Appendix H

Hydrology and Hydraulics

December 2019

Attachment 1—H&H certification

MIDDLE RIO GRANDE FLOOD CONTROL STUDY
HYDRAULIC ANALYSES

COMPLETION OF INDEPENDENT TECHNICAL REVIEW
AND CERTIFICATION

The Albuquerque District of the U.S. Army Corps of Engineers (USACE) has completed the hydraulic analysis for the Middle Rio Grande Flood Control Study. Notice is hereby given that an independent technical review, that is appropriate to the level of risk and complexity inherent to the project, has been conducted as defined in the Quality Control Plan. During the independent technical review, compliance with established policy principles and procedures was verified. This included review of assumptions, methods, procedures, and material used in the analysis, the appropriateness of the data used and the level obtained, and reasonableness of the results, including whether the product meets the customer's needs consistent with law and existing USACE policy. The independent technical review was accomplished by Don Twiss, Hydraulic Engineer in the Hydraulic Design Section of the Sacramento District on 4 December 2007. All comments resulting from the ITR have been resolved.

Joan E. Lotosky
Chief, Hydrology and Hydraulics Section
Albuquerque District
USACE

19 Dec 2007
Date

Kristopher T. Schafer
Chief, Planning Branch
Albuquerque District
USACE

12/19/07
Date

MIDDLE RIO GRANDE, BERNALILLO TO BELEN
FLOOD CONTROL STUDY
HYDROLOGIC ANALYSIS CERTIFICATION

The Albuquerque District of the U.S. Army Corps of Engineers (USACE) completed the hydrologic analysis for the Mountain View, Isleta, and Belen reach of the Rio Grande in support of the Middle Rio Grande Flood Control Study. All quality control activities and an independent technical review, appropriate to the level of risk and complexity inherent to the project, have been completed. The review verified compliance with established policies, principles and procedures, using justified and valid assumptions. Verification included review of analysis assumptions; methods, procedures, and material used in the analysis; the appropriateness of data used; and reasonableness of the results. The independent technical review was performed and completed 1 December 2006 by Robert Collins, senior hydrologist in the Water Management Branch in the Sacramento District.

Joan E. Lotosky
Chief, Hydrology and Hydraulics Section
Albuquerque District

6 December 2006
Date

Kristopher T. Schafer
Chief, Planning Branch
Albuquerque District

12/6/06
Date

WATER MANAGEMENT SECTION
CERTIFICATION FOR INDEPENDENT TECHNICAL REVIEW

Middle Rio Grande Flood Control Project, New Mexico
Mountain View, Isleta and Belen Units
Hydrology, Albuquerque District
December 1, 2006

GENERAL FINDINGS

Compliance with clearly established policy, principles, and procedures, utilizing clearly justified and valid assumptions, has been verified for the subject project. This includes assumptions; methods, procedures and materials used in the analyses; the appropriateness of data used and level of data obtained; and the reasonableness of the results, including whether the product meets the customers' needs consistent with law and existing Corps criteria and policy.

I certify that an independent technical review of the project indicated above has been completed and all technical issues have been identified and resolved. I recommend certification that the quality control process has been completed.

In accordance with CESPD R 1110-1-8, South Pacific Division Quality Management Plan, May 2000, this letter certifies that this hydrology is appropriate as the basis for with- and without-project floodplains flows in the Middle Rio Grande Flood Control Project, New Mexico Mountain View, Isleta and Belen Units.

Robert F. Collins, District Hydrologist, P.H.

12/1/2006
Date

MIDDLE RIO GRANDE FLOOD CONTROL STUDY
HYDRAULIC ANALYSES

COMPLETION OF AGENCY TECHNICAL REVIEW
AND CERTIFICATION

WITH PROJECT HYDRAULICS (LEVEE AND RISK & UNCERTAINTY)

The Albuquerque District of the U.S. Army Corps of Engineers (USACE) has completed the hydraulic analysis for the Middle Rio Grande Flood Control Study. This hydraulics assessment included evaluating the preferred levee alternative and completing the associated risk and uncertainty analysis. Notice is hereby given that an agency technical review, that is appropriate to the level of risk and complexity inherent to the project, has been conducted as defined in the Quality Control Plan. During the agency technical review, compliance with established policy principles and procedures was verified. This included review of assumptions, methods, procedures, and material used in the analysis, the appropriateness of the data used and the level obtained, and reasonableness of the results, including whether the product meets the customer's needs consistent with law and existing USACE policy. The agency technical review was accomplished by Ethan Thompson, Senior Hydraulic Engineer in the Hydraulic Design Section of the Sacramento District on 8 March 2010. All comments resulting from the ATR have been resolved.

Tamara M. Massong
Chief, Hydrology and Hydraulics Section
Albuquerque District
USACE

12 MAR 2010
Date

Kristopher T. Schafer
Chief, Planning Branch
Albuquerque District
USACE

3/12/10
Date

CERTIFICATION OF AGENCY TECHNICAL REVIEW

Project Title: Middle Rio Grande Flood Control Project, New Mexico
Mountain View, Isleta and Belen Units
Product: Hydrology & Hydraulics Report. 8 March 2010

I certify that an agency technical review of the subject study report has been completed. The review was completed only for the hydraulic related analyses. The hydrology was reviewed separately. All comments and technical issues have been identified and have been adequately addressed and resolved. Changes have been incorporated in the final report.

3/8/10
Date

Ethan A. Thompson, P.E.
Senior Hydraulic Engineer
Hydraulic Design Section
Sacramento District, USACE

Attachment 2—Project Hydrology and Hydraulics Report

CORPS OF ENGINEERS

Albuquerque District

EXHIBIT A

Middle Rio Grande Flood Control Project, New Mexico

Mountain View, Isleta and Belen Units

Hydrology & Hydraulics

January 2015

Rio Grande Floodway
Bernalillo to Belen, Mt. View - Isleta to Belen Unit
Bernalillo and Valencia Counties, NM
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1.0. Introduction
The Middle Rio Grande Flood Control Project was authorized in the Water Resources Development Act of 1986. The current feasibility study for this project will evaluate flood control alternatives for the Rio Grande in the project area which includes the three southern units (Mountain View, Isleta, and Belen) located in Bernalillo and Valencia counties, New Mexico.

1.1. Scope of Hydrology and Hydraulics (H&H) Study. An evaluation of hydrology and hydraulics provides an estimate of potential for flooding in a given area. The flood peaks, volume and duration that can be expected, along with the corresponding river depths and velocities, are part of the information that these analyses provide. Sediment movement is a significant factor in the Rio Grande, and so that is included.

Hydrology and hydraulics have been addressed in past studies. However, there are now both new data and improved analytic techniques available for use in analyzing the H&H. Given the present capability, it was decided to revise the hydrology, and also the hydraulic and sediment analyses.

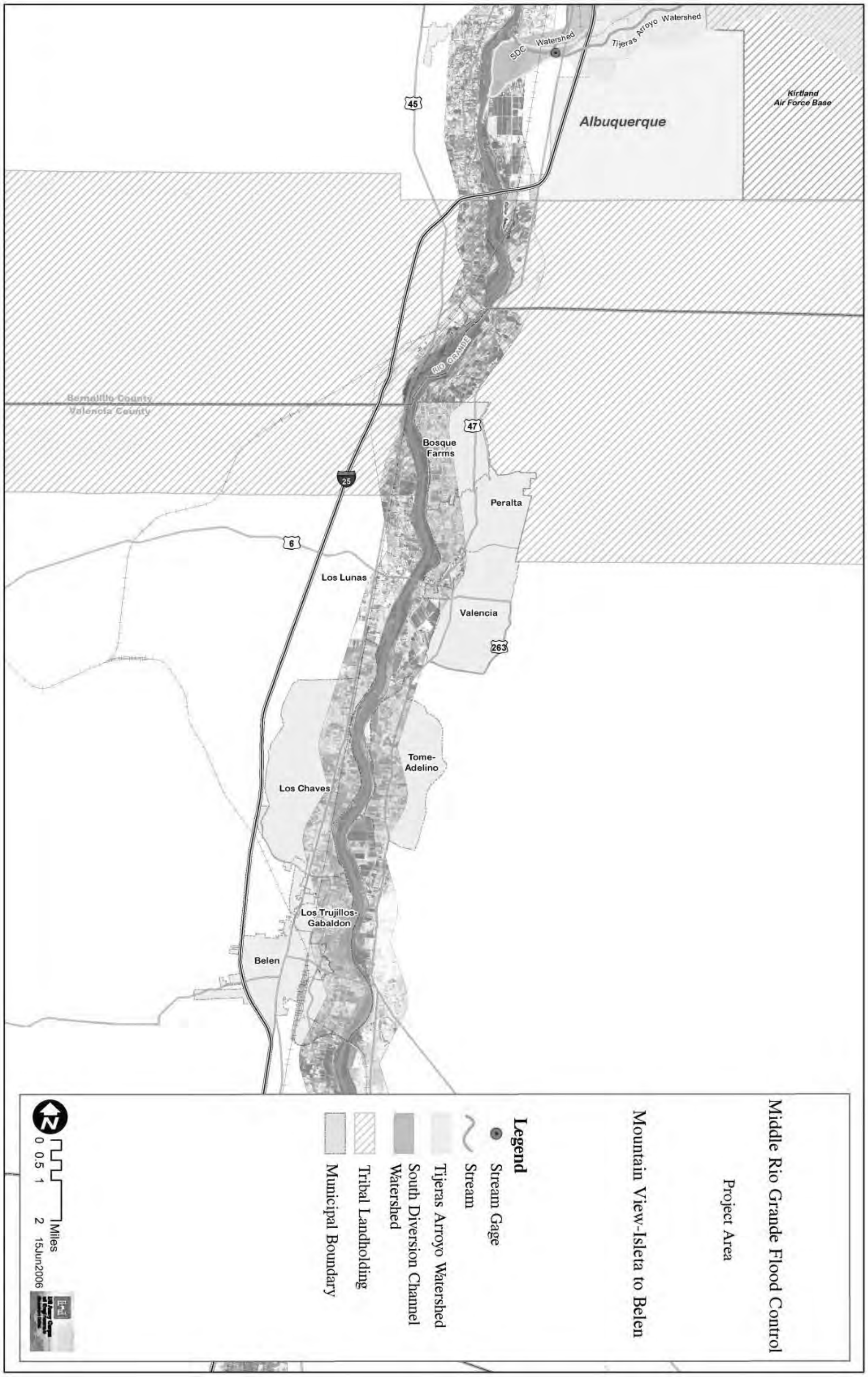
The H&H analysis addresses both with- and without-project conditions. The differences between them allow an evaluation of the benefits of providing proposed project features. The with-project analysis includes the details of the proposed design, so that specific design features can be evaluated.

Both with- and without-project evaluations include present conditions and future conditions (with projected sedimentation). Future conditions are estimated to be 50 years from present.

1.2. Project Area. The project area begins in the Rio Grande floodway just south of the Albuquerque city limits, at the South Diversion Channel. It extends downstream for 35 river miles. The downstream limit of the project area is the Rio Grande floodway 5 river miles south of the Highway 47 Bridge at Belen. The study area includes several small rural communities on both sides of the Rio Grande between Albuquerque and Belen, most of which are unincorporated. Pajarito, Los Padillas, Isleta Pueblo, Los Lentes, Los Lunas, Los Chavez, Belen, Bacaville, Jarales, and Pueblitos are located on the west bank of the Rio Grande; Mountain View, Bosque Farms, Peralta, Valencia, Tome, Adelino, La Constancia, and Madrone are located on the east bank of the Rio Grande.

Figure 1 shows the project area and Figure 2 shows the watershed upstream of the project area, including major subwatersheds. For more detailed mapping of the project area showing significant project features see MRG Map Books at the end of this section.

Figure 1 - Project Area



2.0. Previous Hydrology, Hydraulic and Sediment Analyses

The flooding problems along the Middle Rio Grande between Bernalillo and Belen, New Mexico are documented in a 1979 feasibility report (USACE 1979a). In that report, seven reaches or 'units' were evaluated. These included Bernalillo, Corrales, Albuquerque, Isleta, Mountain View, and Belen East and West units. The 1979 feasibility report recommended no action for the Albuquerque Unit since the existing levees were in good condition and provided flood protection up to and including 42,000 cfs and no action for the Bernalillo Unit because it was not economically justified. Levees to reduce the flood threat for the other five units were found to be economically justified and authorized for construction in the Water Resources Development Act of 1986. A General Design Memorandum (GDM) for these five units was also completed in 1986 (USACE 1986). In this document, it was determined that the Mountain View and Isleta Units were no longer economically justified and were dropped at the completion of the GDM from further design or analysis. A detailed Interior Drainage Analysis was also included as part of the 1986 GDM. Because the Middle Rio Grande Conservancy District (MRGCD), sponsor for these projects, had limited financial capability, design activities for the Corrales Unit continued, with the Belen East and West units being placed on hold until the Corrales Unit was completed. A Limited Reevaluation Report (LRR) for the Corrales Unit was completed in 1994. Construction of the Corrales Unit began in 1996 and was completed in 1997.

During the course of the Limited Reevaluation Report study for the Belen East and West units, several events occurred that impacted the study and have resulted in expanding the scope of the Belen Limited Reevaluation Report study. The following paragraphs summarize these events.

- Endangered Species Status: In 1994 the Southwestern Willow Flycatcher and the Rio Grande Silvery Minnow, both of which inhabit the study area, were added to the Federal Endangered Species List under the provisions of the Endangered Species Act. Additionally, the study reach is within designated critical habitat for both species As a result, the United States Fish and Wildlife Service has repeatedly requested that the Corps of Engineers evaluate the impacts of Corps of Engineers projects (units) on a total project basis and not by individual units.

- Rio Grande Basin Hydrologic Computations: In 1994 and 1995, while conducting the Espanola, New Mexico, feasibility investigations, a revised method of computing the hydrologic outputs of watersheds within the middle Rio Grande valley was developed. This method identifies not only the impacts due to peak flood discharges, but also the impacts due to long duration, spring runoff flood events. As a result, alternatives under consideration in the middle valley must include features to mitigate against both these water resource impacts. Because the authorized plan essentially addressed only impacts due to peak flood discharges, additional analyses and design are now required.

- Sponsor Financial Capabilities: In 1998, as separate limited reevaluation studies were being finalized for the Belen East and West units, and the San Acacia to Bosque del Apache, Rio Grande Floodway projects, the MRGCD made a decision to proceed with the San Acacia project before the Belen project. This decision was largely due to their financial capabilities. It therefore became necessary to focus on the completion of the San Acacia GRR and delay the completion of the GRR studies for the Belen units.

- Cost Analysis: The authorized project included five units (Corrales, Mountain View, Isleta, Belen East and Belen West). While the General Design Memorandum was being completed for the project in 1986, the Mountain View and Isleta Units were dropped from further consideration because changes in cost rendered them economically infeasible. The benefit/cost ratios dropped to 0.7:1 for Mountain View and 0.9:1 for Isleta West. With the completion of the construction of the Corrales Unit in 1997, and analyzing the actual costs of the Corrales Unit, the local sponsor believes that the Mountain View and

Isleta Units would be economically justified. The District also believes this to be true. Preliminary analysis indicates that both the Mountain View and Isleta Units would be economically justified. The Middle Rio Grande Conservancy District has requested by a letter dated December 3, 1997 that the Albuquerque District undertake studies to include the Mountain View and Isleta Units as originally authorized.

As discussed earlier, the Albuquerque District of the Corps of Engineers has previously studied flood protection for the project area. The hydrologic and hydraulic analyses that have been performed, and the sedimentation issues analyzed, have used the most up-to-date hydraulic models that were available, whether HEC-2 or HEC-RAS.

Sediment is a topic of great relevance to the project area. Reports on this topic include USACE (1948) and RTI (1993). Additionally, the USACE Upper Rio Grande Water Operations Project (URGWOP) is presently evaluating sediment in the project area as it relates to reservoir operations.

3.0 Hydrology

3.1. Purpose of Hydrology Study

The purpose of the hydrology study is to estimate frequency flows in the Rio Grande through the project area. Some of the applications are:

- The hydrology will be used to evaluate the potential of flooding with and without a proposed flood control levee in the project area.
- The hydrology will be used as the basis for with- and without-project floodplains.
- Economic benefits for flood frequency events will be estimated based on the hydrology.

Figure 2 is a map that shows the Albuquerque watershed together with major subwatersheds. Figure 3 shows the annual average precipitation in New Mexico.

The Hydrologic Analysis presented herein has undergone independent technical review (ITR) and has been certified by the Albuquerque District U.S. Army Corps of Engineers. Copies of the ITR Certification are included as Attachment 1 of Appendix H.

Figure 2 - Albuquerque Watershed

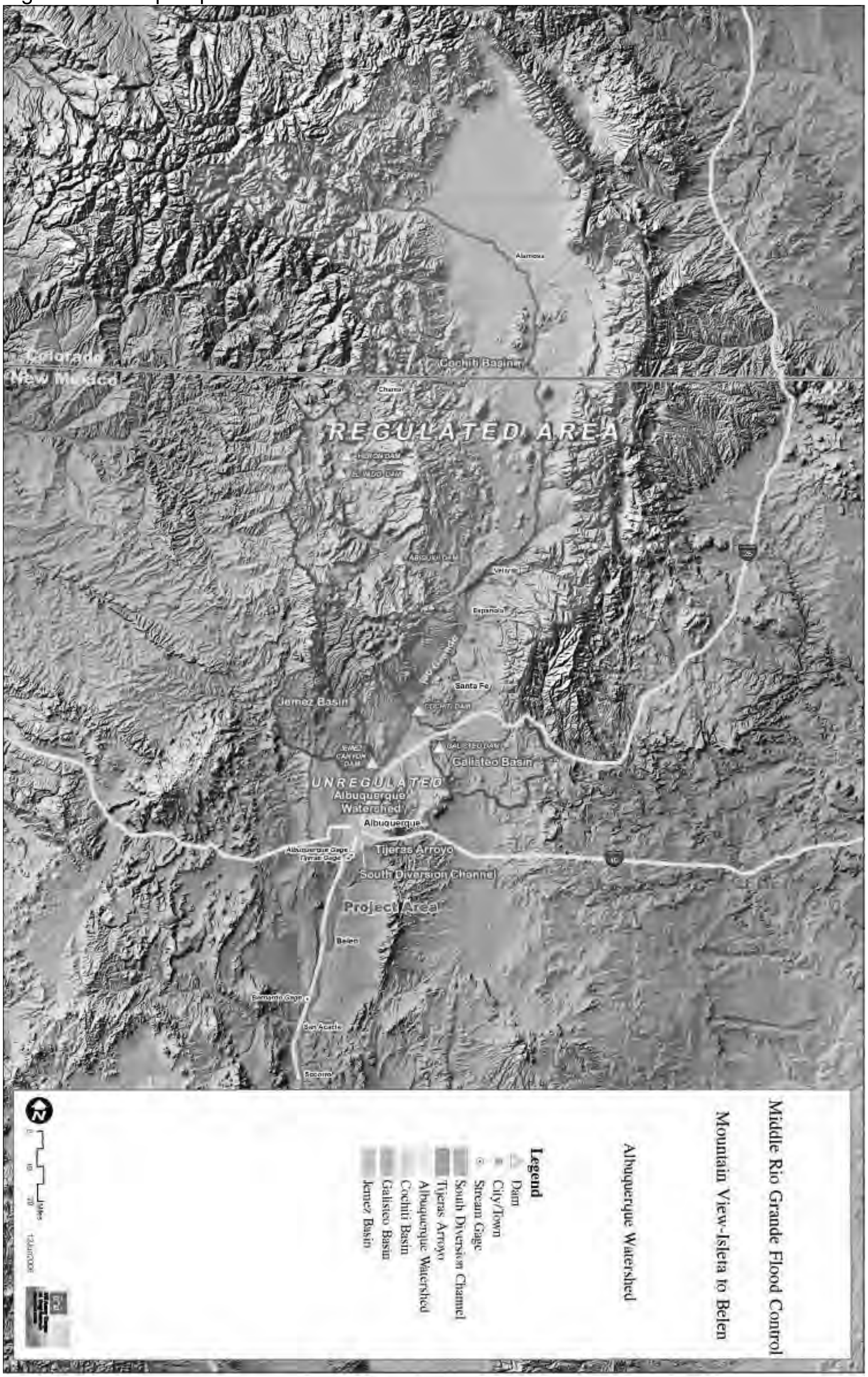
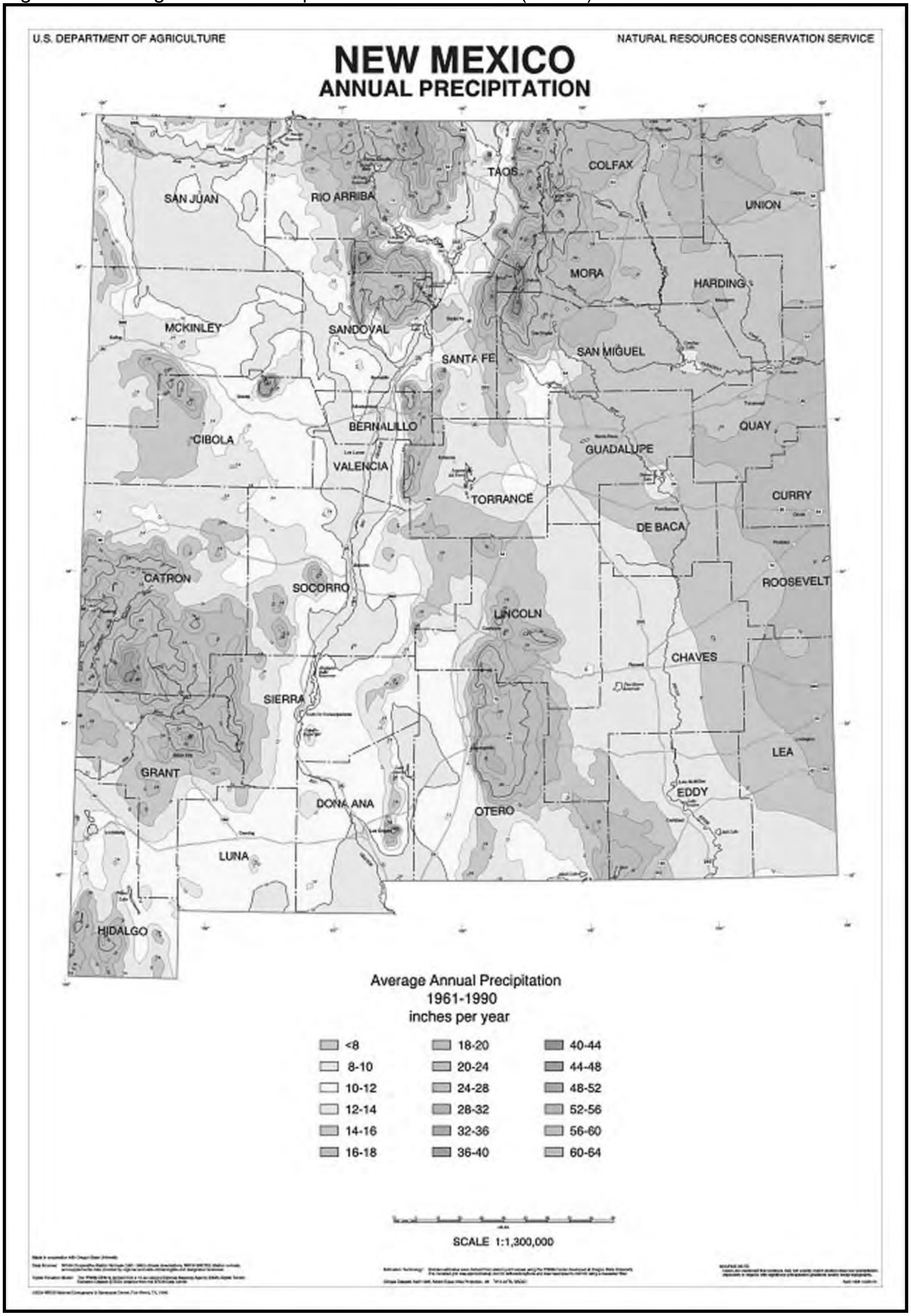


Figure 3 - Average Annual Precipitation in New Mexico (NRCS)



3.2. Overview of Hydrology in the Project Area

3.2.1. Hydrology at Albuquerque

The Rio Grande watershed upstream of Albuquerque is comprised of 17,440 square miles, of which 16,535 square miles is regulated by dams.

- Cochiti Dam is 48 river miles upstream of Albuquerque at the gage, and it directly regulates the Rio Grande upstream of the Albuquerque gage. The area of the Cochiti Dam watershed is 14,900 square miles. The dam is located on the Cochiti Pueblo in Sandoval County, NM.
- The Jemez Canyon Dam, also in Sandoval County, NM, controls 1034 square miles of the Jemez River watershed. The Jemez River confluence with the Rio Grande is 25 river miles upstream of the Albuquerque gage. The Jemez Canyon Dam is presently operated as a dry dam, though it has had a permanent pool for approximately half of the 44 years it has been in service.
- Galisteo Dam is a dry dam that controls 600 square miles of the Galisteo Creek watershed. Galisteo Creek enters the Rio Grande 46.5 miles upstream of Albuquerque.

Downstream of these structures, another 900 square miles are unregulated and also contribute directly to flooding in the Rio Grande floodway in Albuquerque.

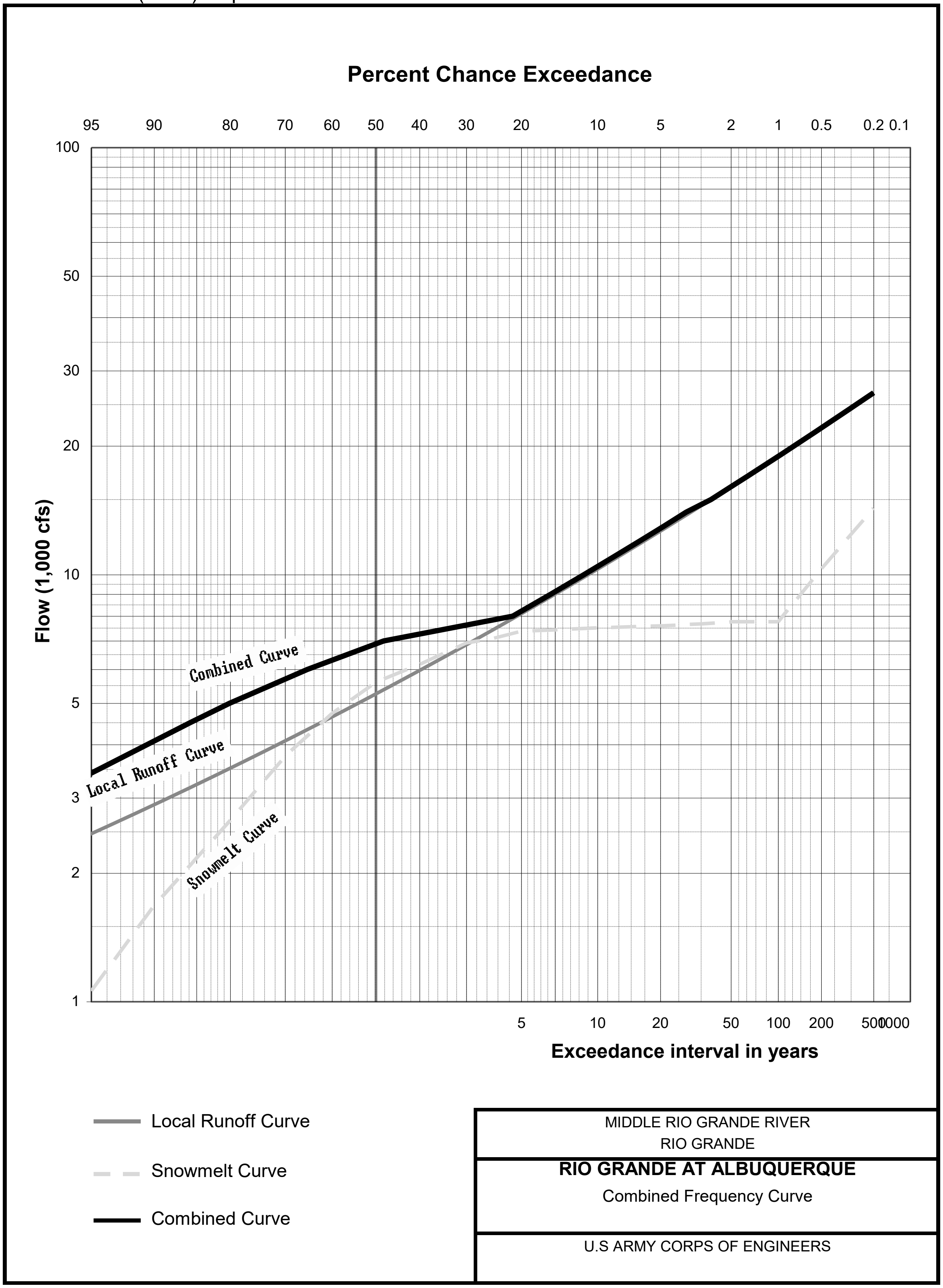
HEC (2006) is a study of flood frequencies for the Rio Grande in Albuquerque. The location in Albuquerque for this hydrology is the Rio Grande at the Central Avenue Bridge, where the Albuquerque gage is located. The HEC Middle Rio Grande flow frequency is a combined frequency based on

- Regulated flood flows from the reservoirs upstream of Albuquerque, predominantly snowmelt floods, and
- Flood flows from unregulated local areas downstream of the reservoirs, primarily from rainfall runoff.

This study was done in 2006 by the Hydrologic Engineering Center (HEC), and the results were used to develop the Rio Grande hydrology in the project area. Figure 4 is the combined frequency curve for the Rio Grande at the Albuquerque gage.

The Albuquerque levee was designed prior to Cochiti Dam being constructed. The design flow for the Albuquerque levee was 42,000 cfs. The present day probability of a flow of 42,000 cfs is significantly different than it was before the dams were put into operation. The probability of a flood flow of 42,000 cfs was determined by extrapolating it from the combined frequency curve. It is 0.000168, and the return period is 5,950 years.

Figure 4 - Combined Peak Flow Frequency Curve for the Rio Grande at the Albuquerque Gage, from the HEC (2006) Report



3.2.2. Summary of Hydrologic Analysis in the Project Area

There are three potential sources of Rio Grande flooding in the project area downstream of Albuquerque. They are:

- Flood events in the regulated area upstream of Albuquerque, as identified in the HEC (2006) report.
- Rainfall-runoff flood events in the unregulated area upstream of Albuquerque, as identified in the HEC (2006) report.
- Rainfall-runoff flood events from the project area, downstream of Albuquerque. A single inflow to the Rio Grande, the South Diversion Channel (SDC) south of Albuquerque, provides the only potential for significant local inflow between the Albuquerque gage and the downstream end of the project area (RTI 1985 and USACE 1986).

Snowmelt events from tributaries are not a factor downstream of Cochiti, Galisteo and Jemez Canyon Dams. The tributary areas are relatively small (typically less than 200 square miles), and all have significant variability in the range of elevations present. For example, the elevation of the Tijeras Arroyo at its downstream gage is 5000 ft. NGVD 1929 and the drainage area is 128 square miles. The elevation in the highest part of the Tijeras Arroyo watershed exceeds 9,500 feet. The snowpack that can accumulate during winter and spring months at high elevations are exposed to warmer temperatures over a period of months depending on their elevation. Rapid warming is not normally experienced over the whole range of elevations. The result for small watersheds is very slow snowmelt over a period of months that does not produce runoff of significance.

Individual flood flow frequencies are available for each of the three flooding sources. These frequencies are explained further in sections 3.2.3, 3.2.4, and 3.2.5 below. The independence of the three flooding sources is discussed in section 3.2.3.

It is necessary to evaluate both with- and without-project conditions in order to meet Corps of Engineers requirements for feasibility level evaluation. In this study, flood flows from the three sources of flooding were routed separately downstream through the project area, in order to estimate their individual contributions to flooding in the project area. There are two separate scenarios for flood routing. One represents without-project conditions, which in this case is without a flood control project. It is assumed that the spoil bank levees, which are not engineered levees, will uniformly fail throughout the area. The second scenario represents with-project conditions, which in this case is assumed to be an engineered levee high enough to contain flooding.

For each of the two flood routing scenarios, estimated flooding from each of the three sources was evaluated at selected locations. Then they could be combined to estimate the total flood flow frequency at each location for without-project and with-project conditions.

3.2.3. Independence of Flooding Sources

Although it is not necessary for these flooding sources to be independent, when they are, it simplifies application of the combined probability equation.

It was established in the HEC (2006) report that the two sources of flooding at the Albuquerque gage (flooding from regulated areas above dams and the unregulated rainfall runoff flooding) are independent of one another. In the project area, downstream of the Albuquerque gage on the Rio Grande, flow from the third source, the SDC, must also be considered.

Table 1 provides data that indicate that flooding from the SDC is independent of flooding in the Rio Grande. The record of instantaneous peak flows is given for years that have gage data for

Table 1 - Record of Instantaneous Peak Flows for the Rio Grande and the Tijeras Arroyo

Water Year	Gage 08330600 Tijeras Arroyo at the SDC		Gage 08330000 Rio Grande in Albuquerque		Difference in Dates of Peak Flow in Days
	Date of Peak	Peak Q (cfs)	Date of Peak	Peak Q (cfs)	
1953	8/12/53	1280	7/18/1953	7920	25
1954	9/26/54	1190	8/11/1954	5720	46
1955	7/27/55	2000	9/25/1955	7960	60
1956	8/18/56	1120	7/20/1956	4880	29
1957	1957	500	7/5/1957	8780	N/A
1958	1958-07	650	5/30/1958	12700	~ 45
1959	8/24/59	980	1959	No Data	N/A
1960	7/15/60	1100	4/14/1960	4800	92
1961	8/14/61	1350	8/23/1961	6770	9
1962	1962-07	750	4/22/1962	6520	~90
1963	1963-09	695	3/30/1963	2480	~ 180
1964	9/4/64	795	5/28/1964	1920	99
1965	7/25/65	825	6/19/1965	8720	36
1966	10/17/65	655	8/2/1966	6650	289
1967	6/24/67	2530	8/10/1967	13300	47
1968	7/2/68	1930	6/3/1968	4360	29
1969	No Data	No Data	6/17/1969	6480	N/A
1970	No Data	No Data	10/23/1969	5840	N/A
1971	No Data	No Data	7/27/1971	6650	N/A
1972	8/18/72	620	9/15/1972	4380	28
1973	9/9/73	380	5/14/1973	8570	118
1974	7/9/74	370	1/1/1974	2080	189
1975	8/12/75	830	5/24/1975	6160	80
1976	8/19/76	1440	5/21/1976	3340	90
1977	8/14/77	490	8/18/1977	2190	4
1978	10/6/77	780	5/24/1978	4580	230
1979	8/17/79	295	6/1/1979	8650	77
1980	8/14/80	980	8/14/1980	7600	0
1981	8/11/81	370	8/21/1981	2750	10
1982	8/1/82	740	6/2/1982	5460	60
1983	7/29/83	340	6/10/1983	7700	49
1984	8/7/84	520	5/28/1984	9500	71
1985	10/15/84	430	4/24/1985	9370	191
1986	8/25/86	550	8/11/1986	5190	14
1987	8/22/87	1140	7/24/1987	7840	29
1988	7/9/88	2930	7/9/1988	4820	0
1989	8/2/89	178	4/25/1989	3730	99
1990	7/14/90	992	9/29/1990	5610	77
1991	7/24/91	687	8/7/1991	6440	14
1992	8/11/92	724	4/29/1992	6250	104
1993	7/20/93	181	6/7/1993	7210	43
1994	7/20/94	181	5/11/1994	7050	70
1995	8/22/95	370	5/25/1995	6570	89
1996	7/28/96	583	6/27/1996	2690	31
1997	7/28/97	583	6/8/1997	6270	50
1998	7/26/98	359	5/9/1998	4060	78
1999	8/3/99	520	5/28/1999	4920	67
2000	6/3/00	297	8/20/2000	2040	78
2001	10/23/00	489	5/22/2001	4970	211
2002	8/19/02	416	9/10/2002	1770	22
2003	3/21/03	331	3/21/2003	1880	0
2004	No Data	No Data	4/3/2004	3590	N/A

both the Tijeras Arroyo gage at the SDC and the Albuquerque gage. The difference calculation between the dates of the peaks is shown in days, to establish which flows are coincident.

The frequencies of the flooding events given above were established graphically, in order to check the significance of coincident flows. The combined frequency curve from the 2006 HEC report for the Rio Grande at the Albuquerque gage, shown previously in Figure 4, was used to estimate frequency of Albuquerque flows. A graph of the USGS flood frequency analysis for the Tijeras Arroyo gage at the SDC, shown in Figure 5, was used to estimate the frequency of floods on the Tijeras Arroyo. More information about flood frequency studies for the Tijeras Arroyo is given in Section 3.5.2.

The 50-year record displayed in Table 1 indicates that there are only three dates on which instantaneous peak flows have occurred on both the Rio Grande at Albuquerque gage and the Tijeras Arroyo gage.

- 8/14/1980. The flow on the Rio Grande peaked at 7600 cfs, approximately equivalent to a 25% chance flood event. The flow from the Tijeras Arroyo peaked at 980 cfs, approximately the 40% chance flood event.
- 7/9/1988. The flow on the Rio Grande peaked at 4820 cfs, less than the 50% chance flood event. The flow from the Tijeras Arroyo peaked at 2930 cfs, approximately the 1.5% chance flood event.
- 3/21/2003. The flow on the Rio Grande peaked at 1880 cfs, less than the 50% chance flood event. The flow from the Tijeras Arroyo peaked at 331 cfs, also less than the 50% chance flood event.

Of these coincident flows, only one from the Tijeras Arroyo is greater than the 20% chance flood event and it coincides with a flow that is less than the 50% chance event on the Rio Grande. None of the coincident flows on the Rio Grande is greater than a 20% chance flood event; therefore coincident flows are not a significant factor in serious flood events. It follows that flooding from the Tijeras Arroyo can be considered to be independent of flooding in the Rio Grande.

3.3. Flood Events from Regulated Areas

The HEC (2006) report includes a frequency analysis for floods from regulated areas contributing to Rio Grande flooding at the Albuquerque gage. These are predominantly snowmelt flood events. Table 2 includes flood peaks associated with the frequency flood events. Hydrographs for these floods are plotted in Figure 6. These hydrographs were provided to the Albuquerque District by the HEC for use in flood routing models.

3.4. Flood Flows from Unregulated Areas - Albuquerque and Upstream

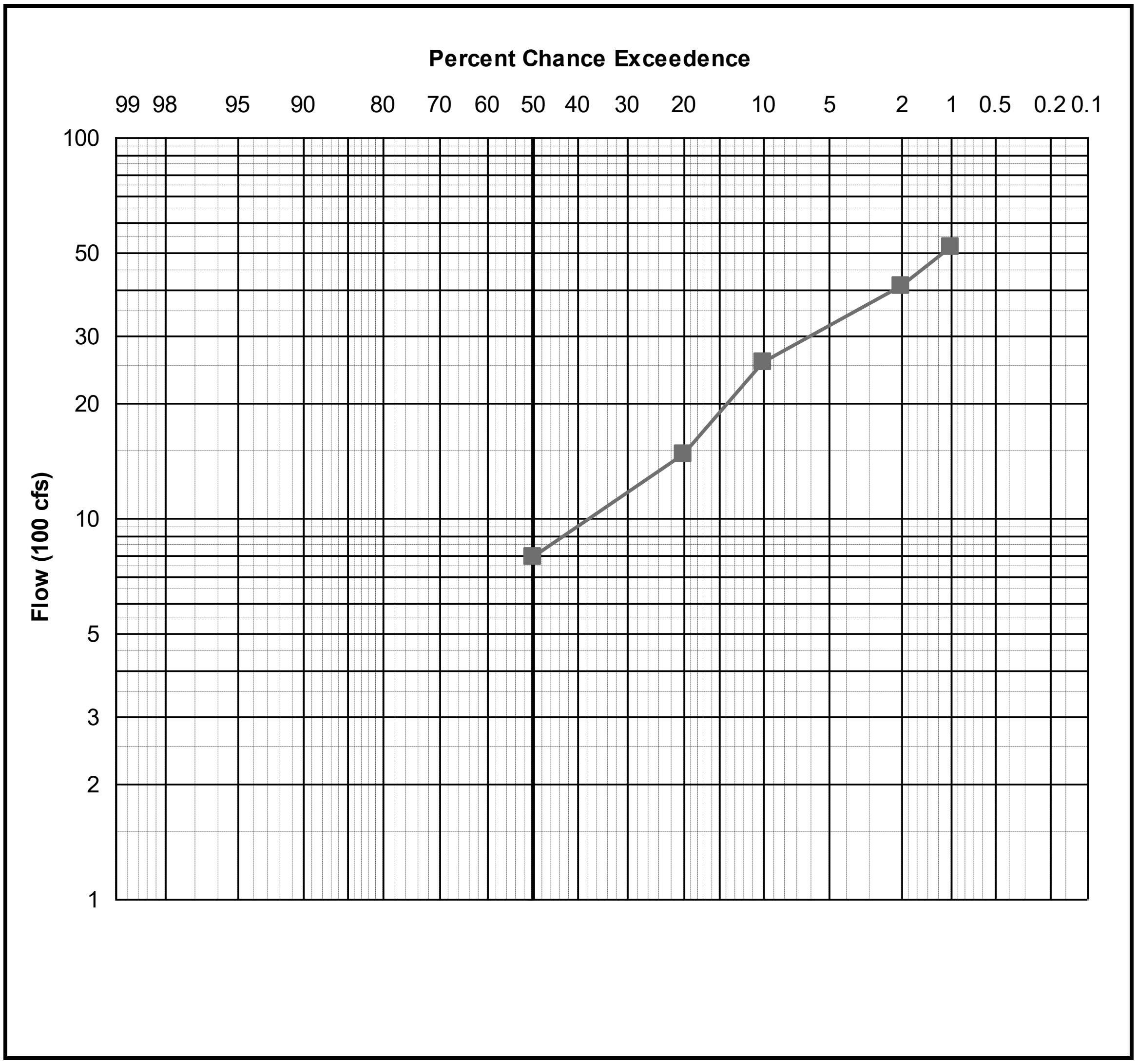
The HEC (2006) hydrology for Albuquerque includes a frequency analysis for floods from unregulated areas contributing to Rio Grande flooding at the Albuquerque gage. These floods are associated with rainfall runoff events. Table 2 includes flood peaks from unregulated areas associated with the frequency flood events. Figure 7 is the HEC dimensionless hydrograph for floods from unregulated areas upstream of Albuquerque. It was provided to the Albuquerque District by the HEC together with the HEC (2006) report.

3.5. Flood Flows from Rainfall Runoff Events in the Project Area

3.5.1. Watershed Characteristics of the South Diversion Channel

The South Diversion Channel (SDC) is the only major arroyo that directly enters the Rio Grande between the Albuquerque gage at Central Avenue and the downstream study limit. Flow from other tributaries to the Rio Grande through the study area is blocked by levees and does not reach the river.

Figure 5. USGS Flood Flow Frequency for Gage 08330600, Tijeras Arroyo near Albuquerque, Located at its Confluence with the SDC



The SDC enters the Rio Grande from the east immediately south of Albuquerque. The SDC includes 2 separate drainage areas.

- The South Diversion Channel watershed proper is 11 square miles, and is not of adequate size to affect the hydrology of the river.
- The Tijeras Arroyo drainage area is 128 square miles.

A map of the SDC and Tijeras Arroyo watersheds is provided in Figure 8.

The Tijeras Arroyo watershed is a fan-shaped area lying to the southeast of Albuquerque. The arroyo is 28 miles in length.

The upstream portion of the Tijeras Arroyo begins on the east side of the mountains, follows the fault line between the Sandia and Manzano mountains and emerges as a canyon onto a broad, sloping plain, or mesa, near Kirtland Air Force Base (KAFB). Mountainous uplands comprise approximately 60% of the area of the Tijeras Arroyo watershed. The channel between the upstream border of KAFB to the upstream limit of the Village of Tijeras is 11.6 miles in length and its slope is approximately .016.

Table 2 - Peak Flood Flows for the Rio Grande Gage in Albuquerque (at Central Avenue) both from Upstream Regulated Areas and Upstream Unregulated Areas

Return Period (Years)	% Chance Exceedance	Flood Events from Regulated Areas-Peak Flows in cfs	Flood Events from Unregulated Areas-Peak Flows in cfs
2	50	5600	5260
5	20	7380	8100
10	10	7510	10300
50	2	7750	16100
100	1	7750	18900
200	0.5	10300	22100
500	0.2	14300	26700

Figure 6. Hydrographs at the Albuquerque Gage for Flood Events from Regulated Areas Upstream of Albuquerque (Provided to the Albuquerque District by the HEC as Supplemental Information to the HEC (2006) Report)

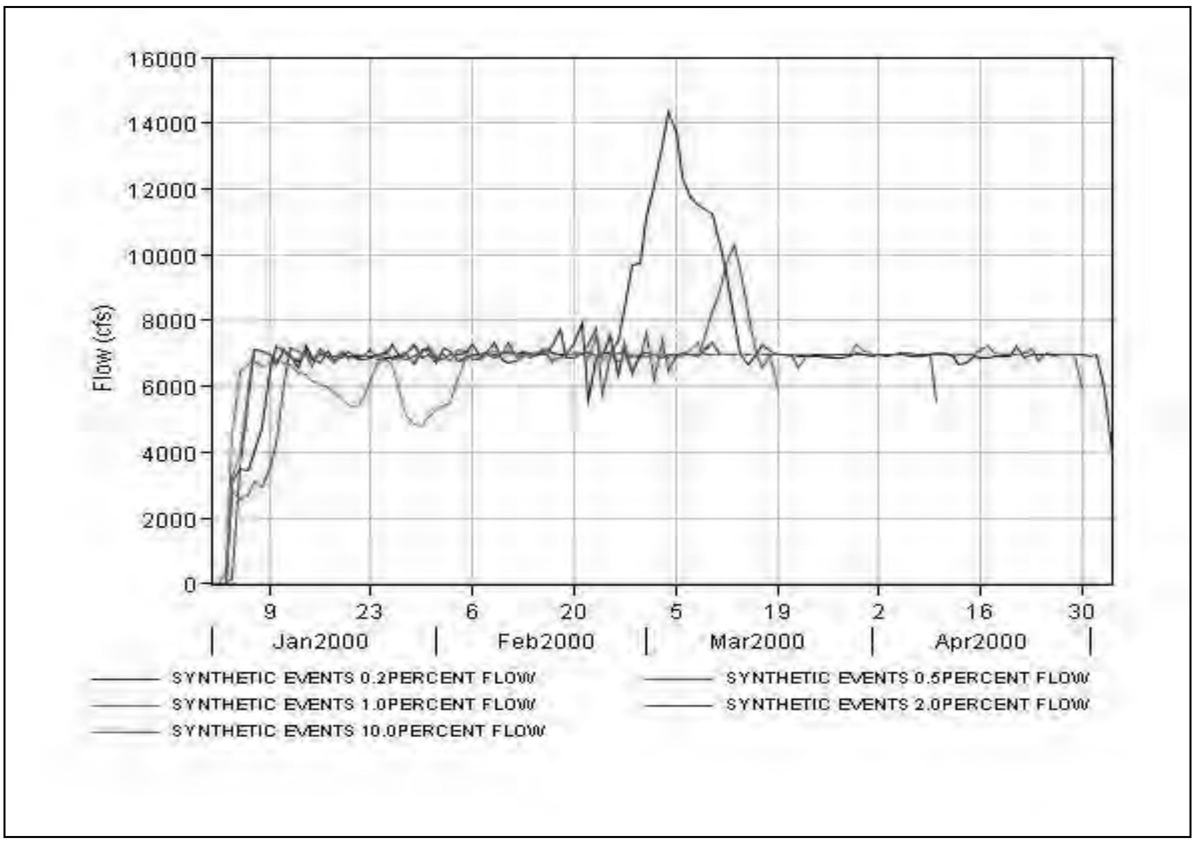


Figure 7. Dimensionless Hydrograph at the Albuquerque Gage for Flood Events from Unregulated Areas Upstream of Albuquerque (Provided to the Albuquerque District by the HEC as Supplemental Information to the HEC 2006 Report)

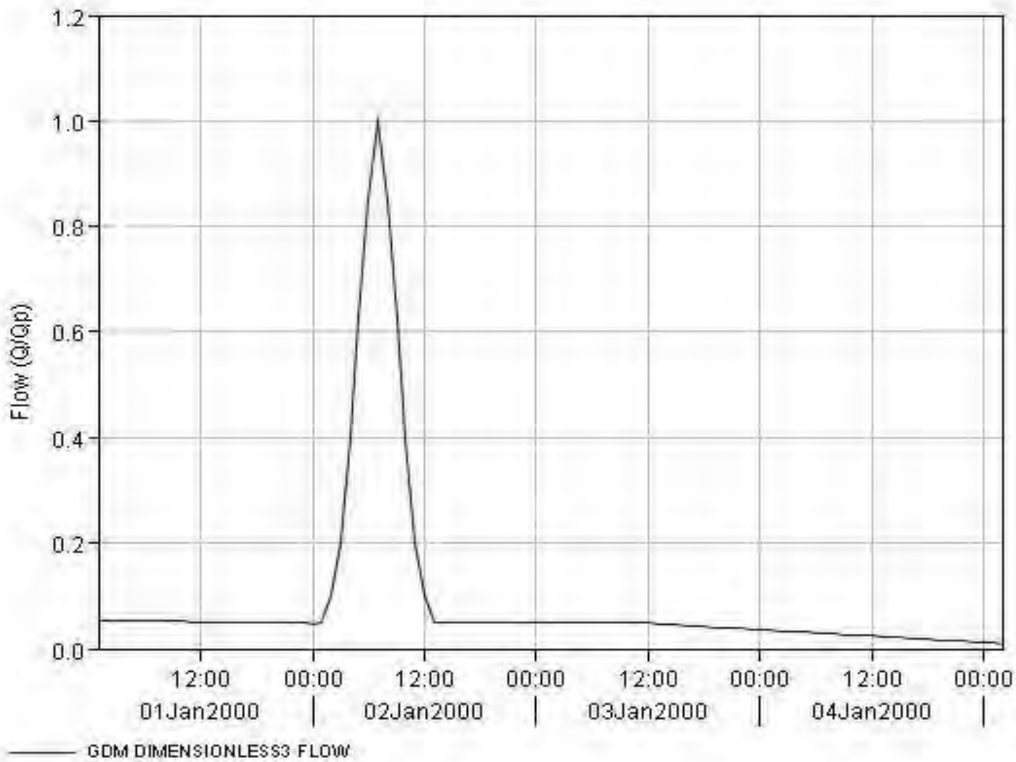
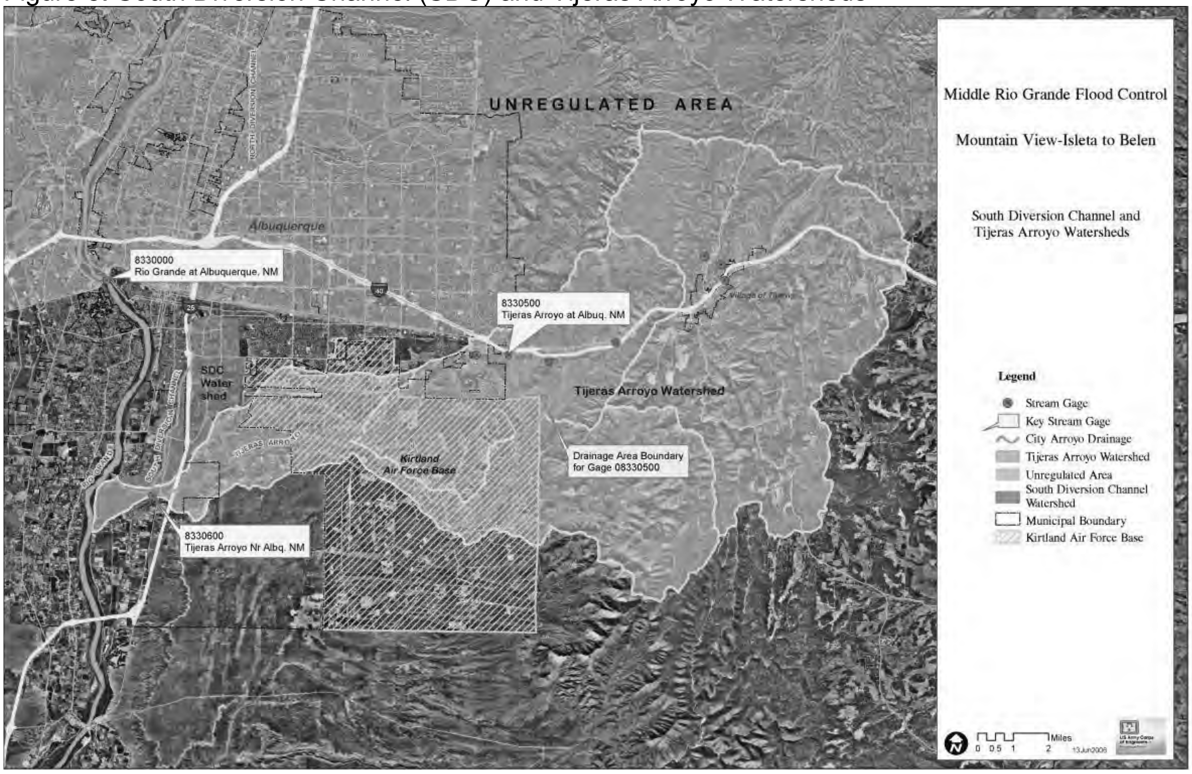


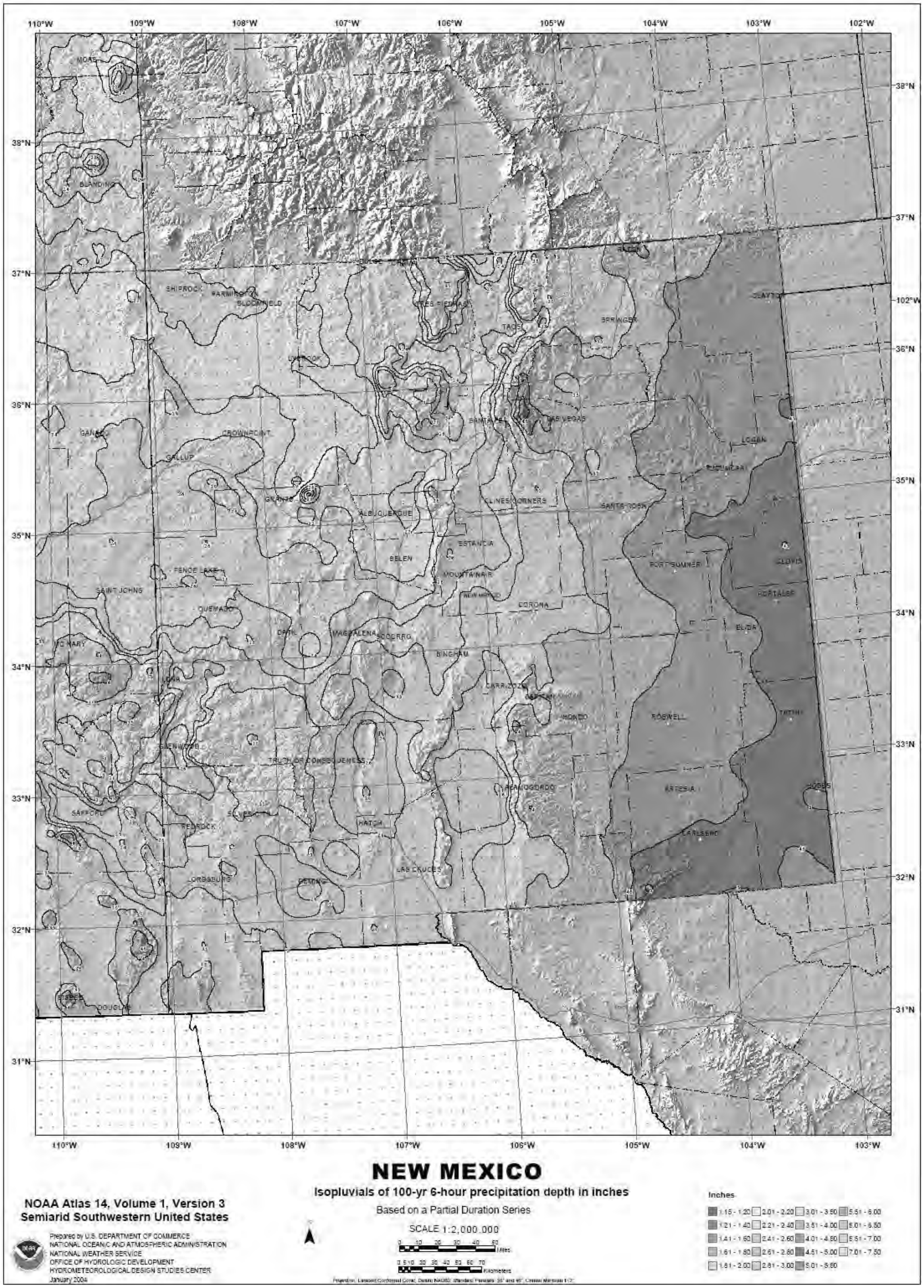
Figure 8. South Diversion Channel (SDC) and Tijeras Arroyo Watersheds



Approximately 13 stream miles are located on the mesa. The mesa is considerably flatter than the reach upstream, and the average slope of the channel is .009. The Tijeras Arroyo valley is generally about 2,000 feet in width, with a channel that is deep and narrow in some reaches and wide, shallow, and poorly defined in others.

Figure 9 shows the 100-year 6-hour isopluvials for New Mexico from NOAA Atlas 14. The Tijeras Arroyo watershed is predominantly semiarid. Average annual precipitation varies from 7 inches at the lowest elevations (4930 ft.) to 18.1 inches at Sandia Park (elevation 7000 ft.) Approximately half of the precipitation occurs from July to September in the form of brief but intense thunderstorms. It is these thunderstorms that produce peak local flows, and are the storms of interest for the SDC hydrology.

Figure 9. Isopluvials of the 100-Year 6-Hour Precipitation Depth in Inches for New Mexico, from NOAA Atlas 14

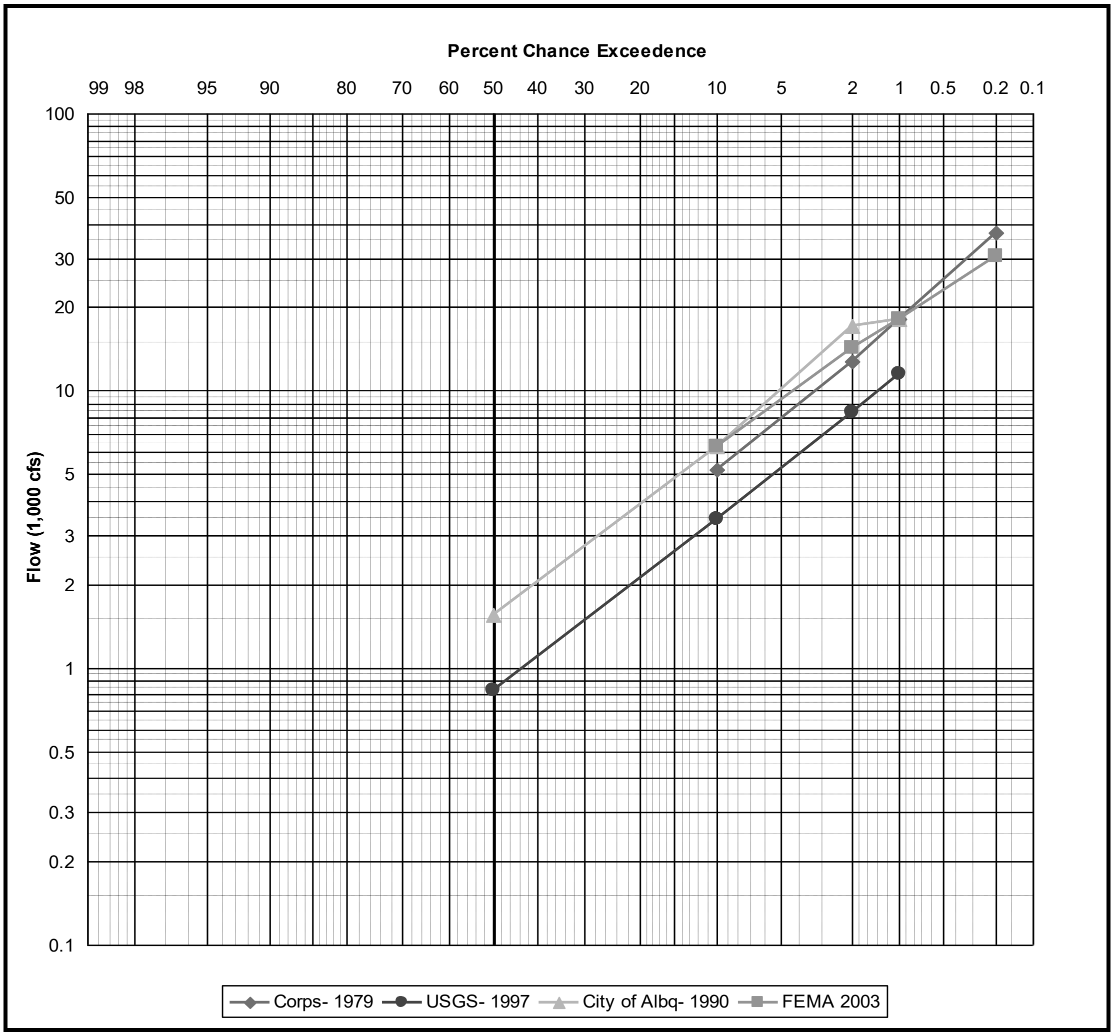


3.5.2. Previous Hydrologic Studies for the Tijeras Arroyo

Previous hydrologic studies that have been performed for the Tijeras Arroyo include USACE (1956a), USACE (1956b), USACE (1979b), Bovay (1981), Leedshill-Herkenoff (1987), Leedshill-Herkenoff (1990), USGS (1997), and FEMA (2003).

These hydrologic studies provide estimates for flood magnitudes based on several analytic methods. The USGS study is a flood frequency analysis based on gage data. The others use numeric hydrologic models. The studies have produced a range of results, though the hydrologic model results have been similar. Figure 10 shows frequencies that have been derived for the Tijeras Arroyo from previous studies.

Figure 10. Peak Flood Frequencies for Tijeras Arroyo at Gage 08330500, Tijeras Arroyo North of Kirtland AFB, from Four Agencies



3.5.3. Factors in the Tijeras Arroyo Hydrology

Floods in the Tijeras Arroyo typically produce flood hydrographs that peak quickly and are low in volume. As a flood moves through the watershed, marked attenuation of the peak occurs. Based on gage data, relatively high peaks come from the mountainous upstream area and attenuate on the mesa before reaching the confluence of the Tijeras Arroyo with the SDC.

The 26-mile channel is unlined, other than a mile of the Tijeras Arroyo on the mesa that is concrete lined. Even so, channel losses through infiltration are not likely to be a major factor. Based on the limited data that is available for peak and one-day flood flows, flood flows are of short duration. Infiltration is time dependent. The time that is needed for infiltration to occur in the 26-mile channel is a limiting factor.

Streams in the Albuquerque area generally produce large quantities of sediment. Several of the studies listed in Section 5.2 provide information about sediment in the Tijeras Arroyo watershed, as well as the hydrology of the watershed.

- DM #1 (USACE 1956a) states that the bed load coming from the Tijeras Arroyo at its confluence with the SDC is predominantly sand.
- According to DM #2 (USACE 1956b), 36 acre-feet of sediment comes from the Tijeras Arroyo annually.
- RTI (1993) computed an average annual sediment load of 44.2 acre-feet for Tijeras Arroyo.

3.5.4. Methods of Evaluating the SDC/ Tijeras Arroyo Hydrology

Hydrology of watersheds is often evaluated either with hydrologic models or by using frequency analysis. For the Albuquerque hydrology, the HEC evaluated the hydrology of local inflows between river gages as a group, based on river gage data. The SDC is the only local flow of significance, but even so its hydrology cannot be analyzed using that methodology. The nearest downstream gage with a meaningful period of record is located at Bernardo, 47 miles downstream from the confluence of the SDC with the Rio Grande. Significant flow attenuation is a factor in the reach, and prevents that data from being useful to analyze the SDC hydrology.

3.5.5. Flood Frequency Analyses

As many as 9 stream gages have been in operation at various times in the Tijeras Arroyo watershed. Two of the gages in the watershed have records of instantaneous peaks that are adequate for flood frequency analyses. These are gages 08330500 and 08330600.

The gage at the downstream end of Tijeras Arroyo and above its confluence with the SDC (gage 08330600) is 'Tijeras Arroyo near Albuquerque, NM'. A second gage within the Tijeras Arroyo watershed (gage 08330500, discontinued) is 'Tijeras Arroyo at Albuquerque, NM'. Because the gage names are so similar, the gage numbers are used for reference in this report.

The drainage area of Tijeras Arroyo immediately above its confluence with the SDC (gage 08330600), is 128 square miles. It has 49 years of instantaneous peaks, as of January, 2006. The average elevation for the channel is 5,930 ft. At this stream gage, the 24-hour precipitation for the 10% chance storm event is 1.72 inches, based on point precipitation frequency estimate from NOAA Atlas 14. Because of flow losses and attenuation, this peak is less than the peak recorded flow 6 miles upstream. Based on approximately 50 years of gage records, the maximum recorded flow is 2,930 cfs. In its 1997 report, the USGS estimated the 1% chance flood flow to be 5,140 cfs.

The drainage area of gage 08330500 is 75 square miles. It has 51 years of recorded instantaneous peaks, as of January, 2006. The gage is located 6 miles upstream of the confluence of the Tijeras Arroyo and the SDC, at the mouth of Tijeras Canyon, where it captures runoff from the upstream mountainous areas. The average elevation for the channel is 6,380 ft. In close proximity to gage 08330500, at the Albuquerque Sunport (airport), the 24-hour precipitation for the 10% chance storm event is 1.74 inches, based on the point precipitation frequency estimate from NOAA Atlas 14. The maximum recorded flow at gage 08330500, based on approximately 50 years of gage records, is plotted in Figure 10, together with other agency flood frequency estimates for the same location. The 1% chance flood flow estimate from the 1997 USGS analysis is 11,500 cfs.

3.5.6. Hydrologic Models from Previous Studies

In order to estimate frequency floods at the mouth of Tijeras Arroyo, HEC-1 models were used for the USACE (1979b) study and for the Leedshill-Herknehoff (1990) study. The FEMA (2003) study used the Arid Lands Hydrology Model (AHYMO) model. The USACE (1979b) study covers a smaller drainage area than the others, stopping upstream of the confluence of Tijeras Arroyo with the SDC at the KAFB boundary. The study results compare well with each other for similar drainage areas. Table 3 provides results of these results. Major differences are:

- Inconsistencies are found in the two studies that extend to the mouth of Tijeras Arroyo at that location. Typically, estimated flows resulting from hydrologic models increase in the downstream direction, as do the model results from the 1990 City of Albuquerque study. The FEMA (2003) study resulted in lower peak flows downstream at the confluence with the SDC than the estimated peak flows 6 miles upstream at gage 08330600. The reduction in flood peaks can be justified based on gage data.
- Dissimilarities are found in the model results for the 0.2% chance floods.

3.5.7. Flood Frequency for the Tijeras Arroyo at its Confluence with the SDC

Because the hydrologic model results are reasonably consistent, it was concluded that a composite of the model results could be used to estimate Tijeras Arroyo flows entering the Rio Grande. The purpose of this study is to evaluate flood reduction alternatives for the Rio Grande. Table 4 shows peak flow frequency values for the Tijeras Arroyo that were used for this study. Figure 11 is a graph of the flood frequency adopted for use in this study.

3.5.8. Volume of Flooding on the Tijeras Arroyo

The Leedshill-Herknehoff (1990) analysis provided flood hydrographs as well as peak flows. These are 6-hour hydrographs that correspond with the 6-hour design storm. The hydrographs are shown in Figure 12 for several frequencies.

Table 3 - Hydrologic Model Results from Studies on the Tijeras Arroyo

Location	Flow on the Tijeras Arroyo in cfs				
	North of Kirtland AFB			Mouth of Tijeras Arroyo	
Recurrence Interval	Corps of Engineers (1979)	City of Albuquerque (1990)	FEMA (2003)	City of Albuquerque (1990)	FEMA (2003)
Hydrologic Model	HEC-1	HEC-1	AHYMO	HEC-1	AHYMO
0.5		1560		1890	
0.1	5200	6285	6285	7100	4340
0.02	12750	17210	14300	13575	9150
0.01	18050	18065	18065	22215	14700
0.002	37000		30500		29400

Table 4 - Peak Flood Flows Entering the Rio Grande from the Tijeras Arroyo

Recurrence Interval	Peak Flow (CFS)	Notes
0.5	1560	From the 1990 City of Albuquerque hydrology
0.2	4200	Graphical solution
0.1	6285	From the 1990 City of Albuquerque hydrology
0.02	14300	FEMA 2003 (coincides w graphical solution)
0.01	18065	From the 1990 City of Albuquerque hydrology
0.005	26000	Graphical solution
0.002	37000	From the 1979 Corps of Engineers hydrology

It should be noted that all of the hydrologic models for the Tijeras Arroyo used 6-hour duration storms. The Albuquerque Airport is located on the northern border of the Tijeras Arroyo watershed, near its western edge. The point precipitation frequency estimates from NOAA Atlas 14 at the Albuquerque Airport were downloaded from the NWS Precipitation Frequency Data Server. Those precipitation frequency estimates are shown in Table 5 for the 6-hour, 12-hour, 24-hour and 48-hour storms, together with the percentage of change in precipitation as compared with the 6-hour storm event for the longer storms. As the storm duration increases, the increase in the amount of precipitation is fairly flat. Significant infiltration occurs over the Tijeras Arroyo watershed, but since infiltration is time-dependent, it is less of a factor in the volume of runoff for a 6-hour storm event than for a longer storm event. It can be concluded that the shorter duration 6-hour storm is appropriate to use as the design storm.

In order to verify that the 6-hour hydrographs are appropriate for the FLO-2D frequency flood flow routing, the volume of historic flood hydrographs was evaluated to the extent possible. One-day flows are provided by the USGS for gage 08330600 from 10/1/1982 through 9/30/2004. For gage 08330500 these data are available from 4/1/1943 through 6/30/1949. The annual maximum one-day flows greater than 40 cfs are shown on Table 6, for both gages. Also shown are corresponding annual instantaneous peaks and ratios of annual instantaneous peaks to annual maximum one-day flows.

Figure 11. Composite Flood Frequency Curve for the Tijeras Arroyo at its Confluence with the SDC

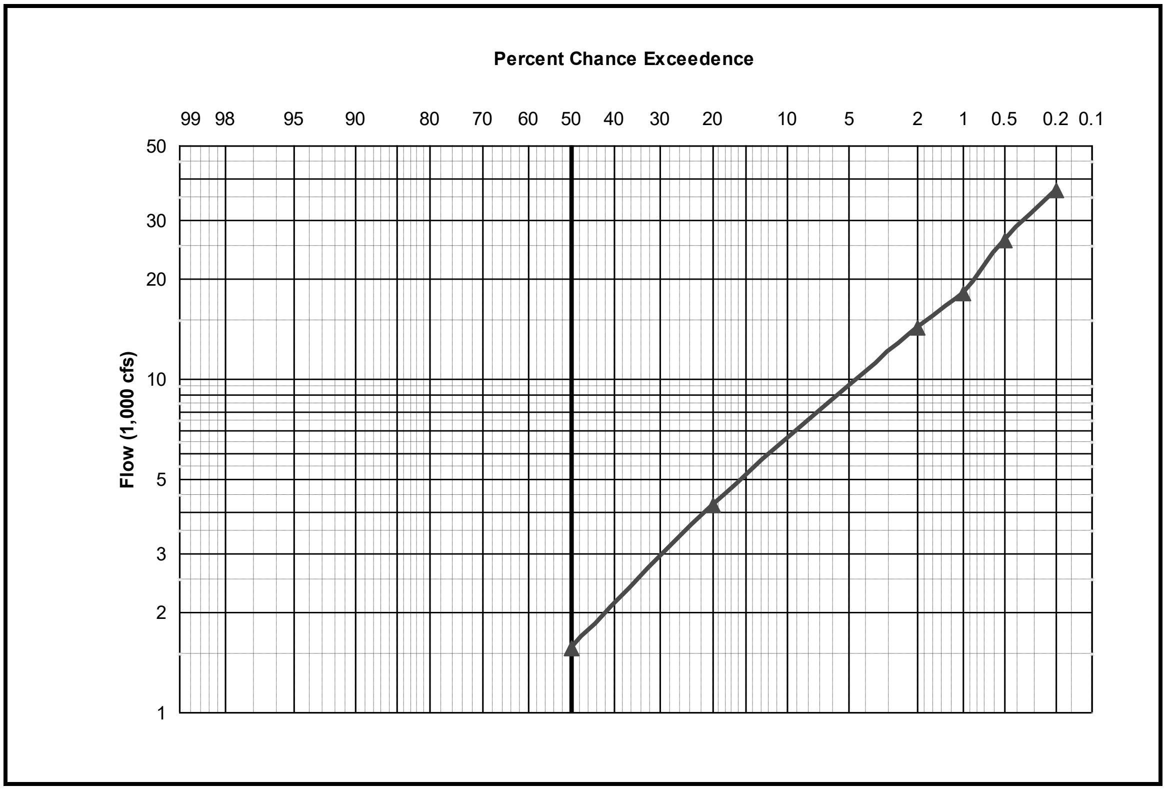


Figure 12. Frequency Flood Hydrographs from the Tijeras Arroyo

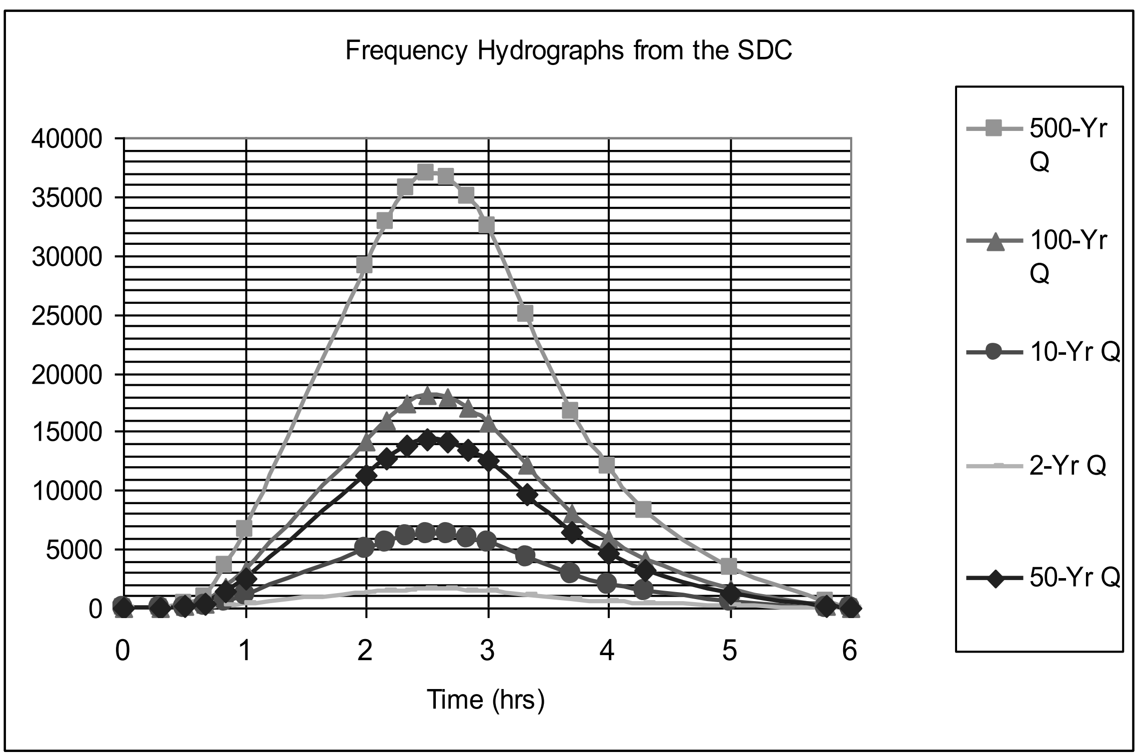


Table 5 - Comparison of Point Precipitation Frequencies for Different Duration Storm Events

Point Precipitation Frequency Estimates at the Albuquerque Airport							
Average Recurrence Interval (Years)	Precipitation Frequency Estimate (Inches)				% Change from 6-Hour Event		
	6 hour	12 hour	24 hour	48 hours	12 hour	24 hour	48 hours
2	0.96	1.06	1.21	1.36	10.4	26.0	41.7
5	1.22	1.33	1.51	1.69	9.0	23.8	38.5
10	1.43	1.54	1.74	1.94	7.7	21.7	35.7
50	1.92	2.04	2.31	2.55	6.3	20.3	32.8
100	2.15	2.27	2.55	2.82	5.6	18.6	31.2
200	2.38	2.49	2.8	3.09	4.6	17.6	29.8
500	2.69	2.8	3.14	3.46	4.1	16.7	28.6

Table 6 - Correspondence of Annual Maximum One-Day Flows with Annual Instantaneous Peak Flows at Gages on Tijeras Arroyo

Annual Peaks Corresponding to Mean Daily Peaks (Mean Daily Peaks Greater Than 40 CFS)				
Gage 8330500	2285 1-Day Flows Recorded from 4/1/1943 to 6/7/1949			
Annual Max 1-Day Q	Date of 1-D Max Q	Inst Peak Q	Date of Inst Peak	Pk/1-d
240	9/27/1944			
200	7/19/1944	6410	7/19/1944	32.1
66	8/22/1947	4810	8/22/1947	72.9
50	7/19/1946			
49	8/11/1944			
42	7/31/1943			
40	8/4/1946	852	8/4/1946	21.3
Gage 08330600	6224 1-Day Flows Recorded from 10/1/1982 to 9/30/2004			
Annual Max 1-Day Q	Date of 1-D Max Q	Inst Peak Q	Date of Inst Peak	Pk/1-d
122	7/9/1988	2930	7/9/1988	24.0
113	8/15/1994			
88	4/3/2004			
73	8/22/1987	1140	8/22/1987	15.6
70	10/11/1985			
68	10/15/1984	430	10/15/1984	6.3
66	7/14/1990	992	7/14/1990	15.0
58	8/4/2004			
48	9/13/1988			
47	8/3/1999	520	8/3/1999	11.1
42	4/28/1985			
40	6/28/1996			
40	7/9/1996			
40	7/23/2004			

Annual maximum one-day flows correspond with annual maximum one-day storm volumes. Figure 4 clearly shows that 1-day peak flows can be expected to be less than an order of magnitude of the corresponding instantaneous peaks. This suggests that flood events on Tijeras Arroyo can be expected to have short durations, and that a 6-hour hydrograph can therefore be used to represent flood flows from this watershed.

3.6. Routing of Flood Components through the Project Area

3.6.1. The FLO-2D Flood Routing Model

Frequency flood events for three sources of flooding (regulated and unregulated floods in Albuquerque and floods from the SDC) were routed downstream in the Rio Grande to evaluate the characteristics of these floods as they move through the project area. A map of the project area was previously shown in Figure 1.

A FLO-2D model was used for routing. FLO-2D is a 2-dimensional unsteady flow numeric model that can be used to evaluate hydraulics in floodplains and open channels. It is used for several projects in the Middle Rio Grande and is well suited for modeling overbank flows. The FLO-2D model reach was the Rio Grande between the Rio Grande gage at Central Avenue in Albuquerque and the Rio Grande gage at Bernardo. The flood routing models are very conservative, in that losses from infiltration and evaporation are not included.

Two model scenarios are provided. The without-project scenario for the hydrologic routing model represents existing conditions. The spoil bank levees were removed from the model, to reflect the Corps of Engineers assumption that non-engineered levees will not remain viable in a flooding situation. A second model scenario represents with-project conditions. It is essentially the same model, but the proposed levee is represented in the model data, to evaluate flood conditions with the proposed project.

Cross sections extending across the floodplain were selected throughout the reach for evaluating the flood routing with the FLO-2D Model. This was done in order to evaluate the maximum flow passing a given section through the channel, the floodway, and the floodplain at a given time. The criteria for selecting cross section locations were based on the need to capture the hydraulic characteristics of the flooding. Cross sections used in this analysis are listed in Table 7.

3.6.2. Hydrographs Used for Routing

The HEC (2006) study provided hydrographs at the Albuquerque gage both for local unregulated flood events and floods from regulated areas.

- The flood hydrographs from the regulated areas are shown in Figure 5. They were patterned using hypothetical flood events.
- The flood hydrographs for the unregulated floods were based on dimensionless hydrographs that the HEC provided. The dimensionless hydrographs, plotted in Figure 6, were scaled to provide flood peaks that correlate with frequency flood flows for Albuquerque floods from unregulated upstream areas, shown in Table 2.

Hydrographs used for routing flood flows from the Tijeras Arroyo are shown in Figure 11. See Section 5 for the derivation of these hydrographs. Normally, rainfall runoff events occur in the months of July through October. A flow of 500 cfs is typical in the Rio Grande during those months, and a steady flow of 500 cfs was used in the FLO-2D model as the base flow in the Rio Grande. The results of routing the flood flows were later used in a combined frequency at locations in the study reach. Before using the routing results in a combined frequency, the 500 cfs was subtracted from the flow at each cross section.

Table 7 - Cross Sections Used in Analyzing the Project Area Downstream of the Albuquerque Gage for the Without-Project Condition

X-sec #	Range Line	River Mile	FLO-2D Grid #	Location
Gage	509	183.4	5033	Central Ave. Bridge, Albuquerque
1	561.5	178.3	6662	Rio Bravo Bridge, Albuquerque
2	575	177.1	7165	Below Tijeras Arroyo Confluence (SDC)
3	591	175.5	7618	End of West Side Albuquerque Levee
4	623	172.6	8602	I-25 Bridge
5	637	171.1	8973	Isleta Railroad Bridge
6	655	169.3	9351	Isleta Bridge (Rt. 147)
7	700	165.1	10497	Bosque Farms
8	738.1	161.4	11979	Bridge at Los Lunas (Rt. 6)
9	799	155.4	14636	Los Chaves
10	858.1	149.5	16447	Bridge at Belen (Rt. 309)
11	877	147.7	16888	Belen RR Bridge
12	908	144.6	17649	Downstream end of project area

3.6.3. Results of FLO-2D Flood Routing for the Without-Project Condition

The FLO-2D routing results for the chosen cross sections are listed in Tables 8 through 10 and plotted in Figures 13 through 15.

Snowmelt flooding is controlled, for the most part, by reservoirs. Reservoir releases from Cochiti Dam resulting from snowmelt flooding typically occur as a steady flow in the Rio Grande that can take place over a period of months. Present guidance for the magnitude of these reservoir releases is 7,000 cfs, though it has been higher at times in the past. The steady long-term portion of snowmelt floods has no significant attenuation.

Flow over the spillway can also result from snowmelt floods coming from upstream of the reservoirs, and is expected to begin between the 1% chance and the 0.5% chance flood events. For the 0.5% and 0.02% chance regulated flow hydrographs spillway flow occurs in addition to reservoir releases, but unlike reservoir releases the flow is not controlled. Routing peak flow which comes from rainfall runoff events shows significant attenuation through the 35-mile study reach.

One factor leading to the high amount of attenuation for the rainfall-runoff events is the relatively low volume of the high peak hydrographs. That is not the case for spillway flow. Figures 13 and 14 illustrate the differences in the flood routings for spillway flow (0.5% and 0.2% chance events in Figure 13) and rainfall-runoff flooding (Figure 14). It can be seen that the attenuation for spillway flow is gradual, where as for rainfall-runoff flooding attenuation is dramatic.

Another of the causes for the attenuation is the large volume of storage available in the channel, which is wide and shallow. Widths typically range from 500 feet to about 600 feet, with flow depths on the order of 4 feet for the 10% chance flood event.

Attenuation is also related to the large amount of flow in the floodplain and overbanks. There is significant storage in the overbanks, even for the with-project model. Overbank flow, because of vegetation in the overbanks, is slower than channel flow, and delays the portion of the flood peak that is not carried in the channel, thus reducing flood peaks.

Table 8 - Without-project Flood Peaks after Routing - Albuquerque Floods from Regulated Areas Upstream

X-sec #	Range Line	River Mile	50% Q_P (cfs)	20% Q_P (cfs)	10% Q_P (cfs)	2% Q_P (cfs)	1% Q_P (cfs)	0.5% Q_P (cfs)	0.2% Q_P (cfs)
Gage	509	183.4	5600	7380	7510	7750	7750	10300	14300
1	561.5	178.3	5585	7365	7480	7720	7720	10270	14235
2	575	177.1	5585	7320	7435	7660	7660	10100	14200
3	591	175.5	5585	7310	7415	7645	7645	10065	14100
4	623	172.6	5540	7275	7355	7600	7600	10060	14070
5	637	171.1	5515	7245	7295	7560	7560	10050	14040
6	655	169.3	5500	7210	7240	7520	7520	10045	14020
7	700	165.1	5475	7160	7210	7425	7425	10040	14015
8	738.1	161.4	5445	7100	7165	7355	7355	9960	14010
9	799	155.4	5345	7040	7120	7280	7280	9875	14005
10	858	149.5	5250	6980	7075	7200	7200	9790	13995
11	877	147.7	5250	6925	7030	7135	7135	9625	13855
12	908	144.6	5240	6870	6985	7075	7075	9460	13155

Table 9 - Without-project Flood Peaks after Routing- Albuquerque Floods from Unregulated Areas Upstream

X-sec #	Range Line	River Mile	50% Q_P (cfs)	20% Q_P (cfs)	10% Q_P (cfs)	2% Q_P (cfs)	1% Q_P (cfs)	0.5% Q_P (cfs)	0.2% Q_P (cfs)
Gage	509	183.4	5260	8100	10300	16100	18900	22100	26700
1	561.5	178.3	4915	7115	8750	12785	15265	18015	22055
2	575	177.1	4830	6980	8515	12015	14085	16650	20345
3	591	175.5	4780	6970	7940	11065	12875	14935	17905
4	623	172.6	4585	6005	6530	7910	8720	9615	11675
5	637	171.1	4385	5870	6325	7165	7570	8530	10465
6	655	169.3	4295	5840	6300	6930	7210	7575	8135
7	700	165.1	4050	5505	5850	6285	6435	6585	6855
8	738.1	161.4	3855	5380	5610	5820	5860	5910	6005
9	799	155.4	3440	5205	5520	5615	5615	5625	5675
10	858	149.5	2930	4730	5240	5415	5415	5430	5475
11	877	147.7	2850	4610	5195	5400	5405	5420	5460
12	908	144.6	2620	4335	4965	5275	5305	5335	5380

In the without-project scenario, there is another factor leading to attenuation of flood peaks. The floodway, between the levees, is generally elevated above the historic floodplain, due to deposition of sediment between the levees. The difference in elevation between the floodway and the historic floodplain is approximately 5 to 10 feet. Other than the Corrales to Albuquerque reach, most of the Rio Grande is bordered by spoil bank levees. In the without-project scenario, the spoil bank levees are assumed to fail and floodwaters are lost from the perched river to the lower floodplains. Their return to the river is greatly delayed or simply does not occur. This phenomenon greatly reduces the flood peaks.

Table 10 - Without-project Flood Peaks after Routing- Floods from the Tijeras Arroyo

X-sec #	Range Line	River Mile	2-YR Q_P (cfs)	5-YR Q_P (cfs)	10-YR Q_P (cfs)	50-YR Q_P (cfs)	100-YR Q_P (cfs)	200-YR Q_P (cfs)	500-YR Q_P (cfs)
Gage	509	183.4	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	561.5	178.3	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	575	177.1	1560	4200	6285	17210	18065	26000	37000
3	591	175.5	1395	3700	5370	11220	11670	16665	23295
4	623	172.6	1170	3220	4705	6910	7035	8095	9470
5	637	171.1	1030	2880	4250	6290	6365	6910	7630
6	655	169.3	940	2670	3995	6165	6225	6620	7145
7	700	165.1	865	2360	3525	5510	5560	5815	6095
8	738.1	161.4	785	2115	3185	5175	5205	5365	5425
9	799	155.4	650	1770	2700	4920	4952	5090	5120
10	858	149.5	580	1515	2275	4400	4455	4745	4895
11	877	147.7	550	1455	2200	4300	4365	4690	4855
12	908	144.6	505	1340	2050	4020	4075	4430	4660

Figure 16 is a graph of the results of the without-project routing of 1% chance floods from the three sources of flooding in the project area. It illustrates the difference in attenuation for flooding from regulated areas, which are primarily steady flow snowmelt flood events, as opposed to flooding from rainfall runoff flood events. It also shows how floods from the three flooding sources can dominate flooding in the river at different locations.

Figure 13. Rio Grande at Albuquerque Routed Without-Project Flood Hydrographs from the Controlled Upstream Area

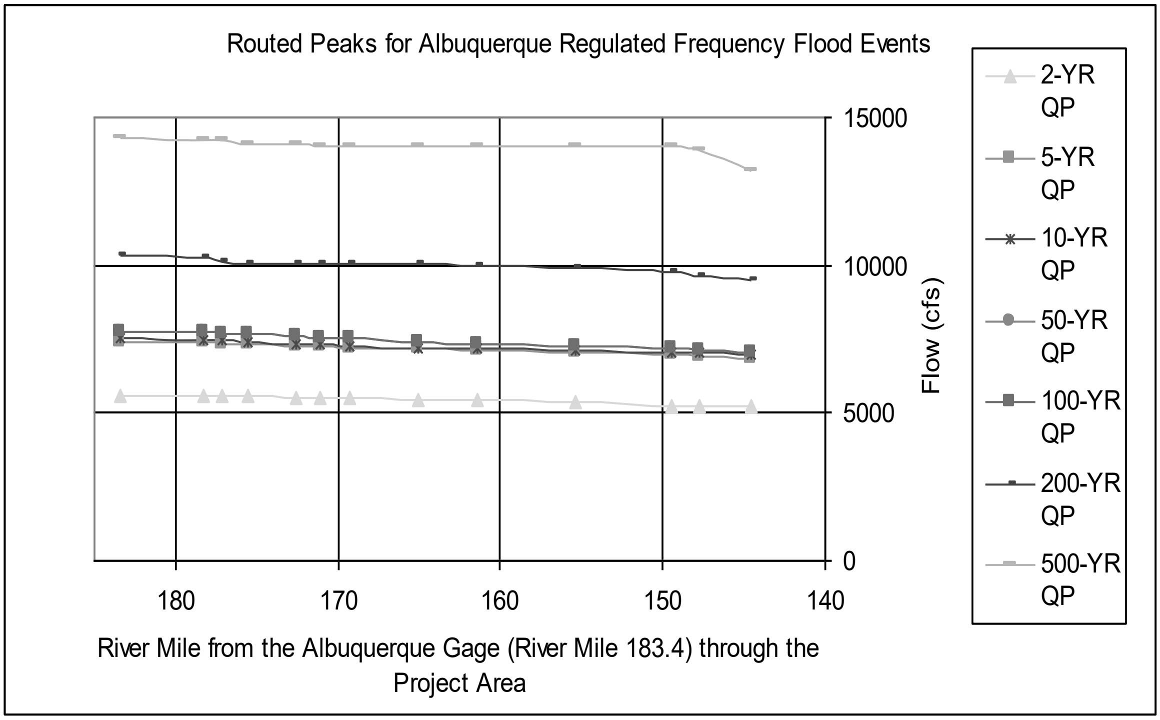


Figure 14. Rio Grande at Albuquerque Without-Project Routed Flood Hydrographs from the Uncontrolled Upstream Area

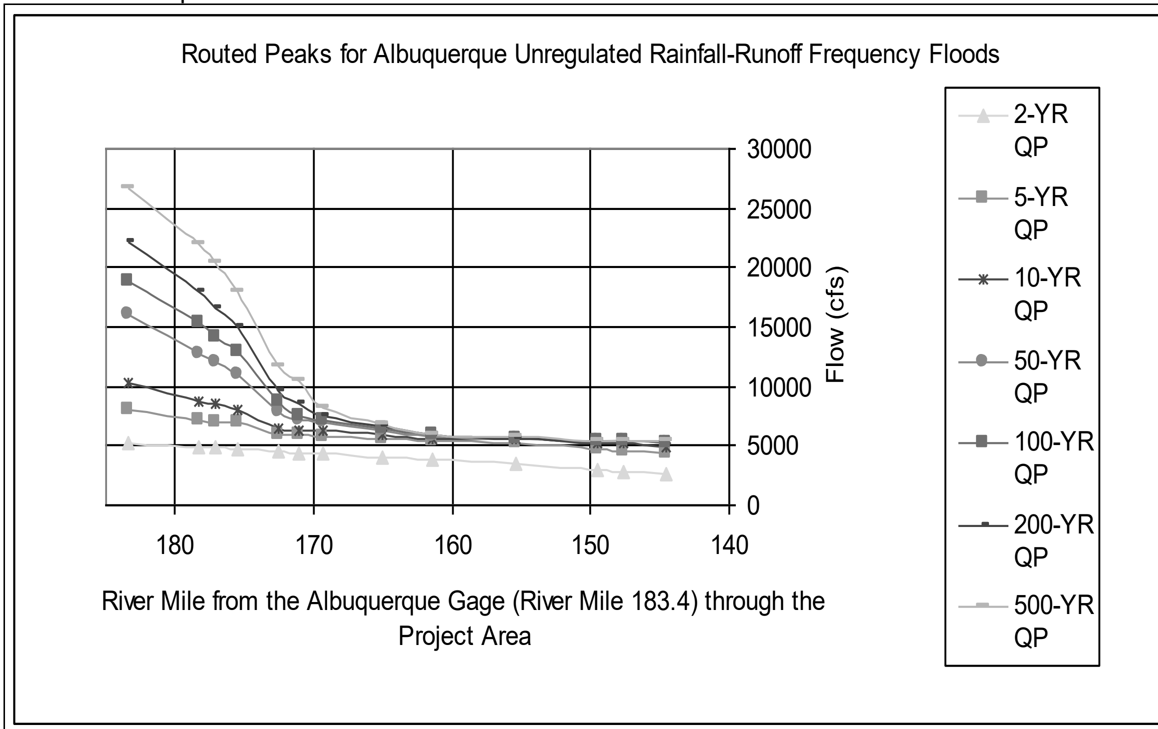


Figure 15. Rio Grande at Albuquerque Routed Without-Project Flood Hydrographs from the Tijeras Arroyo

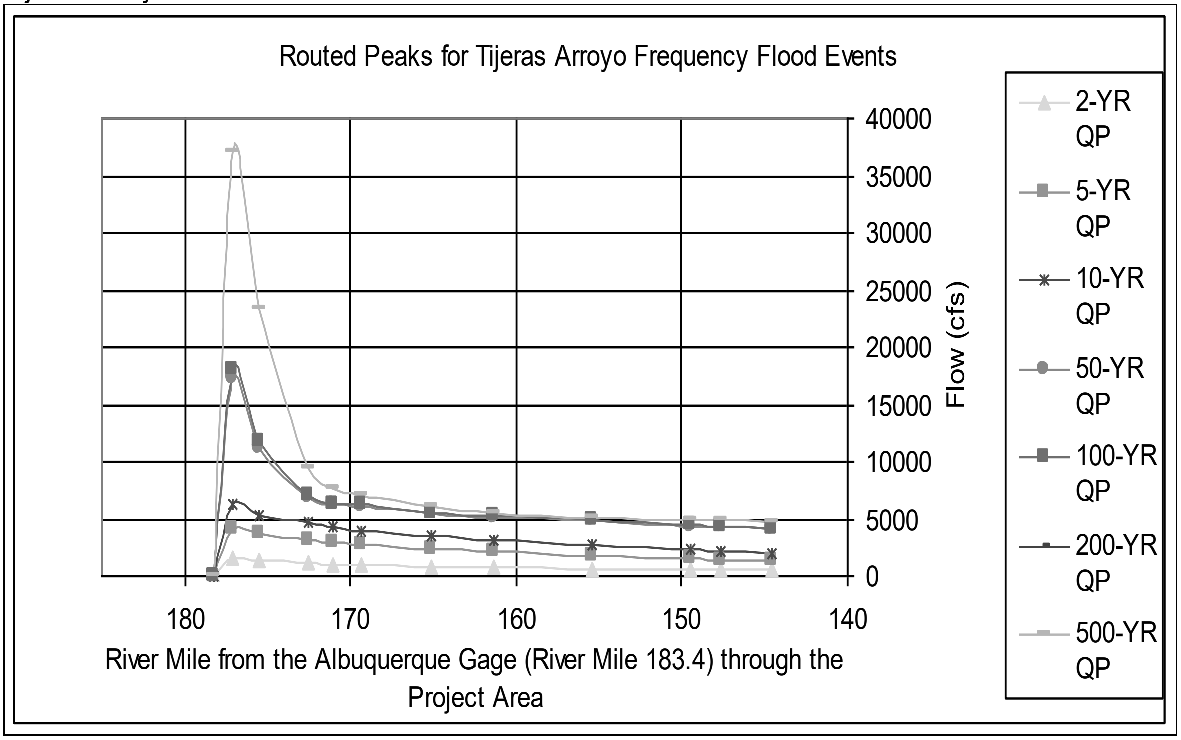
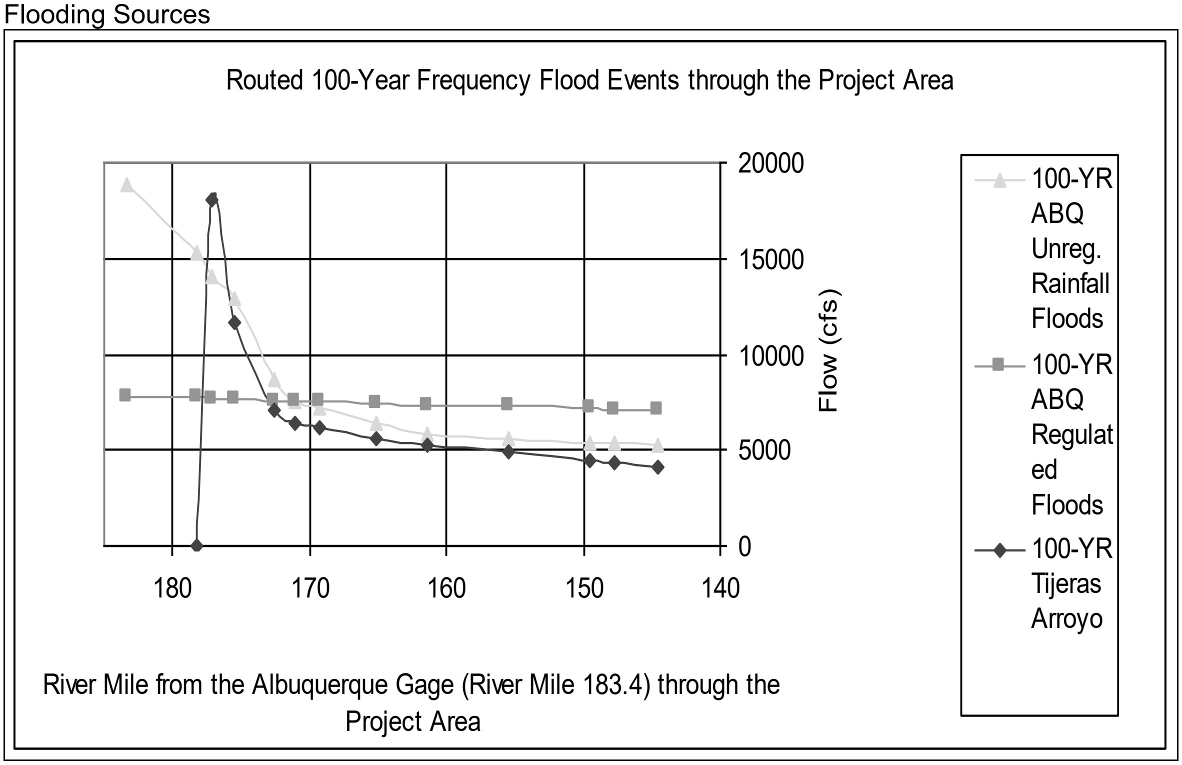


Figure 16. FLO-2D Without-Project Routing of 1% Chance Flooding from the Three Independent Flooding Sources



3.6.4. Results of FLO-2D Flood Routing for the With-Project Condition

The FLO-2D routing was performed for both the current With-Project Condition (2008) and future With-Project Condition (2058). For the purposes of With-Project flood routing, engineered levees were assumed to exist for the entire length of the project in the same location as the existing Rio Grande setback spoil bank levees. This was considered to be the most conservative approach for flood routing to determine levee height. Levee height assumed for the With-Project Condition was set to elevation 5020 (over 100 ft. high) to insure overtopping cannot occur. Results of flood routing for the selected cross sections are listed in Tables 12 through 14 for the current With-Project Condition (2008). These cross sections were selected based on damage reaches and locations where attenuation would suggest a change in levee height and are given in Table 11. The MRG Levee Project will begin at cross section 3, the confluence of the Rio Grande with the South Diversion Channel (SDC) and continue downstream to a railroad crossing of the Rio Grande at the south end of Belen, NM.

Snowmelt flooding is controlled, for the most part, by reservoirs. Reservoir releases from Cochiti Dam resulting from snowmelt flooding typically occur as a steady flow in the Rio Grande that can take place over a period of months. Present guidance for the magnitude of these reservoir releases is 7,000 cfs, though it has been higher at times in the recent past since Cochiti was operational in 1973. There is a future operational target of 10,000 cfs for these reservoir releases. The steady long-term portion of snowmelt floods has no significant attenuation through the project reach. Spillway flow can also result from snowmelt floods coming from upstream of the reservoirs, and is expected to begin between the 1% chance and the 0.5% chance flood events. Spillway flow occurs in addition to reservoir releases, but unlike reservoir releases the flow is not controlled. Spillway flow can also be of long duration resulting in no significant attenuation.

Routing of rainfall runoff events from the unregulated areas, unlike the regulated flow, shows significant attenuation through the 30-mile project reach. One factor leading to the high amount of attenuation for the rainfall-runoff events is the relatively low volume of the peak hydrographs.

Another of the causes for the attenuation is the large volume of storage available in the channel, which is wide and shallow. Widths typically range from 500 feet to about 4,000 feet, with flow depths on the order of 4 feet for the 10% chance flood event. There is significant storage in the overbanks, even for the with-project model. Overbank flow, because of vegetation in the overbanks, is slower than channel flow, and delays the portion of the flood peak that is not carried in the channel, thus reducing flood peaks.

Table 11 - Cross Sections Used in Analyzing the Project Area Downstream of the Albuquerque Gage for the With-Project Condition

X-sec #	Range Line	River Mile	FLO-2D Grid #	Location
1 - Gage	509	183.4	5033	Central Ave. Bridge, Albuquerque
2	562	178.3	6662	Rio Bravo Bridge, Albuquerque
3	576	177.1	7165	Below Tijeras Arroyo Confluence (SDC)
4	624	172.6	8602	I-25 Bridge
5	657	169.3	9351	Below Isleta Bridge (Rt. 147) @ Isleta Diversion
6	700	165.1	10497	Bosque Farms
7	740	161.4	11979	Bridge at Los Lunas (Rt. 6)
8	799	155.4	14636	Los Chaves
9	859	149.5	16447	Bridge at Belen (Rt. 309)
10	878	147.7	16888	Belen RR Bridge

Table 12 - Current With-project Condition (2008) Flood Peaks after Routing - Albuquerque Floods from Regulated Areas

X-sec #	River Mile	2-YR Q_P (cfs)	5-YR Q_P (cfs)	10-YR Q_P (cfs)	20-YR Q_P (cfs)	50-YR Q_P (cfs)	100-YR Q_P (cfs)	200-YR Q_P (cfs)	500-YR Q_P (cfs)
Gage 1	183.4	5595	7373	7500	7605	7735	7735	10297	14305
2	178.3	5579	7344	7461	7569	7689	7690	10206	13907
3	177.1	5583	7337	7452	7557	7679	7684	10226	14015
4	172.6	5569	7322	7419	7486	7649	7652	10237	14228
5	169.3	5980	7698	7771	8068	8119	8232	11096	15234
6	165.1	5541	7303	7389	7462	7615	7616	10224	14248
7	161.4	5528	7294	7375	7452	7601	7604	10202	14243
8	155.4	5506	7256	7329	7408	7551	7554	10112	14014
9	149.5	5499	7266	7332	7416	7571	7573	10218	14460
10	147.7	5500	7263	7331	7414	7569	7572	10180	14196

Table 13 - Current With-project Condition (2008) Flood Peaks after Routing - Albuquerque Floods from Unregulated Areas

X-sec #	River Mile	2-YR Q_P (cfs)	5-YR Q_P (cfs)	10-YR Q_P (cfs)	20-YR Q_P (cfs)	50-YR Q_P (cfs)	100-YR Q_P (cfs)	200-YR Q_P (cfs)	500-YR Q_P (cfs)
1-Gage	183.4	5147	7434	9344	12884	15542	18361	21577	26108
2	178.3	4930	6757	7692	9270	12044	14854	17834	21801
3	177.1	4898	6723	7753	8974	10755	13491	16652	21106
4	172.6	4825	6535	7071	7765	8877	10865	13494	18452
5	169.3	5033	6061	6620	7642	8563	10064	12606	17119
6	165.1	3592	4705	5065	6393	6918	8221	10089	13768
7	161.4	3518	4543	4789	5211	5698	7085	8875	11941
8	155.4	3398	4490	4605	4955	5208	5513	6094	8922
9	149.5	3179	4402	4548	4799	5045	5273	5638	7557
10	147.7	3076	4320	4527	4785	5030	5262	5624	7480

Table 14 - Current With-project Condition (2008) Flood Peaks after Routing - Floods from the Tijeras Arroyo

X-sec #	River Mile	2-YR Q_P (cfs)	5-YR Q_P (cfs)	10-YR Q_P (cfs)	20-YR Q_P (cfs)	50-YR Q_P (cfs)	100-YR Q_P (cfs)	200-YR Q_P (cfs)	500-YR Q_P (cfs)
1-Gage	183.4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	178.3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	177.1	2029	4521	6550	9710	15199	16041	21855	30219
4	172.6	1906	4228	5946	7445	9237	9505	13213	21694
5	169.3	1783	4554	5716	6212	7258	7371	10306	16853
6	165.1	1570	2736	3447	4205	5085	5215	6716	11170
7	161.4	1353	2475	3270	4009	4768	4829	5485	8890
8	155.4	1228	2282	3012	3826	4570	4616	5133	5864
9	149.5	1173	2117	2749	3551	4476	4518	4953	5441
10	147.7	1189	2059	2677	3420	4370	4428	4921	5421

3.7. Combined Frequency in the Project Area

3.7.1 Without-Project Combined Frequency

At any given location, the combined frequency equation using three sources of flooding is:

$$P_C = P_X + P_Y + P_Z - P_X P_Y - P_X P_Z - P_Y P_Z + P_X P_Y P_Z$$

where

P_X = Probability of reaching a given elevation in the Rio Grande with a flood from the regulated area upstream of Albuquerque

P_Y = Probability of reaching a given elevation in the Rio Grande with a flood from the unregulated area upstream of Albuquerque

P_Z = Probability of reaching a given elevation in the Rio Grande with a flood from the South Diversion Channel

P_C = Combined frequency by joint probability theorem

Tables 11, 13, 15, 17, 19, 21, 23, 25, 27, 29, 31, and 33 show the derivation of the combined frequencies (with the associated stage) for each of the selected cross sections.

Rating curves were used to convert the channel flood stage to channel flows. Channel flow was then correlated with total cross section flow across the channel, floodway and floodplain. Tables 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, and 34 show the combined frequencies at each of the selected cross sections.

From the top of the reach downstream to the Isleta Railroad Bridge, at River Mile 172.6, river flooding is dominated by rainfall-runoff storm events, both local to Albuquerque (Tijeras Arroyo) and from the uncontrolled area upstream of Albuquerque. From the Isleta Railroad Bridge downstream, river flooding is dominated by large volume snowmelt floods that are controlled by the reservoirs upstream of Albuquerque. Figure 16 illustrates this phenomenon. Figure 13 illustrates one characteristic of the snowmelt-dominated floods, that the snowmelt flooding is essentially constant as it moves downstream. Because reservoir releases occur over an extended period of time, typically several months for snowmelt frequency floods, losses due to attenuation are virtually non-existent. Other losses, such as water diversion for irrigation, cannot be assumed because they do not always occur. For the purpose of hydrology, then, the flood flows from Isleta downstream do not vary. Therefore the same frequency curve was used for these cross sections.

Frequency curves are plotted in Figures 17 through 21. For a few of the cross sections, the 0.2% chance flood flows were not derived directly from the routed flood elevations. In those locations the 0.2% chance flood flows were derived graphically, and show as “projected frequencies” in the plots.

Figure 21 shows the frequency flood peaks from the Isleta Railroad Bridge downstream to the lower end of the project reach. Also plotted in Figure 21 are points from frequency flows at several cross sections in the lower reach, in order to illustrate the scatter in the routed flows. The cross section flows that are plotted in Figure 21 are located where there is no bridge or other structure to interfere with flood flows.

Table 15 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 1, the Rio Bravo Bridge in Albuquerque

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4923.19	~1	~1	~0	1
4927	.59	.55	~0	0.816
4928	.46	.37	~0	0.660
4929	.007	.09	~0	0.096
4930	.003	.023	~0	0.026
4931	.0014	.0069	~0	0.008
4932	~0	.002	~0	0.002

Table 16 - Without-Project Probability of Flood Flows at Cross Section 1, the Rio Bravo Bridge in Albuquerque

Q (cfs)	4500	5500	7000	10000	12000	14000	16000
Probability	0.99	0.82	0.3	0.045	0.02	0.012	0.007

Table 17 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 2, the South Diversion Channel

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4916.87	~1	~1	~1	1
4921	.58	.55	.23	0.854
4922	.44	.34	.09	0.664
4923	.0057	.06	.035	0.098
4924	.002	.012	.015	0.029
4925	~0	.0027	.0064	0.009
4926	~0	~0	.0028	0.003
4926.4	~0	~0	.002	0.002

Table 18 - Without-Project Probability of Flood Flows at Cross Section 2, the South Diversion Channel

Q (cfs)	5500	7000	8000	10000	12000	15000	20000	24000	28000
Probability	0.99	0.4	0.2	0.11	0.06	0.03	.015	0.005	0.002

Table 19 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 3, the End of the West Side Albuquerque Levee

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4911.16	~1	~1	~1	1
4915	.6	.56	.23	0.864
4916	.48	.38	.095	0.708
4917	.0067	.068	.016	0.089
4918	.002	.0088	.0082	0.019
4919	~0	~0	.0031	0.003
4919.37	~0	~0	.002	0.002

Table 20 - Without-Project Probability of Flood Flows at Cross Section 3, the End of the West Side Albuquerque Levee

Q (cfs)	5800	7000	10600	12000	14000	16000	18000	21500
Probability	0.99	0.4	.095	.04	.03	.019	.011	.002

Table 21 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 4, I-25 Bridge

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4898.95	~1	~1	~1	1
4903	.57	.57	.22	0.856
4904	.43	.38	.06	0.668
4905	.0056	.0075	.004	0.017
4905.68	.002	~0	~0	0.002

Table 22 - Without-Project Probability of Flood Flows at Cross Section 4, I-25 Bridge

Q (cfs)	5700	7000	8000	9000	10000	12000	14500
Probability	0.99	0.3	0.12	0.03	0.018	0.006	0.002

Table 23 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 5, Isleta Railroad Bridge

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4894.07	~1	~1	~1	1
4899	.55	.52	.15	0.816
4900	.37	.2	.036	0.514
4900.5	.009	.02	.008	0.037
4901	.0038	.0027	~0	0.006
4901.23	.002	~0	~0	0.002

Table 24 - Without-Project Probability of Flood Flows at Cross Section 5, Isleta Railroad Bridge

Q (cfs)	3900	4000	5000	6000	7000	8000	9000	11000
Probability	0.99	0.97	0.8	0.5	0.06	0.025	0.015	0.0045

Table 25 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 6, Isleta Bridge

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4880.73	~1	~1	~1	1
4886.5	.53	.43	.096	0.758
4887	0.4	0.2	.037	0.538
4887.5	.008	.01	.045	0.062
4888	.0028	~0	~0	0.003
4888.14	.002	~0	~0	0.002

Table 26 - Without-Project Probability of Flood Flows at Cross Section 6, Isleta Bridge

Q (cfs)	1400	2000	3000	4000	5000
Probability	.99	0.7	0.3	0.02	0.002

Table 27 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 7, Bosque Farms

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4862.62	~1	~1	~1	1
4867.5	.53	.5	.096	0.788
4868	.42	.16	.028	0.526
4868.5	.0082	~0	~0	0.008
4869	.0028	~0	~0	0.003
4869.13	.002	~0	~0	0.002

Table 28 - Without-Project Probability of Flood Flows at Cross Section 7, Bosque Farms

Q (cfs)	4200	5000	6000	7000	7050	8000	9000	10000
Probability	0.99	0.85	0.5	0.2	0.02	0.007	0.004	0.003

Table 29 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 8, Bridge at Los Lunas

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4847.1	~1	~1	~1	1
4851.5	.55	.52	.16	0.819
4852	.52	.35	.054	0.705
4852.5	.1	.0098	.009	0.117
4852.78	.002	~0	~0	0.002

Table 30 - Without-Project Probability of Flood Flows at Cross Section 8, Bridge at Los Lunas

Q (cfs)	4900	5350	8000	10000	12000	14000	16000
Probability	0.99	0.85	0.5	0.1	0.05	0.02	0.007

Table 31 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 9, Intermediate Location between Los Lunas and Belen

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4820.99	~1	~1	~1	1
4826	.53	.45	.05	0.754
4826.5	.46	.17	.02	0.561
4826.57	.3	.01	.002	0.308
4826.79	.002	~0	~0	0.002

Table 32 - Without-Project Probability of Flood Flows at Cross Section 9, Intermediate Location between Los Lunas and Belen

Q (cfs)	4300	5350	7200	11500
Probability	0.99	0.65	0.312	0.002

Table 33 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 10, Bridge at Belen

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4796.23	~1	~1	~1	1
4801.5	.63	.59	.27	0.889
4802	.54	.38	.04	0.726
4802.5	.51	.18	.02	0.606
4803	.39	~0	~0	0.390
4803.5	.23	~0	~0	0.230
4803.57	.01	~0	~0	0.010
4805.6	.002	~0	~0	0.002

Table 34 - Without-Project Probability of Flood Flows at Cross Section 9, Bridge at Belen

Q (cfs)	3900	4000	5500	6000	7000	10000
Probability	.99	0.97	0.3	0.02	0.0085	0.003

Table 35 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 11, Belen Railroad Bridge

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4788.64	~1	~1	~1	1
4794	.57	.46	.06	0.782
4794.5	0.53	0.3	.029	0.681
4795	0.5	.05	.002	0.526
4795.54	0.2	~0	~0	0.200
4795.58	0.1	~0	~0	0.100
4795.62	0.01	~0	~0	0.010
4796.38	0.005	~0	~0	0.005
4797.31	0.002	~0	~0	0.002

Table 36 - Without-Project Probability of Flood Flows at Cross Section 11, Belen Railroad Bridge

Q (cfs)	5000	6000	7000	8000	10000	11000	12000	13400
Probability	0.99	0.85	0.7	0.55	0.25	0.01	0.008	0.006

Table 37 - Without-Project Combined Frequency Analysis Multiple Discrete Event Method at Cross Section 12, Downstream End of Project Area

Elevation (NAVD 88)	Probability of Reaching Elevation with Flooding from the Regulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the Unregulated Area Upstream of Albuquerque	Probability of Reaching Elevation with Flooding from the South Diversion Channel	Combined Frequency by Joint Probability Theorem
4776.4	~1	~1	~1	1
4783.5	.55	.47	.07	0.778
4784	.53	.38	.032	0.718
4784.6	0.5	.02	.001	0.510
4785.07	0.2	~0	~0	0.200
4785.1	0.1	~0	~0	0.100
4785.13	0.01	~0	~0	0.010
4785.79	0.005	~0	~0	0.005
4786.51	.002	~0	~0	0.002

Table 38 - Without-Project Probability of Flood Flows at Cross Section 12, Downstream End of Project Area

Q (cfs)	5000	5800	7800	10000	11400	14000
Probability	.99	0.942	0.1	0.02	0.005	0.002

Figure 17. Without-Project Combined Frequency Curve at Cross Section 1, the Rio Bravo Bridge in Albuquerque

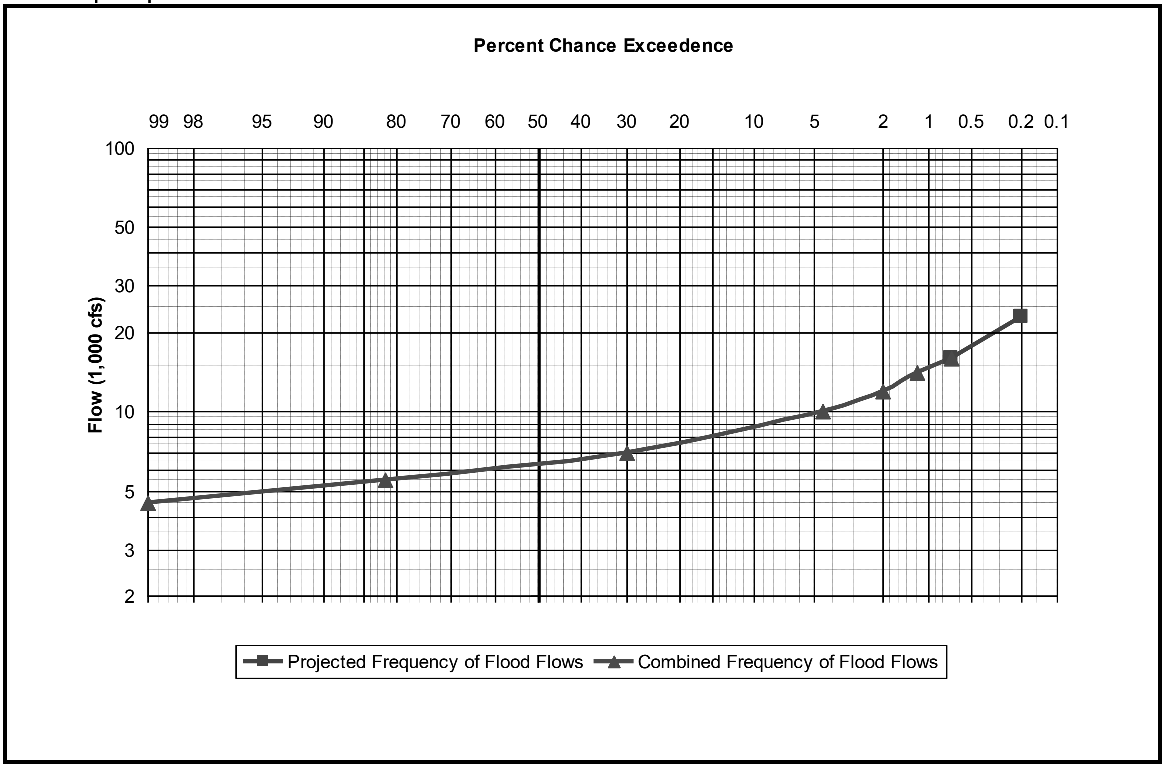


Figure 18. Without-Project Combined Frequency Curve at Cross Section 2, the South Diversion Channel

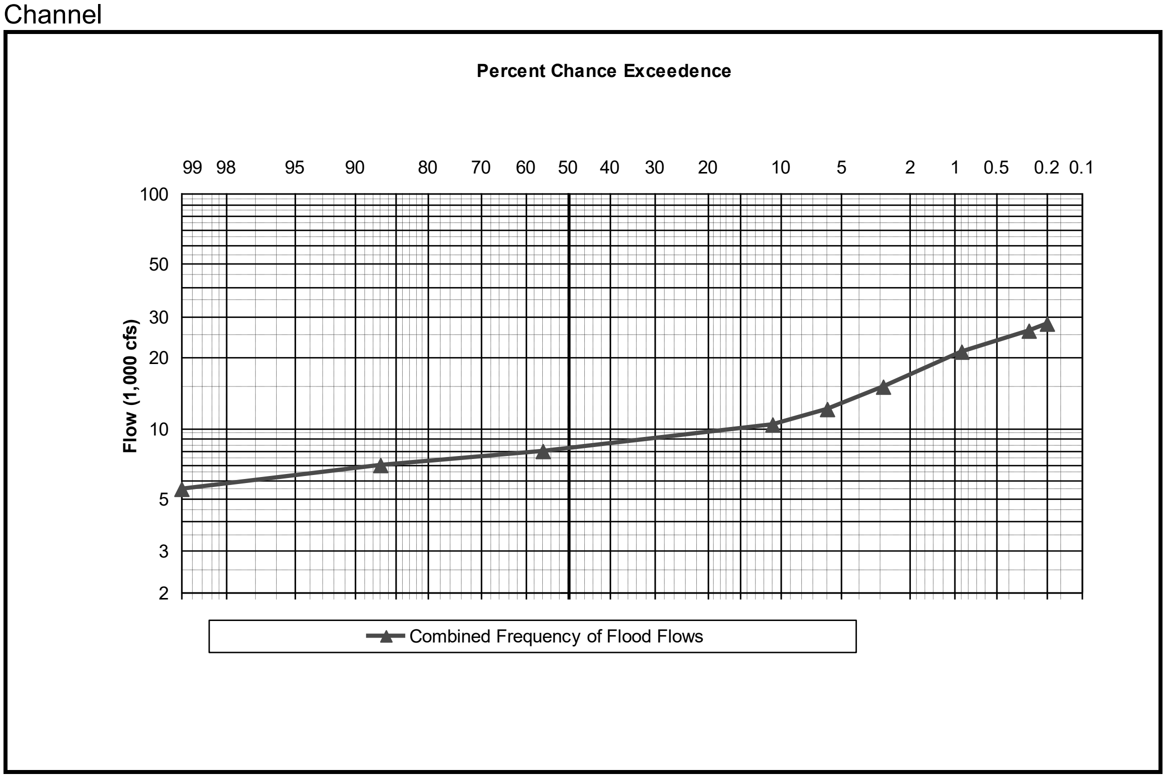


Figure 19. Without-Project Combined Frequency Curve at Cross Section 3, the End of the West Side Albuquerque Levee

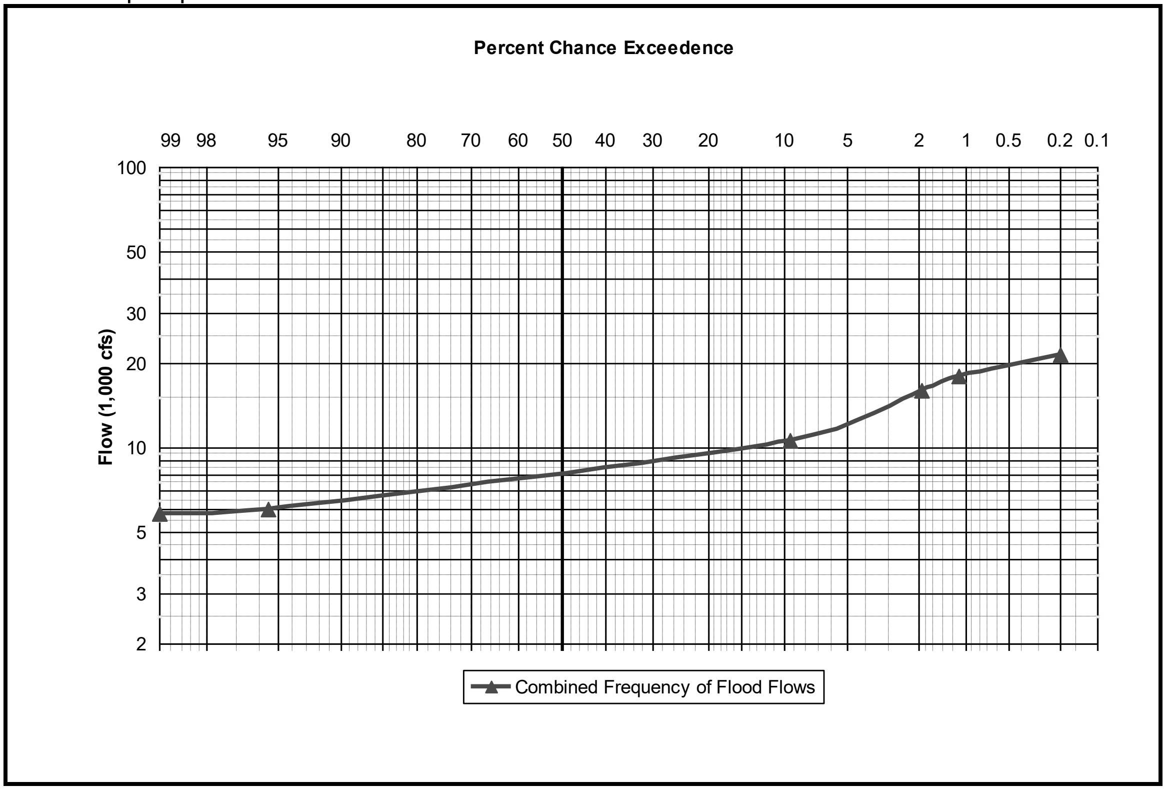


Figure 20 - Without-Project Combined Frequency Curve at Cross Section 4, I-25 Bridge

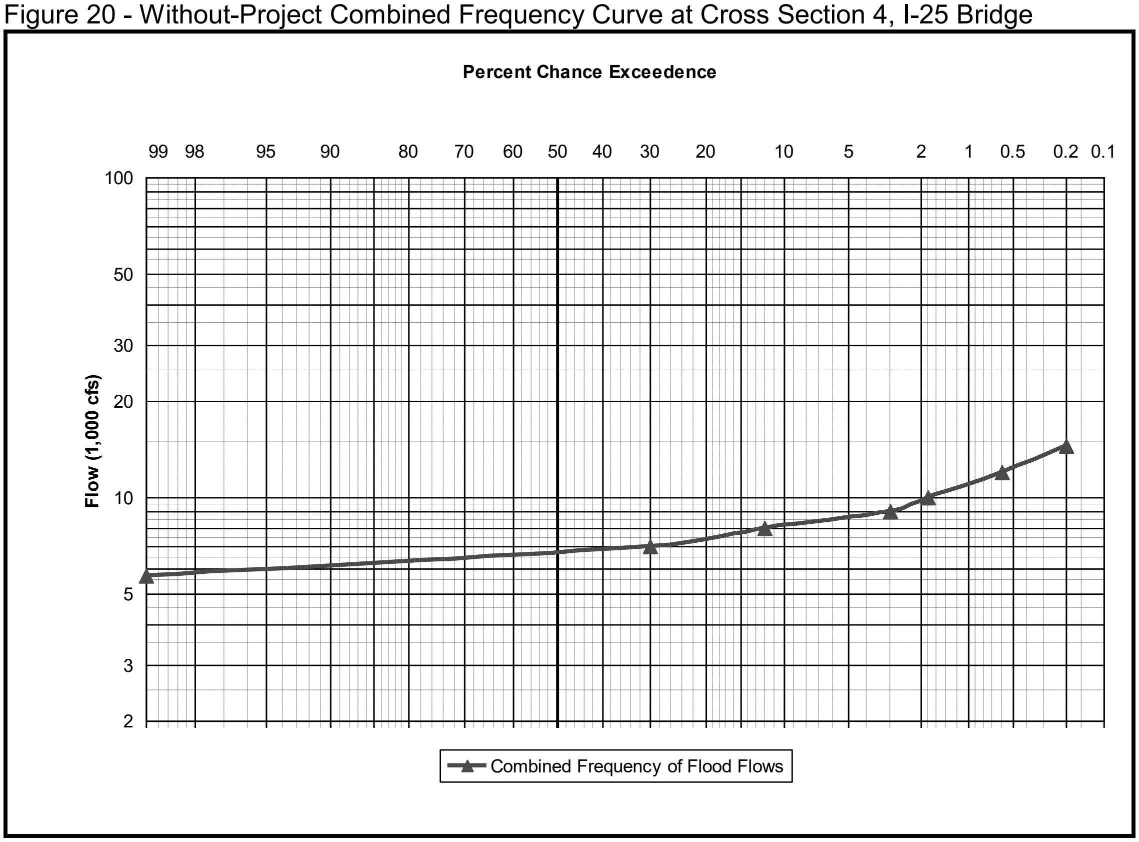
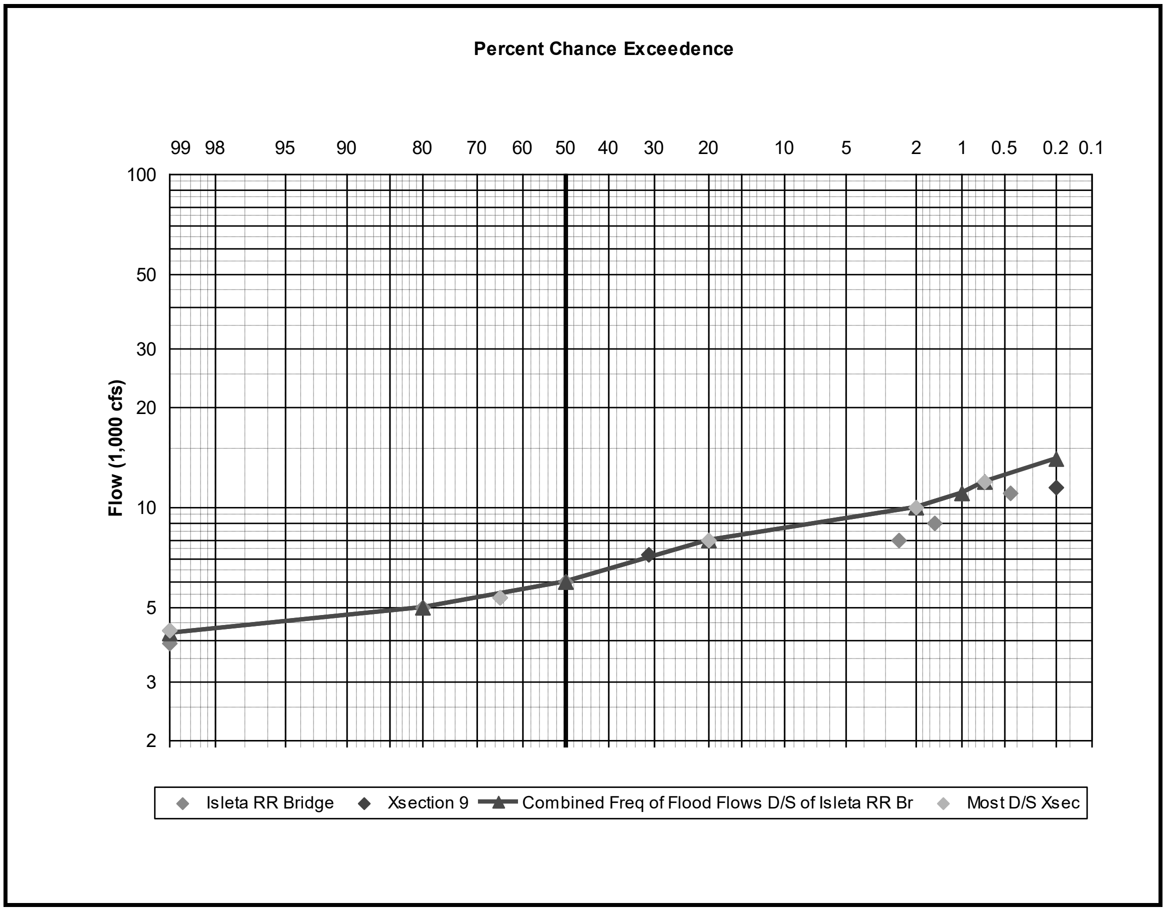


Figure 21. Without-Project Combined Frequency for Cross Sections 5 to 12; Isleta Railroad Bridge to Belen (Plotted Line). Note: Additional Points Shown in Graph Represent Scatter in Data.



3.7.2. With-Project Combined Frequency

The With-Project combined frequency analysis was performed using HEC-FDA for both current conditions (2008) and future conditions (2058). These results will be used by HEC-FDA to determine levee heights using the risk analysis for flood damage reduction studies. The MRG Levee Project will begin at the South Diversion Channel (cross section 3 / Damage Reach 1) and extend through Belen (cross section 10 / Damage Reach 8). These cross sections, damage reaches and levee reaches are identified in the following Table 39.

Table 39 - Cross Sections Used in Analyzing the With-Project Levee from the SDC through Belen

X-sec #	Range Line	River Mile	FLO-2D Grid #	Location	Levee Reach	Damage Reach
3	576	177.1	7165	Below Tijeras Arroyo	Mountain View	1
4	624	172.6	8602	I-25 Bridge	Isleta North	2
5	657	169.3	9351	Isleta Bridge	Isleta South	3
6	700	165.1	10497	Bosque Farms	Bosque Farms	4
7	740	161.4	11979	Los Lunas Bridge	Los Lunas	5
8	799	155.4	14636	Los Chaves	Los Chaves	6
9	859	149.5	16447	Belen Hwy Bridge	Belen	7
10	878	147.7	16888	Belen RR Bridge	Belen South	8

The routed flows used for the current With-Project Condition (2008) are shown in Table 40 and were taken from Tables 12, 13 & 14 based on the highest flow for each frequency event at each of the given cross sections. For the With-Project Condition, river flooding is generally dominated by rainfall-runoff storm events from the South Diversion Channel downstream to the Isleta Diversion (River Mile 169.3). From the Isleta Diversion downstream to the end of the project, river flooding is dominated by large volume snowmelt floods that are regulated by the reservoirs upstream of Albuquerque. The routed flows shown in Table 40 are also the discharge-probability inputs for the HEC-FDA program under the “HydEng” tab for the exceedance probability functions with uncertainty at each cross section for the current With-Project Condition (2008).

Table 40 – Current With-project Condition (2008) Flood Peaks after Routing - Floods from the Tijeras Arroyo

X-sec #	Levee Reach	2-YR Q_P (cfs)	5-YR Q_P (cfs)	10-YR Q_P (cfs)	20-YR Q_P (cfs)	50-YR Q_P (cfs)	100-YR Q_P (cfs)	200-YR Q_P (cfs)	500-YR Q_P (cfs)
3	Mt. View	5583	7337	7753	9710	15199	16041	21855	30219
4	Isleta N	5569	7322	7419	7765	9237	10865	13494	21694
5	Isleta S	5980	7698	7771	8068	8563	10064	12606	17119
6	Bosque Farm	5541	7303	7389	7462	7615	8221	10224	14248
7	Los Lunas	5528	7294	7375	7452	7601	7604	10202	14243
8	Los Chaves	5506	7256	7329	7408	7551	7554	10112	14014
9	Belen	5499	7266	7332	7416	7571	7573	10218	14460
10	Belen South	5500	7263	7331	7414	7569	7572	10180	14196

The “Equivalent Record Length (N)” was set at 15 years in the HEC-FDA program for the Mountain View Reach. The value for “N” was set at 15 years for this reach since the controlling flow for most frequencies comes from the Tijeras Arroyo. Tijeras Arroyo flows were determined through hydrologic modeling and then routed to the project reach. The “Equivalent Record Length (N)” was set at 10 years for all other levee reaches due to the longer distances of flow routing both from the Tijeras Arroyo and from the Albuquerque Gage. The record lengths given were used for both With- and Without-Project Conditions. Selections of values for the equivalent record length were determined after discussions with the Albuquerque District Hydrologist and were based on guidance given in EM 1110-2-1619. At the time of this study the actual record length for the Albuquerque gage following the start of operations of Cochiti Dam was 27 years. Routing hydrographs downstream from the Albuquerque gage into the project reach has the effect of further reducing the equivalent record length. As the routing length increases, the flow rate is further attenuated. Since it was felt this introduces additional uncertainty, the equivalent record length was reduced to account for the increased routing lengths and additional uncertainty. Therefore equivalent record lengths were set at 10 to 15 years depending on the reach and routing length.

The resulting confidence limit curves are given below for the current With-Project Condition (2008) HEC-FDA output for each levee reach. The output is given in both tabular form and plotted form for each levee reach. This analysis was conducted for both the current With-Project Condition (2008) and the future With-Project Condition (2058) and (see Attachment 7 of Appendix H).

MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 | Stream: Rio Grande

Analysis Year: 2008 | Damage Reach: 1 -Mountain View

Function: MountainView1 | Use An Existing Function | Save | Cancel

Description: Routed from SDC with flo-2d

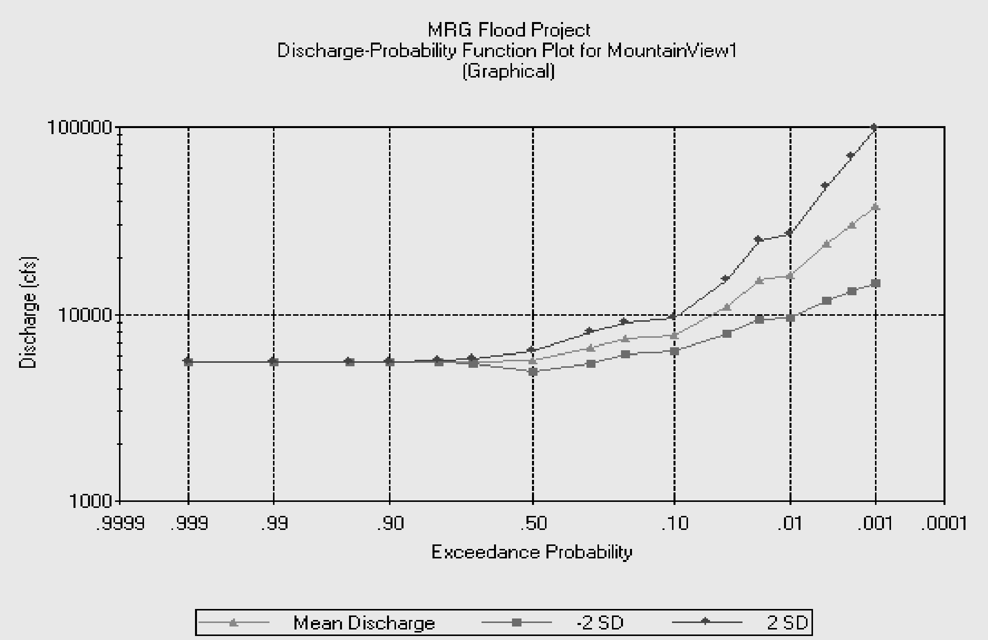
Type: ○ Analytical... ◉ Graphical...

Function Statistics... | Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,535.04	5,516.32	5,525.67	5,544.43	5,553.83
0.9900	5,546.86	5,528.14	5,537.49	5,556.25	5,565.65
0.9500	5,557.42	5,538.70	5,548.05	5,566.81	5,576.21
0.9000	5,563.06	5,544.34	5,553.69	5,572.45	5,581.85
0.8000	5,569.90	5,551.18	5,560.53	5,579.28	5,588.68
0.7000	5,574.83	5,453.26	5,513.71	5,636.63	5,699.12
0.5000	5,583.00	4,939.91	5,251.62	5,935.29	6,309.80
0.3000	6,619.07	5,438.18	5,999.64	7,302.46	8,056.39
0.2000	7,337.00	6,042.06	6,658.12	8,085.10	8,909.47
0.1000	7,753.00	6,306.86	6,992.64	8,596.04	9,530.74
0.0400	10,904.00	7,828.73	9,239.29	12,868.65	15,187.29
0.0200	15,199.00	9,350.05	11,921.05	19,378.31	24,706.78
0.0100	16,041.01	9,612.63	12,417.58	20,721.74	26,768.31
0.0040	23,715.87	11,699.00	16,656.89	33,766.37	48,076.14
0.0020	30,218.99	13,183.39	19,959.69	45,751.58	69,268.02
0.0010	37,931.16	14,733.87	23,640.48	60,860.58	97,650.73

Exceedance Probability Function Plot

File Help



MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 Stream: Rio Grande

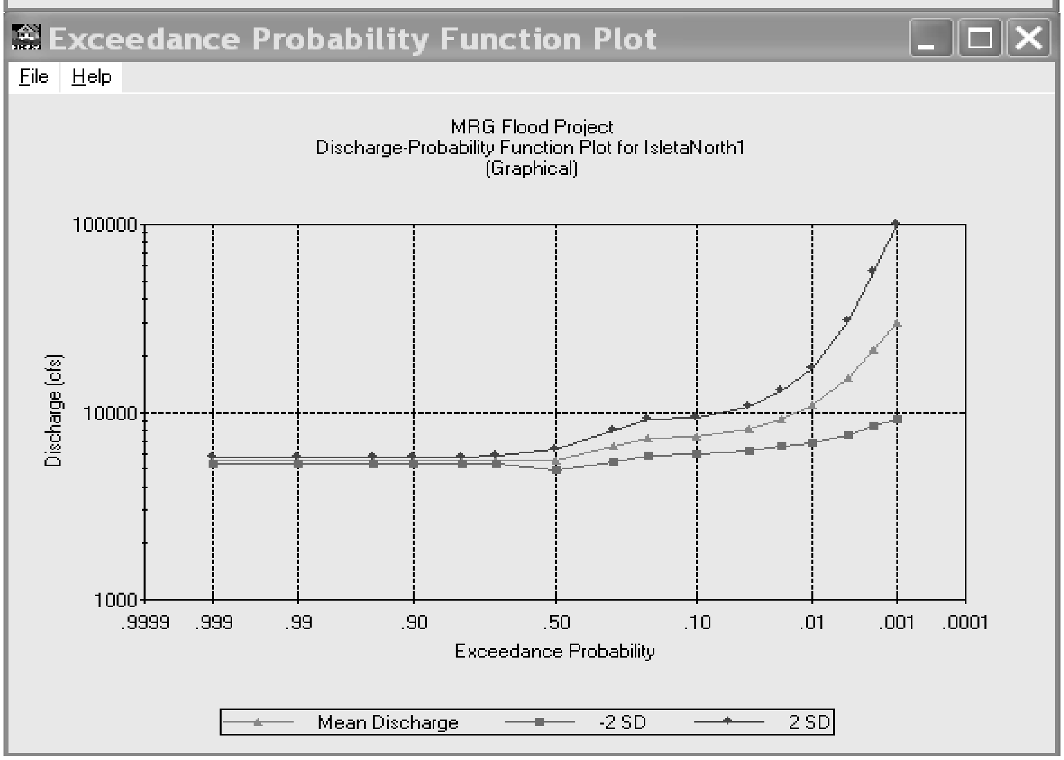
Analysis Year: 2008 Damage Reach: 2 - Isleta North

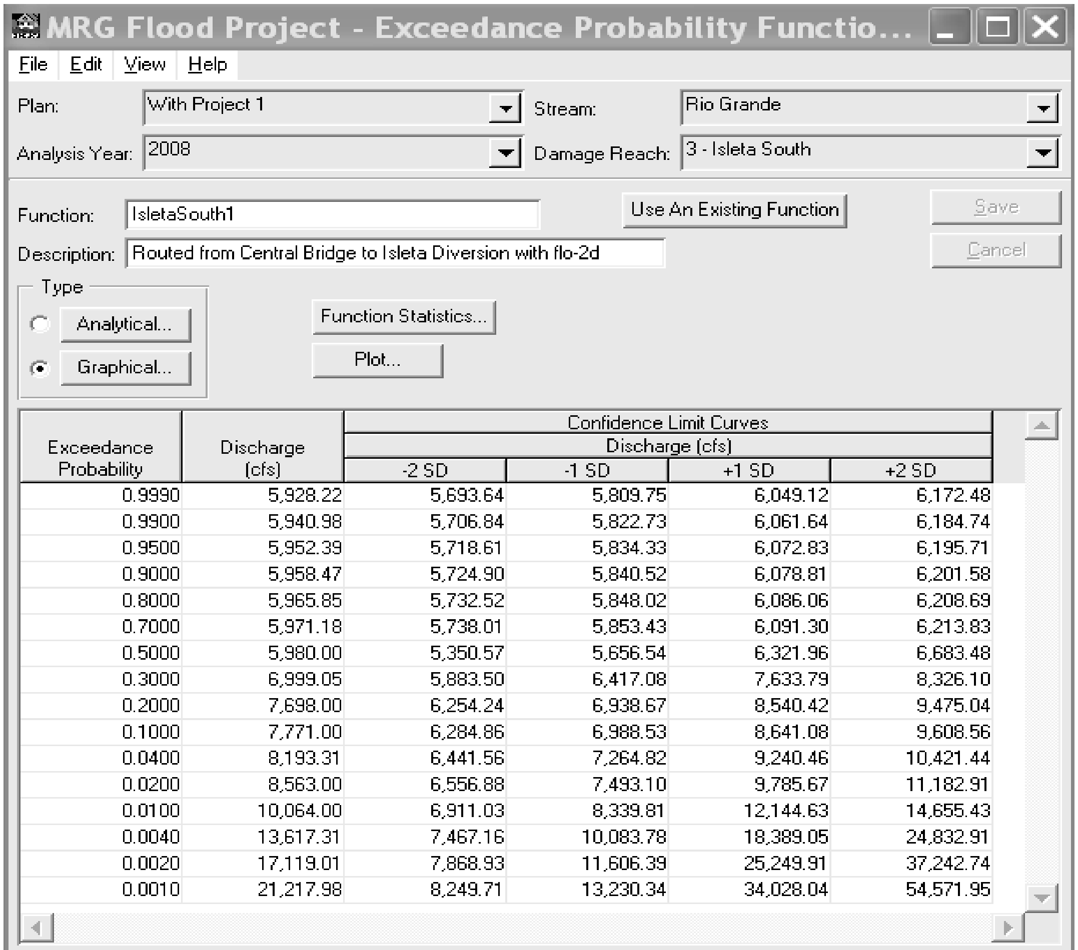
Function: IsletaNorth1 Use An Existing Function Save

Description: Routed from SDC to I-25 Bridge with flo-2d Cancel

Type: Analytical... / Graphical... Function Statistics... Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,521.17	5,285.76	5,402.19	5,642.78	5,767.06
0.9900	5,532.96	5,297.82	5,414.11	5,654.41	5,778.53
0.9500	5,543.49	5,308.59	5,424.77	5,664.81	5,788.78
0.9000	5,549.11	5,314.33	5,430.46	5,670.36	5,794.26
0.8000	5,555.93	5,321.30	5,437.35	5,677.10	5,800.91
0.7000	5,560.85	5,326.32	5,442.33	5,681.96	5,805.71
0.5000	5,569.00	4,936.96	5,243.47	5,914.74	6,281.95
0.3000	6,604.38	5,466.98	6,008.83	7,258.97	7,978.43
0.2000	7,322.00	5,873.69	6,557.98	8,175.03	9,127.44
0.1000	7,419.00	5,918.40	6,626.36	8,306.46	9,300.08
0.0400	8,121.83	6,194.36	7,092.92	9,300.00	10,649.07
0.0200	9,237.00	6,525.19	7,763.58	10,990.04	13,075.80
0.0100	10,865.00	6,894.97	8,655.28	13,638.87	17,120.92
0.0040	15,210.32	7,617.56	10,764.09	21,493.13	30,371.13
0.0020	21,693.99	8,397.03	13,496.85	34,869.57	56,047.12
0.0010	30,267.80	9,175.33	16,664.85	54,974.39	99,848.16




MRG Flood Project - Exceedance Probability Functio...

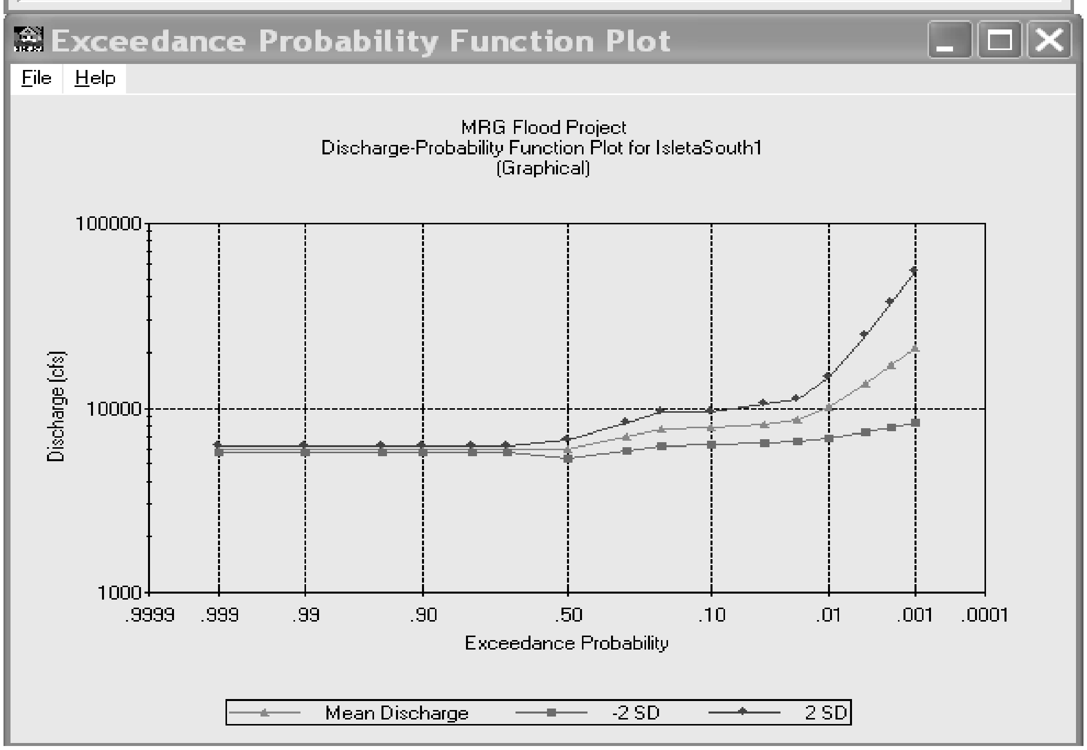
File Edit View Help

Plan: With Project 1
Stream: Rio Grande
Analysis Year: 2008
Damage Reach: 3 - Isleta South

Function: IsletaSouth1
Use An Existing Function
Save
Cancel
Description: Routed from Central Bridge to Isleta Diversion with flo-2d

Type
Analytical...
Graphical...
Function Statistics...
Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,928.22	5,693.64	5,809.75	6,049.12	6,172.48
0.9900	5,940.98	5,706.84	5,822.73	6,061.64	6,184.74
0.9500	5,952.39	5,718.61	5,834.33	6,072.83	6,195.71
0.9000	5,958.47	5,724.90	5,840.52	6,078.81	6,201.58
0.8000	5,965.85	5,732.52	5,848.02	6,086.06	6,208.69
0.7000	5,971.18	5,738.01	5,853.43	6,091.30	6,213.83
0.5000	5,980.00	5,350.57	5,656.54	6,321.96	6,683.48
0.3000	6,999.05	5,883.50	6,417.08	7,633.79	8,326.10
0.2000	7,698.00	6,254.24	6,938.67	8,540.42	9,475.04
0.1000	7,771.00	6,284.86	6,988.53	8,641.08	9,608.56
0.0400	8,193.31	6,441.56	7,264.82	9,240.46	10,421.44
0.0200	8,563.00	6,556.88	7,493.10	9,785.67	11,182.91
0.0100	10,064.00	6,911.03	8,339.81	12,144.63	14,655.43
0.0040	13,617.31	7,467.16	10,083.78	18,389.05	24,832.91
0.0020	17,119.01	7,868.93	11,606.39	25,249.91	37,242.74
0.0010	21,217.98	8,249.71	13,230.34	34,028.04	54,571.95



MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 — Stream: Rio Grande

Analysis Year: 2008 — Damage Reach: 4 - Bosque Farms

Function: BosqueFarms1 — Use An Existing Function — Save — Cancel

Description: Routed from Central Bridge to Bosque Farms with flo-2d

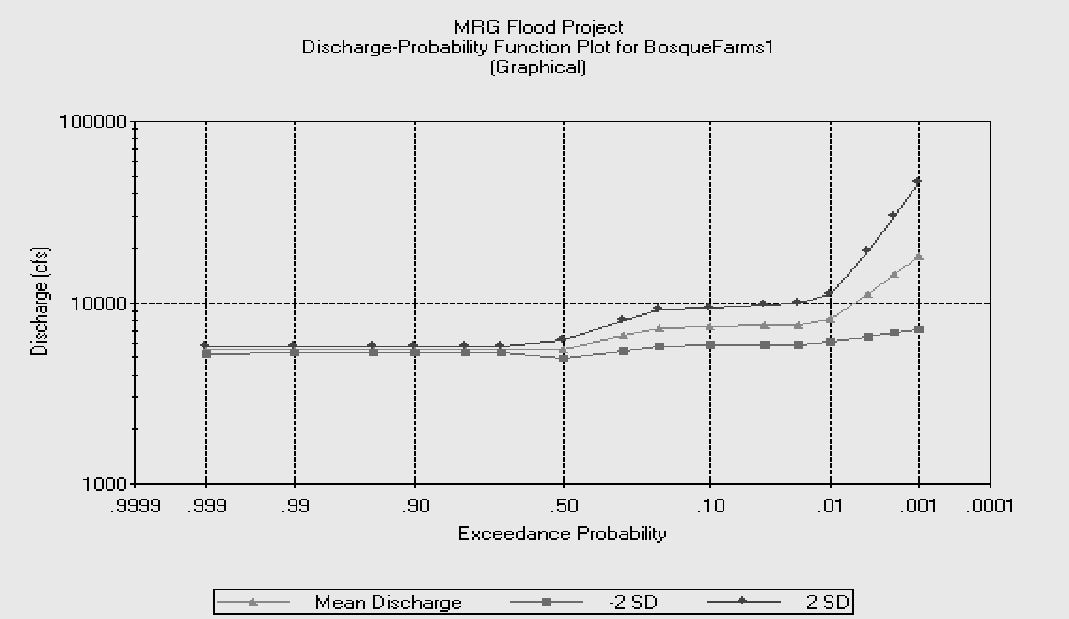
Type: Analytical... / Graphical... (selected)

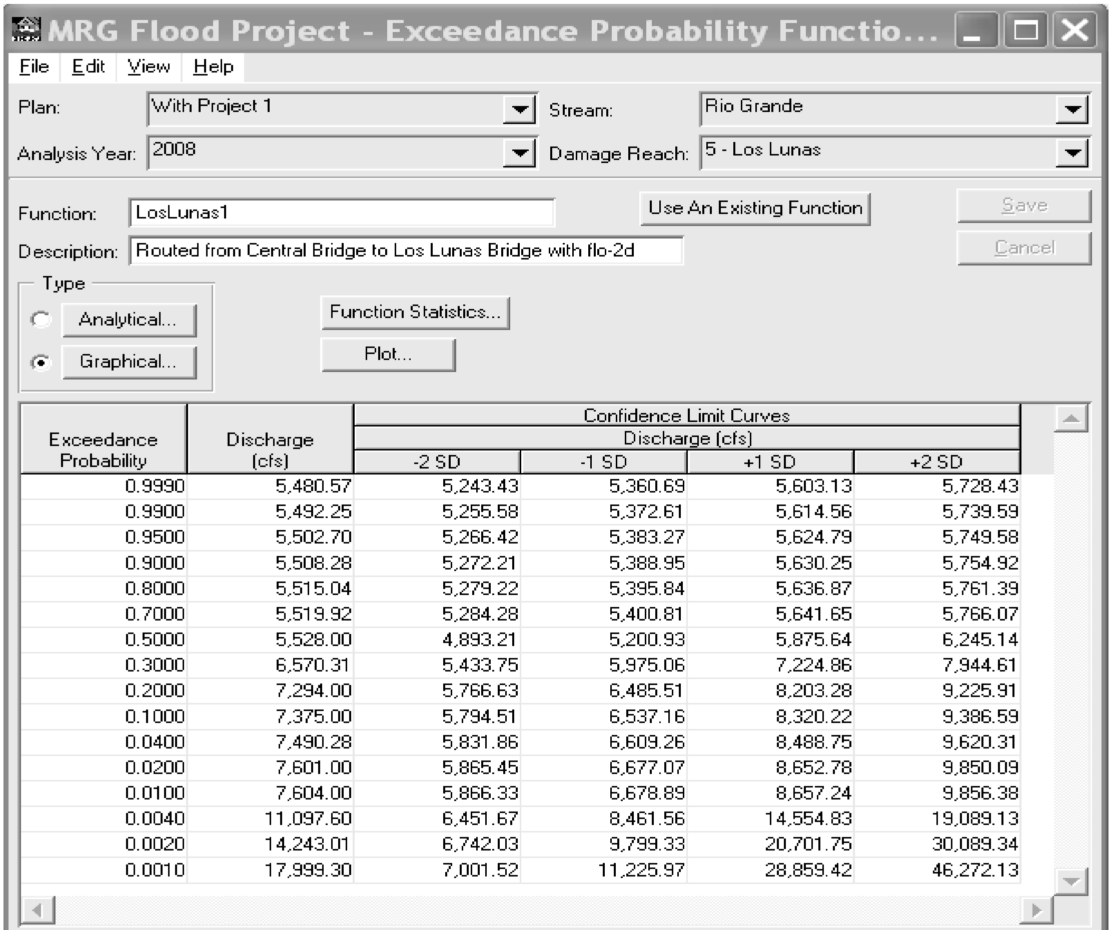
Function Statistics... — Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,493.44	5,256.73	5,373.78	5,615.77	5,740.81
0.9900	5,505.16	5,268.90	5,385.74	5,627.23	5,752.01
0.9500	5,515.63	5,279.77	5,396.41	5,637.49	5,762.03
0.9000	5,521.23	5,285.57	5,402.11	5,642.97	5,767.39
0.8000	5,528.01	5,292.60	5,409.02	5,649.61	5,773.89
0.7000	5,532.90	5,297.66	5,414.00	5,654.41	5,778.58
0.5000	5,541.00	4,907.05	5,214.40	5,888.05	6,256.85
0.3000	6,581.18	5,446.01	5,986.75	7,234.64	7,952.97
0.2000	7,303.00	5,782.13	6,498.22	8,207.45	9,223.91
0.1000	7,389.00	5,812.16	6,553.32	8,331.24	9,393.63
0.0400	7,501.30	5,849.10	6,623.89	8,494.94	9,620.19
0.0200	7,615.00	5,884.15	6,693.86	8,662.90	9,855.00
0.0100	8,221.00	6,040.42	7,046.86	9,590.77	11,188.77
0.0040	11,116.48	6,484.16	8,490.05	14,555.39	19,058.13
0.0020	14,248.00	6,782.24	9,830.23	20,651.13	29,931.90
0.0010	17,982.83	7,049.44	11,259.16	28,721.69	45,873.51

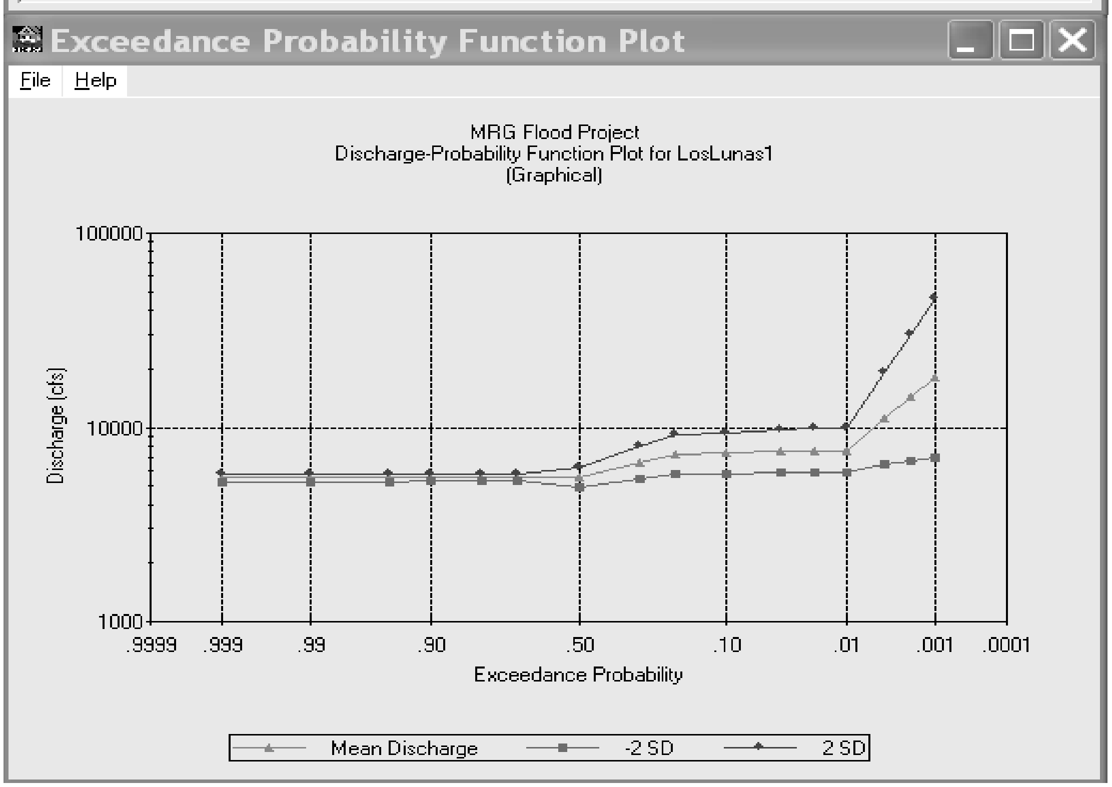
Exceedance Probability Function Plot

File Help





Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,480.57	5,243.43	5,360.69	5,603.13	5,728.43
0.9900	5,492.25	5,255.58	5,372.61	5,614.56	5,739.59
0.9500	5,502.70	5,266.42	5,383.27	5,624.79	5,749.58
0.9000	5,508.28	5,272.21	5,388.95	5,630.25	5,754.92
0.8000	5,515.04	5,279.22	5,395.84	5,636.87	5,761.39
0.7000	5,519.92	5,284.28	5,400.81	5,641.65	5,766.07
0.5000	5,528.00	4,893.21	5,200.93	5,875.64	6,245.14
0.3000	6,570.31	5,433.75	5,975.06	7,224.86	7,944.61
0.2000	7,294.00	5,766.63	6,485.51	8,203.28	9,225.91
0.1000	7,375.00	5,794.51	6,537.16	8,320.22	9,386.59
0.0400	7,490.28	5,831.86	6,609.26	8,488.75	9,620.31
0.0200	7,601.00	5,865.45	6,677.07	8,652.78	9,850.09
0.0100	7,604.00	5,866.33	6,678.89	8,657.24	9,856.38
0.0040	11,097.60	6,451.67	8,461.56	14,554.83	19,089.13
0.0020	14,243.01	6,742.03	9,799.33	20,701.75	30,089.34
0.0010	17,999.30	7,001.52	11,225.97	28,859.42	46,272.13



MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 — Stream: Rio Grande

Analysis Year: 2008 — Damage Reach: 6 - Los Chaves

Function: LosChaves 1 — Use An Existing Function — Save

Description: Routed from Central Bridge to Los Chaves with flo-2d — Cancel

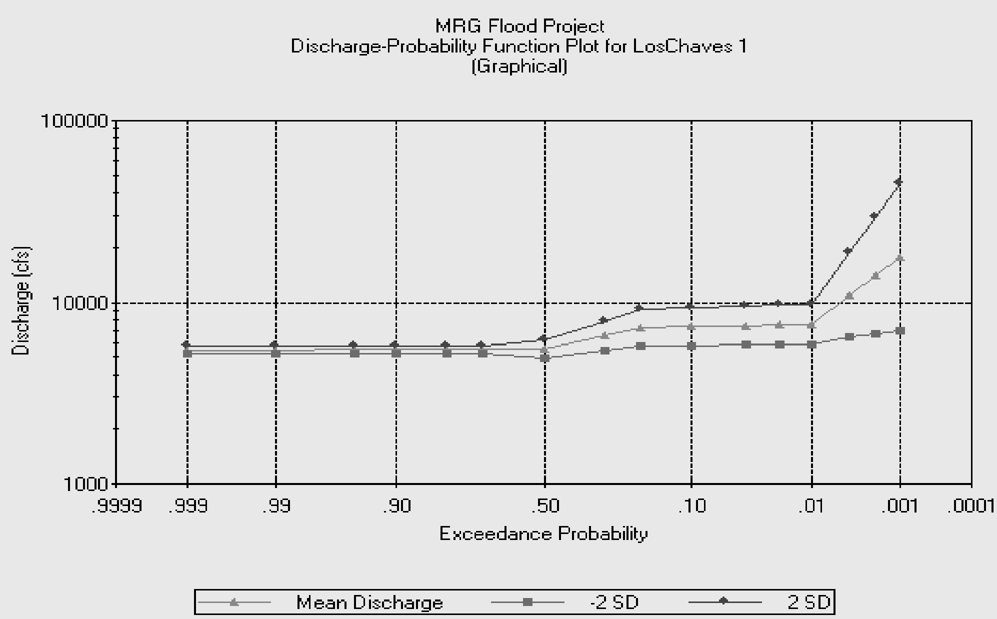
Type: Analytical... / Graphical... (selected)

Function Statistics... — Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,458.78	5,223.57	5,339.88	5,580.32	5,704.57
0.9900	5,470.41	5,235.67	5,351.75	5,591.70	5,715.68
0.9500	5,480.81	5,246.48	5,362.36	5,601.88	5,725.62
0.9000	5,486.37	5,252.24	5,368.02	5,607.32	5,730.93
0.8000	5,493.10	5,259.22	5,374.89	5,613.91	5,737.38
0.7000	5,497.96	5,264.26	5,379.84	5,618.67	5,742.03
0.5000	5,506.00	4,876.44	5,181.67	5,850.63	6,216.83
0.3000	6,539.12	5,413.79	5,949.91	7,186.68	7,898.37
0.2000	7,256.00	5,743.10	6,455.38	8,155.91	9,167.44
0.1000	7,329.00	5,768.23	6,501.95	8,261.25	9,312.08
0.0400	7,444.75	5,805.81	6,574.41	8,430.31	9,546.34
0.0200	7,551.00	5,838.12	6,639.55	8,587.58	9,766.44
0.0100	7,554.00	5,839.00	6,641.37	8,592.04	9,772.72
0.0040	10,979.33	6,415.26	8,392.57	14,363.39	18,790.49
0.0020	14,014.00	6,697.33	9,687.95	20,271.80	29,323.93
0.0010	17,618.84	6,949.09	11,065.03	28,054.49	44,671.16

Exceedance Probability Function Plot

File Help



MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 — Stream: Rio Grande

Analysis Year: 2008 — Damage Reach: 7 - Belen

Function: Belen Reg1 — Use An Existing Function — Save

Description: Routed from Central Bridge to Belen Hwy Bridge with flo-2d — Cancel

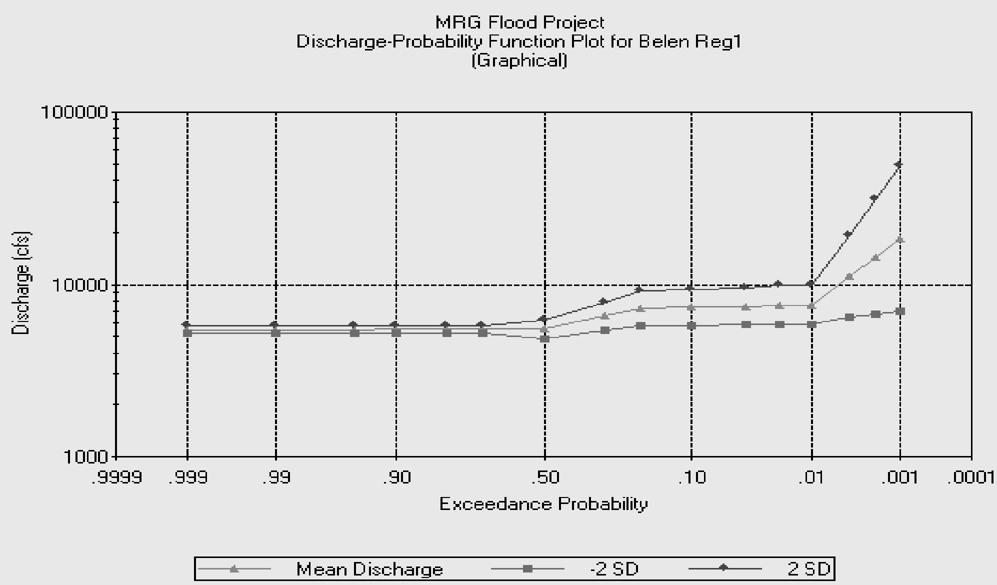
Type: Analytical... / Graphical... (selected)

Function Statistics... — Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,451.84	5,214.77	5,331.99	5,574.39	5,699.69
0.9900	5,463.46	5,226.84	5,343.84	5,585.76	5,710.80
0.9500	5,473.85	5,237.62	5,354.43	5,595.93	5,720.73
0.9000	5,479.39	5,243.37	5,360.09	5,601.36	5,726.04
0.8000	5,486.12	5,250.34	5,366.93	5,607.95	5,732.48
0.7000	5,490.97	5,255.37	5,371.88	5,612.70	5,737.13
0.5000	5,499.00	4,864.70	5,172.14	5,846.52	6,216.00
0.3000	6,541.60	5,407.38	5,947.51	7,195.03	7,913.73
0.2000	7,266.00	5,741.00	6,458.65	8,174.28	9,196.09
0.1000	7,332.00	5,763.88	6,500.83	8,269.44	9,326.74
0.0400	7,455.81	5,804.40	6,578.49	8,450.14	9,577.06
0.0200	7,571.00	5,839.57	6,649.16	8,620.64	9,815.81
0.0100	7,573.00	5,840.16	6,650.38	8,623.62	9,820.00
0.0040	11,153.07	6,440.78	8,475.52	14,676.49	19,313.01
0.0020	14,460.00	6,745.36	9,876.13	21,171.41	30,997.82
0.0010	18,448.06	7,018.74	11,379.03	29,908.61	48,488.89

Exceedance Probability Function Plot

File Help



MRG Flood Project - Exceedance Probability Functio...

File Edit View Help

Plan: With Project 1 Stream: Rio Grande

Analysis Year: 2008 Damage Reach: 8 - Belen RR

Function: BelenRRReg1 | Use An Existing Function | Save

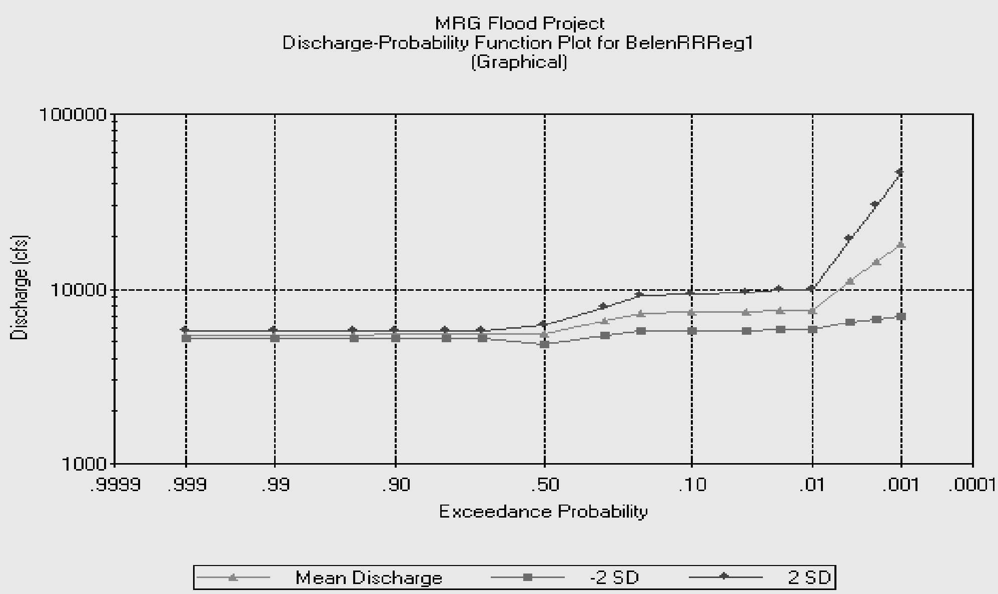
Description: Routed from Central Bridge to Belen RR with flo-2d | Cancel

Type: ( ) Analytical... (•) Graphical... | Function Statistics... | Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.9990	5,452.83	5,216.19	5,333.20	5,575.15	5,700.22
0.9900	5,464.46	5,228.27	5,345.06	5,586.52	5,711.31
0.9500	5,474.85	5,239.05	5,355.65	5,596.69	5,721.25
0.9000	5,480.39	5,244.81	5,361.30	5,602.12	5,726.56
0.8000	5,487.11	5,251.78	5,368.15	5,608.71	5,733.00
0.7000	5,491.97	5,256.81	5,373.10	5,613.46	5,737.65
0.5000	5,500.00	4,866.83	5,173.74	5,846.84	6,215.55
0.3000	6,540.37	5,408.17	5,947.39	7,192.47	7,909.58
0.2000	7,263.00	5,740.22	6,456.87	8,169.76	9,189.74
0.1000	7,331.00	5,763.70	6,500.28	8,267.88	9,324.50
0.0400	7,453.81	5,803.68	6,577.20	8,447.25	9,573.11
0.0200	7,569.00	5,838.67	6,647.77	8,617.90	9,812.14
0.0100	7,572.00	5,839.54	6,649.59	8,622.37	9,818.44
0.0040	11,070.47	6,425.91	8,434.33	14,530.53	19,072.05
0.0020	14,196.00	6,715.32	9,763.74	20,640.27	30,009.96
0.0010	17,925.46	6,974.26	11,181.09	28,738.00	46,072.64

Exceedance Probability Function Plot

File Help



4.0. Hydraulic Analysis

4.1. Overview of Hydraulic Analysis. The hydraulic analysis is part of evaluating the potential of flooding and proposed actions to alleviate high water conditions. It is used in conjunction with the sediment analysis. Specific applications for the hydraulic analysis in the project area include:

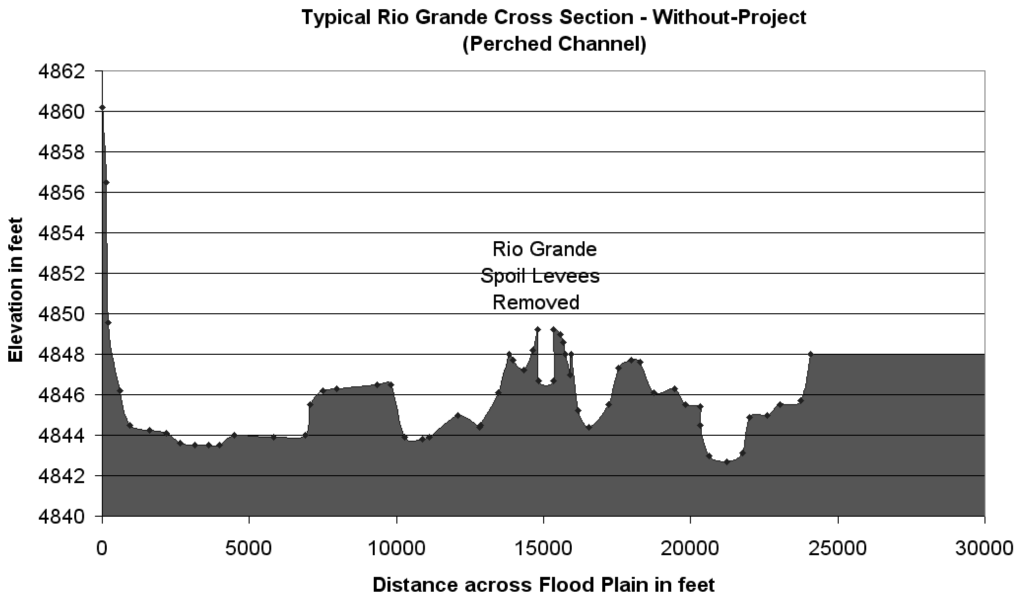
- With and without project floodplains
- Input to economic analysis
- Input to risk assessment, used to determine damage-frequency relationships and design parameters such as levee heights
- Evaluation of design for the proposed project
- Evaluation of environmental impacts of the proposed project
- Estimate damages induced by proposed project

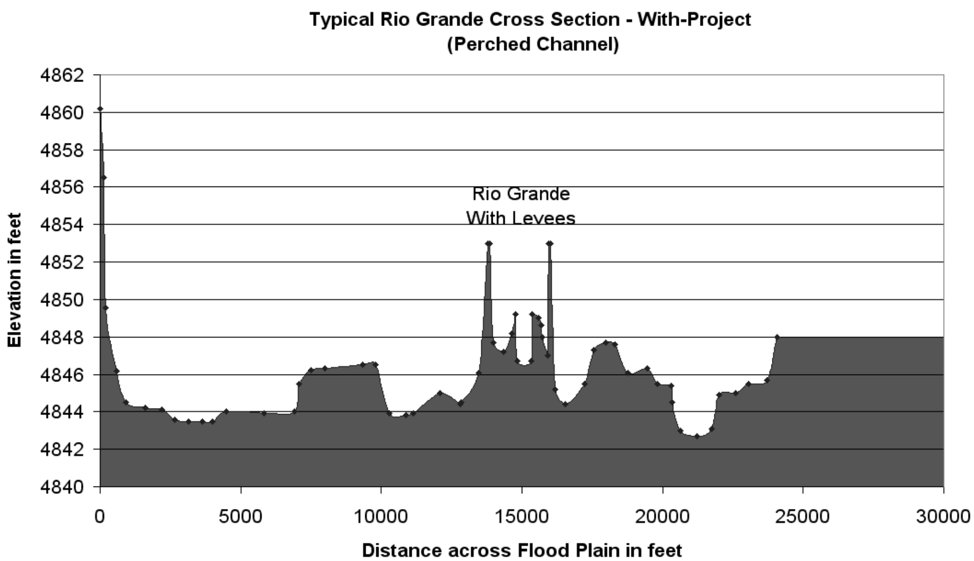
Two numeric models were used for the hydraulic analysis. They are:

- HEC-RAS, the River Analysis System, was used to analyze hydraulics at bridges and will be used to aid in the with-project analysis and design.
- FLO-2D, a 2-dimensional hydraulic model, was used for the hydrologic routing and for the without-project analysis. It was also be used for the with-project hydrologic routing and design.

The Flo-2D Model was used to develop the inundation mapping and provide inputs into the FDA Analysis for the without- project conditions (which included generation of the uncertainty for the without-project conditions). The Flo-2D Model was also used for routing hydrographs through the system for both with- and without- project conditions to determine flows at index points for the various project reaches. The Flo-2D Model had the ability to analyze the entire valley area as well as the area limited by the levees depending on the values set for levee height in the levee.dat file. The HEC-RAS Model was used for the with- project inputs into FDA for the determination of levee heights (which included generation of the uncertainty for the with-project conditions). The HEC-RAS Model was limited to the with- project condition for two reasons. The HEC-RAS cross sections did not extend beyond the levees and the HEC-RAS model did not attenuate hydrographs routed downstream. However, the HEC-RAS Model is approved for design (levee height determination) and is better suited for the with- project analysis once the flow is defined within the various project reaches.

Model scenarios represent both with-project and without-project conditions. Both present and future conditions models are required for the with-project and without-project scenarios. The future conditions model represents the channel and floodplain 50 years into the future. Development of future conditions models is addressed in the section on sediment. Typical Rio Grande Cross Sections are shown below for both the With-Project Condition and the Without-Project Condition. The cross sections also show the perched channel condition of the Rio Grande above the surrounding floodplain.





Mapping is used to represent the terrain for the hydraulic models. All mapping was converted to vertical datum NAVD 1988 and horizontal datum New Mexico State Plane Central 1983.

- The FLO-2D model extends from Cochiti Dam to Elephant Butte Reservoir and was previously developed by the Corps to support the Upper Rio Grande Water Operations Planning Study (URGWOPS) (Tetra Tech, 2004). While the overall model extends from Cochiti Dam to Elephant Butte Reservoir, only the reach needed for this study was used.
- Cross sections for the models are surveyed. The most recent surveys have been supplied by various Federal and local agencies, who have shared their data freely with other agencies.
- A Corps project to study the Albuquerque levees, completed in 2007, contributed HEC-RAS models and surveys through Albuquerque to the Isleta Pueblo.
- A HEC-RAS model was developed from US bureau of Reclamation Range Line Data. A comparison was made between the elevations at specific locations along the project reach given in NGVD29 as compared to NAVD88 (using Corpscon6.0). The difference in

elevation (generally +2.4 to +2.5 for NAVD88) was then added for the appropriate reach in the HEC-RAS Model. The conversion in the HEC-RAS Model was made using the datum adjustment tool in the geometric data window. The cross sections used in the HEC-RAS Model were developed from LiDAR Survey on a date when the Rio Grande was flowing at approximately 300 cfs. Therefore, the bottom of the channel shown is the water surface at 300 cfs. Since the comparison of measured flows with the modeled flows is favorable, it was determined that no adjustment to the cross sections were necessary. Additionally, large design flows will render this low flow condition of 300 cfs to be insignificant. This is also a likely base flow condition that would be present at the onset of a large storm event. See the comparison of modeled flows versus measured flows given below.

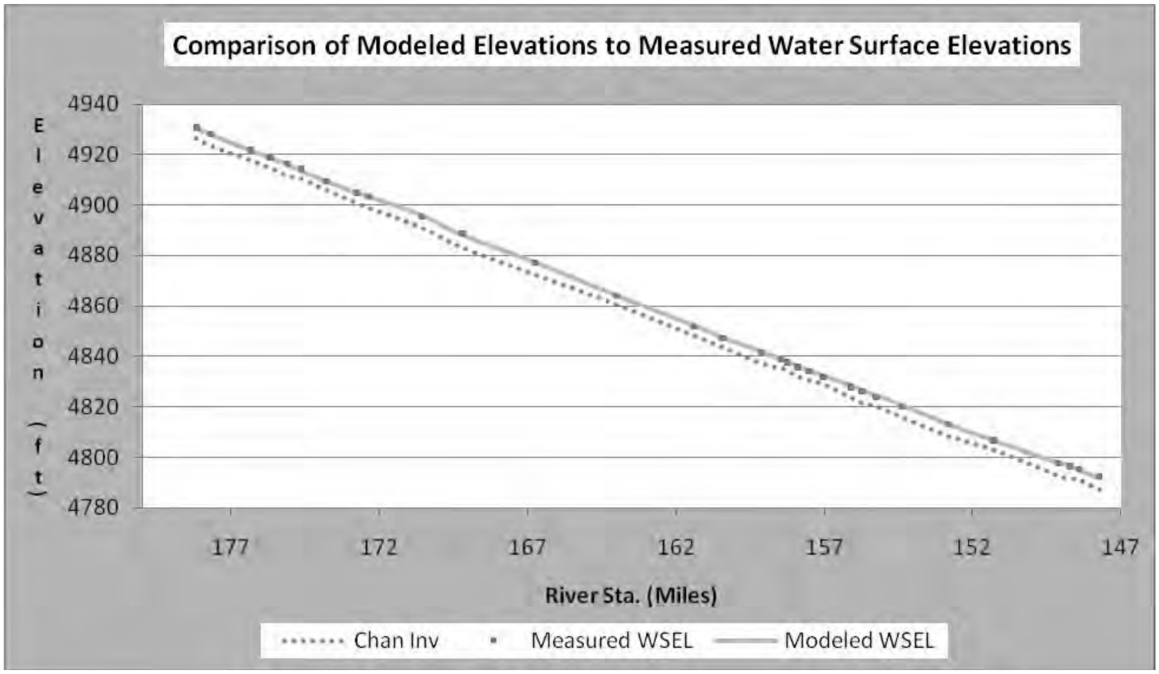
- A 250 foot Grid FLO-2D Model was developed in 2007 for the Upper Rio Grande Water Operations Planning Study which included recently surveyed sections and updated rating curves at bridges that were used for this study.

TABLE 41

Middle Rio Grande Comparison of Modeled Elevations to Measured Water Surface Elevations of May 2005 (NAVD 88) - Flow approx. 5500 cfs to 6000 cfs

Approximate River Sta. (HEC-RAS)	Rangeline	Endpoint	Date Surveyed	Endpoint Elevation	Endpoint Reading	WSEL Reading	Measured WSEL Elevation	Modeled WSEL Elevation	Comparison of Modeled & Measured WSEL's	Approximate Location
178.18	CO-37	LEP	5/25/2005	4931.33	3.87	4.53	4930.67	4930.49	-0.18	Rio Bravo Bridge
177.69	AQ-567	REP	5/26/2005	4929.32	6.59	7.65	4928.26	4927.81	-0.45	
176.32	AQ-582	LEP	5/27/2005	4922.28	5.5	6.01	4921.77	4921.35	-0.42	Tijeras Arroyo
175.68	AQ-589	LEP	5/27/2005	4919.03	4.35	4.53	4918.85	4918.59	-0.26	
175.12	AQ-595	LEP	5/27/2005	4916.72	10.23	10.49	4916.46	4915.9	-0.56	
174.64	AQ-600	LEP	5/27/2005	4914.07	6.13	5.89	4914.31	4913.35	-0.96	
173.8	AQ-608.5	REP	5/26/2005	4909.47	0	-0.08	4909.55	4909.31	-0.24	
	CO-38	REP	5/26/2005	4908.57	10.52	11.76	4907.33			
172.75	AQ-621	LEP	5/27/2005	4904.23	0	-0.7	4904.93	4904.62	-0.31	
172.37	AQ-624.5	REP	5/26/2005	4902.79	0	-0.42	4903.21	4903.3	0.09	I-25 Bridge
170.57	AQ-643	LEP	5/27/2005	4895.07	5.2	5.01	4895.26	4895.84	0.58	
169.2	IS-658	LEP	5/27/2005	4887.29	6.93	5.59	4888.63	4887.93	-0.7	Isleta Diversion
166.76	IS-684	REP	5/27/2005	4876	0	-1.09	4877.09	4877.25	0.16	
164.01	CO-713	REP	5/27/2005	4865.87	3.73	5.76	4863.84	4864.1	0.26	
	CO-738.15		05/25/05	4861.14	4.88	13.46	4852.56			Los Lunas Bridge
161.4	IS-741	LEP	5/25/2005	4851.52	5.11	4.93	4851.7	4852.25	0.55	
160.42	IS-752	LEP	5/25/2005	4847.84	5.48	6.06	4847.26	4847.4	0.14	
159.13	CO-765	REP	5/25/2005	4842.68	3.85	4.8	4841.73	4842.01	0.28	
158.46	IS-772	LEP	5/25/2005	4839.23	4.95	5.47	4838.71	4838.8	0.09	
158.27	LL-774	LEP	5/25/2005	4837.96	5.19	5.52	4837.63	4837.83	0.2	
157.91	LL-778	LEP	5/25/2005	4836.15	5.5	5.93	4835.72	4836.1	0.38	
157.52	IS-782	LEP	5/25/2005	4834.31	5.42	5.54	4834.19	4834.43	0.24	
157.04	CO-787	LEP	5/25/2005	4833.78	3.83	5.75	4831.86	4832.4	0.54	
156.12	IS-797	LEP	5/25/2005	4828.29	5.55	6.03	4827.81	4828.49	0.68	Los Chaves
155.73	IS-801	LEP	5/25/2005	4826.53	5.57	5.99	4826.11	4826.44	0.33	
155.25	CO-806	LEP	5/25/2005	4826.02	3.66	5.85	4823.83	4824.53	0.7	
154.41	IS-815	LEP	5/25/2005	4820.6	8.09	8.73	4819.96	4820.81	0.85	
152.8	CO-833	LEP	5/25/2005	4814.27	4.03	5.1	4813.2	4813	-0.2	
151.28	IS-849	LEP	5/25/2005	4806.92	4.93	5.1	4806.75	4806.64	-0.11	
	CO-858.1	REP	5/25/2005	4807.21	10.33	14.15	4803.39			Belen Hwy Bridge
149.1	IS-872	LEP	5/25/2005	4797.68	6.18	6.06	4797.8	4797.87	0.07	
148.71	CO-877	LEP	5/25/2005	4796.26	3.43	3.23	4796.46	4796.99	0.53	Belen RR Bridge
148.4	IS-880	REP	5/25/2005	4795.21	4.65	4.65	4795.21	4794.68	-0.53	
147.71	IS-887	IS-R-44	5/25/2005	4795.54	7.1	10.34	4792.3	4791.79	-0.51	

Modeled Water Surface Elevation Determined with HEC-RAS Model using Manning's n value = 0.030 for Channel Flow



4.2. Without-Project Hydraulic Analysis

4.2.1. Without-Project Hydraulic Models

The Rio Grande in the study area is characterized by setback spoil bank levees that contain the floodway. The setback levees have been in place for more than 50 years, and in that period of time sediment has deposited between them. As a result of these sediment deposits, the floodway has become elevated above the surrounding floodplain. The difference in elevation varies, but the floodway is elevated by as much as 5 to 10 feet above the surrounding floodplain in many locations.

FLO-2D is a 2-dimensional hydraulic model. It was used for without-project conditions because of its ability to evaluate the flooding once flows leave the river channel and move onto the floodplain.

The basis of the FLO-2D model in the project area was a widely used FLO-2D model that was modified to meet the analytic needs of this project. The URGWOPs FLO-2D model is used by several Federal and State agencies for the purpose of evaluating and coordinating reservoir releases. The model extends upstream and downstream of the project area on the Rio Grande. URGWOPs models the spoil bank levee in the project area as a viable levee.

- With-project conditions will include an engineered levee roughly in the alignment of the spoil bank levees. Therefore the with-project FLO-2D routing scenario is very similar to the URGWOPs scenario, because it represents existing conditions and with a levee in place. The URGWOPs FLO-2D model with the spoil bank levees removed was the basis for the without-project model. The Albuquerque District USACE, Geotechnical Branch (USACE 2000) made the determination that the Probable Failure Point (PFP) is designated as the elevation of a point at the toe of the existing levee just above the point where the water first breaks out of the river channel.
- The FLO-2D URGWOPs model included Manning's n values based on field observations. It was calibrated using the best available flood data.

Because the URGWOPs model is not used for high flows, it was not clear at the outset that the extent of the floodplain data for the URGWOPS model, represented by a grid, would be adequate to model flood flows. Mussetter Engineering, Inc., was contracted to evaluate whether the grid was adequate, or would need to be expanded to use for the Albuquerque to Belen reach of the Rio Grande. Mussetter Engineering concluded that the present URGWOPs grid was adequate. Their report is given in the H&H Appendix as Attachment 3.

Other updates to the FLO-2D model are:

- The channel data were updated using the most recent cross section data in the Isleta reach, downstream of the I-25 Bridge and upstream of the bridge at Los Lunas.
- The hydraulic structures data were updated using rating curves from the most recent HEC-RAS and FLO-2D models.
- Two separate channel and floodplain datasets were produced representing present and future conditions. Estimates of projected sediment aggradation and degradation for a future conditions scenario are shown in the sediment section of this report. The channel data and floodplain data (within the floodway only) were adjusted using the future conditions aggradation and degradation estimates, in order to produce the future conditions scenario.
- Infiltration losses are not included.
- Evaporation losses are not included.

Due to these updates, flows found in the hydraulic analysis do not precisely match those given previously in the Hydrology Analysis. However, the magnitudes are very similar and do not affect the conclusions. Most of these updates were made by Tetra-Tech under contract to the Albuquerque District USACE (Tetra Tech 2006).

4.2.2. Results of FLO-2D Flood Routing for the Without Project Condition

Snowmelt flooding is controlled, for the most part, by reservoirs. Reservoir releases from Cochiti Dam resulting from snowmelt flooding typically occur as a steady flow in the Rio Grande that can take place over a period of months. Present guidance for the magnitude of these reservoir releases is 7,000 cfs, though it has been higher at times in the past. The steady long-term portion of snowmelt floods has no significant attenuation.

Spillway flow can also result from snowmelt floods coming from upstream of the reservoirs, and is expected to begin at approximately the 0.5%-chance flood event. Spillway flow occurs in addition to reservoir releases, but unlike reservoir releases the flow is not controlled.

Routing of rainfall runoff events from non-regulated areas results in significant attenuation through the study reach. One factor leading to the high amount of attenuation for the rainfall-runoff events is the relatively low volume of the high peak hydrographs. That is not the case for spillway flow. Attenuation for spillway flow is gradual, where as for rainfall-runoff flooding attenuation is dramatic.

Another of the causes for the attenuation is the large volume of storage available in the channel, which is wide and shallow. Widths typically range from 500 feet to 600 feet, with flow depths on the order of 4 feet for the 10%-chance flood event.

Attenuation is also related to the large amount of flow in the floodplain and overbanks. There is significant storage in the overbanks, even for the with-project model. Overbank flow, because of vegetation in the overbanks, is slower than channel flow, and delays the portion of the flood peak that is not carried in the channel, thus reducing flood peaks. Table 42 gives the results of routing flows for the 0.2%-chance event through the project for the various flow scenarios discussed. These flows were modeled in 2008 during the study and were used in developing the damage assessment. After the existing condition work had been completed the Albuquerque West Levee was constructed. It is a three mile reach of levee located on the west side of the Rio Grande from approximately the SDC downstream to I-25 and across the river from the Mountain View Reach. Table 43 gives the results of routing flows for the 0.2%-chance event through the project for the same flow scenarios as in Table 42; however Table 43 includes the effects of the Albuquerque West Levee as being in place. Following a review of all flow frequency results, it was determined the effect of the Albuquerque West Levee would not change the damage assessment analysis when considering the controlling flow scenarios.

Table 42 – Results of Routing Flows for the 0.2%-Chance Event for the Without Project condition without the Albuquerque West Levee

X-sec #	Location	Regulated Flow	Unregulated Area Below Cochiti	South Diversion Channel Flow
1 (Gage)	Below Central Ave. Bridge Albuquerque	14305	26086	N/A
2	Above Rio Bravo Bridge, Albuquerque	13905	21713	N/A
3	Below Tijeras Arroyo confluence	14157	21600	31221
4	Above I-25 Bridge	14553	11007	10012
5	Below Isleta Bridge (Rt. 147) @ Isleta Diversion	14802	8290	6219
6	Bosque Farms	14206	6172	3191
7	Bridge at Los Lunas	14165	4591	3160
8	Los Chaves	14122	4588	3110
9	Bridge at Belen (Rt. 309)	13895	4569	3039
10	Belen RR Bridge	14100	4562	3006

Table 43 – Results of Routing Flows for the 0.2%-Chance Event for the Without Project condition with the Albuquerque West Levee in place

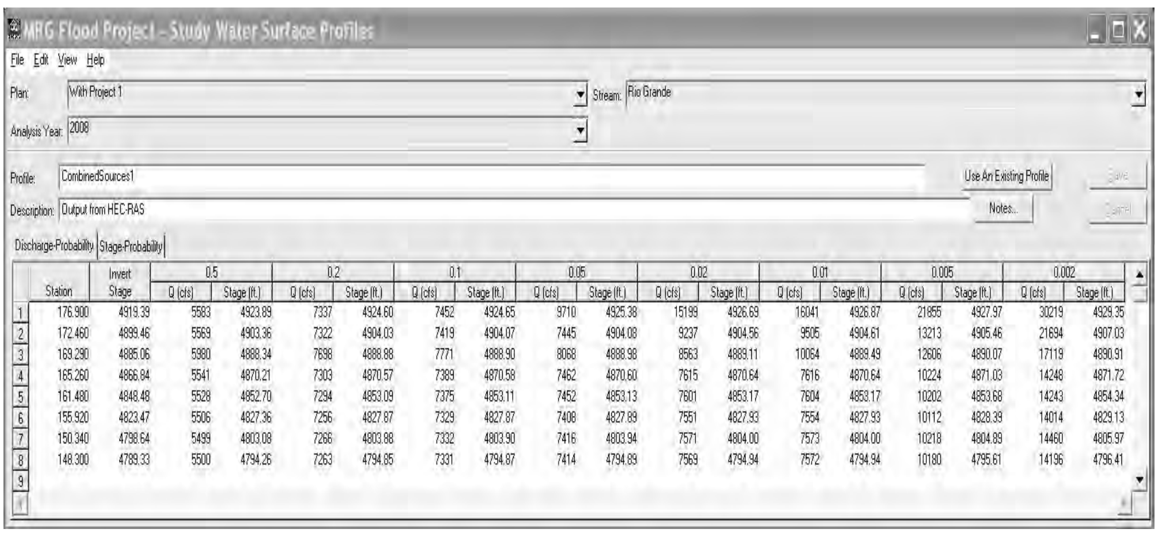
X-sec #	Location	Regulated Flow	Unregulated Area Below Cochiti	South Diversion Channel Flow
1 (Gage)	Below Central Ave. Bridge Albuquerque	14408	26051	N/A
2	Above Rio Bravo Bridge, Albuquerque	14278	22508	N/A
3	Below Tijeras Arroyo confluence	14280	21179	34721
4	Above I-25 Bridge	14550	11744	12480
5	Below Isleta Bridge (Rt. 147) @ Isleta Diversion	14217	8426	6786
6	Bosque Farms	14185	6631	4255
7	Bridge at Los Lunas	14152	5075	3982
8	Los Chaves	14106	4841	3942
9	Bridge at Belen (Rt. 309)	14210	4751	4018
10	Belen RR Bridge	14102	4708	3911

4.3. With-Project Hydraulic Analysis

The location of the existing Rio Grande setback spoil bank levees described above provided the basis for which the alignment was determined for the With-Project engineered levees used in this analysis. The FLO-2D 2-dimensional hydraulic model described previously was also used for the With-Project hydraulic analysis in order to determine the attenuation of the flood hydrograph with the engineered levees in place. The flows determined from FLO-2D at each cross section were then used in the HEC-RAS Model to determine water surface elevations. The HEC-RAS Model was also used for determining the With-Project standard deviations for hydraulic risk. This effort was conducted for both the current With-Project Conditions (2008) and the future With-Project Conditions (2058). Results of With-Project hydraulic risk is presented in Section 6.2 of this report.

The With-Project HEC-RAS model for the expected condition uses a Manning's n = 0.030 for the active channel and a Manning's n = 0.10 for the left and right overbank areas from the channel banks to the levees on either side. The results were then input into the HEC-FDA program at each section for all eight frequencies. The water surface elevations for the current With-Project Conditions (2008) at the index locations are shown below.

For each of the flood frequencies that were considered, a separate project was developed to determine the elevation to achieve a 90% Conditional Non-Exceedance Probability Levee Height for that event. This analysis was conducted for both the current With-Project Condition (2008) and the future With-Project Condition (2058) and is included in the HEC-FDA Risk Analysis Program (see Attachment 7 of Appendix H).



	Station	Invert Stage	0.5		0.2		0.1		0.05		0.02		0.01		0.005		0.002	
			Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)	Q (cfs)	Stage (ft.)
1	176.900	4919.39	5583	4923.89	7337	4924.60	7452	4924.65	9710	4925.38	15199	4926.69	16041	4926.87	21855	4927.97	30219	4929.35
2	172.460	4899.46	5569	4903.36	7322	4904.03	7419	4904.07	7445	4904.08	9237	4904.56	9505	4904.61	13213	4905.46	21694	4907.03
3	169.290	4885.06	5980	4888.34	7698	4888.88	7771	4888.90	8068	4888.98	8563	4889.11	10064	4889.49	12606	4890.07	17119	4890.91
4	165.260	4866.84	5541	4870.21	7303	4870.57	7389	4870.58	7462	4870.60	7615	4870.64	7616	4870.64	10224	4871.03	14248	4871.72
5	161.480	4848.48	5528	4852.70	7294	4853.09	7375	4853.11	7452	4853.13	7601	4853.17	7604	4853.17	10202	4853.68	14243	4854.34
6	155.920	4823.47	5506	4827.36	7256	4827.87	7329	4827.87	7408	4827.89	7551	4827.93	7554	4827.93	10112	4828.39	14014	4829.13
7	150.340	4798.64	5499	4803.08	7266	4803.88	7332	4803.90	7416	4803.94	7571	4804.00	7573	4804.00	10218	4804.89	14460	4805.97
8	148.300	4789.33	5500	4794.26	7263	4794.85	7331	4794.87	7414	4794.89	7569	4794.94	7572	4794.94	10180	4795.61	14196	4796.41
9																		

5.0. Sediment Analysis

A robust design of the proposed engineering levee needs to consider the dynamic and complex interchange of processes that occur on a fluvial system. This understanding helps ensure that the engineered levee system will be able to accommodate reasonably predicted channel adjustments, especially within an alluvial system such as the MRG. There are risks, however, with introducing a static component into a dynamic system. Understanding the historical and current channel conditions and dynamics of a river and the anthropogenic influences (including this levee) on the system are important in developing a robust design and minimizing risk. Evaluating the potential for loss of channel/floodway conveyance, increased frequency of channel overbanking (putting more water against the riverside toe of the engineered levee), and potential channel migration towards the engineered levee are all risks that can be mitigated to some extent when working with fluvial systems by understanding the historical and current channel conditions and dynamics.

The MRG is a dynamic and complex alluvial system where flow and sediment transported from the Upper Rio Grande and MRG tributaries influence the observed form of the river. This flux of water and sediment (magnitude, duration, and frequency) is tempered by bank and bed stability, base level changes, floodplain lateral confinement, and floodplain connectivity, which in turn influences how much, when, and where water and sediment are transported or stored within and through the fluvial system (Leopold et al. 1992, MEI 2002, Charlton 2008, Davies and Korup 2010, Makar and AuBuchon 2012). A detailed analysis for the MRG Bernalillo to Belen project is provided in Attachment 6 of Appendix H and summarized briefly in the sections that follow.

5.1. MRG Conditions and Dynamics

The MRG is primarily a snow-melt influenced fluvial system, but strong monsoonal patterns in the summer and fall bring additional rainfall-runoff that has the potential to influence the morphology. Both anthropogenic and climatic influences have affected the MRG through the decades by influencing both the discharge of flow and sediment on the MRG. Historical conditions have resulted in a significant narrowing of the MRG's floodplain and created a perched channel condition above the historic floodplain. Both the sediment supply and peak flow conditions have been reduced since the early part of the twentieth century and precipitated geomorphic changes within the floodway (includes the active channel and the floodplain within the currently existing spoil levees.

The project reach was divided into four subreaches, with one subdivision of a subreach, based on common morphological characteristics to further evaluate the observed morphological adjustments occurring on the MRG. Subreach 1 is upstream of the project area and ends around the confluence of the Rio Grande with the AMAFCA South Diversion Channel. Subreach 2 extends from subreach 1 and ends around the Isleta Diversion Dam. Subreach 3 was divided into two subdivisions, a and b. Subreach 3a ends just north of Los Lunas, NM and Subreach 3b ends north of Belen, NM. Subreach 4 extends to around Casa Colorado, NM. Main findings of the current morphological adjustments on the MRG are summarized in the following bullets:

- Peak flow conditions (magnitude, and frequency) have been reduced since the 1970s. Low flow conditions (magnitude and frequency) have increased during the same time period.
- Sediment load has decreased since the early twentieth century. Since the 1990s there has been a slight increase in the suspended sediment load.
- Channel gradient throughout the MRG Bernalillo to Belen project area has seen a slope reduction.
- Channel bed material has generally coarsened, but is still primarily sand throughout the MRG Bernalillo to Belen project reach. Subreaches 1, 2, and 3a have a higher percentage pf gravel found in the bed than subreaches 3b and 4.
- Channel width has narrowed for all subreaches. Subreach 1 hasn't experienced the same level of narrowing as the other subreaches.
- Planform changes have shown a shift from a braided system to more of a meandering system, indicating that the meander wavelength may be decreasing.
- Channel depth is currently trending towards decreasing depths within subreaches 1, 2, and 3a and increasing depths within subreaches 3b and 4.
- Sinuosity is generally increasing throughout the MRG Bernalillo to Belen project area, with a slightly higher sinuosity occurring in subreach 1.

5.2. Future Channel Conditions and Dynamics

The long term sediment load reduction and the peak flow reduction are primary drivers of the observed geomorphic change on the MRG within the project reach. These trends have persisted through several decades and are expected to continue into the future time frame associated with the project. Because of the strong width reduction, the depth trend may be masked, but would be expected to remain constant or potentially decrease over the long term. This suggests that the recently observed depth trends may oscillate between degradation and aggradation as the MRG planform adjusts. The increase in sinuosity would be expected to be manifested in a reduction in slope as well, since the meandering increases the channel length. The loss of higher peak flows and increased frequency of low flows is expected to increase vegetation cover within the floodway. This may trigger a positive feedback loop with channel incision, which in turn may increase the potential for lateral migration due to the increased bank height.
Based on an assessment of average channel degradation an average rate of 0.02 feet per year of aggradation throughout the project reach is recommended. This is roughly one foot of deposition within the floodway over the next 50 years.

An assessment of future conditions facilitates a qualitative assessment of risks associated with constructing a static structure within a dynamic system. Three potential risks for the project are: loss of channel/floodway conveyance, increased frequency of channel overbanking, and potential channel migration towards the constructed engineered levee. A summary and proposed mitigation associated with these risks is summarized in the following bullets:

- *Loss of channel/floodway conveyance*—Incorporation of an additional height on the proposed engineering levee that is equivalent to the estimated aggradation that potentially could be experienced within the MRG over the design life of the project helps mitigate this risk. A proposed value of one foot is recommended.
- *Increased frequency of channel overbanking*—There is a cyclical risk associated with waves of deposition and incision that may occur as the MRG continues to adjust through the MRG Bernalillo to Belen project area. The seepage risk is highest in subreaches 2, 3a, and 3b. Adding additional height and incorporating seepage control into the engineered levee helps mitigate this risk. These geomorphic conditions also create opportunities for potential habitat restoration.
- Potential channel migration towards the constructed engineered levee—The geomorphic evaluation of the project reach suggests that there is a risk for potential channel migration. This risk is highest in subreaches 3b and 4. Identifying areas prone to channel migration and incorporating appropriate erosion protection will help mitigate this risk.

5.3. Conclusions

The channel and overbank elevations were relatively stable for the entire reach. Within the project reach, subreaches 3b and 4 were slightly degradational, therefore exclusion of degradation is recommended for future conditions models. This would also be the more conservative approach for evaluating flooding scenarios.

Based on an assessment of average channel degradation an average rate of 0.02 feet per year of aggradation throughout the project reach is recommended. This is roughly one foot of deposition within the floodway over the next 50 years.

6.0. Risk and Uncertainty. A primary purpose of the hydraulic risk analysis is to estimate variability of the water surface. High, expected and low flow scenarios were developed for the without-project hydraulic models. The reaches described below were used as locations to evaluate the standard deviation of the water surface elevations. These standard deviations can then be used in the economic risk evaluation. Standard Deviations were calculated using Equation 5-6 of EM 1110-2-1619. The term "S sub model" in Equation 5-6 was calculated using Equation 5-7 of EM 1110-2-1619. The term "S sub terrain" is calculated using the following equation:

$$SD = 0.0657 \times S_0^{0.592} \times S_n^{0.738}$$

Where:
SD = Standard Deviation (ft) of the error in the computed water surface elevation due solely to the accuracy of the terrain data and the slope of the stream.
S_0 = Slope of the stream in ft/mile.
S_n = The standardized survey accuracy being analyzed – the contour interval 2-, 5-, 10-feet divided by 10. (i.e. if the data is considered to be accurate to the 2 foot contour interval then S_n = 2/10 = 0.2).

The Rio Grande stream slope = 4.6 ft/mile & contour interval = 4 ft. The 1% chance present condition models were used for the risk analysis.

6.1. Without-Project Hydraulic Risk. Hydraulic parameters in the 1% chance present condition FLO-2D model were varied to estimate variability of water surface elevations. Table 44 shows the modifications that were made to develop high and low risk scenarios. The without-project model is the expected scenario risk model. Results of the risk analysis for the channel are shown in Table 45. Results for the floodplain are shown in Table 46 & Table 47.

The sediment plug in the Tijeras Arroyo has as its basis a similar occurrence that took place a short distance upstream at the confluence of the Rio Grande with the Calabacillas Arroyo in the summer of 1988. During a sever summer thunderstorm, a large volume of sediment was washed down the Calabacillas Arroyo into the confluence as an alluvial deposit that filled in the Rio Grande channel for some distance. This same result could occur at the Tijeras Arroyo confluence under similar conditions. In a report entitled, "Sediment-Transport and Scour Analysis of Tijeras Arroyo for Design of University Boulevard Bridge Crossing" conducted by Mussetter Engineering, Inc. dated December 2004, the quantity of sediment available just upstream of the confluence was estimated. This volume was used to simulate the sediment plug for the risk analysis. The referenced report is included as Attachment 5.

As shown in the results below, the effects of the sediment plug are limited to cross sections 3, 4 and 5. It is interesting to note that the "Low n-value Risk Scenario" does not always result in the lowest water surface elevation. This is likely an effect of routing whereby the system more efficiently delivers greater flow downstream. This results in higher flow rates and greater water surface elevations across certain cross sections. Another result which requires some explanation occurs at X-section 4 for the "High Scenario" in Table 45. The value given for "w plug" is one foot lower than the value for the "Expected Scenario". This occurred due to the diversion of flow into the floodplain in the previous section where the plug was introduced. Since the diverted flow did not all return to the channel, the resulting water surface elevation was lower than would otherwise be expected.

The controlling flood events at the various Cross–sections are as follows:

X–Sect	Location	Controlling Flood Event
1.	(Central Ave. Bridge, Alb.)	Unregulated Area below Cochiti
2.	(Rio Bravo Bridge, Alb.)	Unregulated Area below Cochiti
3.	(Tijeras Arroyo Confluence)	Tijeras Arroyo
4.	(I-25 Bridge)	Unregulated Area below Cochiti
5.	(Isleta Diversion Structure)	Regulated Flow, Cochiti & Jemez Releases
6.	(Bosque Farms)	Regulated Flow, Cochiti & Jemez Releases
7.	(Bridge in Los Lunas (Rt. 6))	Regulated Flow, Cochiti & Jemez Releases
8.	(Los Chaves)	Regulated Flow, Cochiti & Jemez Releases
9.	(Bridge in Belen (Rt. 309))	Regulated Flow, Cochiti & Jemez Releases
10.	(Belen RR Bridge)	Regulated Flow, Cochiti & Jemez Releases

Table 44 - Hydraulic Parameters Varied for the Risk Analysis Hydraulic Models

Risk Parameter	Risk Scenario		
	Low n-value	Expected n-value	High n-value
Channel n-value	-.005 (n>.015)	.016-.038	+.005
Overbank n-value	-.02	.065/.1	+.02
Sediment	N/A	N/A	Simulated sediment plug at the Tijeras Arroyo Confluence

Table 45 - Risk Analysis Results for the Channel Water Surface Elevations (WSEL)

X-section	Low Scenario	Expected Scenario	High Scenario	Est. Std. Dev.
1	4955.3	4955.8	4956.3	0.3
2	4933.0	4933.4	4933.7	0.3
3	4925.8	4926.2	4926.6 w/o plug	0.3
3	4925.8	4926.2	4928.1 w plug	0.58
4	4903.3	4902.9	4902.6 w/o plug	0.3
4	4903.3	4902.9	4901.9 w plug	0.36
5	4890.4	4890.1	4890.1 w/o plug	0.3
5	4890.4	4890.1	4890.3 w plug	0.3
6	4869.3	4869.4	4869.7	0.3
7	4853.5	4853.1	4852.5	0.5
8	4825.9	4826.1	4826.2	0.3
9	4801.9	4801.8	4801.7	0.3
10	4796.0	4796.0	4796.1	0.3

Table 46 - Risk Analysis Results for the Right (West) Overbank Floodplain WSEL

X-section	Low Scenario	Expected Scenario	High Scenario	Est. Std. Dev.
1	4955.6	4956.2	4956.9	0.34
2	4932.7	4933.0	4933.3	0.3
3	4925.0	4925.1	4925.7 w/o plug	0.3
	4925.0	4925.1	4927.6 w plug	0.66
4	4902.2	4903.1	4903.6 w/o plug	0.36
	4902.2	4903.1	No right overbank Flow with plug	0.3

	4889.2	4889.5	4889.8 w/o plug	0.3
5	4889.2	4889.5	4889.7 w plug	0.3
6	4868.6	4868.8	4869.0	0.3
7	4851.2	4851.8	4852.1	0.3
8	4823.0	4823.4	4823.8	0.3
9	4803.1	4803.4	4803.8	0.3
10	4793.6	4793.8	4794.1	0.3

Table 47 - Risk Analysis Results for the Left (East) Overbank Floodplain WSEL

X-section	Low Scenario	Expected Scenario	High Scenario	Est. Std. Dev.
1	4955.5	4955.9	4956.4	0.3
2	4933.5	4933.9	4934.3	0.3
	4925.5	4925.8	4926.1 w/o plug	0.3
3	4925.5	4925.8	4927.2 w plug	0.43
	4903.6	4903.9	4904.2 w/o plug	0.3
4	4903.6	4903.9	4906.0 w plug	0.61
	4887.0	4887.2	4887.4 w/o plug	0.3
5	4887.0	4887.2	4887.5 w plug	0.3
6	4867.1	4867.3	4867.7	0.3
7	4849.7	4850.0	4850.2	0.3
8	4824.9	4825.3	4825.5	0.3
9	4804.8	4804.9	4804.9	0.3
10	N/A	N/A	N/A	No Overbank Flow

6.2. With-Project Hydraulic Risk

The HEC-RAS Model described previously was used for determining the With-Project standard deviations for hydraulic risk (see Attachment 4). This effort was conducted for both the current With-Project Conditions (2008) and the future With-Project Conditions (2058). Table 48 shows the modifications that were made to develop high and low risk scenarios.

Table 48 - Hydraulic Parameters Varied for the With-Project Risk Analysis in the HEC-RAS Model

Risk Parameter	Risk Scenario		
	Low n-value	Expected n-value	High n-value
Channel n-value	-.005	0.030	+.005
Overbank n-value	-.02	0.10	+.02
Sediment (Range Line Extent)	2008 – None added 2058 – 1' (653-801)	2008 – None added 2058 – 1' (653-801)	Simulated sediment plug fills channel for 2008 & 2058 condition

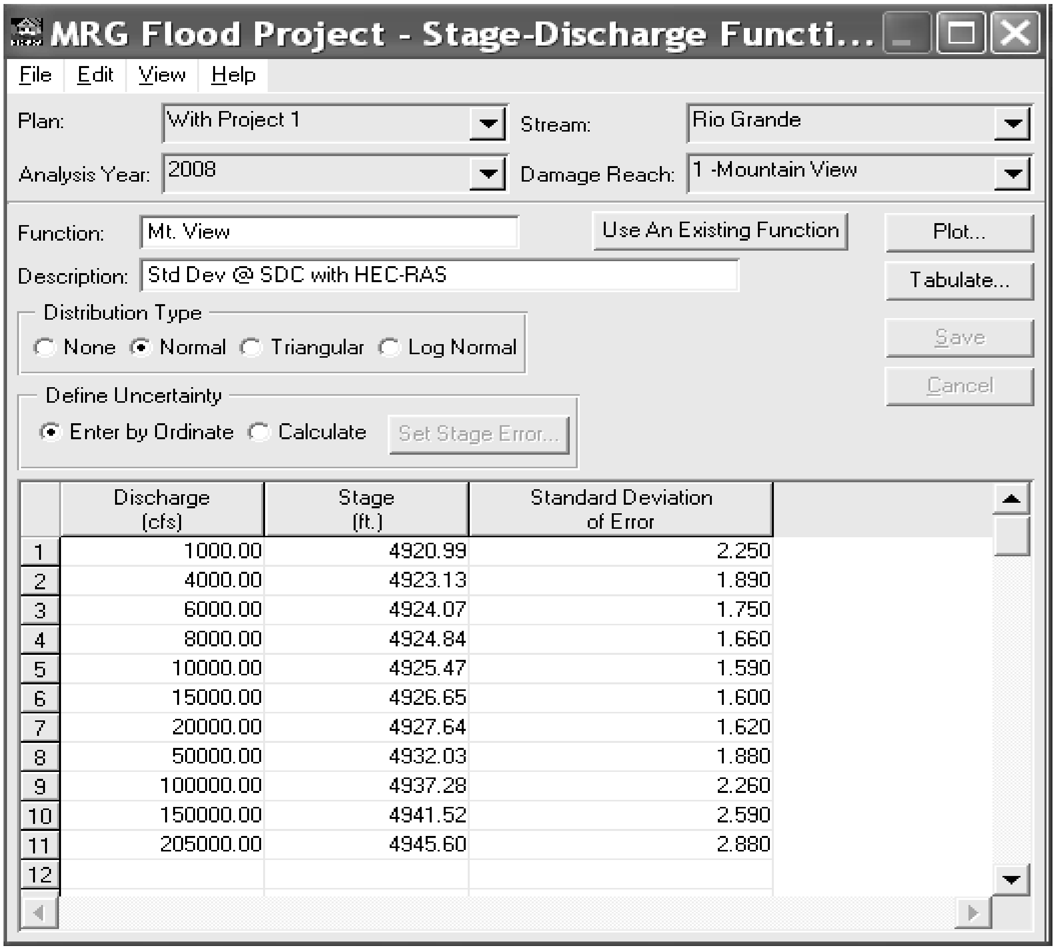
The basis for the Low, High, and Expected n-values given are due to a confined cross section for the Rio Grande Floodway between the levees as shown in the HEC-RAS Model, whereas the without project scenario considered the effects of the entire valley floodplain section in the FLO-2D Model (which varies the n-values much more widely). However, the magnitude of change between Low, High, and Expected n-values for the with-project condition are consistent when compared to the without-project condition. The simulated sediment plug was considered for all sections in the with-project modeling based on recommendations from the Albuquerque District Sedimentation Subject Matter Expert (personal commun., Darrell Eidson, 2009). This was based on recent experience in the San Acacia Project reach of the Rio Grande just downstream from the MRG project reach. In the San Acacia reach, sediment plugs have formed in various locations. Following plug formation, the sediment can subsequently be transported downstream. The sediment condition for the High n-value scenario was handled differently for the with-project condition than for the without-project condition, since the effect of engineered levees will tend to confine flows (and mobilize sediment) within the levee section for large events. Therefore, any sediment deposits are confined within the floodway rather than deposited in the historic valley floodplain under high flow conditions.

The sediment plug which fills the active channel in the high risk scenario has as its basis a similar occurrence that took place a short distance upstream near the confluence of the Rio Grande with the Calabacillas Arroyo in the summer of 1988. During a severe summer thunderstorm, a large volume of sediment was washed down the Calabacillas Arroyo into the confluence as an alluvial deposit that filled in the Rio Grande channel for some distance. This same result could occur at the Tijeras Arroyo confluence under similar conditions and was discussed in the without-project scenario. As discussed above, there are several recent examples of sediment plugs occurring downstream of this reach in the Rio Grande from Bosque Del Apache National Wildlife Refuge to Elephant Butte Reservoir. The most recent occurrences were in the summer of 2007 and the spring of 2009.

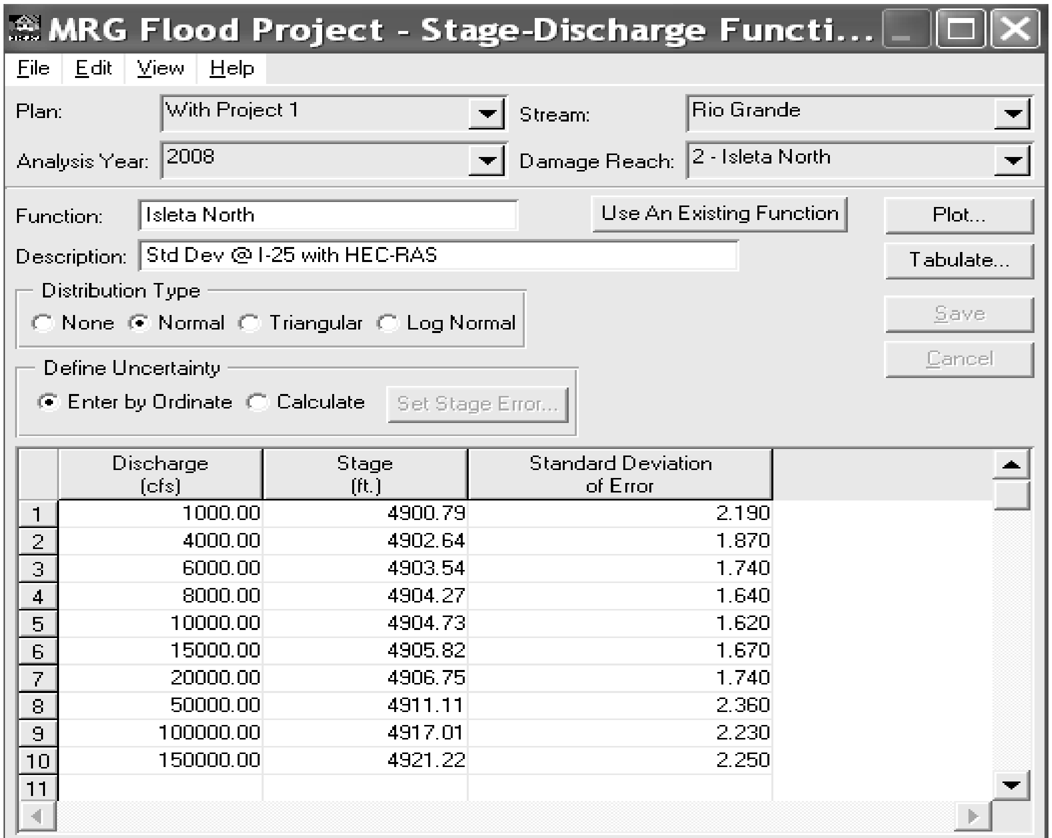
The controlling flood events at most frequencies for the With-Project Condition at the various Cross–sections are as follows:

X–Sect	Damage Reach	Location	Controlling Flood Event
3	1	(Tijeras Arroyo Confluence)	Unregulated Areas below Cochiti & SDC
4	2	(I-25 Bridge)	Unregulated Areas below Cochiti & SDC
5	3	(Isleta Diversion Structure)	Unregulated Areas below Cochiti & SDC
6	4	(Bosque Farms)	Regulated by Cochiti & Jemez Releases
7	5	(Bridge in Los Lunas (Rt. 6))	Regulated by Cochiti & Jemez Releases
8	6	(Los Chaves)	Regulated by Cochiti & Jemez Releases
9	7	(Bridge in Belen (Rt. 309))	Regulated by Cochiti & Jemez Releases
10	8	(Belen RR Bridge)	Regulated by Cochiti & Jemez Releases

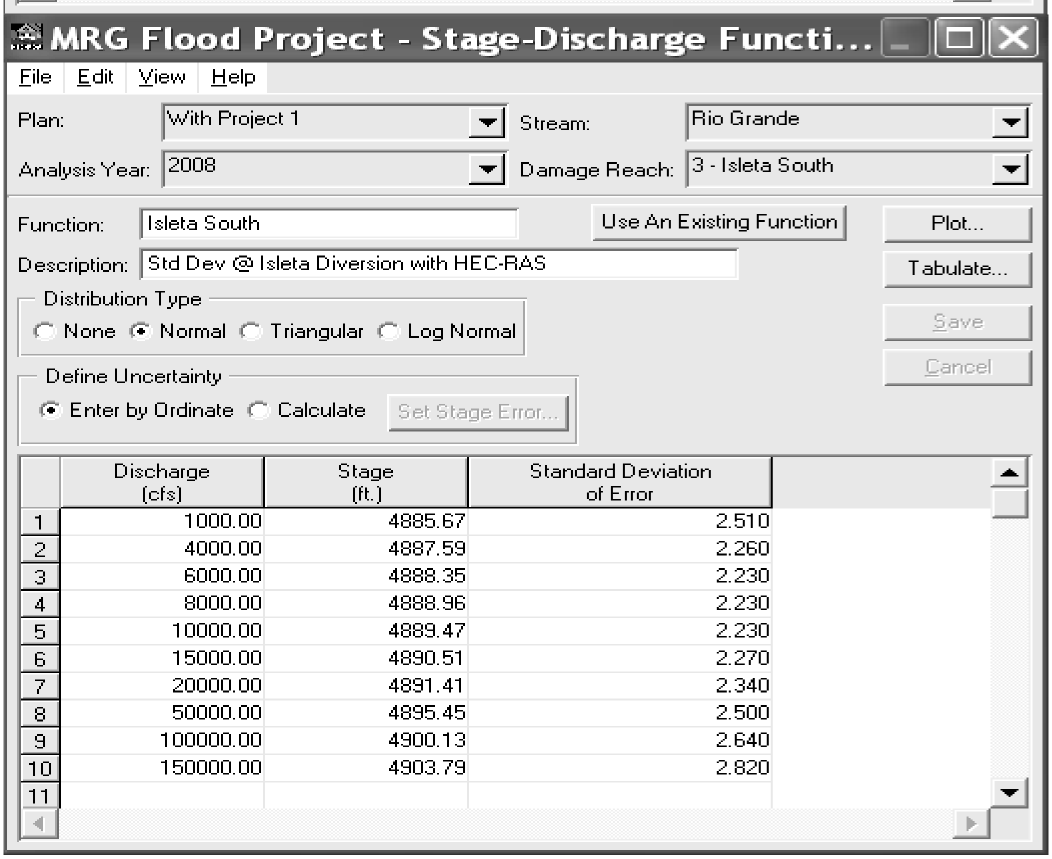
The standard deviations were calculated for each index point in the eight (8) levee reaches for a full range of flows covering the confidence intervals calculated for the discharge-probability curves. The results were then entered into the HEC-FDA program under the “HydEng” tab for “Stage-Discharge Function with Uncertainty” for each damage reach for both the current With-Project Condition (2008) and the future With-Project Condition (2058). The values input for the current With-Project Condition (2008) are shown below.



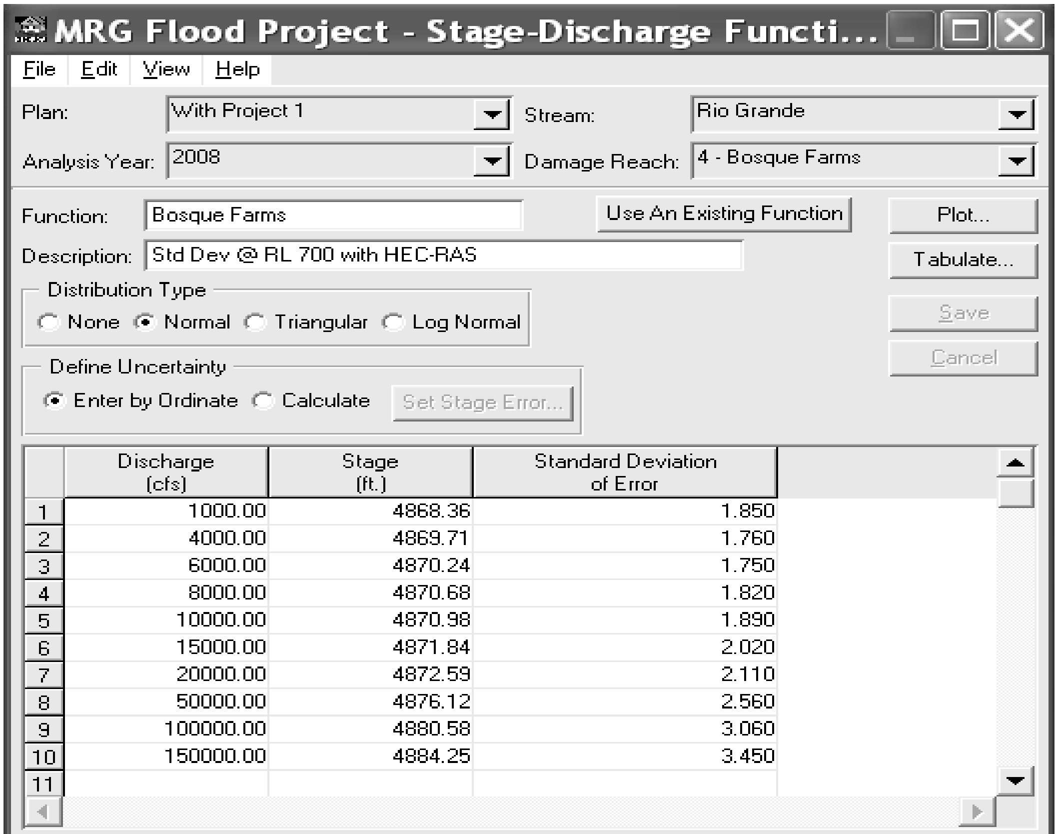
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4920.99	2.250
2	4000.00	4923.13	1.890
3	6000.00	4924.07	1.750
4	8000.00	4924.84	1.660
5	10000.00	4925.47	1.590
6	15000.00	4926.65	1.600
7	20000.00	4927.64	1.620
8	50000.00	4932.03	1.880
9	100000.00	4937.28	2.260
10	150000.00	4941.52	2.590
11	205000.00	4945.60	2.880
12			



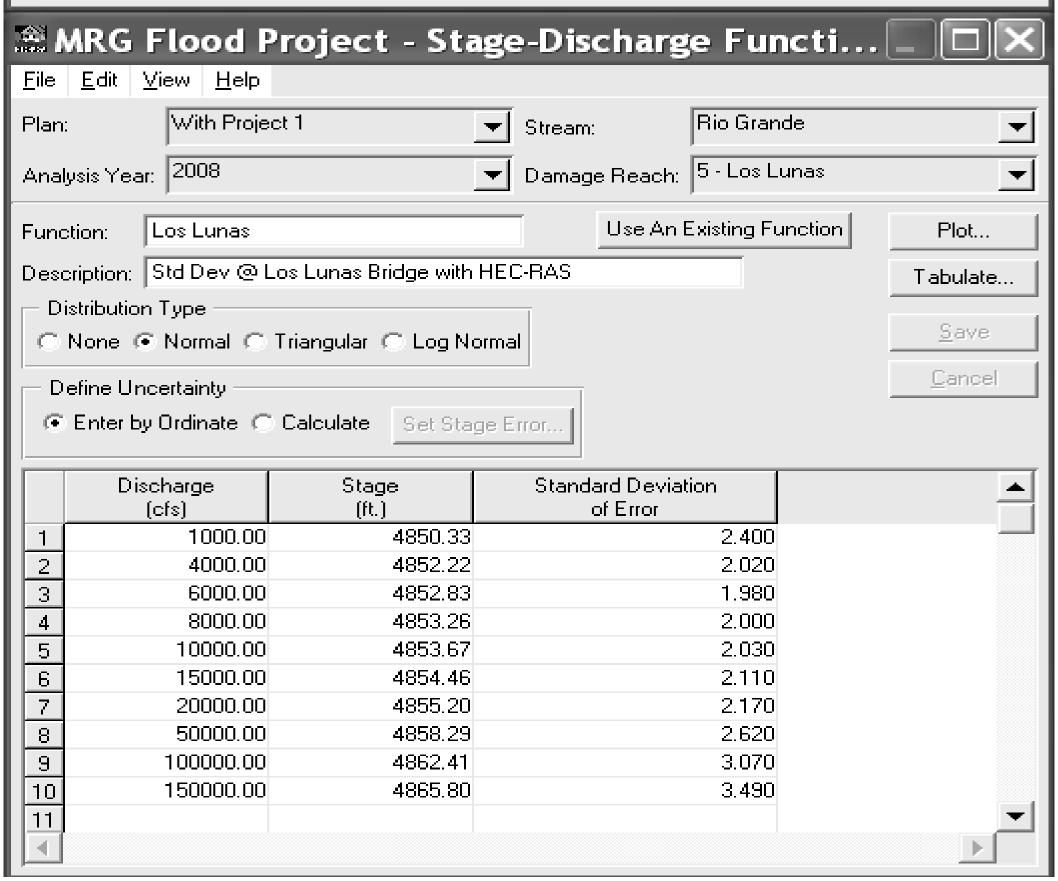
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4900.79	2.190
2	4000.00	4902.64	1.870
3	6000.00	4903.54	1.740
4	8000.00	4904.27	1.640
5	10000.00	4904.73	1.620
6	15000.00	4905.82	1.670
7	20000.00	4906.75	1.740
8	50000.00	4911.11	2.360
9	100000.00	4917.01	2.230
10	150000.00	4921.22	2.250
11			



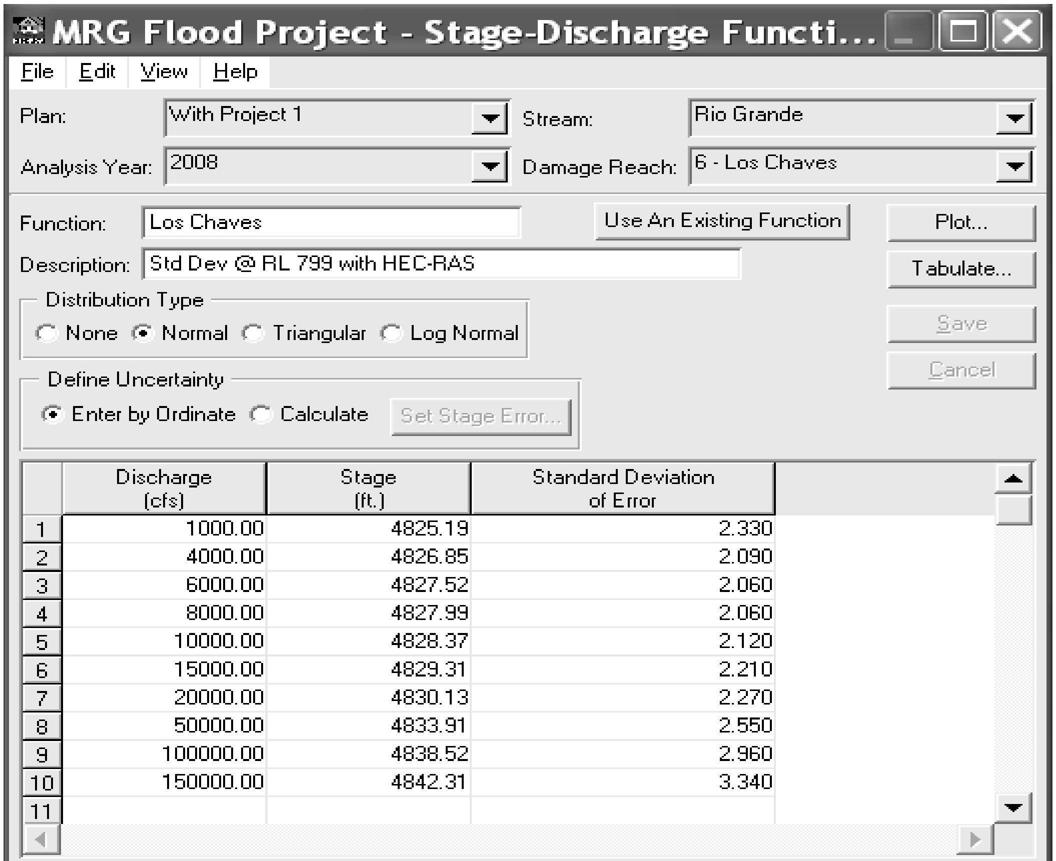
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4885.67	2.510
2	4000.00	4887.59	2.260
3	6000.00	4888.35	2.230
4	8000.00	4888.96	2.230
5	10000.00	4889.47	2.230
6	15000.00	4890.51	2.270
7	20000.00	4891.41	2.340
8	50000.00	4895.45	2.500
9	100000.00	4900.13	2.640
10	150000.00	4903.79	2.820
11			



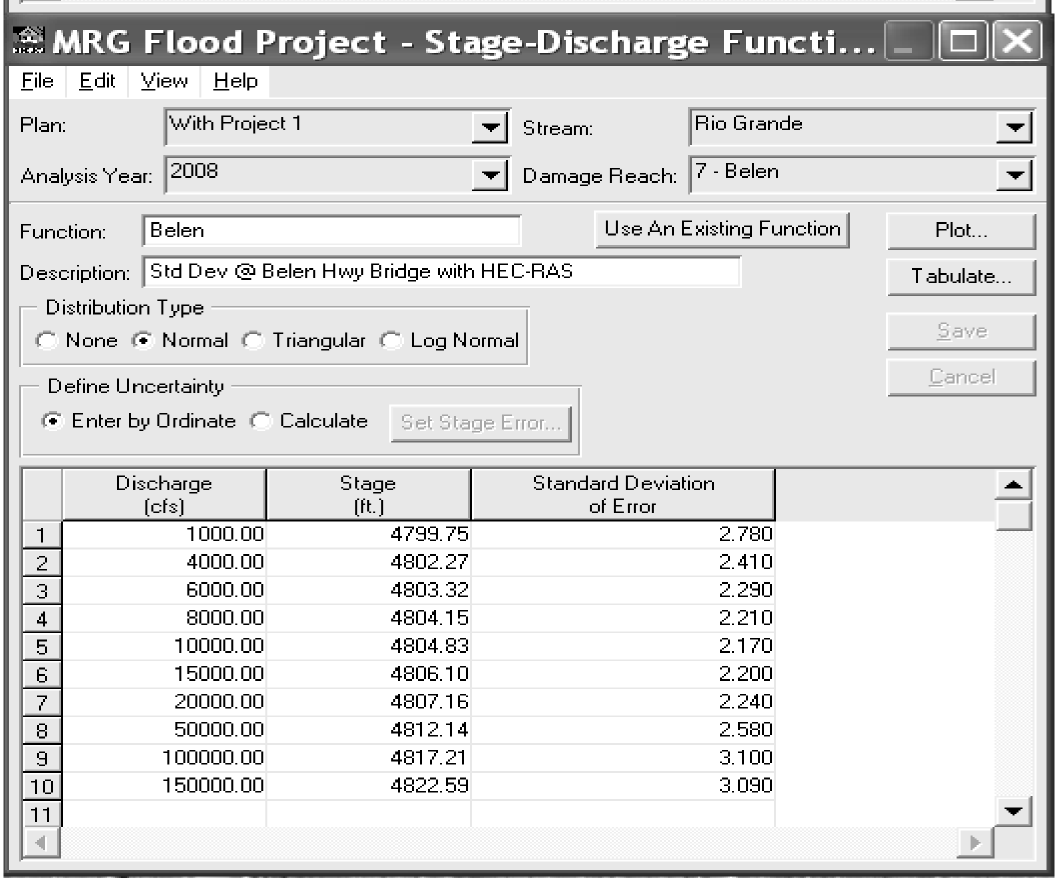
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4868.36	1.850
2	4000.00	4869.71	1.760
3	6000.00	4870.24	1.750
4	8000.00	4870.68	1.820
5	10000.00	4870.98	1.890
6	15000.00	4871.84	2.020
7	20000.00	4872.59	2.110
8	50000.00	4876.12	2.560
9	100000.00	4880.58	3.060
10	150000.00	4884.25	3.450
11			



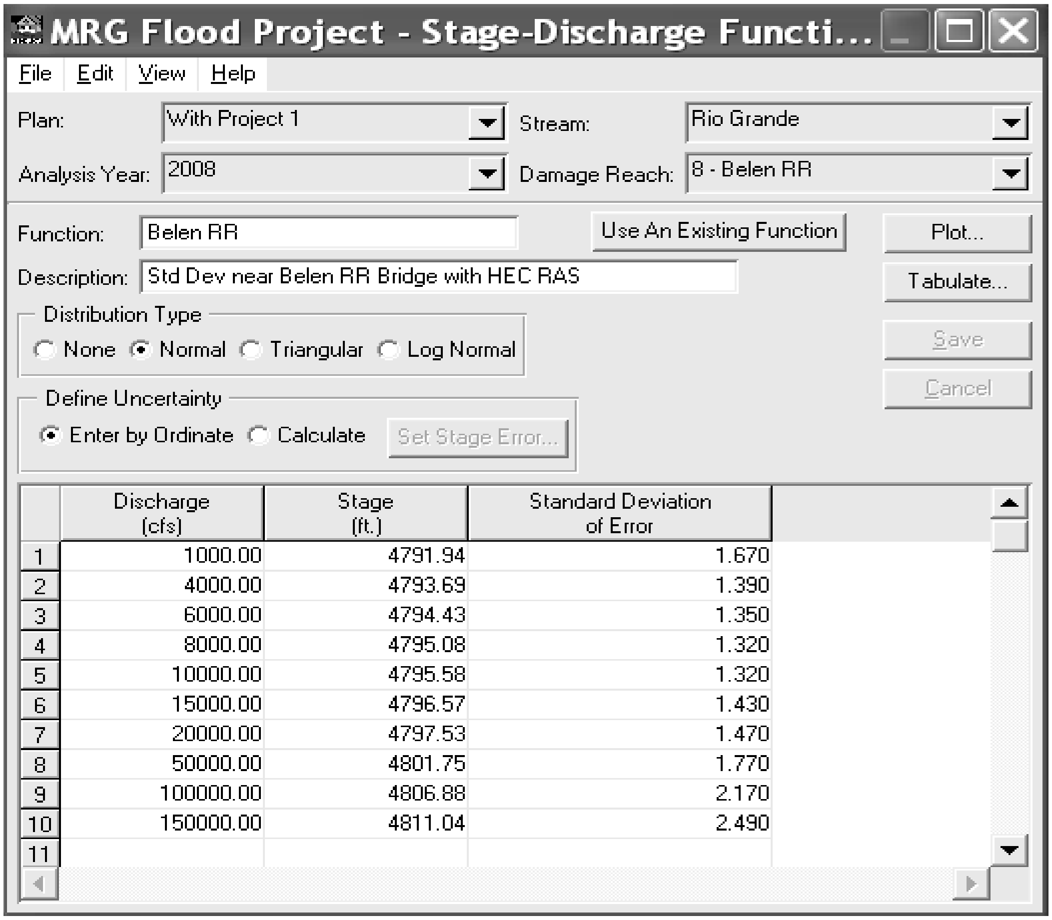
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4850.33	2.400
2	4000.00	4852.22	2.020
3	6000.00	4852.83	1.980
4	8000.00	4853.26	2.000
5	10000.00	4853.67	2.030
6	15000.00	4854.46	2.110
7	20000.00	4855.20	2.170
8	50000.00	4858.29	2.620
9	100000.00	4862.41	3.070
10	150000.00	4865.80	3.490
11			



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4825.19	2.330
2	4000.00	4826.85	2.090
3	6000.00	4827.52	2.060
4	8000.00	4827.99	2.060
5	10000.00	4828.37	2.120
6	15000.00	4829.31	2.210
7	20000.00	4830.13	2.270
8	50000.00	4833.91	2.550
9	100000.00	4838.52	2.960
10	150000.00	4842.31	3.340
11			



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4799.75	2.780
2	4000.00	4802.27	2.410
3	6000.00	4803.32	2.290
4	8000.00	4804.15	2.210
5	10000.00	4804.83	2.170
6	15000.00	4806.10	2.200
7	20000.00	4807.16	2.240
8	50000.00	4812.14	2.580
9	100000.00	4817.21	3.100
10	150000.00	4822.59	3.090
11			



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	1000.00	4791.94	1.670
2	4000.00	4793.69	1.390
3	6000.00	4794.43	1.350
4	8000.00	4795.08	1.320
5	10000.00	4795.58	1.320
6	15000.00	4796.57	1.430
7	20000.00	4797.53	1.470
8	50000.00	4801.75	1.770
9	100000.00	4806.88	2.170
10	150000.00	4811.04	2.490
11			

For each of the flood frequencies that were considered, a separate project was developed to determine the elevation to achieve a 90% Conditional Non-Exceedance Probability Levee Height for that event corresponding to each levee reach. Levee elevations were adjusted in an iterative process until 90% CNP was achieved for each event being investigated. This analysis was conducted for both the current With-Project Condition (2008) and the future With-Project Condition (2058) and is included in the HEC-FDA Risk Analysis Program (see Attachment 7 in Appendix H).

The final hydraulic results using HEC-FDA Risk Analysis for Flood Damage Reduction Projects will be used in determining the most appropriate levee height for design based on the Benefit/Cost Analysis.

Examples of the “Project Performance” output are provided below for current Without-Project Conditions and current With-Project 1 Conditions (500 year project – 2% chance event).

Project Performance

File Help

MRG Flood Project Project Performance
by Damage Reaches for the Without
(Without project condition) plan for Analysis Year 2008
(Stages in ft.)
Plan was calculated with Uncertainty

Without Project Base Year Performance Target Criteria:
Event Exceedance Problability = 0.01
Residual Damage = 5.00 %

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Rio Grande	Channel & Floo	8 - Belen RR	Belen RR Bridge to E	levee	0.0032	0.1285	0.7473	0.9679	0.9990	0.8024	0.7977	0.7894	0.7891	0.5593	0.3922
		7 - Belen	Belen Hwy Bridge to	levee	0.0042	0.2286	0.9254	0.9985	1.0000	0.7042	0.7008	0.6932	0.6930	0.4920	0.3567
		6 - Los Chaves	Los Chaves to Belen	levee	0.0014	0.1675	0.8401	0.9898	0.9999	0.7963	0.7948	0.7903	0.7901	0.6676	0.5585
		5 - Los Lunas	Los Lunas to Los Ch	levee	0.0062	0.3636	0.9891	1.0000	1.0000	0.5866	0.5833	0.5784	0.5782	0.4393	0.3491
		4 - Bosque Farr	Bosque Farms to Los	levee	0.0011	0.1415	0.7825	0.9779	0.9995	0.8121	0.8120	0.8120	0.8120	0.6976	0.5901
		3 - Isleta South	Isleta Diversion to Bc	levee	0.0035	0.2587	0.9499	0.9994	1.0000	0.6866	0.6819	0.6793	0.6769	0.5189	0.4050
		2 - Isleta North	I-25 Bridge to Isleta I	levee	0.0042	0.2145	0.9106	0.9976	1.0000	0.7156	0.6948	0.6514	0.6463	0.4822	0.3484
		1 -Mountain Vie	South Diversion Cha	levee	0.0325	0.1706	0.8460	0.9907	0.9999	0.7715	0.5756	0.3403	0.3029	0.1208	0.0684

Project Performance

File Help

MRG Flood Project Project Performance
by Damage Reaches for the With Project 1
(500 Year Levee Height) plan for Analysis Year 2008
(Stages in ft.)
Plan was calculated with Uncertainty

Without Project Base Year Performance Target Criteria:
Event Exceedance Problability = 0.01
Residual Damage = 5.00 %

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Rio Grande	Channel & Floo	8 - Belen RR	Belen RR Bridge to E	levee	0.0002	0.0017	0.0166	0.0411	0.0805	0.9982	0.9981	0.9979	0.9979	0.9740	0.9045
		7 - Belen	Belen Hwy Bridge to	levee	0.0002	0.0055	0.0532	0.1278	0.2392	0.9922	0.9918	0.9913	0.9913	0.9629	0.9017
		6 - Los Chaves	Los Chaves to Belen	levee	0.0001	0.0111	0.1057	0.2437	0.4280	0.9847	0.9840	0.9834	0.9834	0.9539	0.9043
		5 - Los Lunas	Los Lunas to Los Ch	levee	0.0001	0.0133	0.1251	0.2841	0.4875	0.9819	0.9813	0.9806	0.9806	0.9489	0.9004
		4 - Bosque Farr	Bosque Farms to Los	levee	0.0001	0.0082	0.0786	0.1850	0.3357	0.9875	0.9871	0.9864	0.9825	0.9552	0.9018
		3 - Isleta South	Isleta Diversion to Bc	levee	0.0001	0.0058	0.0562	0.1345	0.2510	0.9923	0.9909	0.9897	0.9833	0.9535	0.9017
		2 - Isleta North	I-25 Bridge to Isleta I	levee	0.0005	0.0007	0.0074	0.0183	0.0364	0.9999	0.9999	0.9997	0.9990	0.9865	0.9041
		1 -Mountain Vie	South Diversion Cha	levee	0.0003	0.0007	0.0074	0.0185	0.0366	0.9999	0.9999	0.9996	0.9992	0.9740	0.9009

Output summaries from FDA output shown below provide levee heights for the 1% chance, 0.5% chance, and 0.2% chance events. However, FDA does not directly calculate the 0.5% CNP, rather the 0.4% CNP. Therefore, it is necessary to interpolate between the 1% chance and 0.4% chance events in order to obtain the 0.5% CNP. The results of that effort are also provided below for both the current With-Project (2008) Condition and the future With-Project (2058) Condition.

FDA Results for With-Project Analysis -Current (2008)
Calculation for 0.5% CNP

Determine by Interpolation	1%	0.5% *	0.40%
90% CNP for 0.5% Levee	0.01	0.005	0.004
Current With-Project (2008)	100yr	200yr	250yr
Tijeras Arroyo Confluence	0.9879	0.905983	0.8896
I-25 River Crossing	0.9733	0.903217	0.8892
Isleta Diversion Structure	0.9549	0.905567	0.8957
Bosque Farms	0.9495	0.902167	0.8927
Los Lunas Bridge	0.9523	0.903217	0.8934
Los Chaves	0.9563	0.9073	0.8975
Belen Highway Bridge	0.964	0.901917	0.8895
Belen RR Bridge	0.9796	0.908017	0.8937

* 0.5% CNP = ((1% CNP - 0.4% CNP) / 6) + .4% CNP

FDA Results for With-Project Analysis - Future (2058)
Calculation for 0.5% CNP

Determine by Interpolation	1%	0.50%	0.40%
90% CNP for 0.5% Levee	0.01	0.005	0.004
Future With-Project (2058)	100yr	200yr	250yr
Tijeras Arroyo Confluence	0.9876	0.902267	0.8852
I-25 River Crossing	0.9775	0.9075	0.8935
Isleta Diversion Structure	0.9656	0.902183	0.8895
Bosque Farms	0.9599	0.903067	0.8917
Los Lunas Bridge	0.9687	0.909867	0.8981
Los Chaves	0.967	0.909333	0.8978
Belen Highway Bridge	0.9641	0.902933	0.8907
Belen RR Bridge	0.9797	0.907367	0.8929

* 0.5% CNP = ((1% CNP - 0.4% CNP) / 6) + .4% CNP

A summary of results from the FDA output is provided below which gives levee heights for the 1% chance, 0.5% chance, and 0.2% chance events. Results are provided for both the current With-Project (2008) Condition and the future With-Project (2058) Condition.

FDA Results for With-Project Analysis - Current 2008 With-Project Levee Heights
Elevations are Given in Feet - NAVD88 Datum

Location	River Station	Channel Invert	0.002 500 year Flow Rate (CFS)	500 year WSEL	500 year Levee Elev.	Levee Height above expected 500 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	30219	4929.35	4932.85	3.5
I-25 River Crossing	172.46	4899.46	21694	4907.03	4910.45	3.42
Isleta Diversion Structure	169.29	4885.06	17119	4890.91	4894.3	3.39
Bosque Farms	165.26	4866.84	14248	4871.72	4874.6	2.88
Los Lunas Bridge	161.48	4848.48	14243	4854.34	4857.3	2.96
Los Chaves	155.92	4823.47	14014	4829.13	4832.3	3.17
Belen Highway Bridge	150.34	4798.64	14460	4805.97	4809.3	3.33
Belen RR Bridge	148.3	4789.33	14196	4796.41	4798.8	2.39

Location	River Station	Channel Invert	0.005 200 year Flow Rate (CFS)	200 year WSEL	200 year Levee Elev.	Levee Height above expected 200 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	21855	4927.97	4931	3.03
I-25 River Crossing	172.46	4899.46	13213	4905.46	4908.3	2.84
Isleta Diversion Structure	169.29	4885.06	12606	4890.07	4893.3	3.23
Bosque Farms	165.26	4866.84	10224	4871.03	4873.7	2.67
Los Lunas Bridge	161.48	4848.48	10202	4853.68	4856.5	2.82
Los Chaves	155.92	4823.47	10112	4828.39	4831.4	3.01
Belen Highway Bridge	150.34	4798.64	10218	4804.89	4808	3.11
Belen RR Bridge	148.3	4789.33	10180	4795.61	4797.7	2.09

Location	River Station	Channel Invert	0.01 100 year Flow Rate (CFS)	100 year WSEL	100 year Levee Elev.	Levee Height above expected 100 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	16041	4926.87	4929.25	2.38
I-25 River Crossing	172.46	4899.46	9505	4904.61	4907.2	2.59
Isleta Diversion Structure	169.29	4885.06	10064	4889.49	4892.4	2.91
Bosque Farms	165.26	4866.84	7616	4870.64	4873.1	2.46
Los Lunas Bridge	161.48	4848.48	7604	4853.17	4855.8	2.63
Los Chaves	155.92	4823.47	7554	4827.93	4830.6	2.67
Belen Highway Bridge	150.34	4798.64	7573	4804	4806.9	2.9
Belen RR Bridge	148.3	4789.33	7572	4794.94	4796.7	1.76

FDA Results for With-Project Analysis - Future (2058) With-Project Levee Heights
Elevations are Given in Feet - NAVD88 Datum

Location	River Station	Channel Invert	500 year Flow Rate (CFS)	500 year WSEL	500 year Levee Elev.	Levee Height above expected 500 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	30407	4929.38	4933	3.62
I-25 River Crossing	172.46	4899.46	23170	4907.26	4910.9	3.64
Isleta Diversion Structure	169.29	4886.06	18310	4892.12	4895.1	2.98
Bosque Farms	165.26	4867.84	14237	4872.72	4874.9	2.18
Los Lunas Bridge	161.48	4849.48	14223	4855.34	4857.7	2.36
Los Chaves	155.92	4824.47	14082	4830.01	4832.6	2.59
Belen Highway Bridge	150.34	4798.64	14415	4805.96	4809.3	3.34
Belen RR Bridge	148.3	4789.33	14514	4796.46	4798.9	2.44

Location	River Station	Channel Invert	0.005 200 year Flow Rate (CFS)	200 year WSEL	200 year Levee Elev.	Levee Height above expected 200 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	21915	4927.98	4931	3.02
I-25 River Crossing	172.46	4899.46	13483	4905.52	4908.4	2.88
Isleta Diversion Structure	169.29	4886.06	12914	4891.13	4893.8	2.67
Bosque Farms	165.26	4867.84	10220	4872.03	4874	1.97
Los Lunas Bridge	161.48	4849.48	10205	4854.68	4856.9	2.22
Los Chaves	155.92	4824.47	10136	4829.29	4831.7	2.41
Belen Highway Bridge	150.34	4798.64	10182	4804.88	4808	3.12
Belen RR Bridge	148.3	4789.33	10180	4795.61	4797.7	2.09

Location	River Station	Channel Invert	0.01 100 year Flow Rate (CFS)	100 year WSEL	100 year Levee Elev.	Levee Height above expected 100 yr WSEL
Tijeras Arroyo Confluence	176.9	4919.39	16025	4926.87	4929.3	2.43
I-25 River Crossing	172.46	4899.46	10853	4904.61	4907.2	2.59
Isleta Diversion Structure	169.29	4886.06	10273	4889.49	4892.9	3.41
Bosque Farms	165.26	4867.84	7614	4870.64	4873.4	2.76
Los Lunas Bridge	161.48	4849.48	7600	4853.17	4856.1	2.93
Los Chaves	155.92	4824.47	7555	4827.93	4830.9	2.97
Belen Highway Bridge	150.34	4798.64	7564	4804	4806.9	2.9
Belen RR Bridge	148.3	4789.33	7563	4794.94	4796.7	1.76

7.0 Delineation of Inundation Mapping

7.1 Without Project Inundation Mapping

Flo-2d runs were conducted to reflect the "Albuquerque West Levee" which is not part of this project but was constructed after this study began. The hydrology for the study area is a combined frequency that is based on Albuquerque hydrology (separate snowmelt and rainfall-runoff hydrographs) and rainfall-runoff flooding from the Albuquerque South Diversion Channel which includes the Tijeras Arroyo. Flooding in the study reach is dominated by one of the three sources of flooding at any given location: (1) regulated spring snowmelt runoff floods (2) unregulated and primarily rainfall-runoff floods, or (3) rainfall-runoff from the Albuquerque South Diversion Channel (SDC). Four frequencies were mapped; the 10%-chance flood, the 1.0%-chance flood, the 0.5%-chance flood and the 0.2%-chance flood.

The output from the final merged floodplains for each frequency event indicates that the floodplain extents and depths are generally dictated by the rainfall events in the northern reaches (South Diversion Channel to Interstate Hwy 25) and by the regulated flow (spring runoff) in the southern reaches of the study area (Interstate Hwy 25 to Belen).

The approach taken to delineate Without-Project inundation maps were accomplished using the FLO-2D, 2-dimensional hydraulic model described previously. The Without-Project floodplain mapping (flood inundation maps) considered the three different flooding sources described above. This was accomplished by running each event separately. The model was run using the 10-year, 100-year, 200-year and 500-year (10%, 1%, 0.5%, & 0.2% chance events) hydrographs for each flooding source. Inundation maps were developed for combined sources that considered the affects from all three flooding sources superimposed onto one map. This resulted in a total of four separate inundation maps. The results are shown in Attachment 5 of Appendix H.

7.2 Tentative Selected Plan Inundation Mapping

Flo-2d runs were conducted to reflect the Tentative Selected Plan with the "Albuquerque West Levee" in place which is not part of this project but was constructed after this study began.
The hydrology for the study area is a combined frequency that is based on Albuquerque hydrology (separate snowmelt and rainfall-runoff hydrographs) and rainfall-runoff flooding from the Albuquerque South Diversion Channel which includes the Tijeras Arroyo. Flooding in the study reach is dominated by one of the three sources of flooding at any given location: (1) regulated spring snowmelt runoff floods (2) unregulated and primarily rainfall-runoff floods, or (3) rainfall-runoff from the Albuquerque South Diversion Channel (SDC). Four frequencies were mapped: the 10%-chance flood, the 1.0%-chance flood, the 0.5%-chance flood and the 0.2%-chance flood.

The output from the final merged floodplains for each frequency event indicates that the floodplain extents and depths are generally dictated by the rainfall events in the northern reaches (South Diversion Channel to Interstate Hwy 25) and by the regulated flow (spring runoff) in the southern reaches of the study area (Interstate Hwy 25 to Belen).

The approach taken to delineate the Tentatively Selected Plan inundation maps were accomplished using the FLO-2D, 2-dimensional hydraulic model described previously. The Tentative Selected Plan floodplain mapping (flood inundation maps) considered the three different flooding sources described above. This was accomplished by running each event separately. The model was run using the 10-year, 100-year, 200-year and 500-year (10%, 1%, 0.5%, & 0.2% chance events) hydrographs for each flooding source. Inundation maps were developed for combined sources that considered the affects from all three flooding sources superimposed onto one map. This resulted in a total of four separate inundation maps. The results are shown in Attachment 7 of Appendix H.

7.3 Residual With-Project Inundation Mapping Extents

The approach taken to delineate With-Project floodplain extents were accomplished using the FLO-2D, 2-dimensional hydraulic model described previously. Levee heights within the model were set at the 200-year (0.5% probability) water surface elevation (WSEL). The residual With-Project floodplain mapping (flood inundation maps) considered the three different flooding sources – Flows from the unregulated area below Cochiti (substantially rainfall runoff), Regulated flow (substantially spring runoff), and the South Diversion Channel flow (substantially rainfall runoff from Tijeras Arroyo). This was accomplished by running each event separately. The model was run using the 500-year (0.2% chance event) hydrographs for each flooding source. Inundation maps were developed for each flooding source separately as well as a combined flood inundation map that considered the affects from all three flooding source superimposed onto one map. This resulted in a total of four separate inundation maps. The results are shown in Attachment 9 of Appendix H

8.0 Interior Drainage Analysis

The analysis in USACE (1986) was considered to be applicable for the current study and is discussed in Paragraph 6-01, Interior Flooding Evaluation and 8-03 Interior Flooding (MRG 1986 GDM - Interior Flooding Evaluation). Additionally, RTI (1985) provided a useful summary of tributary drainages that was useful for this study.

The USACE 1986) report states in the Interior Flooding Evaluation, section 8-03 on page VIII-2:

> *"The phrase 'interior flooding' refers to flooding caused by blocked drainage behind the riverside levees. The floodplain behind these man-made structures is protected from exterior (Rio Grande) flood water, but the interior runoff still remains as a potential problem. Tributary arroyos draining large watersheds usually have direct outlets through the levees, but smaller drainage areas usually do not have a direct outlet and could create interior flooding. Approximately 60 of these smaller drainage areas were identified and analyzed in this study. However, only in one area do tributary floodwaters reach the levee and pond behind it, even under SPF conditions. In the Belen East Unit, where the largest watersheds are located, the valley floor is so wide that SPF flows of nearly 30,000 cfs reach the levee only at Pond 4 at the lower end of the project. See Plate 13H."*

In section 8-17.5 on page VIII-13:

> *"The selected plan consists of leaving 2800 feet of existing levee unimproved at the lower end of Pond 4 (see Plates 17H and 18H). This would allow flows in the Rio Grande in excess of approximately 11,000 cfs to flow into Pond 4. This is the only feasible plan formulated that would not induce interior flooding and still be in accordance with EC 1110-2-247."*

These reports provide detailed hydrologic and hydraulic information for drainage areas outside the existing spoil banks and recommendations concerning the conveyance of these flows to the Rio Grande.

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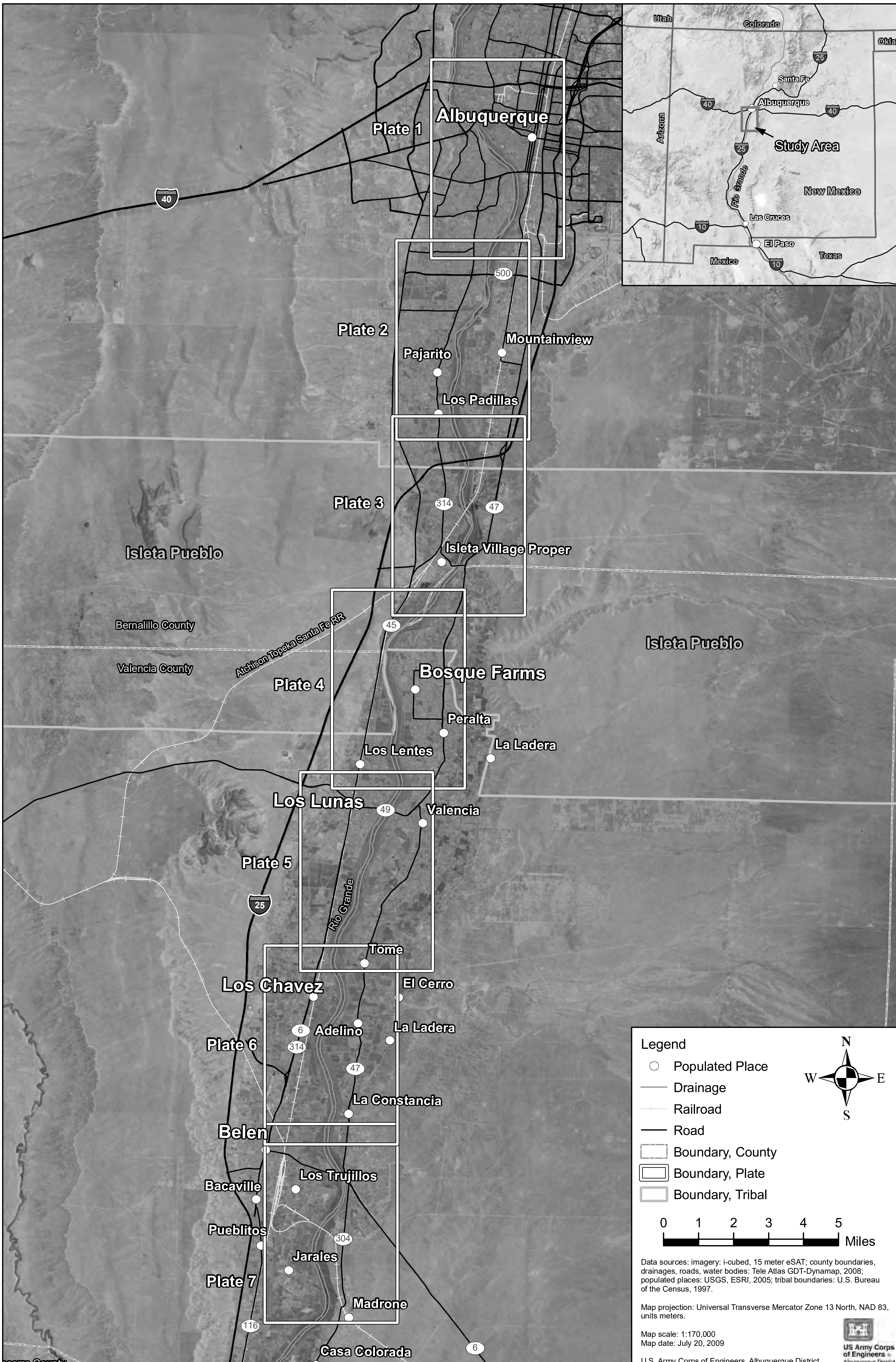
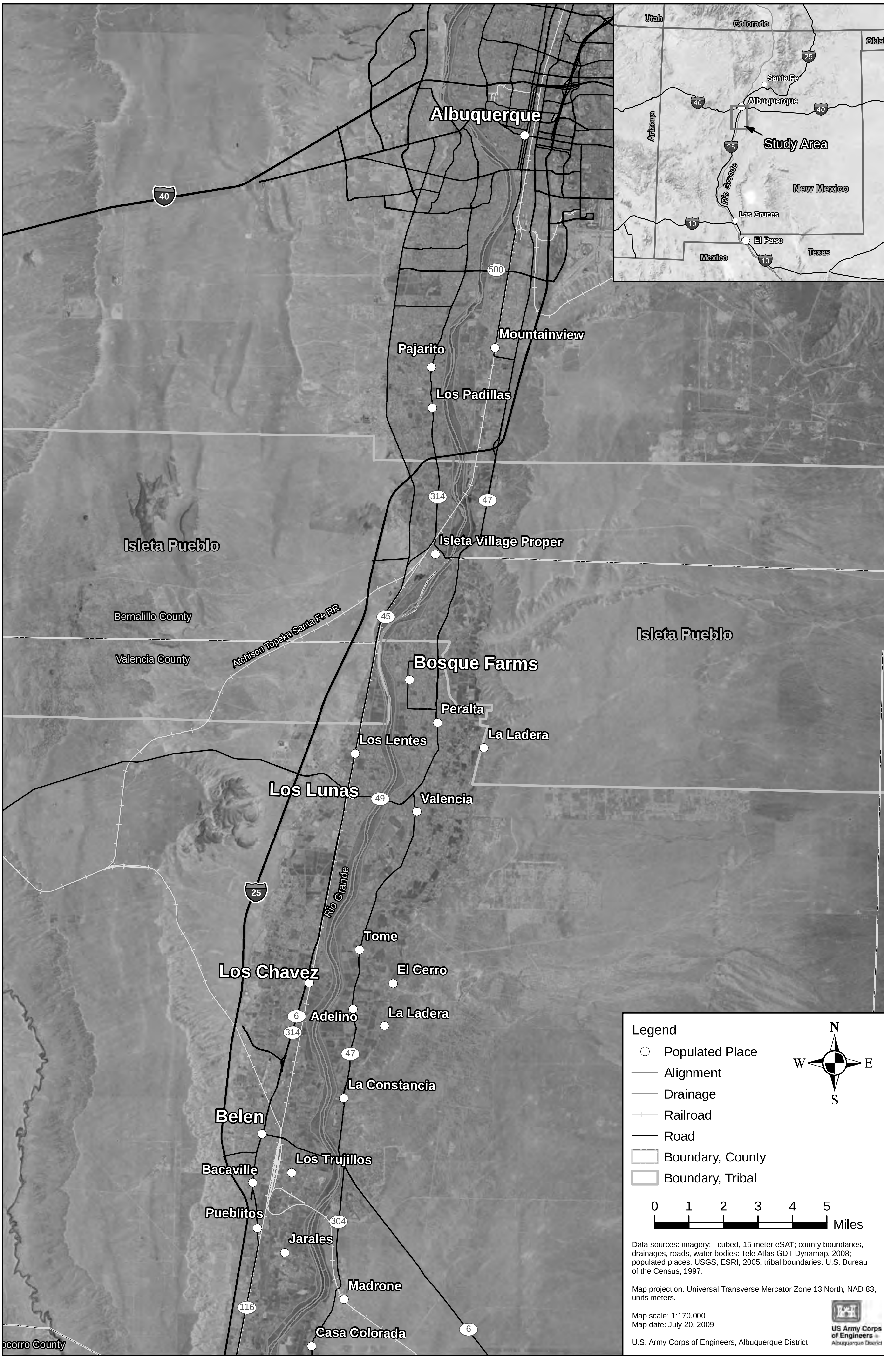
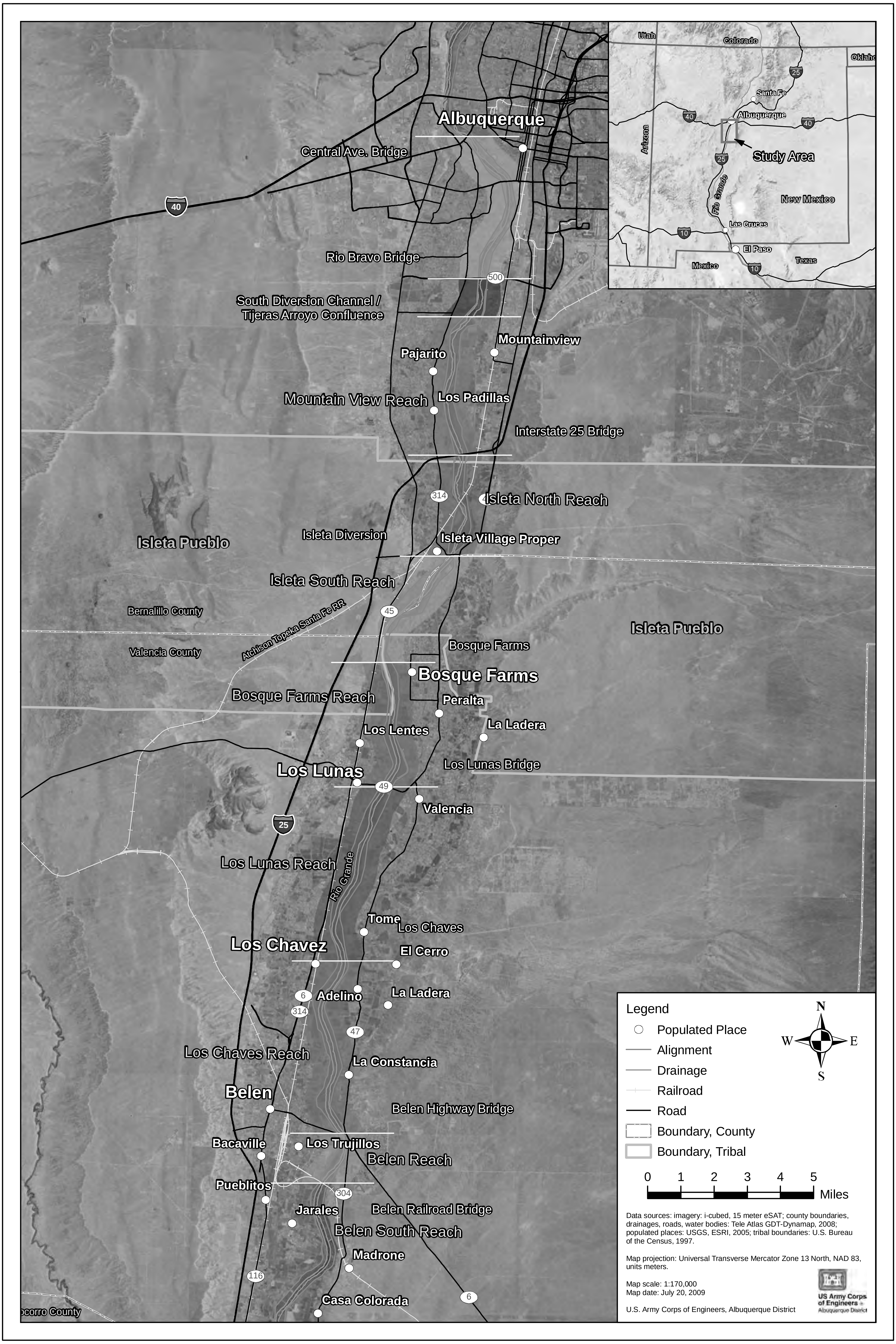


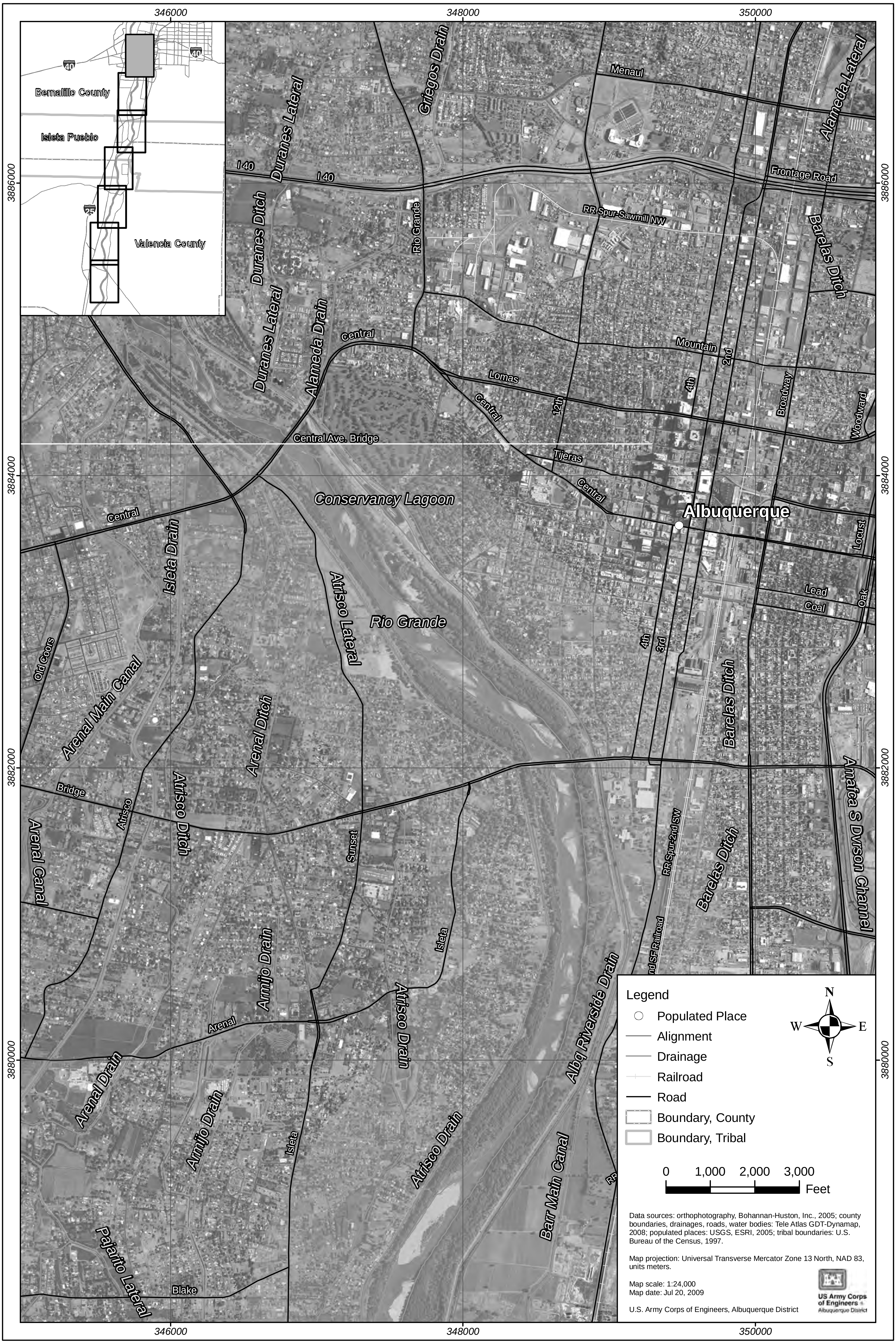
Middle Rio Grande Study Area Plate Index Map



Middle Rio Grande Study Area Vicinity Map



Middle Rio Grande Levee/Damage Reaches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Bernalillo County
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Menaul
Griegos Drain
Duranes Lateral
Alameda Lateral
I 40
Frontage Road
Duranes Ditch
Rio Grande
RR Spur-Sawmill NW
Barelas Ditch
Central
Mountain
Alameda Drain
Lomas
4th
2nd
12th
Broadway
Woodward
Central Ave. Bridge
Tijeras
Conservancy Lagoon
Albuquerque
Isleta Drain
Locust
Lead
Coal
Oak
Atrisco Lateral
Old Coors
3rd
Arenal Main Canal
Arenal Ditch
Bridge
Atrisco Ditch
Amafca S Dvrson Channel
Arenal Canal
Atrisco
Sunset
RR Spur-2nd SW
Isleta
Armijo Drain
Atrisco Drain
Albq Riverside Drain
Arenal
Arenal Drain
Bar Main Canal
Pajarito Lateral
Blake
Legend
Populated Place
Alignment
Drainage
Railroad
Road
Boundary, County
Boundary, Tribal
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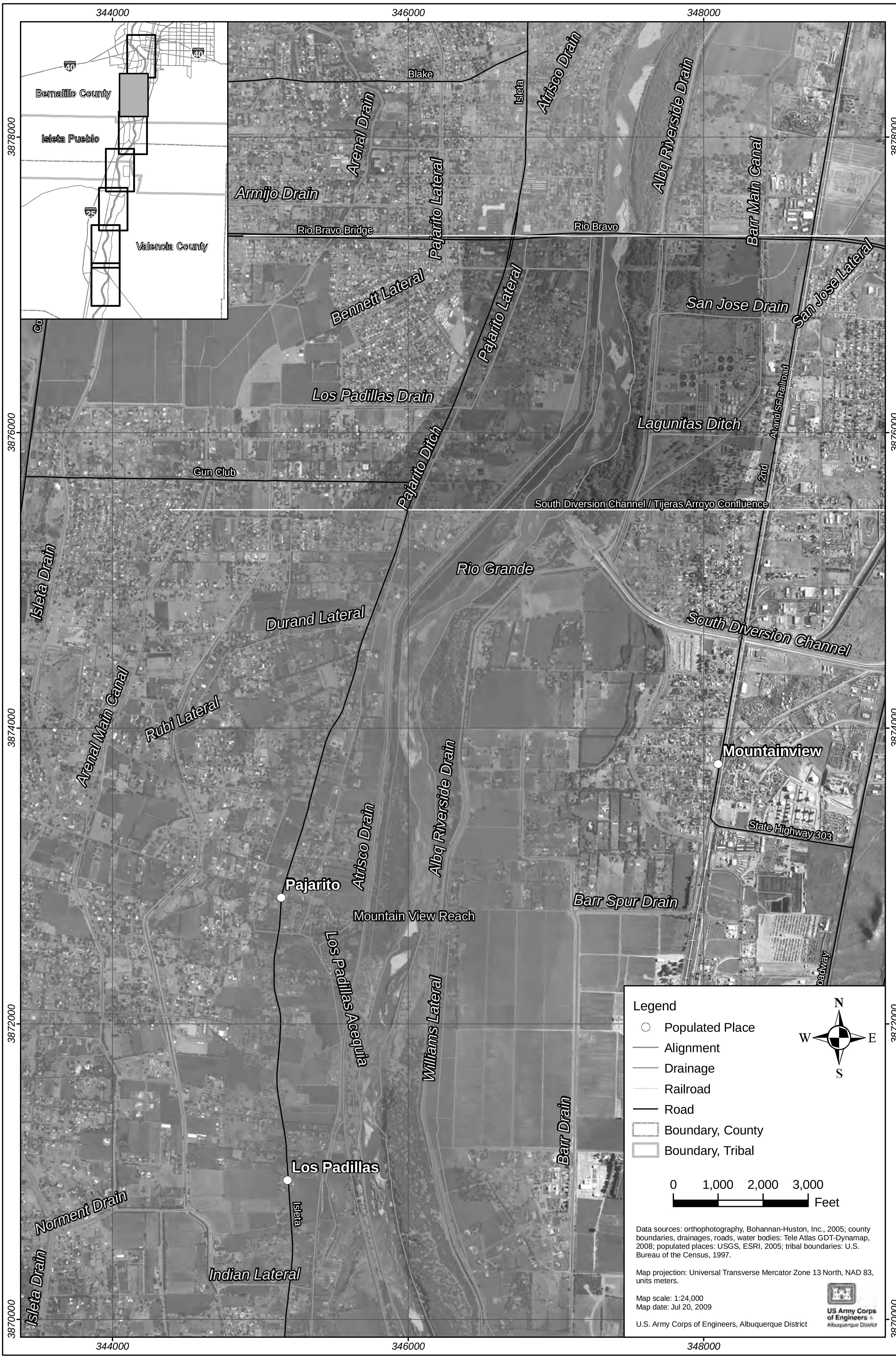
Feet
Data sources: orthophotography, Bohannan-Huston, Inc., 2005; county boundaries, drainages, roads, water bodies: Tele Atlas GDT-Dynamap, 2008; populated places: USGS, ESRI, 2005; tribal boundaries: U.S. Bureau of the Census, 1997.
Map projection: Universal Transverse Mercator Zone 13 North, NAD 83, units meters.
Map scale: 1:24,000
Map date: Jul 20, 2009
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers
Albuquerque District

Plate 1



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Bernalillo County
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Valencia County
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Arenal Drain
Albq Riverside Drain
Barr Main Canal
Armijo Drain
Pajarito Lateral
Rio Bravo Bridge
Rio Bravo
Bennett Lateral
San Jose Drain
San Jose Lateral
Los Padillas Drain
Lagunitas Ditch
At and SF Railroad
Pajarito Ditch
Gun Club
2nd
South Diversion Channel / Tijeras Arroyo Confluence
Rio Grande
Isleta Drain
Durand Lateral
South Diversion Channel
Arenal Main Canal
Rubi Lateral
Mountainview
State Highway 303
Pajarito
Atrisco Drain
Albq Riverside Drain
Barr Spur Drain
Mountain View Reach
Los Padillas Acequia
Williams Lateral
Barr Drain
Los Padillas
Isleta
Norment Drain
Indian Lateral
Isleta Drain
Legend
Populated Place
Alignment
Drainage
Railroad
Road
Boundary, County
Boundary, Tribal
N
S
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Data sources: orthophotography, Bohannan-Huston, Inc., 2005; county boundaries, drainages, roads, water bodies: Tele Atlas GDT-Dynamap, 2008; populated places: USGS, ESRI, 2005; tribal boundaries: U.S. Bureau of the Census, 1997.
Map projection: Universal Transverse Mercator Zone 13 North, NAD 83, units meters.
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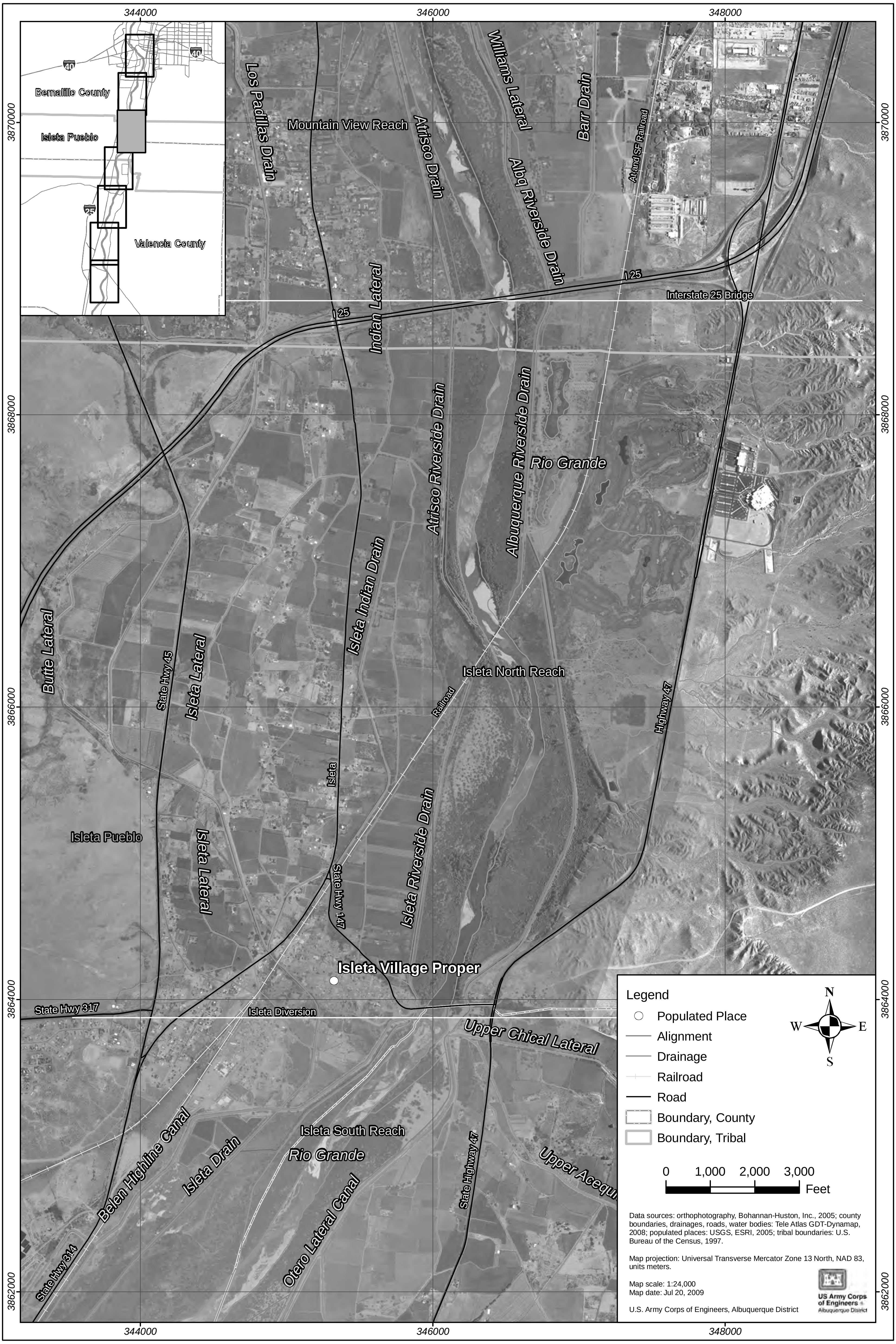
Map date: Jul 20, 2009
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers
Albuquerque District

Plate 2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Bernalillo County
Isleta Pueblo
Valencia County
Los Padillas Drain
Mountain View Reach
Atrisco Drain
Williams Lateral
Barr Drain
At and SF Railroad
Albq Riverside Drain
I 25
Interstate 25 Bridge
Indian Lateral
Atrisco Riverside Drain
Albuquerque Riverside Drain
Rio Grande
Isleta Indian Drain
Butte Lateral
State Hwy 45
Isleta Lateral
Isleta North Reach
Railroad
Highway 47
Isleta
Isleta Pueblo
Isleta Lateral
State Hwy 147
Isleta Riverside Drain
Isleta Village Proper
State Hwy 317
Isleta Diversion
Upper Chical Lateral
Isleta South Reach
Rio Grande
Belen Highline Canal
Isleta Drain
Otero Lateral Canal
State Highway 47
State Hwy 314
Legend
Populated Place
Alignment
Drainage
Railroad
Road
Boundary, County
Boundary, Tribal
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Feet
Data sources: orthophotography, Bohannan-Huston, Inc., 2005; county boundaries, drainages, roads, water bodies: Tele Atlas GDT-Dynamap, 2008; populated places: USGS, ESRI, 2005; tribal boundaries: U.S. Bureau of the Census, 1997.
Map projection: Universal Transverse Mercator Zone 13 North, NAD 83, units meters.
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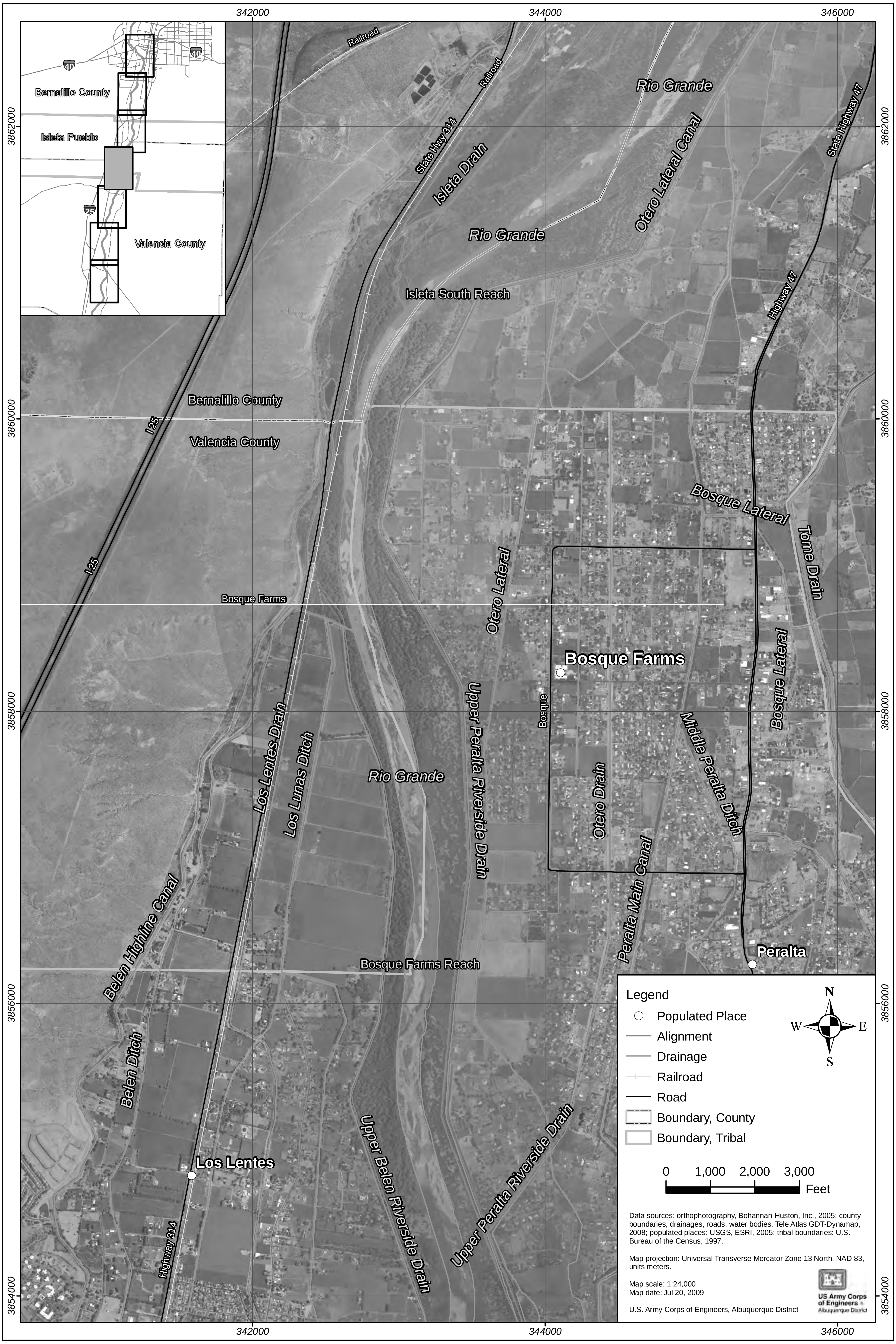
Map date: Jul 20, 2009
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers Albuquerque District

Plate 3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Bernalillo County
Isleta Pueblo
Valencia County
Railroad
Rio Grande
State Hwy 314
Isleta Drain
Otero Lateral Canal
State Highway 47
Isleta South Reach
Highway 47
Bernalillo County
Valencia County
I 25
Bosque Lateral
Tome Drain
Bosque Farms
Otero Lateral
Bosque Farms
Upper Peralta Riverside Drain
Bosque
Middle Peralta Ditch
Bosque Lateral
Los Lentes Drain
Los Lunas Ditch
Otero Drain
Peralta Main Canal
Belen Highline Canal
Bosque Farms Reach
Peralta
Belen Ditch
Upper Belen Riverside Drain
Upper Peralta Riverside Drain
Los Lentes
Highway 314
Legend
Populated Place
Alignment
Drainage
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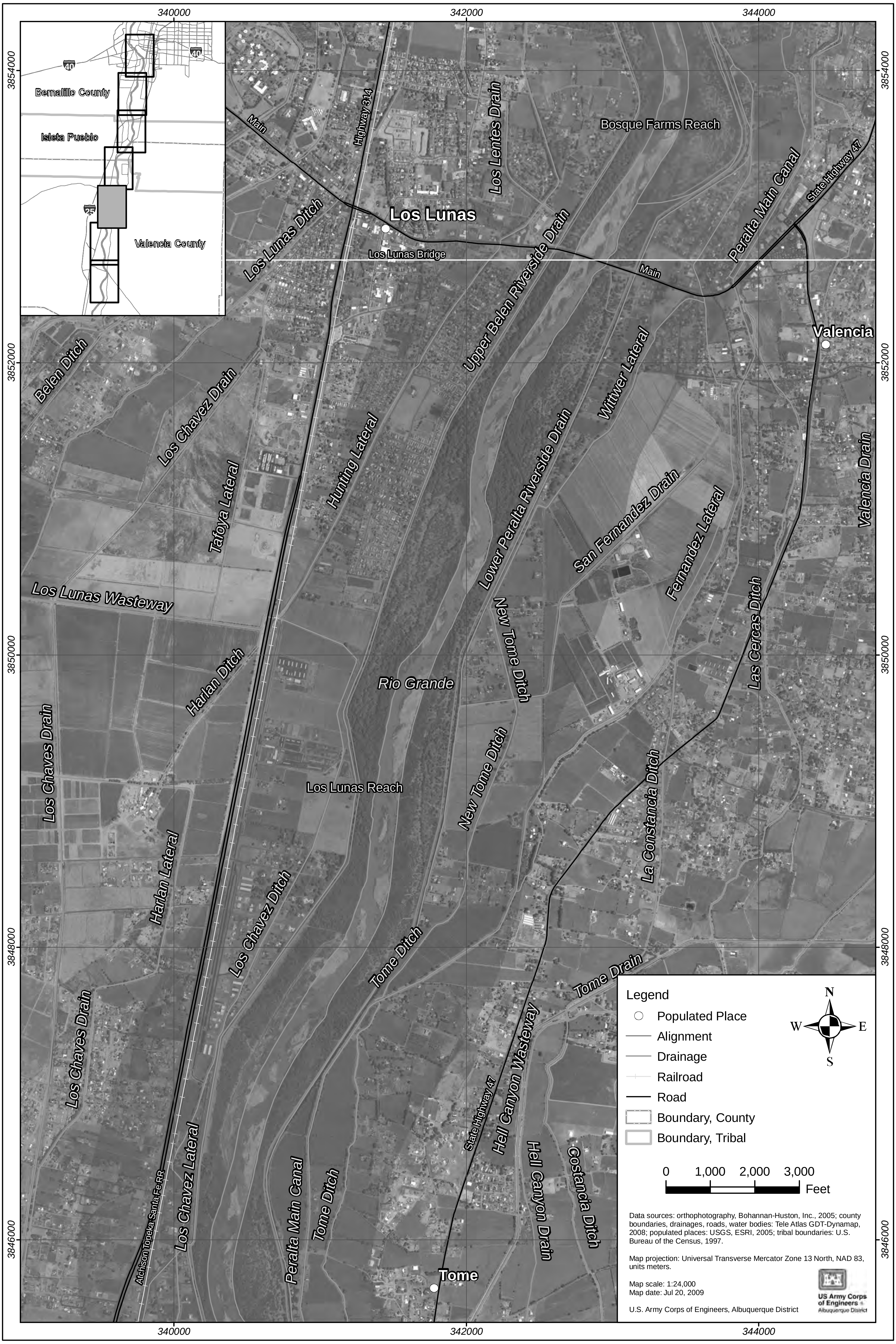
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers Albuquerque District

Plate 4



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Los Lunas Bridge
Upper Belen Riverside Drain
Peralta Main Canal
State Highway 47
Valencia
Belen Ditch
Los Chavez Drain
Wittwer Lateral
Hunting Lateral
Lower Peralta Riverside Drain
Tafoya Lateral
San Fernandez Drain
Fernandez Lateral
Valencia Drain
Los Lunas Wasteway
New Tome Ditch
Las Cercas Ditch
Harlan Ditch
Rio Grande
Los Chaves Drain
Los Lunas Reach
La Constancia Ditch
Harlan Lateral
Los Chavez Ditch
Tome Ditch
Tome Drain
Hell Canyon Wasteway
Los Chavez Lateral
Atchison Topeka Santa Fe RR
Tome
Hell Canyon Drain
Costancia Ditch
Legend
Populated Place
Alignment
Drainage
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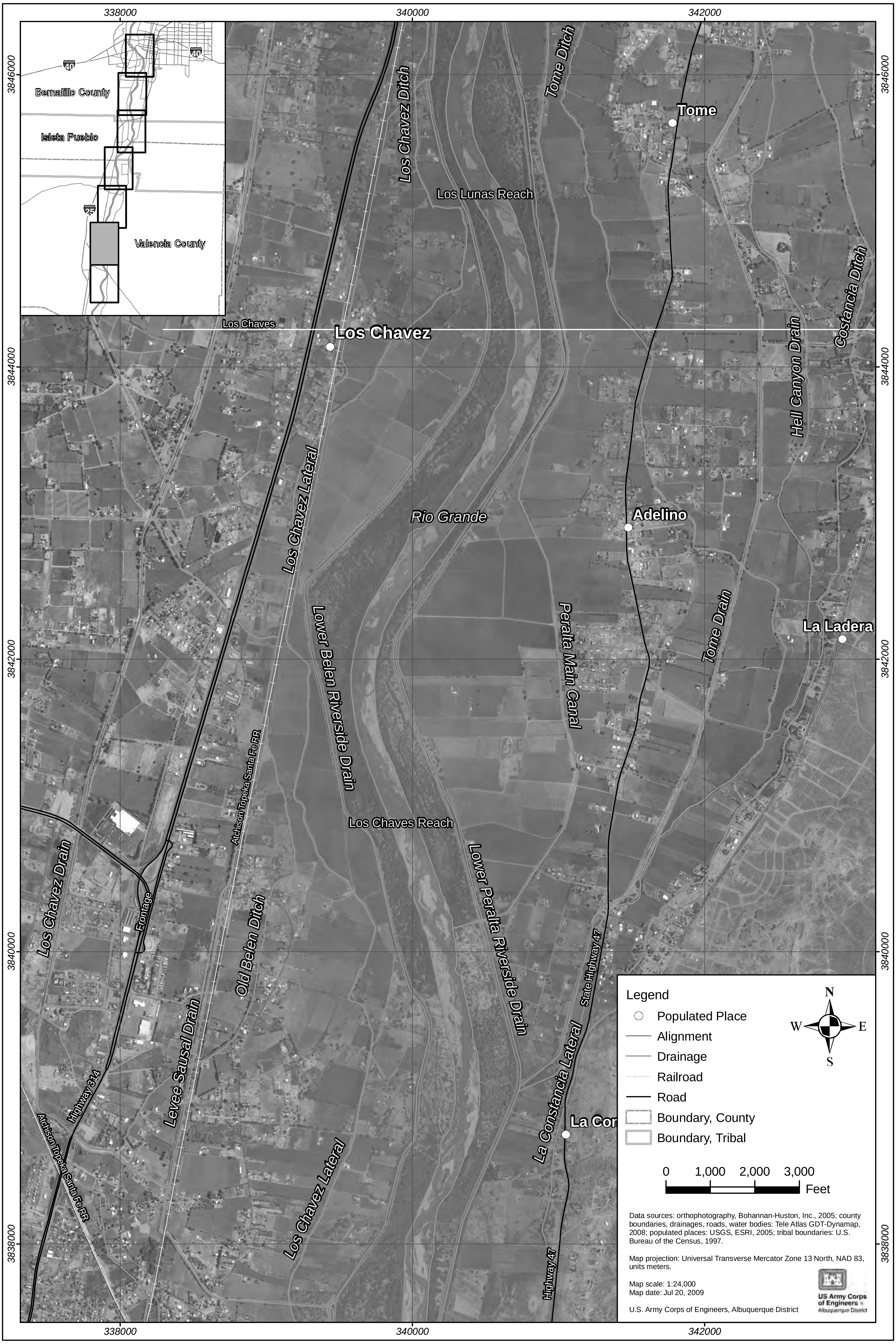
Map date: Jul 20, 2009
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers Albuquerque District

Plate 5



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Hell Canyon Drain
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Peralta Main Canal
Tome Drain
La Ladera
Atchison Topeka Santa Fe RR
Los Chaves Reach
Lower Peralta Riverside Drain
Los Chavez Drain
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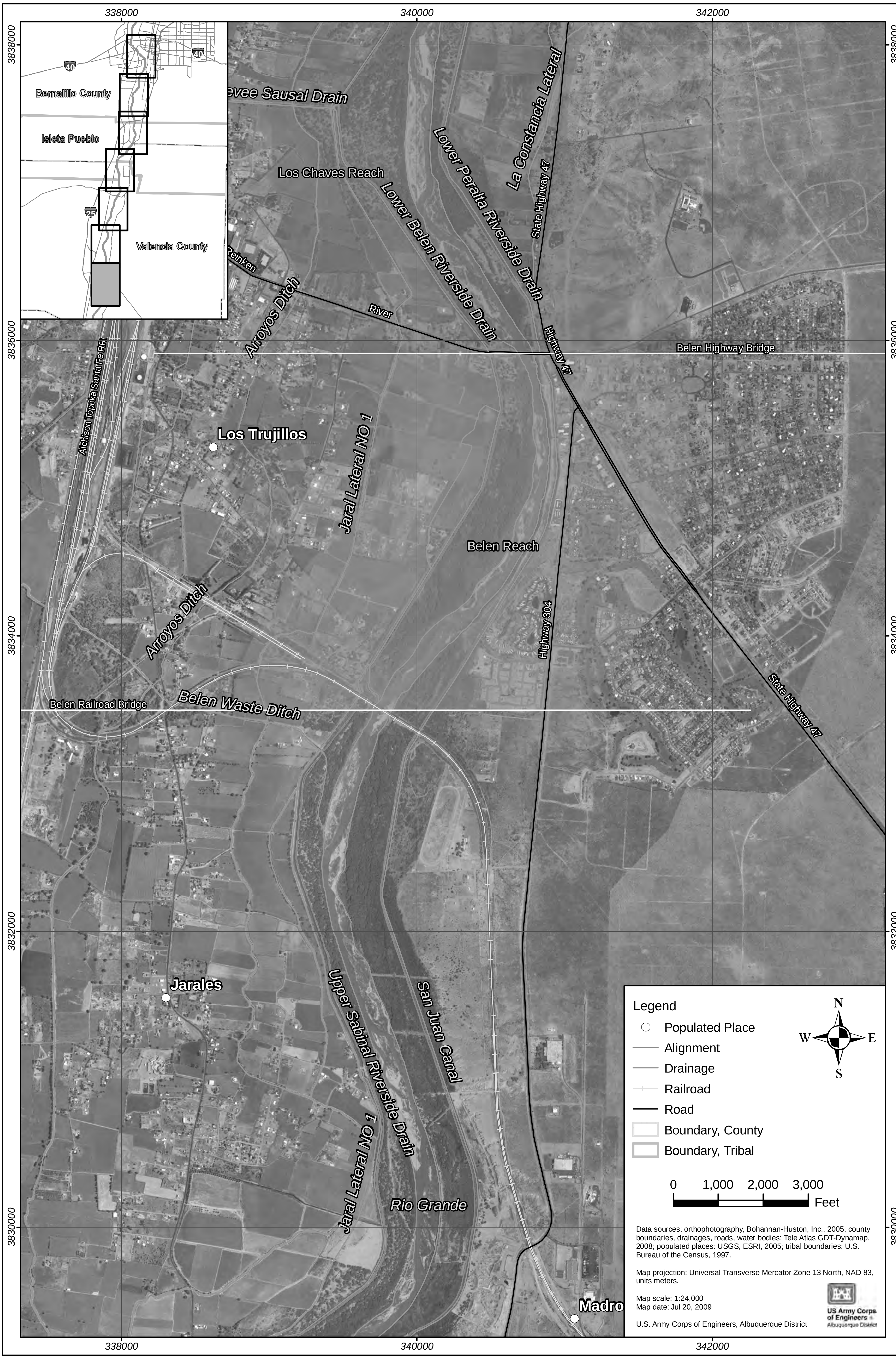
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers
Albuquerque District

Plate 6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Sausal Drain
Los Chaves Reach
Lower Belen Riverside Drain
Lower Peralta Riverside Drain
La Constancia Lateral
State Highway 47
Reinken
River
Arroyos Ditch
Belen Highway Bridge
Highway 47
Atchison Topeka Santa Fe RR
Los Trujillos
Jaral Lateral NO 1
Belen Reach
Highway 304
Belen Railroad Bridge
Belen Waste Ditch
State Highway 47
Jarales
Upper Sabinal Riverside Drain
San Juan Canal
Jaral Lateral NO 1
Rio Grande
Madro
Legend
Populated Place
Alignment
Drainage
Railroad
Road
Boundary, County
Boundary, Tribal
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Feet
Data sources: orthophotography, Bohannan-Huston, Inc., 2005; county boundaries, drainages, roads, water bodies: Tele Atlas GDT-Dynamap, 2008; populated places: USGS, ESRI, 2005; tribal boundaries: U.S. Bureau of the Census, 1997.
Map projection: Universal Transverse Mercator Zone 13 North, NAD 83, units meters.
Map scale: 1:24,000
Map date: Jul 20, 2009
U.S. Army Corps of Engineers, Albuquerque District
US Army Corps of Engineers
Albuquerque District

Plate 7

Attachment 3—Summary of evaluated FLO-2D project scenarios

FLO-2D

Middle Rio Grande Flood Control Project, New Mexico
Mountain View, Isleta and Belen Units

These folders contain the complete files for the FLO-2D runs used in the computations for the:

1) Inundation Mapping
   - Albuquerque Floods from Regulated Areas
   - Albuquerque Floods from Unregulated Areas
   - Floods from the Tijeras Arroyo (SDC)

2) Damage Reach Analysis
   - Albuquerque Floods from Regulated Areas
   - Albuquerque Floods from Unregulated Areas
   - Floods from the Tijeras Arroyo (SDC)

3) Attenuated Flow rate Analysis
   - Albuquerque Floods from Regulated Areas
   - Albuquerque Floods from Unregulated Areas
   - Floods from the Tijeras Arroyo (SDC)

This analysis was conducted for both the current With-Project and Without-Project Conditions (2008) and the future With-Project and Without-Project Conditions (2058).

The models can be viewed in FLO-2D version 2006.01.

A listing of files is as follows:

Without-Project Flo-2d

(2-yr, 5-yr, 10-yr, 20-yr, 50-yr, 100-yr, 200-yr, 500-yr events)

Existing Rain	Future Rain	- Albuquerque Floods from Unregulated Areas
Existing SDC	Future SDC	- Floods from the Tijeras Arroyo (SDC)
Existing Reg	Future Reg	- Albuquerque Floods from Regulated Areas

Tijeras SED Plug (Simulated Sediment Plug from Tijeras Arroyo for Existing Condition Without-Project 100-yr events)

SedPlug Rain 100	- Albuquerque Floods from Unregulated Areas
SedPlug SDC 100	- Floods from the Tijeras Arroyo (SDC)
SedPlug Reg 100	- Albuquerque Floods from Regulated Areas

High and Low FLO-2D Runs (Existing Condition Without-Project 100-yr events)

High Rain 100	Low Rain 100	- Albuquerque Floods from Unregulated Areas
High SDC 100	Low SDC 100	- Floods from the Tijeras Arroyo (SDC)
High Reg 100	Low Reg 100	- Albuquerque Floods from Regulated Areas

Inundation Maps – Flo-2d data used to develop without project inundation mapping for 10-yr, 100-yr, 200-yr, 500-yr events.

With-Project Flo-2d

(2-yr, 5-yr, 10-yr, 20-yr, 50-yr, 100-yr, 200-yr, 500-yr events)

Existing Rain	Future Rain	- Albuquerque Floods from Unregulated Areas
Existing SDC	Future SDC	- Floods from the Tijeras Arroyo (SDC)
Existing Reg	Future Reg	- Albuquerque Floods from Regulated Areas

Tentatively Selected Plan (Runs were made for the purpose of developing inundation maps for the Tentatively Selected Plan)

(2-yr, 5-yr, 10-yr, 20-yr, 50-yr, 100-yr, 200-yr, 500-yr events)

Existing Rain	Future Rain	- Albuquerque Floods from Unregulated Areas
Existing SDC	Future SDC	- Floods from the Tijeras Arroyo (SDC)
Existing Reg	Future Reg	- Albuquerque Floods from Regulated Areas

Inundation Maps – Flo-2d data used to develop Tentatively Selected Plan inundation mapping for 10-yr, 100-yr, 200-yr, 500-yr events.

Attachment 4—Summary of evaluated HEC-RAS project scenarios

MRG HEC-RAS

Plan Definitions:

Plan 1 - (geom01) (Flow1)	Existing Condition w/ Expected Manning's n values
Plan 2 - (geom02) (Flow1)	Existing Condition w/ High n & Sediment Plug
Plan 3- (geom03) (Flow1)	Existing Condition w/ Low Manning's n values
Plan 4 - (geom01) (Flow2)	Existing Condition w/ Expected Manning's n values
Plan 8 - (geom05) (Flow1)	Future Condition w/ High n & Sediment Plug
Plan 9 - (geom04) (Flow1)	Future Condition w/ Expected Manning's n values
Plan 10 - (geom06) (Flow1)	Future Condition w/ Low Manning's n values
Plan 11 - (geom04) (Flow3)	Future Condition w/ Expected Manning's n values

Flow1 – Stepped Flow from 1,000 cfs to 205,000 cfs
Flow2 – Existing With-Project Frequency Flow
Flow3 – Future With-Project Frequency Flow

Geom01 – Existing Condition w/ Expected n
Geom02 – Existing Condition w/ High n (Includes sediment plug)
Geom03 – Existing Condition w/ Low n
Geom04 – Future Condition w/ Expected n (Includes 1' sediment)
Geom05 – Future Condition w/ High n (Includes 1' sediment & sediment plug)
Geom06 – Future Condition w/ Low n (Includes 1' sediment)

Plans 1, 2 & 3 were used as Existing Condition (Year 2008) With-Project to determine Standard Deviation at index locations.

Plan 4 was used for Existing Condition (Year 2008) With-Project expected n for channel Water Surface Elevations WSEL at index locations.

Plans 8, 9 &10 were used as Future Condition (Year 2058) With-Project to determine Standard Deviation at index locations.

Plan 11 was used for Future Condition (Year 2058) With-Project expected n for channel Water Surface Elevations WSEL at index locations.

Manning's n values for expected, high and low are as follows:

	Channel n	Overbank n
Expected	0.03	0.10
High	0.035	0.12
Low	0.025	0.08

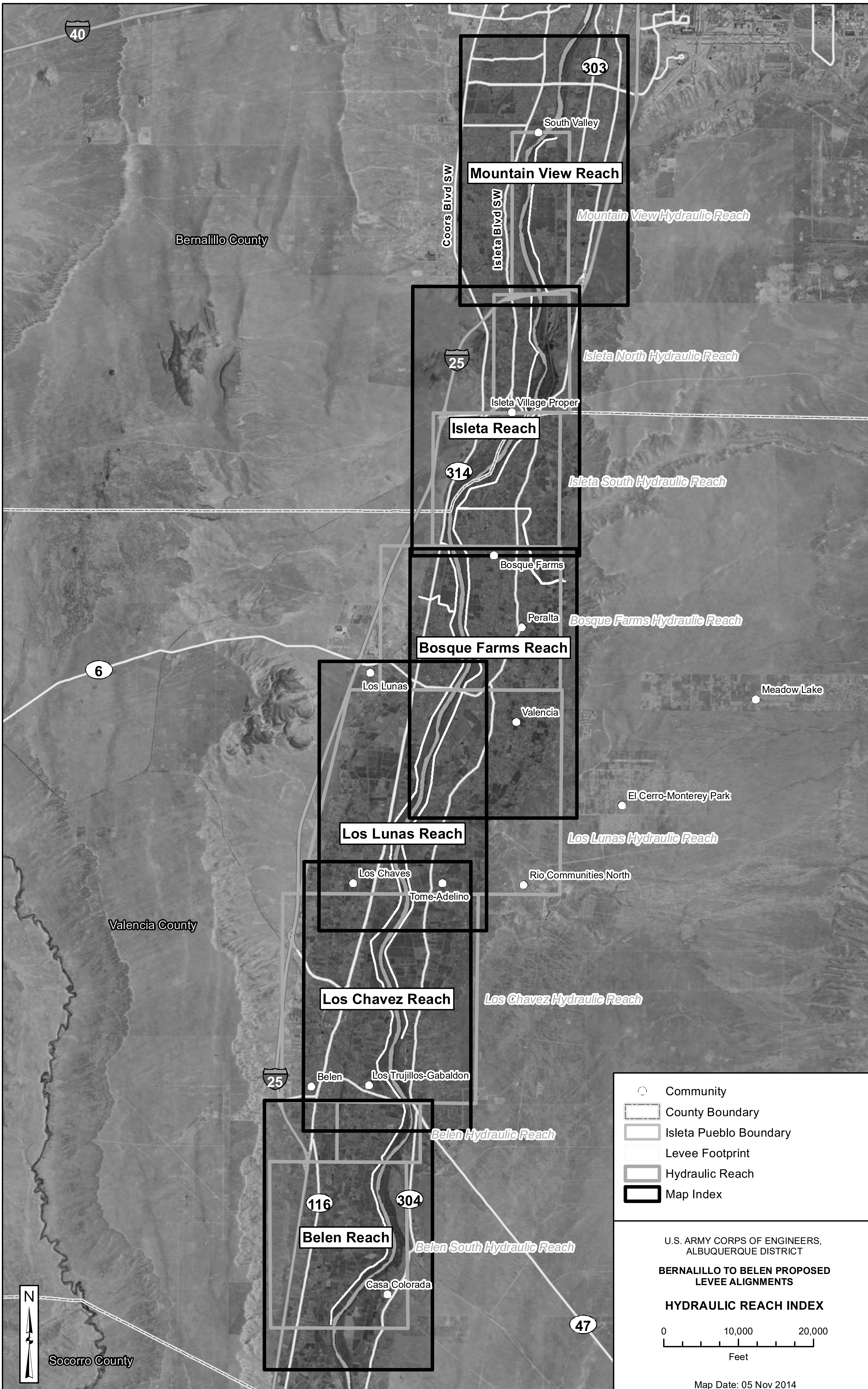
HEC-FDA Damage Reaches and Index Points are as follows:

Damage Reach Name	Index Location Station	Beginning & Ending Stations	Location
1-Mountain View	176.9	172.56 to 177.00	SDC to I-25
2-Isleta North	172.46	169.41 to 172.53	I-25 Bridge
3-Isleta South	169.29	165.44 to 169.38	I-25 to Isleta Diversion
4-Bosque Farms	165.26	161.57 to 165.35	Isleta Diversion to Bosque Farms
5-Los Lunas	161.48	156.12 to 161.54	Bosque Farms to Los Lunas
6-Los Chaves	155.92	150.43 to 156.02	Los Lunas to Los Chaves
7-Belen	150.34	148.50 to 150.40	Los Chaves to Belen Hwy Bridge
8-Belen RR	148.3	147.04 to 148.40	Belen Hwy to Belen RR Bridge

Important Study Locations and Stations are as follows:

X-sec #	HEC-RAS Station	FLO-2D Grid #	Location
1 - Gage	183.4	5069	Central Ave. Bridge Albuquerque
2	178.3	6662	Rio Bravo Bridge, Albuquerque
3	176.9	7199	SDC – Tijeras Arroyo Confluence
4	172.46	8629	I-25 Bridge
5	169.29	9385	Isleta Diversion
6	165.26	10498	Bosque Farms
7	161.48	12027	Los Lunas Bridge
8	155.92	14635	Los Chaves
9	150.34	16505	Belen Hwy Bridge (Rt. 309)
10	148.3	16991	Belen Railroad Bridge

Attachment 5—FLO-2D Inundation maps without project



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South Valley
Mountain View Reach
Coors Blvd SW
Isleta Blvd SW
Mountain View Hydraulic Reach
Bernalillo County
25
Isleta North Hydraulic Reach
Isleta Village Proper
Isleta Reach
314
Isleta South Hydraulic Reach
Bosque Farms
Peralta
Bosque Farms Hydraulic Reach
Bosque Farms Reach
6
Los Lunas
Meadow Lake
Valencia
El Cerro-Monterey Park
Los Lunas Reach
Los Lunas Hydraulic Reach
Los Chaves
Tome-Adelino
Rio Communities North
Valencia County
Los Chavez Reach
Los Chavez Hydraulic Reach
25
Belen
Los Trujillos-Gabaldon
Belen Hydraulic Reach
116
304
Belen Reach
Belen South Hydraulic Reach
Casa Colorada
47
N
Socorro County
Community
County Boundary
Isleta Pueblo Boundary
Levee Footprint
Hydraulic Reach
Map Index
U.S. ARMY CORPS OF ENGINEERS,
ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN PROPOSED
LEVEE ALIGNMENTS
HYDRAULIC REACH INDEX
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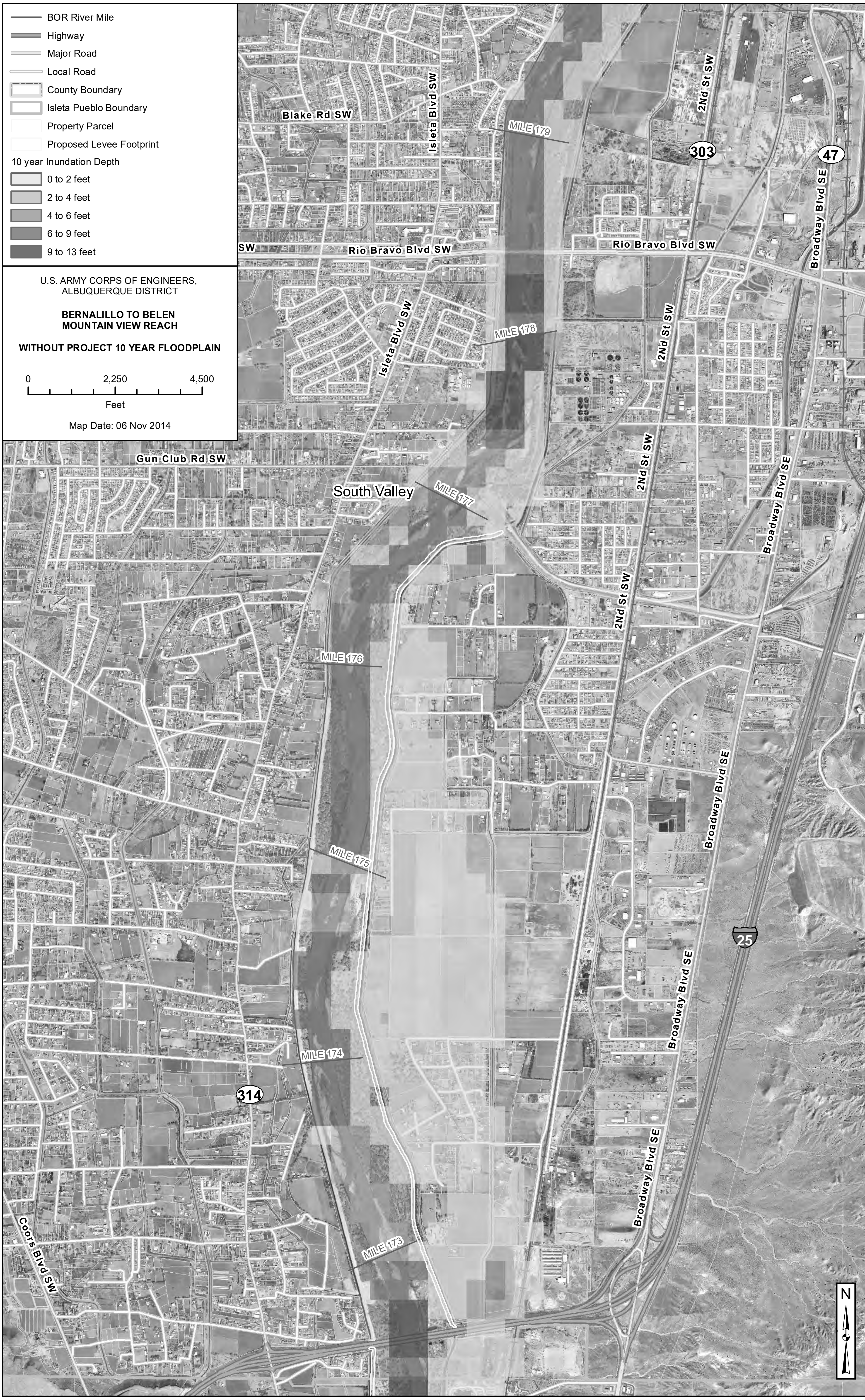
20,000
Feet
Map Date: 05 Nov 2014



BOR River Mile
Highway
Major Road
Local Road
County Boundary
Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
10 year Inundation Depth
0 to 2 feet
2 to 4 feet
4 to 6 feet
6 to 9 feet
9 to 13 feet
U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
MOUNTAIN VIEW REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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2,250
4,500
Feet
Map Date: 06 Nov 2014
Blake Rd SW
Isleta Blvd SW
MILE 179
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303
47
Rio Bravo Blvd SW
Broadway Blvd SE
MILE 178
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South Valley
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2Nd St SW
MILE 176
MILE 175
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MILE 174
314
Coors Blvd SW
MILE 173
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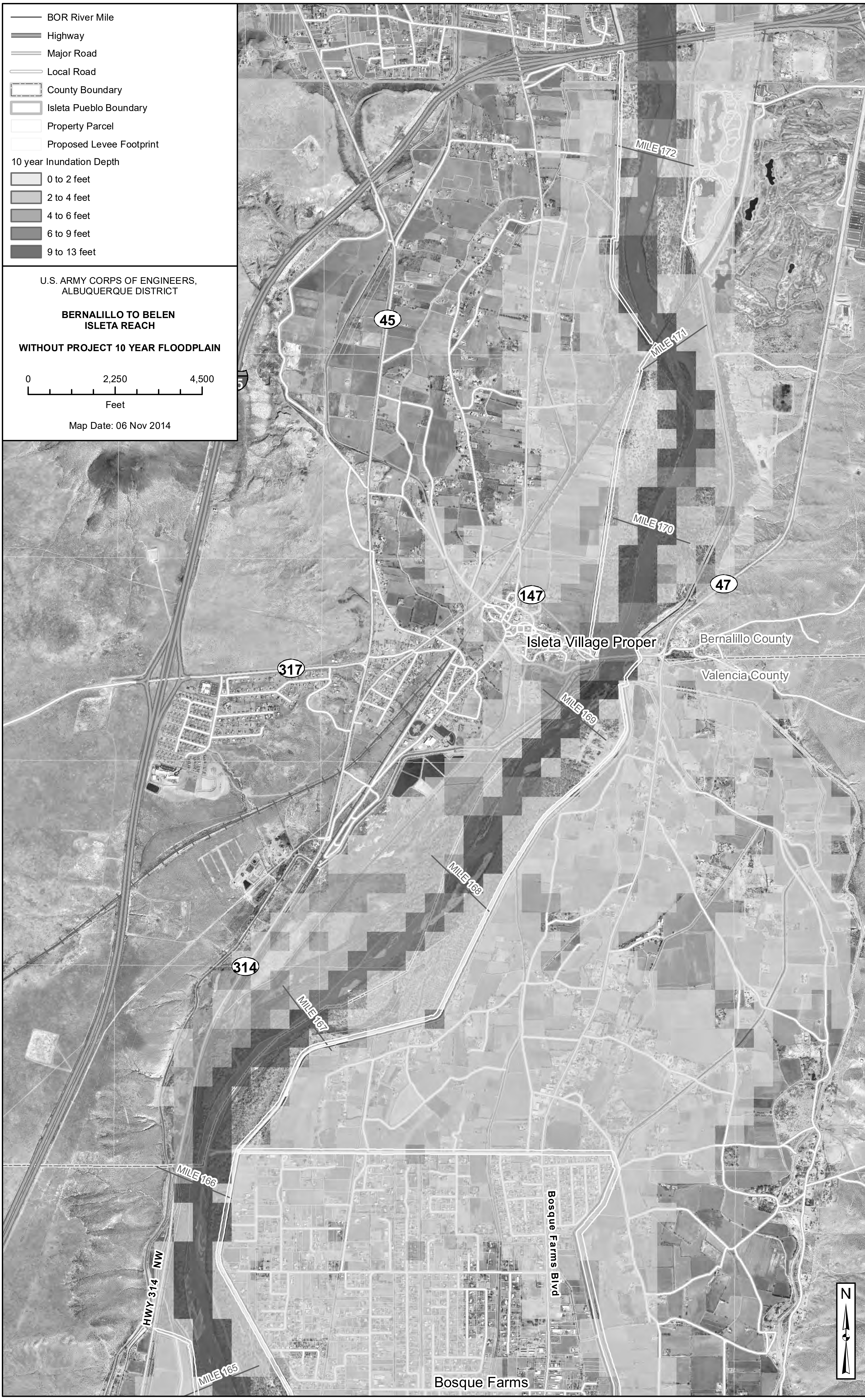
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County Boundary
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Property Parcel
Proposed Levee Footprint
10 year Inundation Depth
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U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN ISLETA REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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Feet
Map Date: 06 Nov 2014
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MILE 170
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MILE 168
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314
Isleta Village Proper
Bernalillo County
Valencia County
Bosque Farms Blvd
HWY 314 NW
Bosque Farms
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Bosque Farms
Peralta
Valencia
314
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HWY 314 NW
HWY 314 SW
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MILE 164
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MILE 162
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MILE 160
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158
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BOR River Mile
Highway
Major Road
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County Boundary
Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
10 year Inundation Depth
0 to 2 feet
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U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
BOSQUE FARMS REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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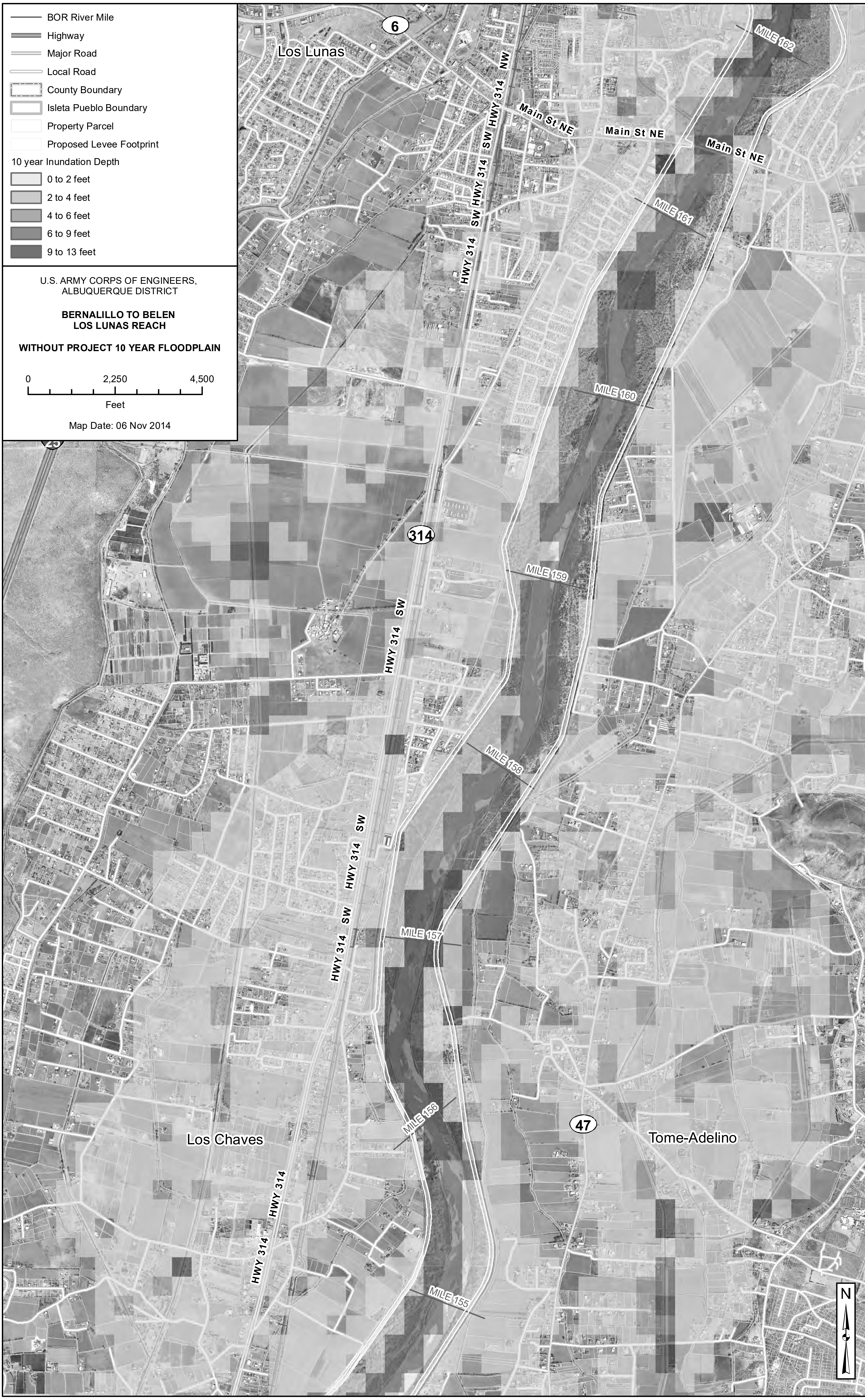
Map Date: 06 Nov 2014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Property Parcel
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2 to 4 feet
4 to 6 feet
6 to 9 feet
9 to 13 feet
U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
LOS LUNAS REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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Feet
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Main St NE
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MILE 160
MILE 159
MILE 158
MILE 157
MILE 156
MILE 155
314
HWY 314 SW
HWY 314 SW
HWY 314 SW
Los Chaves
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Tome-Adelino
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HWY 314
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Los Chaves
314
Tome-Adelino
MILE 156
HWY 314
MILE 155
HWY 314
HWY 314
MILE 154
MILE 153
47
Frontage Rd
HWY 314
MILE 152
MILE 151
N Main St
W Reinken Ave
E Reinken Ave
Belen
Los Trujillos-Gabaldon
MILE 150
E River Rd
548
309
304
MILE 149
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S Main St
BOR River Mile
Highway
Major Road
Local Road
County Boundary
Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
10 year Inundation Depth
0 to 2 feet
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ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
LOS CHAVEZ REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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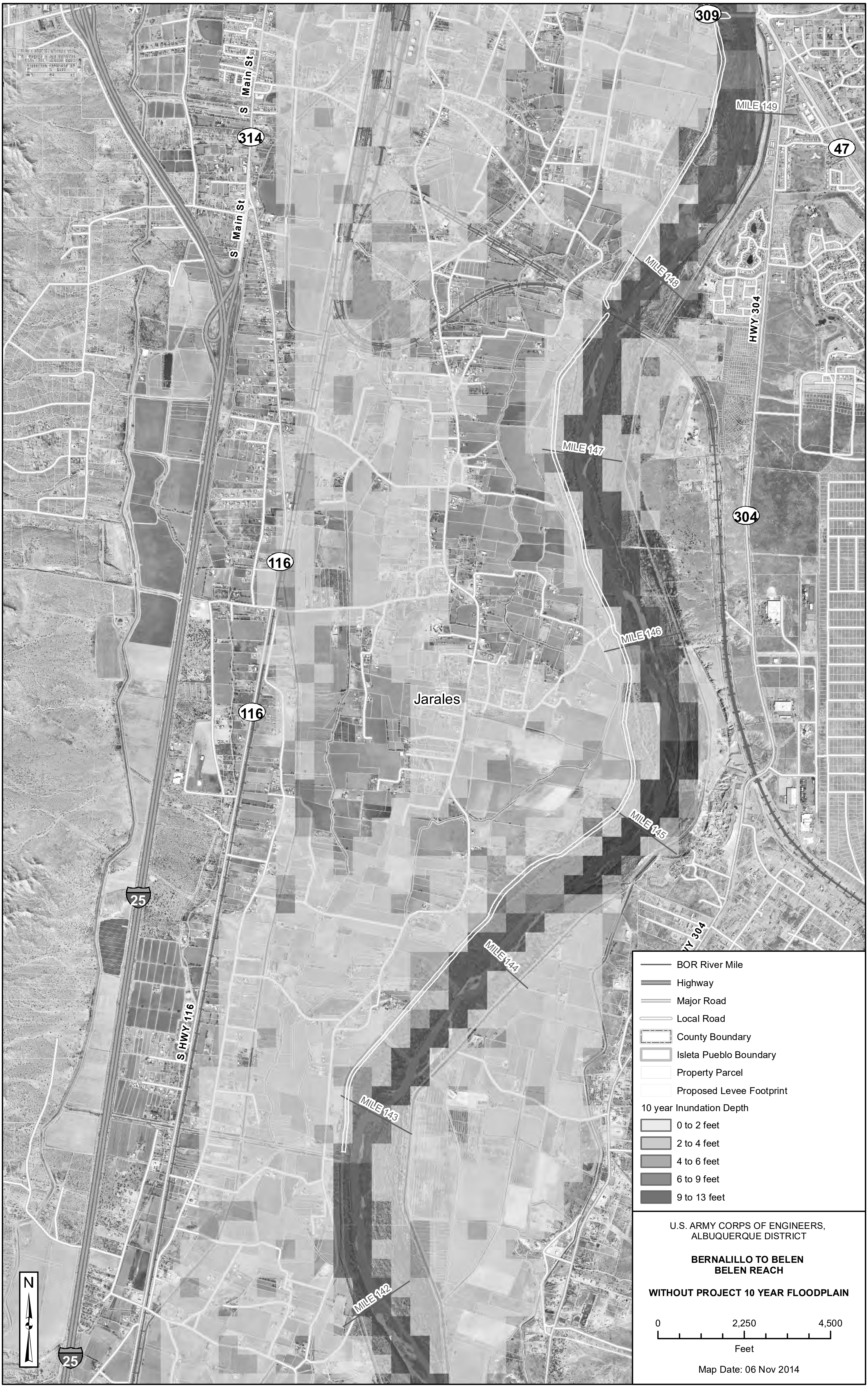

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HWY 304
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MILE 142
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BOR River Mile
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County Boundary
Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
10 year Inundation Depth
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U.S. ARMY CORPS OF ENGINEERS,
ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
BELEN REACH
WITHOUT PROJECT 10 YEAR FLOODPLAIN
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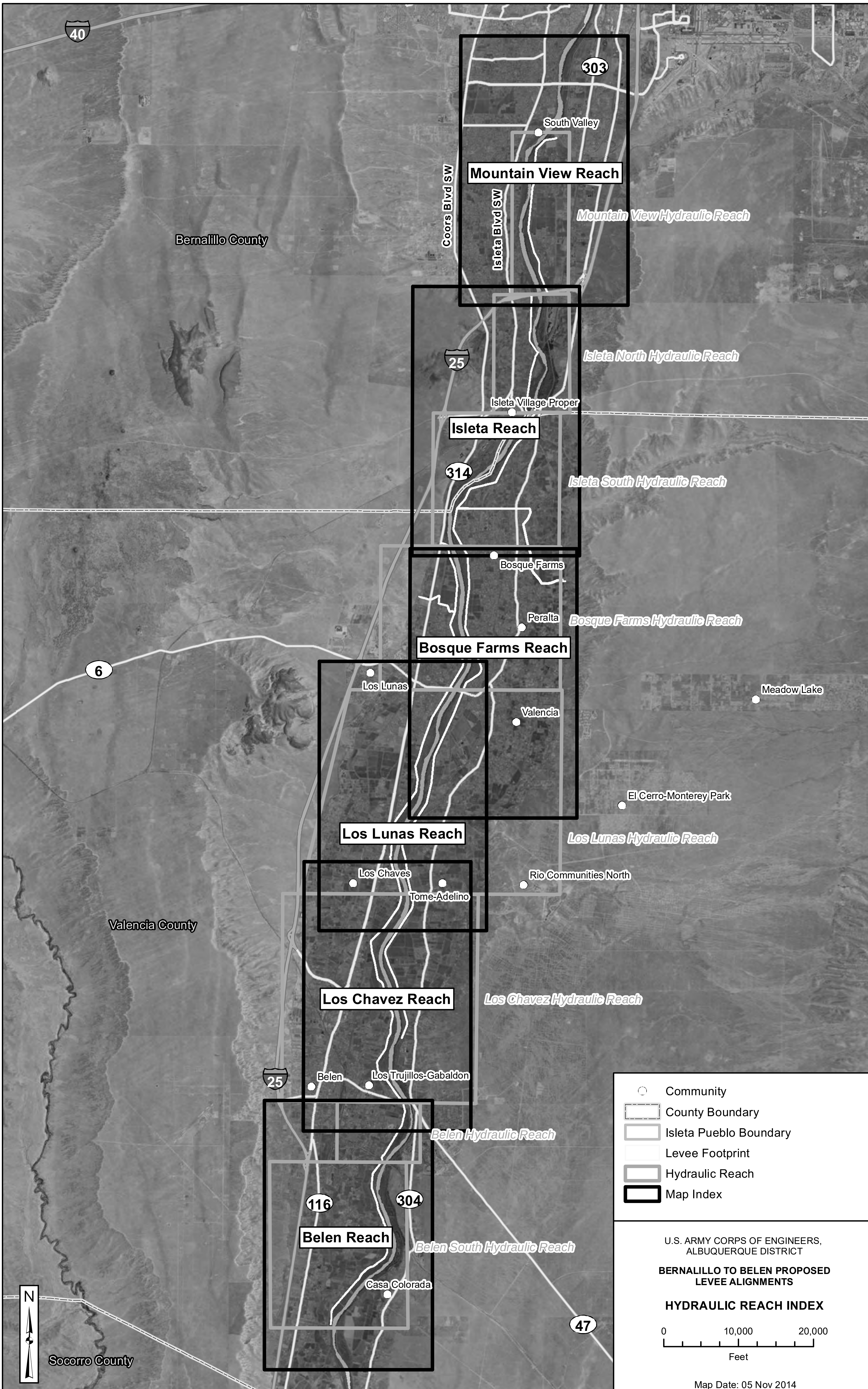

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Bernalillo County
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Isleta Reach
314
Isleta South Hydraulic Reach
Bosque Farms
Peralta
Bosque Farms Hydraulic Reach
Bosque Farms Reach
6
Los Lunas
Meadow Lake
Valencia
El Cerro-Monterey Park
Los Lunas Reach
Los Lunas Hydraulic Reach
Los Chaves
Tome-Adelino
Rio Communities North
Valencia County
Los Chavez Reach
Los Chavez Hydraulic Reach
25
Belen
Los Trujillos-Gabaldon
Belen Hydraulic Reach
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Belen Reach
Belen South Hydraulic Reach
Casa Colorada
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Socorro County
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Isleta Pueblo Boundary
Levee Footprint
Hydraulic Reach
Map Index
U.S. ARMY CORPS OF ENGINEERS,
ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN PROPOSED
LEVEE ALIGNMENTS
HYDRAULIC REACH INDEX
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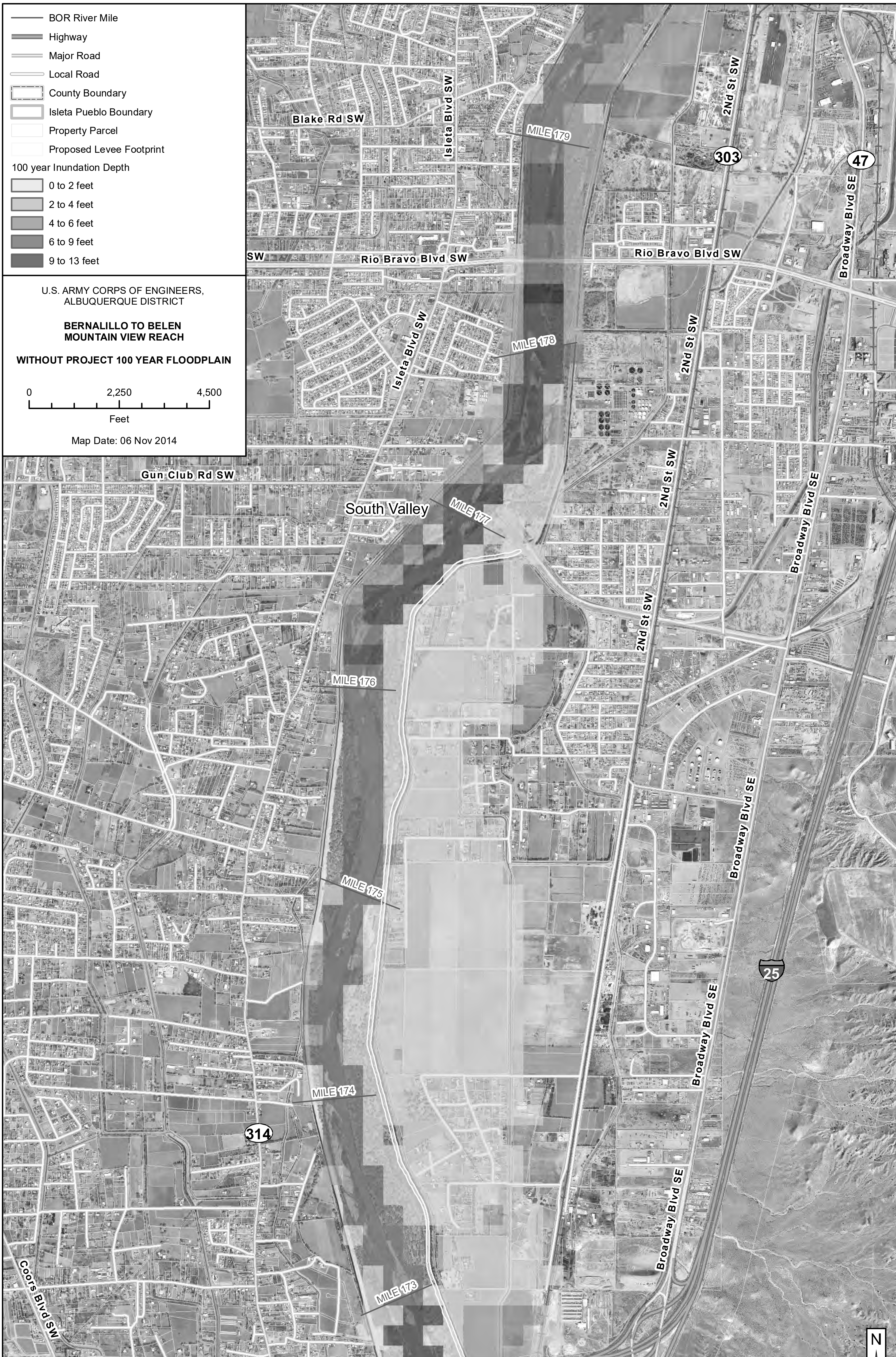
20,000
Feet
Map Date: 05 Nov 2014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Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
100 year Inundation Depth
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6 to 9 feet
9 to 13 feet
U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
MOUNTAIN VIEW REACH
WITHOUT PROJECT 100 YEAR FLOODPLAIN
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4,500
Feet
Map Date: 06 Nov 2014
Blake Rd SW
Isleta Blvd SW
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Rio Bravo Blvd SW
Broadway Blvd SE
MILE 178
Gun Club Rd SW
South Valley
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MILE 173
Coors Blvd SW
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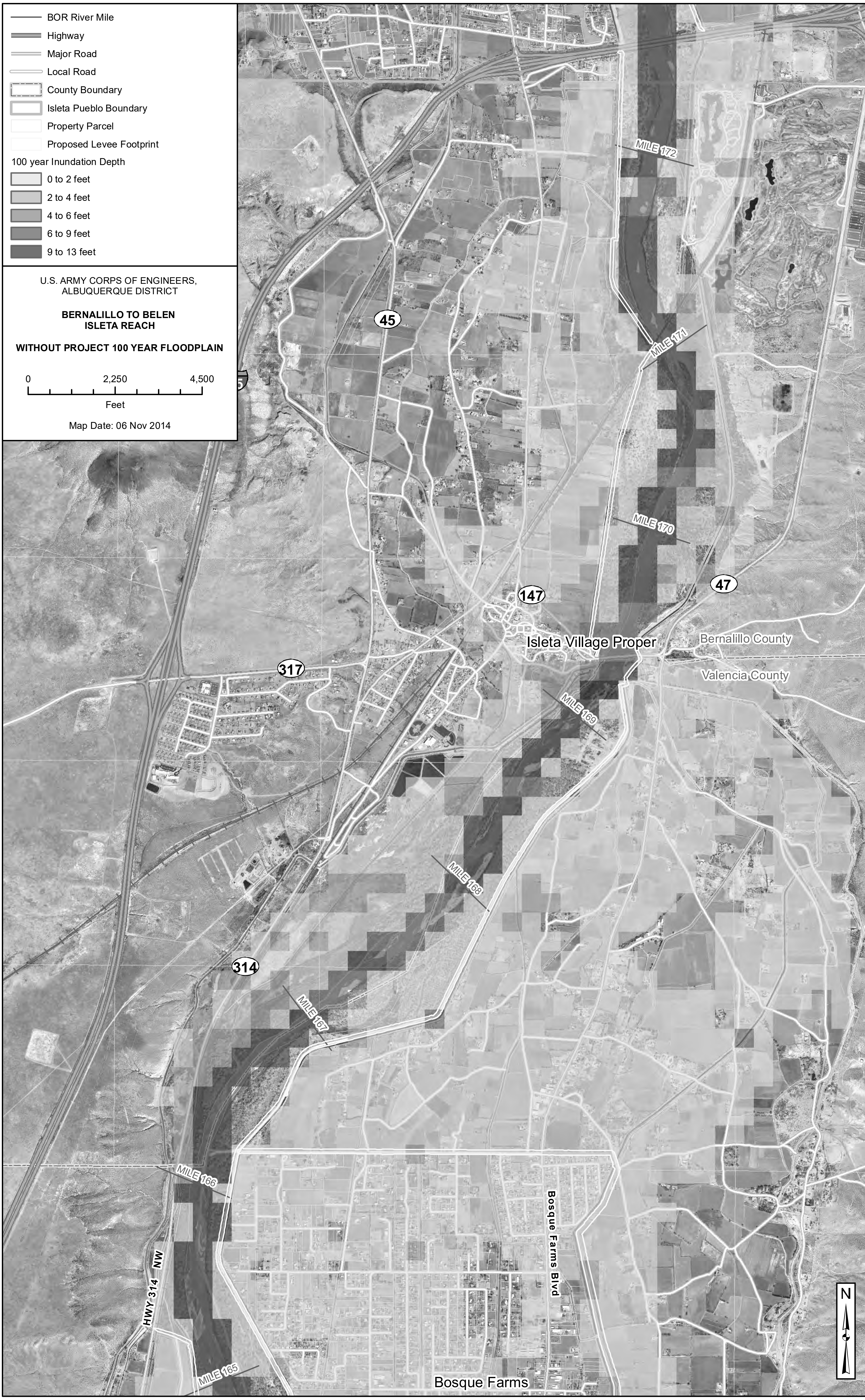

BOR River Mile
Highway
Major Road
Local Road
County Boundary
Isleta Pueblo Boundary
Property Parcel
Proposed Levee Footprint
100 year Inundation Depth
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4 to 6 feet
6 to 9 feet
9 to 13 feet
U.S. ARMY CORPS OF ENGINEERS,
ALBUQUERQUE DISTRICT
BERNALILLO TO BELEN
ISLETA REACH
WITHOUT PROJECT 100 YEAR FLOODPLAIN
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2,250
4,500
Feet
Map Date: 06 Nov 2014
MILE 172
MILE 171
MILE 170
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100 year Inundation Depth
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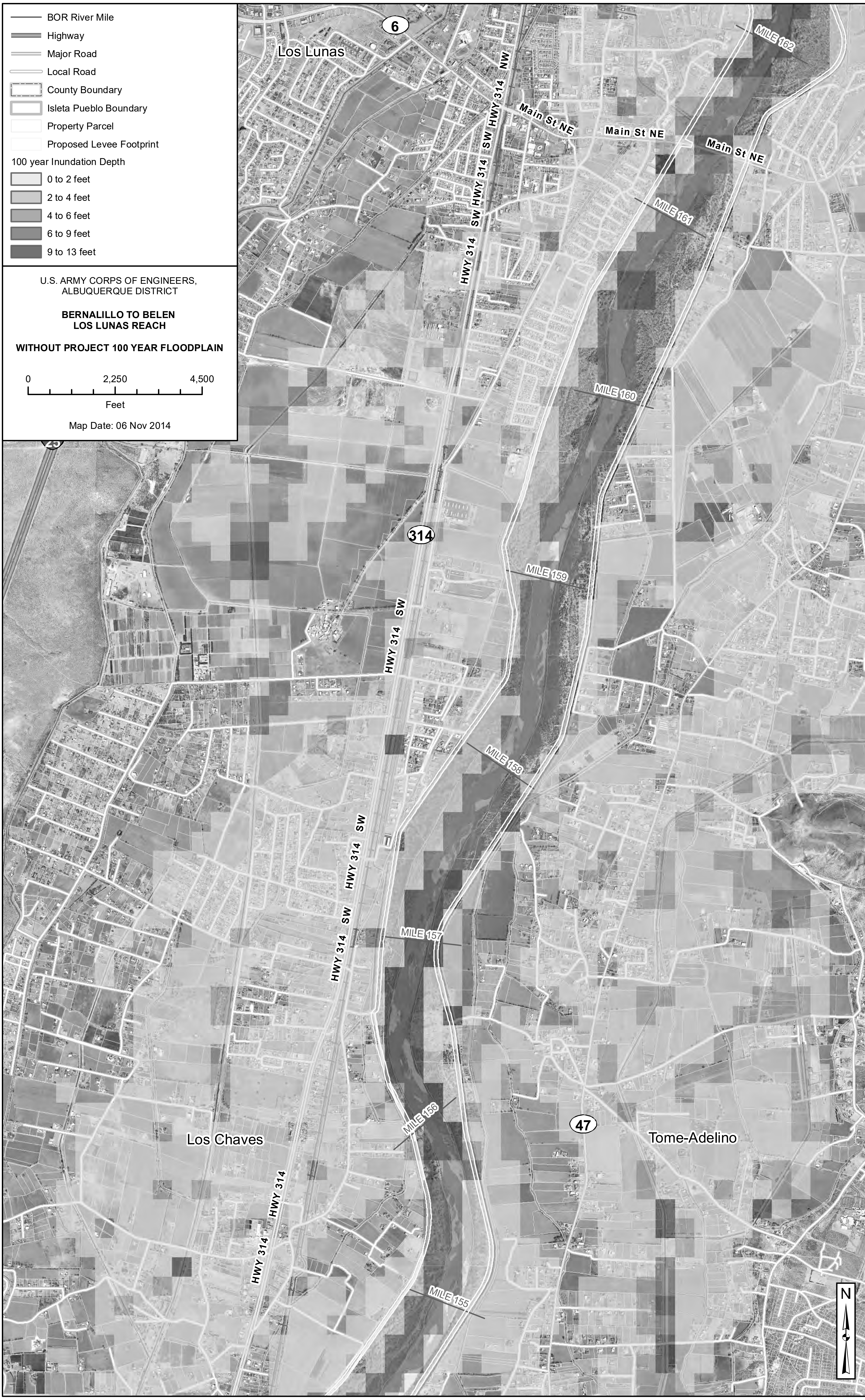
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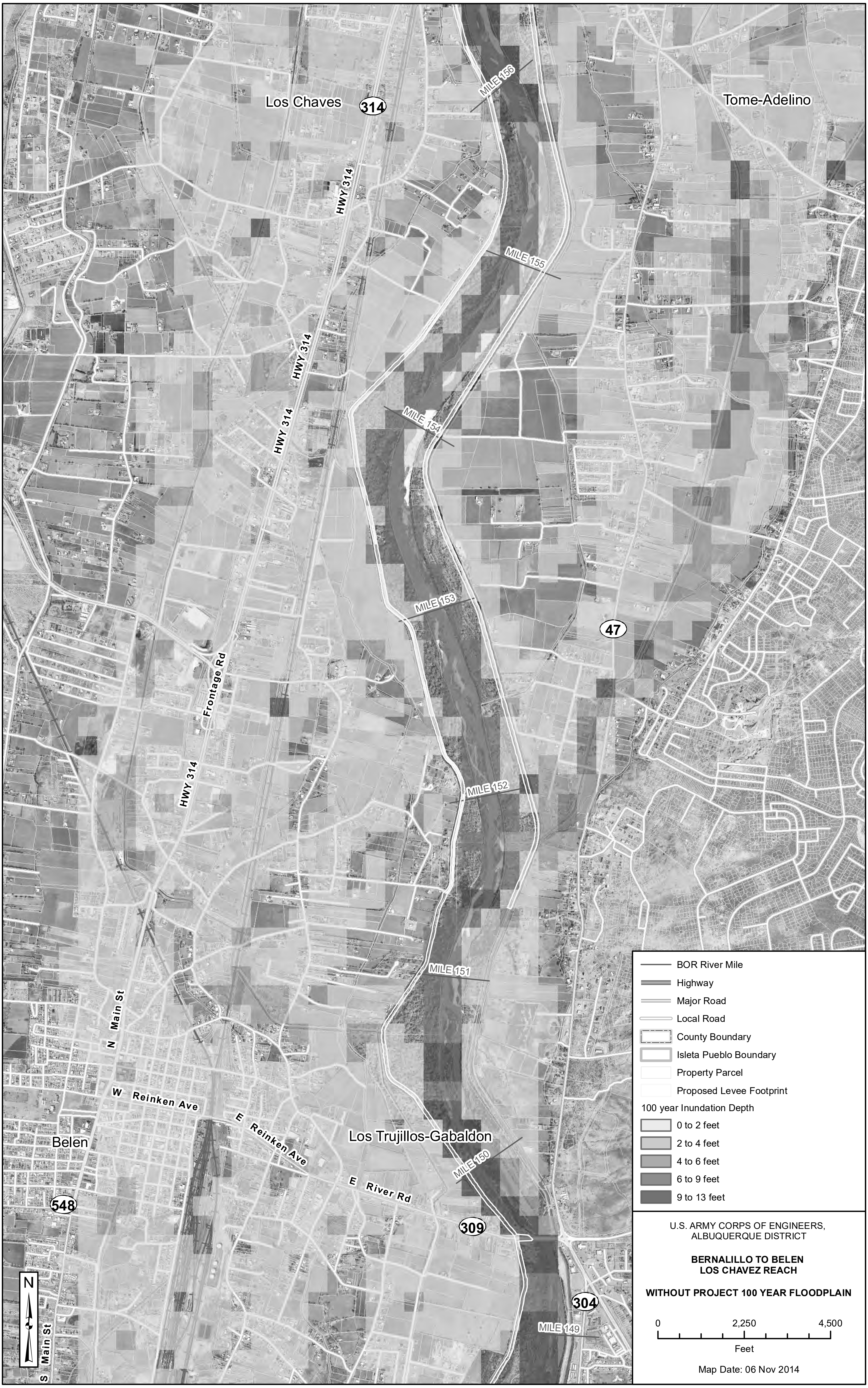
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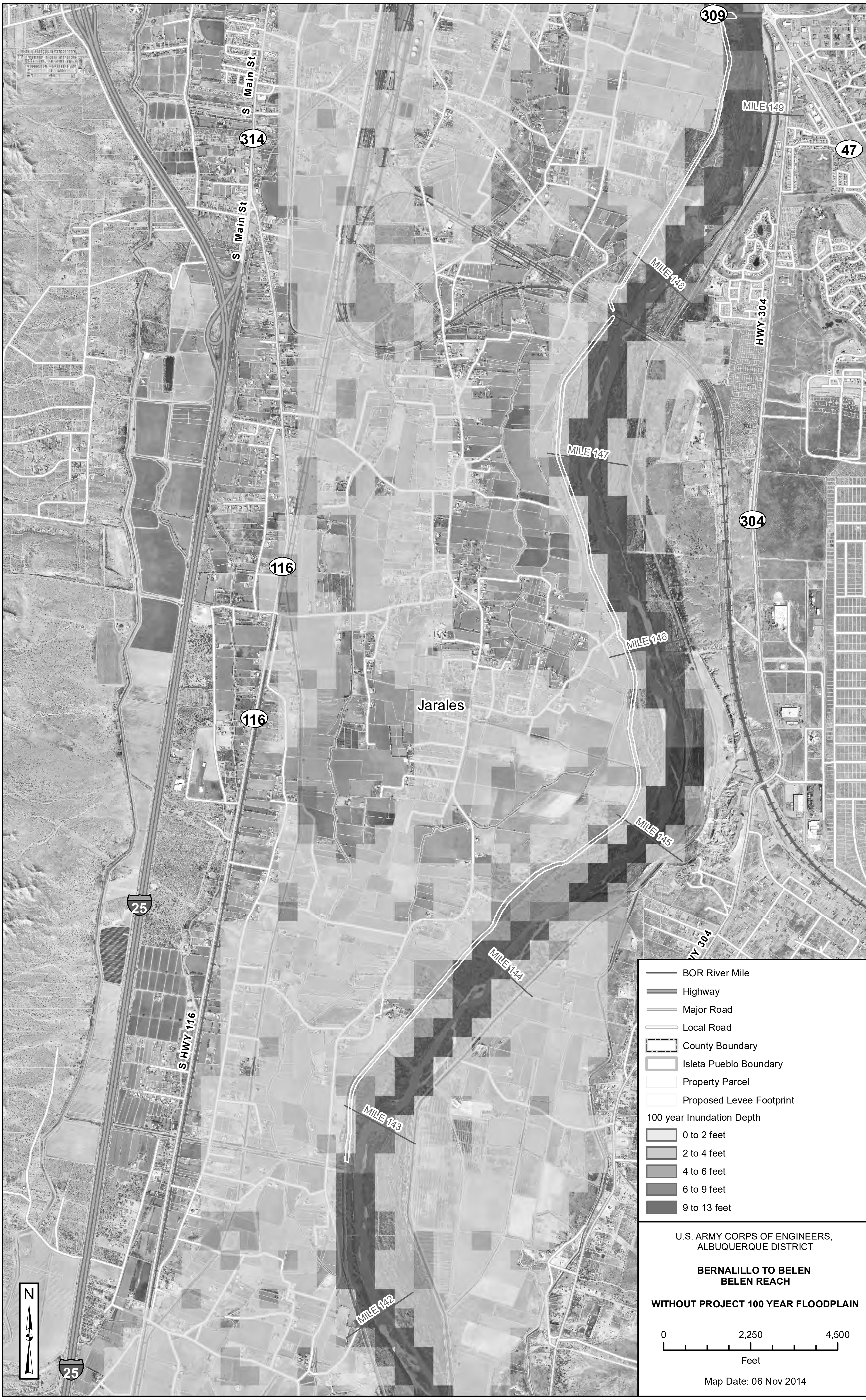

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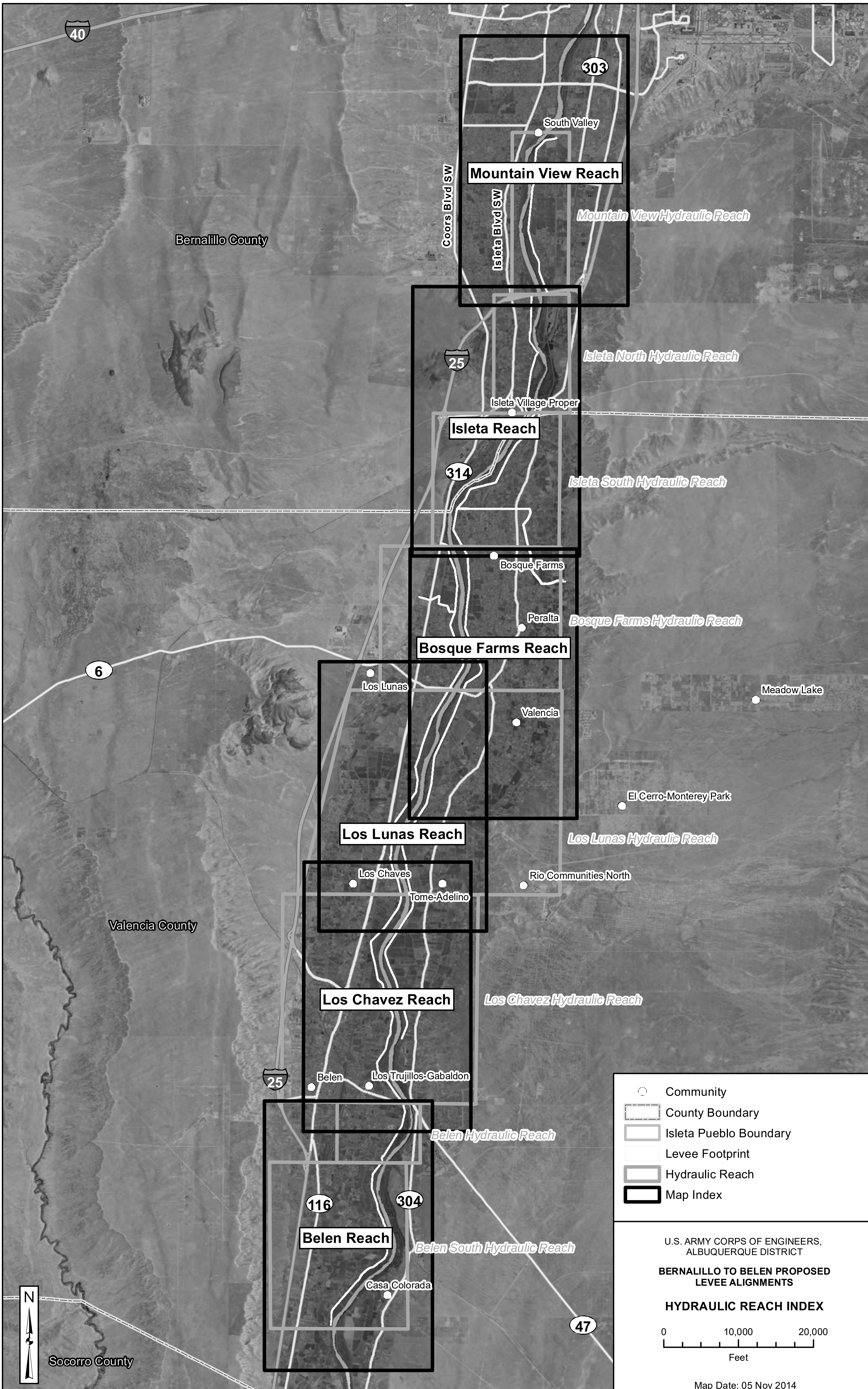
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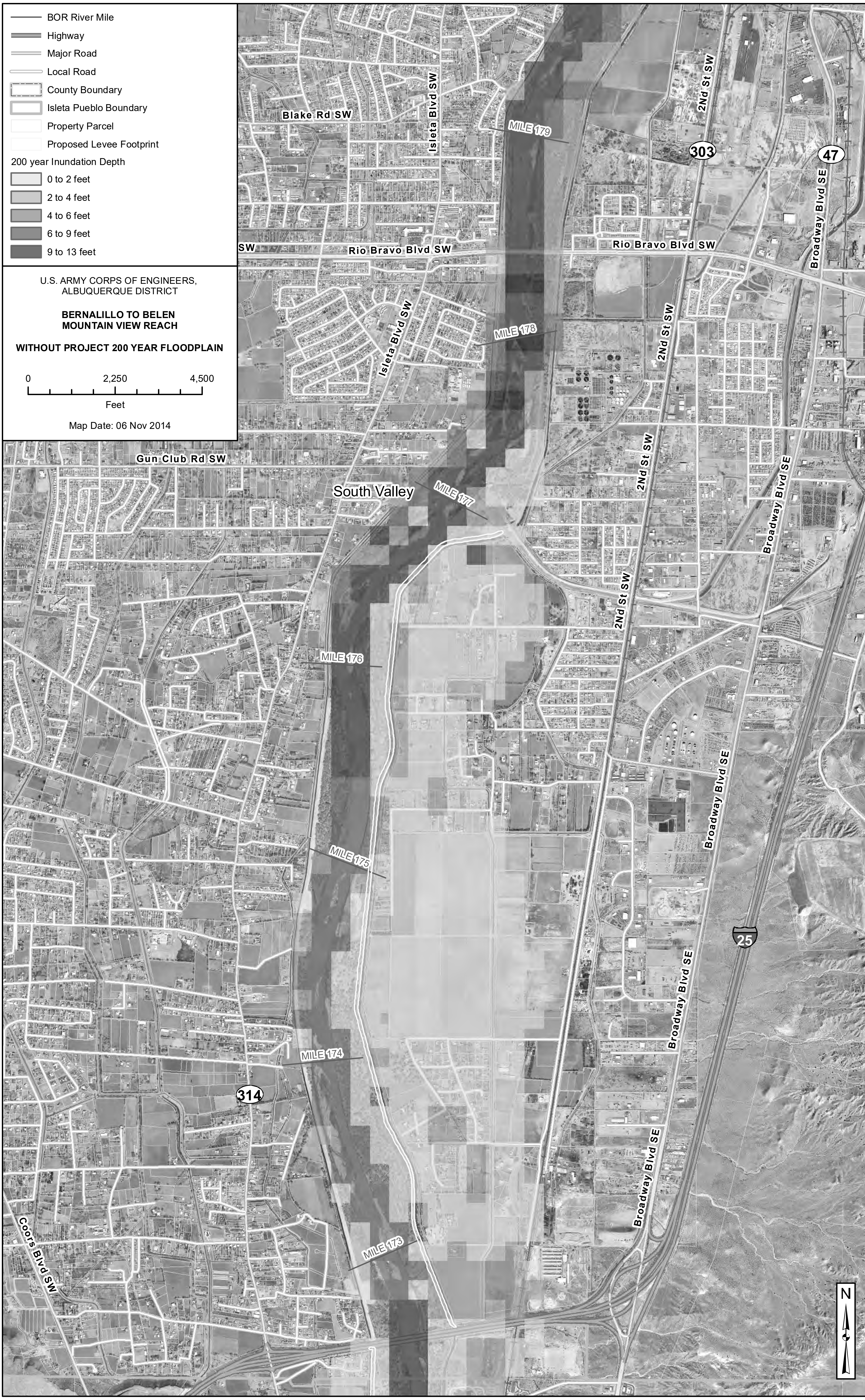

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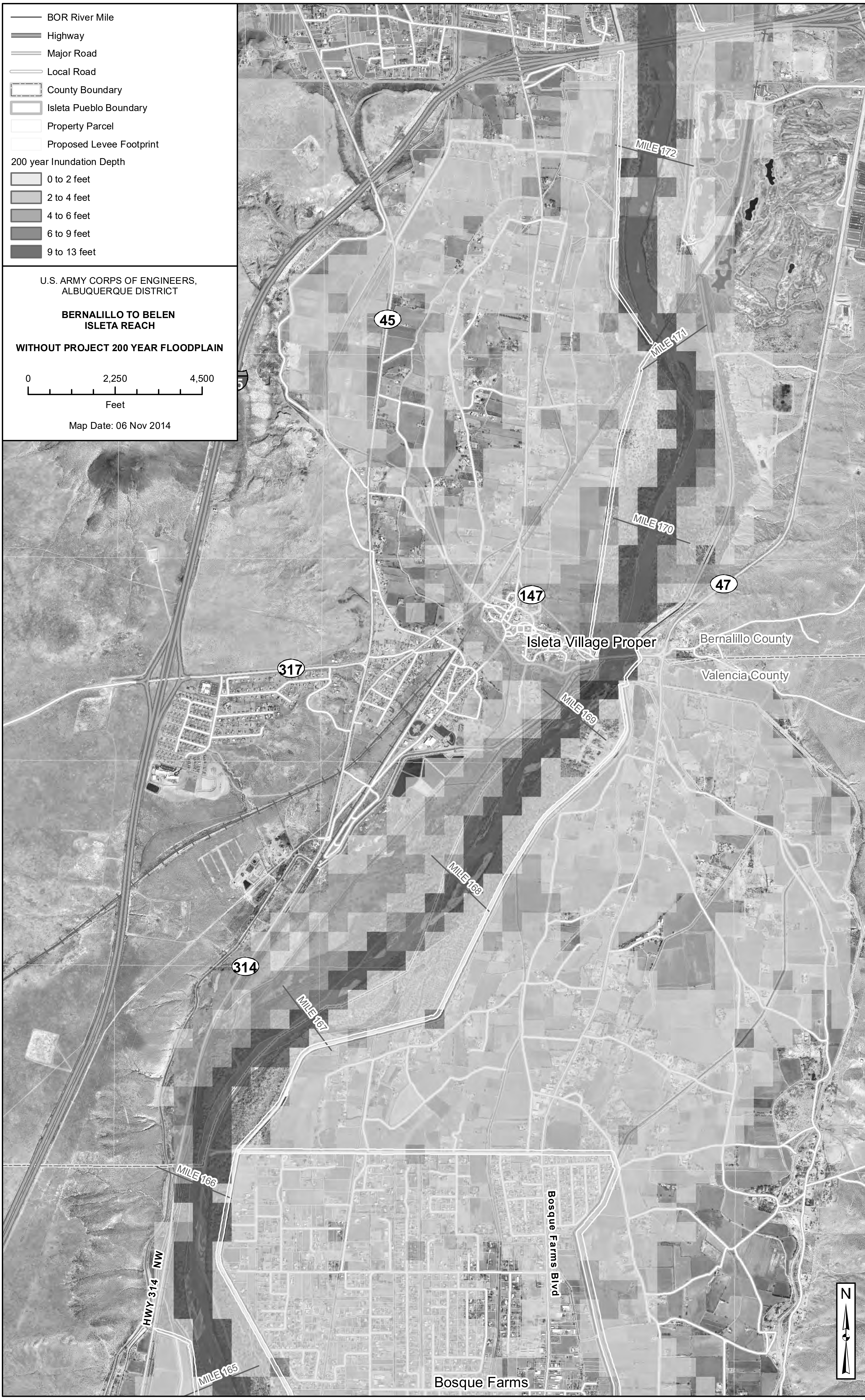

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Proposed Levee Footprint
200 year Inundation Depth
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BERNALILLO TO BELEN
BOSQUE FARMS REACH
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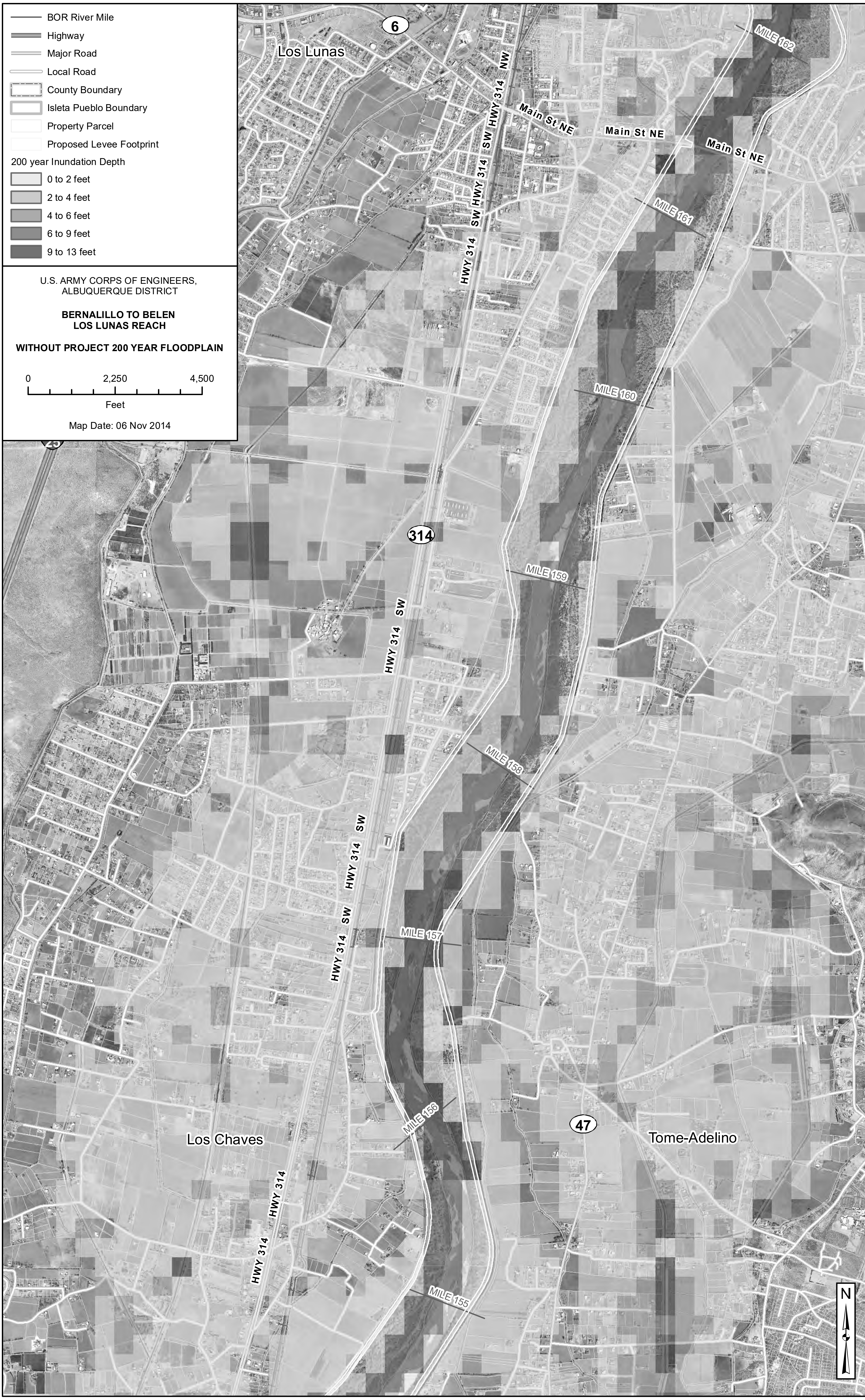

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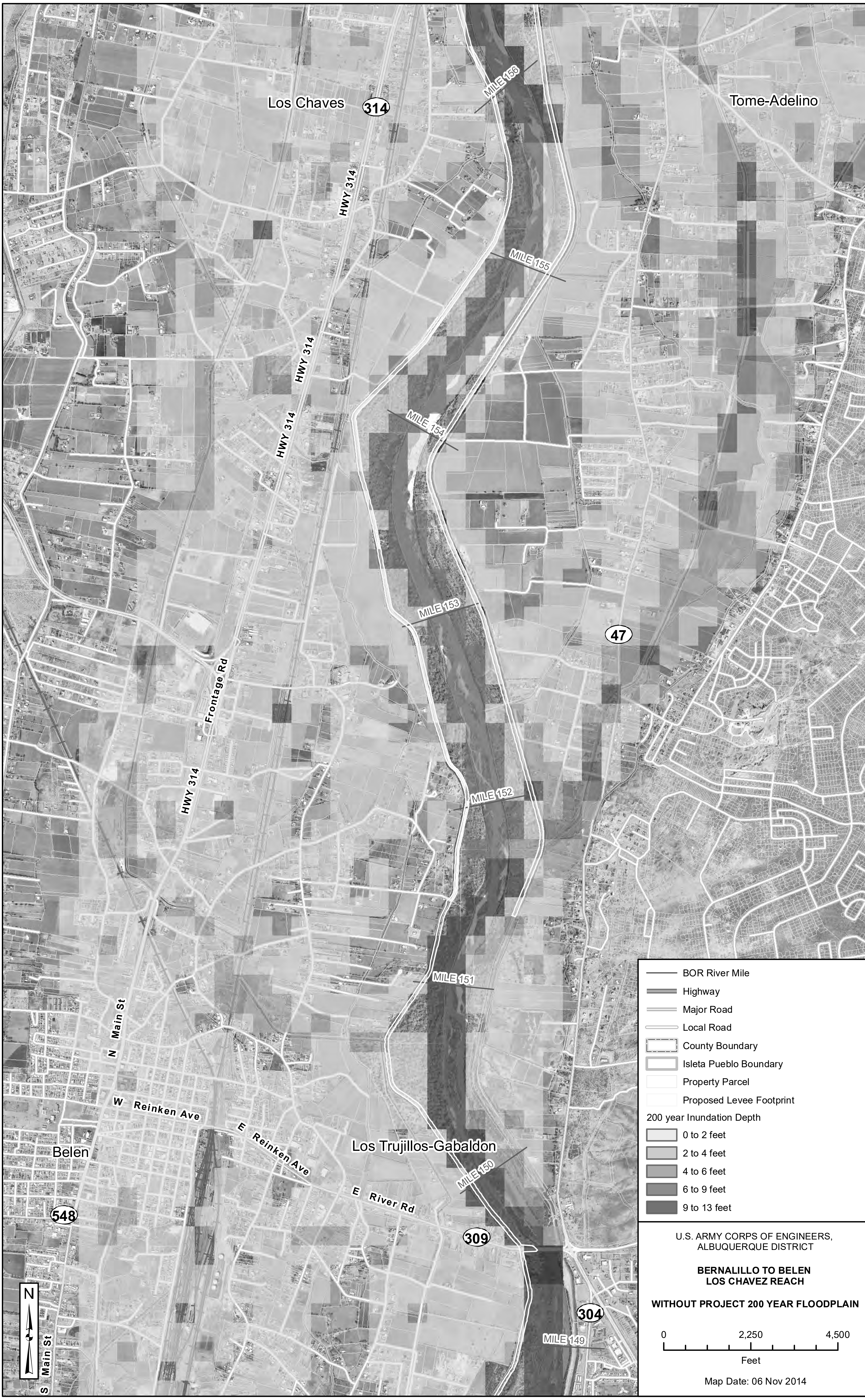

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U.S. ARMY CORPS OF ENGINEERS,
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LOS CHAVEZ REACH
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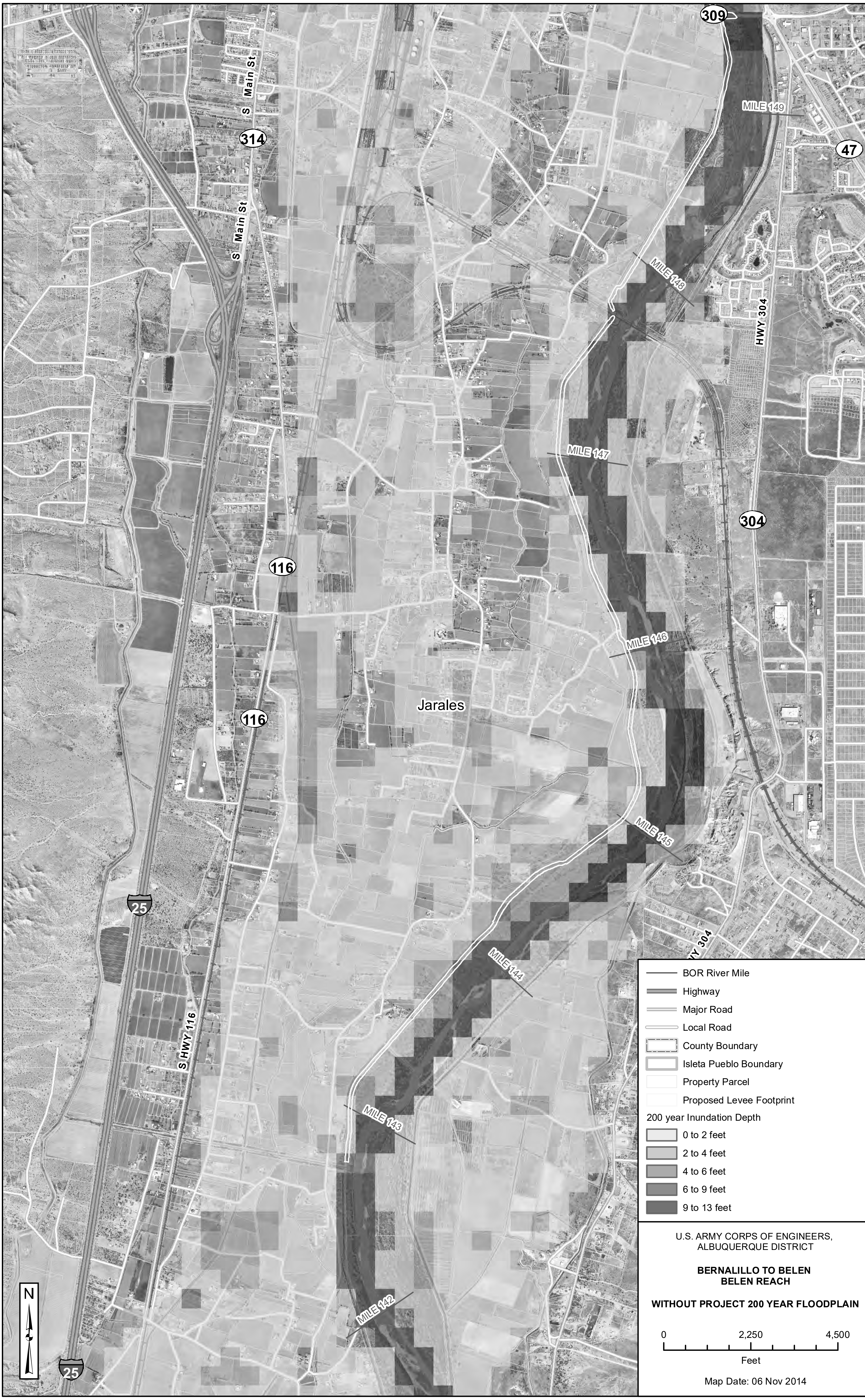

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BOR River Mile
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U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
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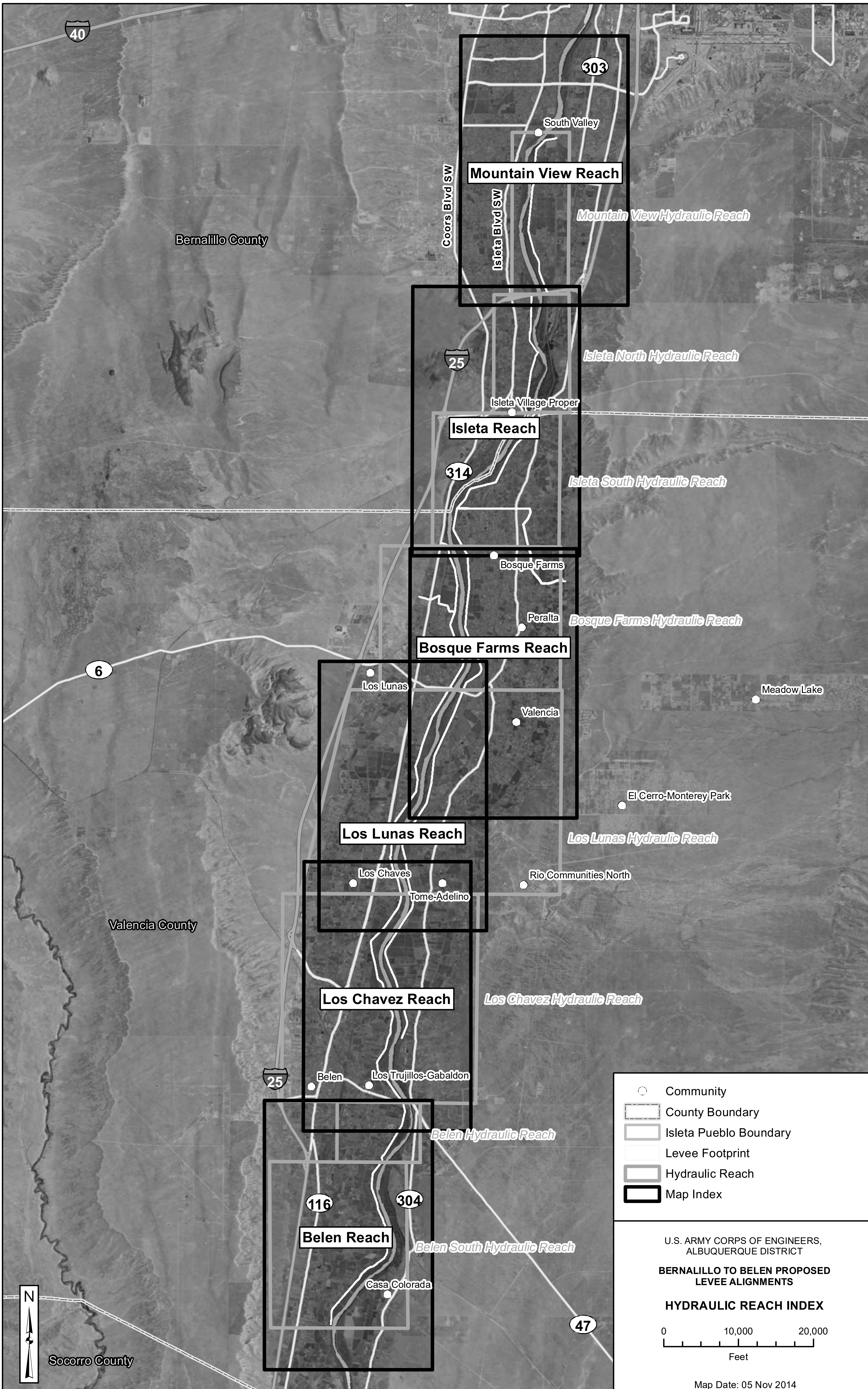

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Bosque Farms
Peralta
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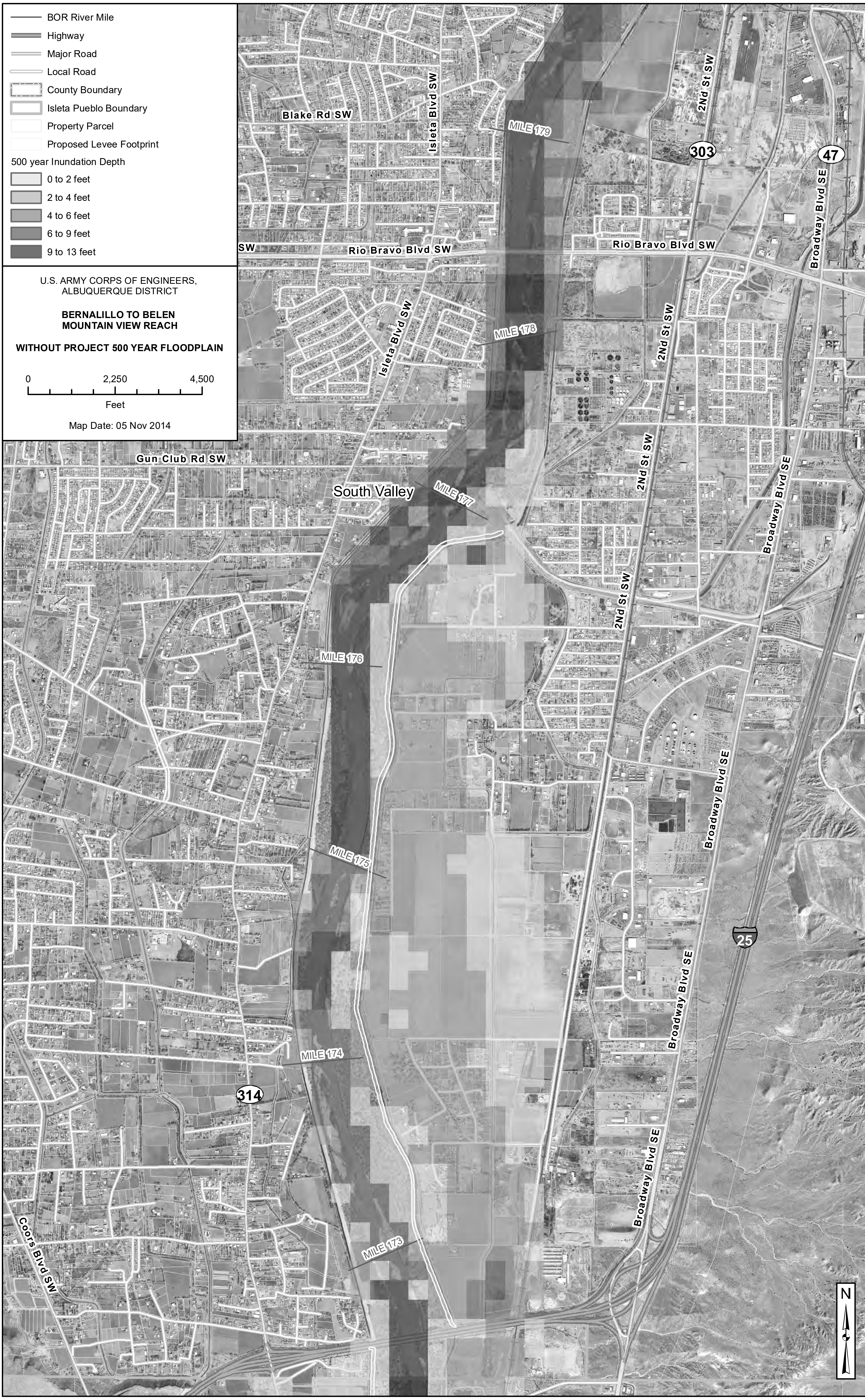

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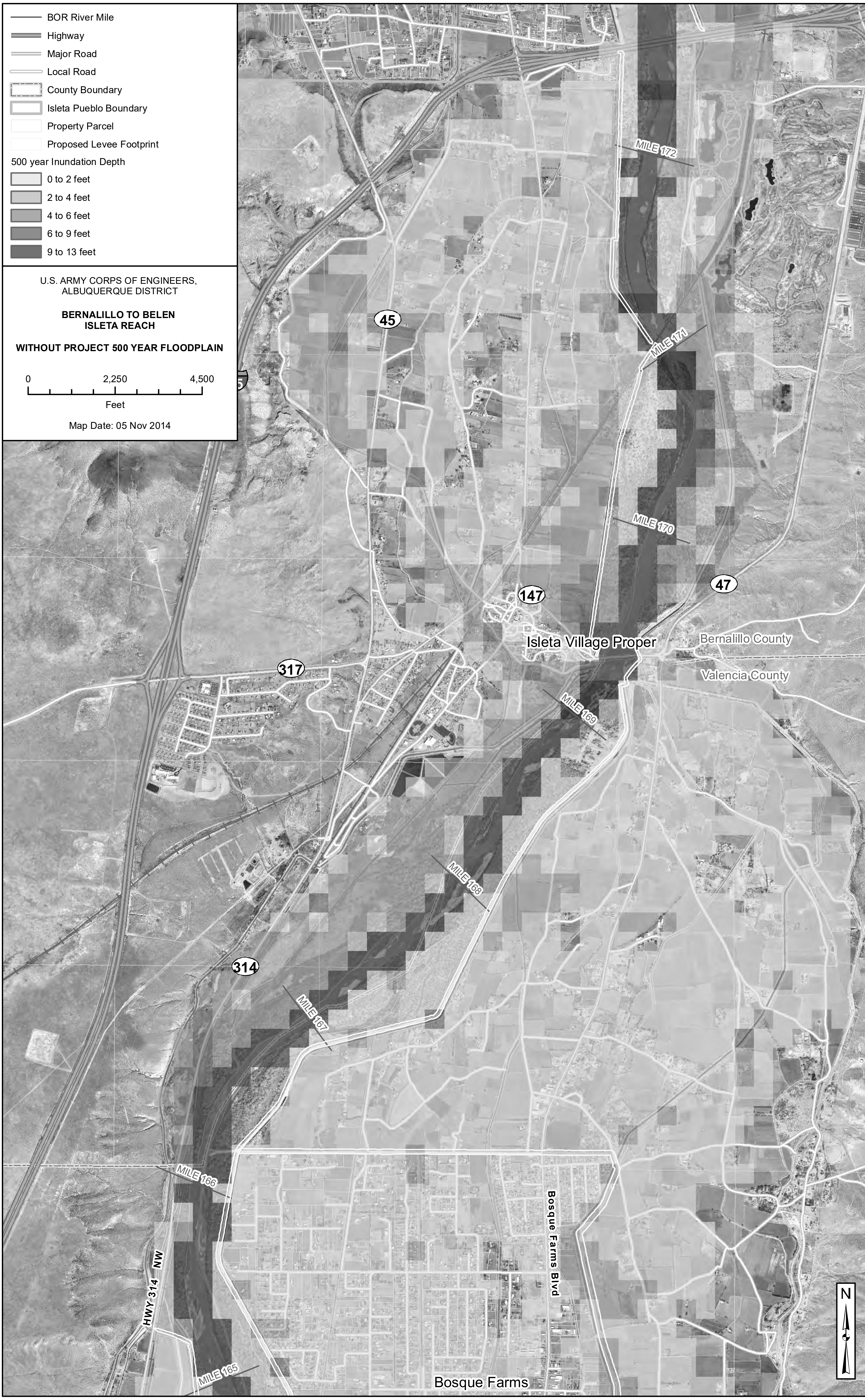

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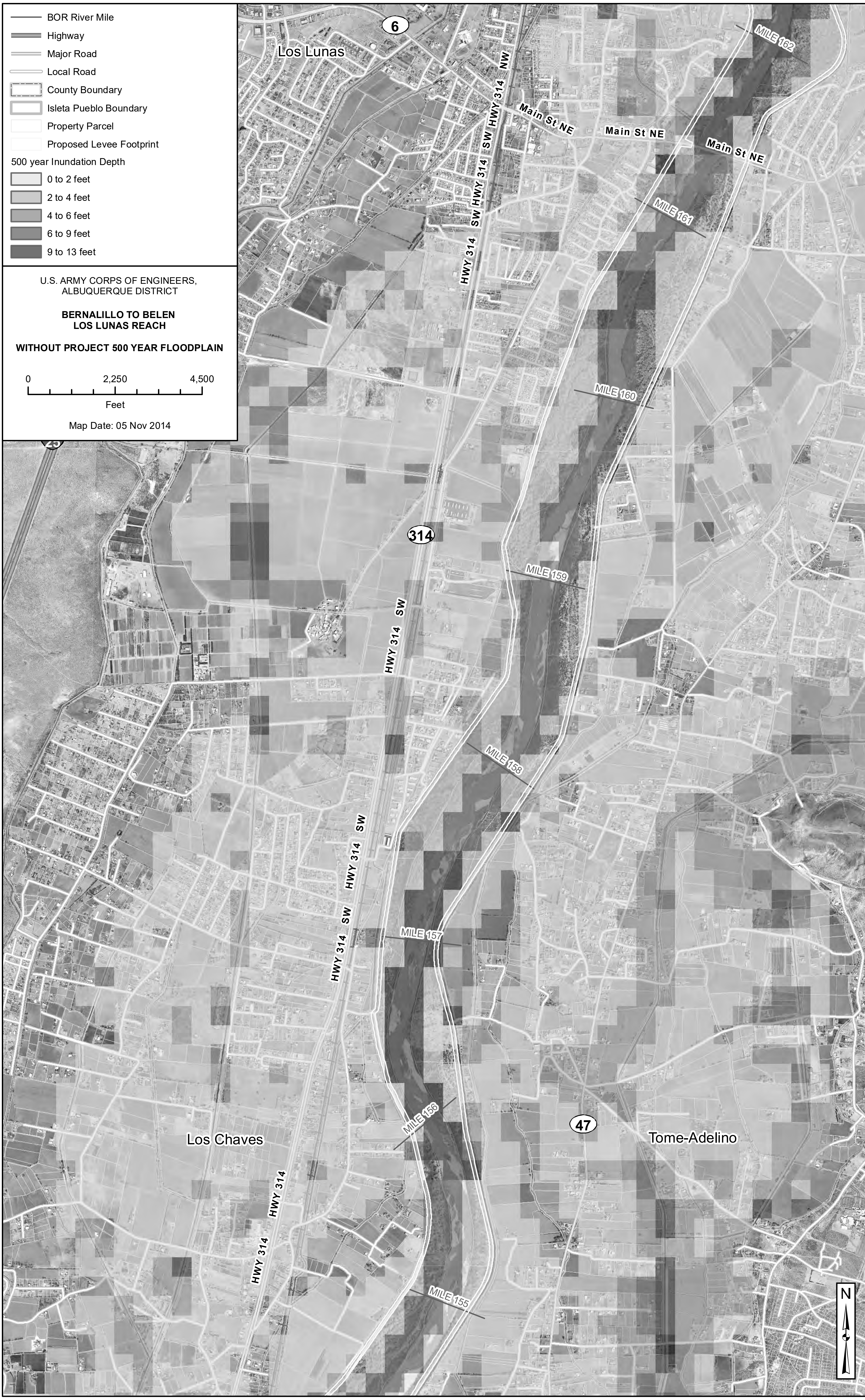

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Los Chaves
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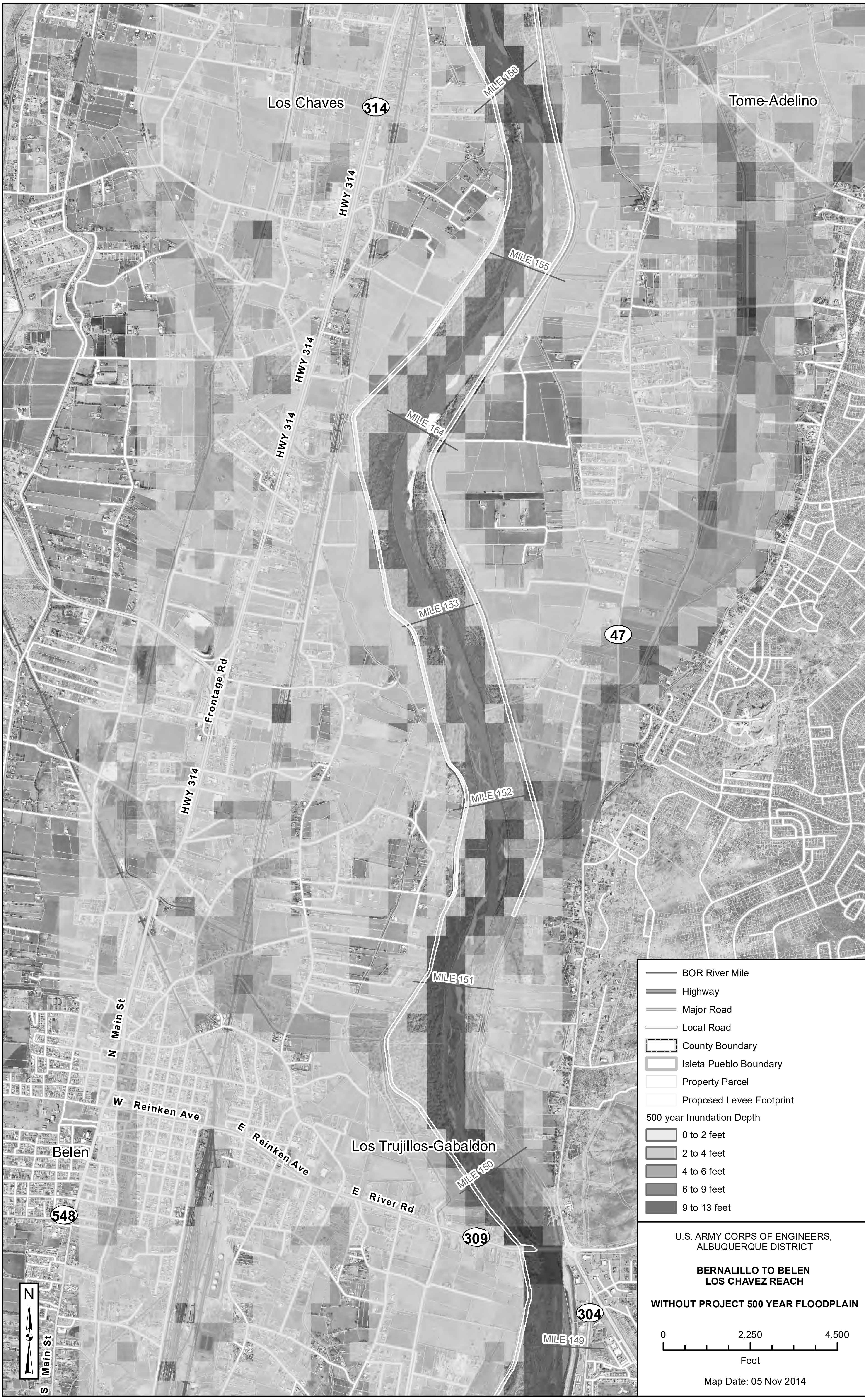

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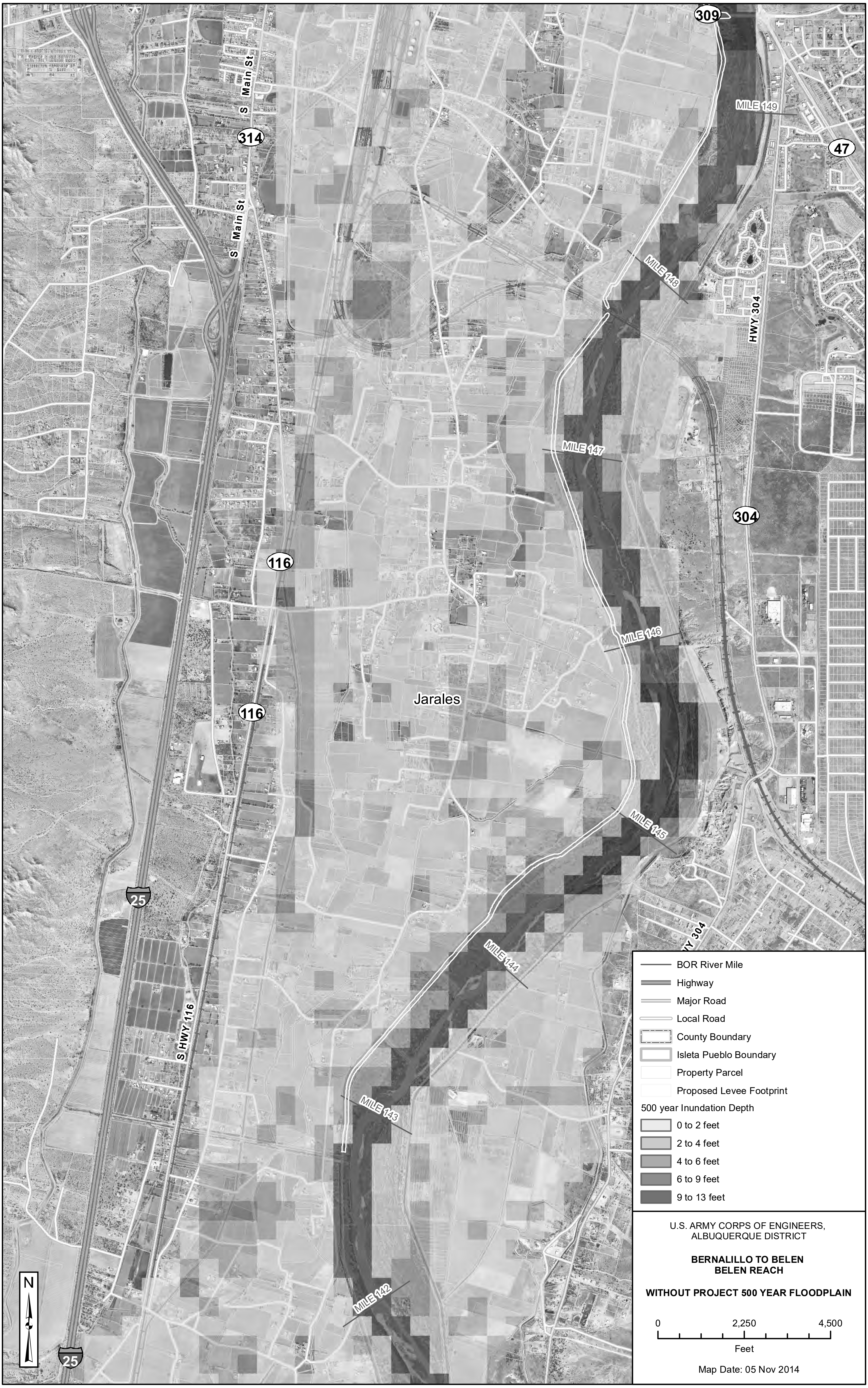

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500 year Inundation Depth
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U.S. ARMY CORPS OF ENGINEERS, ALBUQUERQUE DISTRICT
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BELEN REACH
WITHOUT PROJECT 500 YEAR FLOODPLAIN
0
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Feet
Map Date: 05 Nov 2014

Attachment 6—Geomorphology-Sediment Analysis

Middle Rio Grande Flood Protection H&H Supplemental Geomorphic Assessment

Bernalillo to Belen, New Mexico: Mountain View, Isleta, and Belen Units

AUGUST 2018 **[DRAFT]**

US Army Corps of Engineers

US ARMY CORPS OF ENGINEERS
ALBUQUERQUE DISTRICT
HYDROLOGY AND HYDRAULICS SECTION

REPORT PREPARED BY

Jonathan AuBuchon, PE
Hydraulic Engineer/Sediment Specialist

REPORT REVIWED BY

Jame Eisenberg, PE
Hydraulic Engineer

Cover Picture: Middle Rio Grande looking upstream from Central Bridge in Albuquerque, NM (picture taken 5 June 2018)

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Introduction

The U.S. Army Corps of Engineers (USACE) is looking to pursue flood protection on the Middle Rio Grande (MRG) between Bernalillo to Belen, NM (B2B). The effort was initiated as part of the Water Resources Development Act (WRDA) of 1986 to provide flood damage reduction for the MRG. The currently proposed work includes about 48 miles of engineered levee installation between the South Diversion Channel and Casa Colorado, NM. The proposed engineered levee will have a levee footprint that is about 60 feet wider than the current spoil levee footprint, with about half of this width (~ 30 feet) encroaching into the floodway (active channel and floodplain within the constructed spoil levees).

A robust design of the levee needs to consider the dynamic and complex interchange of processes that occur on a fluvial system. This understanding helps ensure that the engineered levee system will be able to accommodate reasonably predicted channel adjustments, especially within an alluvial system such as the MRG. There are risks, however, with introducing a static component into a dynamic system. Understanding the historical and current channel conditions and dynamics of a river and the anthropogenic influences (including this levee) on the system are important in developing a robust design and minimizing risk. Evaluating the potential for loss of channel/floodway conveyance, increased frequency of channel overbanking (putting more water against the riverside toe of the engineered levee), and potential channel migration towards the engineered levee are all risks that can be mitigated to some extent when working with fluvial systems by understanding the historical and current channel conditions and dynamics.

Fluvial systems develop primarily with the job of moving excess water back to the ocean (Phillips 2009). On the way, however, there is a complex interaction between the flowing water, the underlying geology, and the biota (plants and animals) that creates various channel forms (Wheaton et al. 2011). Various processes (such as geological uplift, climate change, mass wasting, fluvial erosion, energy dissipation of flood pulse, etc.) interact over different temporal and spatial scales to produce channel forms. The similar nature of these forms has given rise to a variety of classification efforts (Schumm 1981; Schumm 1985; Rosgen and Silvey 1996; Montgomery and Buffington 1997; Brierley and Fryirs 2005) that provide insight into the current channel form and the underlying processes (Schumm 1969; Schumm 1977; Leopold et al. 1992; Charlton 2008). On the temporal and spatial scale of the MRG relative to the B2B project, the primary drivers of change to channel form are water and sediment (Schumm 1977; Makar and AuBuchon 2012).

The MRG is a dynamic and complex alluvial system where flow and sediment transported from the Upper Rio Grande and MRG tributaries influence the observed form of the river. This flux of water and sediment (magnitude, duration, and frequency) is tempered by bank and bed stability, base level changes, floodplain lateral confinement, and floodplain connectivity, which in turn influences how much, when, and where water and sediment are transported or stored within and through the fluvial system (Leopold et al. 1992, MEI 2002, Charlton 2008, Davies and Korup 2010, Makar and AuBuchon 2012).

Knowledge of historical anthropogenic changes (such as upstream reservoirs, river straightening, irrigation practices, floodplain constraints, upland land practices, etc.), coupled with climatic influences such as large floods, helps to understand the historical shape (or morphology) of the river. Assessing changes in the morphology (width, location/planform, slope, sinuosity, bed material size and type, and channel and floodway topography) in light of relationships that have been developed through observations of fluvial systems (Lane 1954; Schumm 1969; Schumm 1977; Massong et al. 2010) provides the ability to estimate future changes and allow for the incorporation of known risks into the design of the B2B engineered levee.

MRG Conditions and Dynamics

The MRG is primarily a snow-melt influenced fluvial system (Bauer 2000; Klein et al. 2018a). The Rio Grande drains the eastern edge of the southern Rockies, collecting the snow that falls in the mountainous regions of Colorado and New Mexico and conveying it downstream to the Gulf of Mexico. The snow-pack driven peaks of the spring melt consist of large peak and long duration floods (Bauer 2000; Klein et al. 2018a) with instantaneous and daily average discharges that are close to unity (HEC 2006). The southwestern United States can also develop strong monsoonal patterns in the summer and fall that bring additional rainfall-runoff into the MRG (Bauer 2000; Mosley 2000; Smith and Finch 2016; Klein et al. 2018a). These events are often associated with higher peak flows, but smaller durations (Mosley 2000; Klein et al. 2018a). The rainfall-runoff events tend to be more localized, with potential contributions of flow throughout the MRG drainage basin. The flashy nature of these flow events results in instantaneous discharge peaks on the MRG around Albuquerque, NM that are around four times higher than the daily average discharge (HEC 2006).

Anthropogenic influences on the MRG have occurred for centuries and vary from small, localized irrigation facilities to larger irrigation and flood control measures placed on the main stem of the Rio Grande and primary tributaries (Graf 1994; Scurlock 1998). While most of the measures, such as flood control and water supply reservoirs or diversions have been to store or extract water from the MRG, there has also been a project (San-Juan Chama Project) to add water from the Colorado Basin (San Juan River) into the MRG basin (Graf 1994; Scurlock 1998; MEI 2002; Makar and AuBuchon 2012). These influences, coupled with climatic changes have influenced the magnitude, duration, and frequency of flow on the MRG (MEI 2002; Makar and AuBuchon 2012; Klein et al. 2018a). Typically these have resulted in lower peaks, shorter duration and less frequent large flows and higher peak, longer duration, and more frequent low flows (MEI 2002; Makar and AuBuchon 2012; Klein et al. 2018a).

While influenced by the flow that can transport it, the sediment supply into the MRG has also varied. Anecdotal accounts of deforestation, timed with climatic influences at the turn of the 20th century brought sediment loads, estimated at about 40 million tons of sediment each year, into the MRG (Scurlock 1998). This resulted in channel aggradation close to seven feet near the Isleta Diversion Dam (Happ 1948). The influx of sediment resulted in a river planform that was wide and braided through most of the valley (Scurlock 1998; MEI 2002; Makar and AuBuchon 2012).

By the middle of the 20th century additional anthropogenic controls (including flood control dams on the main stem of the MRG and on tributaries upstream of Albuquerque, NM, channelization and river training techniques, and irrigation and drainage infrastructure) meant to control the widespread flooding resulted in a significant narrowing of the MRG's floodplain. The aggradation that occurred was confined, causing problems with waterlogged soils in agricultural areas (Graf 1994; Scurlock 1998). It also created a perched channel condition above the historic floodplain that had been cut off through the construction of spoil levees (MEI 2002; Makar and AuBuchon 2012). While sediment influxes into the MRG were still large, estimated at around 32 million tons per year (Finch and Tainter 1995), the magnitude of sediment had begun to decrease. The rate of aggradation was noted to have decreased from a rate of 0.15 feet per year in the 1920s (Scurlock 1998) to around 0.04 feet per year in Albuquerque by the early 1960s (LaGasse 1980).

By the 1970s, degradation of the active channel was observed north of Albuquerque (Lagasse 1980).The sediment supply on the Rio Grande was noted to decrease (Makar and AuBuchon 2012) with a current suspended sediment yield ranging between two to five million tons per year. The sequences of changes in the sediment and flow regimes resulted in significant changes in the channel morphology that were varied in the downstream direction, but which are generally summarized within the project area as follows:

- pre 1800s—Tributaries to the Rio Grande were noted as being characterized by grassy bottomlands and perennial streams (Bryan and Post 1927). The Rio Grande was noted to be wide and shallow (Scurlock 1998).
- 1800s to 1920s—Arroyo down cutting is assumed to have begun in this period, creating deep, entrenched channels (Bryan and Post 1927; Elliott 1979). Surveys from the 1910s indicate a wide, braided channel with a relatively high water table that promoted wet riparian areas (Crawford et al. 1993; MEI 2002; Klein et al. 2018a).
- 1930s to 1940s—Rio Grande is wide and braided, although the active channel is only 2/3 of the width from previous decades (Makar and AuBuchon 2012). Flooding due to aggradation is very problematic for urban and agricultural areas along the MRG (Graf 1994; Scurlock 1998). Rainfall is less than in previous decades, but large floods still occur (Graf 1994).
- 1950s to early 1970s—Active channel continues to narrow (Swanson et al. 2011), with some of the larger medial bars becoming attached to the channel banklines (Massong et al. 2010; Makar and AuBuchon 2012; Klein et al., 2018a). Mobile medial bars are still visible within the active channel and stands of woody vegetation are not common (Klein et al. 2018a). Larger trees are noted to form in the floodplains as the peak flood pulse are cut off and older floodplain surfaces becoming disconnected from the channel (Graf 1994, Scurlock 1998).

- Mid-1970s to 1990s— Precipitation increases and a transbasin diversion comes on line (Graf 1994; MEI 2002). Active channel width is relatively constant, with minimal vegetation growth on bars. Vegetation on the floodplain becomes more established, causing vertical accretion immediately adjacent to the active channel. While flooding of the floodplain is not as common, when flooding occurs, low spots are evident against the riverside toes of the constructed spoil levees (Makar and AuBuchon 2012; Klein et al. 2018a).
- 2000s to present—Drier precipitation patterns become predominant with some wetter years interspersed (Makar and AuBuchon 2012). Medial bars become less mobile and vegetation starts to "lock" them in place, causing vertical accretion (Meyer and Hepler 2007). The active channel is predominantly a single channel planform. Reach sinuosity has increased slightly, while the reach-averaged slope has decreased (Makar and AuBuchon 2012; Klein et al. 2018a). Channel adjustments have varied by reach, with both channel incision and deposition having been observed (MEI 2008; Klein et al. 2018a). Floodplain deposition has been noted more consistently in the active channel through the study reach (USACE 2018; Klein et al. 2018a). Median bed material sizes have also increased, with the noted occurrence of gravel on bars, especially north of Albuquerque, NM (Makar and AuBuchon 2012; Klein et al. 2018a).

The combination of climatic and anthropogenic changes throughout the Rio Grande watershed has caused changes in both the flow and sediment regime that have manifested themselves in concurrent morphological adjustments as described above.

MRG Morphological Adjustments

Alluvial systems have complex responses that may lag behind obvious system changes (Biedenharn et al. 2008; Charlton 2008; Owen et al. 2012), making it difficult to interpret underlying processes that are causing the observed geomorphic changes. Adding to this complexity is the singularity of channel response (Schumm 1983), in that alluvial channel responses are not always uniform, being dampened by controls on the response (such as an armored bed, a limited floodplain connection, or additional stability added by vegetation) that are heterogeneous in their spatial distribution. The MRG also seems to adjust episodically (MEI 2002) which provides further difficulties in understanding the nature of these adjustments (MEI 2002) and expectations of future adjustments.

Lane (1954) and Schumm (1969; 1977) proposed relationships for alluvial channel responses based on observations of different fluvial systems. These relationships help interpret the observed geomorphic responses and provide a means of forecasting future channel responses. These relationships are predicated on the assumption that alluvial systems will tend to adjust towards an equilibrium condition (Lane 1954; Schumm 1977). Because of the channel complexity, alluvial channels may never reach this equilibrium condition before other system changes occur, or they may reach a dynamic oscillation around an equilibrium value (Leopold et al. 1992; Charlton 2008). The qualitative relationships developed by Lane (1954) and Schumm (1977) are provided in Equation 1 through Equation 3. These relationships illustrate that changes in the sediment and water regimes would be expected to have observable geomorphic channel responses, as illustrated in Table 1.

Equation 1. Lane's relationship (Lane 1954)

$$Q_s d_{50} \propto QS$$

Where Q_s = sediment load,
d_{50} = median sediment size,
Q = water discharge, and
S = channel gradient.

Equation 2.Schumm's relationship for water discharge (Schumm 1969; 1977)

$$Q \propto \frac{b, \lambda, d}{S}$$

Where Q = water discharge,
b = channel width,
λ = meander wavelength,
d = channel depth, and
S = channel gradient.

Equation 3.Schumm's relationship for sediment discharge (Schumm 1969; 1977)

$$Q_s \propto \frac{b, \lambda, S}{d, P}$$

Where Q_s = sediment load,
b = channel width,
λ = meander wavelength,
S = channel gradient,
d = channel depth, and
P = channel sinuosity.

Table 1. Expected geomorphic responses based on an increase or decrease in the water or sediment regime (Lane 1954; Schumm 1969). Plus (+) indicates an increase in the geomorphic parameter, while a minus (–) implies decrease. N.C. means there is the assumption that no change occurs. An indeterminate answer has both a plus and minus indication.

Geomorphic Parameters	Q	Q_s	S	d_{50}	b	λ	d	P
Reduction in Q_s	N.C.	-	-	+	-	-	+	+
Increase in Q_s	N.C.	+	+	-	+	+	-	-
Reduction in Q	-	N.C.	+	-	-	-	-	+/-
Increase in Q	+	N.C.	-	+	+	+	+	+/-

But the geomorphic parameter responses listed in Table 1 may give conflicting responses. For example there is a strong narrowing trend on the MRG (MEI 2002; Makar and AuBuchon 2012; Klein et al. 2018a), which would indicate that the sediment load and/or the flow discharge have decreased from previous decades. This is supported in the data discussed previously that shows a decrease in the sediment load and the peak flows, but there are also a number of indeterminate variables with this combination suggesting a complex system response or local variations in the parameters. For instance, while the peak discharges have decreased, there has also been an increase in the base flows, which would indicate that the width reduction shouldn't be as strong, which was the case for the MRG from the mid-1970s through the 1990s in the B2B study reach. A combination of different changes to the water and sediment regimes and the expected geomorphic responses are shown in Table 2.

Table 2. Expected geomorphic responses based on a combination of water or sediment regime changes (Lane 1954; Schumm 1969). Plus (+) indicates an increase in the geomorphic parameter, while a minus (–) implies decrease. An indeterminate answer has both a plus and minus indication.

Geomorphic Parameters	**Q**	**Q_s**	**S**	**d_{50}**	**b**	**λ**	**d**	**P**
Reduction in Q_s, Reduction in Q	-	-	+/-	+/-	-	-	+/-	+
Reduction in Q_s, Increase in Q	+	-	-	+	+/-	+/-	+	+
Increase in Q_s, Reduction in Q	-	+	+	-	+/-	+/-	-	-
Increase in Q_s, Increase in Q	+	+	+/-	+/-	+	+	+/-	-

The B2B study reach was divided into four subreaches based on common morphological characteristics to further evaluate the observed morphological adjustments with respect to the expected geomorphic responses based on alluvial relationships developed by Lane (1954) and Schumm (1969). The reaches are shown in Figure 1 and described in the paragraph below.

Subreach 1 is upstream of the project area, extending downstream from aggradation-degradation (agg-deg) line 514 to agg-deg line 580, which is around the confluence of the Rio Grande with the AMAFCA South Diversion Channel.

Subreach 2 extends from subreach 1 to the Isleta Diversion Dam (roughly between agg-deg lines 581 and 656).

Subreach 3 was divided into two sections because of the presence of gravels found in the upstream section (Occam and Tetra Tech 2017). Subreach 3a extends from agg-deg line 657 to 724, while subreach 3b is approximately between agg-deg 725 and agg-deg 801. Downstream of agg-deg line 787 the percent of gravel found in the bed is less than 2% by weight, while upstream, including subreaches 2 and 3, the gravel content was generally greater than 2% (Occam and Tetra Tech 2017).

Subreach 4 begins downstream of subreach 3 and extends between agg-deg lines 802 and 1000.

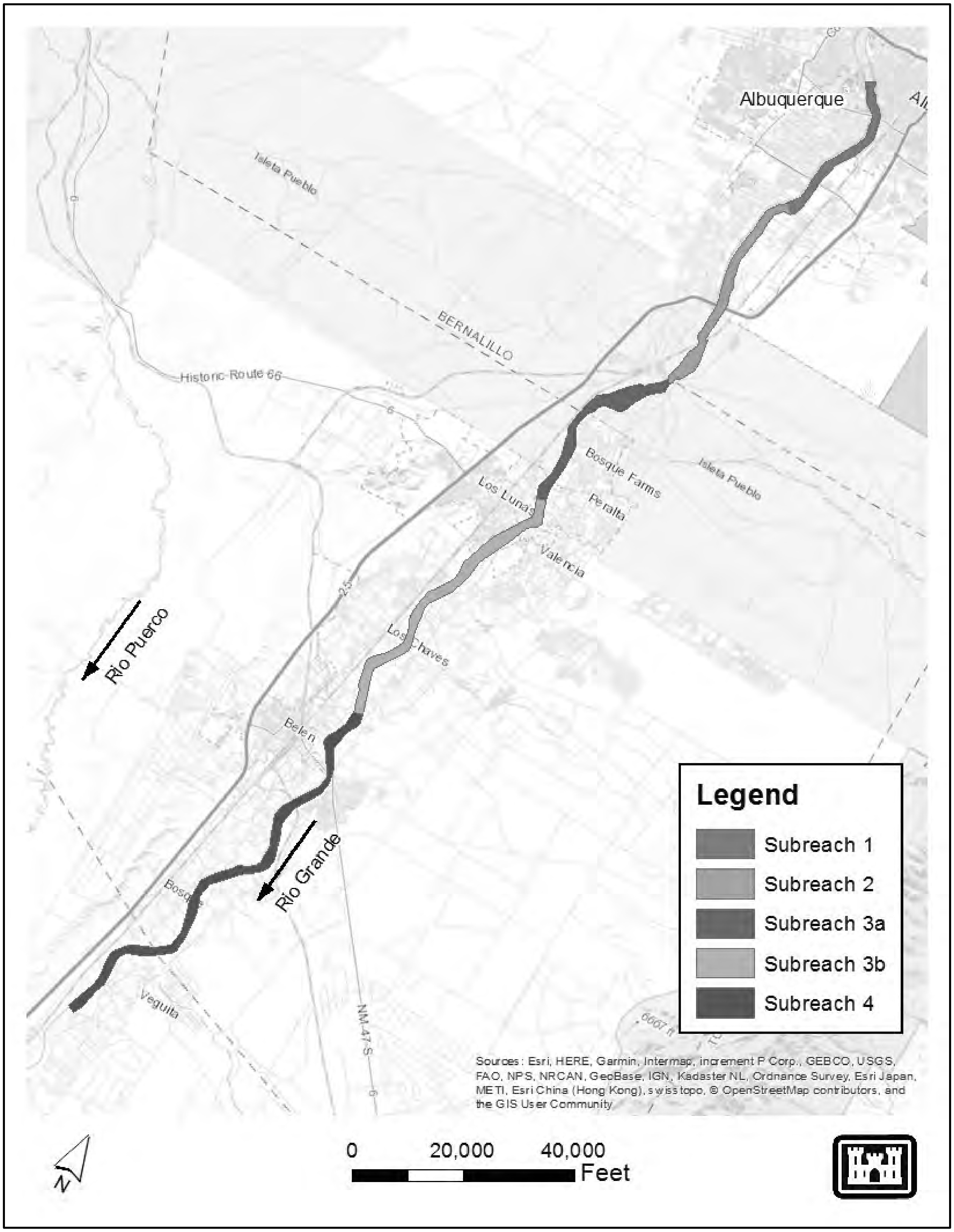

Figure 1. Subreach designations for the B2B project. Base topography map is from ESRI (accessed online 8 August 2018).

Current geomorphic parameters on the MRG for the B2B reach are described briefly in the following sections.

Water Discharge

Since the construction of upstream reservoirs that have controlled peak flood pulses there has been a decrease in the annual peak flows (MEI 2002; Makar and AuBuchon 2012). Klein et al. (2018) also indicated a climatic shift in the annual flow volumes with a wetter period occurring in the late 1970s/early 1980s to mid-1990s, followed by a return to previous flow volume conditions. A flow frequency analysis of the USGS gages upstream and downstream of the B2B project area indicate an increase in the frequency of flows at the lower discharges and a reduction in the frequency of flows at the higher discharges (MEI 2002; Bui 204; Klein et al. 2018).

Sediment Load

The Rio Grande is located in the Rio Grande Rift Zone in New Mexico, with an interconnected series of basins separated by geological constrictions (Graf 1994). Upstream of the B2B project site the Espanola basin drains erodible sediments into the MRG upstream of Cochiti Dam. Downstream of Cochiti Dam, the Santo Domingo Basin has numerous tributaries that bring in erodible sediment to the MRG. The Santo Domingo basin ends around Angostura Diversion Dam and another valley widening begins with the Albuquerque Basin. The Albuquerque Basin has several large tributaries that can bring in sediment into the MRG, including the Jemez River, the Calabacillas Arroyo, and both the North and South AMAFCA Diversion Channels. The Albuquerque Basin ends around the Isleta Diversion Dam. On the downstream side of the Isleta geological constriction, the Belen Basin begins. The Belen basin pinches out around the San Acacia Diversion Dam (Bauer 2000; MEI 2002). The geology within these basins and through which the MRG tributaries flow are erodible, consisting of rocks within or similar to the Santa Fe Group (Graf 1994).

Graf (1994) listed tributaries draining the Espanola Basin and the Jemez River as major sources of sediment to the MRG upstream of the B2B project area. Episodic events on other tributaries, such as Peralta Creek and Calabacillas Arroyo) have also been known to bring in large quantities of sediment to the MRG (Swanson et al. 2010; AuBuchon and Bui 2013). These events have been noted to constrict the Rio Grande (Swanson et al. 2010) and block the river entirely (AuBuchon and Bui 2013). AuBuchon and Bui (2013) also noted that that the September 13, 2013 rainfall-runoff event on Peralta Arroyo deposited two to three feet of sediment in the MRG channel up to about 3000 feet downstream. While there is potential for episodic sediment pulses from the larger tributaries to the MRG, there influence is relatively local at the time of the event, although subsequent flows, especially the spring snow-melt runoff tend to re-mobilize finer material and transport it downstream into the B2B reach. Within the B2B reach, however, there are no major tributaries, apart from the AMAFCA South Diversion Channel, whose confluence with the MRG marks the upper boundary of the B2B project area, until Abo Arroyo, which is south of the B2B project. This is a result of the Joyita Uplift that causes a higher elevation complex of exposed rock on the east side of the Rio Grande that keeps the majority of tributaries from entering the Rio Grande until around Bernardo, NM (MEI 2002).

Historically tributary flows have caused significant flooding and sedimentation downstream on the MRG (Harden 2006). The largest documented floods in the 1920s and 1940s, however, were prior to the construction of upstream reservoirs that currently provide flow regulation during peak events (Makar and AuBuchon 2012). Analysis of suspended sediment measurements from the USGS gages on the MRG from the 1950s through 2005 show a reduction in the sediment load in the 1970s (Makar and AuBuchon 2012). A general channel degradational trend is observed after 1972 due to a variety of reasons, including the closure of a large upstream flood control reservoir, importation of transbasin flows, and continued channel narrowing (Bauer 2000; MEI 2002; Parmetrix 2008; Varyu 2013). There is also a slight increase in the suspended sediment load in the 1990s, but this is more pronounced at the USGS gage at Otowi, NM (USGS # 08313000), upstream of Cochiti Flood Control Reservoir, than the downstream gages. Klein et al. (2018) extended an analysis of the collected suspended sediment data at the Albuquerque (USGS # 08330000), Bernardo (USGS # 08332010), and San Acacia (USGS # 08354900) USGS gages to the early 2010s. This analysis indicates that while the suspended load has fluctuated since the mid-1970s, the increase of suspended sediment in the 1990s has continued, with a sharper rise in the suspended sediment concentration downstream of the B2B project site than upstream. A table of the results extracted from Makar and AuBuchon (2012) and Klein et al. (2018) is shown in Table 3. The Otowi and Albuquerque USGS gages are upstream of the B2B project area, while the Bernardo and San Acacia gages are downstream.

Table 3. Average annual suspended sediment concentration of the Rio Grande at the Otowi, Albuquerque, Bernardo, and San Acacia USGS gages (extracted from tables in Makar and AuBuchon 2012; Klein et al. 2018).

Time period	**Average annual concentration (mg/L) at USGS gaging station on the MRG**			
	Otowi	Albuquerque	Bernardo	San Acacia
*1955-1975**	2,033	3,377		10,022
*1976-1990**	817	499		3,010
*1991-2005**	1,999	831		2,675
2004-2014†		937	825	3,483

Notes:
*—from Makar and AuBuchon (2012)
†—from Klein et al. (2018)

Klein et al. (2018) also compared the suspended sediment flux between the Albuquerque and San Acacia USGS gages and the Albuquerque and Bernardo USGS gages. This assessment indicated the loss of suspended sediment between Albuquerque and the Bernardo USGS stations and an increase in the suspended sediment from Albuquerque to San Acacia. There are large tributaries downstream of Bernardo, however that may account for the increase in suspended sediment at the San Acacia USGS gage. The difference between the Albuquerque and Bernardo gage oscillates with time, indicating that the suspended sediment load oscillates between periods of gaining and losing sediment. The B2B study area is within this reach of the MRG, suggesting that there may be cyclical aggradational and degradational tendencies within the reach. Others (Vensel et al. 2006; Varyu 2013; Huang 2016) have also found a slight aggradational trend from Isleta to San Acacia between 2002 and 2012 that suggests an increase in the sediment load.

While the MRG has seen a historical lowering of the sediment load since the 1960s/1970s there are still significant tributaries upstream of the project site that have the potential to bring in large quantities of sediment. The stream gradients for the tributaries to the MRG tend to be an order of magnitude greater than the MRG gradient (Simons et al. 1981; MEI 2002; Makar and AuBuchon 2012), resulting in a reduction of the transport energy at the confluences. Since the tributary events are also typically caused by late summer/early fall rainfall-runoff they result in sediment deliveries that are not timed with sustained main stem flows on the MRG, which tend to occur during the spring snow melt runoff. The result is a decrease in the energy gradient, causing sediment deposition near the tributary confluence with the MRG. This creates local geomorphic changes. Julien and O'Brien (1997) have noted that a lack of high velocity gradients is a limiting condition for debris flow mobility. The lower energy gradient of the MRG, coupled with upstream flow regulation and the lack of major tributaries within the B2B reach, would therefore imply that the MRG through the B2B reach, while prone to longer term sedimentation issues, is not as susceptible to large, episodic sediment failures, such as debris flows.

Channel Gradient

The channel gradient within the B2B study reach has a general trend showing a slight reduction in the channel bed slope from 1936 to 2007 (Makar 2010). Klein et al. (2018a) indicated slight variability in the reach average energy grade slope between Isleta Diversion Dam and the Rio Puerco Confluence, with an overall trend showing a slope reduction for a discharge of 5000 cfs. The energy grade slope calculated by Klein et al. (2018a) was similar to the values found by MEI (2008).

Channel Bed Material

In general the channel material within the B2B study reach consists of sand (Makar 2010). Klein et al. (2018a) found that collected bed samples in the 2000s tended to have a larger range than in the 1990s and noted the presence of gravels. Samples collected in the 2010s tended to be coarser than the 1990 samples, but only a limited gravel presence. Sampling in 2016/2017 (Occam and Tetra Tech 2017) found that the majority of the bed material in this reach is sand. Gravels were found in all samples, except in the upstream portion of subreach 4. The percentage by weight of gravels in subreaches 1, 2, and 3a generally ranged from two to five percent. The percentage of gravels in subreach 3b and 4 was generally less than two percent.

Channel Width

As described previously, widths on the MRG have decreased since the turn of the twentieth century. A summary of the wetted channel widths since 1962 for the B2B study reaches is provided in Table 3. A cumulative width graph of active channel widths is provided in Figure 2. This information indicates that subreach 1 has been more constant than the other reaches, showing only a slight average active channel width decrease between 1962 and 2012 and a slight increase in the average active channel width from 1962 to 1972. The other subreaches have shown a decrease in the average active channel width between 1962 and 2012, with only subreach 3a also showing an average active channel width increase between 1962 and 1972. At the same time the range in the active channel width values for the reach has narrowed. The largest shift in the active channel width between 1962 and 2012 occurred between 1992 and 2002, which was during a drought period on the MRG (Makar and AuBuchon 2012). Klein et al. (2018a) also found a significant decrease between 2012 and 2016 in the average active channel width between Isleta Diversion Dam and the Rio Puerco from around 380 feet to less than 200 feet. Klein et al. (2018a) also noted that the percentage of woody vegetation on the MRG increased close to 20% between 1992 and 2016 between the Isleta and San Acacia Diversion Dams.

Table 4. Active channel widths for the B2B study reach. Width values are rounded to nearest ten. Modified from tabular results compiled by unpub. work by A. Posner, U.S. Department of the Interior, Bureau of Reclamation.

Subreach Number	Width Description	Widths (feet) by year				
		1962	1972	1992	2002	2012
1	Average	470	560	500	480	450
	Maximum	950	1,010	650	690	640
	Minimum	160	150	200	200	220
2	Average	590	520	430	440	430
	Maximum	890	870	620	620	610
	Minimum	350	170	220	150	180
3a	Average	430	490	500	420	360
	Maximum	790	680	700	540	560
	Minimum	140	150	190	210	220
3b	Average	660	560	560	410	430
	Maximum	1,280	730	780	550	570
	Minimum	170	253	360	300	280
4	Average	450	450	470	350	300
	Maximum	1,080	670	680	540	600
	Minimum	170	160	190	120	150

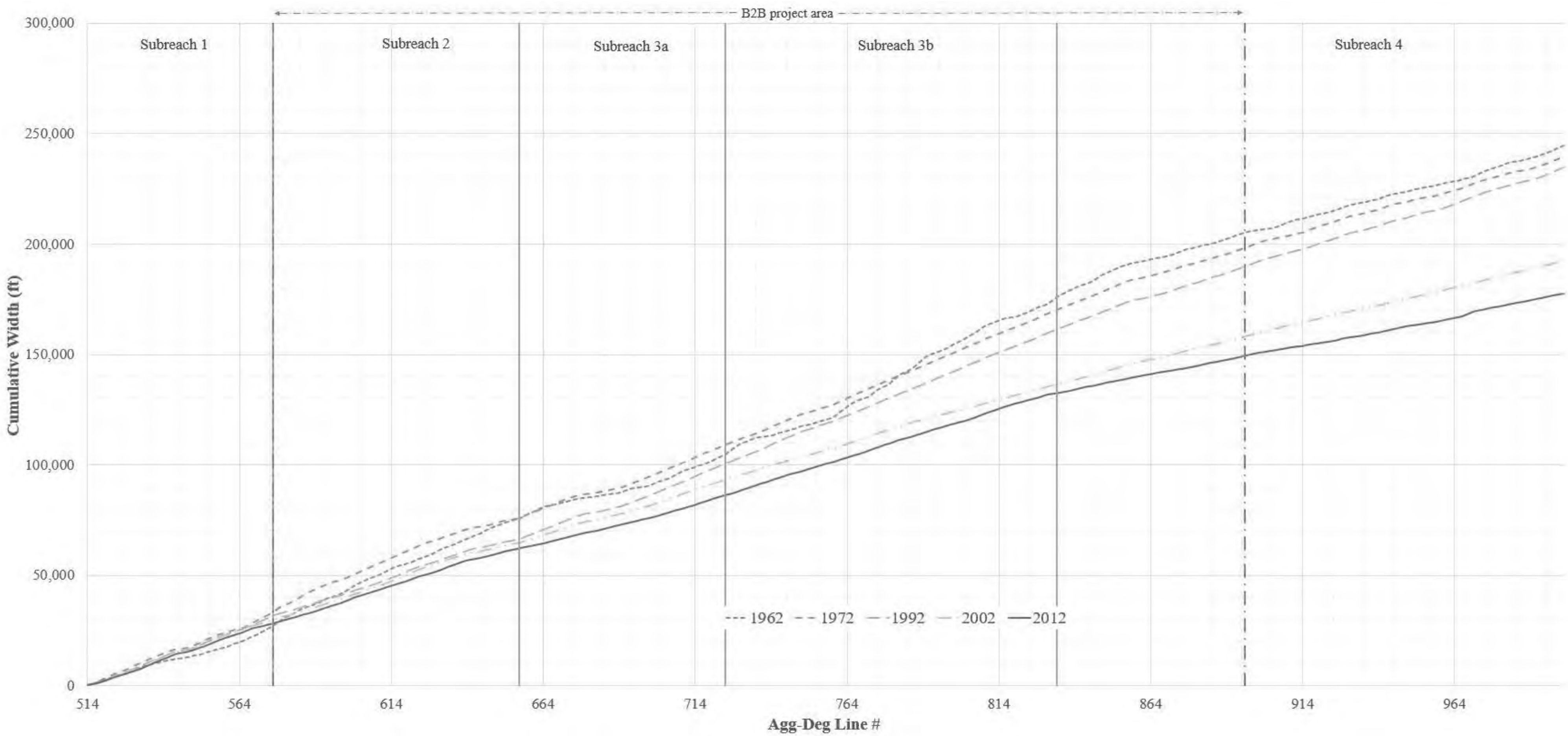

Figure 2. Cumulative active channel widths on the MRG through the B2B study reach. Figure modified from unpub. work by A. Posner, U.S. Department of the Interior, Bureau of Reclamation.