DRAINAGE REPORT – STAGE 4
FOR
VALENCIA ROAD : WADE ROAD TO MARK ROAD
(PIMA COUNTY WO # 4RTVMW)
PIMA COUNTY, ARIZONA

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September 2012

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ARIZONA U.S.A.

EXPIRES 12-31-2013

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HYDROLOGY & GEOMORPHOLOGY, INC.

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I. INTRODUCTION

1.1 Project Description

The Pima County Department of Transportation (PCDOT) contracted with HDR Inc. (HDR) to provide roadway design services for two segments of Valencia Road between Mountain Eagle Drive and Mark Road (see Figure 1). The project reach occupies portions of Section 8-17 of Township 15 South, Range 12 East, Gila and Salt River Meridian, Pima County, Arizona. The PCDOT work order numbers associated with these two projects are # 4RTVWE, which applies to the segment between Mountain Eagle Drive and Wade Road, and # 4RTVMW, which applies to the segment between Wade Road and Mark Road (see Figure 2).

HDR contracted with JE Fuller/Hydrology and Geomorphology, Inc. (JEF) to address the drainage requirements associated with the two projects. This drainage report documents the Stage I drainage design requirements, as outlined in Reference 1, for the segment from Wade Road to Mark Road (4RTVMW). However, the actual project reach extends from Star Diamond Place, which is approximately 1800 feet west of Wade Road, to a point on Valencia Road located approximately 1000 feet east of S. Ignacio M. Baumea where Valencia Road has already been constructed with a five-lane section and sidewalk on the south side of the road.

1.2 Major Drainage Features

This segment of Valencia Road is subject to shallow-sheet flooding from watersheds extending south to the Sierrita Mountains. All cross drainage is across the existing Valencia Road pavement and there are no existing cross-drainage structures within the study reach. The proposed drainage design scheme will use a series of cross-drainage structures to accommodate offsite runoff. The roadway will be designed to accommodate cross-drainage through a combination of raising the existing profile, and depressing the inlets of the cross-drainage structures to accommodate the required flows.

1.3 Proposed Improvements

Cross-drainage structures will be used to capture and convey offsite runoff beneath the new roadway section. The structures will consist of reinforced concrete box culverts (RCBC) and spiral rib steel pipe (SRSP). Drop inlets will be used where needed to accommodate the proposed roadway profile.

1.4 Design Criteria

Each cross-drainage structure was designed to accommodate the 100-year peak flow collecting between it and the next upslope cross-drainage structure to the east. Where needed, spur dikes are provided at the culvert inlets to help convey flow into the culvert structure.

Pavement drainage design criteria will be in accordance with the criteria outlined in Chapter 2 of the Pima County Roadway Design Manual (Reference 1). No storm drains are proposed as part of the improvements. During the 10-year event, 1 lane equivalent width of pavement will be

kept clear of drainage in each direction. During the 100-year event, flowing or ponded water will not exceed one foot in depth anywhere within the paved section and flow will not be allowed to overflow to adjacent basins.

II. EXISTING CONDITIONS

2.1 Overview

This segment of Valencia Road is impacted by runoff originating from watersheds located to the south, including the Black Wash watershed, which extends onto the San Xavier District of the Tohono O'Odham reservation located south of Hermans Road. The general drainage pattern in this area is from the south-southeast to the north-northwest and the majority of the roadway is impacted by shallow sheet-flow. Developed subdivisions located upstream of the roadway, including Star Valley and Star Valley Villages (see Figure 2), were designed to capture offsite runoff in a series of detention/retention basins and collector channels. Generally speaking, flows conveyed in these channels were redistributed to the maximum extent possible to approximate pre-developed flow conditions at the downstream end of the subdivisions. In addition, approximately 1.5 miles of the south frontage of Valencia Road, between Wade Road and the Casino del Sol, is undeveloped state land. Existing development downstream of the roadway consists primarily of parcels which are not part of a formal subdivision process with little to no drainage improvements except for a single cell arch concrete culvert crossing of the Black Wash on Camino Verde approximately 800 feet north of Valencia Road.

Figure 3 shows the project alignment on the Flood Insurance Rate Map (FIRM) for the area (Panel #2265L, Revised June 16, 2011). As shown on Figure 3, the entire project alignment is located within floodprone areas, primarily Zones AO1, AO2 and AO3. These zones are approximate and, as such, the Flood Insurance Study (FIS) does not list flood discharges for this area. A Conditional Letter of Map Revision (CLOMR) will be needed for the project prior to construction, with a Letter of Map Revision (LOMR) needed upon completion of construction.

Existing and projected land-use in the four major watersheds that currently impact the study reach ranges from rural to medium-intensity urban. However, the majority of the watershed area is located in the rural category. Therefore, future development will have little impact on runoff volumes or magnitudes. Existing and projected land-use in the local watersheds ranges from low-intensity rural to medium intensity urban, with the majority being within the latter category.

The majority of natural vegetation within both the local and major watershed is in the desert brush or scrub vegetation type. From a hydrologic standpoint, soils in the four major watersheds range from 100% Group D to 100% Group B, with the majority being in the latter group. Soils in the local watersheds are classified as Hydrologic Group B, C and D.

2.2 Existing Conditions Analysis

The existing conditions hydrologic and hydraulic analysis was conducted using FLO-2D, Version 2009.06 (Reference 2). The FLO-2D software package was used to evaluate both rainfall and runoff within the boundary of the study area (i.e., the contributing watershed area) and the project area. FLO-2D was selected to model the study area, since the lack of definition within the contributing watershed and project area does not lend to the use of traditional lumped-parameter methods such as HEC-1 or HEC-HMS, and the FLO-2D model offers a greater level of detail in modeling the broad shallow flow from the upstream contributing watershed. The total model area measures approximately 37 square miles and is shown on Figure 4. Table 2.1 summarizes the data and parameters used to develop the model.

Table 2.1 - FLO-2D Modeling Parameters

Parameter/Data	Description
Topographic Data	PAG 2008 DEM data. Data processing included elevation resampling to reduce data density.
FLO-2D Grid	100-foot grid developed from PAG 2008 DEM data using FLO-2D GDS program. Data adjusted where needed to eliminate ponding grids. 103,992 total grids in model.
Rainfall Data	NOAA14 Upper 90% confidence interval rainfall data was used (Reference 3). Two rainfall distributions were modeled separately as follows; • 24-hour storm; SCS Type-1 rainfall distribution per Tech Policy 018 (Reference 4). Total Storm Depth = 4.55 inches • 3-hour storm; SCS 3-hour distribution per Reference 4. Total Storm Depth = 3.34 inches As a conservative measure, aerial reduction was not applied.
Soils Data	NRCS soil survey data (Reference 5) as found in Pima County GIS shape file soilshyd.shp.
SCS Curve Number	The SCS option available within FLO-2D was utilized to convert rainfall to runoff. SCS curve numbers were determined using shape files containing information on (1) hydrologic soil group (HSG) from the Pima County GIS soilshyd.shp file, which contains soils information from NRCS Soil Survey #AZ669, (2) vegetative cover type (from Arizona Game & Fish data), and (3) impervious cover from county land use data. The resulting CN values ranged from approximately 77 to 96, depending on HSG, cover and land use.
Roughness Coefficient	A roughness coefficient of 0.035 was used with adjustment in isolated locations to aid in model stability.
Structures	A limited number of channel and culvert structures were included in the model to reflect drainage improvements associated with the Star Valley development.
Special Conditions	As indicated previously the Black Wash is poorly defined along much of its length. As a result flow often spreads over wide areas. At two particular locations in the model area, flow spreads out and leaves the model area without contributing to the project reach of Valencia Road. As a conservative measure, flow blocks were built into the FLO-2D model (using the levee option) to prevent this flow from leaving the model.

2.3 Summary of Existing Conditions

The results of the existing condition FLO-2D model are shown on Exhibit 1.

Exhibit 1 shows the 100-year flow depths within the model area ranging from zero (0) to over three (3) feet in a color-coded format. Exhibit 1 also shows the quantities of flow that cross the existing roadway over the lengths indicated by the heavy black lines, above which the discharge values appear. Because modeling was performed for both the 3-hour and 24-hour storms, both discharge values are shown at each location. It was determined that maximum flow depths and discharges were produced by the 3-hour storm in some locations, while in others, particularly those locations inundated by the main stem of the Black Wash, the 24-hour storm produced the greatest flood depths and discharges. Because of this dichotomy, both discharge values are shown on Exhibit 1. The flow depths shown on Exhibit 1 are a composite of both the 3-hour and 24-hour models wherein the greatest flow depth associated with the two models is shown at any given point.

III. PROPOSED CROSS DRAINAGE IMPROVEMENTS

3.1 Offsite Drainage Approach

The following approach was taken to the offsite drainage;

1. Identify logical points for cross-drainage structures based on (a) existing dips and crossing points in the roadway, (b) flow patterns from Exhibit 1 which identify flow crossings and (c) existing channels and/or thalwegs available on the downstream (north) side of the roadway to accept discharges from the cross-drainage structures. This was performed as an iterative process through coordination between HDR and JE Fuller.
2. Determine the approximate quantities of flow from the FLO-2D model that collects at the crossing points, as determined in (1) above. The discharges shown on Exhibit 1 were determined based on the crossing points determined in (1) above such that the discharges shown reflect the quantity of flow crossing the roadway between each proposed drainage crossing location.
3. Perform hydraulic analysis of conceptual cross-drainage structures using HY-8 (Reference 6) to determine a size and configuration that would convey the flow determined in (2) above at a headwater elevation similar to that which occurs under existing conditions (based on review of the existing condition FLO-2D model). HEC-RAS (Reference 9) was used to determine tailwater conditions at selected crossings.
4. Develop a proposed condition FLO-2D model. This was done by developing a stage-discharge rating table for each of the crossing structures determined in (3) from the HY-8 rating and entering that cross-drainage structure into the existing condition FLO-2D model. The proposed condition FLO-2D model also reflects adjusted elevations for the proposed Valencia Road and Wade Road design profiles and, where applicable, elevated wing dikes on the upstream side of each proposed cross-drainage structure, which force the required headwater to drive the design discharge for the culvert.

Offsite drainage will be collected at the key concentration points and conveyed beneath the roadway in reinforced concrete box culverts (RCBC) and spiral rib steel pipe (SRSP) of varying sizes and configurations. In two locations SRSP crossings will be used for the express purpose of maintaining flow to existing downstream vegetated wash segments. To minimize changes in the roadway profile, drop inlets will be provided. Rock riprap aprons or pre-formed plunge basins have been sized for the culvert outlets to dissipate energy and minimize erosion.

3.2 Proposed Conditions Analysis

Table 3.1, following page, summarizes the results of the selected cross-drainage structures determined per the process described in Section 3.1 above. More-detailed output from the HY-8 analysis is provided in Appendix C.

The culverts listed in Table 3.1 were entered into the proposed condition FLO-2D model, along with the proposed Valencia Road profile elevations and associated wing dike elevations at each culvert. Table 3.2, below, provides summary information on the wing dikes included in the model.

Table 3.1 - Summary of Hydraulic Design of Selected Culverts

Roadway Station	Existing FLO-2D Discharge (cfs)	Design Structure Discharge (cfs)	Structure Type	Span (feet)	Rise (feet)	Length (feet)	# Cells	Hydraulic Control	Outlet Invert Elev	Inlet Invert Elevation	Design Headwater Elev.	Design Headwater Depth (ft)	Approx. Existing WSEL
232+80	151	141	SRSP	NA	3	116	2	Inlet	2475.00	2475.60	2482.28	6.68	2482.14
237+30	353	302	RCBC	10	5	112	2	Outlet	2476.00	2476.60	2482.06	5.46	2483.51
241+82	NA	41	SRSP	NA	3	135	1	Inlet	2481.00	2482.00	2485.69	3.68	2485.57
253+66	312	196	RCBC	10	5	145	4	Inlet	2483.00	2484.20	2487.66	3.45	2489.40
257+80	NA	66	SRSP	NA	3	262	1	Outlet	2485.00	2486.50	2492.22	5.72	2491.78
278+40	82	84	RCBC	8	4	137	1	Inlet	2491.50	2492.20	2496.25	4.05	2497.68
281+75	103	106	SRSP	NA	4	135	1	Inlet	2492.30	2493.50	2499.02	5.52	2498.93
289+55	162	158	RCBC	6	4	119	1	Inlet	2494.60	2496.30	2500.85	4.55	2500.52
296+70	1962	1952	RCBC	10	5	169	9	Inlet	2495.50	2497.50	2502.85	5.35	2503.14
310+50	1027	1046	RCBC	10	5	136	9	Inlet	2504.50	2505.20	2508.76	3.56	2509.11

Table 3.2 – Wing Dike Summary

Roadway Station	Top-of-Dike Elev. (NAVD)	Approximate Lateral Extent to South of Culvert Headwall	Notes
232+80	NA		No wing dike needed
237+30	2484.0	70 feet	
241+82	NA		No wing dike needed
253+66	NA (see notes)		Assumes elevation of Wade Road to provide flow containment at culvert
257+80	NA		No wing dike needed
278+40	2496.8	130 feet	
281+75	NA		No wing dike needed
289+55	NA		No wing dike needed
296+70	2504.0	300 feet	
310+50	2509.0	300 feet	Alignment jogs east at south end

The results of the proposed condition FLO-2D model are shown on Exhibit 2. Comparison of Exhibit 2 with Exhibit 1 shows that flood depths are increased in some areas and decreased in others as a result of collecting and conveying flow across the roadway at discreet locations rather than in sheet fashion as currently occurs. Because the entire project alignment is within existing federally mapped flood zones, no mitigation is to be provided.

Two alternative outlet protection structures were considered as part of the design analysis – a horizontal apron and a pre-formed plunge basin. Both structures can be constructed using dumped rock riprap. The apron is the easiest to construct, but typically requires a larger D_{50} rock diameter and footprint (see Appendix D). The design parameters for a horizontal apron or blanket are summarized in Table 3.3. The design parameters for a pre-formed plunge basin are summarized in Table 3.4.

Table 3.3 - Outlet Protection: Riprap Apron or Blanket Alternative

Roadway	Design Q	D50	Dimensions, in feet (see Appendix D)			
Station	(cfs)	(ft)	k	Lsb	W1	W2
232+80	141	0.7	5	41	14	30
237+30	302	0.2	5	20	31	39
241+82	41	0.6	2	35	9	44
253+66	196	0.1	2	39	53	92
257+80	66	0.9	2	44	9	53
278+40	84	0.2	5	16	16	22
281+75	106	0.6	5	40	12	28
289+55	158	0.6	5	40	14	30
296+70	1952	0.2	5	29	108	120
310+50	1046	0.2	2	44	108	152

Note: 1. Variable 'k' is the flare ratio (i.e., 2:1 for min. & 5:1 for max. tailwater.)
2. W1 is upstream width at culvert outlet and W2 is downstream width at length Lsb.

Table 3.4 - Outlet Protection: Riprap Pre-Formed Plunge Basin Alternative

Roadway	Design Q	D50	Dimensions, in feet (see Appendix D)		
Station	(cfs)	ft	hs	L	W
232+80	141	0.5	1.5	18	20
237+30	302	0.1	2.5	30	46
241+82	41	0.4	1.5	18	15
253+66	196	0.1	2.5	30	68
257+80	66	0.5	1.5	18	15
278+40	84	0.1	2	24	28
281+75	106	0.4	2	24	20
289+55	158	0.4	2	24	24
296+70	1952	0.2	2.5	30	123
310+50	1046	0.1	2.5	30	123

1. Plunge basin values for outlet depression (hs) equal to 0.5*Dc.
2. L = 3Dc+6hs and W = 2Wc+6hs, where Dc = culvert height or diameter and Wc = culvert width.

As a practical matter, a minimum rock size of 6” (0.5’) should be used for all outlet installations.

3.4 Sedimentation

Improved inlets at a 2:1 incline, as opposed to vertical drop inlets, will be provided to minimize sedimentation at the culvert inlets. Since the design headwater elevation for each culvert will be at or below the existing water-surface elevation on the upstream side of the roadway, the existing velocity of approach flows will be maintained. During the more-frequent flow events, the velocity of flow through the barrel will exceed the existing velocity of approach flows and the downstream tailwater elevation will be at a minimum; therefore, sedimentation in the culvert barrel or box should not be a significant problem. However, during the less-frequent flow events and during the 100-year event, some sedimentation at the inlet and just inside the barrel or box can be expected and will need to be addressed as part of regular maintenance.

3.5 Channelization

Varying degrees of channelization will be required downstream of all of the proposed culverts. As a result drainage easements will be required downstream of each culvert. As previously noted, some minor grading may be required upstream at the inlets to ensure positive drainage of offsite flows to the inlet structures; however no channelization is required.

3.6 Outlet Protection

Based on a review of existing flow velocities in the vicinity of the proposed outlets and the outlet velocities associated with the proposed structures, as summarized in Table 3.5, outlet protection is warranted at only two locations, per Reference 7. As previously noted in Section 3.3, outlet protection was designed using Reference 8, which is an updated version of Pima County's methodology as outlined in Reference 7. The scour protection guidelines presented in Chapter VI of Reference 7 notes that if the ratio of the outlet velocity to the natural velocity is less than 1.5 outlet protection is not normally required. However, if the ratio is between 1.5 and 2.5 outlet protection is warranted. When the ratio exceeds 2.5, an energy dissipation structure similar to the pre-formed plunge basin is recommended. Although the ratio associated with most of the crossings is less than 1.5, flows discharging from the culverts will directly impact the banks of the downstream sections. Therefore, it is recommended that outlet protection be provided at all culvert outlets.

Table 3.5 - Ratio of Outlet Velocity to Tailwater Velocity

Roadway Station	Design Discharge (cfs)	Flow Velocity Outlet (Vo) (fps)	Flow Velocity Tailwater (Vt) (fps)	Ratio Vo/Vt
232+80	141	10.7	3.6	3.0
237+30	302	4.1	3.3	1.2
241+82	41	9.4	2.6	3.6
253+66	196	7.7	2.3	3.3
257+80	66	10.2	3.1	3.3
278+40	84	8.5	3.0	2.8
281+75	106	12.2	2.4	5.1
289+55	158	13.9	2.1	6.6
296+70	1952	11.2	3.8	2.9
310+50	1046	9.0	2.3	3.9

3.7 Right-of Way Requirements

The right-of-way requirements are addressed under a separate cover.

3.8 Mitigation Measures

Mitigation measures are addressed under a separate cover.

3.9 Permitting

Permitting is addressed under a separate cover.

IV. REFERENCES

1. Pima County Department of Transportation, *Pima County Roadway Design Manual*, Third Edition, 2010.

2. FLO-2D Software, Inc., FLO-2D Reference Manual, Version 2009 and FLO-2D Data Input Manual, Version 2009.06, October, 2009.

3. National Oceanic and Atmospheric Administration (NOAA), National Weather Service, NOAA Atlas 14, Precipitation Frequency Atlas of the United States, U.S. Department of Commerce, 2004.

4. Acceptable Model Parameterization for Determining Peak Discharges, Technical Policy, TECH-018, Pima County Regional Flood Control District, April 2011

5. Natural Resources Conservation Service (NRCS), Soil Survey of Pima County, Arizona, Eastern Part, 2003.

6. U.S. Department of Transportation, Federal Highway Administration, HY-8, Version 7.2.4, Co-developed under FHWA Task Order DTFH61-05-T-63053, Build Date: July 14, 2010.

7. Pima County Department of Transportation and Flood Control District, *Drainage and Channel Design Standards for Local Drainage for Flood Plain Management Within Pima County, Arizona*, Adopted May 15, 1984, Effective June 1, 1984.

8. Grace, John L., U.S. Army Corps of Engineers, Waterways Experiment Station, Erosion Control at Culvert Oulets, paper presented at the ASCE Annual Meeting, Hollywood, FL, October 1980.

9. U.S. Army Corps of Engineers, Hydraulic Engineering Center, *HEC-RAS River Analysis System*, Version 4.1.0, January 2010.

Appendix A

Existing Conditions Model Files (FLO-2D)

Appendix B

Design Conditions Model Files (FLO-2D)

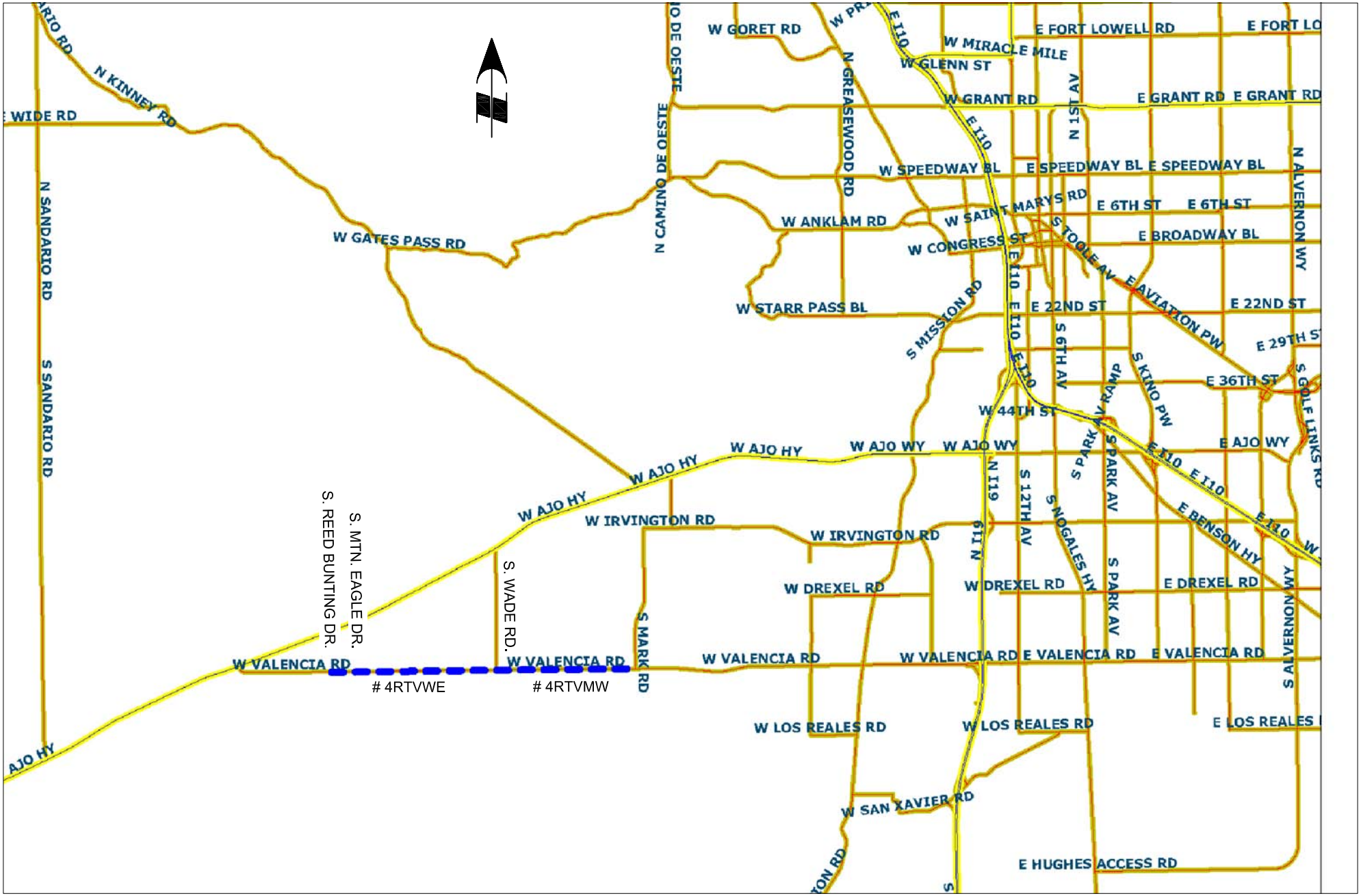
Appendix C

Hydraulic Data for Design Conditions (HY-8 & HEC-RAS)

Appendix D

Outlet Protection Computation Sheets for Design Conditions

- All appendices are provided on CD



10000' 0' 10000' 20000'

SCALE: 1" = 10,000'

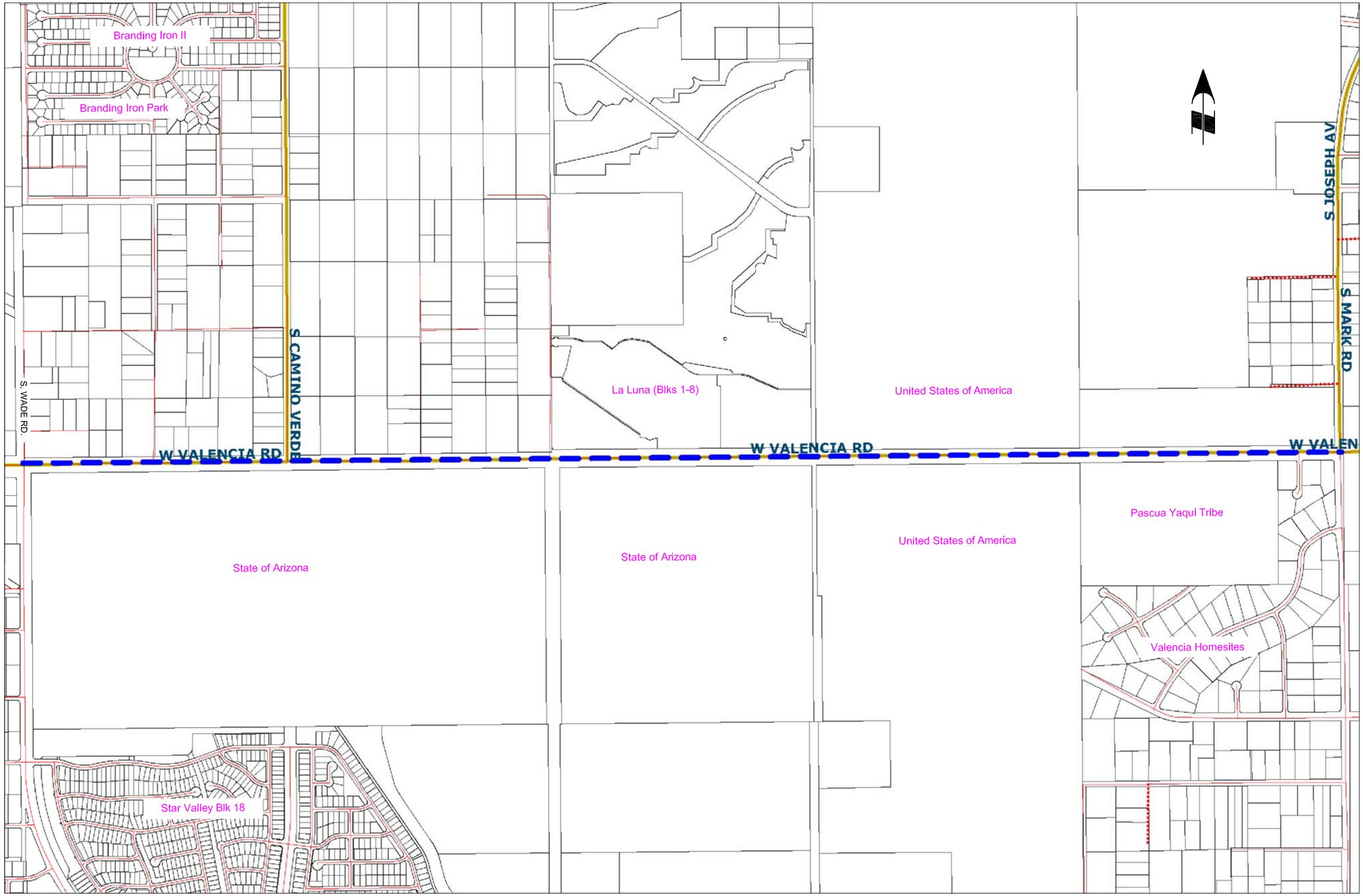
Legend

Project Reach

Figure 1

Vicinity Map

JE Fuller / Hydrology & Geomorphology, Inc.



1000' 0' 1000' 2000'

SCALE: 1" = 1,000'

Legend

Project Reach

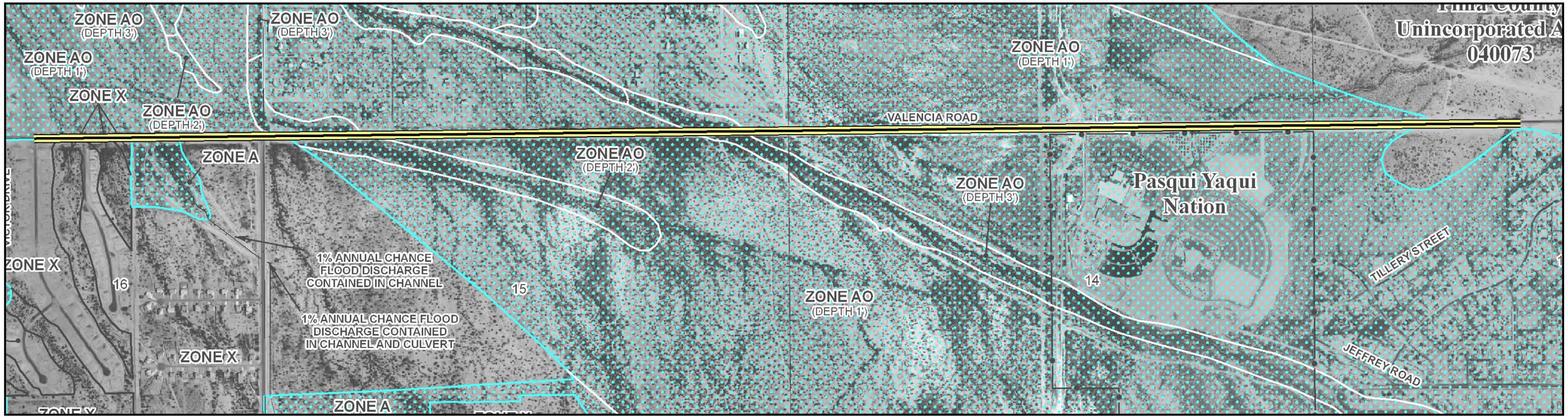
Figure 2

Location Map

WO # 4RTVMW

JE Fuller / Hydrology & Geomorphology, Inc.

Figure 3 - FIRM Showing Valencia Wade to Mark Project Alignment

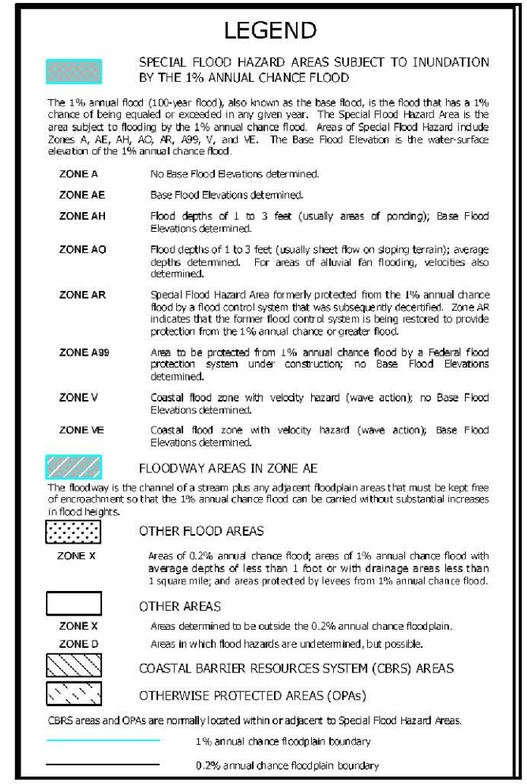


N

1 inch = 1,000 feet

Legend

Valencia_Wade-to-Mark Project Alignment



LEGEND

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

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

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

FLOODWAY AREAS IN ZONE AE

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

OTHER FLOOD AREAS

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

OTHER AREAS

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

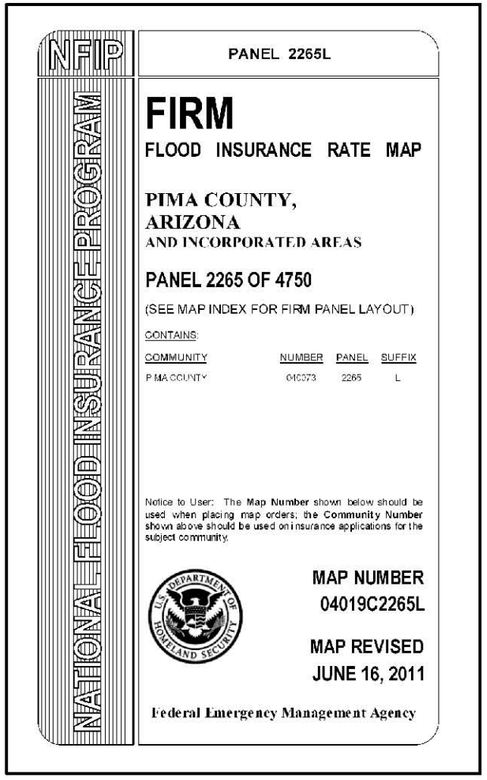
COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

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

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary



NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 2265L

FIRM

FLOOD INSURANCE RATE MAP

PIMA COUNTY, ARIZONA
AND INCORPORATED AREAS

PANEL 2265 OF 4750

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
PIMA COUNTY	040073	2265	L

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

MAP NUMBER
04019C2265L

MAP REVISED
JUNE 16, 2011

Federal Emergency Management Agency

Figure 4 - FLO-2D Model Boundary Overview

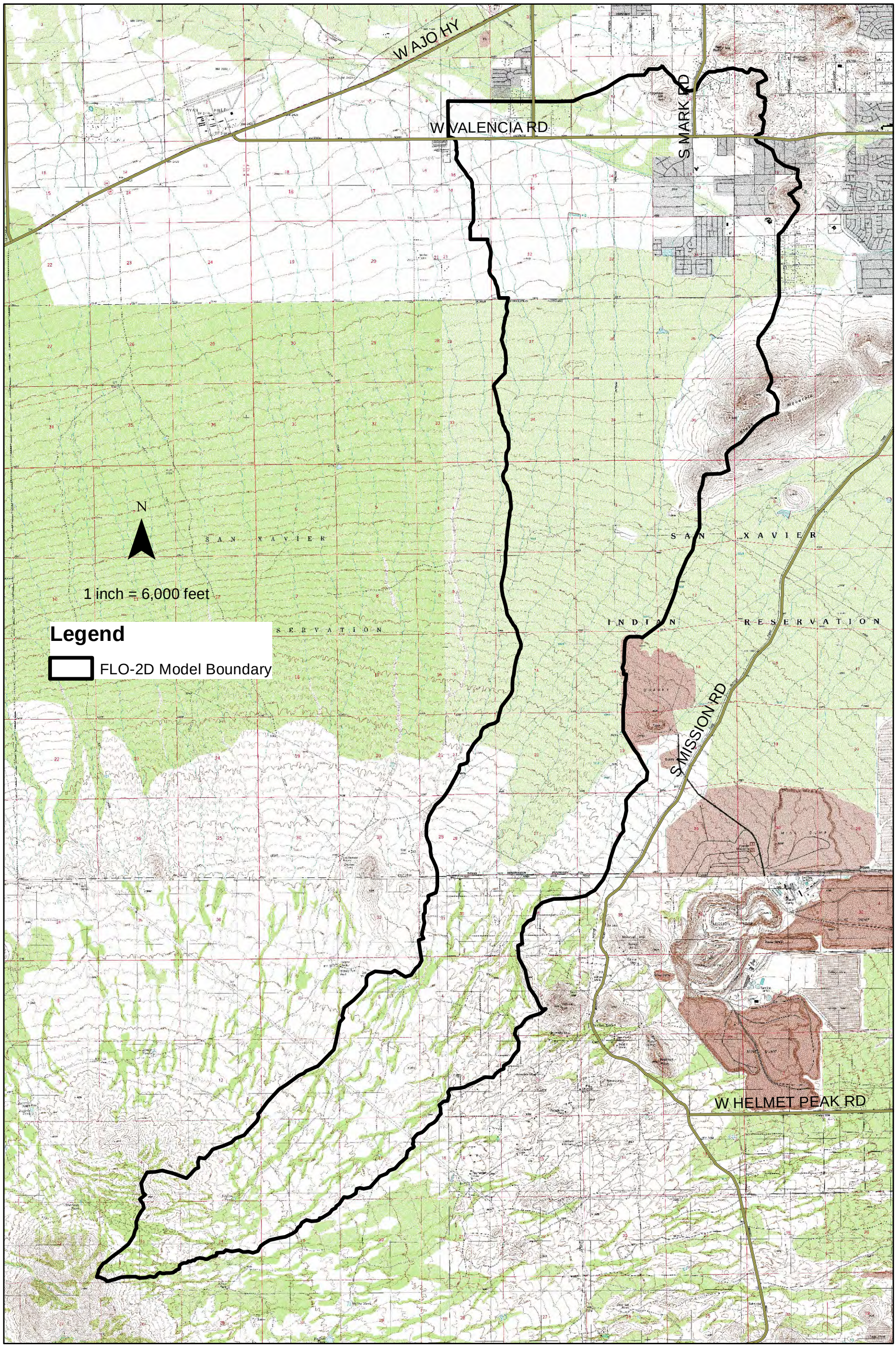


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Exhibit 1 - Valencia - Wade to Mark - Existing Condition FLO-2D Flow Depths

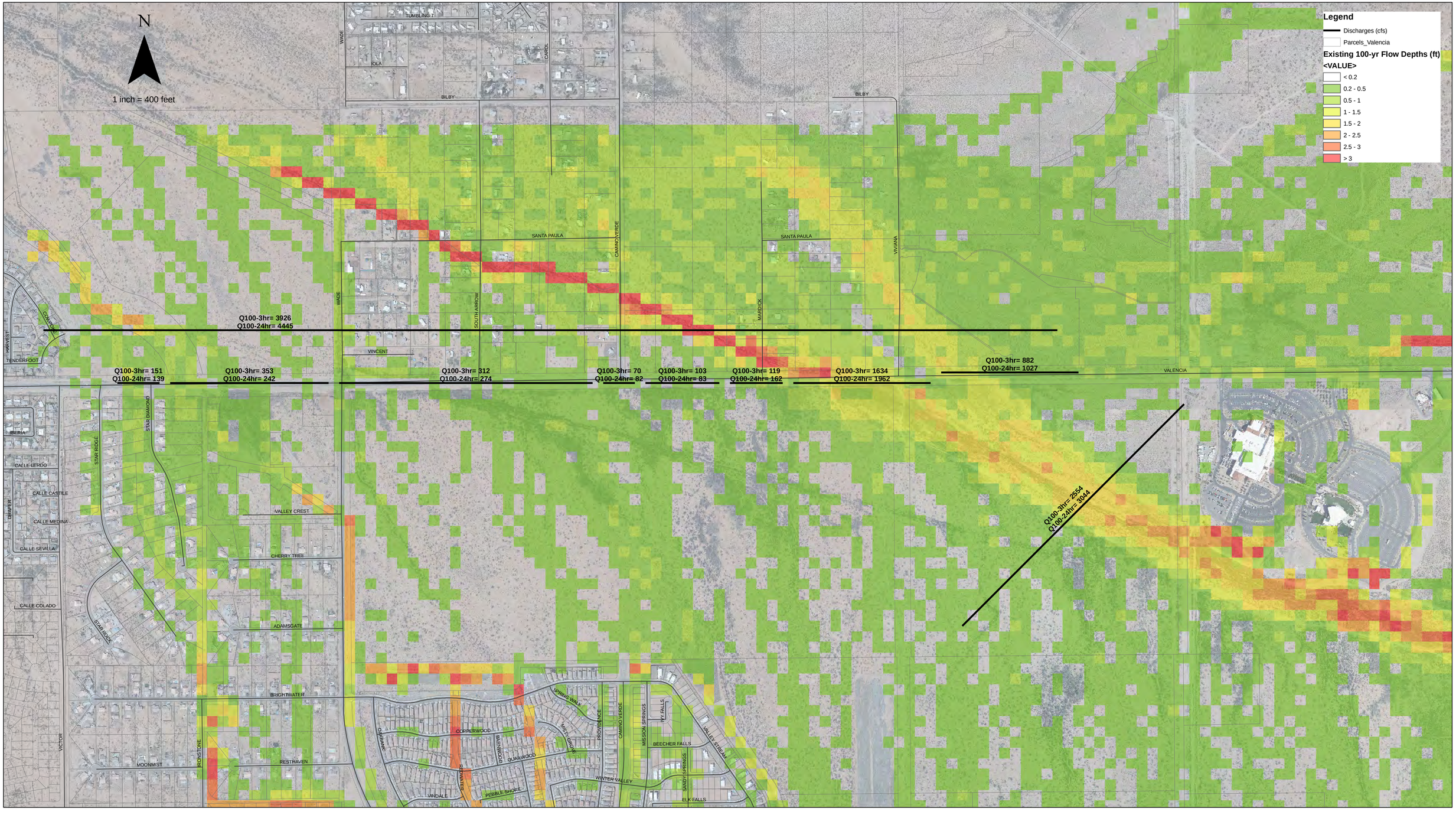

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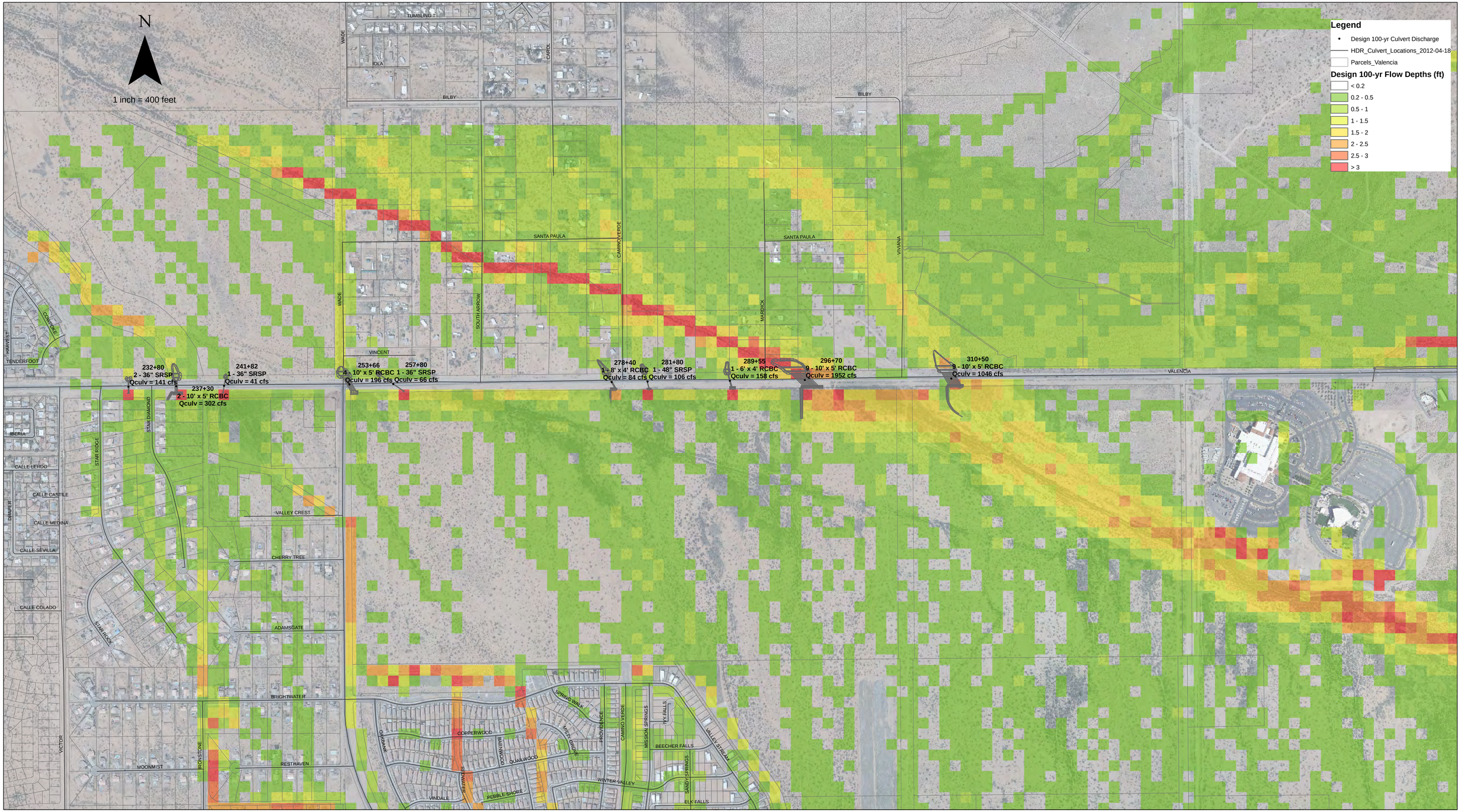
Exhibit 2 - Valencia - Wade to Mark - Design Condition FLO-2D Flow Depths



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