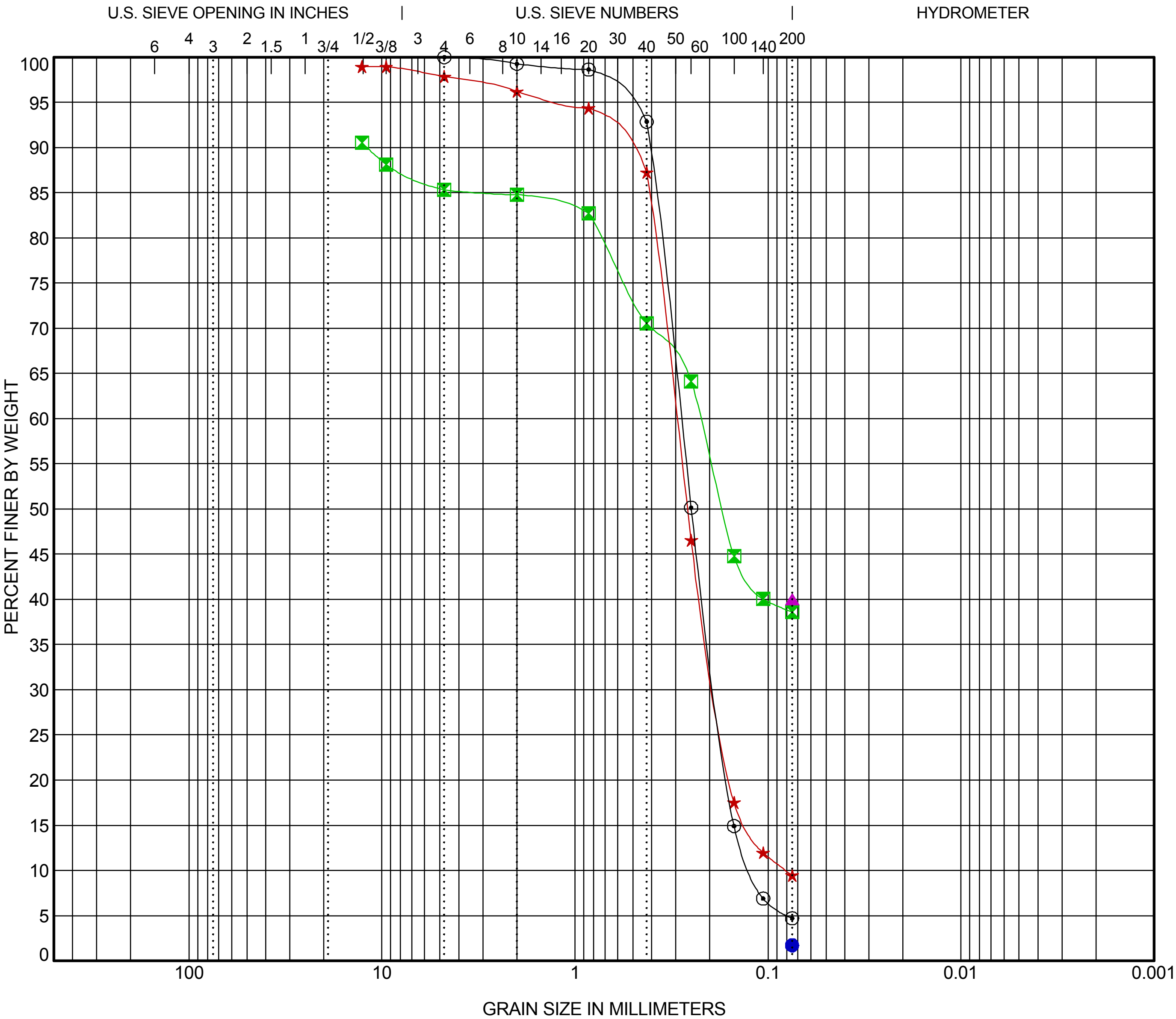
GRAIN SIZE DISTRIBUTION

The Rio Grande Canalization Project
Hatch/Tonuco Segment



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**HT-161**	**10.0**				**NP**		
⊠	**HT-161**	**15.0**	**SILTY SAND (SM)**			**NP**		
▲	**HT-162**	**0.0**	**CLAYEY SAND (SC)**	**30**	**19**	**11**		
★	**HT-162**	**5.0**	**POORLY GRADED SAND with SILT (SP-SM)**			**NP**	**1.45**	**3.69**
⊙	**HT-162**	**10.0**	**POORLY GRADED SAND (SP)**			**NP**	**1.02**	**2.33**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**HT-161**	**10.0**	**0.075**				**0.0**	**0.0**	**1.7**	
⊠	**HT-161**	**15.0**	**12.7**	**0.224**			**5.2**	**46.7**	**38.6**	
▲	**HT-162**	**0.0**	**0.075**				**0.0**	**0.0**	**40.0**	
★	**HT-162**	**5.0**	**12.7**	**0.298**	**0.187**	**0.081**	**1.1**	**88.4**	**9.5**	
⊙	**HT-162**	**10.0**	**4.75**	**0.283**	**0.187**	**0.121**	**0.0**	**95.3**	**4.7**	

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El Paso, Texas 79915
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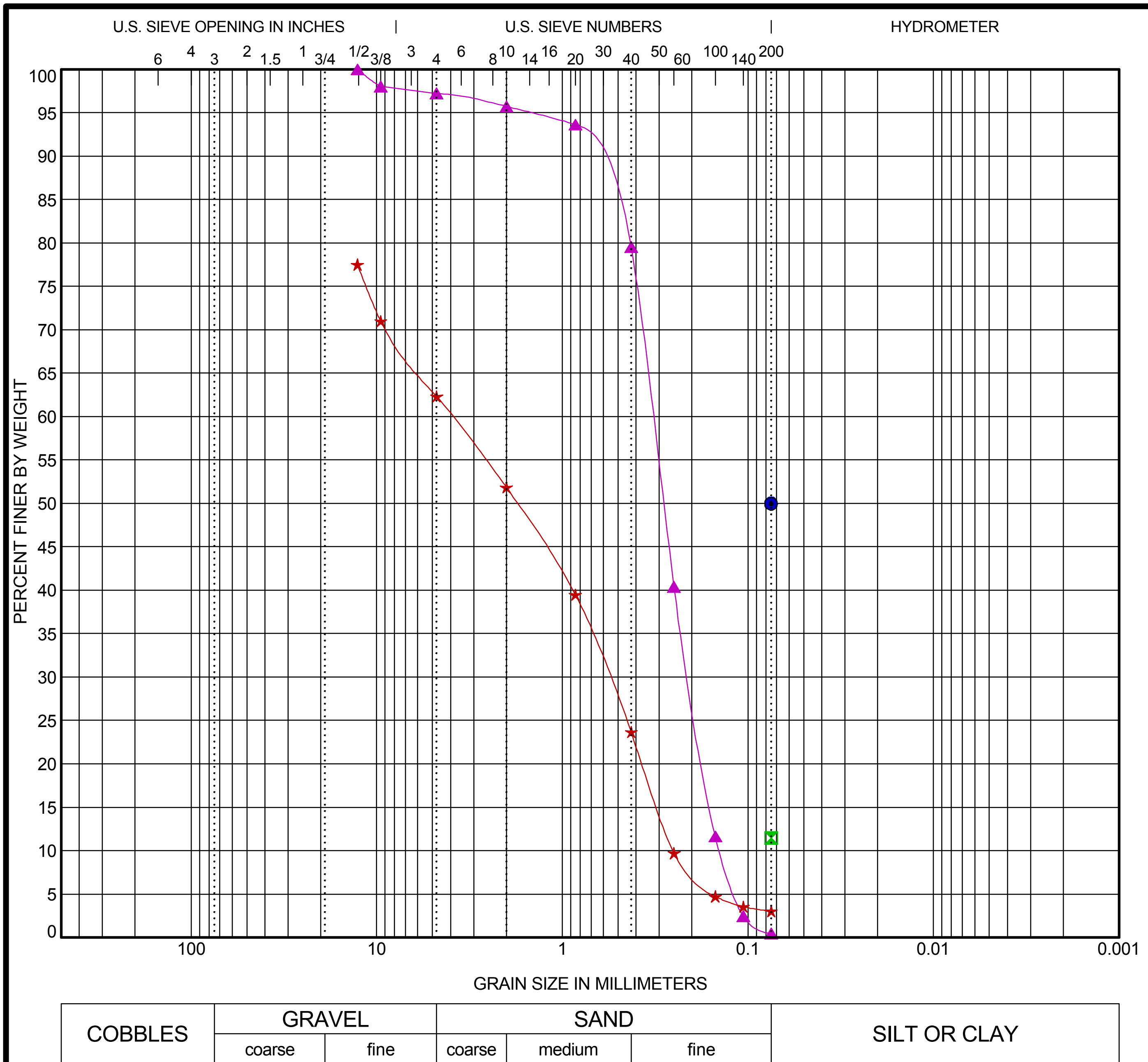
(915) 779-8301 fax
www.rkci.com

GRAIN SIZE DISTRIBUTION

The Rio Grande Canalization Project
Hatch/Tonuco Segment



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	HT-163	0.0	**SANDY LEAN CLAY (CL)**	28	19	9		
⊠	HT-163	2.5				NP		
▲	HT-163	10.0	**POORLY GRADED SAND (SP)**			NP	0.94	2.31
★	HT-163	18.5	**POORLY GRADED SAND with GRAVEL (SP)**			NP	0.32	15.50
⊙	HT-164	0.0	**CLAYEY SAND (SC)**	25	15	10		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HT-163	0.0	0.075				0.0	0.0	50.0	
⊠	HT-163	2.5	0.075				0.0	0.0	11.5	
▲	HT-163	10.0	12.7	0.326	0.208	0.141	2.8	96.8	0.4	
★	HT-163	18.5	12.7	3.92	0.561	0.253	15.2	59.3	3.0	
⊙	HT-164	0.0	0.075				0.0	0.0	50.0	

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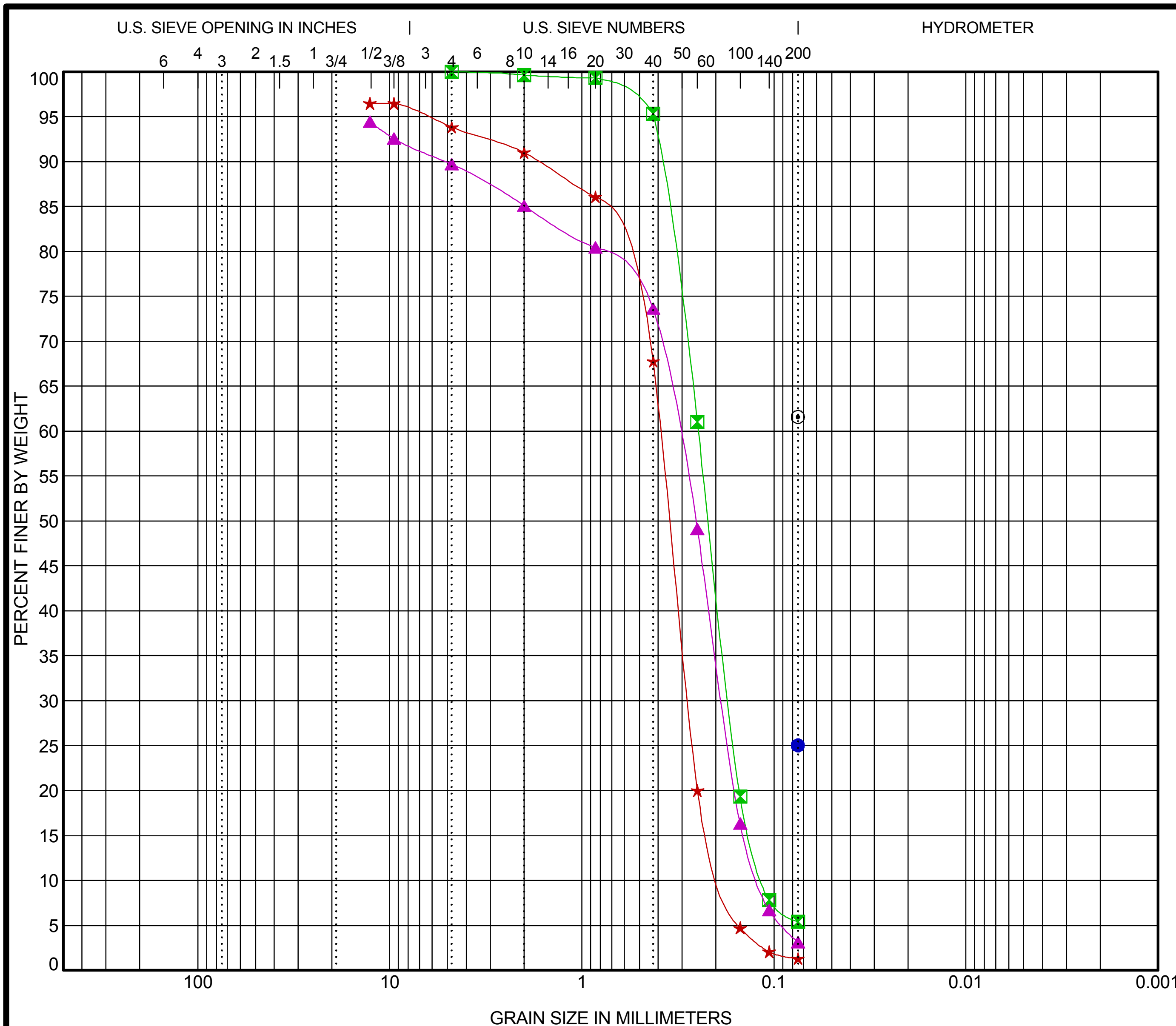

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● HT-164	2.5	**SILTY SAND (SM)**			NP		
⊠ HT-164	7.5	**POORLY GRADED SAND with SILT (SP-SM)**			NP	1.05	2.18
▲ HT-164	15.0	**POORLY GRADED SAND (SP)**			NP	0.91	2.65
★ HT-164	18.5	**POORLY GRADED SAND (SP)**			NP	1.12	2.18
⊙ HT-165	0.0	**SANDY SILT (ML)**			NP		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● HT-164	2.5	0.075				0.0	0.0	25.0	
⊠ HT-164	7.5	4.75	0.247	0.171	0.113	0.0	94.6	5.4	
▲ HT-164	15.0	12.7	0.316	0.186	0.119	4.7	86.6	3.1	
★ HT-164	18.5	12.7	0.39	0.279	0.179	2.6	92.5	1.3	
⊙ HT-165	0.0	0.075				0.0	0.0	61.6	

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GRAIN SIZE DISTRIBUTION

The Rio Grande Canalization Project
Hatch/Tonuco Segment

Permeability Test Data

Project: IBWC Levees - AEA08-020-02 Date: 4/1/2008

Reported To: Raba-Kistner Consultants, Inc. Job No.: 6437-B

Boring No.:							
Sample No.:	HT-70	HT-80	HT-85	HT-90	HT-95	HT-160	HT-75
Depth (ft)	10-15	10-15	10-15	0-5	0-5	10-15	7.5-9
Location:							
Sample Type:	Bag	Bag	Bag	Bag	Bag	Bag	Bag
Soil Type:	Sand w/a trace of gravel, medium grained (SP)	Sand w/Silt, fine grained (SP-SM/SP)	Sand w/Silt and a little gravel, fine to medium grained (SP-SM)	Silty Sand w/a trace of gravel (SM)	Silty Sand w/gravel (SM)	Sand w/Silt, fine grained (SP-SM)	Sand, fine grained (SP)
Atterberg Limits							
LL							
PL							
PI							
Permeability Test							
Before Test Conditions: Saturation %:							
Porosity:							
Ht. (in):	3.99	3.99	3.99	3.99	3.99	3.99	3.99
Dia. (in):	2.89	2.89	2.89	2.89	2.89	2.89	2.89
Dry Density (pcf):	101.0	92.8	99.0	102.6	103.1	92.7	90.1
Water Content:	14.5%	3.1%	3.1%	3.0%	3.7%	3.3%	7.2%
Test Type:	Constant	Constant	Constant	Constant	Constant	Constant	Constant
Max Head (cm):	8.7	11.9	11.3	51.2	48.6	12.2	7.3
Confining press. (Effective-psi):	None	None	None	None	None	None	None
Trial No.:	7-11	7-11	7-11	7-11	7-11	7-11	7-11
Water Temp ℃:	22.9	21.9	21.8	21.7	21.5	21.3	20.4
% Compaction							
% Saturation (After Test)							
Coefficient of Permeability							
K @ 20 ℃ (cm/sec)	**3.4×10^{-3}**	**2.1×10^{-3}**	**2.0×10^{-3}**	**3.0×10^{-4}**	**1.6×10^{-4}**	**3.0×10^{-3}**	**8.1×10^{-3}**
K @ 20 ℃ (ft/min)	**6.8×10^{-3}**	**4.1×10^{-3}**	**4.0×10^{-3}**	**5.9×10^{-4}**	**3.1×10^{-4}**	**5.8×10^{-3}**	**1.6×10^{-2}**
Notes:							

9301 Bryant Ave. South Suite 107 SOIL ENGINEERING TESTING, INC. Bloomington, Minnesota 55420-3436

Direct Shear Test

ASTM: D3080

Job No.: 6437-B

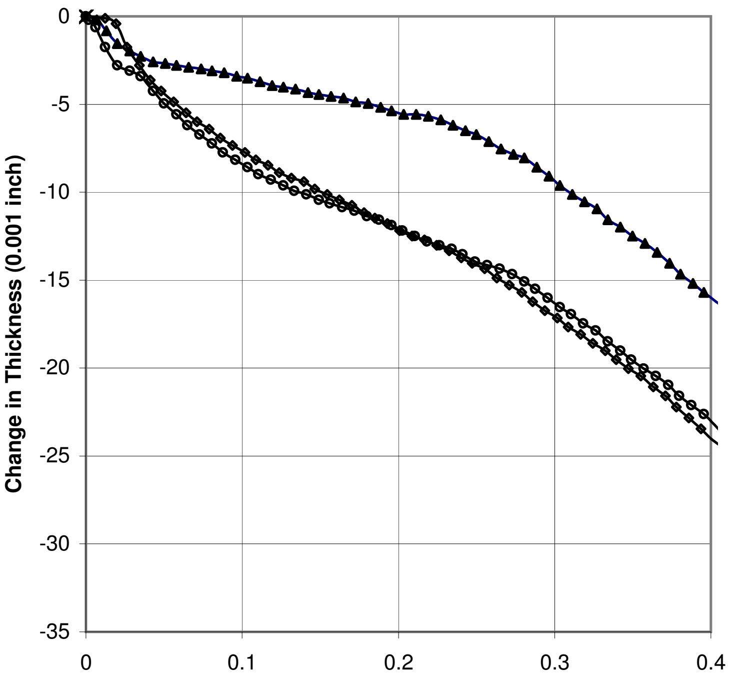
Project:	**IBWC Levees - #AEA08-020-02**				
Boring No.:	**HT-75**	Sample No.		Depth:	**7.5-9**
Location:				Sample Type:	**Bags**
Soil Type:	**Sand, Fine Grained (SP)**				

Test Date:	**4/15/2008**
Date Reported:	**4/21/2008**
Shear Rate	
0.007 (in/min)	

Remarks: Specimens compacted using -#4 material to given densites at as received water content. Inundated after applying normal load; Allowed to consolidate past T-100. Sheared to given diplacements at a constant rate of 0.007"/min.

(*) = Assumed Specific Gravity

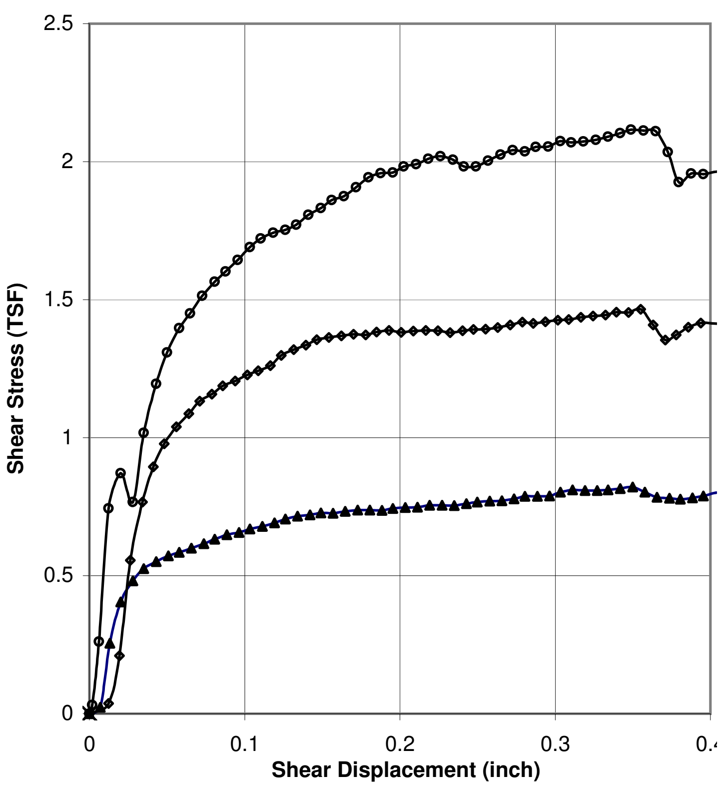
Liquid Limit:	
Plastic Limit:	
Plasticity Index:	
Specific Gravity (*):	**2.65**

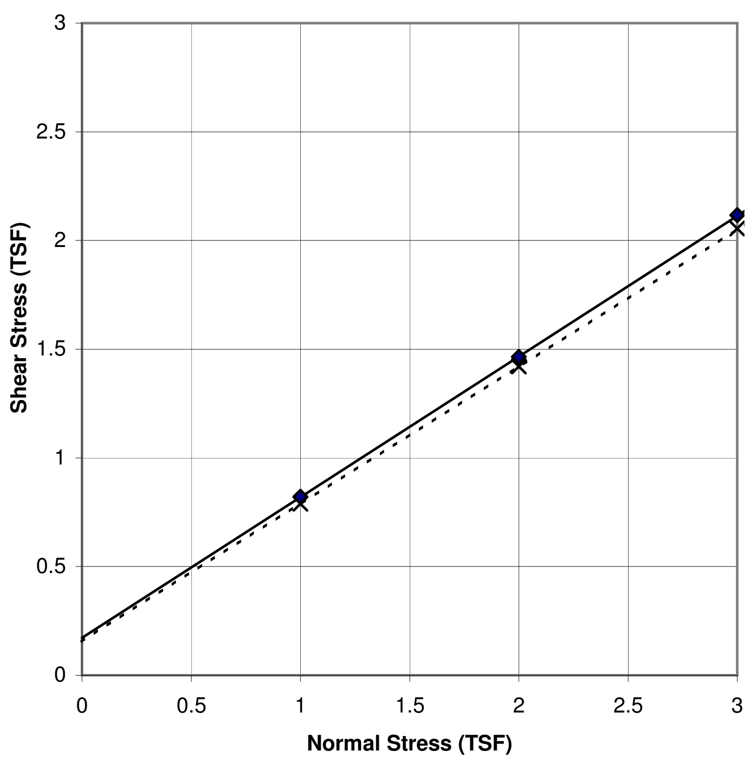


Failure Criterion:				
Max Shear Stress	A	B	C	D
Initial	▲	◇	○	X
Diameter (In.)	2.50	2.50	2.50	
Thickness (In.)	1.10	1.10	1.10	
Water Content (%)	7.2	7.2	7.2	
Dry Density (pcf)	90.1	90.1	90.1	
Before Shear				
Thickness (In.)	1.09	1.09	1.09	
Water Content (%)	31.2	30.9	30.8	
Dry Density (pcf)	90.5	90.9	91.1	
Normal Stress	1.00	2.00	3.00	
Shear Stress	0.82	1.47	2.12	

"These tests are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design."

Peak Conditions			At Given Shear Disp. Of:		0.3
Friction Angle:	ϕ= **32.9**	**deg.**	Friction Angle:	ϕ= **32.3**	**deg.**
Apparent Cohesion	**0.173**	**TSF**	Apparent Cohesion	**0.155**	**TSF**





B4. Site Seismicity Data

USGS Design Maps Summary Report

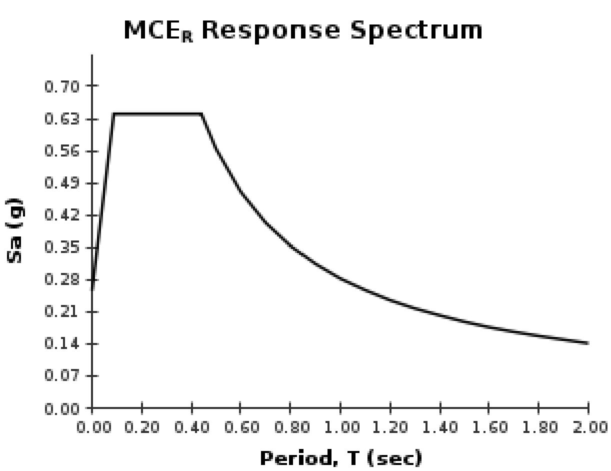
User–Specified Input

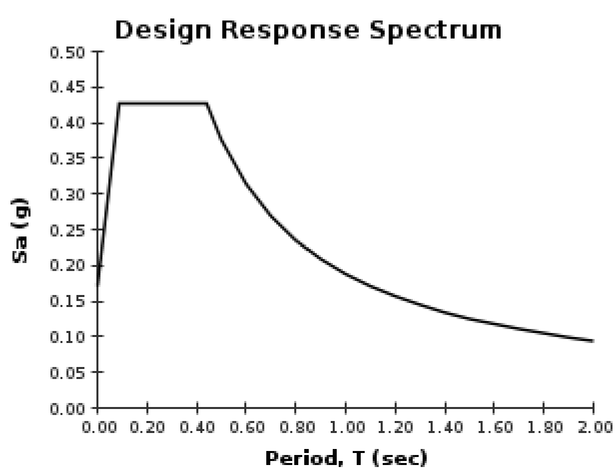
Report Title	Thurman Arroyo I Fri December 2, 2016 20:23:41 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.683°N, 107.177°W
Site Soil Classification	Site Class E – "Soft Clay Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.259 g	**S_{MS} =**	0.640 g	**S_{DS} =**	0.427 g
S_1 =	0.081 g	**S_{M1} =**	0.282 g	**S_{D1} =**	0.188 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (32.683°N, 107.177°W)

Site Class E – "Soft Clay Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 0.259 g
From Figure 22-2 [2]	S_1 = 0.081 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class E, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = E and S_S = 0.259 g, F_a = 2.471

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = E and S_1 = 0.081 g, F_v = 3.500

Equation (11.4–1): $S_{MS} = F_aS_S = 2.471 \times 0.259 = 0.640$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 3.500 \times 0.081 = 0.282$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

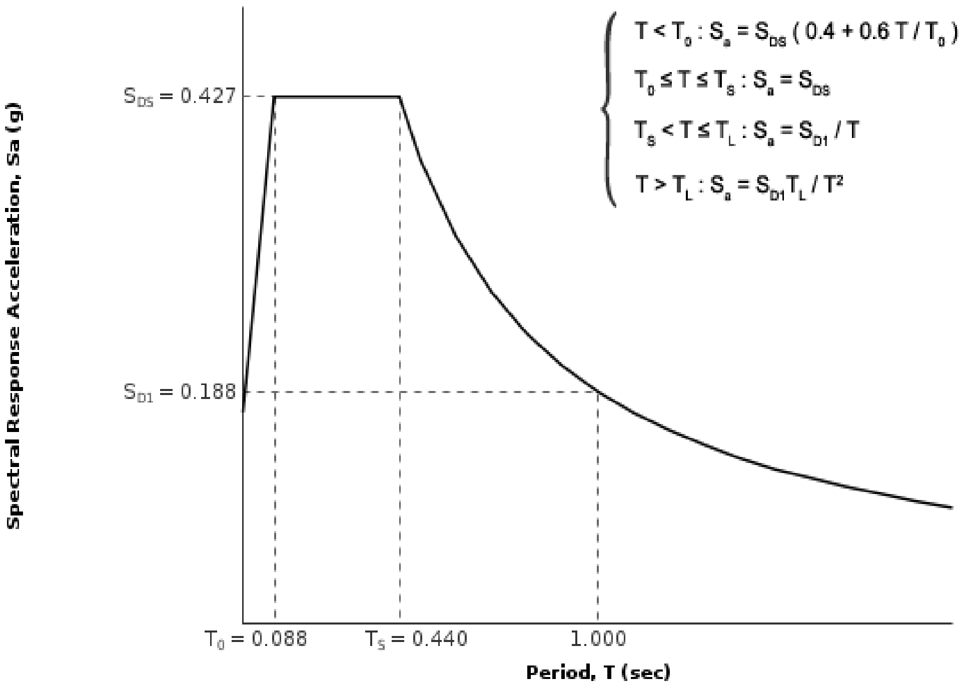
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 0.640 = 0.427$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.282 = 0.188$ g

Section 11.4.5 — Design Response Spectrum

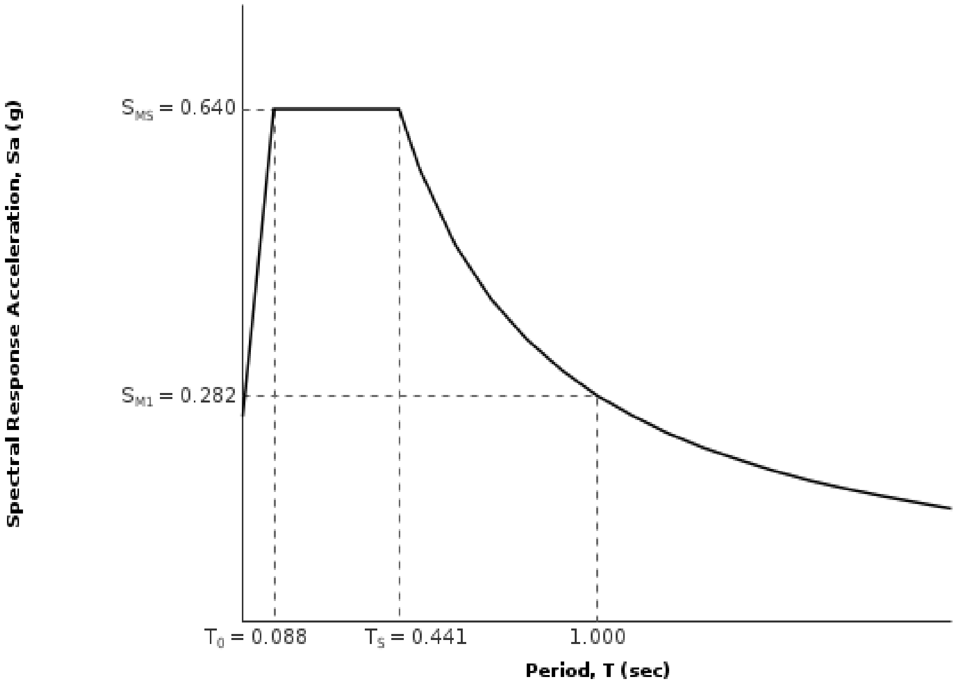
From Figure 22-12 [3] $T_L = 6$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.107

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 2.446 \times 0.107 = 0.261$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = E and PGA = 0.107 g, F_{PGA} = 2.446

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.901

From Figure 22-18 [6] C_{R1} = 0.916

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.427 g, Seismic Design Category = C

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.188 g, Seismic Design Category = C

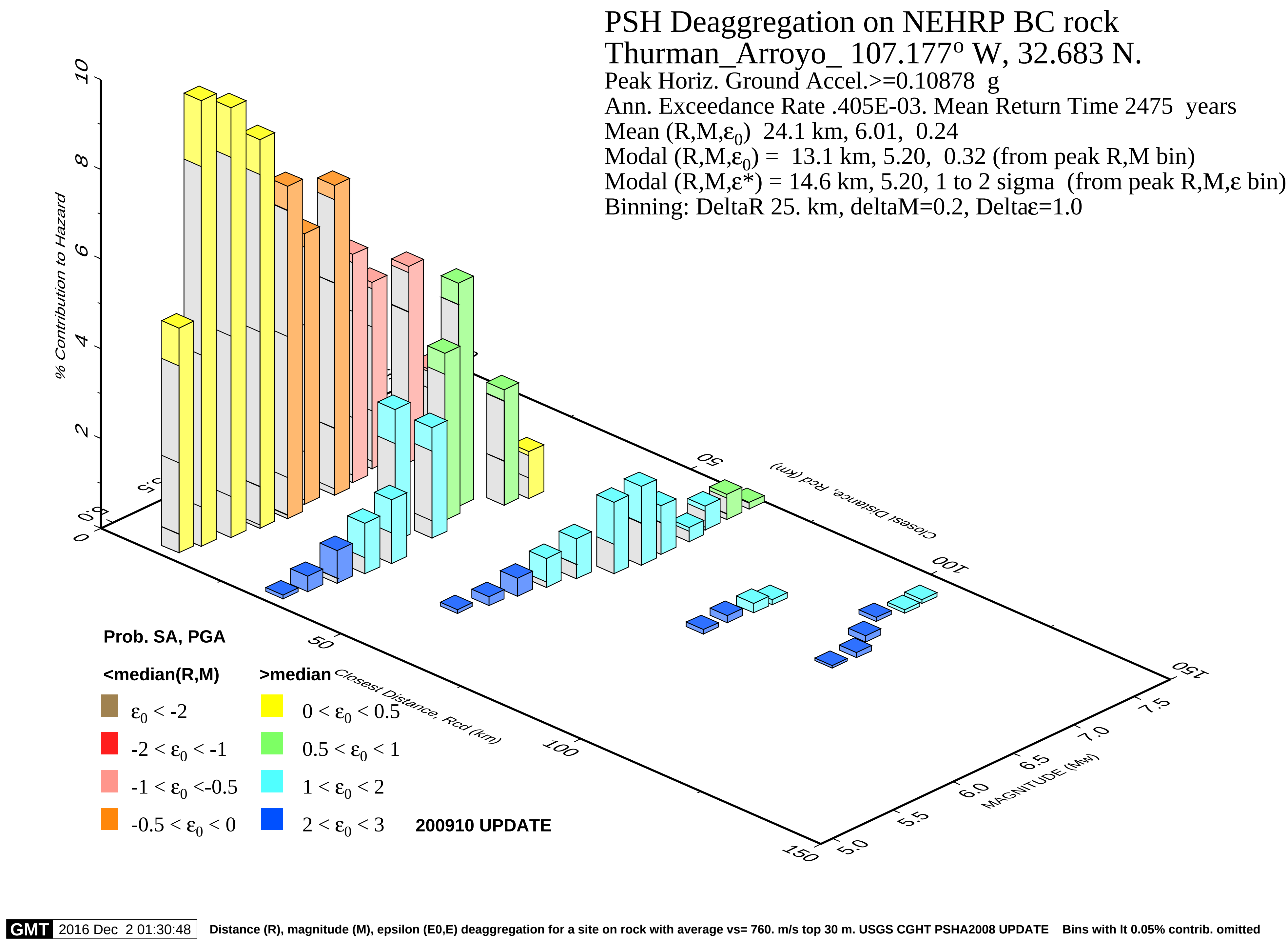
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = C

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf


PSH Deaggregation on NEHRP BC rock
Thurman_Arroyo_ 107.177° W, 32.683 N.
Peak Horiz. Ground Accel.>=0.10878 g
Ann. Exceedance Rate .405E-03. Mean Return Time 2475 years
Mean (R,M,ε0) 24.1 km, 6.01, 0.24
Modal (R,M,ε0) = 13.1 km, 5.20, 0.32 (from peak R,M bin)
Modal (R,M,ε*) = 14.6 km, 5.20, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 25. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
0
2
4
6
8
10
Closest Distance, Rcd (km)
50
100
150
MAGNITUDE (Mw)
5.0
5.5
6.0
6.5
7.0
7.5
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Dec 2 01:30:48
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

Appendix C
Operations and Maintenance Plan

United States Section of the International Boundary and Water Commission

Design for the Construction of Channel Maintenance Alternatives within the Rio Grande Canalization Project Doña Ana County, New Mexico

Contract No. IBM15D0003
Order No. IBM16T0018

Appendix C

Operations and Maintenance Plan
Final Submittal

April 27, 2018

Prepared by:

URS
9400 Amberglen Blvd
Austin, TX 78729

Table of Contents

List of Tables

List of Figures

C.1 PROJECT DESCRIPTION

C.1.1 Historical Background

The Rio Grande Canalization Project (RGCP), which extends 105.4 miles from Percha Dam in Sierra County, New Mexico to American Dam in El Paso, Texas, was constructed between 1938 and 1943 as authorized by the Act of Congress approved June 4, 1936 (Public Law 648, 49 Stat 1463) to: "facilitate compliance with the convention between the United States and Mexico concluded May 21, 1906, providing for the equitable division of the waters of the Rio Grande, and to properly regulate and control, to the fullest extent possible, the water supply for use in the two countries as provided by treaty." The Act authorizes the United States International Boundary and Water Commission (USIBWC) to construct, operate and maintain the RGCP in accordance with the plan in the Engineering Record of December 14, 1935. The USIBWC objectives for the RGCP can be summarized by: Flood Conveyance and Flood Protection, Channel Conveyance Reliability, Delivery Efficiency, Compliance with U.S. Regulations, and Minimizing Costs.

C.1.2 Sediment Basin Project Background

There is ongoing sediment inflow from the tributary arroyos, resulting in sediment deposition forming sediment plugs at arroyo confluences along sections of the Rio Grande. Sediment inflow also results in island formations and raising of river beds. Sediment accumulation prevents draining of irrigation return flow to the Rio Grande and may result in increases in water surface elevations, which could impact levee freeboard and increase the flooding risk to adjoining communities. A study entitled *Channel Maintenance Alternatives and Sediment Transport Studies for the RGCP Final Report* was completed in 2015 by Tetra Tech, Inc. (hereafter "Tetra Tech 2015"). The report identified nine (9) representative problem locations experiencing sediment accumulation along the 105.4 miles of the RGCP that were evaluated in the study. The report then evaluated, scored, and ranked various Channel Maintenance Alternatives (CMAs) for each of the nine (9) problem locations. The report presented a conceptual sediment trap as one of the CMAs, and because of the high benefit-to-cost consequence of the sediment trap as determined in the report, it was recommended as an alternative to be used at all of the problem locations.

USIBWC contracted URS Group, Inc. (URS) to perform design of one or more of the CMAs at each of two (2) selected locations within USIBWC's ROW. The two selected sites are referred to as "Thurman I Arroyo" and "Thurman II Arroyo" and are located within Problem Location 2, which extends a distance of approximately 3.3 miles from the Salem Bridge at NM Highway 391 downstream to the confluence with Placitas Arroyo (see Figure 1-1).

The two CMAs designed and implemented at Thurman I Arroyo and Thurman II Arroyo were:

1. Construct an excavated sediment basin at the mouth of each arroyo; and
2. Remove localized sediment within the Rio Grande main channel at each arroyo.

From an O&M perspective, the intent of the project is to remove the existing sediment that has accumulated in the Rio Grande near the mouth and immediately downstream of each arroyo; prevent future accumulation of arroyo sediment in the river by trapping it in the sediment basins; and as such, reduce or simplify the overall operations and maintenance at each site. Additional benefits of the sediment basins are that they will improve conveyance efficiency, hydraulic capacity, drainage return flows, and levee infrastructure, and will decrease flood risk.

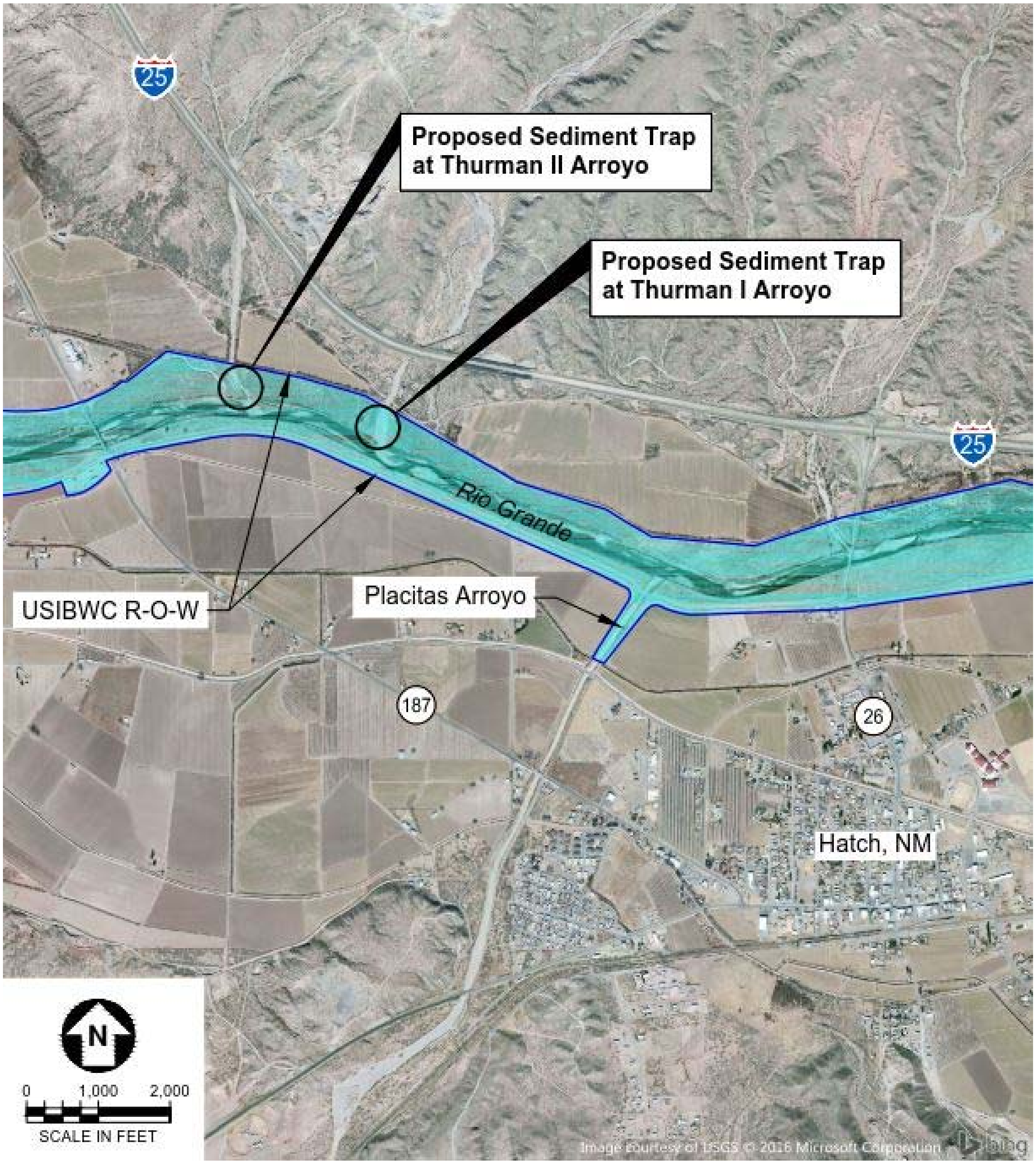


Figure C-1. Project Location Map

C.2 SEDIMENT BASIN INSPECTIONS

This section provides standard procedures for:

- Inspection of the sediment basins and their appurtenances;
- Routine monitoring; and
- Maintenance of the sediment basin and their appurtenances.

C.2.1 Inspections

The inspection program for the two sediment basins at the Thurman I Arroyo and Thurman II Arroyo includes both routine inspections and special inspections. The routine inspections include:

- Ongoing surveillance and monitoring by USIBWC field staff; and
- Periodic condition and maintenance inspections.

The special inspections are performed when conditions lead to a need for an inspection outside the prescribed routine. These special inspections are described in Section C.2.1.2. Table C-1 provides a recommended schedule interval for inspections.

Table C-1. Inspection Schedule

Inspection Type	Frequency								
	Continuous	Daily	Weekly	Monthly	Quarterly	Semi-Annually	Annually	Every 5 Years	Occasional
Ongoing Surveillance	X								
Condition/Maintenance					X				
Special Inspection – Unusual or Emergency Event									X

C.2.1.1 Routine Inspections

C.2.1.1.1 Ongoing Surveillance by Field Staff

Ongoing surveillance and monitoring by area staff involves routine checking of sediment basin components when USIBWC staff are in the vicinity. Unusual conditions or changes from the routine are reported to the Operations and Maintenance Division (O&M) or the Engineering Department, as appropriate.

C.2.1.1.2 Condition and Maintenance Inspections

Condition and maintenance inspections will be regularly scheduled and will include assessing the current condition of the sediment basins and their components to identify any potential problems that need to be addressed. These inspections will be performed by USIBWC staff from either the O&M Division or the Engineering Department who are knowledgeable in the function of the sediment basins and can identify potential problems.

- Inspect the earthen side slopes for failures, erosion damage, and animal damage.
- Inspect the concrete end walls for damage or obstructions, such as tree limbs and trash. Remove all obstructions from the weirs.
- Inspect rock riprap and gabion mattresses at the inlet flumes and at the concrete end walls for excessive stone displacement or erosion.
- Inspect for the growth of woody vegetation within the sediment basins, in the vicinity of the end walls, or within the limits of the rock riprap downstream of the end walls. Any woody vegetation that does take root within these areas should be removed as soon as possible.
- Note depth of sediment at each sediment depth gauge.

C.2.1.2 Special Inspections

Special inspections are performed when site conditions or forecasted weather conditions lead to a need for an inspection outside the prescribed routine. These conditions include:

- **Prior to major storms:** Inspect the overall condition of the basins, including:
  - Flumes at entrances of basins;
  - Basin side slopes;
  - End walls;
  - Downstream of end walls; and
  - Record the depth of the accumulated sediment at the sediment depth gauges.
- **During major storms:** Inspect and observe the overall performance of the basin.
- **After major storms:** Again inspect the overall condition of the basins, including:
  - Flumes at entrances of basins;
  - Basin side slopes;
  - End walls;
  - Downstream of end walls; and
  - Record the depth of the accumulated sediment at the sediment depth gauges.

C.3 OPERATIONS AND MAINTENANCE

The following are recommended methods and strategies for operating and maintaining the constructed sedimentation basins. The sedimentation basins have been designed to require minimum operator attention and minimize long-term maintenance.

C.3.1 Operational Procedures

There are no consistent ongoing operational procedures associated with the two sediment basins.

C.3.2 Maintenance Activities

C.3.2.1 Sediment Removal Maintenance Intervals

The primary maintenance activity to be performed is the periodic removal of accumulated sediment from the sediment basins. To maintain the trapping efficiency of the sediment basins, it is recommended that a basin have the trapped sediment removed when the basin has lost 75% of its design volume. Below are the assumed maintenance intervals for the basins:

> **Thurman I Arroyo Sediment Basin:** The total design sediment capacity of the Thurman I Arroyo Sediment Basin is 5.21 acre-feet. Based on a mean annual sediment yield of 1.12 acre-feet coming from the Thurman I Arroyo, and only allowing the basin to fill to 75% capacity before cleaning, the maintenance interval for the Thurman I Arroyo basin is estimated to be **3.5 years**.

> **Thurman II Arroyo Sediment Basin:** The total design sediment capacity of the Thurman II Arroyo Sediment Basin is 5.41 acre-feet. Based on a mean annual sediment yield of 1.98 acre-feet coming from the Thurman II Arroyo, and only allowing the basin to fill to 75% capacity before cleaning, the maintenance interval for the Thurman II Arroyo basin is estimated to be **2.0 years**.

The above stated maintenance intervals could vary significantly depending on the frequency of major monsoon events over the arroyo watersheds capable of delivering significant quantities of sediment down the arroyos to the basins. Two (2) sediment depth gauges have been installed in each of the sediment basins to aid USIBWC staff in continual monitoring of the depth of the accumulated sediment in the basins (see Figure 3-1). It is recommended that when the depth of the sediment reaches approximately four (4) feet the basins should be cleaned.

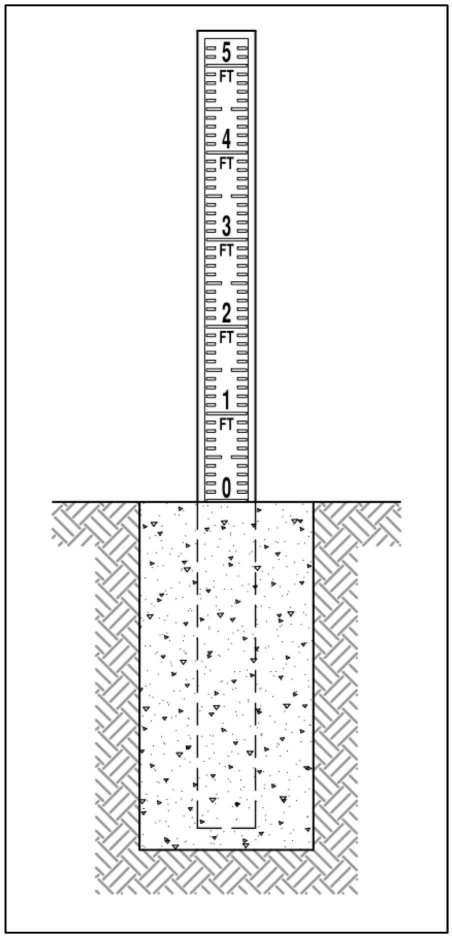


Figure C-2. Sediment Depth Gauge Detail

C.3.2.2 Sediment Removal Method

There are three main methods of sediment removal: hydraulic dredging, drag-line dredging, and land-based excavating with earth-moving equipment. Given the anticipated normally dry conditions of the sediment basins in the arid environment, and due to the relatively small size of the basins, the use of land-based equipment would be the most practical method of sediment removal. Front-end loaders will load the sediment into dump trucks for removal and disposal offsite. Skid steer loaders can be used in tighter, less accessible spaces such as near the entrance flumes, sediment depth gauges, and end walls. The skid steer loader can consolidate sediment from these locations to a location where the front-end loader can then load it into trucks.

An access road and ramp have been provided at each sediment basin to allow access to the bottom of the basins for cleaning. The sediment should only be removed down to the top of the sediment depth gauge footings and no lower.

C.3.2.3 Permitting for Sediment Removal

Sediment removal operations will require a permit from the U.S. Army Corps of Engineers since the arroyos and basins are most likely below the Ordinary High Water Mark (OHWM) and would be considered waters of the United States under the Clean Water Act. This permit is required even when sediment spoil is disposed of in an upland site.

C.3.2.4 Disposal of Sediment

Sediment removed from the basins will be disposed of at unidentified offsite location(s). The disposal site(s) will need to be cleared for the presence of protected cultural resources by the New Mexico Historic Preservation Division and cleared of protected wetlands prior to disposal.

C.3.2.5 Concrete End Walls

Changes in concrete condition will be noted in regular monitoring and inspection and reviewed by a structural engineer at USIBWC’s discretion.

C.3.2.6 Rock Riprap

All exposed riprap should be inspected for stability on an annual basis. Any riprap that is misplaced or that has been moved should be replaced (if possible with heavier stones). Where erosion has occurred, protective measures should be installed to minimize further erosion.

C.3.2.7 Maintenance Records

Maintenance performed on the basins should be documented and records maintained by USIBWC's O&M Division.