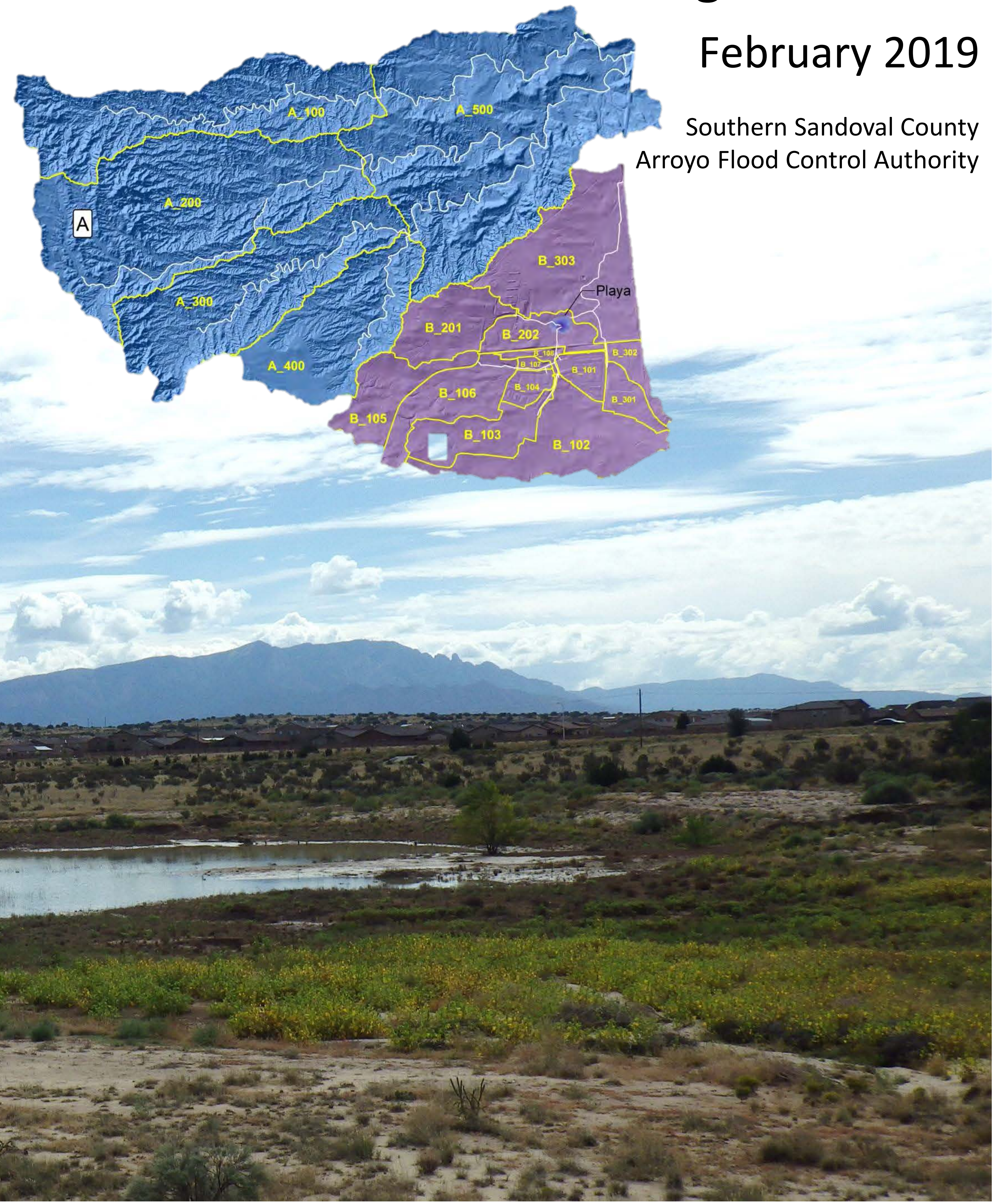
Zia Watershed Park Management Plan

February 2019

Southern Sandoval County
Arroyo Flood Control Authority



Disclaimer

This is a planning document. Nothing herein constitutes any commitment by SSCAFCA to construct any project, study any area, acquire any right of way or enter into any contract. This watershed park management plan does not obligate SSCAFCA in any way.

Drainage facility alignments, conveyance treatments, corridors, locations, rights-of-way and cost estimates are conceptual only, and may be altered or revised based upon future project analysis, changed circumstances or otherwise. Land uses included in this document were assumed for the basis of hydrologic modeling only. This document does not grant "free discharge" from any proposed development. Naturalistic channel treatments and piped storm drains are to be used for conveyance stabilization, unless otherwise authorized by SSCAFCA.

To ensure public health, safety and welfare, SSCAFCA develops and maintains the adopted regional hydrology for all watersheds within its jurisdiction. Updates and revisions are made and tracked by SSCAFCA, or their designee. A copy of the regional hydrology model is available for reference or use by others. Contact SSCAFCA to obtain copies of the model and see the SSCAFCA website for the Watershed Management Plan status. Use of electronic media provided by SSCAFCA is solely at the user's risk.

Note:

The Zia Watershed Park Management Plan was originally published in February 2019 and corrected in December 2020 after an error in the flood routing procedure was discovered.

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1. Introduction

The Zia watershed is located at the northern boundary of Rio Rancho, west of US Highway 550, and includes portions of Zia and Santa Ana Reservation lands (see Figure 1.3, blue and green shading). The watershed consists of two major tributaries (A and B) with distinct geomorphology. Tributary A features steeply sloped terrain (Figure 1.1) and an intricate network of dry stream channels (arroyos) that drain to a bridge under US 550. Storm flows then continue to the Jemez River and eventually the Rio Grande. Tributary B, on the other hand, is comprised of gently sloping terrain and largely lacks defined flow paths (Figure 1.2).

Figure 1.1: Tributary A is characterized by steep slopes and a network of gullies and arroyos.

Figure 1.2: Gently sloping terrain characteristic of Tributary B.

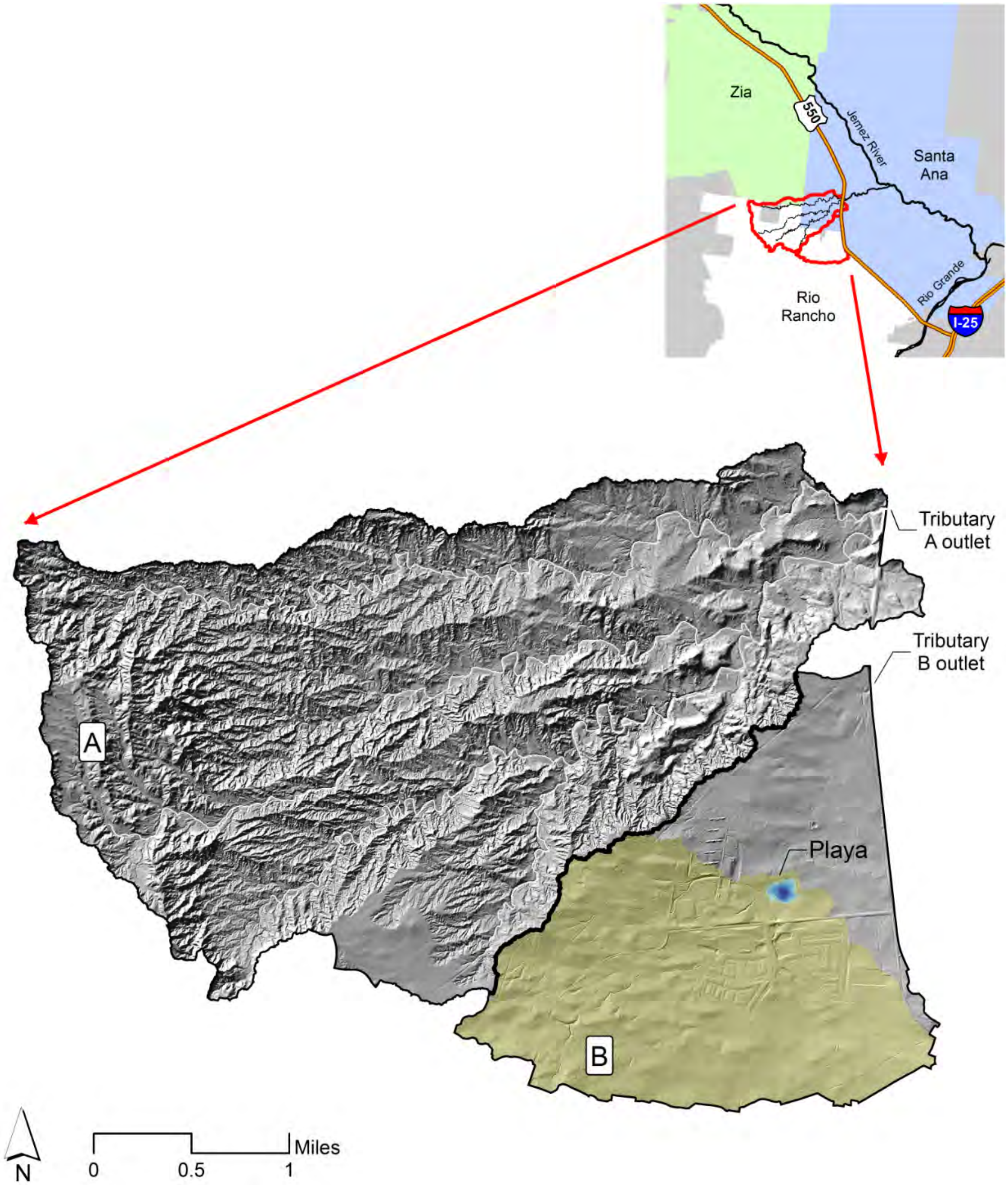

Figure 1.3: Overview map of the Zia watershed (area highlighted in yellow drains to the playa).

A large portion of the catchment (see Figure 1.3, yellow shading) drains to a natural playa with an estimated storage volume of more than 50 acre-feet. Any overflow from the playa and runoff from the remaining basin travels to a set of concrete box culverts under US 550 (see Figure 1.3, Tributary B outlet). East of the highway, storm flows continue towards the Jemez River. The arroyo, clearly defined at first, disappears before reaching the river (Figure 1.4). This so-called "flood-out" area indicates that all storm flows infiltrate into the ground before reaching the Jemez River. No observed discharge data is available for the Zia catchment. Nevertheless, the clearly defined, wide arroyo downstream of Tributary A suggests that this system flows frequently. On the other hand, comparatively little flow appears to leave Tributary B.

Figure 1.4: Arroyos connecting Tributaries A and B with the Jemez River.

2. Watershed Hydrology

The hydrology of the Zia watershed was modeled using *HEC-HMS* version 4.2.1.

2.1. Basin Delineation

Orthophotography used for this project consists of tiled images which depict color digital aerial photographs acquired in the spring of 2018 during leaf-off conditions. LiDAR-derived elevation data (2-foot contour interval, 2010) were used to delineate watersheds and sub-basins as well as for calculating hydrologic parameters. Both orthophotogarphy and elevation data are part of the Mid-Region Council of Governments (MRCOG) Digital Orthophotography and Elevation Data Project. Figure 2.1 shows subbasin delineation and major flow paths for tributaries A and B of the Zia watershed. Initial watershed and subbasin boundary delineation was accomplished using *HEC Geo-HMS* software with a digital elevation model (DEM) created from 2010 MRCOG LiDAR data. All basin boundaries were checked based on 2010 2-ft elevation contours. Questionable boundaries were verified in the field, especially at locations where graded roads influence flow paths, and where a dominant flow path was not immediately obvious from 2-ft contours.

2.2. Reach Routing

Routing reaches were delineated, and slopes were estimated in *Arc-GIS* based on 2010 2-ft contours. Channel reaches were modeled using idealized cross-sections that most closely resembled the natural geometry of the reach (trapezoidal and rectangular). Roughness coefficients (Manning's n-values) were estimated based on orthoimagery and field investigations. In general, the following n-values were used in the model:

Table 2.1: Roughness coefficients for routing reaches.

Surface Type	Manning's n-value
Concrete pipe	0.013
Road (asphalt)	0.017
Corrugated metal pipe	0.025
Major arroyo, sandy bed and vertical banks	0.020
Natural channel, moderate to heavy vegetation in channel bed and along banks	0.025 - 0.035

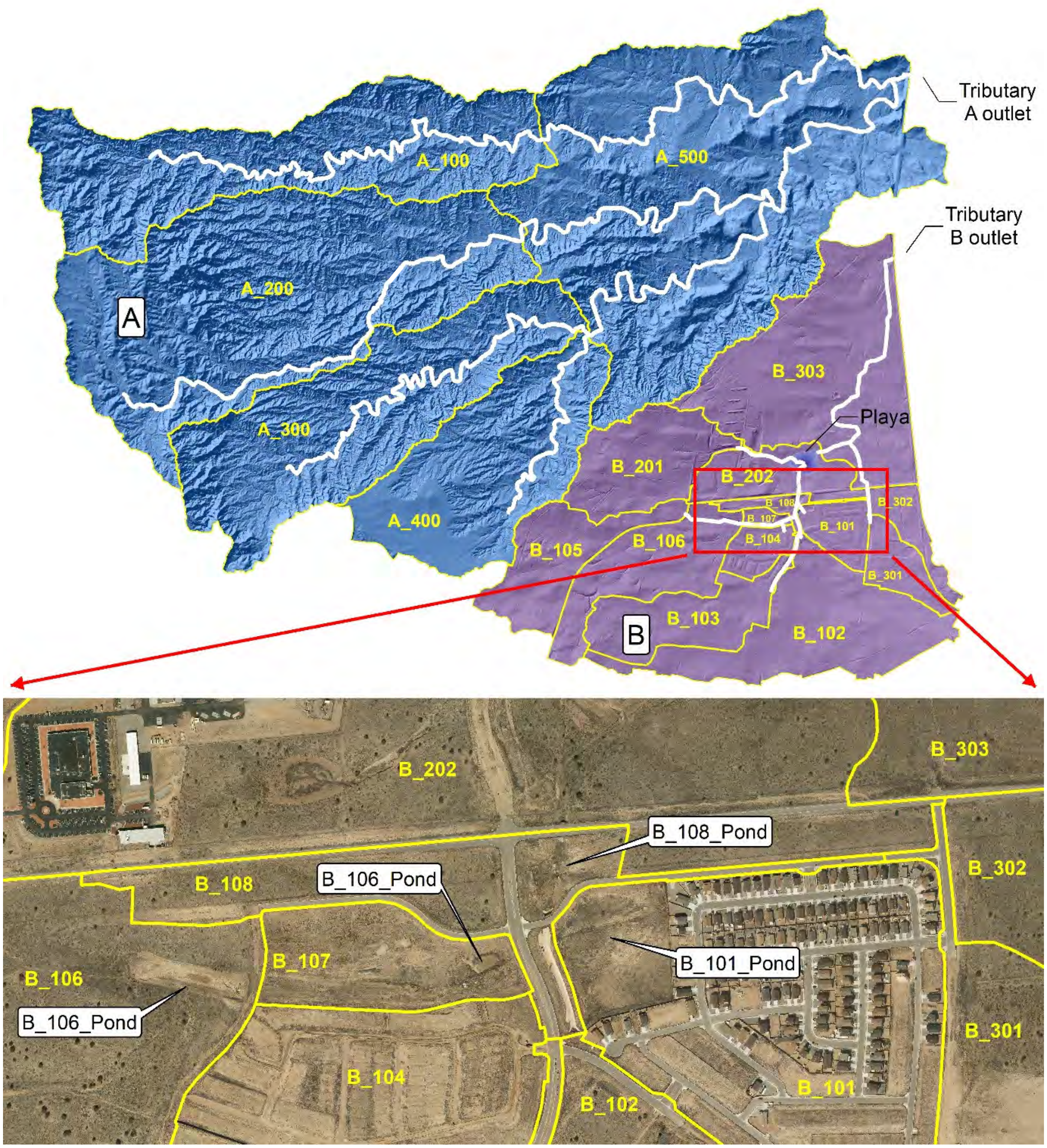

Figure 2.1: Subbasin delineation and major flow paths (white) for tributaries A and B of the Zia watershed.

2.3. Existing Land Use

Figure 2.2 illustrates that under existing conditions, the majority of Tributary A is undeveloped. Land use was quantified by manual digitization using orthoimagery and based on GIS data obtained from the City of Rio Rancho. Special emphasis was placed on impervious coverage: directly connected impervious areas (DCIA) were specified explicitly for each subbasin rather than including them in a composite loss calculation. A SSCAFCA study on the impacts of urban imperviousness showed that this approach yielded satisfactory results (Schoener, 2017). Disturbed or mass graded areas (Figure 2.2, orange) were digitized manually.

2.4. Existing Conditions Loss Parameters

Parameters for the Curve Number (CN) loss method (USDA, 2004) were estimated based on 2018 land use conditions in the Zia watershed. Table 2.2 lists land use types and associated loss parameters. Loss parameters for graded areas were estimated based on guidance contained in Technical Release 55 (USDA, 1986). Curve numbers for open space were based on results from two studies (SSCAFCA 2018, Schoener and Stone 2018) indicating that curve numbers for natural areas in the study area range from 68-80. For this study, an intermediate CN of 74 was assumed for open space areas, but model results for dry conditions (CN = 68) and wet conditions (CN = 80) are also reported to provide confidence bounds.

Table 2.2: Land use categories and associate loss parameters for existing conditions 2018.

Land Use Type	Data Source	Pervious CN	% of Total Area	
			Trib. A	Trib. B
Paved roads with curb, residential driveways, other DCIA	CoRR curb coverage, parcels, manual digitization	-	<1	3
Building footprint	CoRR building footprints, manual digitization	98	-	<1
Graded areas	Manual digitization	86	1	17
Open space < 10% slope	GIS	74	21	80
Open space > 10% slope	GIS	74-77	78	-

Almost 80 percent of the area in Tributary A has slopes exceeding 10% (see Figure 2.3). Open space curve numbers were therefore scaled based on land slope using the method proposed by Huang (2006), whereby curve numbers increase with increasing slope. A table of model parameters is included in Appendix A.

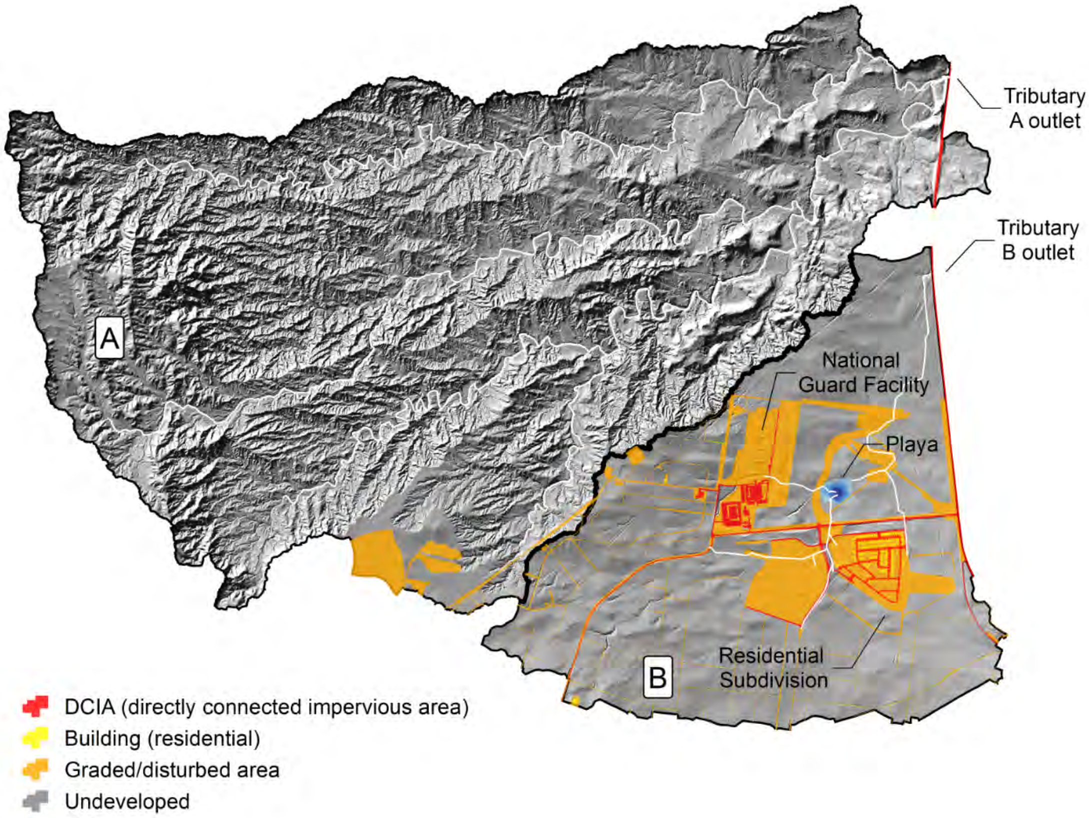

Figure 2.2: Map of the Zia watershed and major land use types for existing conditions 2018.

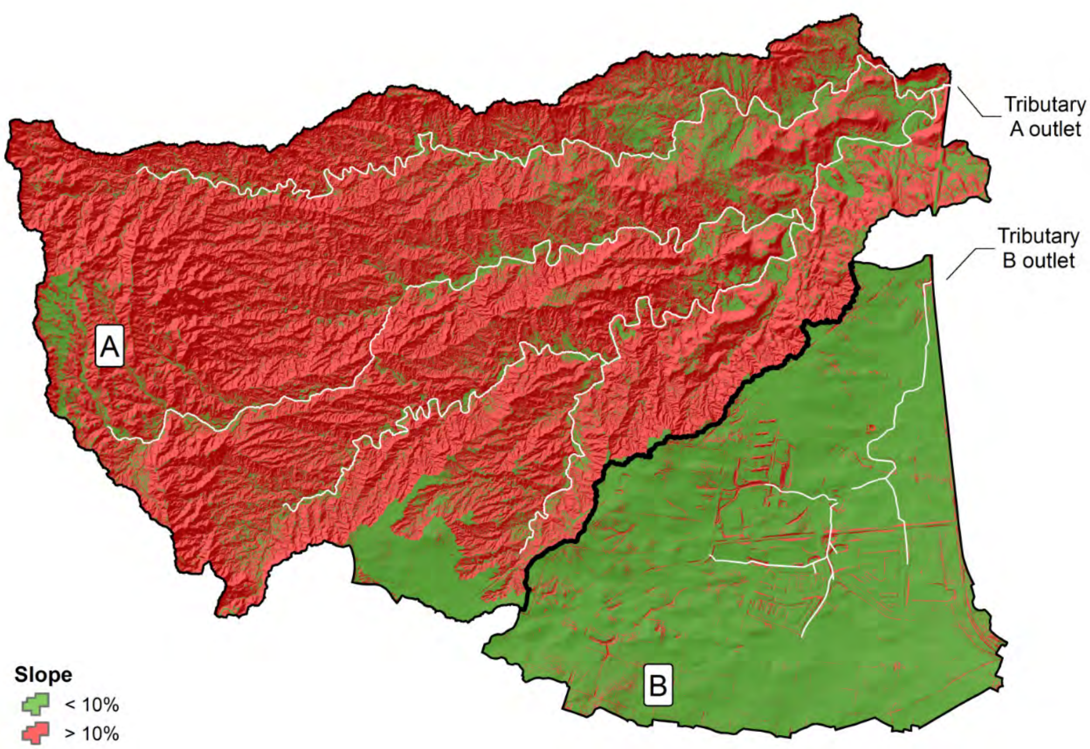

Figure 2.3: Map of percent land slope for tributaries A and B of the Zia watershed.

2.5. Projected Future Land Use

Future land use was not assessed as part of this study.

2.6. Transform Method

In *HEC-HMS*, the SCS unit hydrograph was selected to transform excess precipitation into a runoff hydrograph for each subbasin. Lag time was estimated as 60% of the time of concentration (T_c). Times of concentration in turn were estimated in *Arc-GIS* based on the watershed DEM using the methodology outlined in TR-55 (USDA, 1986).

2.7. Sediment Bulking

Sediment bulking factors were added as flow ratios to clearwater discharges in *HEC-HMS* to account for the increase in runoff volume due to suspended sediment in storm flows. Bulking factors of 18% were added to all subbasins in Tributary A; 6% bulking factors were used for Tributary B due to mild slopes found throughout this basin.

2.8. Existing Ponds

The Zia watershed model contains four stormwater detention ponds (see Figure 2.1) and one natural playa. In *HEC-HMS*, pond routing was simulated using elevation-storage curves and outlet structures. Pond and playa volumes were determined in *Arc-GIS* based on the watershed DEM. Parameters for modeling outlet structures were based on field surveys. A list of all ponds included in the watershed model is contained in Appendix B. Ponds were assumed to be dry at the start of each simulation.

2.9. Design Storm

The design storm is used as a planning tool. It is a hypothetical storm event based on point precipitation frequency estimates from NOAA Atlas 14 (NOAA, 2018). Precipitation estimates for the Zia watershed are displayed in Table 2.3. The design storm was modeled in HEC-HMS using the built-in frequency storm option with an intensity position of 25 percent and intensity duration of five minutes.

Table 2.3: Point precipitation frequency estimates for the 100-year recurrence interval in the Zia watershed.

Duration	Point precipitation estimate (in)
5 min	0.593
15 min	1.120
1 h	1.860
2 h	2.120
3 h	2.200
6 h	2.380
12 h	2.540
24 h	2.890

2.10. Existing Conditions Results

Figure 2.4 shows hydrographs at the outlets of Tributaries A (left) and B (right) for existing land use conditions based on the design storm. Black lines in Figure 2.4 are results from model runs with intermediate moisture conditions (base CN = 74) for all open space areas. Grey areas represent model runs with a base curve number range of 68-80 for open space. This range was selected to provide an uncertainty envelope around the estimated 100-year runoff associated with initial moisture conditions.

It is important to note that simulation results only provide a best estimate of the watershed runoff response from the design storm for current land use conditions. Model results are intended to be used for planning and design of flood control infrastructure but need to be interpreted with the underlying uncertainty in mind.

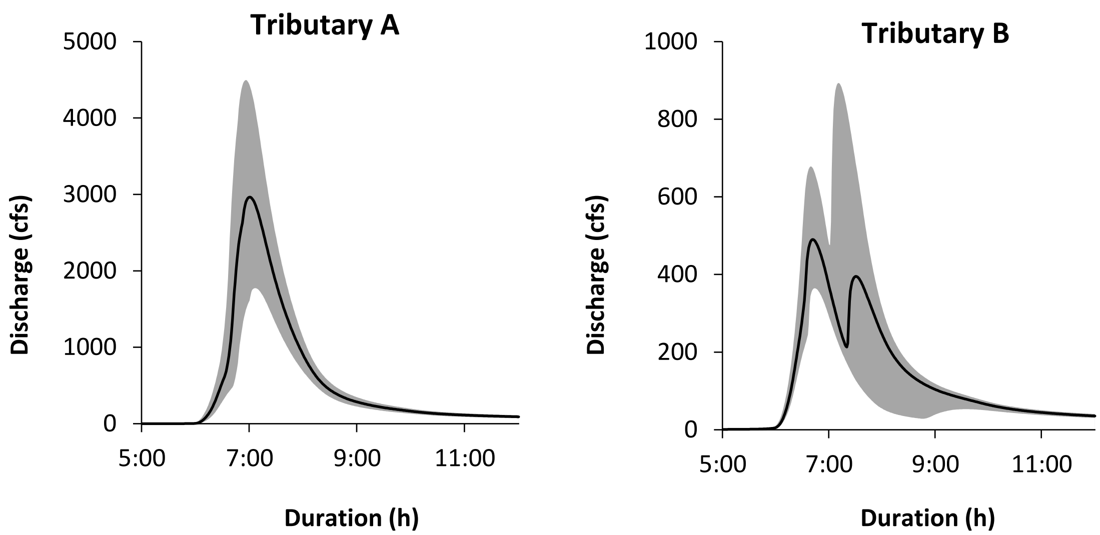

Figure 2.4: Simulated design storm discharge for existing conditions from Tributary A (left) and Tributary B (right). Black lines represent intermediate moisture conditions (open space CN = 74), grey areas are results ranging from dry (CN = 68) to wet (CN = 80).

Based on this analysis, expected peak discharge is approximately 2,965 cfs (range: 1,775 – 4,497 cfs) for Tributary A and 490 cfs (range: 365 – 893 cfs) for Tributary B at each basin outlet. Results are based on depth-area reduction factors for an eight square mile basin (Tributary A) and a three square-mile catchment (Tributary B).

Figure 2.5 shows model results for selected analysis points. Results represent model simulations with no depth-area reduction factors, with the exception of the basin outlets, where depth-area reduction factors for 8 and 3 square mile catchments were used for Tributaries A and B, respectively. Tables with all model results are contained in Appendix C.

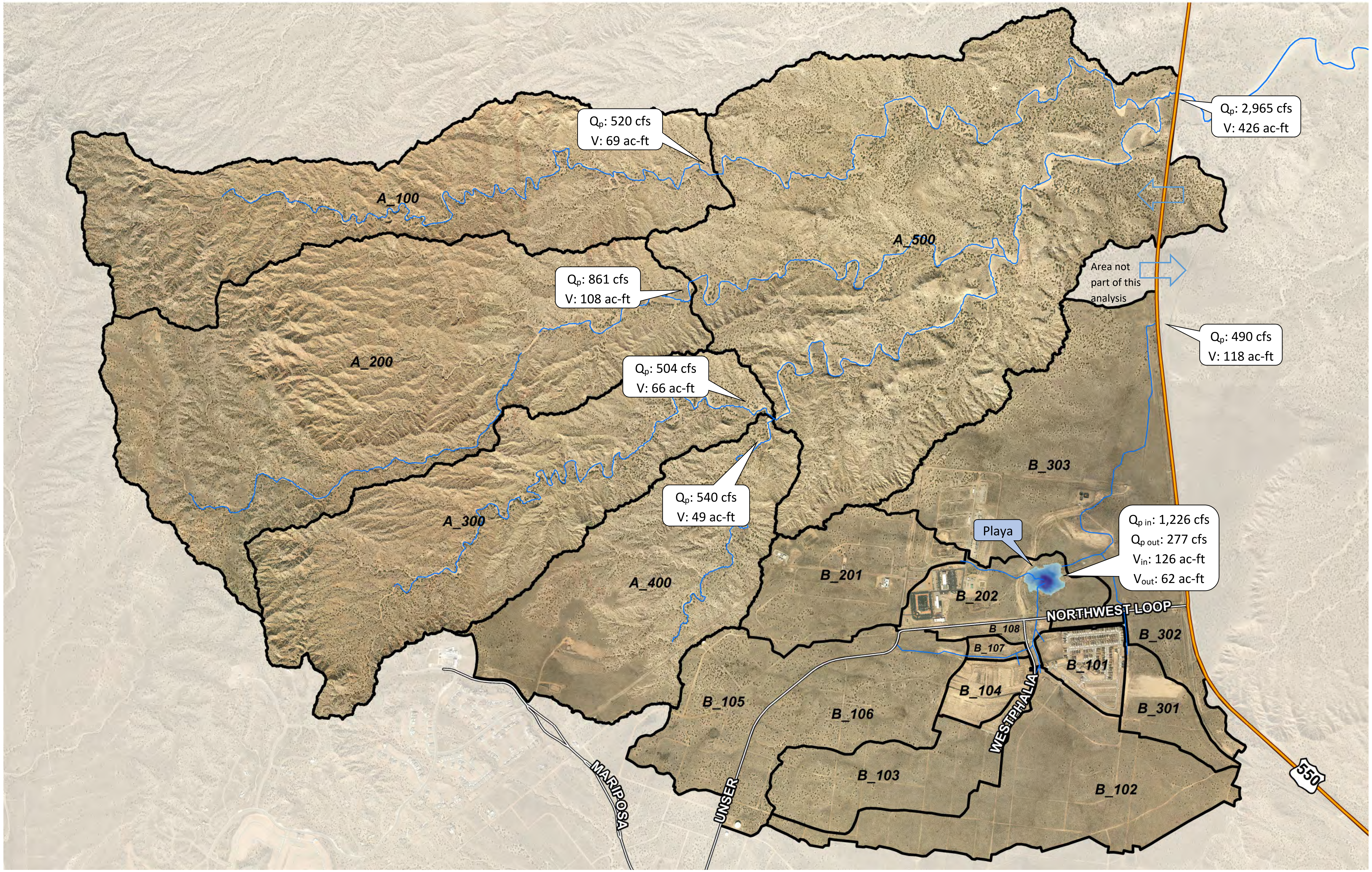

Figure 2.5: Peak discharge (Q_p) and runoff volume (V) results at selected analysis points. Results represent model simulations with no depth-area reduction factors, with the exception of the basin outlets, where depth-area reduction factors for 8 and 3 square mile catchments were used for Tributaries A and B, respectively.

The playa in Tributary B (see Figure 2.6) is an important drainage feature of that basin. It plays a crucial hydrologic role by storing stormwater runoff and attenuating peak flows from 2.3 square miles of contributing area. SSCAFCA holds a drainage easement over portions of the playa (Figure 2.8).

Figure 2.6: Photo of the natural playa in Tributary B (looking north from Northwest Loop), taken in October 2018.

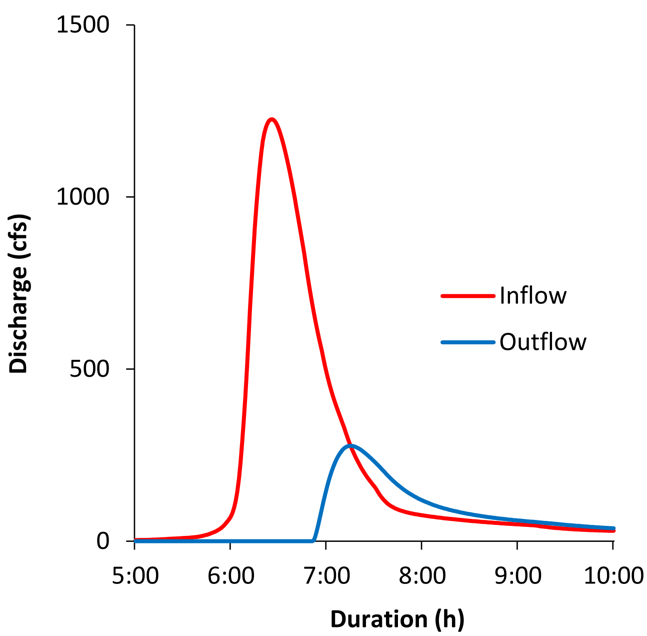

Figure 2.7: Simulated playa inflow (red) and outflow (blue) for intermediate moisture conditions.

Figure 2.7 illustrates that during the 100-year design storm (assuming intermediate moisture conditions), the playa reduces peak discharge from 1,226 cfs to 277 cfs, a reduction of 77%. In addition to hydrologic impacts, the playa likely contributes to habitat and biological diversity by providing a source of water.

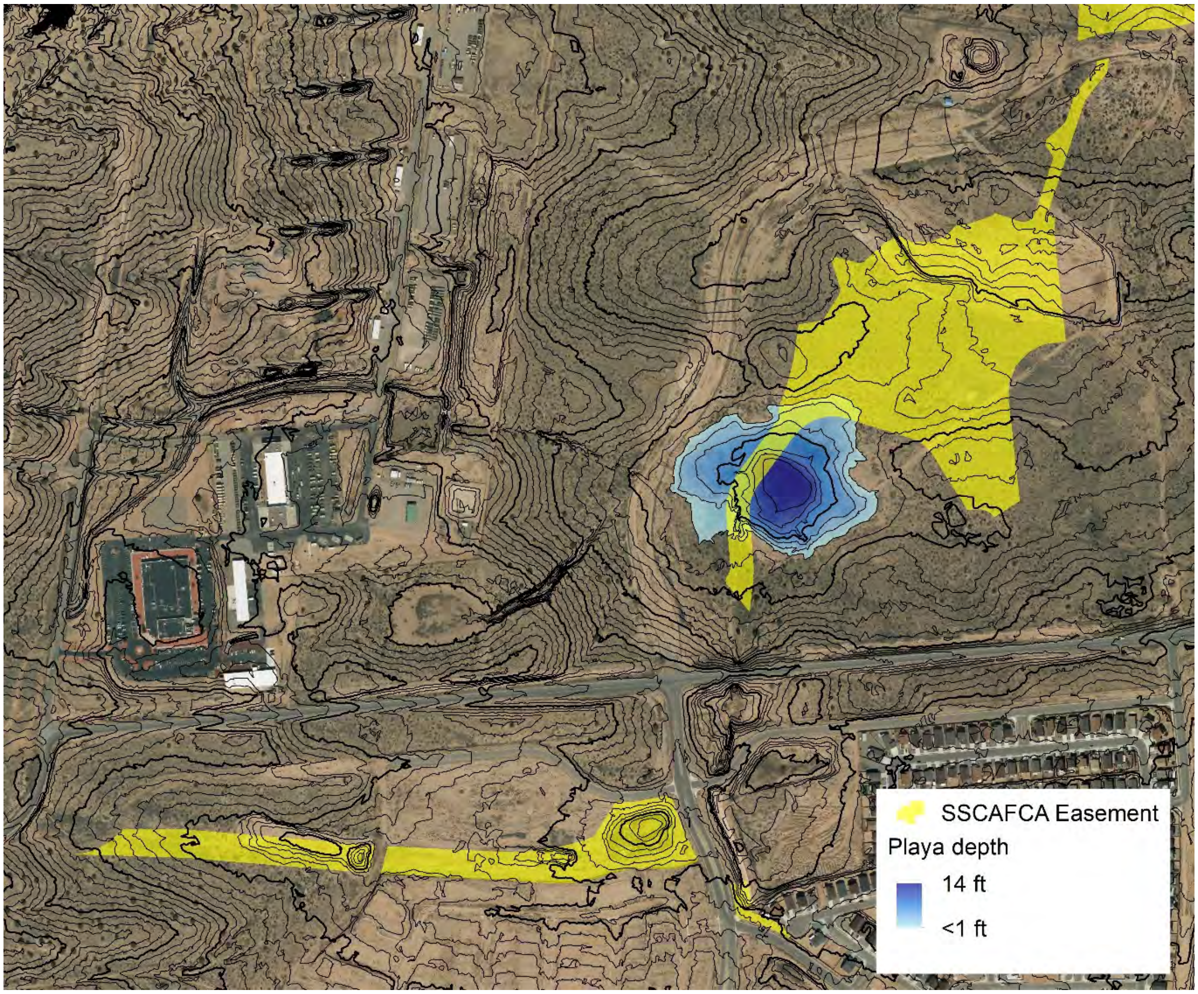

Figure 2.8: Overview map of natural playa location (blue) and SSCAFCA easements (yellow).

2.11. Structure Capacities and Major Deficiencies

Culvert capacities were analyzed based on existing conditions model runs. Results are summarized in Table 2.4 and Figure 2.9. Structure capacities were estimated for planning purposes only to establish approximate maximum allowable flow rates at each location. Capacity calculations are based on field investigations (see Appendix E). All structures have sufficient capacity under existing conditions.

Table 2.4: Major crossing structures, capacities and peak discharges.

Crossing	Location	Structure Description	HMS_ID	Existing Q_p (cfs)	Capacity (cfs)
1	Tributary B at Unser Frontage	3 – 10' W x 8' H CBC	B_108_J	915	2,470
2	Tributary B at Northwest Loop	1 – 11' DIA CPM	B_108_Pond	1,042	1,280
3	Tributary B at US 550	3 – 8' W x 6' H CBC	Tributary_B	490	1,460
4	Tributary A at US 550	Bridge	Tributary_A	2,965	n/a*

* Capacity not analyzed because minimal flow constriction occurs at this location

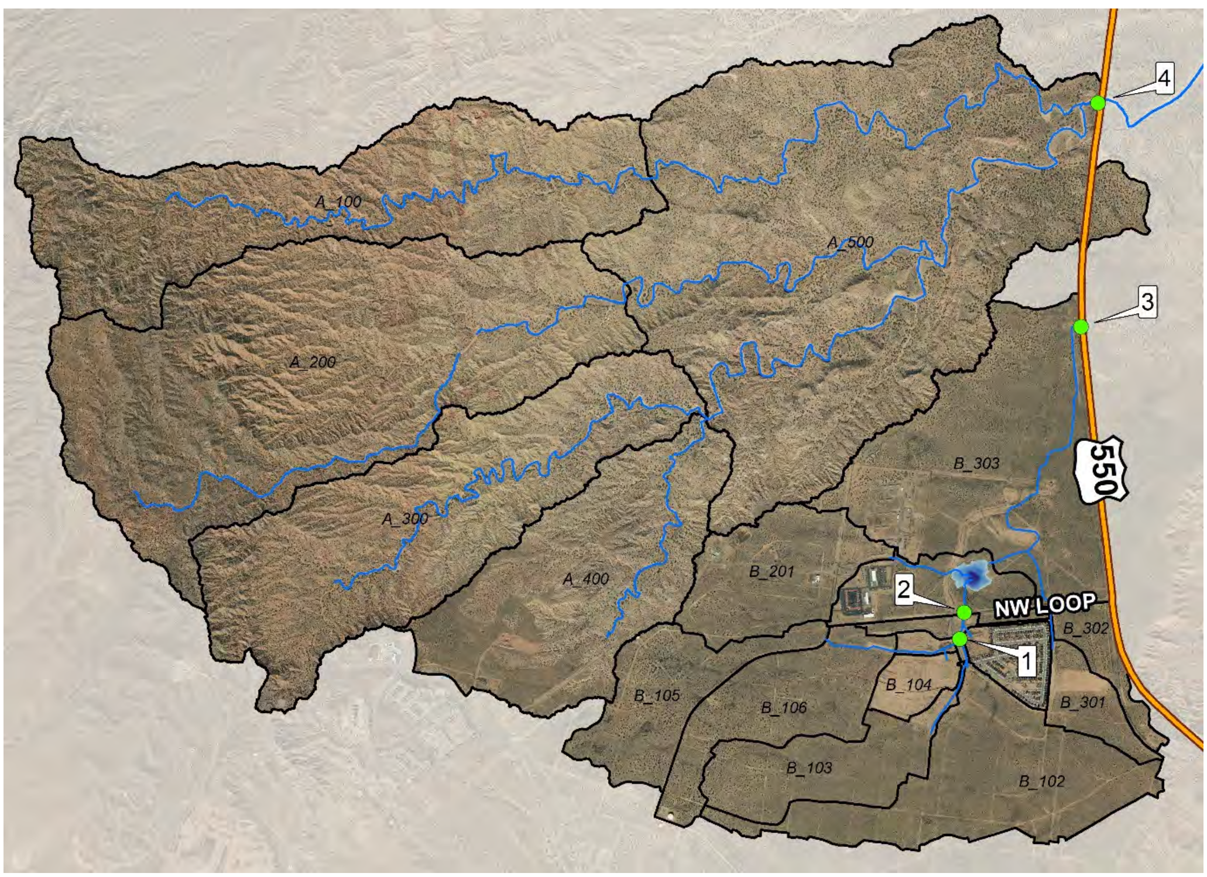

Figure 2.9: Map of crossing structure locations in the Zia watershed.

2.12. Lateral Erosion Envelope

Lateral Erosion Envelopes (LEE) for major arroyos within the Zia Watershed Park were delineated in accordance with the procedures described in SSCAFCA's Sediment and Erosion Design Guide (Mussetter, 2008). The LEE represent the maximum lateral migration distance of an arroyo that can be expected over the next 30-50 years and identifies a corridor where properties and infrastructure are potentially at risk from erosion. The LEE was mapped in *ArcGIS* by calculating the expected maximum lateral erosion distance for each reach and applying a buffer zone of corresponding width on either side of the arroyo (see Appendix D). Discharge rates for individual reach was estimated as the fraction of the total subbasin runoff based on percent contributing area draining to a particular reach.

The LEE does not predict the future course of an arroyo, nor does it guarantee that the arroyo will remain in its limit. The purpose of the LEE is to identify areas in the proximity of major arroyos that are at higher risk from erosion damage.

No major incised natural channels currently exist in tributary B. In tributary A, lateral erosion envelopes were only delineated for basins A_300 and A_400; the remainder of the area is at present not adjacent to urban development. LEE for the remaining basins may be delineated at a future date. Please note that there may be other areas at risk that are not identified in this document, particularly along smaller tributaries.

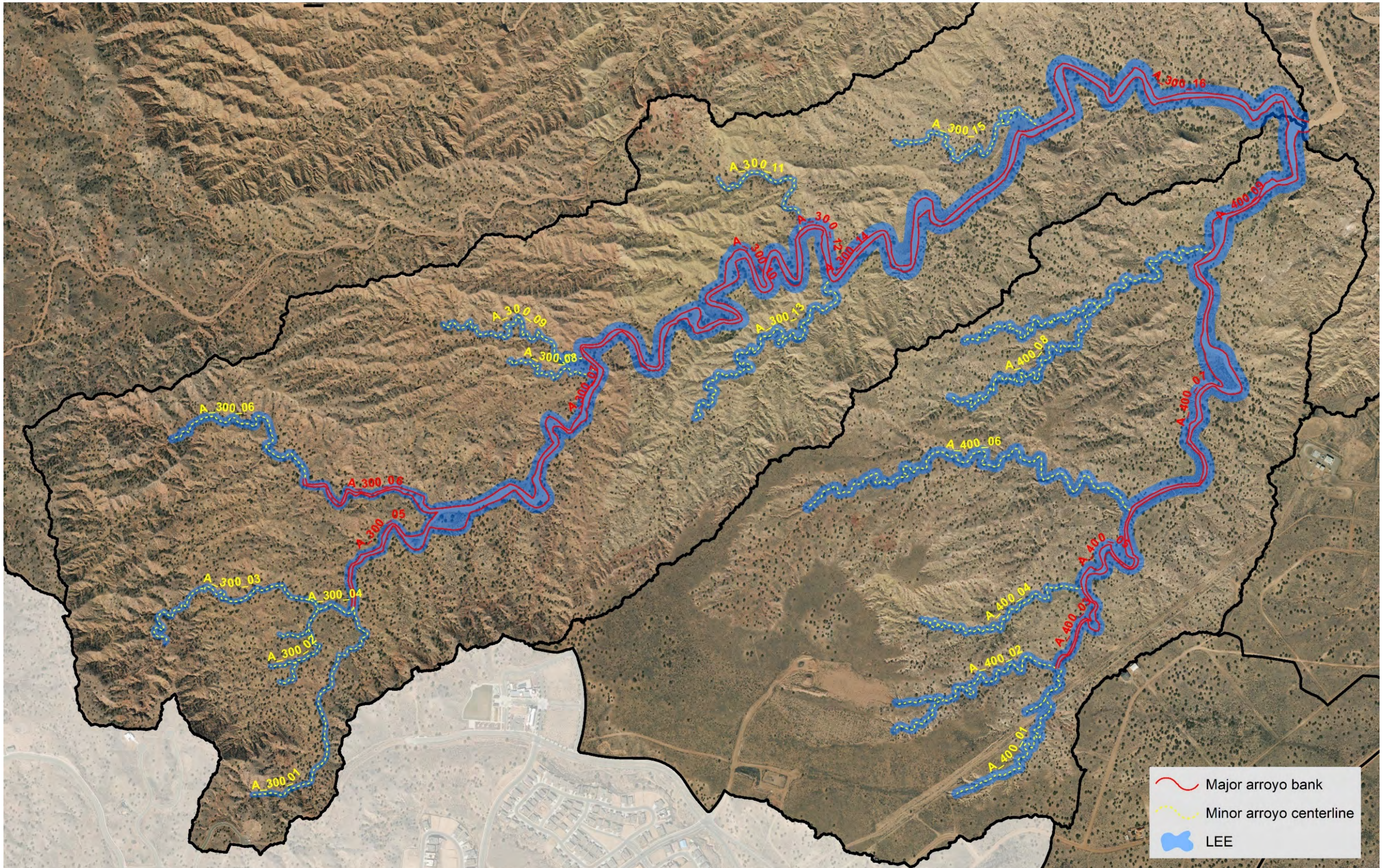

Figure 2.10: Bank line of major arroyos (red), centerlines of minor arroyos (dashed yellow) and lateral erosion envelopes (shaded blue) for Zia subbasins A_300 and A_400.

2.13. References

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USDA (U.S. Department of Agriculture) (1986). Urban hydrology for small watersheds. Natural Resources Conservation Service, Technical Release 55, Washington, D.C.

Appendix A

Model Parameters

Subbasin Parameters

Subbasin Parameters			Loss Model Parameters		Unit Hydrograph Parameters		
Basin ID	Area		Existing Conditions	Existing Cond. Impervious	Time of concentration	Lag [a]	Sediment Bulking Factor
	(ac)	(mi^2)	(CN)	(%)	(h)	(min)	
A_100	803	1.254	75	0.0	0.96	35	1.18
A_200	1257	1.964	75	0.0	0.88	32	1.18
A_300	763	1.192	75	0.0	0.94	34	1.18
A_400	534	0.834	76	0.0	0.54	20	1.18
A_500	1683	2.629	75	0.2	1.16	42	1.18
B_101	59	0.093	88	17.7	0.31	11	1.06
B_102	352	0.550	75	0.2	0.64	23	1.06
B_103	183	0.285	75	1.7	0.45	16	1.06
B_104	46	0.072	86	0.2	0.24	9	1.06
B_105	184	0.287	75	0.8	0.40	14	1.06
B_106	206	0.321	75	0.9	0.35	13	1.06
B_107	13	0.020	86	0.0	0.17	6	1.06
B_108	23	0.036	78	25.1	0.30	11	1.06
B_201	205	0.320	76	1.0	0.34	12	1.06
B_202	125	0.196	78	13.1	0.35	13	1.06
B_301	69	0.107	77	1.8	0.42	15	1.06
B_302	56	0.088	75	6.5	0.37	13	1.06
B_303	561	0.876	76	1.3	1.01	36	1.06

[a] Lag = 0.6 * Time of concentration

Routing Parameters

Routing Reach ID	Length	Slope	Manning's n	Shape	Diameter	Width	Side Slope	Channel Loss
	(ft)	(ft/ft)	( )		(ft)	(ft)	(xH : 1V)	(cfs/ac)
A_500_R1	11840	0.012	0.020	Trapezoid		30	2	0
A_500_R2	12744	0.011	0.020	Trapezoid		30	2	0
A_500_R3	8830	0.010	0.020	Trapezoid		60	6	0
A_500_R4	18386	0.011	0.020	Trapezoid		40	6	0
B_106_R	1047	0.017	0.030	Trapezoid		10	6	0
B_107_R1	1192	0.005	0.013	Circle	7.5			0
B_107_R2	384	0.005	0.013	Circle	7.5			0
B_107_R3	840	0.005	0.013	Circle	5.0			0
B_108_R	625	0.014	0.013	Trapezoid		10	3	0
B_202_R1	780	0.026	0.030	Trapezoid		15	5	0
B_202_R2	1801	0.025	0.030	Trapezoid		10	3	0
B_303_R1	955	0.005	0.030	Trapezoid		60	10	0
B_303_R2	1244	0.015	0.030	Trapezoid		60	10	0
B_303_R3	6358	0.010	0.030	Trapezoid		25	10	0

Appendix B

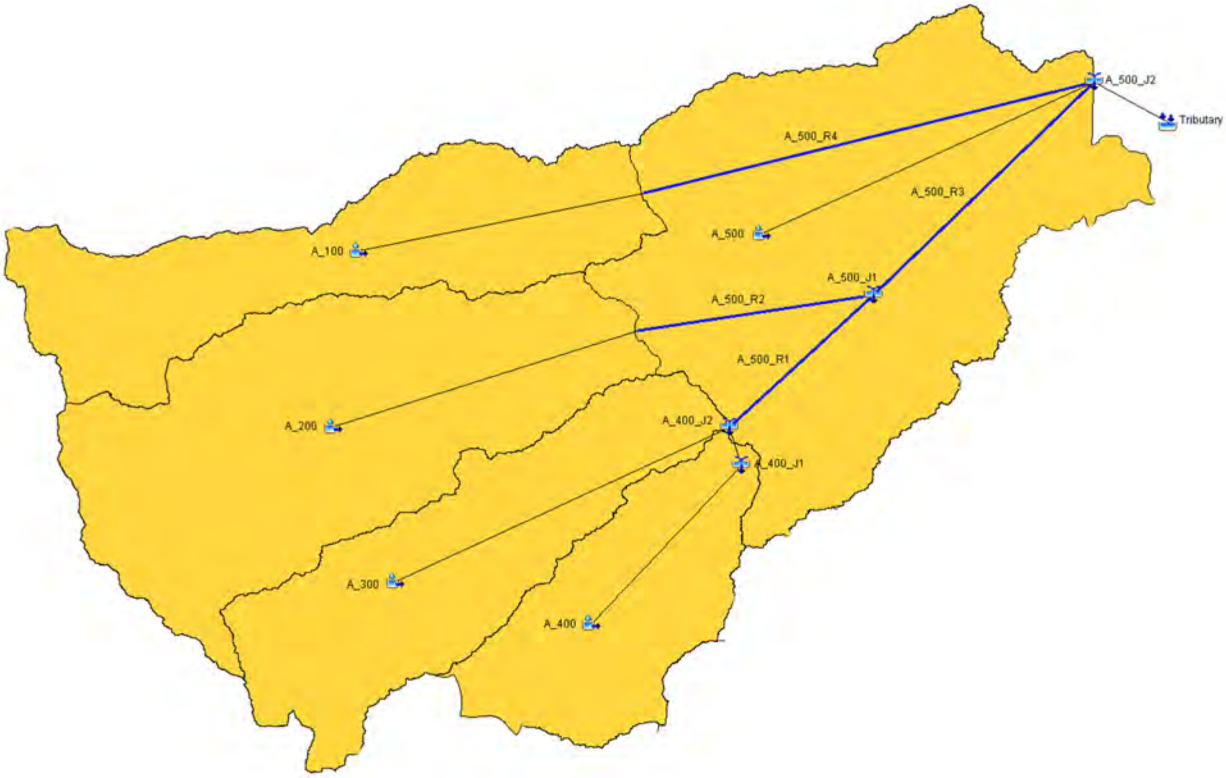
Ponds

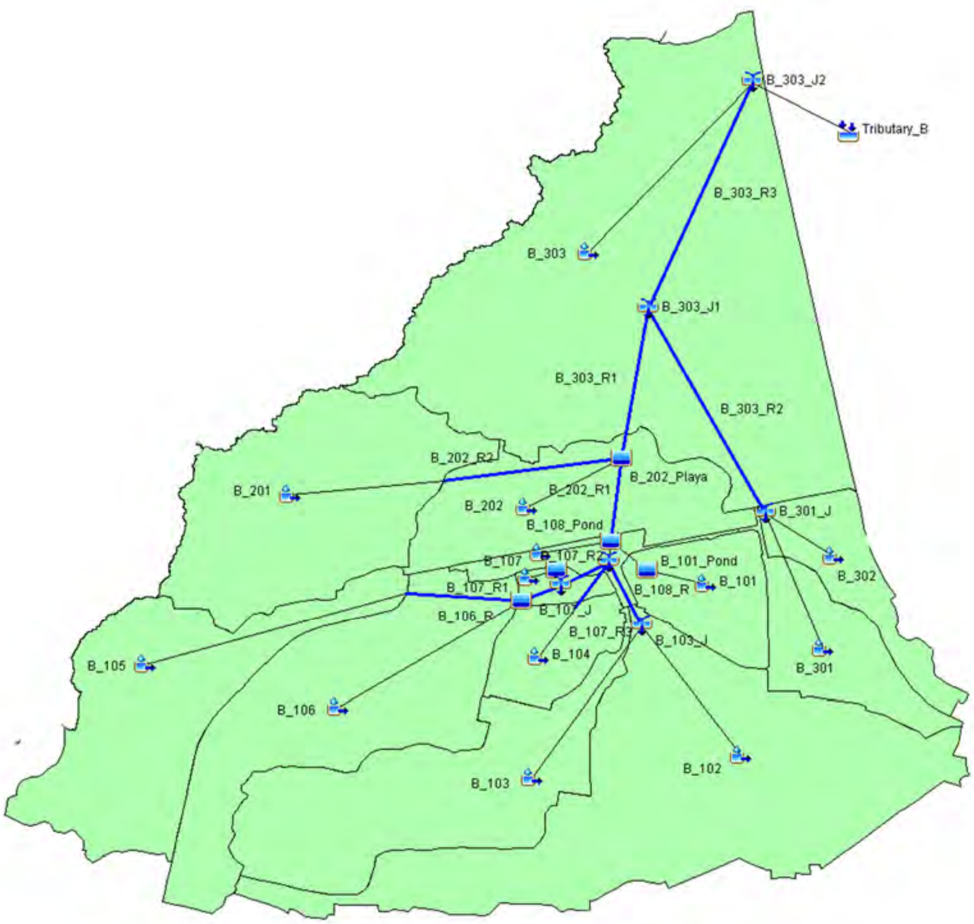
Existing Ponds

Hydro ID	Data Source	Emergency Spillway Elevation (ft)	Storage Volume @ Emergency Spillway	Top of Embankment Elevation (ft)	Storage Volume @ Top of Embankment (ac-ft)	Peak Storage (ac-ft)
B_101_Pond	GIS / field survey	n/a	n/a	5596	8.8	2.7
B_106_Pond	GIS / field survey	n/a	n/a	5612	2.2	0.1
B_107_Pond	GIS / field survey	n/a	n/a	5604	4.4	0.7
B_108_Pond	GIS / field survey	n/a	n/a	5698	12.5	8.1

Appendix C

Design Storm Model Results


A_500_J2
Tributary A
A_500_R4
A_500_R3
A_100
A_500
A_500_J1
A_500_R2
A_500_R1
A_200
A_400_J2
A_400_J1
A_300
A_400


B_303_J2
Tributary_B
B_303_R3
B_303
B_303_J1
B_303_R1
B_303_R2
B_202_R2
B_202_Playa
B_201
B_202_R1
B_202
B_301_J
B_108_Pond
B_107
B_107_R2
B_101_Pond
B_302
B_107_R1
B_108_R
B_101
B_106_R
B_107_J
B_107_R3
B_103_J
B_105
B_104
B_301
B_106
B_102
B_103

Notes:

(1) Model results reported in this table are for the 100-year design storm using no depth-area reduction factor. **Please modify the storm area** in the HEC-HMS model for analyses with larger contributing areas.

(2) Model results area for intermediate moisture conditions (CN=74 for open space, landscaping and residential yards).

(3) Q_p and V values for ponds correspond to peak outflow and outflow volume, respectively. For detailed pond routing including peak inflow, peak storage and peak elevation values, please consult the HEC-HMS model.

Existing Conditions			
HMS ID	Area (mi^2)	Q_p (cfs)	V (ac-ft)
A_100	1.254	520	69.1
A_200	1.964	861	108.1
A_300	1.192	504	65.7
A_400	0.834	540	49.0
A_400_J1	0.834	540	49.0
A_400_J2	2.025	930	114.7
A_500	2.629	950	144.6
A_500_J1	3.989	1740	221.6
A_500_J2	7.873	3115	433.5
A_500_R1	2.025	928	114.1
A_500_R2	1.964	860	107.5
A_500_R3	3.989	1738	220.6
A_500_R4	1.254	519	68.3

Existing Conditions			
HMS ID	Area (mi^2)	Q_p (cfs)	V (ac-ft)
B_101	0.093	179	10.1
B_101_Pond	0.093	93	10.0
B_102	0.550	274	27.5
B_103	0.285	184	14.8
B_103_J	0.835	435	42.3
B_104	0.072	127	6.4
B_105	0.287	196	14.6
B_106	0.321	230	16.4
B_106_Pond	0.608	423	30.8
B_106_R	0.287	196	14.6
B_107	0.020	42	1.8
B_107_J	0.628	431	32.3
B_107_Pond	0.020	13	1.6
B_107_R1	0.608	418	30.8
B_107_R2	0.628	430	32.3
B_107_R3	0.072	126	6.4
B_108	0.036	51	3.1
B_108_J	1.535	915	81.0
B_108_Pond	1.664	914	94.0
B_108_R	0.835	434	42.3
B_201	0.320	259	17.3
B_202	0.196	212	14.2
B_202_Playa	2.180	277	61.5
B_202_R1	1.664	913	94.0
B_202_R2	0.320	258	17.3
B_301	0.107	84	6.2
B_301_J	0.195	154	11.2
B_302	0.088	71	5.0
B_303	0.876	355	47.2
B_303_J1	2.375	293	72.5
B_303_J2	3.251	500	118.9
B_303_R1	2.180	277	61.3
B_303_R2	0.195	153	11.2
B_303_R3	2.375	292	71.7

Appendix D

Lateral Erosion Envelop Calculations

Reach	% of Contributing Area	EXISTING Q_{100} [a]	Dominant Discharge Q_d	Slope S_0	Critical Slope S_c	Maximum lateral erosion distance Δmax	Est. channel width W_D	Offset
		(cfs)	(cfs)	(ft/ft)	(ft/ft)	(ft)	(ft)	(ft)
A_300_01	4%	21	4	0.049	0.031	21	8	25
A_300_02	4%	19	4	0.034	0.031	19	8	23
A_300_03	5%	25	5	0.040	0.030	22	9	26
A_300_04	9%	45	9	0.029	0.028	28	11	33
A_300_05	18%	93	19	0.024	0.025	37	15	45
A_300_06	13%	66	13	0.033	0.026	32	13	39
A_300_07	40%	202	40	0.022	0.023	51	20	61
A_300_08	4%	22	4	0.026	0.030	22	9	26
A_300_09	5%	24	5	0.025	0.030	22	9	27
A_300_10	59%	298	60	0.015	0.021	63	25	76
A_300_11	3%	18	4	0.030	0.031	19	8	23
A_300_12	63%	319	64	0.016	0.021	64	26	77
A_300_13	10%	52	10	0.025	0.027	30	12	36
A_300_14	81%	411	82	0.014	0.021	72	29	87
A_300_15	5%	26	5	0.026	0.030	23	9	27
A_300_16	100%	504	101	0.015	0.020	77	31	93
A_400_01	17%	90	18	0.022	0.025	37	15	45
A_400_02	11%	60	12	0.023	0.027	32	13	38
A_400_03	29%	158	32	0.022	0.023	46	18	56
A_400_04	7%	36	7	0.026	0.028	26	10	31
A_400_05	40%	214	43	0.026	0.022	52	21	62
A_400_06	21%	115	23	0.026	0.024	40	16	48
A_400_07	78%	423	85	0.020	0.021	69	27	82
A_400_08	13%	69	14	0.029	0.026	33	13	39
A_400_09	100%	540	108	0.023	0.020	75	30	90

[a] estimated by scaling subbasin discharge using % of contributing area

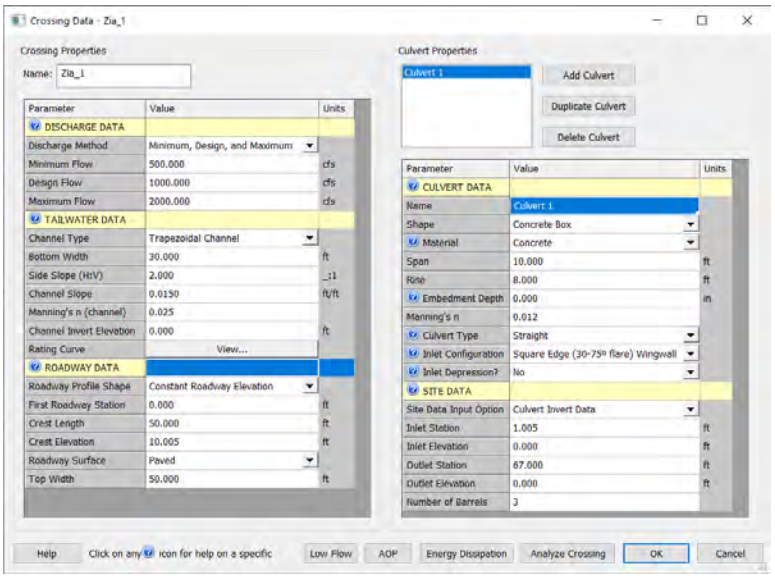
Appendix E

Existing Structure Capacities

Appendix E contains capacity analyses of culvert crossings in the Zia watershed for the 100-year storm event. Please note that this analysis was performed for planning purposes only to establish approximate maximum allowable flow rates at each location. Culvert dimensions were measured during a field visit on 10/01/2018. Capacities were estimated using HY-8 software version 7.5. The analysis was based on the following assumptions:

- Culverts are free of sediment and debris unless otherwise noted in the data tables; actual capacities may be less than those reported due to sediment accumulation, vegetation, and debris caught at culvert entrances.
- For simplicity, downstream channels were assumed to be trapezoidal with a bottom width and slope equal to that of the culvert crossing and a Manning's value of 0.025.
- Overtopping of roadways was not modeled in HY-8. Maximum capacities correspond to maximum upstream water levels before flow starts overtopping the road or break out of the channel upstream of the crossing.

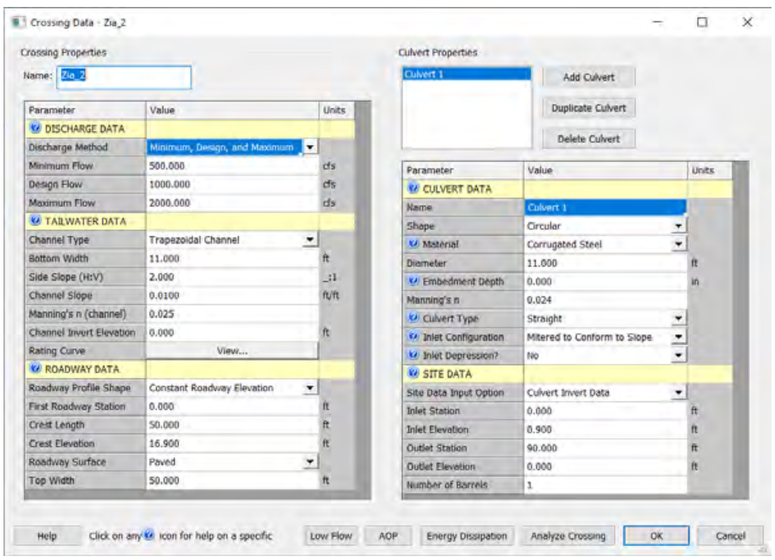
Tributary B at Unser Frontage (crossing 1)



Summary of Flows at Crossing - Zia_1

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
3.47	500.00	500.00	0.00	1
4.12	650.00	650.00	0.00	1
4.73	800.00	800.00	0.00	1
5.30	950.00	950.00	0.00	1
5.48	1000.00	1000.00	0.00	1
6.36	1250.00	1250.00	0.00	1
6.85	1400.00	1400.00	0.00	1
7.33	1550.00	1550.00	0.00	1
7.80	1700.00	1700.00	0.00	1
8.25	1850.00	1850.00	0.00	1
8.68	2000.00	2000.00	0.00	1
10.01	2474.09	2474.09	0.00	Overtopping

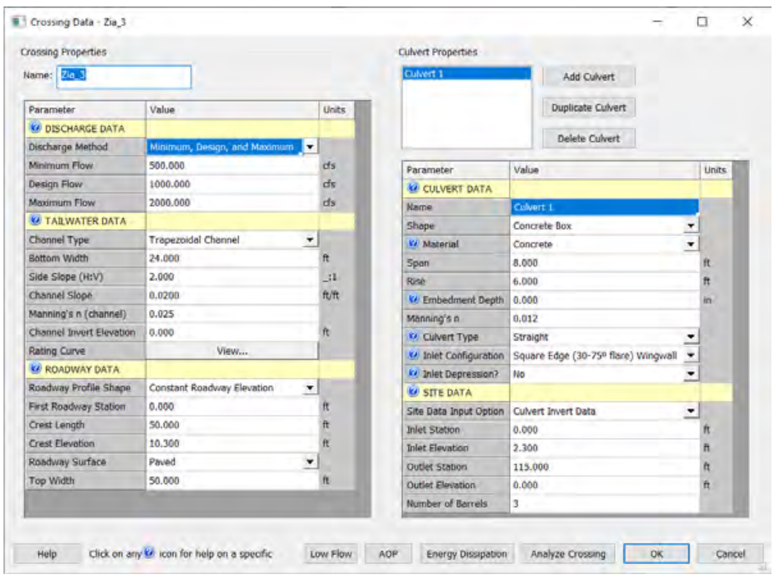
Tributary B at Northwest Loop (crossing 2)



Summary of Flows at Crossing - Zia_2

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
8.59	500.00	500.00	0.00	1
9.87	650.00	650.00	0.00	1
11.20	800.00	800.00	0.00	1
13.46	950.00	950.00	0.00	1
13.84	1000.00	1000.00	0.00	1
16.40	1250.00	1250.00	0.00	1
17.52	1400.00	1326.19	73.65	5
18.06	1550.00	1361.43	188.51	4
18.51	1700.00	1390.38	309.58	4
18.92	1850.00	1415.81	434.16	4
19.29	2000.00	1438.85	560.74	3
16.90	1284.55	1284.55	0.00	Overtopping

Tributary B at US 550 (crossing 3)



Crossing Data - Zia_3

Crossing Properties

Name: Zia_3

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	500.000	cfs
Design Flow	1000.000	cfs
Maximum Flow	2000.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	24.000	ft
Side Slope (H:V)	2.000	_:1
Channel Slope	0.0200	ft/ft
Manning's n (channel)	0.025	
Channel Invert Elevation	0.000	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	50.000	ft
Crest Elevation	10.300	ft
Roadway Surface	Paved	
Top Width	50.000	ft

Culvert Properties

Culvert 1

Add Culvert

Duplicate Culvert

Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1	
Shape	Concrete Box	
Material	Concrete	
Span	8.000	ft
Rise	6.000	ft
Embedment Depth	0.000	in
Manning's n	0.012	
Culvert Type	Straight	
Inlet Configuration	Square Edge (30-75° flare) Wingwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	2.300	ft
Outlet Station	115.000	ft
Outlet Elevation	0.000	ft
Number of Barrels	3	

Help | Click on any icon for help on a specific | Low Flow | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel

Summary of Flows at Crossing - Zia_3

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
5.92	500.00	500.00	0.00	1
6.62	650.00	650.00	0.00	1
7.28	800.00	800.00	0.00	1
7.92	950.00	950.00	0.00	1
8.13	1000.00	1000.00	0.00	1
9.24	1250.00	1250.00	0.00	1
9.97	1400.00	1400.00	0.00	1
10.62	1550.00	1523.21	26.59	4
11.05	1700.00	1601.37	98.56	4
11.43	1850.00	1667.22	182.74	4
11.78	2000.00	1725.46	274.10	3
10.30	1463.89	1463.89	0.00	Overtopping

Tributary A at US 550 (crossing 4)

The capacity of crossing 4 was not analyzed because minimal flow constriction occurs at this location.