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CALABACILLAS WEST BRANCH ARROYO DRAINAGE AND STORM WATER QUALITY MANAGEMENT PLAN - FINAL

AMAFCA

PHASE 2, TASK A
DEVELOPED CONDITIONS HYDROLOGY REPORT: AMAFCA 2014 “WHITE PAPER” METHODOLOGY

Submitted by:

TETRA TECH

and

Bohannan Huston

6121 Indian School Road NE
Suite 205
Albuquerque, NM 87110
(505) 881-3188

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EXECUTIVE SUMMARY

The Calabacillas West Branch Arroyo (CWB) watershed is located in the northwest area of Albuquerque, and straddles the city limits of Albuquerque and Rio Rancho. The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) contracted Tetra Tech to provide engineering services to develop the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan to identify arroyo and storm water quality improvements needed for the CWB as the watershed develops.

The CWB has one point of discharge to the AMAFCA Swinburne Dam. The CWB drainage basin includes diversions from the Piedras Marcadas Arroyo watershed via the Las Ventanas Detention Dam and Outfall Pipe.

Tetra Tech completed the Field Reconnaissance of the CWB on March 5, 2013. A longitudinal profile of the CWB was also created to facilitate assessment of the existing channel. These data were analyzed and compared to the data presented by Mussetter Engineering, Inc. (MEI) in the 1999 Calabacillas Arroyo Prudent Line Study and Related Work – Development of a Prudent Line for the West Branch ("1999 Prudent Line Study"). The Field Reconnaissance Report identifies locations where existing prudent lines are being encroached upon by lateral migration, and where vertical degradation is problematic. The 1999 Prudent Line Study is the basis for recommended sediment bulking factors for various reaches of the arroyo for the 2015 existing conditions hydrology modeling

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Developed conditions hydrologic modeling used the HEC-HMS methodology presented in the draft White Paper – Migrating from AHYMO '97 to HEC-HMS (and USEPA SWMM), Easterling Consultants LLC, September 29, 2014 ("AMAFCA White Paper"). This methodology uses a synthetic frequency storm distribution and uses the SCS Curve Number for the Loss Method.

Fifty-two sub basins were modeled, with a total area of 11.88 sq. mi. This includes eight basins, totaling 1.92 sq. mi., that drain through the existing AMAFCA Las Ventanas or Little Window Detention Dams. 2-year (yr), 10-yr, and 100-yr, (50%, 10% and 1% probability events, respectively) 6-hour and 24-hour storm events were modeled for this study. In addition, the 5-yr, 25-yr, and 50-yr, 24-hour storm events were modeled for this study. The precipitation depths, for the analyzed events, were extracted from the NOAA Atlas 14. For the 6-hour and 24-hour storms respectively, the rainfall depth for the 2-year event is 0.968 inches and 1.23 inches, the 10-year event is 1.47 inches and 1.79 inches, and the 100-yr event is 2.27 inches and 2.66 inches. In this analysis, no depth-area reduction factor was used as the analysis results will ultimately be used for planning and development of recommended infrastructure.

At Swinburne Dam, the predicted developed conditions 100-year, 6-hour peak flow from the 1999 Prudent Line Study was 4,950 cfs (5,450 cfs bulked), and the volume was 680 ac-ft. The 2015 HEC-HMS developed conditions model #1 (DCM#1), which does not have any new detention ponds modeled, predicts a peak 100-year, 6-hour flowrate of 6,653 cfs (7,645 cfs bulked) and a volume of 765 ac-ft and a peak 100-year, 24-hour flowrate of 6,951 cfs (7,987 cfs bulked) and a volume of 941 ac-ft at the same location. Table A below compares the 1999 AHYMO developed conditions results to the DCM #1 results. The analysis point locations in this table are shown on Figure 2.

Table A – Developed Conditions 100-year Peak Flow and Volume Comparisons
1999 AHYMO Model Compared to 2016 HEC-HMS Results

1999 Developed Condtions Analysis Points AHYMO (6-hr. storms)				2016 Developed Condtions Model #1 HEC-HMS AMAFCA 2014 "White Paper" Methodology (6-hr. storms)				2016 Developed Condtions Model #1 HEC-HMS AMAFCA 2014 "White Paper" Methodology (24-hr. storms)			
BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)	BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)	BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)
	Unbulked	Bulked			Unbulked	Bulked			Unbulked	Bulked	
Concentration Point 4				AP2 (QR3_20)				AP2 (QR3_20)			
10.2%	2,560	2,820	184	6.8%	2,531	2,703	187	6.8%	2,651	2,832	234
Concentration Point 2				AP4 (QR3_25_PW4)				AP4 (QR3_25_PW4)			
10.0%	2,990	3,290	258	22.4%	4,191	5,130	327	22.4%	4,381	5,363	406
Concentration Point 1				AP5 (PW1_12)				AP5 (PW1_12)			
10.1%	4,460	4,910	465	12.1%	5,824	6,528	521	12.1%	6,085	6,821	643
Concentration Point L				AP9 (VR_TVI_PW_SEV)				AP9 (VR_TVI_PW_SEV)			
10.0%	4,810	5,290	551	13.1%	6,556	7,415	631	13.1%	6,853	7,750	776
Concentration Point 0				AP10 (SWINBURNE_INFLOW)				AP10 (SWINBURNE_INFLOW)			
10.1%	4,950	5,450	680	14.9%	6,653	7,645	765	14.9%	6,951	7,987	941

The increase in peak runoff and volume as compared to the AHYMO model results in the 1999 Prudent Line Study can be contributed to several factors, including:

- The use of a different hydrologic modeling software, AHYMO was used in 1999 and HEC-HMS was utilized for the current study;
- The use of different rainfall distributions by the two models;
- The current study includes an additional 973 acres (1.52 sq. mi.) added based on the master plan layouts for Quail Ranch and Paradise West;
- The use of smaller, more detailed basins in the current model (52 basins) compared to 7 basins in the 1999 model. Modeling additional basins in hydrologic models typically results in higher flows. The additional basins were required to provide better planning and options analysis in the watershed; and
- The land treatment assumption of higher concentration in developed conditions than assumed in the 1999 model. The 1999 Prudent Line Study assumed single family residential with an average of 4 dwelling units per acre (DU/ac), while the current development (Las Ventanas and Seville), as well as the master plans for Quail Ranch and Paradise West, have areas with 5 to 6 DU/ac.

In addition to DCM #1, three other developed condition hydrologic models were created for this analysis. The four hydrologic models evaluated for developed conditions include:

- DCM #1, a developed conditions hydrology model with existing flood control facilities and the elimination of the Paseo del Volcan Diversion to the Calabacillas Middle Branch.
- DCM #2, a developed conditions hydrology model with existing flood control facilities and the Quail Ranch Pond assumed to be in place. The Quail Ranch Pond was sized to discharge the existing conditions 100-yr, 24-hr peak flowrate as determined in the *Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan – Final, Phase 1, Task C, Existing Conditions Hydrology Report: AMAFCA 2014 "White Paper" Methodology* ("2016 CWB Existing Conditions Hydrology Report"), Tetra Tech/BHI, March 2016.
- DCM #3, a developed conditions hydrology model with existing flood control facilities, the Quail Ranch Pond, and the Paradise West Pond assumed to be in place. The Quail Ranch Pond was sized in DCM#2. The Paradise West Pond was sized to reduce the 100-yr, 24-hr peak flowrate to minimize improvements to the existing grade control structures downstream of Universe Blvd.

- DCM#4, a developed conditions hydrology model with individual ponds placed in each basin within the watershed. These individual ponds were sized to discharge the developed conditions 100-yr, 24-hr peak flowrate to match the existing conditions 100-yr, 24-hr peak flowrates as determined in the CWB Existing Conditions Hydrology Report.

Table B below compares the 100-yr. event results for all four Developed Conditions Models.

Table B – Developed Conditions 100-year Peak Flow and Volume Comparisons

DCM #1				DCM #2				DCM #3				DCM #4			
BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft)	BULKING FACTOR*	Peak Discharge		Runoff Volume (ac-ft)	BULKING FACTOR*	Peak Discharge		Runoff Volume (ac-ft)	BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft)
	Unbulked	Bulked			Unbulke d	Bulked			Unbulked	Bulked			Unbulked	Bulked	
AP1 (QR3_15)				AP1 (QR3_15)				AP1 (QR3_15)				AP1 (QR3_15)			
3.6%	2,840	2,942	211	3.6%	2,840	2,942	211	3.6%	2,840	2,942	211	3.6%	638	661	209
AP2 (QR3_20)				AP2 (QR3_20)				AP2 (QR3_20)				AP2 (QR3_20)			
6.8%	2,651	2,832	234	6.7%	2,651	2,829	234	6.8%	2,651	2,832	234	3.0%	723	744	243
				AP3 (QR3_24)- Flow into QRP				AP3 (QR3_24)- Flow into QRP							
				4.6%	4,288	4,485	379	4.6%	4,288	4,485	379				
AP3 (QR3_24)- QRP				AP3 (QRP_OUT)				AP3 (QRP_OUT)				AP3 (QR3_24)- QRP			
13.2%	4,288	4,853	379	4.6%	837	875	378	4.6%	837	875	378	5.5%	1,179	1,244	386
AP4 (QR3_25_PW4)				AP4 (QR3_25_PW4)				AP4 (QR3_25_PW4)				AP4 (QR3_25_PW4)			
22.4%	4,381	5,363	406	8.2%	865	936	406	8.1%	865	935	406	9.5%	1,193	1,306	413
AP5 (PW1_12)				AP5 (PW1_12)				AP5 (PW1_12) - Flow into PWP				AP5 (PW1_12)			
12.1%	6,085	6,821	643	9.0%	4,046	4,410	645	5.2%	4,046	4,256	645	6.6%	2,059	2,195	648
								AP5 (PWP_OUT) - Flow out of PWP							
								5.2%	1,620	1,704	643				
AP6 (PW_VRW3)				AP6 (PW_VRW3)				AP6 (PW_VRW3)				AP6 (PW_VRW3)			
11.9%	6,217	6,957	668	8.9%	4,248	4,627	670	4.6%	1,701	1,779	669	5.6%	2,151	2,272	674
AP7 (PW14_VRW3)				AP7 (PW14_VRW3)				AP7 (PW14_VRW3)				AP7 (PW14_VRW3)			
16.1%	6,619	7,685	734	12.9%	4,875	5,504	737	8.4%	2,127	2,305	736	8.6%	2,254	2,448	740
AP8 (VR_TVI_PW)				AP8 (VR_TVI_PW)				AP8 (VR_TVI_PW)				AP8 (VR_TVI_PW)			
16.1%	6,802	7,897	766	12.9%	5,143	5,807	769	8.4%	2,418	2,621	769	8.8%	2,356	2,563	772
AP9 (VR_TVI_PW_SEV)				AP9 (VR_TVI_PW_SEV)				AP9 (VR_TVI_PW_SEV)				AP9 (VR_TVI_PW_SEV)			
13.1%	6,853	7,750	776	10.6%	5,213	5,766	780	8.6%	2,498	2,713	780	8.1%	2,370	2,562	783
AP10 (SWINBURNE_INFLOW)				AP10 (SWINBURNE_INFLOW)				AP10 (SWINBURNE_INFLOW)				AP10 (SWINBURNE_INFLOW)			
14.9%	6,951	7,987	941	13.3%	5,306	6,011	944	9.2%	2,597	2,836	944	8.8%	2,472	2,690	947

The resulting peak flows for both DCM #1 and DCM #2 exceed the design capacities of the crossing structures at Kayenta and Universe Blvd, as well as the twelve existing grade control structures in the arroyo.

The resulting peak flows for both DCM #3 and DCM #4 do not exceed the design capacities of either crossing structure or any of the grade control structures in the arroyo.

The report following this Developed Conditions Hydrology Report, “Development of Options and Recommendations”, will provide cost estimates for the full suite of detention basins, channel armoring options, and right of way needs, to determine the most efficient combinations of new and upgraded flood control structures to stabilize the CWB.

Abbreviations and Definitions

Abbreviations

Ac-ft.:	acre-feet; a volume of water one foot deep covering one acre or 43,560 cubic feet
AHYMO:	Albuquerque version of HYMO (hydrologic model program)
AMAFCA:	Albuquerque Metropolitan Arroyo Flood Control Authority
BHI:	Bohannan-Huston, Inc.
cfs:	cubic feet per second, used to quantify flow of water
CWB:	Calabacillas West Branch Arroyo
DMP:	Drainage management plan
HEC-HMS:	U.S. Army Corps of Engineers Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS:	U.S. Army Corps of Engineers Hydrologic Engineering Center River Analysis System
PMP:	Probable maximum precipitation
Q:	variable used to represent flow of water, units are cfs
RCP:	reinforced concrete pipe
MEI:	Mussetter Engineering, Inc.
SSCAFCA	Southern Sandoval County Arroyo Flood Control Authority
Tetra Tech	Tetra Tech, Inc.

Definitions

basin:	a region in which runoff flows to a common point
hydrology:	an earth science dealing with occurrence and distribution of the earth's water, including rainfall and the resulting runoff
hydraulics:	operated by or employing water. Hydraulic structures in this report are those which convey runoff (pipes, channels, streets, and dams). Hydraulics is the behavior of water in the hydraulic structures.
model:	a set of numerical data that describes the watershed conditions. Input data for the model includes rainfall, area of basins, slopes, and land usage. Output includes volume and flow of runoff.
watershed:	region in which many basins drain to a common point

1.0 INTRODUCTION

1.1 Scope

The Calabacillas West Branch Arroyo (CWB) watershed is located on the northwest area of Albuquerque, and straddles the city limits of Albuquerque and Rio Rancho. The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) contracted Tetra Tech to provide engineering services to develop the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan to identify arroyo improvements needed for the CWB as the watershed develops.

The CWB is a major tributary to the Calabacillas Main Branch Arroyo. Its confluence with the main branch is within the reservoir area of the AMAFCA Swinburne Detention Dam. The CWB, while small in comparison to the Calabacillas Main Branch, is a substantial part of the west side drainage system. With an area of approximately 5,960 acres, 1999 AHYMO hydrology models predict the CWB watershed would generate a peak flow of about 1,400 cfs in a 100-yr. event under current conditions, and this would increase to about 5,000 cfs under 2036 development conditions. The current floodplain covers approximately 165 acres, and impacts 100 different parcels. The 1999 Prudent Line limits span roughly 270 acres, and impact 184 parcels. As such, compilation of a Drainage Management Plan (DMP) for this arroyo will impact many private owners, multiple jurisdictions, and gives AMAFCA a great opportunity to extend the open space character of the arroyo west to the Rio Puerco divide. This DMP is also a key part of the AMAFCA/SSCAFCA joint effort to evaluate the Calabacillas Main Branch and resulting inflows to Swinburne Dam.

The watershed area is relatively long and linear in layout. The general limits of the watershed are the Rio Puerco divide to the west, the Calabacillas Middle Branch divide to the north, Swinburne Dam (located on Unser Blvd.) to the east, and Irving Blvd to the south. The CWB has one point of discharge to the AMAFCA Swinburne Dam. The CWB drainage basin includes diversions from the Piedras Marcadas Arroyo watershed via the Las Ventanas Detention Dam and Outfall Pipe.

Tetra Tech completed the Field Reconnaissance of the CWB on March 5, 2013. Representatives from AMAFCA, Tetra Tech, BHI, and SSCAFCA performed a reconnaissance-level investigation of the CWB. The field reconnaissance trip included qualitative observations and sediment sampling. A longitudinal profile of the CWB was also created to facilitate assessment of the existing channel. These data were analyzed and compared to the data presented by Mussetter Engineering, Inc. (MEI) in the Calabacillas Arroyo Prudent Line Study and Related Work – Development of a Prudent Line for the West Branch, MEI, 1999 (“1999 Prudent Line Study”). The Field Reconnaissance Report identifies locations where existing prudent lines are being encroached upon by lateral migration, vertical degradation is problematic, and was able to extrapolate the 1999 sediment bulking factors for various reaches of the arroyo for the 2013 existing conditions hydrology modeling.

1.2 Authorization

This Developed Conditions Hydrology Report, intended to support the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan was conducted by Tetra Tech with subconsultant assistance from BHI. Tetra Tech teamed with BHI on this project, with BHI subcontracted to perform HEC-HMS clear water modeling, prepare a PMP Hydrology Report to the NM Office of the Sate Engineer, and to assist in the development of storm drainage and storm water quality facility options. Tetra Tech will focus on the sediment transport through the arroyo, and will evaluate vertical stability, equilibrium slopes and complete all other tasks for the resulting Drainage and Storm Water Quality Management Plan.

Mr. Brad Bingham, PE, was the Project Manager for AMAFCA, and Mr. John Kelly, PE, was Tetra Tech’s Project Manager. Tetra Tech staff who contributed significantly to the work included Dr. Robert Mussetter, PE, Mr. Stuart Trabant, PE, and Mr. Kyle Shour, PE. BHI staff included Mr. Craig Hoover, PE, Ms. Alandren Etlantus, PE, and Ms. Sarah Ganley, PE.

Figure 1 – Vicinity Map of Calabacillas West Branch Arroyo

The vicinity map shows the overlapping jurisdictions of AMAFCA, Bernalillo County, the city of Albuquerque, and the city of Rio Rancho in the CWB watershed.

1.3 List of Tasks for Calabacillas West Branch Arroyo Drainage Management Plan

The following is a list and brief description of tasks required for the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan:

a. Coordination and Communication

The completion of the project includes coordinating this work with a coincident and complementary effort being conducted by the Southern Sandoval County Arroyo Flood Control Authority ("SSCAFCA"). SSCAFCA is performing similar analyses on the Calabacillas Middle Branch and Main Branch Arroyos, with the intent to combine the West Branch model into the SSCAFCA model to evaluate impacts to the AMAFCA Swinburne Dam. This will include evaluation of 100-year and PMP hydrology, and sediment transport to the dam on an annualized basis as well as for selected storm events. Coordination and communication among AMAFCA, Bernalillo County, and the city of Rio Rancho is also anticipated to resolve developed conditions land treatments that are different in various master planning documents.

b. Literature Review and As-Built Drawing Collection

A literature review document has been produced in order to better understand the history of drainage, development, open space and multiuse planning objectives for the CWB and its watershed. This review looked at eleven relevant planning documents and fourteen relevant technical documents addressing the CWB watershed. The review also identified 29 drainage structures or pipe discharges as existing features within the CWB. As-built drawings have also been obtained for all drainage structures within or discharging to the arroyo. This Literature Review Report was produced by Tetra Tech and BHI and submitted to AMAFCA on March 1, 2013.

c. Mapping

The BHI-produced 2012 Mid-Region Council of Governments (MRCOG) digital aerial photography and 2010 LiDAR topography was used as base mapping for this project. Since this is a master planning project, field surveys were not used to verify pipe inverts or slopes, as that level of detail is best suited for future design projects.

d. Field Reconnaissance

The field reconnaissance included qualitative observations and sediment sampling. A longitudinal profile of the CWB was also created to facilitate assessment of the existing channel. These data were analyzed and compared to the data presented by the 1999 Prudent Line Study. The intent was to make recommendations on the validity of the 1999 Prudent Line Study with regard to aggradation/degradational zones, prudent limits, and appropriate bulking factors for the DMP design storms. The field reconnaissance is documented in the *Calabacillas West Branch Arroyo Field Reconnaissance Report,* Tetra Tech, September 5, 2013.

e. Existing Conditions Hydrology and Hydraulics 2013

This report evaluated existing conditions hydrology for the CWB, using land treatments and storm drainage facilities that exist in 2013. The 2013 hydrology analysis used the HEC-HMS model based on SSCAFCA's Development Process Manual (DPM), Chapter 22, Drainage, Flood Control and Erosion Control (Revised April 2010), and is to be compatible with separate analysis being completed for the Calabacillas Watershed by SSCAFCA. Results from this modeling are documented in *Calabacillas West Branch Arroyo Drainage & Storm Water Quality Management Plan, Phase I, Task C, Existing Conditions Hydrology Report*, Tetra Tech and BHI, 2013 (2013 Existing Conditions Hydrology Report). Sediment bulking factors have been extrapolated from the 1999 Prudent Line Study. Flows and volumes have been determined at key analysis points for the 2-year, 10-year, and 100-year, 6-hr and 24-hr duration storm events.

f. Revised Existing Conditions Hydrology and Hydraulics (2016)

This report evaluated existing conditions hydrology for the CWB, using land treatments and storm drainage facilities that exist today. The hydrologic analysis was completed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS Version 4.0), based on the methodology presented in *AMAFCA's White Paper*. Results from this modeling are documented in *Calabacillas West Branch Arroyo Drainage & Storm Water Quality Management Plan, Phase I, Task C, Existing Conditions Hydrology Report: AMAFCA 2014 "White Paper Methodology*, Tetra Tech and BHI, 2016 (2016 Existing Conditions Hydrology Report). Sediment bulking factors have been interpolated from the 1999 Prudent Line Study. Flows and volumes have been determined at key analysis points for the 2-year, 10-year, and 100-year, 6-hr and 24-hr duration storm events.

g. PMP Hydrology Report to the NM Office of the State Engineer

This report developed hydrologic input parameters for the PMP design storms. This included delineation of drainage basins, land treatment, rainfall duration and intensity, reach lengths and sediment bulking. HEC-HMS modeling was done for the PMP design storms, which were the 6-hr. Local HMR-5, 6 hr. Local EM-1110-2-1411, and the 72 hr. General distribution. PMP volumes and peaks were provided at the Swinburne Dam reservoir. The PMP hydrologic analysis is documented in the *PMP Hydrologic Analysis Report for West Branch Calabacillas Arroyo – Inlet to Swinburne Dam Located in Bernalillo County, New Mexico, D457,* BHI, November 14, 2013.

h. Resolution of Developed Conditions Land Treatments

The CWB watershed has multiple overlapping jurisdictions. The watershed is wholly within AMAFCA and Bernalillo County jurisdiction. The upper reach of the watershed was annexed by the city of Rio Rancho in 2008. As such, the Rio Rancho Comprehensive Plan, and the Bernalillo County Comprehensive Plan, as well as privately produced Paradise West Master Plan and the Quail Ranch Master Plan were reviewed, and the conflicting land uses have been identified. The resolution of these conflicts is detailed in Section 3.1 herein.

This work is also being conducted simultaneously with a complementary SSCAFCA effort on the Calabacillas Main and Calabacillas Middle Branches, in order to have one unified hydrologic model for the Swinburne Dam watershed to support dam safety and emergency action plan efforts for the dam. Coordination of these land treatments and all other hydrologic inputs is ongoing with SSCAFCA.

i. Paseo del Volcan Diversion Conceptual Cost Estimates

This letter report evaluates the conceptual costs of the proposed Paseo del Volcan Diversion, which was envisioned as diverting flow resulting from the PMP event (approximately 17 inches of rain in a 6 hour event.) from the upper portions of the Boca Negra and the CWB watershed north to the Calabacillas Middle Branch Arroyo (CMB). The CWB PMP model was updated to include only the portion of the basins upstream of the PdV corridor and to reflect fully developed conditions. The AMAFCA White paper methodology was used, with the 6-hr EM-1110-2-1411 rainfall distribution. Resulting PMF flowrates were just over 19,000 cfs. Two alignments for the diversion were analyzed, and conceptual cost estimates were prepared for comparisons. A rip rap lined section was used for both alternatives. Results of the cost estimates indicated the diversion cost ranged from $21,560,000 to over $50M.

The AMAFCA Board of Directors was briefed on this conceptual cost estimate during their meeting of February 26, 2015, and concurred with AMAFCA staff that the Paseo del Volcan Diversion be removed from the AMAFCA Project Schedule as an AMAFCA-funded project. Results from this modeling are documented in the letter report titled *Paseo del Volcan Diversion Conceptual Cost Estimates*, BHI, January 21, 2015.

The AMAFCA Board subsequently authorized a Request for Proposals to modify the existing Boca Negra Drainage Management Plan to remove the diversion. The scope of the CWB Developed Conditions Hydrology and Hydraulic Modeling was also amended to include the full watershed with no diversion, resolve drainage basin limits on both the Boca Negra and CMB watersheds, and to incorporate the White Paper Methodology.

j. Developed Conditions Hydrology and Hydraulics (this report)

This task will analyze developed-conditions peak flows and volumes, based on the land treatments agreed to among AMAFCA, SSCAFCA, the city of Rio Rancho, and Bernalillo County. Four developed conditions scenarios are evaluated:

- DCM #1, a developed conditions hydrology model with existing flood control facilities and the elimination of the Paseo del Volcan Diversion to the Calabacillas Middle Branch.
- DCM #2, a developed conditions hydrology model with existing flood control facilities and the Quail Ranch Pond assumed to be in place. The Quail Ranch Pond will be sized to discharge the existing conditions 100-year, 24-hour peak flowrate as determined in this report.
- DCM #3, a developed conditions hydrology model with existing flood control facilities and the Quail Ranch Pond and the Paradise West Pond assumed to be in place. The Quail Ranch Pond will be sized to discharge the existing conditions 100-year, 24-hour peak flowrate as determined in the 2016 Existing Conditions Hydrology Report. The Paradise West Pond will be sized to be non-jurisdictional under the Rules and Regulations of the New Mexico Office of the State Engineer Dam Safety Bureau.

- DCM #4, a developed conditions hydrology model with existing flood control facilities in place and assuming one detention pond in each of the 52 developed conditions basins in the watershed for basins that are not bisected by the CWBA. Basins bisected by the CWBA are subdivided into two basins with a pond added for each new basin. The ponds are be sized to reduce each basin's developed conditions peak runoff to match 2016 existing conditions peak runoff for the 100-year, 24-hour event.

k. Sedimentation and Erosion Analyses (this report)

This includes sediment continuity analysis for both developed conditions scenarios. Sediment transport relationships have been developed for appropriate subreaches of the arroyo. Sediment supply from upstream and local tributaries will be estimated. Equilibrium slope analysis will be performed for each subreach. For DCM#1, the Engineer will compute prudent limits for the arroyo from Universe Blvd. (approx. Sta. 55+00) upstream to approximately Sta. 381+00, the alignment of future Paseo Del Volcan. For DCM #2, the Engineer will compute prudent limits from Universe Blvd. (approx. Sta. 55+00) to outlet of the proposed Quail Ranch Pond, at approximately Sta. 284+00. For DCM#3, the Engineer used the same quantitative assessments used in DCM 32 and #3 (rather than the qualitative assessment in the project scope) to compute the prudent limits from Universe Blvd to the outlet of the proposed Paradise West Pond at approximately Sta. 185+00.

l. Development of Facility Options for Developed Conditions

This task will provide recommendations for options for proposed drainage facilities within the watershed and set policy for drainage management in the CWB. This will include consideration of planning documents, storm water quality considerations, watershed management, right of way needs and construction estimates.

2.0 PROJECT AREA DESCRIPTION

2.1 Calabacillas West Branch Watershed

For the purposes of discussion, the CWB can be divided in to two general subreaches based on geomorphic and anthropogenic characteristics:

1. From the mouth at Station 0+00 to Universe Blvd at Station 55+00, the lower reach
2. From Universe Blvd at Station 55+00 to Station 350+00, the upper reach

The lower reach of the CWB has been modified by the construction of Swinburne Dam, where excavation of the Main Branch within the reservoir lowered the base level of the CWB by approximately 6 feet. This was identified in the 1999 Prudent Line Study. This resulted in incision in the lower reach. The 1999 Prudent Line Study compared the bed profile from a 1996 survey performed by BHI with the profile developed from a 1986 topography (taken from the Leedshill-Herkenhoff, Inc. HEC-2 model used in the previous prudent line study). MEI noted that the lower 500 feet of the arroyo had degraded by approximately 5 feet, and that the degradation over the next 4,500 feet ranged from 1-3 feet, The lower reach of the CWB has since been stabilized by ten grade control structures ("GCS") and two road crossing structures (Kayenta Pl. and Universe Blvd.)

Tetra Tech has compared the bed profile for the lower reach from the 1996 survey to the bed profile developed from the 2010 MRGCOG LiDAR topography as shown in the *Calabacillas West Branch Arroyo Field Reconnaissance Report,* Tetra Tech, Inc. 2013.

2.2 Piedras Marcadas Watershed Diversions to Calabacillas West Branch (via Las Ventanas Detention Dam Diversion)

The Las Ventanas Detention Dam and Outfall Pipe captures flows from the North Branch of the Piedras Marcadas Arroyo and a small portion of the CWB watershed. Flows are routed through the 172 ac-ft. (storage at crest of spillway) detention dam reservoir which discharges to a 42" RCP through an inclined-port principle spillway inlet structure. Flows are diverted north in the outfall pipe. The outfall pipe intercepts flows from the Little Window Detention Dam and from the Seville Unit 9 subdivision. The outfall storm drain is a 66" RCP that conveys 149 cfs to the CWB in the 100-year event.

3.0 DEVELOPED CONDITIONS HYDROLOGIC ANALYSES

3.1 Previous Drainage Studies

The primary drainage planning document used to date in this watershed is the *Calabacillas Arroyo Prudent Line Study and Related Work: Development of a Prudent Line for the West Branch*, MEI, 1999. The 1999 Prudent Line Study used the AHYMO program. MEI used an AHYMO model previously developed by Avid Engineering (Avid, 1995). The Avid model, developed to support design of bank protection at the old New Mexico Utilities Well Site, covered the upper CWB watershed beginning at the well site (9,500 feet upstream of Swinburne Dam) to the watershed limits. MEI revised the Avid model to include the arroyo downstream to Swinburne Dam and to account for the effects of the Las Ventanas subdivision.

The MEI model predates the *Quail Ranch Phase I DMP,* BHI, 2005, which included the proposed Paseo del Volcan Diversion of the Boca Negra to the Calabacillas Middle Branch, and included the proposed 75 ac-ft Quail Ranch detention dam with a developed conditions 100-yr, 24-hr inflow of 1,320 cfs and outflow of 295 cfs, with the developed conditions 5-yr event retained in its reservoir. Based on development of recent cost estimates and discussions with AMAFCA, the proposed Paseo del Volcan Diversion of the Boca Negra to the Calabacillas Middle Branch is not considered a viable option. Given this and a region wide shift to the public domain U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) hydrology model, AMAFCA directed the preparation of new existing and developed conditions HEC-HMS models for the CWB watershed, without the Paseo del Volcan Diversion, in April 2015 and July 2015.

Proposed land use densities for the developed conditions hydrology were identified within the watershed using the following four plans:

1. Rio Rancho Comprehensive Plan (RRCP), 2010, as amended 2015
2. Albuquerque Bernalillo County Comprehensive Plan (ABCCP), as amended 2013
3. Quail Ranch Special Area Plan (QRMP), 2005
4. Paradise West Master Plan (PWMP), 2004

The proposed land use and conflicts between the above documents were discussed in July 2013 with AMAFCA, City of Rio Rancho Planning Division, and BHI, and decisions regarding land use to be used for the West Branch Calabacillas hydrologic analysis are summarized below:

1. The specific master plans (which limit the density to levels below what is outlined in the RRCP) should be used for land use densities. For example, the QRMP has an average density of 5 DU/Ac for medium density residential while the RRCP limits medium density residential to a max of 16 DU/Ac.
2. For areas not covered by the specific master plans, the RRCP densities should be used.
3. Only areas outside of the City of Rio Rancho should use the ABCCP.

In 2014, AMAFCA began using the HEC-HMS methodology presented in the draft *White Paper – Migrating from AHYMO '97 to HEC-HMS (and USEPA SWMM)*, Easterling Consultants LLC, September 29, 2014 ("AMAFCA White Paper"). The use of the HEC-HMS model is required by the New Mexico Office of the State Engineer Dam Safety Bureau and its use is encouraged by local regulatory agencies. This AMAFCA White Paper methodology differs from the SSCAFCA methodology used in the 2013 Existing Conditions hydrologic modeling (documented in *Calabacillas West Branch Arroyo Drainage & Storm Water Quality Management Plan, Phase I, Task C, Existing Conditions Hydrology Report*, Tetra Tech and BHI, 2013) in several ways, including using a synthetic frequency storm distribution verses SSCAFCA's temporal distribution and using the Soil Conservation Service (SCS) Curve Number (CN) for the Loss Method versus SSCAFCA's Initial and Constant Loss Method. In 2015-2016, the existing conditions hydrologic analysis was updated using the AMAFCA White Paper methodology. Existing conditions results are documented in the *Calabacillas West Branch Arroyo Drainage & Storm Water Quality Management Plan, Phase 1, Task C, Existing Conditions Hydrology Report, AMAFCA 2014 "White Paper"* Methodology, Tetra Tech and BHI, 2016.

3.2 Hydrologic Model

The CWB watershed was analyzed to identify existing conditions flows to facilitate the development of a Drainage and Storm Water Quality Management Plan, including the determination of drainage improvements needed in the area. The hydrologic analysis was completed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS Version 4.0), based on the methodology presented in *AMAFCA's White Paper*. Several elements need to be considered to build a complete hydrologic model. The elements developed for the CWB model include basin delineation, curve numbers, precipitation, lag time, and routing. The developed conditions watershed analyses and data inputs are discussed in the following sections.

3.3 HEC-HMS Input Parameters

a. Basin Delineation

The developed conditions basins were established starting with the existing conditions delineated basins and adjusting, as appropriate, based on the master plans and comprehensive planning documents for the watershed. The two master plans used were the Paradise West Master Plan (June 2004) and the Quail Ranch Special Area Plan, Phase One (April 2005). The Albuquerque/Bernalillo County Comprehensive Plan (as amended through 2013) and the City of Rio Rancho Comprehensive Plan (November 2010, amended 2015) were also reviewed and utilized during the development of basins for this hydrologic analysis.

The basin delineation methodology that was used in the 2013 and 2016 existing conditions hydrologic analysis by Tetra Tech / BHI followed the subsequent process. The topographic surface data used for determining the basin boundaries was taken from the 2010 LiDAR aerial mapping of Mid-Region Council of Governments (2010 MRCOG Mapping). The data was resampled using a 10-foot grid size within the project area digital elevation model (DEM). Resampling the data to a 10-foot grid size ensures a capture of the required detail needed for analysis without adversely affecting processing time in ArcGIS. Preliminary basin boundaries were determined using ArcGIS loaded with HEC-GeoHMS and Arc Hydro software. Developing basin boundaries using the HEC-GeoHMS program consists of two main processing routines: DEM preprocessing steps, followed by the sub basin processing steps. In the preprocessing steps, a HydroDEM is created to correct for isolated low points in the Raw DEM. The resampled 10-foot MRCOG surface was used to create the HydroDEM and determine grids for flow direction, flow accumulation, streams, stream links and catchments (basins). This raster data was then used to create polyline and polygon shapefiles for streamlines and basin boundaries, respectively.

The basins developed for this model have been named to correspond with the master plans or subdivision that they fall within. For example, "QR" corresponds to basins within the Quail Ranch; "PW" for Paradise West; "VRW" for Ventana Ranch West; "VR" for Ventana Ranch; "TVI" for Albuquerque Technical Vocational Institute's West Side Campus (currently named Central New Mexico Community College – CNM); and "SEV" for the Seville Subdivision.

The basins were further refined and boundaries were modified to account for analysis points such as culverts, roadways, ponds, and storm sewers. The basin boundaries determined using the HEC-GeoHMS tools were compared with field observations, previous drainage analysis reports, U.S. Geologic Survey (USGS) National Hydrography Datasets (NHD), and aerial imagery to verify that the boundaries were in good agreement with watershed conditions and other data sources.

For developed conditions in the watershed, basins were modified based on the proposed future road alignments and crossings, as presented in the master plans for Quail Ranch and Paradise West. For the Quail Ranch area, when basin boundaries were adjusted to match the master plan roadway layout, two additional basins (QR1 and QR2) were delineated in the southwest corner of the watershed. This was done assuming that the future roadway and new infrastructure would accommodate this area's runoff within the Quail Ranch development. This same assumption resulted in increasing the size of several basins, as compared to existing conditions basins, along the proposed roadway alignments (QR9, QR20, QR21, and QR26). Two basins in Paradise West (PW1 and PW11) and one in Ventana Ranch West (VRW1) were also modified slightly to adjust for developed conditions roadway layouts. Table B below summarizes the basins that have different areas in developed conditions as compared to existing conditions.

Table C – Basins with Different Areas for Developed Conditions compared to Existing Conditions Models

Basin Name	Developed Conditions Basin Area (sq. mi.)	Existing Conditions Basin Area (sq. mi.)	Increase in Area with Developed Conditions (sq. mi.)
QR1	0.27	Not in Model	0.27
QR2	0.06	Not in Model	0.06
QR9	0.63	0.43	0.20
QR20	0.32	0.23	0.09
QR21	0.33	0.29	0.03
QR26	0.23	0.14	0.09
PW1	0.10	0.10	0.00
PW11	0.13	0.12	0.01
VRW1	0.33	0.30	0.03
		TOTAL	**0.78**

A total of 52 basins were delineated for the developed conditions models with basin sizes ranging from 0.01 sq. mi. to 0.48 sq. mi. with an average basin size of approximately 0.22 sq. mi. The total watershed area for developed conditions is 11.88 sq. mi. This general basin layout applies to three of the developed conditions models (refer to Section 3.4 for details – DCM #1, DCM #2, and DCM #3). The fourth model, DCM #4, divides some of the basins into smaller areas (refer to Section 3.4.d for details).

Figure 2 – Calabacillas West Branch DCM #1, DCM #2, and DCM #3 Drainage Basin MapFigure 2 and Figure 3 show the general drainage basins developed in this study.

Table D – HEC-HMS Basin Input Parameters below summarizes the HEC-HMS input parameters for these basins.

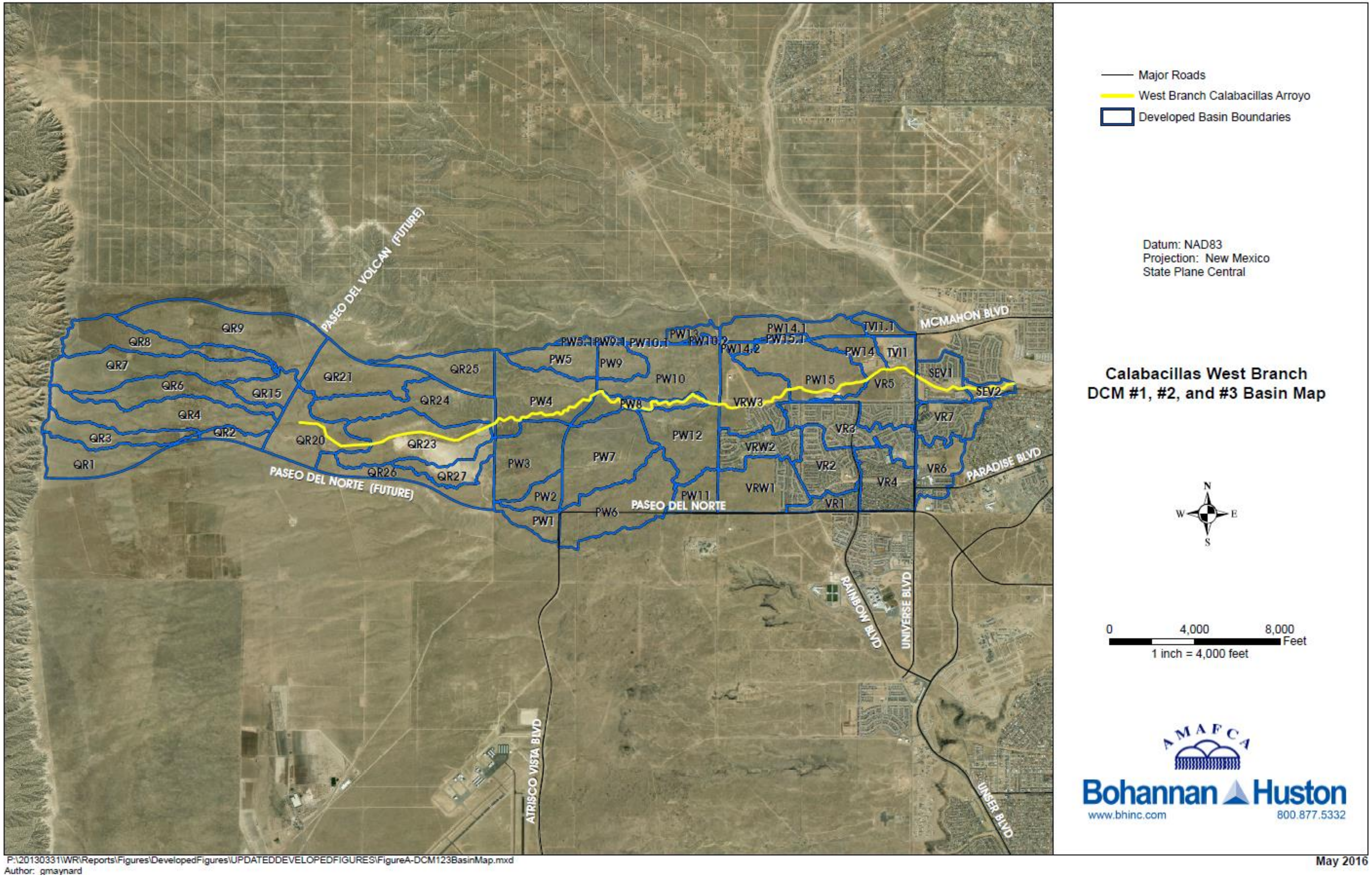
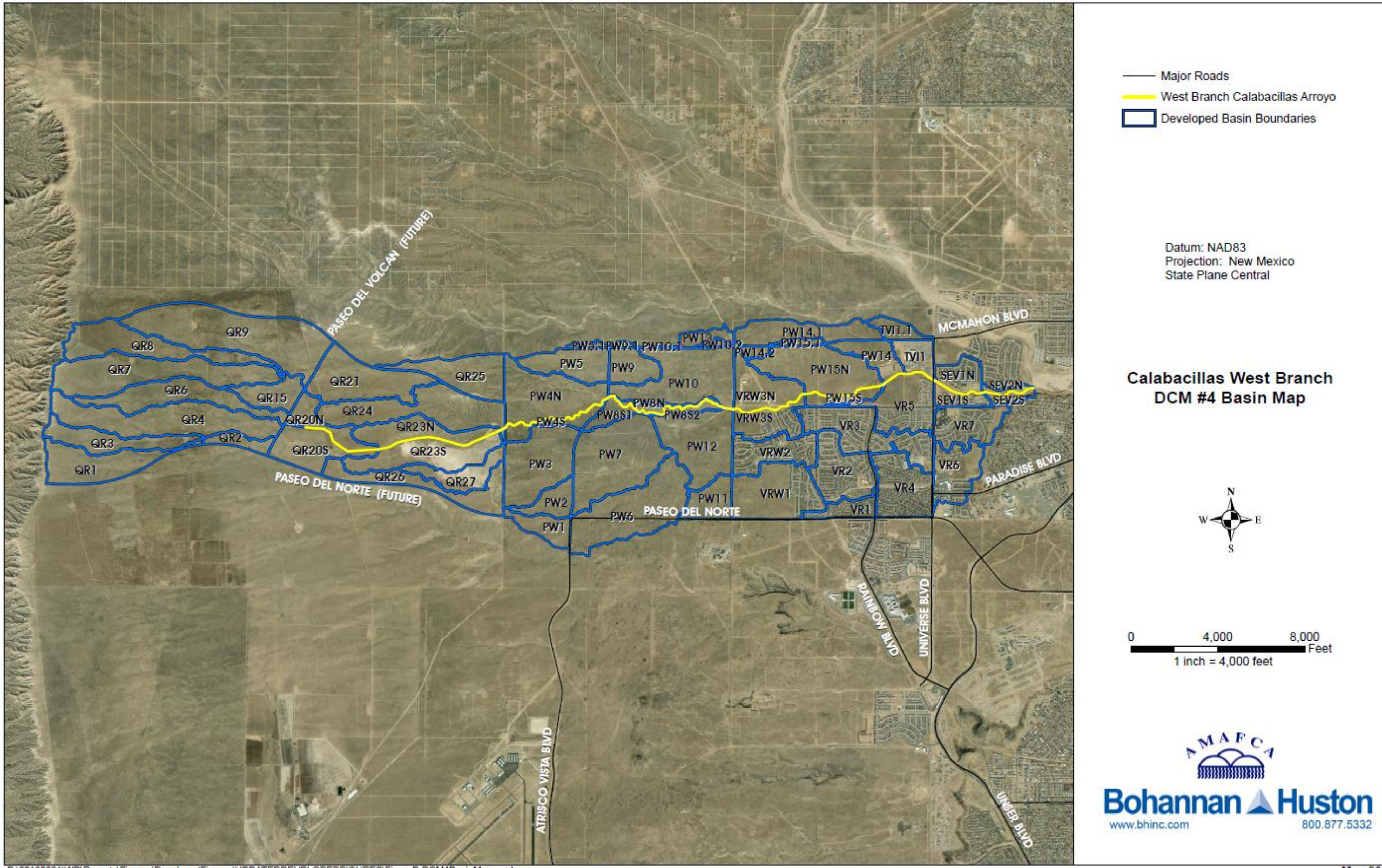


Figure 2 – Calabacillas West Branch DCM #1, DCM #2, and DCM #3 Drainage Basin Map



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Author: gmaynard
May 2016

Figure 3 – Calabacillas West Branch DCM #4 Drainage Basin Map

Table D – HEC-HMS Basin Input Parameters

Basin / Junction	Developed Conditions			Basin / Junction	Developed Conditions		
Name	Basin Area (mi²)	CN	Lag Time (min)	Name	Basin Area (mi²)	CN	Lag Time (min)
QR9_P	0.3345	72	15.0	QR24	0.4061		
QR9_I	0.2966	98	15.0	QR21_P	0.1435	72	25.2
QR9	0.6311			QR21_I	0.1826	98	25.2
QR7_P	0.2841	69	16.8	QR21	0.3261		
QR7_I	0.1862	98	16.8	PW4_P	0.2187	56	13.3
QR7	0.4703			PW4_I	0.2035	98	13.3
QR8_P	0.2071	69	17.7	PW4	0.4222		
QR8_I	0.1122	98	17.7	QR25_P	0.1312	62	11.4
QR8	0.3193			QR25_I	0.1024	98	11.4
QR4_P	0.2437	70	20.6	QR25	0.2336		
QR4_I	0.2249	98	20.6	PW3_P	0.1919	71	10.4
QR4	0.4686			PW3_I	0.1536	98	10.4
QR6_P	0.1370	72	16.1	PW3	0.3455		
QR6_I	0.1436	98	16.1	QR26_P	0.0797	72	17.2
QR6	0.2806			QR26_I	0.1547	98	17.2
QR1_P	0.1318	67	13.7	QR26	0.2344		
QR1_I	0.1372	98	13.7	QR27_P	0.0478	72	17.7
QR1	0.2690			QR27_I	0.0925	98	17.7
QR3_P	0.1003	68	13.6	QR27	0.1403		
QR3_I	0.0952	98	13.6	PW2_P	0.0494	72	9.3
QR3	0.1955			PW2_I	0.0511	98	9.3
QR2_P	0.0312	69	10.5	PW2	0.1005		
QR2_I	0.0325	98	10.5	PW7_P	0.1809	72	13.4
QR2	0.0637			PW7_I	0.2944	98	13.4
QR15_P	0.0668	72	16.9	PW7	0.4753		
QR15_I	0.0627	98	16.9	PW6_P	0.2093	77	16.9
QR15	0.1295			PW6_I	0.2457	98	16.9
QR20_P	0.0961	71	23.1	PW6	0.4550		
QR20_I	0.2242	98	23.1	PW1_P	0.0352	72	10.7
QR20	0.3203			PW1_I	0.0684	98	10.7
QR23_P	0.2265	66	18.1	PW1	0.1036		
QR23_I	0.2380	98	18.1	PW10_P	0.1486	67	12.6
QR23	0.4645			PW10_I	0.2594	98	12.6
QR24_P	0.2066	66	16.5	PW10	0.4080		
QR24_I	0.1995	98	16.5	PW12_P	0.1395	73	14.0

Basin / Junction	Developed Conditions		
Name	Basin Area (mi²)	CN	Lag Time (min)
PW12_I	0.2062	98	14.0
PW12	0.3457		
PW5_P	0.0635	62	10.9
PW5_I	0.1171	98	10.9
PW5	0.1806		
PW5.1_P	0.0068	55	6.7
PW5.1_I	0.0097	98	6.7
PW5.1	0.0165		
PW9_P	0.0465	68	11.4
PW9_I	0.0738	98	11.4
PW9	0.1203		
PW9.1_P	0.0039	57	5.4
PW9.1_I	0.0056	98	5.4
PW9.1	0.0095		
PW8_P	0.0611	60	14.3
PW8_I	0.0720	98	14.3
PW8	0.1331		
PW10.2_P	0.0055	56	13.6
PW10.2_I	0.0089	98	13.6
PW10.2	0.0144		
PW10.1_P	0.0021	55	5.1
PW10.1_I	0.0037	98	5.1
PW10.1	0.0058		
VRW3_P	0.1745	66	14.5
VRW3_I	0.1758	98	14.5
VRW3	0.3503		
PW15_P	0.1688	56	14.2
PW15_I	0.2323	98	14.2
PW15	0.4011		
PW14.1_P	0.0879	67	14.5
PW14.1_I	0.1245	98	14.5
PW14.1	0.2124		
PW13_P	0.0238	66	14.9
PW13_I	0.0313	98	14.9
PW13	0.0551		
PW14.2_P	0.0095	61	12.7

Basin / Junction	Developed Conditions		
Name	Basin Area (mi²)	CN	Lag Time (min)
PW14.2_I	0.0149	98	12.7
PW14.2	0.0244		
PW14_P	0.0499	55	14.0
PW14_I	0.0692	98	14.0
PW14	0.1191		
PW15.1_P	0.0089	56	6.7
PW15.1_I	0.0144	98	6.7
PW15.1	0.0233		
VR5_P	0.1197	60	17.3
VR5_I	0.1379	98	17.3
VR5	0.2576		
TVI1_P	0.0459	55	13.8
TVI1_I	0.0500	98	13.8
TVI1	0.0959		
TVI1.1_P	0.0149	62	10.8
TVI1.1_I	0.0487	98	10.8
TVI1.1	0.0636		
SEV1_P	0.0904	55	13.9
SEV1_I	0.0792	98	13.9
SEV1	0.1696		
VRW1_P	0.1559	69	13.8
VRW1_I	0.1758	98	13.8
VRW1	0.3317		
PW11_P	0.0444	81	12.4
PW11_I	0.0825	98	12.4
PW11	0.1269		
VRW2_P	0.0864	64	14.6
VRW2_I	0.1139	98	14.6
VRW2	0.2003		
VR2_P	0.1126	62	13.9
VR2_I	0.1377	98	13.9
VR2	0.2503		
VR4_P	0.1165	75	20.8
VR4_I	0.1367	98	20.8
VR4	0.2532		
VR1_P	0.0360	74	11.5

Basin / Junction	Developed Conditions		
Name	Basin Area (mi^2)	CN	Lag Time (min)
VR1_I	0.0497	98	11.5
VR1	0.0857		
VR3_P	0.1530	66	15.6
VR3_I	0.1746	98	15.6
VR3	0.3276		
VR6_P	0.1236	81	19.4
VR6_I	0.0691	98	19.4
VR6	0.1927		
VR7_P	0.0886	62	18.6
VR7_I	0.1062	98	18.6
VR7	0.1948		
SEV2_P	0.0476	55	9.4
SEV2_I	0.0076	98	9.4
SEV2	0.0552		

b. Loss Methods (Land Treatment)

The loss method in HEC-HMS provides an estimate of the precipitation that is intercepted, or infiltrates into the soil, and therefore is not part of the total storm runoff. The rainfall loss for this study was accomplished using the SCS CN Method outlined in AMAFCA's White Paper. The SCS is currently known as the Natural Resources Conservation Service (NRCS). This rainfall loss is associated with the CN and major factors that determine the CN are hydrologic soil group, cover type (land use), and hydrologic condition. Tables 2-2a through 2-2d in NRCS's *Urban Hydrology for Small Watersheds, Technical Report 55* (referred to as TR-55) define the recommended CNs for various cover types, hydrologic conditions, and hydrologic soil groups. Following *AMAFCA's White Paper* methodology, a CN was determined for the pervious portion of each basin. Based on cover type, hydrologic condition, and hydrologic soil groups (A, B, C, or D), the CNs used are shown in Table E. The underlying pervious portion of the watershed, in developed conditions, is classified as Desert Shrub.

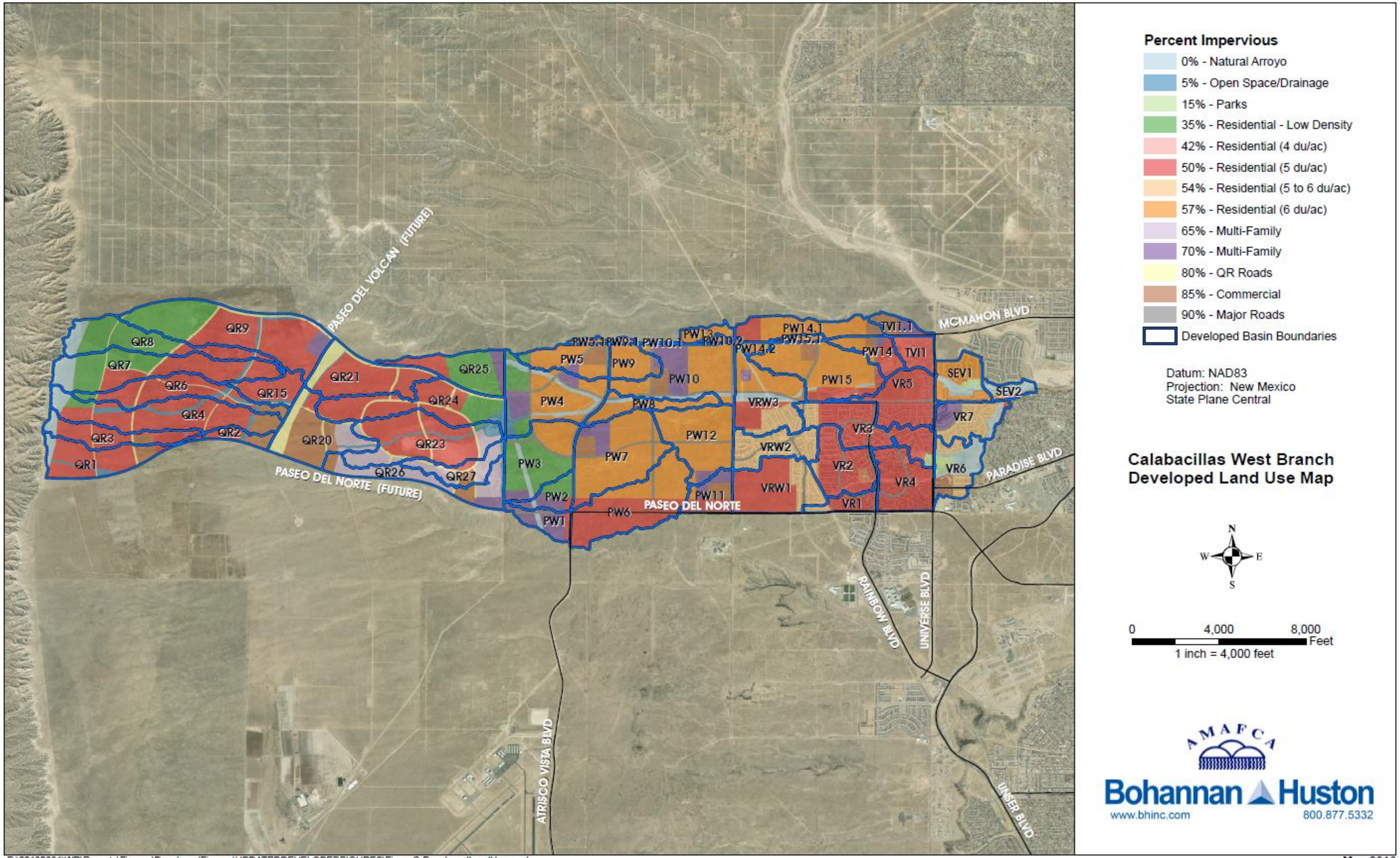
Table E – Curve Numbers for Pervious Portions of CWB Existing Conditions Basins

Cover Type and Hydrologic Condition	Curve Number for Hydrologic Soil Group			
	A	B	C	D
Pervious - Desert Shrub, Fair	55	72	81	86

Following AMAFCA's White Paper methodology, basins are modeled using three components:

1) the pervious portion of the basin is modeled as a sub-basin (naming convention – Basin ID_P),
2) the impervious portion of the basin is modeled as a sub-basin (naming convention – Basin ID_I), and
3) a junction that adds the pervious and impervious sub-basin elements.

The percent impervious for the developed areas in the watershed were classified from available subdivision drainage reports for existing development. For developed areas for which reports were not accessible, aerial imagery was used to determine the DU/ac. For future developed areas, the Quail Ranch and Paradise West Master Plans, as well as the Rio Rancho Comprehensive Plan (RRCP) and Albuquerque Bernalillo County Comprehensive Plan (ABCCP) were used to determine DU/ac development for residential as well as define areas planned for commercial, parks, and open space. Based on the DU/ac for residential areas, the percentage of impervious area could be calculated for each existing and proposed subdivision using equations from Table A-5 of the City of Albuquerque DPM, Section 22.2. For commercial, multi-family, parks, and open space, Table A-5 of the City of Albuquerque DPM, Section 22.2 was also used to determine the percent impervious for these land uses. Table F lists the percent impervious used for typical land use categories in the basins. The impervious percentages by category were then utilized in ArcGIS to calculate a percentage of impervious area for each basin. Using this, an impervious and a pervious area were calculated for each basin so that the two sub-basin components for each basin could be modeled in HEC-HMS. This information was compiled in a developed Land Treatment ArcGIS shapefile; refer to Figure 4. The basins' pervious and impervious sub-basin components/areas and CNs are summarized in Table D.



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Author: gmaynard

May 2016

Figure 4 – Calabacillas West Branch Developed Land Use Map

Table F – Percent Impervious for Land Uses

Land Use Category	% Impervious
Commercial	85 %
Drainage Ponds	5 %
Major Roads	90 %
Multi Family	65 to 70 %
Natural Arroyo	0 %
Open Space / Trail in Developed Conditions	5 %
Park	15 %
Quail Ranch MP Roadway Corridors	80 %
Residential	35 % to 57%

c. Precipitation

2-yr, 10-yr, and 100-yr (50%, 10% and 1% probability events, respectively), 6-hr and 24-hr storm events were simulated for this study. In addition, the 5-yr, 24-yr, and 50-yr, 24-hr storm events were also modeled. The precipitation depths, for the analyzed events, were extracted from the National Oceanic and Atmospheric Administration (NOAA) online data server using the NOAA Atlas 14, Volume 1, Version 5 Point Precipitation Frequency Estimates, accessed on March 11, 2013 (values were confirmed as current by accessing the NOAA online server on July 18, 2016). For the 6-hr and 24-hr storms, rainfall depth for the simulated events are summarized in Table G below.

Table G– NOAA Atlas 14 Point Precipitation Frequency Estimates

Storm Recurrence Interval	NOAA 6-hour duration precipitation estimate (inches)	NOAA 24-hour duration precipitation estimate (inches)
2-year	0.968	1.23
5-year	6-hour event not modeled	1.54
10-year	1.47	1.79
25-year	6-hour event not modeled	2.13
50-year	6-hour event not modeled	2.39
100-year	2.27	2.66

In this analysis, no depth-area reduction factor was used since the analysis results will ultimately be used for planning and development of recommended infrastructure. Precipitation data is provided in Appendix A. A synthetic frequency storm at 25 percent intensity position distribution, as specified in HEC-HMS, following the AMAFCA White Paper methodology, was used for the meteorological model.

The cumulative rainfall distribution was plotted for the HEC-HMS frequency storm for the 100-year, 6-hour and 100-year, 24-hour events and is shown in Figure 5. Using the AMAFCA White Paper methodology with the frequency storm at 25 percent intensity position, the 24-hr event results in higher peak discharges than the 6-hr event. Since, the 24-hr distribution has the peak intensity occurring later with the same magnitude as the 6-hr event; the peak intensity occurs after initial rainfall has occurred and the ground is more saturated resulting in higher peak runoff.

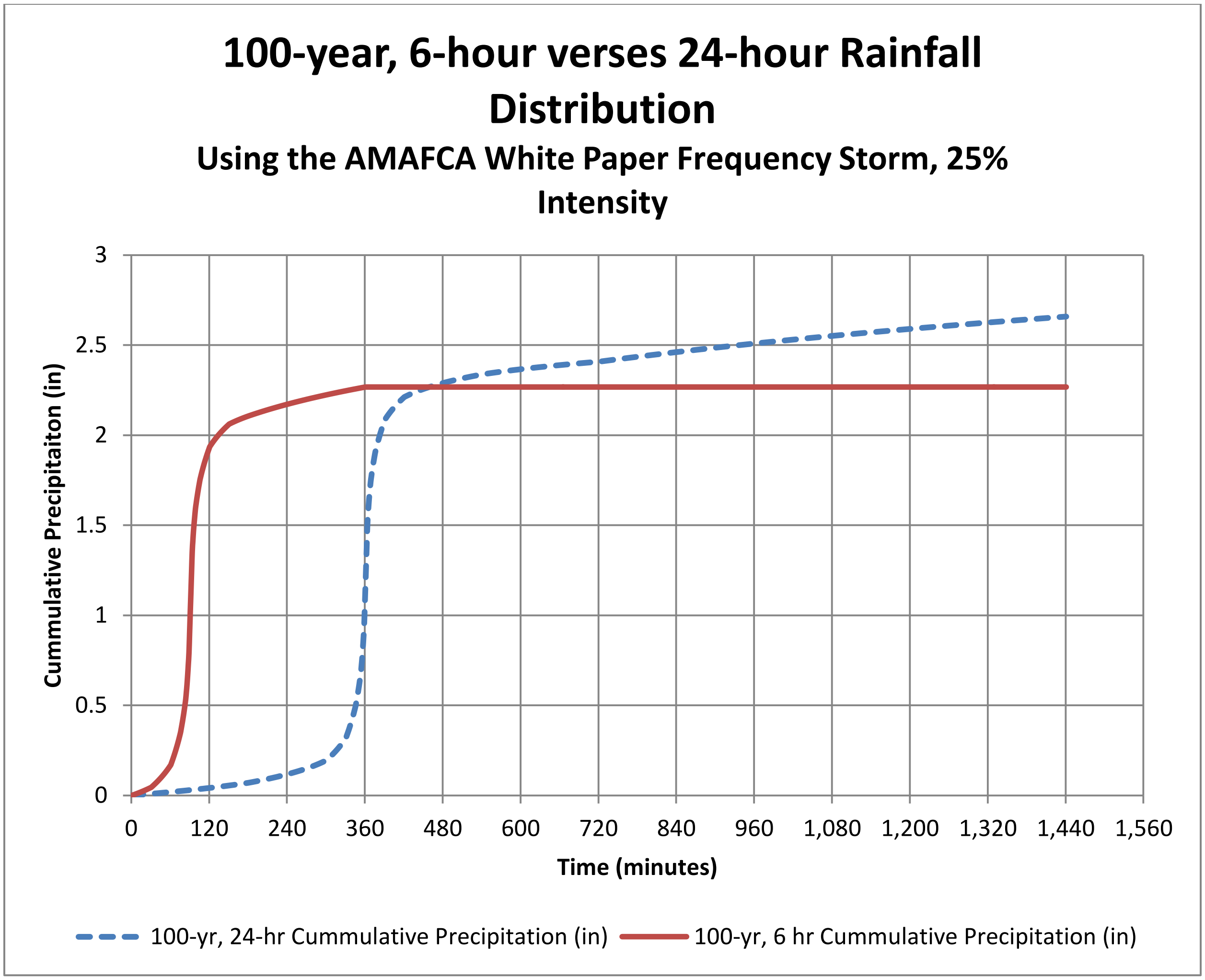

Figure 5 – 100-year, 6-hour versus 24-hour Rainfall Distribution Using the AMAFCA White Paper Frequency Storm, 25% Intensity

d. Lag Time

Basin model transform method uses the SCS unit hydrograph, which is input into HEC-HMS as Basin Lag Time. Following the AMAFCA White Paper methodology, Basin Lag Time is a function of time of concentration (Tc); (Lag Time = 0.6 x Tc). The Tc is calculated using the method prescribed in TR55, which is the same method used in the City of Albuquerque DPM. The HydroDEM and HEC-GeoHMS subbasin processing tools were used to determine the input parameters for the Tc and basin Lag time calculations such as: subbasin area, longest flow path, flow path slope, and basin centroid. As directed by AMAFCA, for this project, the Basin Lag Time was assumed to be the same for both the pervious and impervious sub-basin components of each basin. The lag time calculations apply to three of the developed conditions models (refer to Section 3.4 for details – DCM #1, DCM #2, and DCM #3). The fourth model, DCM #4, divides some of the basins into smaller areas (refer to Section 3.4.d for details) and changes the lag time calculations. DCM #4 lag time values were edited for the split basins and calculations are provided in Appendix B and are summarized in Appendix C. Tc and Basin Lag Time calculations are provided in Appendix B and summarized in Table D – HEC-HMS Basin Input Parameters. The values summarized in Table D refer to DCM #1, DCM #2, and DCM #3.

e. Routing

The Muskingum-Cunge Routing method was used for this study; this method is appropriate for this study and is consistent with the AMAFCA White Paper methodology. The required input for this method includes the cross-section geometry, channel length and slope, and Manning's n-values for routes. The arroyo routes were drawn in a GIS shapefile, shown in Appendix D, and are based on existing drainage infrastructure and proposed drainage infrastructure from area master plans. For all arroyo routes, the shape, length, and average slope were determined from the aerial imagery, the DEM for the area, and pertinent drainage reports and master plans. The Manning's n value used for the main branch of the CWB, which is assumed to remain a natural arroyo even in developed conditions, was obtained from the developed conditions HEC-RAS model (n=0.042). For the other route reaches in the watershed, not within the main arroyo channel, a Manning's n value of 0.03 was used. For natural arroyos, the cross-section was approximated as a rectangular channel with each arroyo bottom width determined from the 2012 MRCOG aerial imagery. For storm drain conveyances, the diameter was taken from the applicable drainage reports or from City of Albuquerque GIS Storm Drain shapefile for existing conveyances and from the Quail Ranch Drainage Master Plan for the proposed storm drain in the Quail Ranch area. For improved channel conveyances, the side slopes and bottom widths were determined from applicable drainage reports. Table H below outlines the routing parameters used in the hydrologic analysis.

Table H – HEC-HMS Routing Parameters

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Shape	Dia. (ft)	Width (ft)	Side Slope
RT.QR8	1,276	0.007	0.013	Circular	3.5	--	--
RT.QR1	1,500	0.01	0.013	Circular	3.5	--	--
RT.QR3	3,810	0.014	0.013	Circular	4	--	--
RT.QR4	5,044	0.005	0.013	Circular	4	--	--
RT.QR20	5,198	0.014	0.013	Circular	6	--	--
RT.QR21	4,586	0.020	0.013	Circular	6	--	--
RT.QR24	5,769	0.020	0.042	Rectangular	--	10	--
RT.PW4	6,709	0.018	0.042	Rectangular	--	12	--
RT.PW3	5,825	0.019	0.03	Rectangular	--	5	--
RT.PW7	3,599	0.018	0.03	Rectangular	--	12	--
RT.PW5	2,871	0.021	0.03	Rectangular	--	30	--
RT.PW9	3,417	0.019	0.03	Rectangular	--	12	--
RT.PW10	3,783	0.013	0.042	Rectangular	--	15	--
RT.VRW3	3,970	0.015	0.042	Rectangular	--	15	--
RT.PW.13	6,662	0.018	0.03	Rectangular	--	10	--
RT.PW14.1	1,701	0.017	0.03	Rectangular	--	6	--
RT.PW14	1,569	0.013	0.042	Rectangular	--	15	--
RT.TVI	1,108	0.010	0.013	Circular	5	--	--
RT.VR5	2,440	0.012	0.042	Rectangular	--	40	--
RT.SEV1	2,506	0.023	0.042	Rectangular	--	45	--
RT.PW11	3,184	0.023	0.03	Rectangular	--	15	--
RT.PW11A	1,234	0.01	0.013	Circular	7	--	--
RT.VRW1	2,094	0.029	0.013	Trapezoidal	--	10	3
RT.VRW2	2,862	0.019	0.013	Circular	7	--	--

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Shape	Dia. (ft)	Width (ft)	Side Slope
RT.VRW2A	1,246	0.015	0.013	Trapezoidal	--	10	3
RT.VR2	2,606	0.019	0.013	Trapezoidal	--	10	3
RT.LVDD	2,257	0.006	0.013	Circular	3.5	0	--
RT.LWRD	1,549	0.010	0.013	Circular	5	0	--

3.4 Developed Conditions Models

Four HEC-HMS developed conditions hydrologic models were developed for this study. The general parameters for each of these models are discussed in the preceding sections. 2-yr, 10-yr and 100-yr, 6-hr and 24-hr storm events were simulated. In addition, 5-yr, 25-yr, and 50-yr, 24-hr events were also modeled.

The CWB has a developed drainage area of approximately 11.9 square miles. In general, runoff flows from the west to the east and will discharge from the study area at the CWB inlet to the Swinburne Dam. Currently, flow from the watershed reaches the CWB either through natural drainage ways or drainage facilities that have been constructed to divert flow from the Las Ventanas Detention Dam and Little Window Dam that ultimately are conveyed to the CWB through a single outfall located downstream of Kayenta Road. This diverted flow accounts for a drainage area of 1.92 square miles and includes Basins PW11, VRW1, VRW2, VR1, VR2, VR3, VR4, VR6 and VR7. Runoff from all other basins in the study area is directly conveyed to the CWB.

For developed conditions scenarios, the five hydrologic models evaluated include:

- DCM #1, a developed conditions hydrology model with existing flood control facilities and the elimination of the Paseo del Volcan Diversion to the Calabacillas Middle Branch.
- DCM #2, a developed conditions hydrology model with existing flood control facilities and the Quail Ranch Pond assumed to be in place. The Quail Ranch Pond was sized to discharge the existing conditions 100-yr, 24-hr peak flowrate as determined in the 2016 CWB Existing Conditions Hydrology Report.
- DCM #3, a developed conditions hydrology model with existing flood control facilities and the Quail Ranch Pond and the Paradise West Pond assumed to be in place. The Quail Ranch Pond was sized in DCM#2. The Paradise West Pond was sized to reduce the 100-yr, 24-hr peak flowrate in the downstream arroyo reach to minimize required improvements to the existing drainage structures downstream of Universe Blvd.
- DCM #4, a developed conditions hydrology model with individual ponds placed in each basin within the watershed. These individual ponds were sized to discharge the developed conditions 100-yr, 24-hr peak flowrate to match the existing conditions 100-yr, 24-hr peak flowrates from the 2016 CWB Existing Conditions Hydrology Report.

Digital HEC-HMS models for the five developed conditions scenarios are included on a CD in Appendix D. The HEC-HMS output files are included in Appendix E. Analysis points, as shown in Figure 6 through Figure 9, are located at key reaches along the CWB. A summary of the 24-hr event runoff results at the analysis points for each model is presented in Table N– Developed Conditions 100-year Peak Flow and Volume Comparisons**.** For each analysis point, the table provides the bulking factor in percent, unbulked and bulked peak discharge in cfs, and runoff volume in ac-ft for the 100-yr, 24-hr event.

a. Developed Conditions Model #1 (DCM #1)

DCM #1 portrays developed conditions in the watershed with existing flood control facilities. Further down the watershed, in the subreaches near Swinburne Dam, there are several existing structures including bridge culverts at Universe Blvd and Kayenta St. and 12 grade control structures in between Universe Blvd. and Swinburne Dam. For developed conditions, the flow was analyzed using the AMAFCA White Paper methodology as described in Section 3.3 above. This model predicts the flow throughout the watershed when no new flood control facilities are added to current conditions and determines whether additional facilities are needed. The DCM #1 results, for 24-hr storms, are summarized in Table H and shown in Figure 6. As seen in Table NJ, many of the downstream existing structure capacities are unable to convey the developed conditions flows. The sections following, that present the results of DCM #2, #3 and #4, provide different developed conditions models that will reduce flows throughout the watershed to minimize impacts to existing drainage facilities.

Table H – Developed Conditions Model #1 (DCM #1) Results

Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)		Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)
		Unbulked	*Bulked*					*Unbulked*	*Bulked*	
AP1 (QR3_15)						**AP6 (PW_VRW3)**				
2	1.8%	759	772	61		2	4.7%	1,721	1,802	208
10	2.7%	1,525	1,566	120		10	8.1%	3,467	3,748	394
100	3.6%	2,840	2,942	211		100	11.9%	6,217	6,957	668
AP2 (QR3_20)						**AP7 (PW14_VRW3)**				
2	3.2%	791	817	71		2	6.9%	1,815	1,940	231
10	4.6%	1,512	1,581	136		10	10.6%	3,693	4,085	434
100	6.8%	2,651	2,832	234		100	16.1%	6,619	7,685	734
AP3 (QR3_24)- QRP						**AP8 (VR_TVI_PW)**				
2	6.0%	1,228	1,301	116		2	6.9%	1,855	1,983	242
10	9.9%	2,395	2,632	222		10	10.6%	3,797	4,199	454
100	13.2%	4,288	4,853	379		100	16.1%	6,802	7,897	766
AP4 (QR3_25_PW4)						**AP9 (VR_TVI_PW_SEV)**				
2	11.1%	1,273	1,414	125		2	7.5%	1,866	2,005	245
10	17.7%	2,473	2,910	238		10	10.8%	3,824	4,237	461
100	22.4%	4,381	5,363	406		100	13.1%	6,853	7,750	776
AP5 (PW1_12)						**AP10 (SWINBURNE_INFLOW)**				
2	6.0%	1,692	1,794	200		2	8.1%	1,931	2,087	296
10	8.4%	3,395	3,680	379		10	11.1%	3,907	4,340	557
100	12.1%	6,085	6,821	643		100	14.9%	6,951	7,987	941

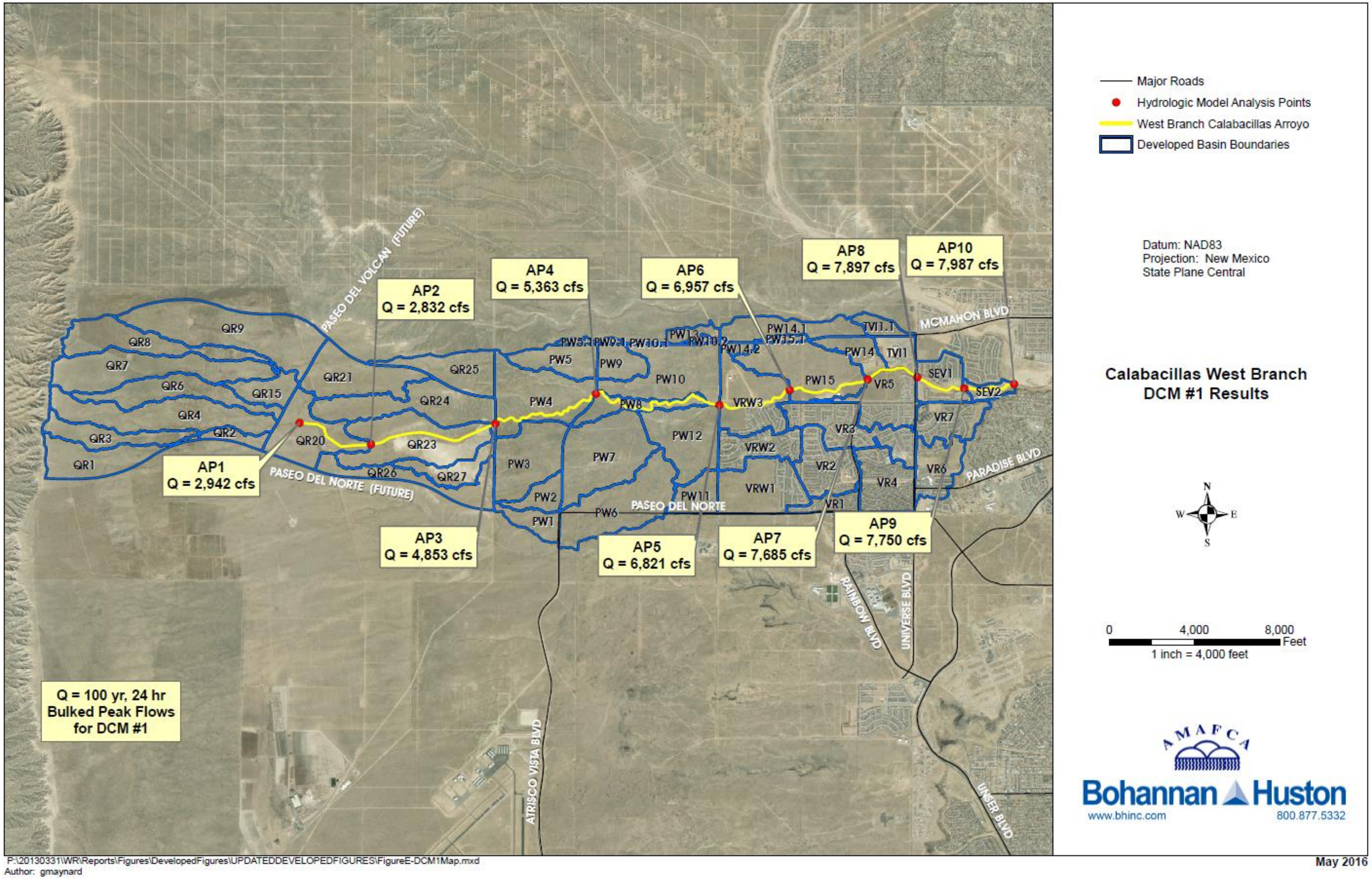

Figure 6 – Calabacillas West Branch DCM #1 Results

b. Developed Conditions Model #2 (DCM #2)

The second developed conditions model involved the addition of the Quail Ranch Pond (QRP) to the WBC Watershed at Analysis Point 3 (AP3), near the east boundary of Quail Ranch property – see Figure F. After analyzing DCM #1, it was determined that the flow through downstream structures was well above existing capacities, and an intermediate flood control facility was needed. This pond was introduced to manage the increase in outflow due to new residential development upstream of this proposed pond. The goal was to configure QRP in such a way that the developed conditions peak 100-yr, 24-hr outflow from the pond is equal to the existing conditions 100-yr, 24-hr outflow at that same location (837 cfs unbulked, 950 cfs bulked) as agreed to by AMAFCA in a March 8, 2016, letter to Tetra Tech, Inc. Maintaining the outflow at a constant level reduces the need for larger and/or additional downstream structures and allows for continued utilization of existing and natural channels. The existing conditions flow was determined in the 2016 CWB Existing Conditions Hydrology Report.

An initial pond size was proposed for the QRP in the Quail Ranch Master Plan. The sizing of the initial Quail Ranch 75 ac-ft detention pond included the proposed Paseo del Volcan Diversion of the Boca Negra to the Calabacillas Middle Branch and was sized for a developed conditions 100-yr, 24-hr inflow of 1,320 cfs. Based on development of recent cost estimates and discussions with AMAFCA, the proposed Paseo del Volcan Diversion of the Boca Negra to the Calabacillas Middle Branch is not considered a viable option and has been removed from this analysis. The removal of the PDV diversion increases the inflow into the proposed QRP by nearly 3.5 times, making it necessary to increase the size of the previously proposed pond. Through an iterative process using HEC-HMS, a storage size 3.5 times the original storage proposed in the Quail Ranch Master Plan was found to best replicate the required conditions. The final conceptual configuration of QRP consists of a four-sided conical reservoir: approximately 925 feet by 800 feet surface footprint (17 acres), 20-foot height, with assumed 4:1 side slopes. These conceptual pond dimensions do not account for freeboard, spillway or maintenance access requirements. These are conceptual recommendations based on preliminary information and are intended to provide a starting point for actual pond design at a later date.

The DCM #2 results, for the modeled 24-hour storms, are summarized in Table I and shown in Figure 7. As seen in Table N, the downstream existing structure design capacities are still unable to convey the developed conditions flows. The sections following, that present the results of DCM #3 and #4, provide additional developed conditions models that will reduce flows throughout the watershed to minimize impacts to existing drainage facilities.

Table I – Developed Conditions Model #2 (DCM #2) Results

Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)
		Unbulked	*Bulked*	
AP1 (QR3_15)				
2	1.8%	759	772	61
10	2.7%	1,525	1,566	120
100	3.6%	2,840	2,942	211
AP2 (QR3_20)				
2	3.2%	791	817	71
10	4.6%	1,512	1,581	136
100	6.7%	2,651	2,829	234
AP3 (QR3_24) - Flow into QRP				
2	2.4%	1,228	1,257	116
10	3.5%	2,395	2,479	222
100	4.6%	4,288	4,485	379
AP3 (QRP_OUT) - Flow out of QRP				
2	2.4%	301	308	115
10	3.5%	577	597	221
100	4.6%	837	875	378
AP4 (QR3_25_PW4)				
2	3.9%	311	323	125
10	5.9%	594	629	238
100	8.2%	865	936	406
AP5 (PW1_12)				
2	4.2%	1,073	1,118	199
10	6.8%	2,241	2,393	379
100	9.0%	4,046	4,410	645

Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)
		Unbulked	*Bulked*	
AP6 (PW_VRW3)				
2	3.6%	1,138	1,179	207
10	6.0%	2,366	2,508	394
100	8.9%	4,248	4,627	670
AP7 (PW14_VRW3)				
2	6.0%	1,314	1,393	230
10	9.3%	2,730	2,984	435
100	12.9%	4,875	5,504	737
AP8 (VR_TVI_PW)				
2	6.0%	1,392	1,476	241
10	9.4%	2,892	3,164	454
100	10.6%	5,143	5,688	769
AP9 (VR_TVI_PW_SEV)				
2	6.4%	1,410	1,500	245
10	9.4%	2,931	3,206	461
100	10.6%	5,213	5,766	780
AP10 (SWINBURNE_INFLOW)				
2	7.1%	1,471	1,576	295
10	9.8%	3,007	3,302	557
100	13.3%	5,306	6,011	944

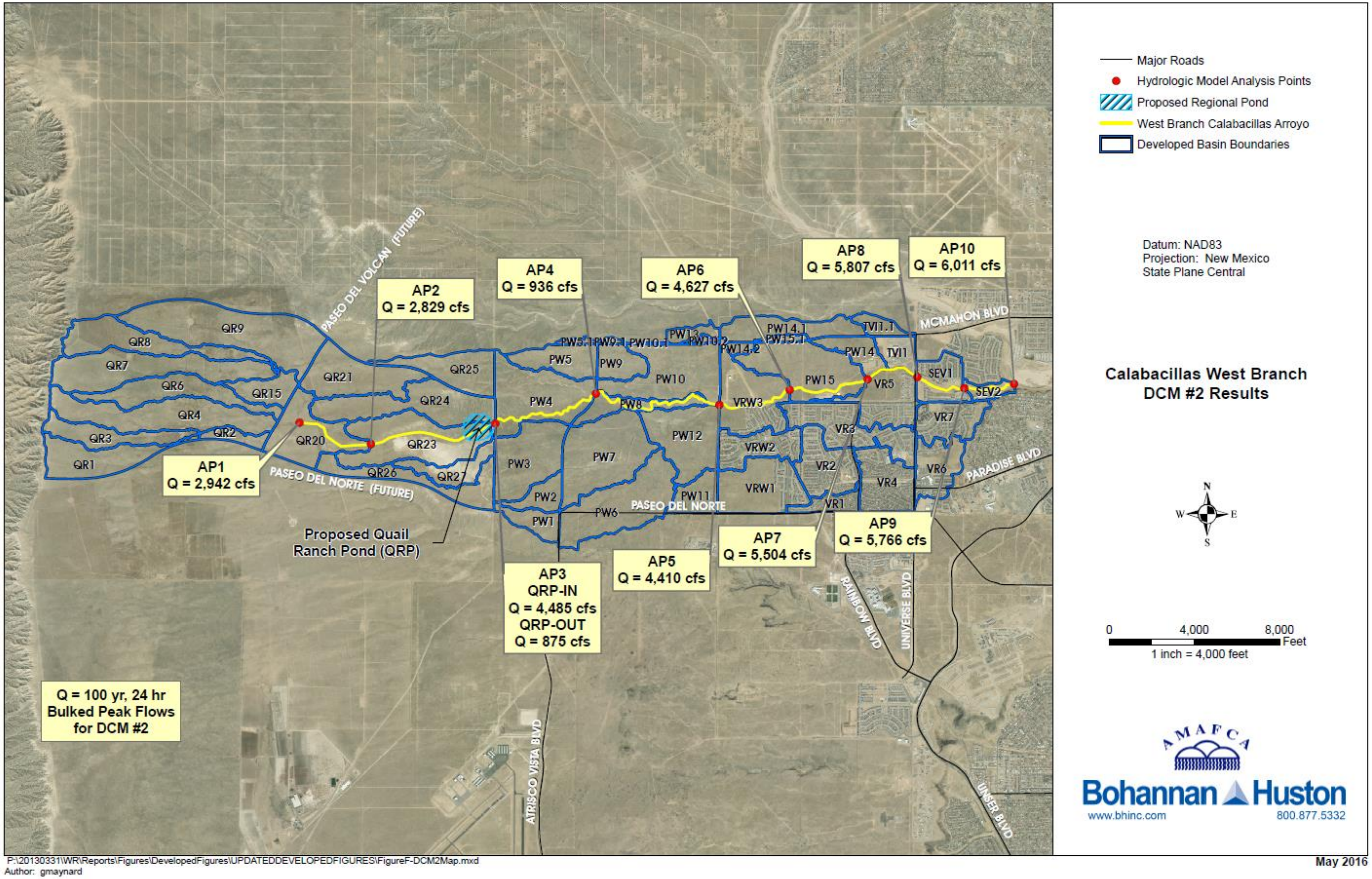

Figure 7 – Calabacillas West Branch DCM #2 Results

c. Developed Conditions Model #3 (DCM #3)

Developed Conditions Model #3 (DCM#3) is similar to DCM#2; however, it involves the addition of the Paradise West Pond (PWP) to the hydrologic model. Although the outflow requirements at Quail Ranch Pond were met in DCM #2, there was too much buildup of runoff downstream of QRP to the extent that existing downstream bridge culverts and drop structures do not have the capacity for the developed conditions flow. Therefore, an additional pond was modeled at an intermediate point between QRP and Swinburne Dam to manage and reduce flows. The PWP addition would be located just upstream of AP5. PWP was sized and configured in such a way that the developed conditions peak 100-yr, 24-hr outflow is less than the controlling existing capacities at these downstream infrastructure locations (3,000 cfs).

Per an original request of AMAFCA, PWP was initially proposed to be a non-jurisdictional pond under the Rules and Regulations of the New Mexico Office of the State Engineer Dam Safety Bureau. However, after the initial hydrologic analysis, it was determined that 50 ac-ft of storage was not enough to reduce downstream flows to the extent required. Therefore, the final pond configuration modeled is jurisdictional and the flows were adequately reduced to meet the downstream structures existing capacities. Paradise West Pond was conceptually modeled as a four-sided conical reservoir: 500-feet by 600-feet surface footprint (6.9 acres), 15-foot height, assumed 4:1 side slopes, and 120 ac-ft of storage. These conceptual pond dimensions do not account for freeboard, spillway or maintenance access requirements. These are conceptual recommendations based on preliminary information and are intended to provide a starting point for actual pond design at a later date.

The DCM #3 results, for 24-hr storms, are summarized in Table J and shown in Figure 8. As seen in Table P, the downstream existing structure design capacities are met with the developed conditions flows from DCM #3.

Table J– Developed Conditions Model #3 (DCM #3) Results

Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)	Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)
		Unbulked	*Bulked*				*Unbulked*	*Bulked*	
AP1 (QR3_15)					**AP5 (PWP_OUT) - Flow out of PWP**				
2	1.9%	759	773	61	2	4.3%	475	496	198
10	2.7%	1,525	1,566	120	10	7.0%	1,070	1,145	377
100	3.6%	2,840	2,942	211	100	9.3%	1,620	1,770	643
AP2 (QR3_20)					**AP6 (PW_VRW3)**				
2	3.2%	791	817	71	2	2.1%	499	509	206
10	4.6%	1,512	1,581	136	10	3.5%	1,122	1,161	393
100	6.8%	2,651	2,832	234	100	4.6%	1,701	1,779	669
AP3 (QR3_24)- Flow Into QRP					**AP7 (PW14_VRW3)**				
2	2.5%	1,228	1,258	116	2	4.2%	572	596	229
10	3.4%	2,395	2,476	222	10	6.0%	1,319	1,398	434
100	4.6%	4,288	4,485	379	100	8.4%	2,127	2,305	736
AP3 (QRP_OUT) - Flow out of QRP					**AP8 (VR_TVI_PW)**				
2	2.5%	301	309	115	2	4.2%	607	633	240
10	3.4%	577	597	221	10	6.0%	1,420	1,505	453
100	4.6%	837	875	378	100	8.4%	2,418	2,621	769
AP4 (QR3_25_PW4)					**AP9 (VR_TVI_PW_SEV)**				
2	3.9%	311	324	125	2	3.8%	615	638	243
10	5.9%	594	629	238	10	6.4%	1,445	1,538	460
100	8.1%	865	935	406	100	8.6%	2,498	2,713	780
AP5 (PW1_12) - Flow into PWP					**AP10 (SWINBURNE_INFLOW)**				
2	4.3%	1,074	1,120	199	2	4.4%	681	711	294
10	7.0%	2,241	2,398	379	10	7.2%	1,529	1,639	556
100	9.3%	4,046	4,422	645	100	9.2%	2,597	2,836	944

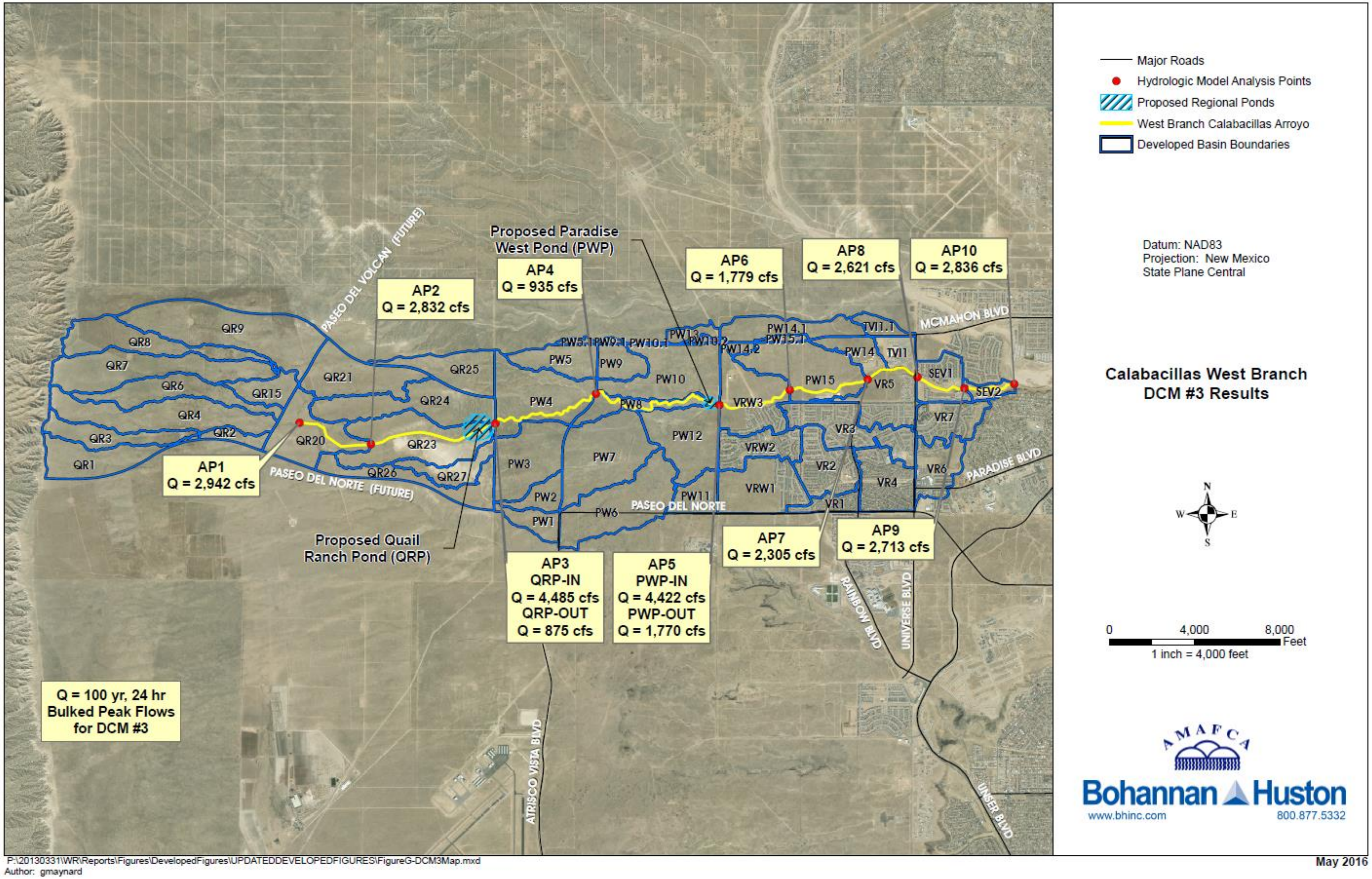

Figure 8 – Calabacillas West Branch DCM #3 Results

d. Developed Conditions Model #4 (DCM #4)

The fourth developed conditions model involves the addition of a single pond at the base of each basin within the watershed, to more closely mimic existing conditions flows for the 100-year, 24-hour storm event throughout the watershed and length of the CWA. In addition to adding the ponds, any basin that was intersected by the main arroyo was split into two new and separate basins (each of which would also have an individual pond). These basins were split to better quantify how much flow is being conveyed from the north and south portions of the basins into the main arroyo channel. There are 61 basins in the DCM #4 model (
Figure **9**). These basins were split to better quantify how much flow is being conveyed from the north and south portions of the basins into the main arroyo channel. The pervious and impervious areas, curve numbers, and lag times were re-calculated to reflect the basin changes.

An updated revised existing conditions model was created to supply peak discharges for the split basins to use in sizing the conceptual basin ponds. Each pond was sized with the intention of reducing each basin's developed conditions 100-year, 24-hour peak discharge down to the 100-year, 24-hour existing conditions peak discharge as determined in the model from the 2016 CWB Existing Conditions Hydrology Report, revised in this analysis for the sub-divided basins. Each pond was modeled as a four-sided conical reservoir with 4:1 side slopes; surface footprints varying from 0.29 acres to 2.6 acres; heights from 1 ft to 10 ft; storage from 1 ac-ft to 28 ac-ft; and various reinforced concrete pipe outlet configurations. These conceptual pond dimensions do not account for freeboard, spillway or maintenance access requirements. These are conceptual recommendations only and are not intended to be used for design. They are intended to provide an estimation of pond volume necessary to reduce developed condition 100-year, 24-hour peak flows. The pond shape and style will be determined during future design by the land owner. When the ponds are being finalized, they can be designed to be deeper and smaller with sloped bottoms to ensure full drainage within 96 hours. Table K – Developed Conditions Model #4 Proposed Pond Volume SummaryTable K below summarizes the proposed pond volumes for each subbasin. A full summary of the pond storage for each subbasin is provided in Appendix H. Note that for 10 basins, located in the eastern portion of the watershed where existing conditions are the same as developed conditions, ponds were not needed because the developed conditions peak flowrate equals the existing conditions peak flowrate.

Though the small ponds in each basin function to reduce the peak 100-year, 24-hour discharge from each basin to mimic the existing conditions peak discharges, the developed conditions peak flows and volumes in the CWB arroyo, as expected, are higher than in existing conditions. There are several reasons for the increased runoff peak and volume including:

1) Added basins in developed conditions - the developed conditions model has additional area of 0.785 sq. mi. (see Table C) resulting in increased runoff, as compared to the existing conditions model;
2) Split several basins along the main arroyo into smaller basins – splitting basins into smaller areas typically increases the peak runoff rate in hydrologic models; this change alone increased the existing conditions peak flow at Swinburne Dam by 125 cfs, roughly a 9 percent increase.
3) Added runoff volume due to increased impervious area - in developed conditions, there is increased impervious area, resulting in less initial abstraction, less precipitation loss volume, and increased direct runoff volume.
4) Different time of peak and basin routing –the routing in the hydrologic models also impacts the peak flows. The developed conditions model assumed lower manning's n values in some of the developed routes, resulting in less flow attenuation and higher peak flows. In addition, the routing

in developed conditions combines basins such that the time of peak flows are more coincidental, resulting in higher peak flows in the main arroyo.

Table K – Developed Conditions Model #4 Proposed Pond Volume Summary

AP	Basin/Pond ID	Proposed Pond Volume (ac-ft)	AP	Basin/Pond ID	Proposed Pond Volume (ac-ft)
	QR9	28		PW8S1	3
	QR7	16		PW8S2	2
	QR8	11		PW10.2	1
	QR4	20		PW10.1	1
	QR6	12	AP5	PW1_12	
	QR1	11	AP6	PW_VRW3	
	QR3	8		VRW3N	8
	QR2	4		VRW3S	4
	QR15	5		PW15N	21
AP1	QR3_15			PW15S	3
	QR20N	7		PW14.1	11
	QR20S	13		PW13	3
AP2	QR3_20			PW14.2	2
	QR23N	8		PW14	6
	QR23S	8		PW15.1	2
	QR24	19	AP7	PW14_VRW3	
	QR21	12		VR5	7
	QR26	13		TVI1	2
	QR25	10		TVI1.1	4
	QR27	5	AP8	VR_TVI_PW	
AP3-QRP	QR3_24			SEV1N	None Ex = Dev
	PW4N	18		SEV1S	1
	PW4S	3	AP9	VR_TVI_PW_SEV	
AP4	QR3_25_PW 4			VRW1	12
	PW3	12		PW11	2
	PW2	4		VRW2	None Ex = Dev
	PW7	23		VR2	None Ex = Dev
	PW6	18		VR4	None Ex = Dev
	PW1	5		VR1	None Ex = Dev
	PW10	22		VR3	None Ex = Dev
	PW12	15		VR6	None Ex = Dev

AP	Basin/Pond ID	Proposed Pond Volume (ac-ft)
	PW5	11
	PW5.1	1
	PW9	6
	PW9.1	1
	PW8N	2

AP	Basin/Pond ID	Proposed Pond Volume (ac-ft)
	Las Ventanas Dam	
	VR7	None Ex = Dev
	Little Window Dam	
	SEV2	None Ex = Dev
AP10	SWINBURNE_INFLOW	

The DCM #4 results, for 24-hr storms, are summarized in Table M and shown in Figure 9. As seen in Table O, the downstream existing structure design capacities met with the developed conditions flows from DCM #3.

Table L – Developed Conditions Model #4 (DCM #4) Results

Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)		Event	Bulking Factor	Peak Discharge (cfs)		Runoff Volume (ac-ft)
		Unbulked	*Bulked*					*Unbulked*	*Bulked*	
AP1 (QR3_15)						**AP6 (PW_VRW3)**				
2	1.8%	195	199	60		2	2.2%	582	595	204
10	2.7%	416	427	118		10	4.0%	1,320	1,372	393
100	3.6%	638	661	209		100	5.6%	2,151	2,272	674
AP2 (QR3_20)						**AP7 (PW14_VRW3)**				
2	1.4%	218	221	71		2	4.2%	616	642	226
10	2.3%	473	484	139		10	6.6%	1,388	1,479	433
100	3.0%	723	744	243		100	8.6%	2,254	2,448	740
AP3 (QR3_24)- QRP						**AP8 (VR_TVI_PW)**				
2	2.4%	343	351	114		2	4.3%	644	672	237
10	4.1%	760	791	222		10	6.7%	1,445	1,541	452
100	5.5%	1,179	1,244	386		100	8.8%	2,356	2,563	772
AP4 (QR3_25_PW4)						**AP9 (VR_TVI_PW_SEV)**				
2	3.9%	350	364	124		2	4.3%	648	676	240
10	5.9%	771	817	239		10	6.4%	1,452	1,545	459
100	9.5%	1,193	1,306	413		100	8.1%	2,370	2,562	783
AP5 (PW1_12)						**AP10 (SWINBURNE_INFLOW)**				
2	3.0%	561	578	196		2	4.7%	712	745	291
10	4.9%	1,270	1,332	377		10	6.9%	1,537	1,643	555
100	6.6%	2,059	2,195	648		100	8.8%	2,472	2,690	947

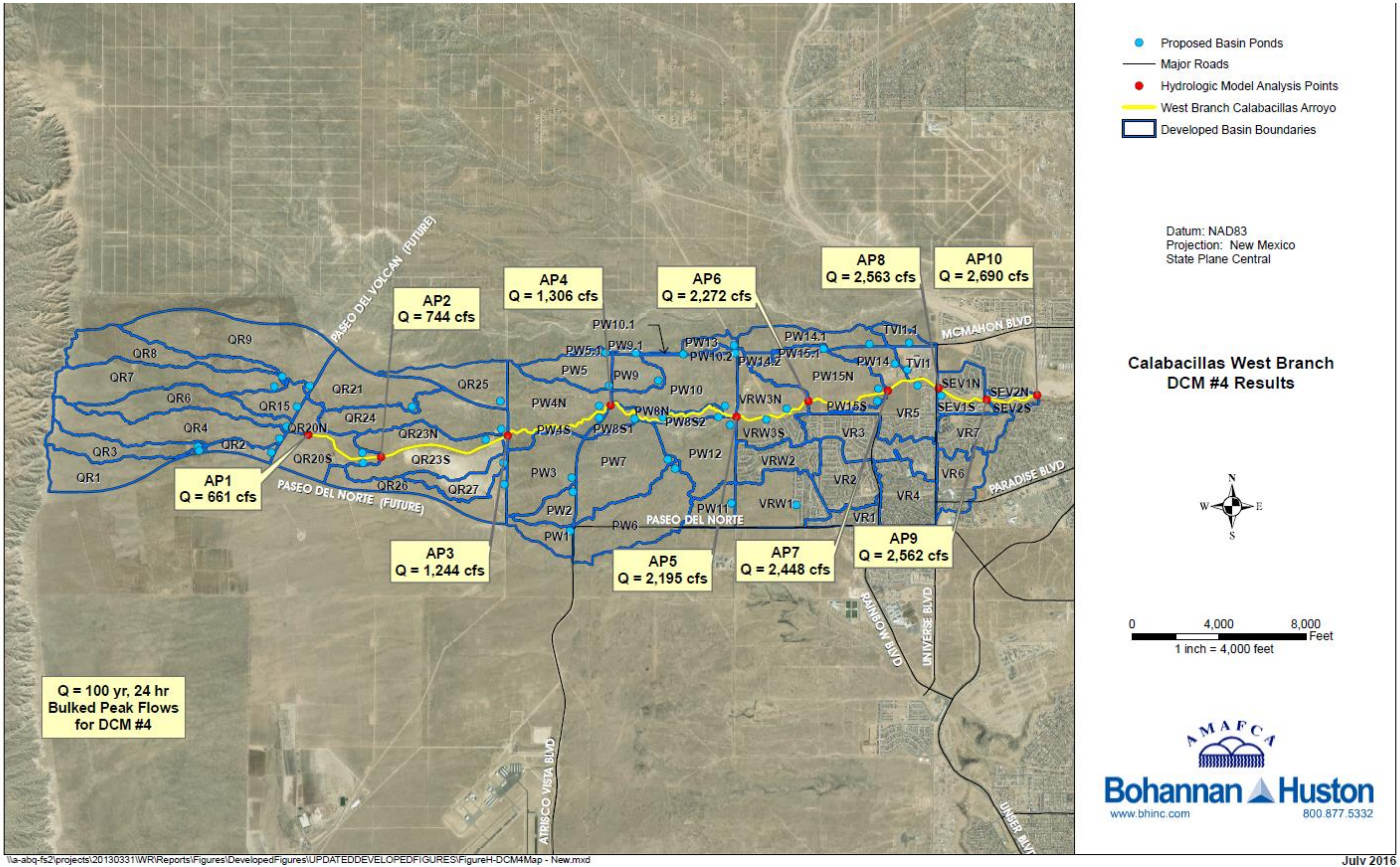

Figure 9 – Calabacillas West Branch DCM #4 Results

e. Summary of Bulking Factor

Bulking factors for all developed conditions models are presented in Table M.

Table M – Bulking Factors

Analysis Reach	Event	Bulking Factor			
		DCM #1	*DCM #2*	*DCM #3*	*DCM #4*
QR3_15	2-year	1.8%	1.8%	1.9%	1.8%
	10-year	2.7%	2.7%	2.7%	2.7%
	100-year	3.6%	3.6%	3.6%	3.6%
QR3_20	2-year	3.2%	3.2%	3.2%	1.4%
	10-year	4.6%	4.6%	4.6%	2.3%
	100-year	6.8%	6.7%	6.8%	3.0%
QR3_24	2-year	6.1%	2.5%	2.5%	2.4%
	10-year	10.0%	3.5%	3.4%	4.1%
	100-year	13.3%	4.7%	4.6%	5.5%
QR3_25_ PW4	2-year	11.1%	4.0%	3.9%	3.9%
	10-year	17.7%	6.0%	5.9%	5.9%
	100-year	22.4%	8.4%	8.1%	9.5%
PW1_12	2-year	6.0%	4.3%	4.3%	3.0%
	10-year	8.4%	6.8%	7.0%	4.9%
	100-year	12.1%	9.1%	9.3%	6.6%
PW_ VRW3	2-year	4.7%	3.6%	2.1%	2.2%
	10-year	8.1%	6.0%	3.5%	4.0%
	100-year	11.9%	9.0%	4.6%	5.6%
PW14_ WRW3	2-year	6.9%	6.0%	4.2%	4.2%
	10-year	10.6%	9.4%	6.0%	6.6%
	100-year	16.1%	13.1%	8.4%	8.6%
VR_TVI _PW	2-year	6.9%	6.0%	4.2%	4.3%
	10-year	10.6%	9.4%	6.0%	6.7%
	100-year	16.1%	13.1%	8.4%	8.8%
VR_TVI_ PW_SEV	2-year	7.5%	6.4%	3.8%	4.3%
	10-year	10.8%	9.3%	6.4%	6.4%
	100-year	13.1%	10.7%	8.6%	8.1%
SWINBURNE _INFLOW	2-year	8.1%	7.1%	4.4%	4.7%
	10-year	11.1%	9.8%	7.2%	6.9%
	100-year	14.9%	13.4%	9.2%	8.8%

f. Flow and Volume Comparisons to 1999 AHYMO Model

The 2016 developed conditions HEC-HMS hydrology is compared to the 1999 developed conditions AYHMO hydrology from the 1999 Prudent Line Study in Table N. At Swinburne Dam, the 1999 100-yr, 6-hr peak flow was 4,950 cfs (5,450 cfs bulked), and the volume was 680 ac-ft. The 2016 HEC-HMS developed conditions model #1 (DCM#1), which does not have any new detention ponds modeled, predicts a peak 100-yr, 6-hr flowrate of 6,653 cfs (7,645 cfs bulked) and a volume of 765 ac-ft and a peak 100-yr, 24-hr flowrate of 6,951 cfs (7,987 cfs bulked) and a volume of 941 ac-ft at the same location.

Table N– Developed Conditions 100-year Peak Flow and Volume Comparisons
1999 AHYMO Model Compared to 2016 HEC-HMS Results

1999 Developed Condtions Analysis Points AHYMO (6-hr. storms)				2016 Developed Condtions Model #1 HEC-HMS AMAFCA 2014 "White Paper" Methodology (6-hr. storms)				2016 Developed Condtions Model #1 HEC-HMS AMAFCA 2014 "White Paper" Methodology (24-hr. storms)			
BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)	BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)	BULKING FACTOR*	Peak Discharge (cfs)		Runoff Volume (ac-ft.)
	Unbulked	Bulked			Unbulked	Bulked			Unbulked	Bulked	
Concentration Point 4				AP2 (QR3_20)				AP2 (QR3_20)			
10.2%	2,560	2,820	184	6.8%	2,531	2,703	187	6.8%	2,651	2,832	234
Concentration Point 2				AP4 (QR3_25_PW4)				AP4 (QR3_25_PW4)			
10.0%	2,990	3,290	258	22.4%	4,191	5,130	327	22.4%	4,381	5,363	406
Concentration Point 1				AP5 (PW1_12)				AP5 (PW1_12)			
10.1%	4,460	4,910	465	12.1%	5,824	6,528	521	12.1%	6,085	6,821	643
Concentration Point L				AP9 (VR_TVI_PW_SEV)				AP9 (VR_TVI_PW_SEV)			
10.0%	4,810	5,290	551	13.1%	6,556	7,415	631	13.1%	6,853	7,750	776
Concentration Point 0				AP10 (SWINBURNE_INFLOW)				AP10 (SWINBURNE_INFLOW)			
10.1%	4,950	5,450	680	14.9%	6,653	7,645	765	14.9%	6,951	7,987	941

The increase in peak runoff and volume as compared to the AHYMO model results in the 1999 Prudent Line Study can be contributed to several factors, including:

- The use of a different hydrologic modeling software, AHYMO was used in 1999 and HEC-HMS was utilized for the current study;
- The use of different rainfall distribution used by the two models;
- The current study includes an additional 973 acres (1.52 sq. mi.) added based on the master plan layouts for Quail Ranch and Paradise West;
- The use of smaller, more detailed basins in the current model (52 basins) compared to 7 basins in the 1999 model. Modeling additional basins in hydrologic models typically results in higher flows. The additional basins were required to provide better planning and options analysis in the watershed; and
- The land treatment assumption of higher concentration in developed conditions than assumed in the 1999 model. The 1999 Prudent Line Study assumed single family residential with an average of 4 dwelling units per acre (DU/ac), while the current development (Las Ventanas and Seville), as well as the master plans for Quail ranch and Paradise West, have areas with 5 to 6 DU/ac.

As HEC-HMS is now the AMAFCA-accepted hydrologic modeling method, further analysis of the reasons for the differences between the 1999 and 2016 model results contained in this study in terms of flows and volumes within the CWB watershed is not part of this project's scope.

g. HEC-RAS Model Comparison

Tetra Tech developed a new HEC-RAS model incorporating the 2010 LiDAR mapping and new structures and bank protection that have been constructed since the 1999 model was developed. The new model was set up to overlap cross sections from the 1999 model as often as was reasonable. Some cross sections did not overlap between the two models because of arroyo realignment or the construction of grade control structures and road crossings. The model was expanded approximately two miles upstream. The 2-year, 10-year, and 100-year 6-hour peak flows were simulated with the 2015 and 1999 model (MEI 1999). The 2-year and 10-year events are shown because they have an appreciable impact on sediment transport. Hydraulic results were averaged by sub reach and compared. The models are in digital form in Appendix F.

The comparisons show both upward and downward changes in all hydraulic parameters and confirm the need for updated hydraulic modeling for the developed conditions hydrologic modeling and the subsequent equilibrium slope analyses, prudent line computations, and reach-specific sediment bulking factors. This analysis was performed for all scenarios of the developed conditions hydrologic and hydraulic modeling.

h. Structure Capacity vs. Flow

As would be expected, some of the existing structures design capacities are exceeded by the 100-year, 24-hour developed conditions peak flows for DCM #1 and DCM #2 as shown in Table O. However, the existing structures have adequate capacity to convey the 100-year, 24-hour developed conditions peak flows for DCM #3 and DCM #4 as shown in Table P.

It is noted that GCS-5, GCS-6, and GCS-7 were designed for less than the 1999 Prudent Line Study developed conditions flow of 4,900 cfs. Per negotiations with the developer, each was sized for 3,000 cfs, which was more than two times the then existing (1999) 100-yr event of 1,440 cfs, and well over the ten year developed conditions event of 2,310 cfs. This approach was justified by the regulatory agencies, considering the risk and consequence of failure, as well as the dedication of right of way to AMAFCA and the City of Albuquerque in excess of the 1999 prudent limits.

It is also noted that the Field Reconnaissance Report identifies GCS-9 as the failed structure. Failure of the structure was not due to exceedance of the design inflow, but rather the location of a subsequently constructed storm drain outfall that undermined the downstream sill of the structure.

Table O – DCM #1 & DCM #2 Results Compared to Existing Structure Capacities

		DCM #1 (No new Infrastructure)			DCM #2 (QRP Only)				DCM #1	DCM #2
Analysis Point	**HEC-HMS Hydrologic Element - Description**	**Unbulked Q_{100} (cfs)**	**Bulked Q_{100} (cfs)**	**Volume (ac-ft)**	**Unbulked Q_{100} (cfs)**	**Bulked Q_{100} (cfs)**	**Volume (ac-ft)**	**Structure Design Q (cfs)**	**Structure Capacity Handles Q_{100}?**	**Structure Capacity Handles Q_{100}?**
AP3	QR3_24 - QRP in	4,288	4,855	379	4,288	4,485	379			
	QRP out				837	875	378			
AP4	QR3_25_PW4	4,381	5,363	406	865	936	406			
AP5	PW1_12 - PWP	6,085	6,821	643	4,046	4,410	645			
AP6	PW_VRW3	6,217	6,957	668	4,248	4,627	670			
AP7	PW14_VRW3	6,619	7,685	734	4,875	5,504	737			
AP8	VR_TVI_PW – Universe Xing	6,802	7,897	766	5,143	5,807	769	4,700	Too Small	Too Small
Universe GCS	Riprap Drop Structure	6,802	7,897		5,143	5,807		2,900	Too Small	Too Small
Conc. Channel	Univ Plaza, 40' w. conc channel	6,802	7,897		5,143	5,807		5,300	Too Small	Too Small
GCS - 10	Univ Plaza, Riprap/conc. GCS	6,802	7,897		5,143	5,807		5,300	Too Small	Too Small
GCS - 9	Riprap Grade Control Structure	6,802	7,897		5,143	5,807		2,900	Too Small	Too Small
GCS - 8	Riprap Grade Control Structure	6,802	7,897		5,143	5,807		2,900	Too Small	Too Small
AP9	VR_TVI_PW_SEV – Kayenta Xing	6,853	7,750	776	5,213	5,766	780	4,800	Too Small	Too Small
Kayenta GCS	Riprap Grade Control Structure	6,853	7,750		5,213	5,766		3,000	Too Small	Too Small
GCS - 6	Riprap Grade Control Structure	6,853	7,750		5,213	5,766		3,000	Too Small	Too Small
GCS - 5	Riprap Grade Control Structure	6,853	7,750		5,213	5,766		3,000	Too Small	Too Small
GCS -4	Soil cement GC structure	6,951	7,987		5,306	6,011		5,450	Too Small	Too Small
GCS -3	Soil cement GC structure	6,951	7,987		5,306	6,011		5,450	Too Small	Too Small
GCS-2	Soil cement GC structure	6,951	7,987		5,306	6,011		5,450	Too Small	Too Small
GCS -1	Soil cement GC structure	6,951	7,987		5,306	6,011		5,450	Too Small	Too Small
AP10	SWINBURNE_INFLOW	6,951	7,987	941	5,306	6,011	944			

Table P – DCM #3 & DCM #4 Results Compared to Existing Structure Capacities

		DCM #3 (QRP & PWP)			DCM #4 (Individual Basin Ponds)				DCM #3	DCM #4
Analysis Point	**HEC-HMS Hydrologic Element - Description**	**Unbulked Q_{100} (cfs)**	**Bulked Q_{100} (cfs)**	**Volume (ac-ft)**	**Unbulked Q_{100} (cfs)**	**Bulked Q_{100} (cfs)**	**Volume (ac-ft)**	**Structure Design Q (cfs)**	**Structure Capacity Handles Q_{100}?**	**Structure Capacity Handles Q_{100}?**
AP3	QR3_24 - QRP in	4,288	4,485	379	1,179	1,244	386			
	QRP out	837	875	378						
AP4	QR3_25_PW4	837	875	378	1,193	1,306	413			
AP5	PW1_12 - PWP in	4,046	4,442	645	2,059	2,195	648			
	PWP out	1,620	1,770	643						
AP6	PW_VRW3	1,701	1,779	669	2,151	2,272	674			
AP7	PW14_VRW3	2,127	2,305	736	2,254	2,448	740			
AP8	VR_TVI_PW - Universe Xing	2,418	2,621	769	2,356	2,563	772	4,700	OK	OK
Universe GCS	Riprap Drop Structure	2,418	2,621		2,356	2,563		2,900	OK	OK
Conc. Channel	Univ Plaza, 40' w. conc channel	2,418	2,621		2,356	2,563		5,300	OK	OK
GCS - 10	Univ Plaza, Riprap/conc. GCS	2,418	2,621		2,356	2,563		5,300	OK	OK
GCS - 9	Riprap Grade Control Structure	2,418	2,621		2,356	2,563		2,900	OK	OK
GCS - 8	Riprap Grade Control Structure	2,418	2,621		2,356	2,563		2,900	OK	OK
GCS - 7	Riprap Grade Control Structure	2,418	2,621		2,356	2,563		2,900	OK	OK
AP9	VR_TVI_PW_SEV – Kayenta Xing	2,498	2,713	780	2,370	2,562	783	4,800	OK	OK
Kayenta GCS	Riprap Grade Control Structure	2,498	2,713		2,370	2,562		3,000	OK	OK
GCS - 6	Riprap Grade Control Structure	2,498	2,713		2,370	2,562		3,000	OK	OK
GCS - 5	Riprap Grade Control Structure	2,498	2,713		2,370	2,562		3,000	OK	OK
GCS -4	Soil cement GC structure	2,597	2,836		2,472	2,690		5,450	OK	OK
GCS -3	Soil cement GC structure	2,597	2,836		2,472	2,690		5,450	OK	OK
GCS-2	Soil cement GC structure	2,597	2,836		2,472	2,690		5,450	OK	OK
GCS -1	Soil cement GC structure	2,597	2,836		2,472	2,690		5,450	OK	OK
AP10	SWINBURNE_INFLOW	2,597	2,836	944	2,472	2,690	947			

4.0 PRUDENT LINE DEVELOPMENT

The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) contracted Tetra Tech to develop the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Master Plan, a portion of which is to develop three sets of prudent lines adjacent to the CWB. Prudent lines define the boundary beyond which the risk of development is considered acceptably low. The boundary is determined by calculating 1) the arroyo's potential to widen, laterally migrate or erode (based on computations across a suite of hydrologic events between the two-year and 100-year recurrence interval), and 2) the flood extents of a bulked 100-year recurrence interval peak flow. Prudent lines were not developed for DCM #4.

The most current methodology to develop prudent lines for arroyos in the Albuquerque area is described in the Southern Sandoval County Arroyo Flood Control Authority's (SSCAFCA) Sediment and Erosion Design Guide (MEI 2008). The hydrology described in Section 3.4 was used as input to the prudent line analyses. The steps below outline the methodology applied to create the prudent lines.

1. Extend Existing Condition Hydraulic Model. An existing conditions hydraulics model was developed for an earlier phase of this work using HEC-RAS (Tetra Tech 2013). This model was extended upstream from station 355+00 (approximately one mile upstream of the Quail Ranch pond) to station 477+90 (in the CWB's headwaters). This expanded the model's capability to simulate developed conditions hydraulics.
2. Compute Hydraulic Characteristics of the Arroyo. Peak flows, including the two-year, five-year, 10-year, 25-year, 50-year, and 100-year recurrence interval events, from DCM #1, DCM #2, and DCM #3 hydrology were input into the extended HEC-RAS model. Modeled flows were not bulked to account for sediment transport because sediment load capacity relationships could not be calculated before this step. The model results were used to calculate reach-averaged hydraulic parameters for all the flood events from two-year to 100-year. These reaches match the reaches used in the previous prudent line study (MEI 1999) with the extended portion of the model appended as a new reach as shown in Table Q:

Table Q – Reach Boundaries Used to Calculate Reach-averaged Hydraulic Parameters

Reach		**Upstream Station**	**Upstream HEC-RAS Station**	
1999	*2015*		*1999*	*2015*
	0	477+90		47789.72
0	1	358+67	76	35867.14
1	2	294+69	67	29468.52
2	3	233+06	52	23305.7
3	4	190+73	40	19072.78
4	5	153+60	32	15360.13
5	6	111+02	25	11102.16
6	7	65+33	17	6533.169
7	8	36+01	16	3600.664
8	9	10+99	4	1099.06

3. Compute Total Sediment Load Capacity Relationship Using Reach-Averaged Hydraulics. This was done in two steps. The first was to calculate wash load capacity for each reach and hydrologic event. The wash load is the portion of the sediment load that remains suspended in the flow, comes from the watershed, and is composed largely of silt and clay particles. Wash load capacity was calculated using the Modified Universal Soil Loss Equation (MUSLE) (Williams and Berndt 1972). The second step was to calculate the bed material load capacity, which is the sediment load that moves along the bed or in suspension, comes from the arroyo bed, and is sand-sized or coarser. The MPM-Woo equation was used to calculate the bed material load capacity for all reaches and

hydrologic events. This same equation was used to calculate bed material load capacity in the previous prudent line study (MEI 1999). The wash load and bed material load capacities were summed for each hydrologic event to create a total sediment load capacity relationship for each reach.

4. Compute Total Sediment Yield for each Hydrologic Event and Reach. Sediment loading capacity relationships computed in step three were applied to calculate the total sediment yield from each reach for each hydrologic event by integrating the sediment load across the hydrograph for each hydrologic event. This step assumes that the arroyo is capacity limited (i.e., there is a sufficient sediment supply for the arroyo to transport its sediment load capacity).
5. Compute Dominant Discharge and Width for each Reach. The dominant discharge is the discharge which carries the most sediment over a long period of time, and the dominant width is the channel width corresponding to the dominant discharge. Dominant discharge is a function of sediment yields and corresponding peak flows of each hydrologic event. The dominant width is a function of dominant discharge. Dominant width and discharge were calculated for each reach.
6. Compute Equilibrium Slopes and Bank Heights. A reach-averaged equilibrium slope is calculated by reach and is a function of the existing slope, the sediment yield, and the sediment transport capacity through a reach. Existing reach-averaged slopes were computed using topography information contained in the extended HEC-RAS model. The sediment yields and transport capacities were computed in step three. The reach-averaged equilibrium slope was applied to actual existing slopes (i.e., not reach-averaged) to compute long-term changes in bed elevation. Future bank heights were computed for each bend as the sum of current bank height and long-term bed elevation change (note: degradation was considered a negative change in bed elevation).
7. Compute Lateral Erosion Envelope (LEE) Extents for each Bend. The LEE defines the maximum potential lateral erosion of the arroyo at a single location. Maximum lateral erosion is the sum of lateral erosion from channel widening, bank erosion, and bend migration. Channel widening is a computed as the difference between current channel width and dominant width. Bank erosion is a function of current bank height, future bank height, and critical bank height. Bank erosion occurs when the bank height exceeds the critical bank height and is constrained when features to prevent bank erosion are in place (e.g., rip rap, grade control structure, etc.). The LEE analysis considers bend migration to be a function of future bend geometry, which is a function of dominant discharge and width. If features are in place to prevent bend migration, the bend migration is constrained. Finally, the bend sharpness (defined as radius of curvature of the bend divided by width of the arroyo) will typically adjust to a value of two (MEI 2008). MEI (2010) presents a relationship for constraining the maximum bend migration distance based on bend sharpness, which they applied to the Las Huertas Creek near Placitas, New Mexico. All these potential sources of lateral erosion are summed after constraints are applied to create the LEE.
8. Compute Bulking Factors and Bulked Peak Flows. Based on the total sediment load capacity relationships computed during step three, bulking factors were calculated for each hydrologic event and reach. The bulking factors were applied to calculate bulked peak flows for each hydrologic event and reach.
9. Delineate Bulked 100-year Recurrence Interval Flood Extents. The 100-year recurrence interval peak flows were inputted to the extended HEC-RAS model. The simulated water surface elevation was used to delineate flood extents.
10. Create Prudent Line. The prudent line was created to follow the extents of one of the following two items based on whichever was furthest from the arroyo: the LEE or the 100-year bulked flood extents. If both the LEE and 100-year bulked flood extents were less than one dominant width away from the top of the arroyo bank (typically 50 feet to 100 feet depending upon the reach) without a feature to prevent bank migration, the prudent line was set at one dominant width away from the top of the arroyo bank.

4.1 DCM #1 PRUDENT LINES

The DCM #1 prudent line is presented in Figure 10 to Figure 16. The prudent line was created following the steps outlined in Section 4.0. Some additional revisions were made to the prudent line using engineering judgement. There are some locations where the DCM #1 prudent line varies appreciably from the 1999 prudent line. These revisions and differences are described in Table R. In the "Side" column of Table R, left and right are from the perspective of someone looking upstream.

The limits of the DCM #1 Prudent Line are from the Swinburne Dam Reservoir up to nearly the top of the watershed at Sta 355+00.

Table R – Summary of Prudent Line Revisions and Appreciable Differences from DCM #1 to 1999

Station	Side	Comment
60+00 to 76+10	Left	The DCM #1 bulked, 100-year flood extents cannot be accurately simulated by HEC-RAS because of one-dimensional, steady-flow model limitations (Figure 16). It is likely that the flow will spill to overland flow and follow the red arrow in Figure 16 if the peak, bulked, 100-year flow persists for long enough to spill (steady-flow water surface elevations may be conservative). This resulted in the prudent line initially passing through part of the Cantacielo subdivision. However, an estimate suggests that only 11 cfs to 17 cfs may spill, resulting in depths of 0.1 feet to 0.3 feet reaching the subdivision if there is no loss of flow to overbank infiltration and peak flows persist for long enough to produce enough volume of flow to reach the subdivision. Additionally, this assumes that DCM #1 flows passes through the existing channel which is expected to degrade as watershed develops. In summary, the neighborhood appears safe even if DCM #1 flows occur now, and there is not likely to be spill of flow under future degraded conditions.
104+00	Left	The DCM #1 prudent line initially overlapped part of the Prickly Brush Subdivision. It is assumed that AMAFCA will perform maintenance of the bank and / or storm water quality pond at this location to alleviate any threats to the subdivision prior to damage occurring.
129+00	Left	The DCM #1 prudent line initially overlapped four houses in the Ventana Ranch West Subdivision. It is assumed that AMAFCA will perform maintenance of the bank and / or storm water quality ponds at this location to alleviate any threats to the subdivision prior to damage occurring.
225+00	Right	Increased flows and improved topographic data result in a large slack-water (or ponded water) area in the floodplain. No structures impacted.
235+00	Left	Increased flows and improved topographic data result in a large slack-water (or ponded water) area in the floodplain. No structures impacted.
295+00	Left	Flood extents increased appreciably because of the construction of the Quail Ranch Pond.

GCS #5 and #6 were designed to contain a flow of 3,000 cfs. The bulked 100-year peak flow at these GCSs is 7,749 cfs. It was therefore assumed that these structures would fail and not provide protection from lateral migration. GCS #7 was in the process of being flanked during the field reconnaissance. This GCS was also assumed to fail and not considered in the prudent line analysis. GCS #8 was designed to contain a flow of 5,300 cfs. The bulked 100-year peak flow at these GCSs is 7,896 cfs. It was therefore assumed that these structures would fail and not provide protection from lateral migration. GCS #9 was in the process of being undercut during the field reconnaissance. This GCS was assumed to fail and not considered in the prudent line analysis.

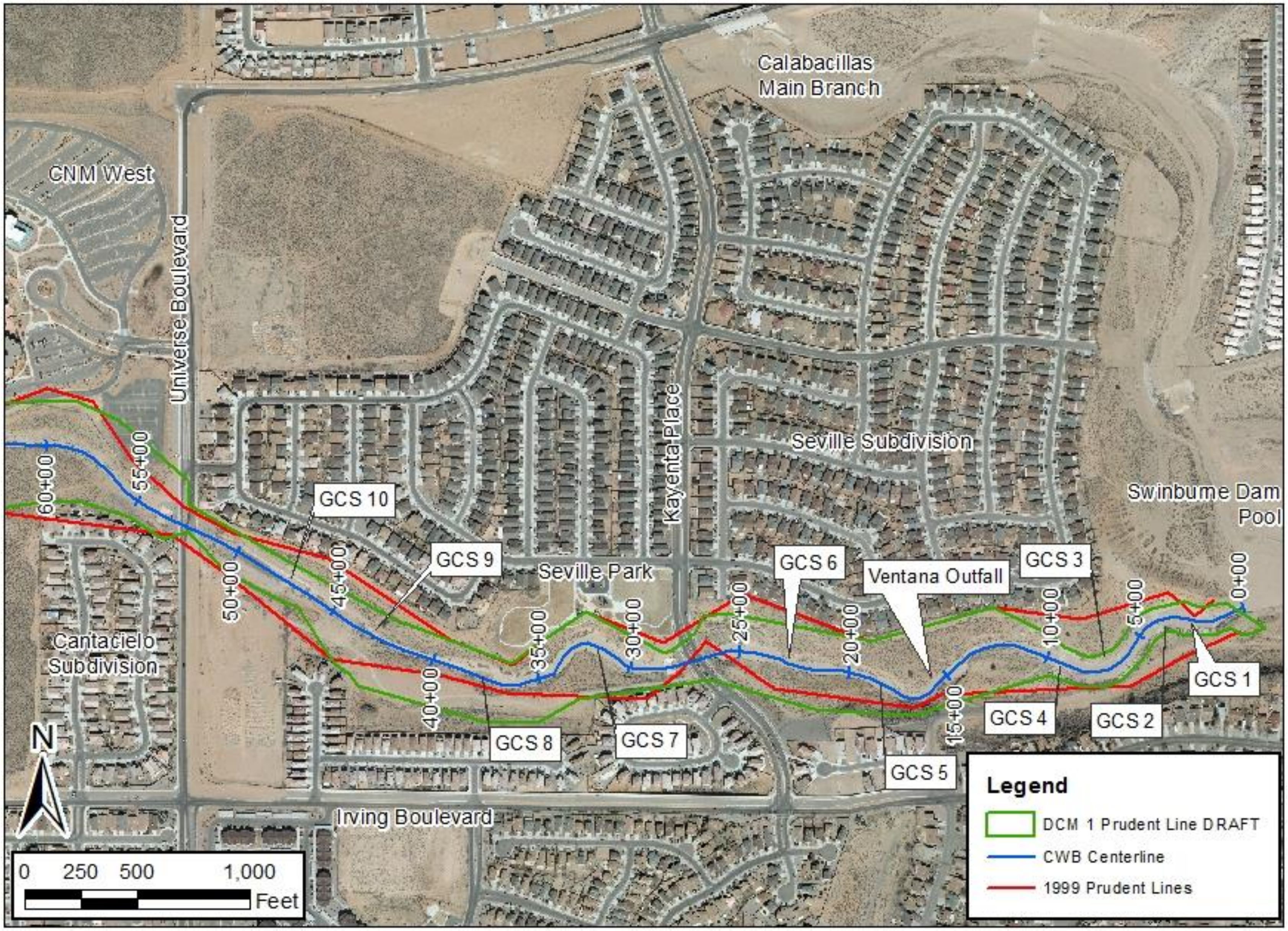

Figure 10 – DCM #1 and 1999 Prudent Lines from Swinburne Dam to Station 60+00.

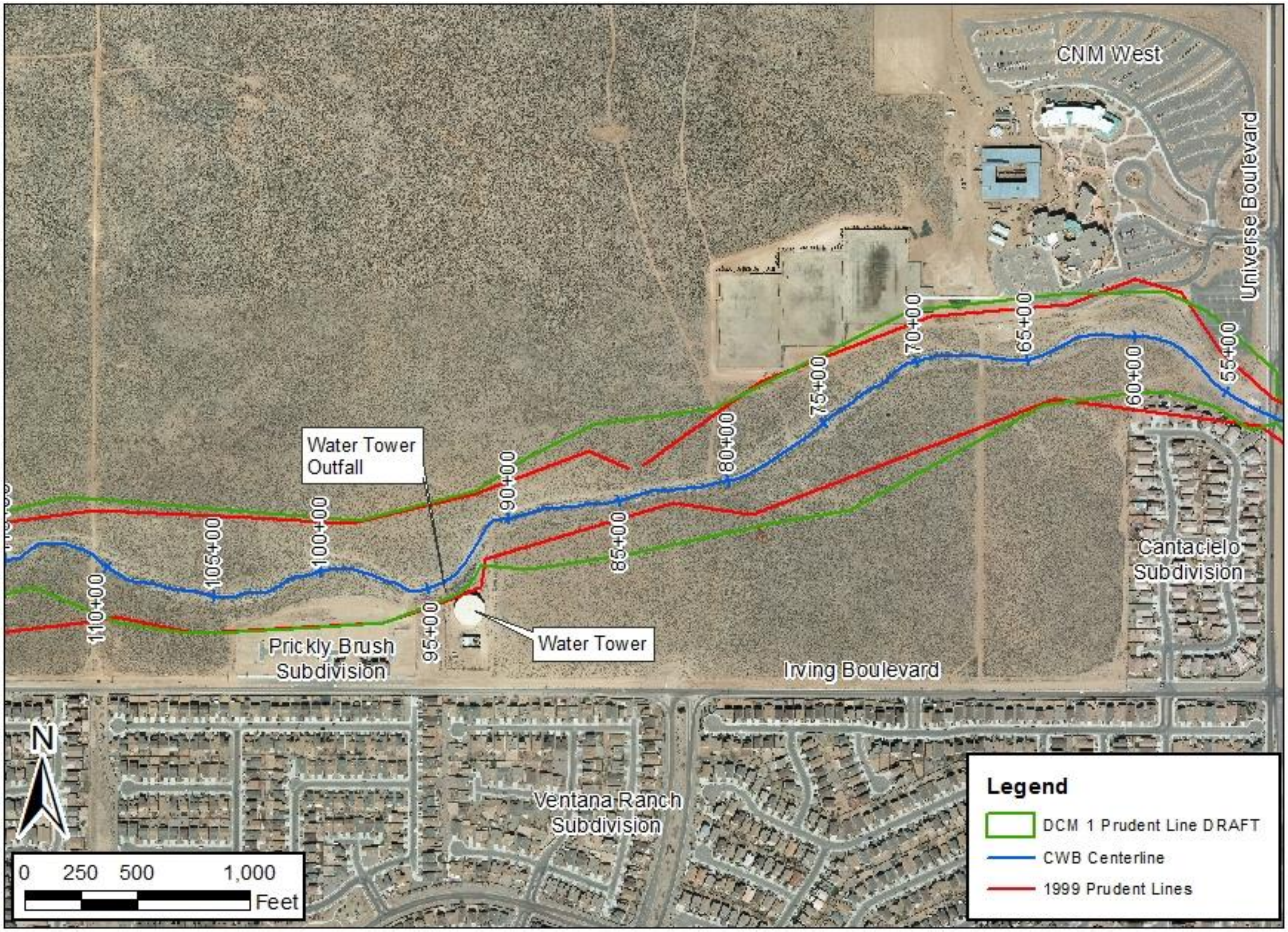

Figure 11 – DCM #1 and 1999 Prudent Lines from Station 55+00 (Universe Boulevard) to Station 110+00.

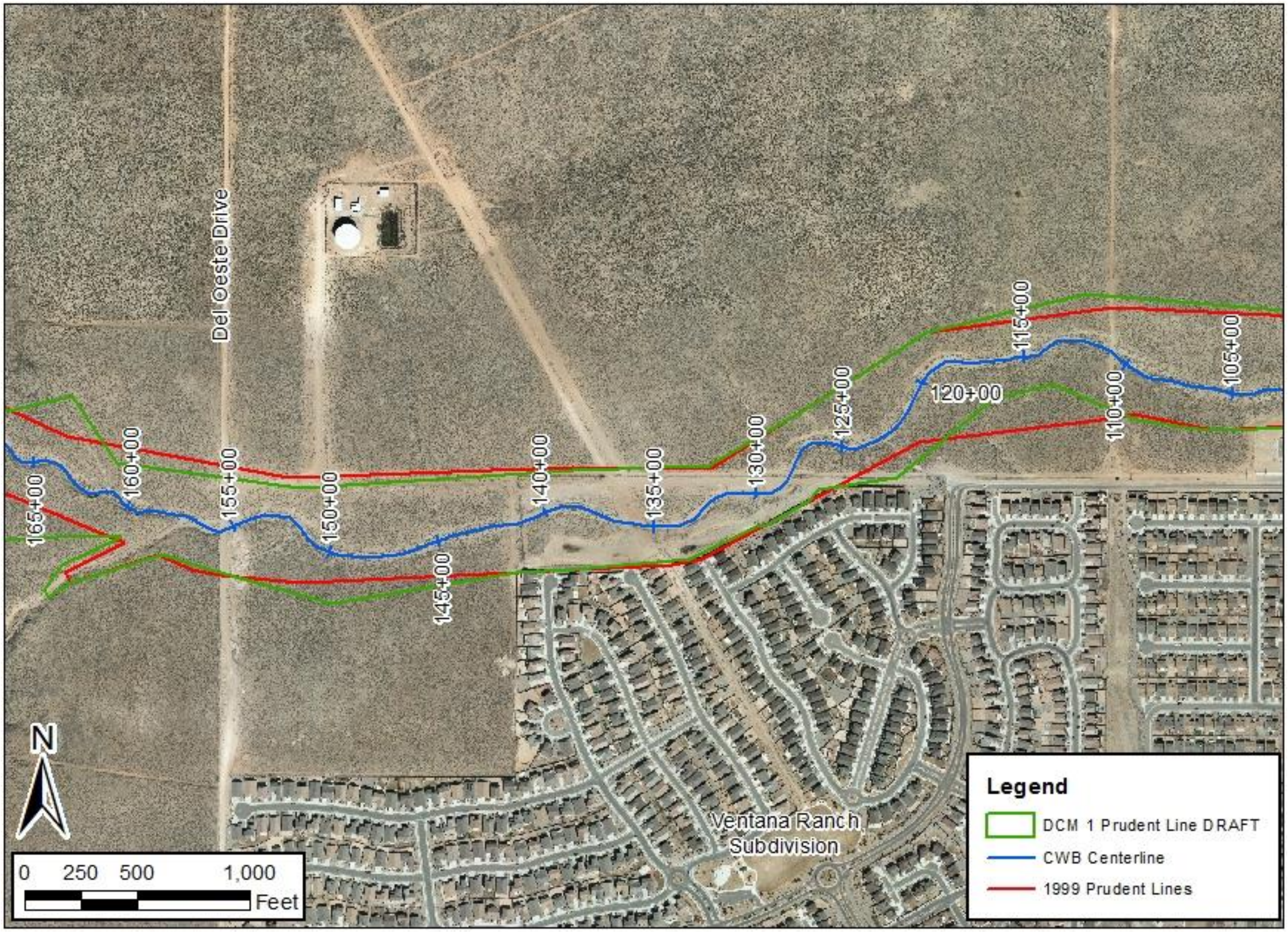

Figure 12 – DCM #1 and 1999 Prudent Lines from Station 105+00 to Station 165+00.

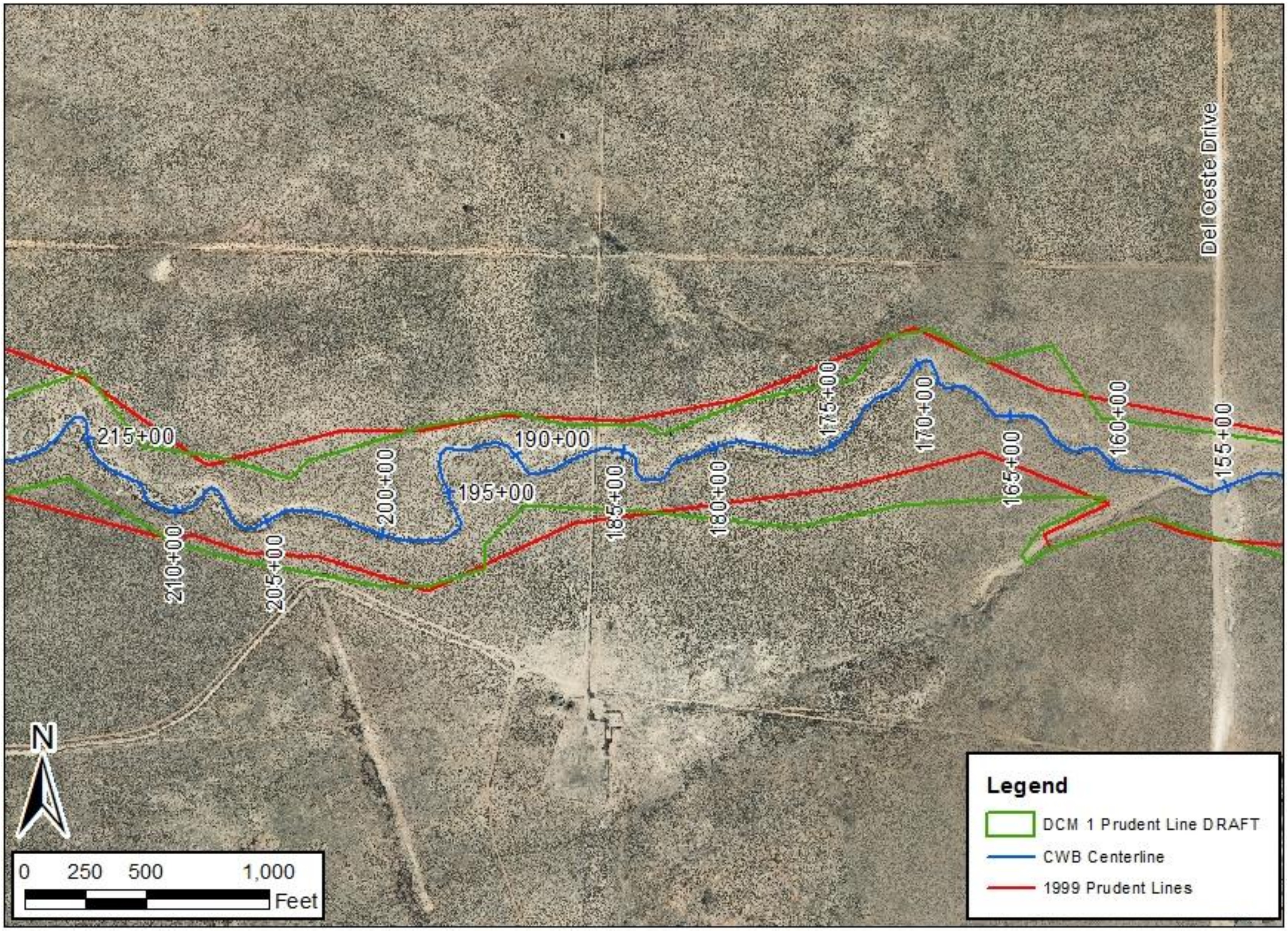

Figure 13 – DCM #1 and 1999 Prudent Lines from Station 155+00 to Station 215+00.

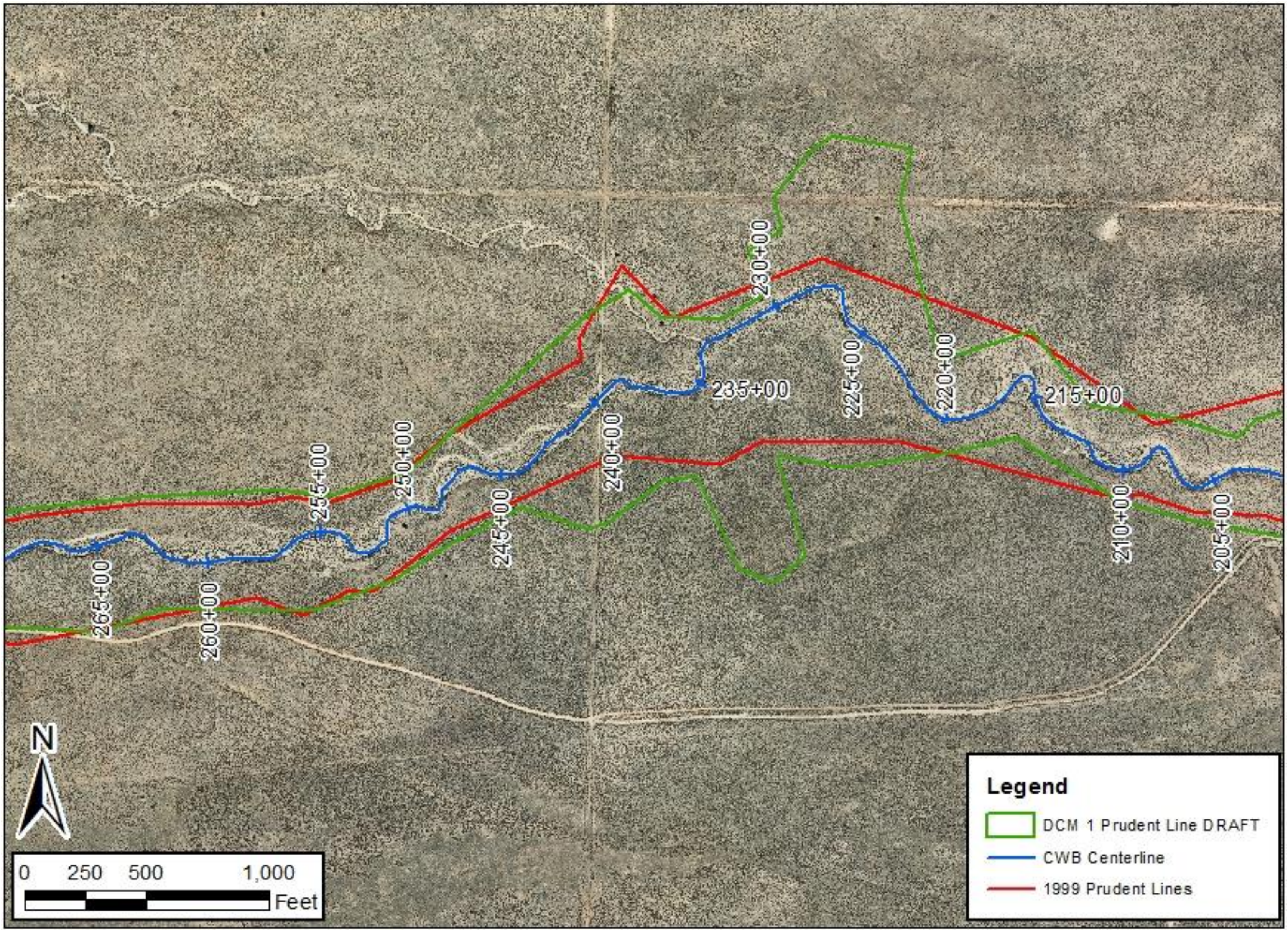

Figure 14 – DCM #1 and 1999 Prudent Lines from Station 205+00 to Station 265+00.

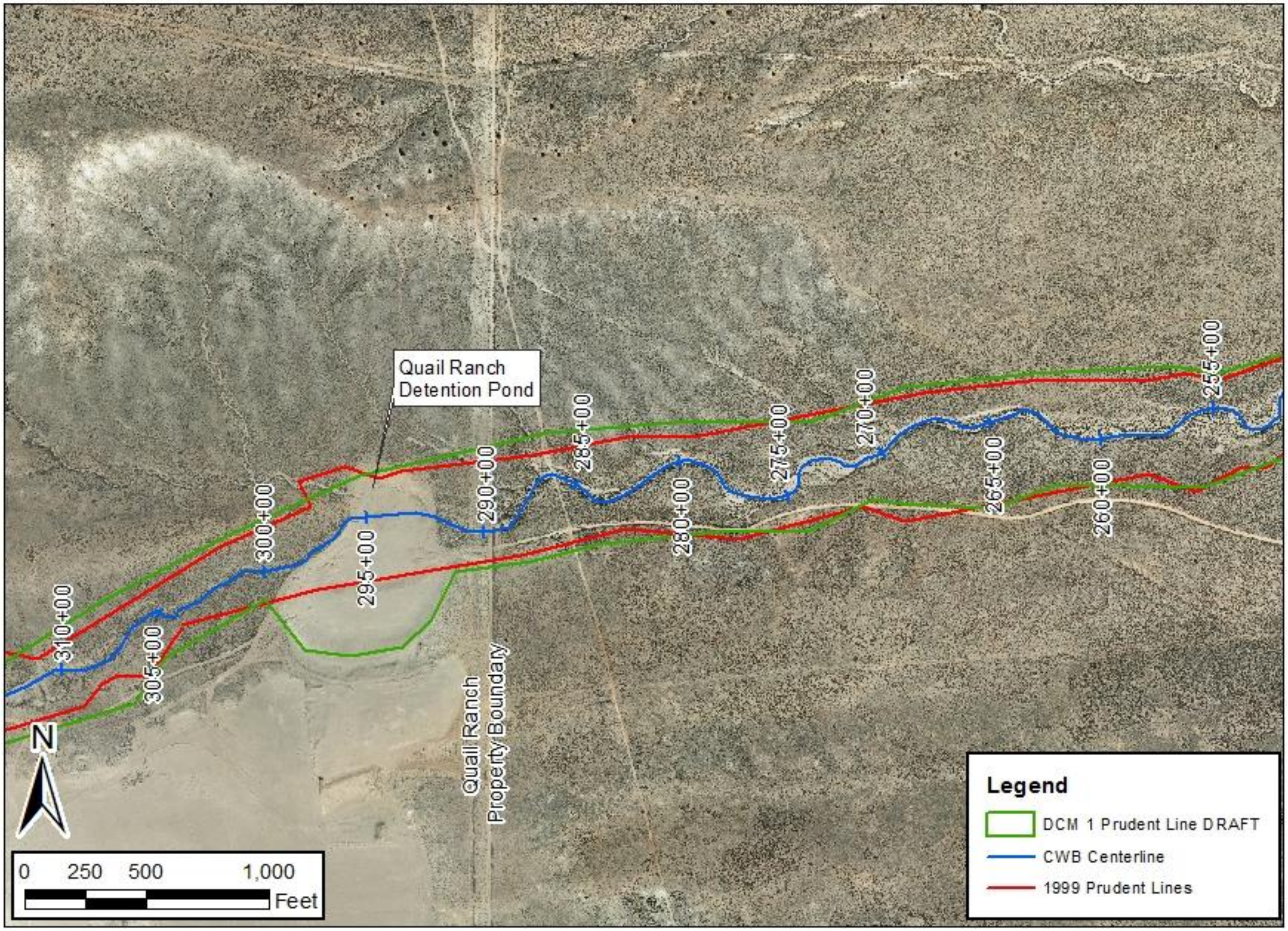

Figure 15 – DCM #1 and 1999 Prudent Lines from Station 255+00 to Station 310+00.

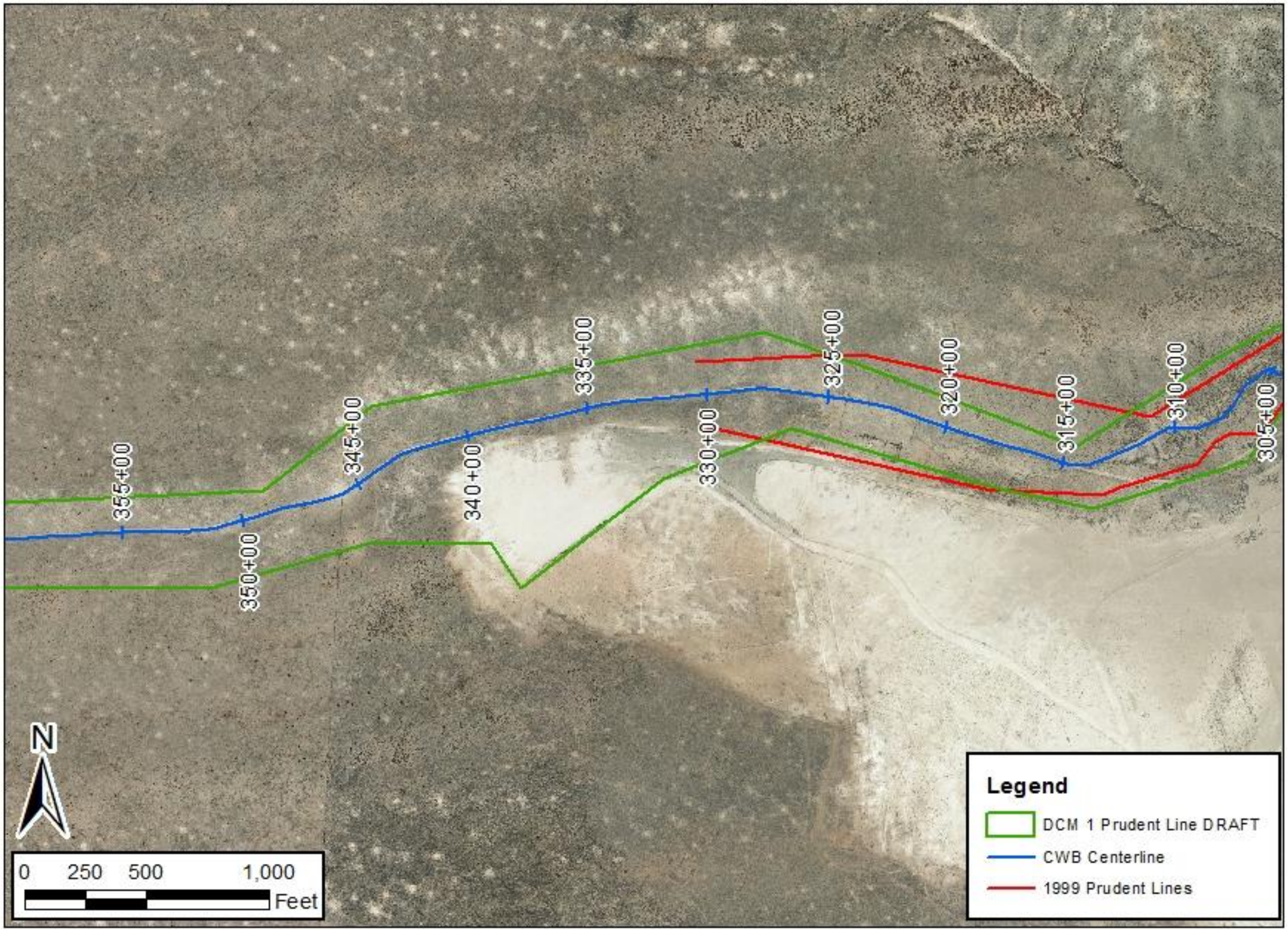

Figure 16 – DCM #1 and 1999 Prudent Lines from Station 305+00 to Station 355+00.

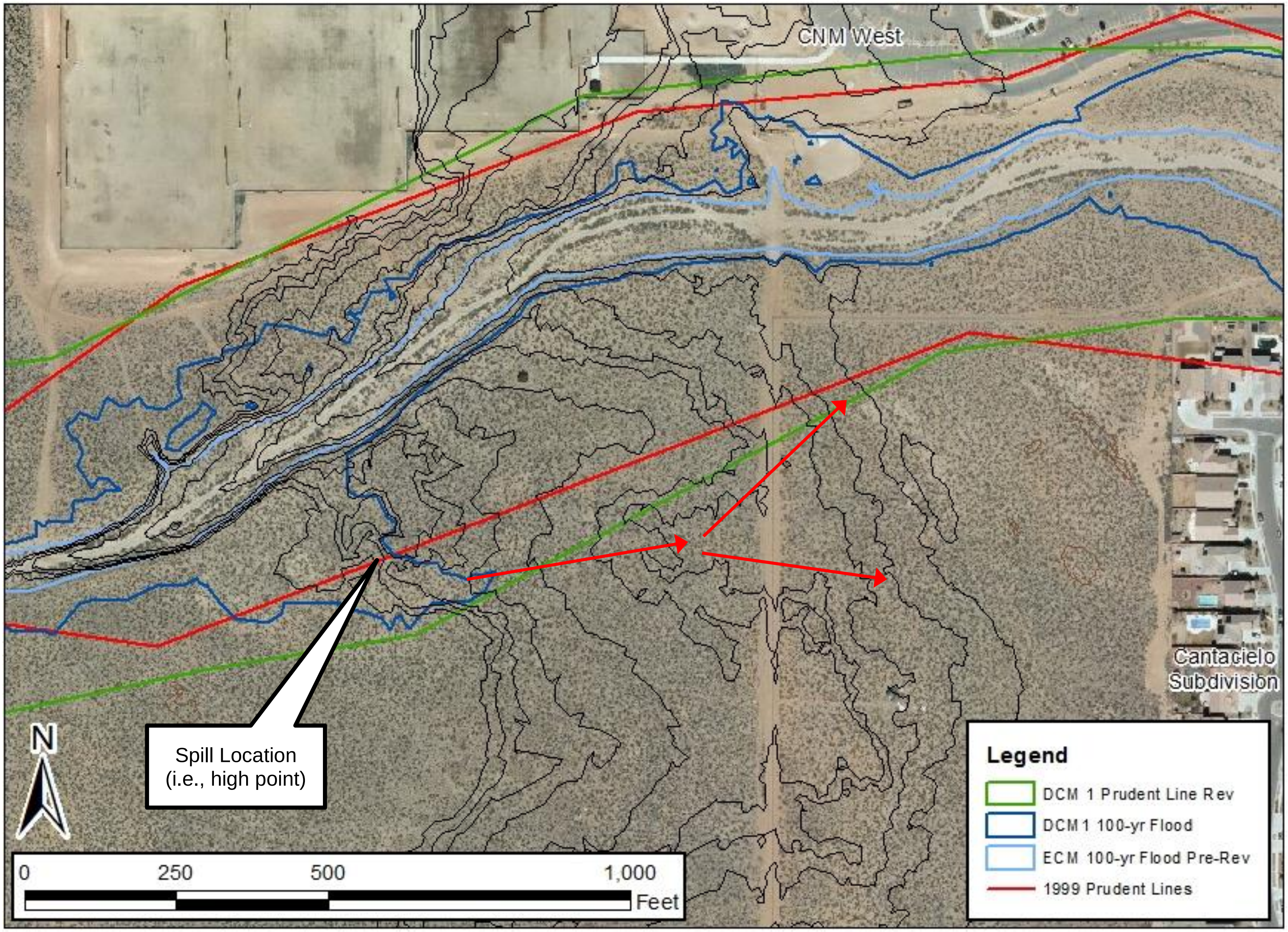

Figure 17 – Potential Spill Location in Left Floodplain near Station 76+00 (Note: Contours don't represent a specific interval).

4.2 DCM #2 PRUDENT LINES

The DCM #2 prudent line is presented in Figure 18 to Figure 22. The prudent line was created following the steps outlined in Section 2. Some additional revisions were made to the prudent line using engineering judgement. There are some locations where the DCM #2 prudent line varies appreciably from the 1999 prudent line. These revisions and differences are described in Table S. In the "Side" column in Table S, left and right are from the perspective of someone looking upstream.

The limits of the DCM #1 Prudent Line are from Universe Blvd. up to the location of the proposed Quail Ranch Detention Pond at Sta. 295+00.

Table S – Summary of Prudent Line Revisions and Appreciable Differences from DCM #2 to 1999

Station	Side	Comment
104+00	Left	The DCM #2 prudent line initially overlapped part of the Prickly Brush Subdivision. It is assumed that AMAFCA will perform inspection and maintenance of the bank and / or storm water quality pond at this location to alleviate any threats to the subdivision prior to damage occurring.
129+00	Left	The DCM #2 prudent line initially overlapped a few back yards in the Ventana Ranch West Subdivision. It is assumed that AMAFCA will perform inspection and maintenance of the bank and / or storm water quality ponds at this location to alleviate any threats to the subdivision prior to damage occurring.
295+00	Left	Flood extents increased appreciably within the Quail Ranch Pond because of the construction of the pond.

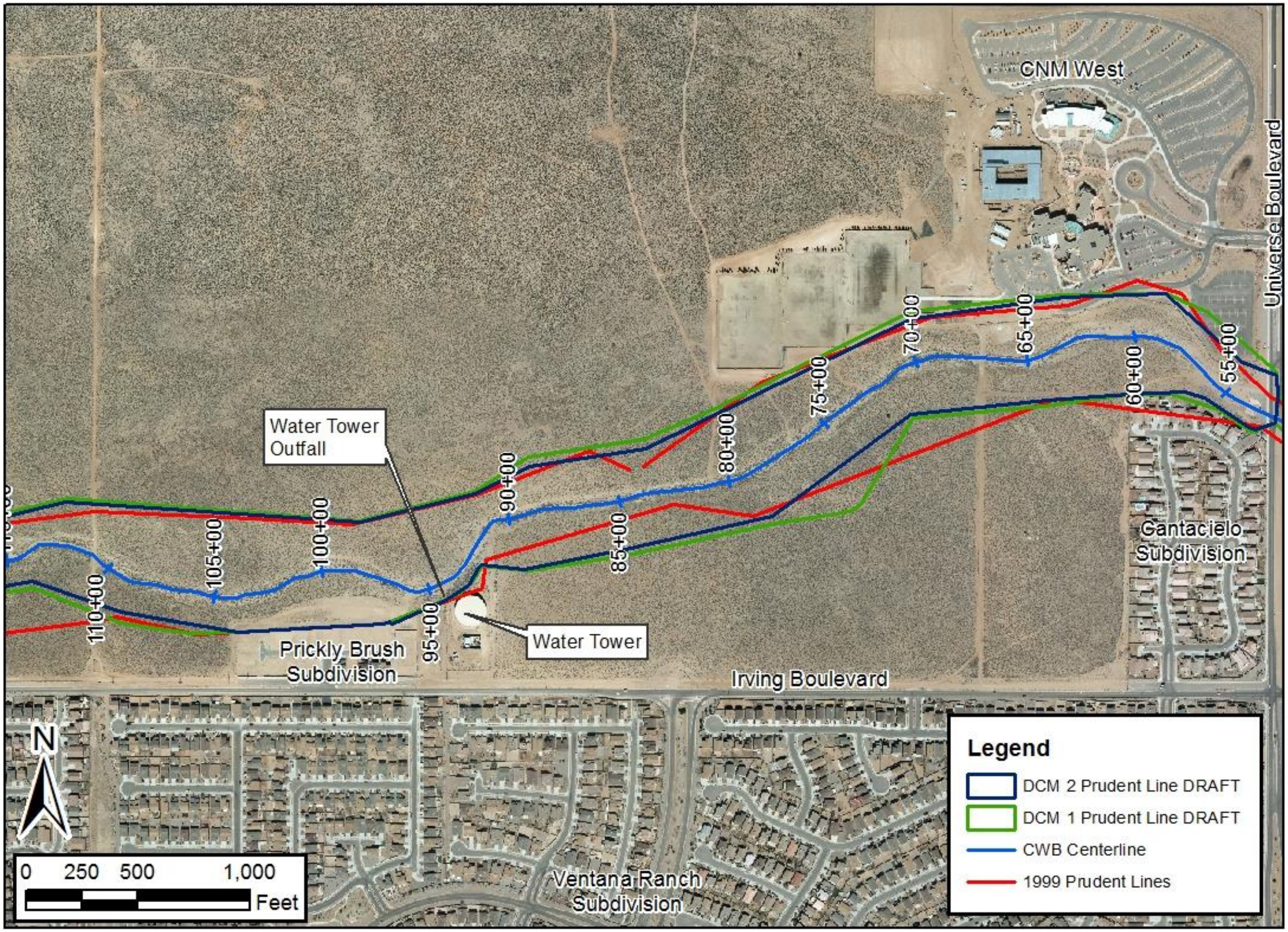

Figure 18 – DCM #2, DCM #1, and 1999 Prudent Lines from Station 55+00 (Universe Boulevard) to Station 110+00.

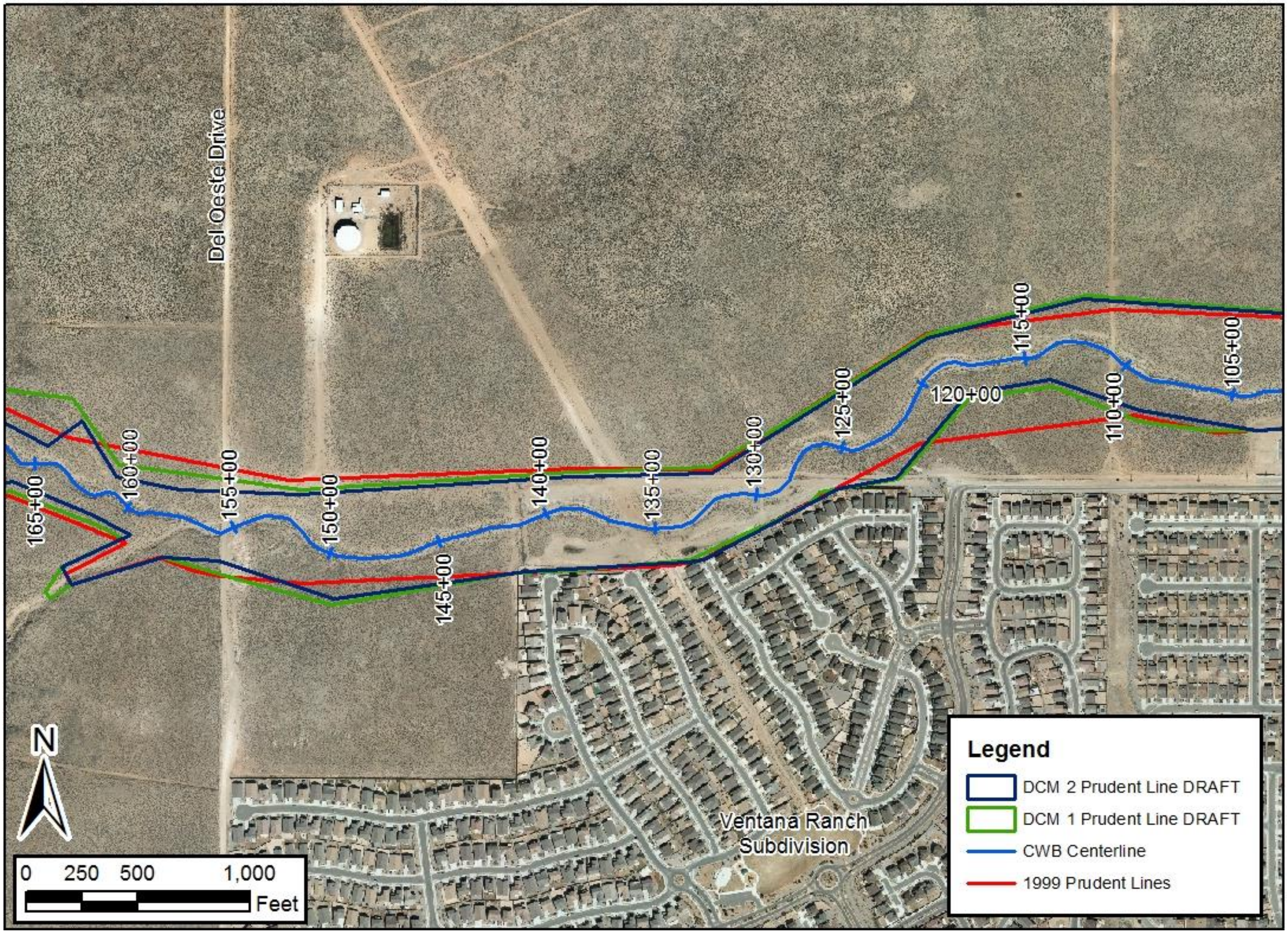

Figure 19 – DCM #2, DCM #1, and 1999 Prudent Lines from Station 105+00 to Station 165+00.

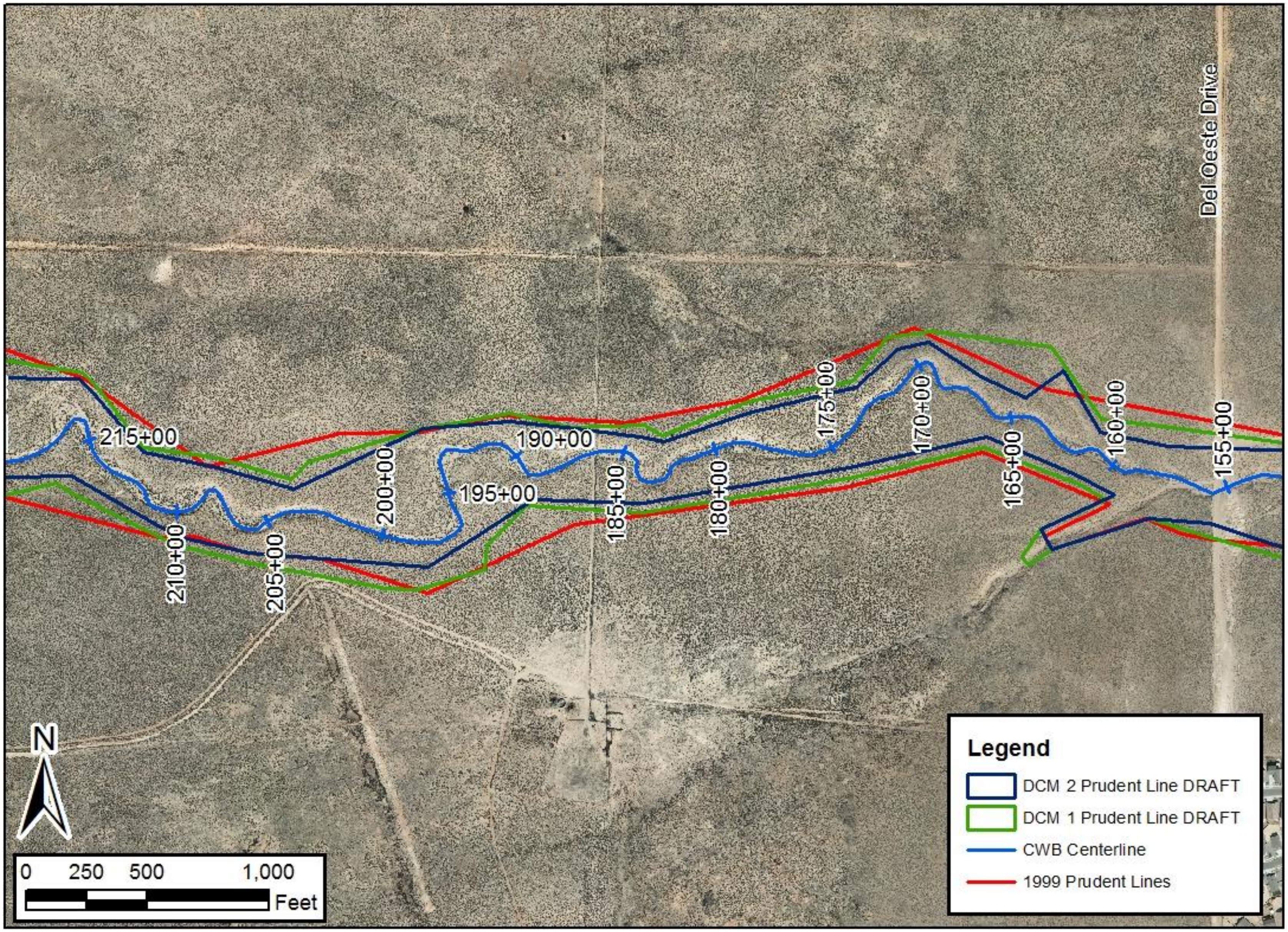

Figure 20 – DCM #2, DCM #1, and 1999 Prudent Lines from Station 155+00 to Station 215+00.

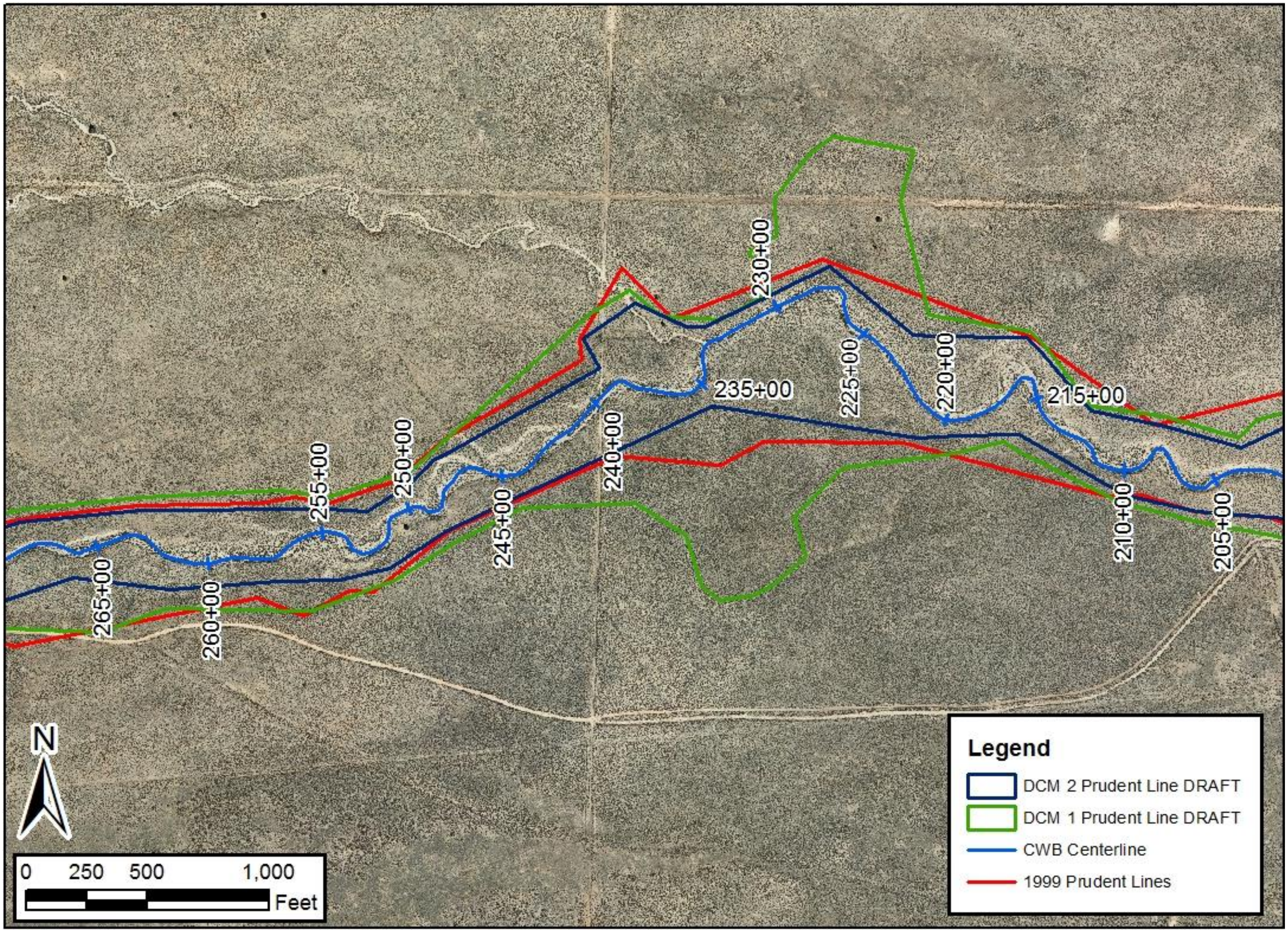

Figure 21 – DCM #2, DCM #1, and 1999 Prudent Lines from Station 205+00 to Station 265+00.

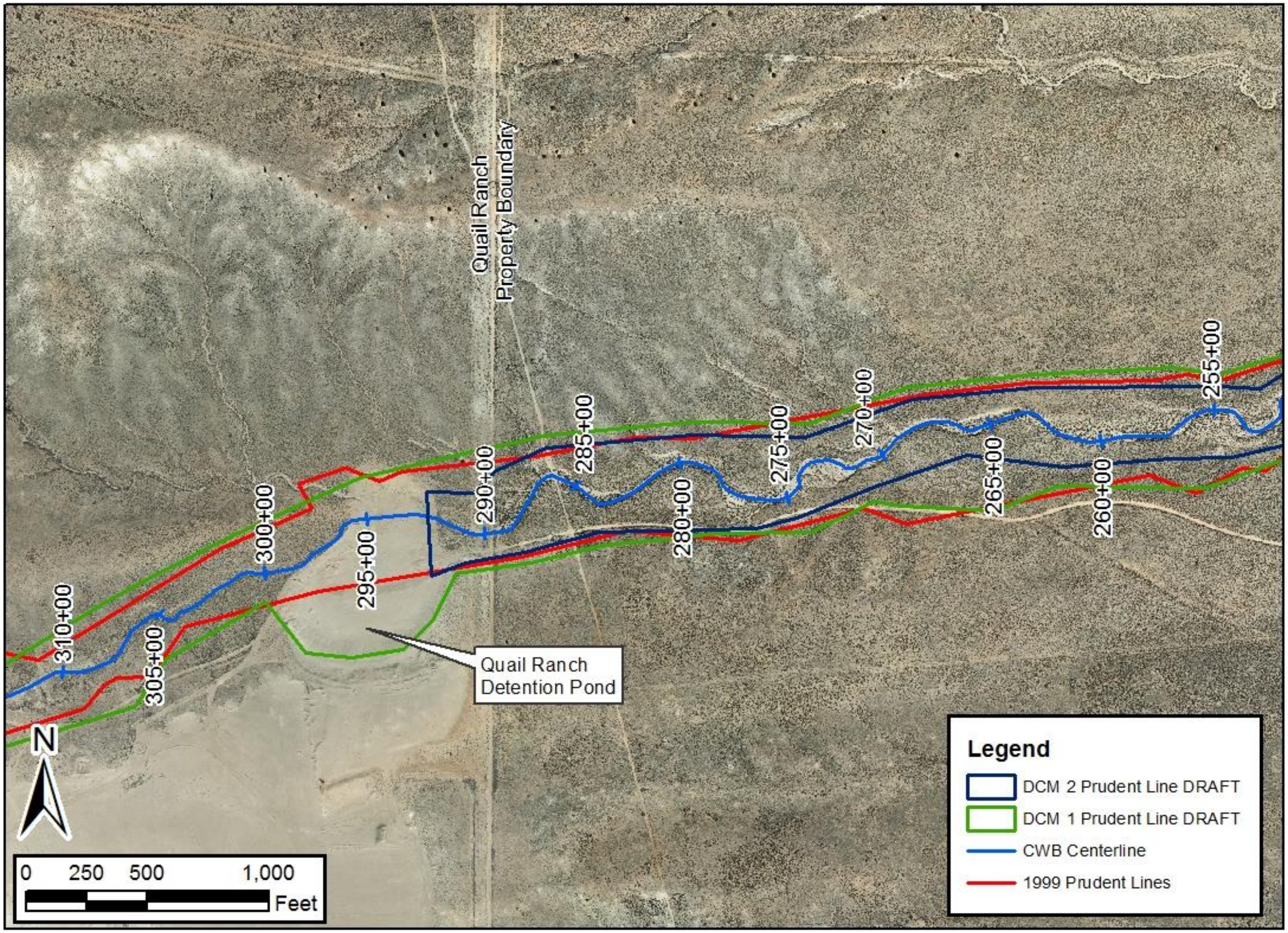

Figure 22 – DCM #2, DCM #1, and 1999 Prudent Lines from Station 255+00 to Station 310+00.

4.3 DCM #3 PRUDENT LINES

The DCM #3 prudent line is presented in Figure 23 to Figure 24. The prudent line was created following the steps outlined in Section 2. Some additional revisions were made to the prudent line using engineering judgement. There are some locations where the DCM #3 prudent line varies appreciably from the 1999 prudent line. These revisions and differences are described in Table T. In the "Side" column in Table T, left and right are from the perspective of someone looking upstream.

Table T – Summary of Prudent Line Revisions and Appreciable Differences from DCM #3 to 1999

Station	Side	Comment
104+00	Left	The DCM #3 prudent line initially overlapped part of the Prickly Brush Subdivision. It is assumed that AMAFCA will perform inspection and maintenance of the bank and / or storm water quality pond at this location to alleviate any threats to the subdivision prior to damage occurring.
129+00	Left	The DCM #3 prudent line initially overlapped a few back yards in the Ventana Ranch West Subdivision. It is assumed that AMAFCA will perform inspection and maintenance of the bank and / or storm water quality ponds at this location to alleviate any threats to the subdivision prior to damage occurring.

4.4 COMPARISONS OF PRUDENT LINES

Upstream from Universe Boulevard (Station 55+00), the DCM #1 prudent line is the widest, and the DCM #3 prudent line is the narrowest. The DCM #3 prudent line extents are generally 100 feet to 150 narrower than the DCM #1 prudent line extents from Universe to Del Oeste Road (Station 155+00). The DCM #2 prudent line lies approximately half way between the DCM #1 and DCM #3 prudent lines. Upstream of Del Oeste Road, the DCM #3 prudent line would match the DCM #2 prudent line. From Del Oeste to the Quail Ranch Pond, the DCM #2 prudent line extents are generally 90 feet to 130 feet narrower than the DCM #1 prudent line extents. Upstream from the Quail Ranch Pond all the prudent lines would match. There are some locations where the DCM #1 prudent line extents are much wider than the DCM #2 and DCM #3 prudent line extents (e.g., Station 220+00 to Station 240+00, Station 75+00). In these locations, the DCM #1 bulked 100-year flood is not contained by the channel and flows spill into a broader floodplain.

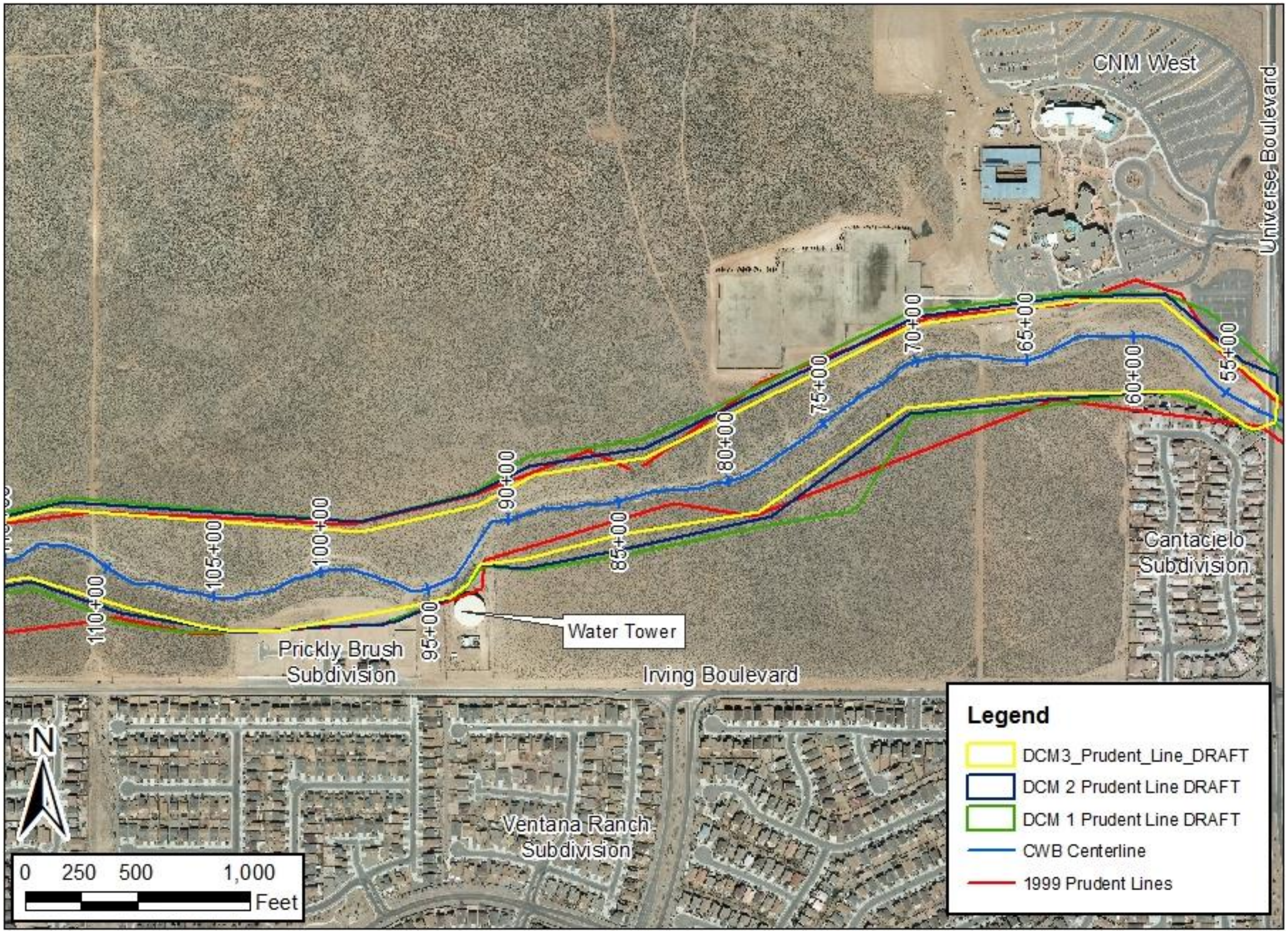

Figure 23 – DCM #3, DCM #2, DCM#1, and 1999 Prudent Lines from Station 55+00 (Universe Boulevard) to Station 110+00.

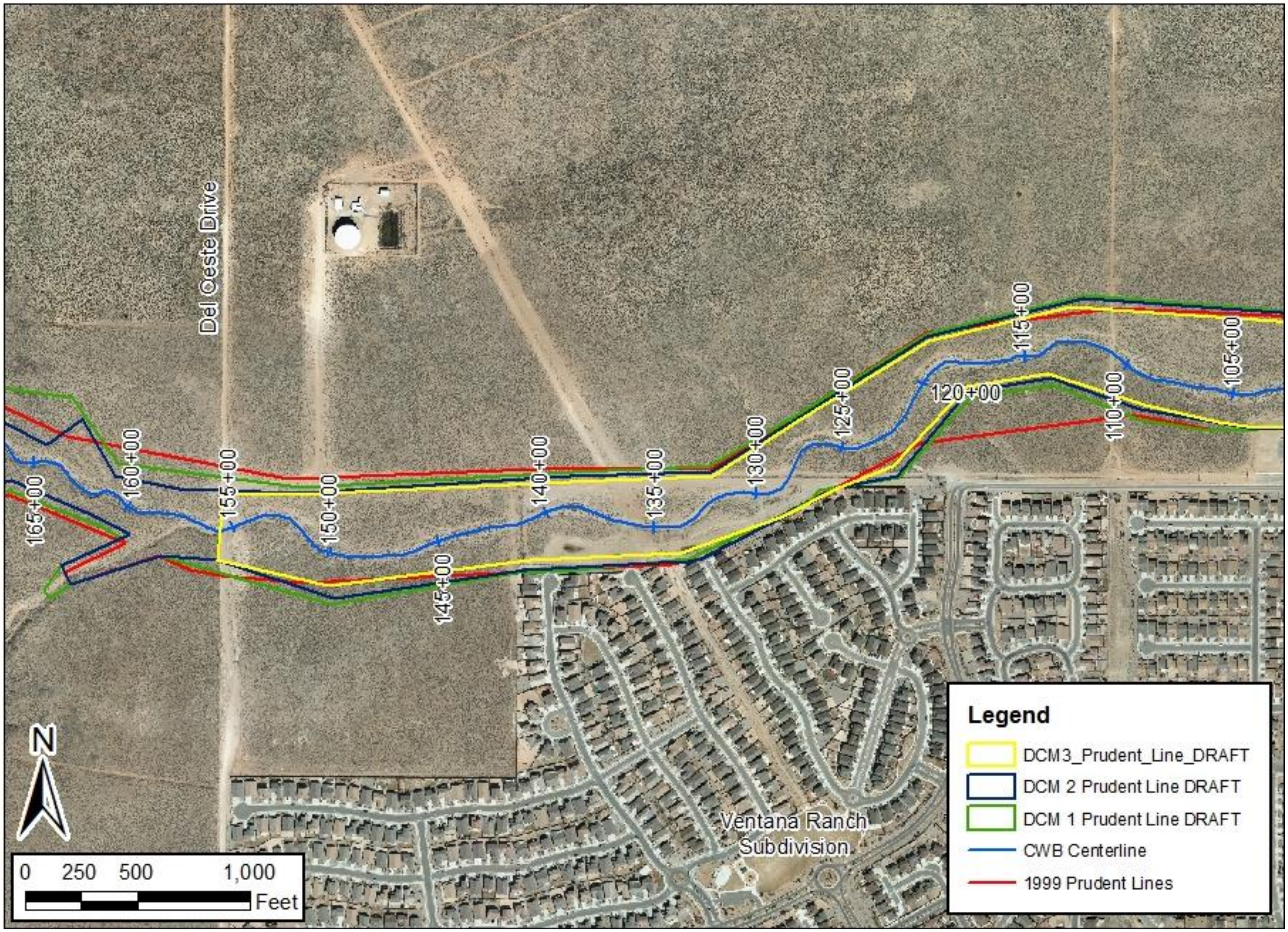

Figure 24 – DCM #3, DCM #2, DCM#1, and 1999 Prudent Lines from Station 105+00 to Station 165+00.

5.0 REFERENCES

Calabacillas Arroyo Prudent Line Study and Related Work: Development of a Prudent Line for the West Branch, Albuquerque, New Mexico. Mussetter Engineering, Inc. 1999. Report prepared for the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA).

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White Paper – Migrating from AHYMO '97 to HEC-HMS (and USEPA SWMM), Easterling Consultants LLC, Sept. 29, 2014

6.0 APPENDICES

(All Appendices are on attached CD)

6.1 Appendix A, Precipitation Data for HEC-HMS Models

6.2 Appendix B, Basin Lag Time Calculations

6.3 Appendix C, Basin Curve Numbers

6.4 Appendix D, HEC-HMS Digital Models

6.5 Appendix E, HEC-HMS Output Files

6.6 Appendix F, HEC-RAS Digital Models

6.7 Appendix G, Bulking Factors and Lateral Erosion Envelope Calculations

6.8 Appendix H, Supporting Data for Conceptual Ponds

(This report is also contained on the attached CD)