Agua Fria River
Hydrology Revision Feasibility Study – Phase II
FCD 2015C008, Work Assignment No. 1

Summary Memorandum

September 9, 2016

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Executive Summary

In March 2014 a Feasibility Study was completed by JE Fuller/Hydrology and Geomorphology, Inc. that identified the technical feasibility of reducing the 100-year peak discharge in the Agua Fria River by making use of sand and gravel mine storage volume adjacent to the river. In this Phase II study, further refinement of the potential for existing sand and gravel mines to reduce the 100-year regulatory flow rates and the corresponding 100-year floodplain limits in the Agua Fria River was performed.

Potential storage volume within these mines was estimated at over 38,000 acre-feet. Reevaluation of the 100-year flows in the River downstream of New Waddell Dam found that outflows from the Dam should be included with runoff from the downstream watershed when determining the flow rates for the 100-year floodplain. Moreover, the Dam outflow volume exceeds the available potential storage volume from existing mine pits. Therefore, a minimum bypass flow for the Dam outflows should be provided.

Use of the potential storage volume could result in a reduction of the 100-year peak flow rates for segments of the Agua Fria River between Happy Valley Road and the New River. However, not all of the pits are strategically located to leverage all the potential storage volume effectively. In order to use the effective storage, significant engineering measures would be required to comply with local and federal floodplain regulations. Estimates of construction costs of such measures would likely exceed $66 million excluding land acquisition and assuming pits can be sufficiently drained by percolation alone. Additionally, implementation would require significant collaboration and coordination among land owners, mine operators, and various levels of government.

Introduction

This Phase II analysis was authorized by the Flood Control District of Maricopa County (FCDMC) to further investigate initial findings of the Agua Fria River Hydrology Revision Feasibility Study completed by JE Fuller Hydrology and Geomorphology (JEF) in 2014. The objectives of Phase II were to evaluate whether existing sand and gravel mines could reduce the 100-year regulatory flow rates and the corresponding 100-year floodplain limits in the Agua Fria River and to estimate the necessary modifications to the pits, including costs, required to function to FCDMC and FEMA standards. The tasks completed as a part of this analysis were

1) the major storms of August 19, 2014 and September 8, 2014 were assessed,
2) the available storage in the Agua Fria River was quantified using the latest available topography,
3) the operations of New Waddell Dam and the risk of coincident releases from the Dam were examined,
4) the base hydrology for the Agua Fria was investigated,
5) from 3 and 4 above, a 100-year hydrograph was developed,
6) using this hydrograph, reduced peak flows were developed with an HEC-1 model that modeled the storage basins as diversion records,
7) a preliminary floodplain was delineated using the reduced peak flows and the latest available topography, and
8) an issues assessment was performed.

Each section of this report gives a summary of key items from each task, while the memoranda from each task are contained in the Appendices. The electronic files used in the analyses are also provided.

Identification of Available Storage

Under Task 2.1 of the Scope of Work (SOW), an initial estimate of 32 existing gravel mines were identified in the study reach. Twenty-three of these mines were classified as offline basins, seven would function as inline basins and two could function as either inline or offline. However, as more details became clear during the completion of Task 2.2, some basins were eliminated from consideration as potential storage facilities because:

a) they do not appear in the latest available topography,
b) groundwater is continually present in them or
c) one basin is reserved as part of another flood control project that controls local inflows.

The final total is 22 offline storage areas, 5 inline areas and 1 storage area that could be designed as either type. A comparison of the potential basins identified during each task is shown in Figure 1. Based on the locations of available storage in the Agua Fria River, the study reach was reduced to the section of river between Happy Valley Parkway and the confluence with New River.

Storage volumes were computed based on a mosaic of the latest available topography for the study reach. Dates of the topography range from 2000 to 2011. The total potential storage volume within the 22 pits was estimated as:

- 33,246 ac-ft as offline storage,
- 2,995 ac-ft as inline storage, and
- 1,841 ac-ft as either type.

This results in a total potential storage volume of 38,082 ac-ft in the reach of the Agua Fria River upstream of the New River confluence. It should be noted that from examination of the 2014 aerial photography, some large mines have increased their volume since the latest topography. However, it is not believed that the potential additional volume would greatly affect the results of this study.

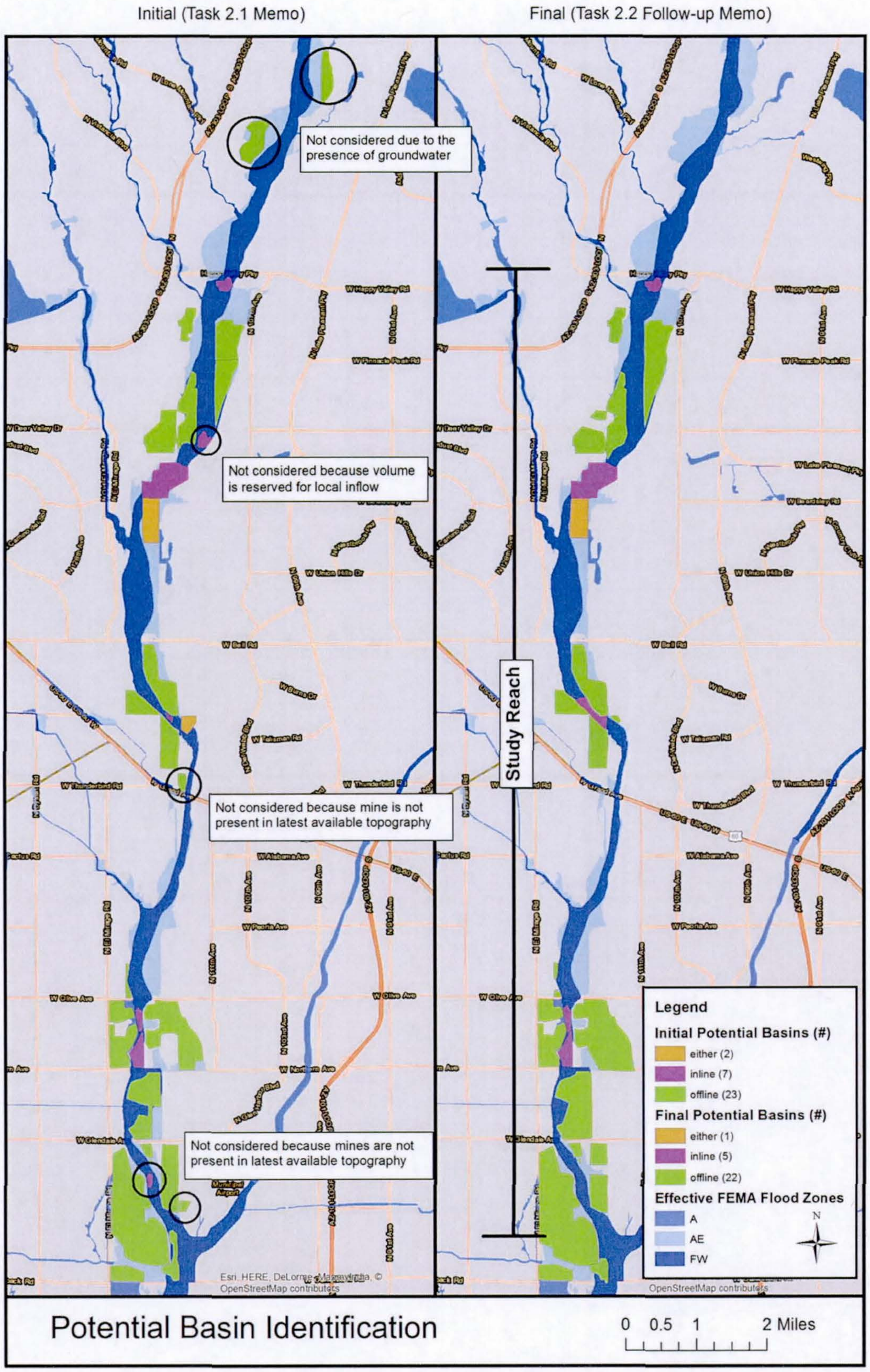

Figure 1. Comparison of potential basins between Tasks 2.1 and 2.2

Hydrology

Under Task 2.2 of the SOW, the hydrology of the watershed was revised, and the August and September 2014 storms were classified. From the results of this task, two conclusions were made:

1) based on the downstream watershed size and County hydrologic modeling requirements, the design hydrology for the Agua Fria watershed below New Waddell Dam would be the 100-year 24-hour event with a base flow of 9,000 cfs from the Dam, and
2) when viewed in the context of a design storm for a large watershed, the storm of September 8, 2014 was between a 5- and 10-year event.

After discussion with Central Arizona Project and review of the data on New Waddell Dam, it was determined that outflow from New Waddell Dam should be included when determining the 100-year event on the lower (downstream of the Dam) watershed; and, if this outflow did occur, it could have a volume greater than 40,000 ac-ft based on storage curves of the reservoir. This volume is greater than all available storage in the study reach. Therefore, the 100-year hydrograph used as the basis for all subsequent analyses was the 100-year 24 hour storm event with a 3 day base flow of 9,000 cfs from New Waddell Dam.

To maintain a realistic bypass flow near 9,000 cfs in the river and thereby keep outflows from New Waddell Dam out of any potential storage areas, a generalized rating table for a lateral weir was developed. This rating table was used in the reduced HEC-1 model to obtain reduced 100-year peak flows in the Agua Fria River. A comparison of these reduced peak flows with the corresponding flows in the existing conditions no-storage HEC-1 model is shown in Table 1. Note that the existing conditions no-storage flow rates derived from the HEC-1 model are similar to the current FEMA effective flow rates in the river, which are based on the 1995 Corps hydrology.

Table 1. Comparison of peak flows

Concentration Point	Location	Effective FEMA (cfs)	Existing no-storage (cfs)	Reduced (cfs)
AFR512	downstream of Happy Valley Parkway	31000	29372	29372
RGLMMO	Rose Garden Lane	35000	29180	11276
AFR820	Confluence with McMicken Wash	37500	36692	17386
AFRGND	Grand Avenue	36000	36637	15653
AFRPEO	Peoria Avenue	34500	36674	15547
AFROLV	Olive Avenue	34500	36626	15521
AFRNOR	Northern Avenue	34500	36407	10178
AFRGLN	Glendale Avenue	30000	36455	11952
AFRBH/Pit19	Bethany Home alignment	30000	36416	9921
AFRCML	Camelback Road	54400	54400[1]	52000[2]

[1] New River inflows control peak at this location, no change from Effective FEMA flow and no hydrograph derived at this location.
[2] Estimated based on peak flows, no hydrograph derived at this location.

Hydraulics and Floodplain

The development of the hydraulic model and the preliminary floodplain is outlined in the Task 2.2 Follow-up & Task 2.3 Preliminary Hydraulic Modeling Memorandum. Key items are summarized in the text below. Since the outflow from New Waddell Dam can have a total volume larger than the available storage in the existing mines, it was assumed that a minimum bypass flow capacity of 9,000 cfs (the maximum controlled release to the River from New Waddell Dam) should be maintained in the river downstream of the dam.

Using the reduced flow peaks from Table 1, an existing conditions HEC-RAS model was developed. This model applied blocked obstructions in large mines to an elevation equal to the downstream containment elevation of each mine (i.e. limit of effective storage). Ineffective flow areas were applied to areas with abrupt contraction or expansion and to areas where flow was not actively conveyed. Finally, the HEC-RAS levee option was applied to berms and other features to contain flows within the main channel, regardless of whether the berm was engineered (i.e. FEMA certifiable) or not. With the water surfaces from this model, an estimate of a reduced floodplain was developed (see Figure 2). From this figure, it can be seen that the floodplain can be reduced in some areas, but the overall effect is limited. In addition, some areas have an increased floodplain (compared to the effective floodplain) due to gravel mines that have been extended outside of the current effective floodplain limits since the effective floodplain delineation study (CVL, 1996). This means that even with the reduced flows there is some expansion of the floodplain due to the gravel mines enlarged surface area. To maximize the reduction in floodplain, some levees would need to be constructed to isolate some mines from the floodplain.

If these gravel mines were to be removed from the floodplain, an engineered structure (i.e., a levee) would need to be constructed to prevent the possibility of flow capturing the low areas of the gravel mine. Otherwise, the entire mine surface area should be shown in the floodplain. Therefore, in order to maximize the reduction in floodplain, some form of channelization would be needed.

Issues Assessment (Implementation Considerations)

Implementation considerations are outlined in detail in the Task 2.4 Memorandum, but major issues are summarized in this section. The primary consideration that increases the complexity and cost of utilizing the mines as storage areas is preservation of the integrity and safety of the floodplain, which is codified through FEMA and Maricopa County requirements and regulations. Compliance with these requirements, necessitates other constraints which must be resolved before any mine could be utilized and the benefits realized. The major constraints are:

- Land rights – access and maintenance of the storage basins must be codified through fee ownership or some type of easement to ensure the storage volume remains available.
- Maintenance – the basins must be maintained by a municipal entity to ensure the storage volume remains available and functions as designed.
- Cost – a) to dedicate the storage volume the inflow must be controlled by armored weirs to prevent erosion/headcut from water flowing into the pit, and b) to maximize land removed from the floodplain engineered levees must be provided. A levee is estimated to cost $900-$1050 per lineal foot, while a weir is estimated to cost around $1300 per lineal foot. Total costs have been estimated in the range of $66-80 million.
- Basin drain time – the basins must be able to drain sufficiently fast to prevent vector control issues. The assumption is made here that this can be achieved by percolation into the alluvium beneath the basins. If this is not the case, additional outlets would be required, increasing construction costs and potentially right-of-way requirements.
- Timing/Phasing – the timing of mine activity, construction phasing, etc. could constraint or delay successful implementation (i.e. official changes to the regulatory flow rates and/or floodplain extents).

There are other issues (such as permitting) typical of any engineering project. These should be able to be worked through during the design process. However, a plan for the transfer from an active mine site to a dedicated storage area needs to be achieved, and a funding source identified before proceeding to a detailed design. All of these items are likely to add to the overall cost.

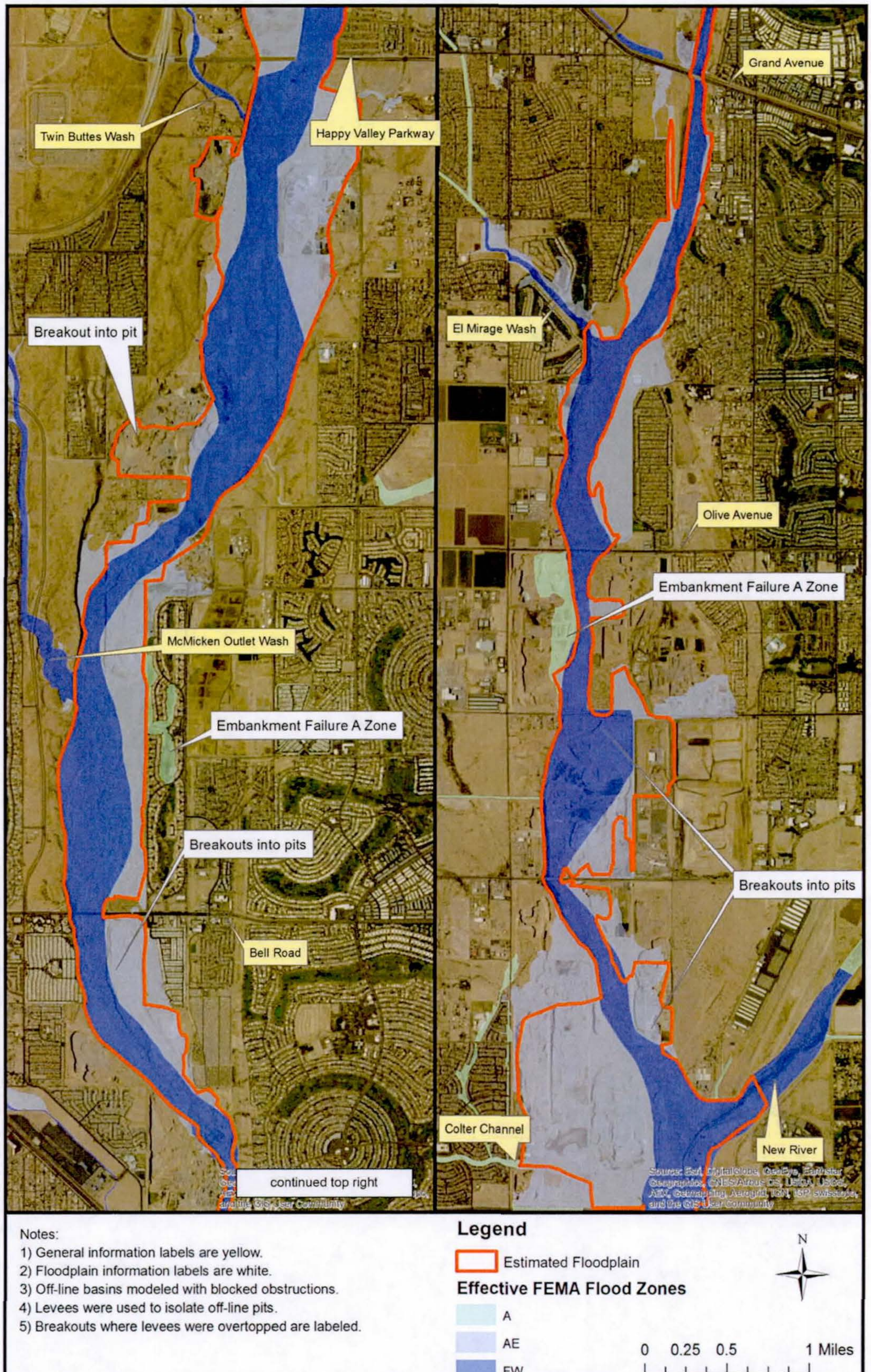

Figure 2. Estimated floodplain based on reduced peak flows in the Agua Fria River in the study reach

Results

The major conclusions from this Phase II analysis are:

1) The storms of August 19, 2014 and September 8, 2014 were less than the 100-year 24-hour design storm for the watershed of the Agua Fria River below New Waddell Dam.
2) There is a significant storage volume of 38,082 ac-ft in the reach of the Agua Fria River downstream of Happy Valley Parkway and upstream of the New River confluence.
3) Significant outflow from New Waddell Dam and a 100-year event on the lower Agua Fria watershed may occur coincidentally.
4) Based on 3, outflows from New Waddell Dam should be included in the 100-year regulatory hydrograph for the Agua Fria River.
5) Given 4, a minimum bypass flow of 9,000 cfs (the maximum controlled release to the River from New Waddell Dam) should be provided along the river.
6) With this bypass flow, inline pits should not be considered as storage areas since the volume of water that can be released from the Dam can be much greater than the volumes in these inline pits.
7) If off-line pits were used as storage areas, some engineered structures would be required to protect the River from erosion and to allow a 9,000 cfs bypass flow.
8) These engineered structures would have significant cost (around $15-18 million for one pit) even excluding land acquisition and potential outlet drains if needed.
9) Even with reduced peak flows, the floodplain extents would not change much unless some form of engineered channelization or flow containment (i.e., levees) was provided.
10) Some mines are more effective at reducing the peak flow than other mines due to geography and size (see Table 2 and Figure 3).

As further information for number 10, the potential storage basins were ranked on their level of effectiveness for reducing the flow rate (see Figure 3). In this figure, the basins are labeled as very effective, effective, and minimally effective, where each term is defined in Table 2. Basically, the potential storage areas were ranked high if there was a large, currently available volume in the storage area; and these areas were positioned in locations where the volume could be used effectively to reduce flows in the river. That is, large mines need to be located downstream of large inflow points. For example, a large volume is available downstream of Glendale Avenue, but there are no significant inflows in this reach.

Table 2. Effectiveness definitions

Term	Definition
Very Effective	Offline, has a large available volume, is well-located relative to areas where significant inflows occur
Effective	Offline, has a large available volume, location is not as ideal as primary locations
Minimally Effective	Inline or located upstream of major inflows or near Camelback Road where New River flow controls peak

In summary, if existing gravel mines were used as storage areas, the 100-year regulatory flow rate for the Agua Fria River could be reduced within the study reach. In fact, a cost of construction that utilizes the very effective and effective mines is estimated to range from $66-80 million, excluding land acquisition costs. Therefore, while peak flow reduction is possible, some mines are more useful than others. Utilizing mines as storage areas comes with significant costs and will require thoughtful collaboration among owners, mine operators, and various levels of government.

References

Coe and Van Loo Consulting Engineers (CVL), Agua Fria River Floodplain Delineation Re-Study, Between the Gila River Confluence and New Waddell Dam, Technical Data Notebook, 1996.

Flood Control District of Maricopa County (FCDMC), Drainage Design Manual for Maricopa County, Hydrology, August 15, 2013.

Flood Control District of Maricopa County (FCDMC), Drainage Design Manual for Maricopa County, Hydraulics, August 15, 2013.

JE Fuller Hydrology and Geomorphology, Inc. (JEF), Agua Fria River Hydrology Revision Feasibility Study, 2014

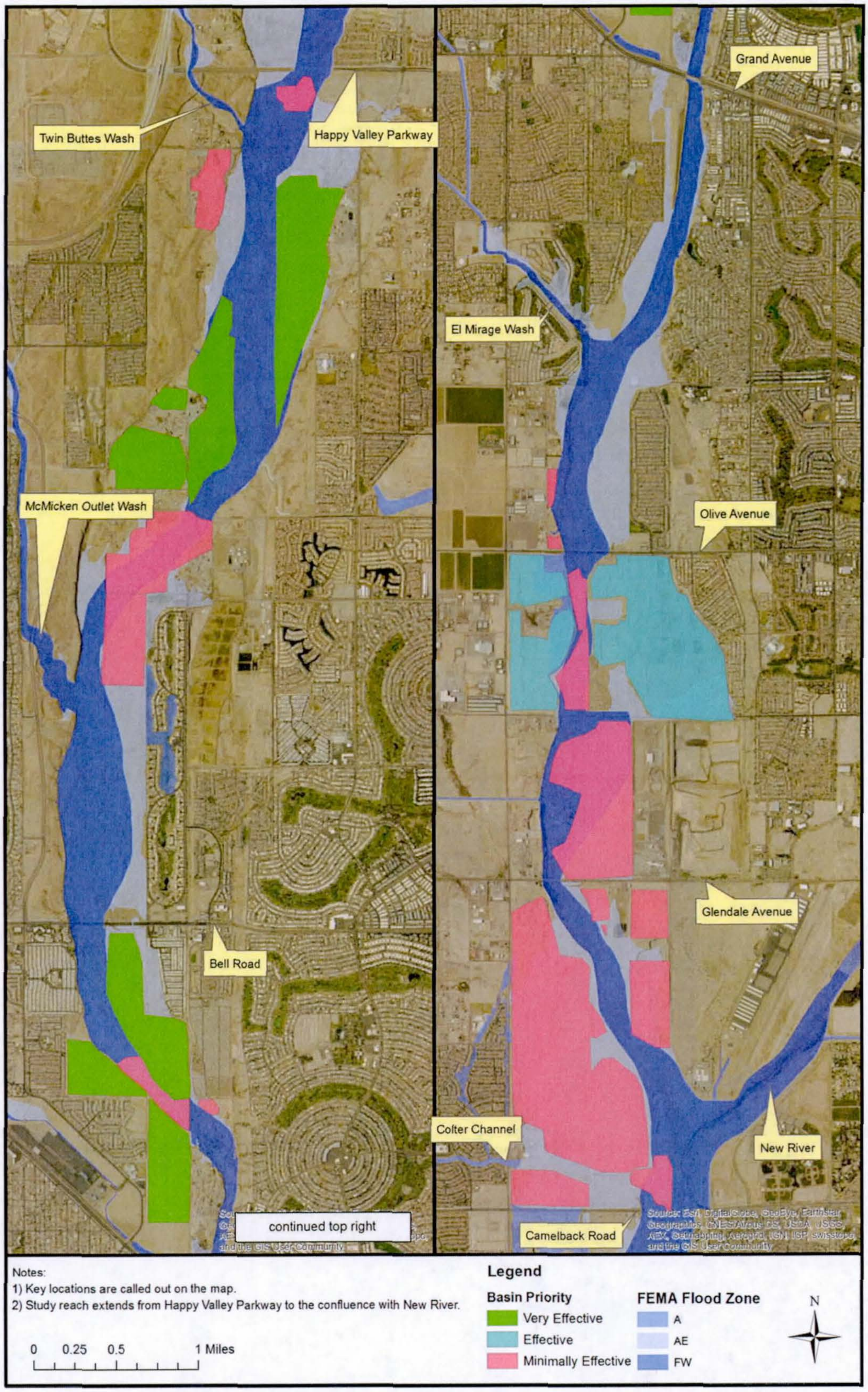

Figure 3. Effectiveness rankings of potential storage basins

Agua Fria River Hydrology & Hydraulics Feasibility Studies

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Task 2.1 Memorandum

This memorandum presents the results of Task 2.1. Data Collection & Review. The major items of Task 2.1 are identified in bold underline at the beginning of each paragraph. Pertinent figures referred to in the text follow the end of this memo.

<u>Identify inline and offline basins for potential modeling</u> – Approximate planimeteric locations were delineated from Nov. 2014 aerial photographs. Fate and extent of processing vs. mining areas is difficult to judge from the aerials alone. Depths might be able to be estimated from available topography for some, but not all, either due to obvious changes since the topography was flown and/or due to the presence of water in pits (see Map Figure 1). A total of 32 basins were delineated with a total of 2,757 acres in surface area. The vast majority (23) were characterized as candidates for offline basins with a total area of 2,386 acres.

<u>Review FCDMC topo and assess adequacy</u> – Large areas of the available topography are very old and highly discontinuous between vintages of datasets. None of the available topography is newer than Nov. 2011. Most is much older (see Map Figure 2). The old Agua Fria Watercourse Master Plan (WCMP) HEC-RAS model (using 1995 FDS topo) does not reflect significant changes to the floodplain including development near Jomax (Tierra del Rio and CAP recharge project), the Loop 303 crossing, nor extensive changes due to mine operations throughout the study reach. Some of these changes are reflected in the newer mapping (e.g. 2008 mapping downstream of Jomax), others are not (e.g. Loop 303 bridge crossing). We recommend new topography be obtained for entire study reach if new floodplain mapping is the objective and/or accurate assessment of the hydraulics are needed. Other more piecemeal, approximate approaches may be adequate to provide approximate hydrologic and hydraulic impacts along the study reach.

<u>Review of Hydrology</u> – The Phase I hydrology used North Peoria ADMP, Glendale-Peoria ADMS, and Wittmann ADMSU HEC-1 models. None of these studies used NOAA 14 rainfall. Comparison of NOAA 2 vs. NOAA 14 rainfall depths shows some areas underestimated in the higher terrain in the northwest while the lower elevation areas and urban areas are overestimated by NOAA 2 (see Map Figure 3). New models collected from the District also reveal updated modeling that uses NOAA 14 for the Wittmann area (McMicken Outlet Wash FDS, 2013) and the Glendale-Peoria area (Glendale Stormwater Management Plan, 2011). In addition, updated hydrology from the Loop 303/White Tanks area was recently completed (July 2015) for areas downstream of McMicken Outlet Wash. The treatment of McMicken Dam outflows may need to be altered to meet the objectives of this study. More than one scenario may also be considered (e.g. with outflows or without outflows from McMicken Dam) in a manner similar to the Phase I hydrology.

The North Peoria ADMP models used one design storm with a 100-year 24-hour point rainfall of 4.2 inches for all subbasins. Computation of the average rainfall for all subbasins using DDMSW and NOAA 14 rainfall data computed an average 100-year 24-hour rainfall depth of 4.234 inches. Given the similarity of rainfall depths, it is therefore recommended that the North Peoria ADMP models be used as-is.

It is further recommended that the Phase I hydrology be updated to reflect the newest inflows from the McMicken, Glendale, and Loop 303 models and the existing North Peoria models for inflows to the Agua Fria downstream of New Waddell Dam and upstream of the New River confluence.

Comparison of revised hydrology with the results of the USACE *Agua Fria River Study, New Waddell Dam to Gila River Confluence, AZ: Hydrologic Evaluation of Impacts of the New Waddell Dam on Downstream Peak Discharges in the Agua Fria River* (1995) should then be performed. The USACE study concluded that the downstream peak discharges for the 100-year event were controlled by local inflows rather than releases from New Waddell Dam.

References

JEF, 2001, Agua Fria River Watercourse Master Plan, Lateral Migration Report, June 2001, report prepared by JE Fuller/Hydrology & Geomorphology, Inc. (JEF) under contract FCD 99-24 as subconsultant to Kimley Horn Associates.

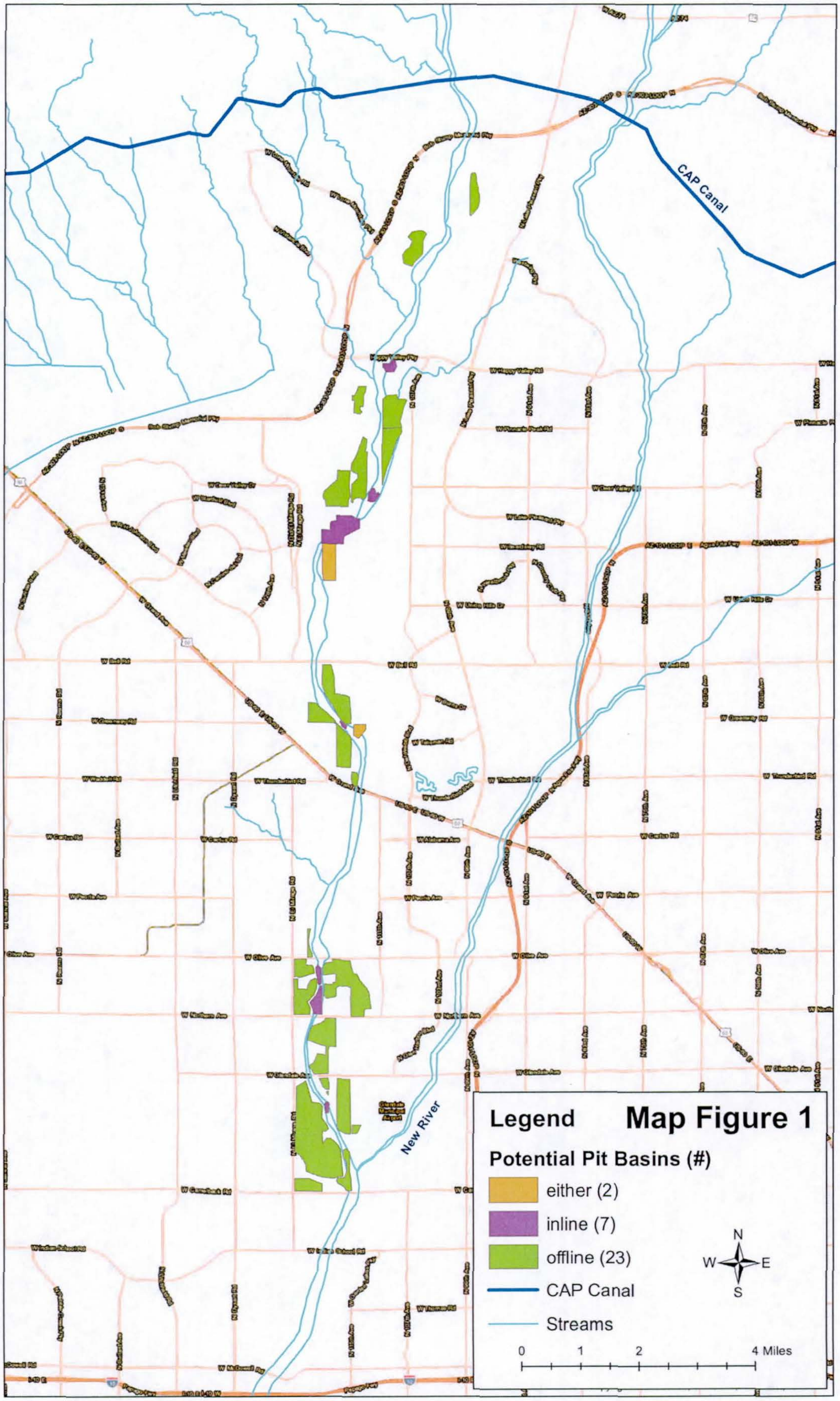
Kimley Horn Associates, 2011, Glendale Area Stormwater Management Plan, report prepared for City of Glendale in association with FCDMC.

Parsons Brinkerhoff, 2015, Loop 303 Corridor/White Tanks Hydrology and Delineation Update, report prepared for FCDMC under contract 2014C003.

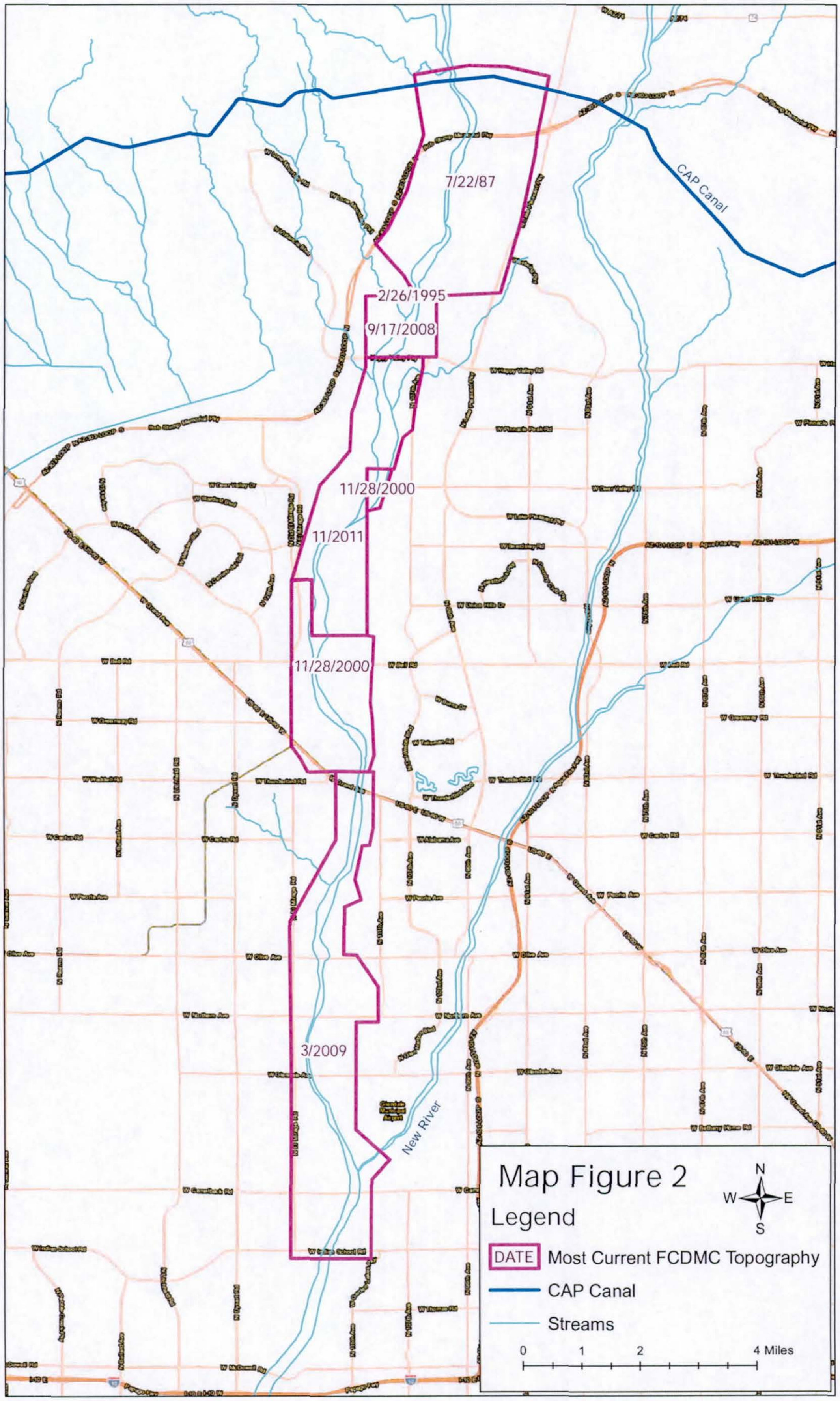
RBF, 2013, McMicken Dam Outlet Wash Floodplain Delineation Study (FDS), report prepared for FCDMC under contract FCD 2011C003.

Stantec, 2001, North Peoria Area Drainage Master Plan, prepared for FCDMC under contract FCD 99-45.

USACE, 1995, *Agua Fria River Study, New Waddell Dam to Gila River Confluence, AZ: Hydrologic Evaluation of Impacts of the New Waddell Dam on Downstream Peak Discharges in the Agua Fria River,* US Army Corps of Engineers, Los Angeles District.



Legend
Map Figure 1
Potential Pit Basins (#)
either (2)
inline (7)
offline (23)
CAP Canal
Streams
N
W
E
S
0
1
2
4 Miles
CAP Canal
New River


7/22/87
2/26/1995
9/17/2008
11/28/2000
11/2011
11/28/2000
3/2009
CAP Canal
New River
Map Figure 2
Legend
N
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DATE Most Current FCDMC Topography
CAP Canal
Streams
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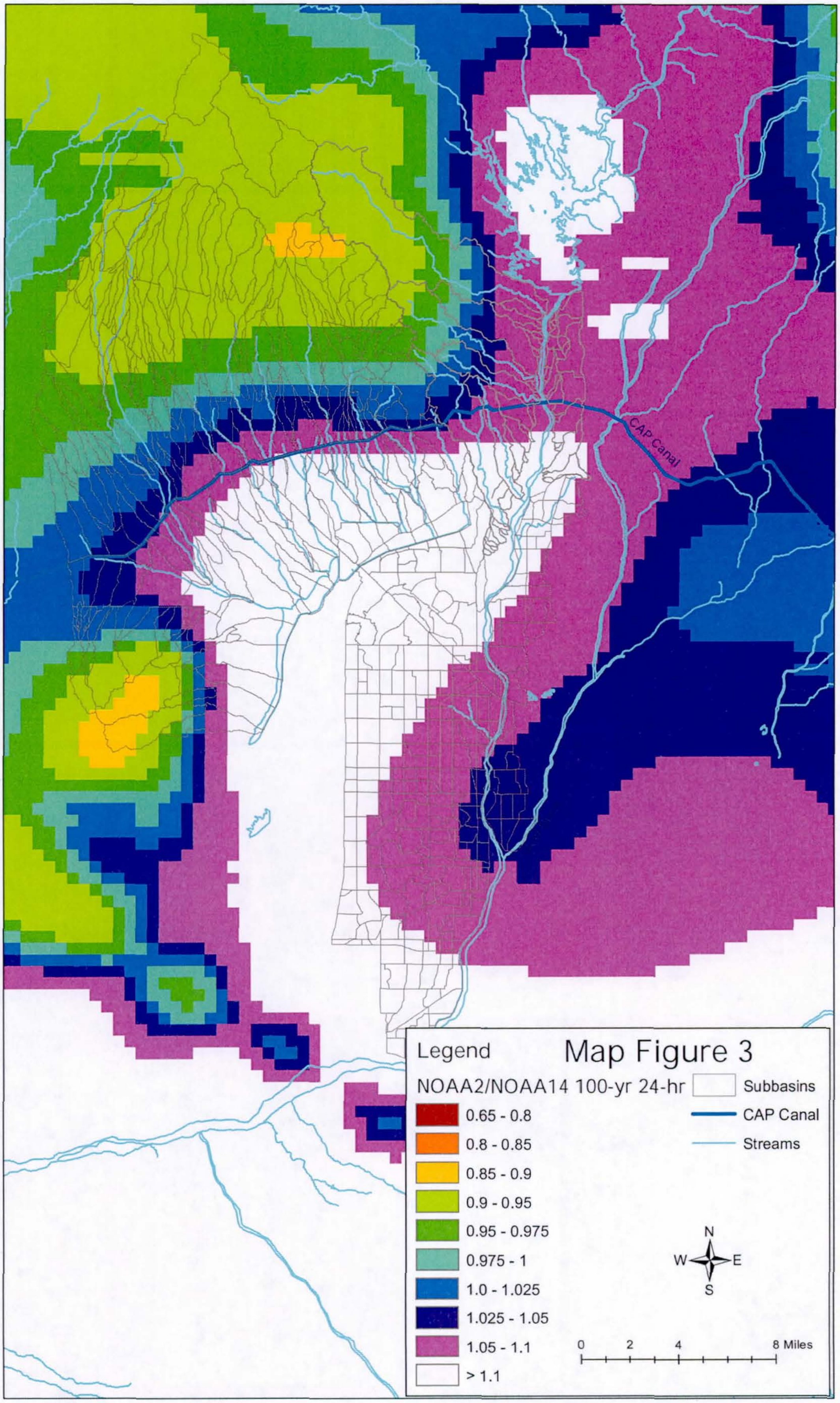
4 Miles


CAP Canal
Legend
Map Figure 3
NOAA2/NOAA14 100-yr 24-hr
0.65 - 0.8
0.8 - 0.85
0.85 - 0.9
0.9 - 0.95
0.95 - 0.975
0.975 - 1
1.0 - 1.025
1.025 - 1.05
1.05 - 1.1
> 1.1
Subbasins
CAP Canal
Streams
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S
0
2
4
8 Miles

Agua Fria River Hydrology & Hydraulics Feasibility Studies

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Task 2.2 Memorandum

This memorandum briefly presents the results of Task 2.2. Hydrologic Modeling and Evaluation. The major items of Task 2.2 are discussed in the paragraphs below.

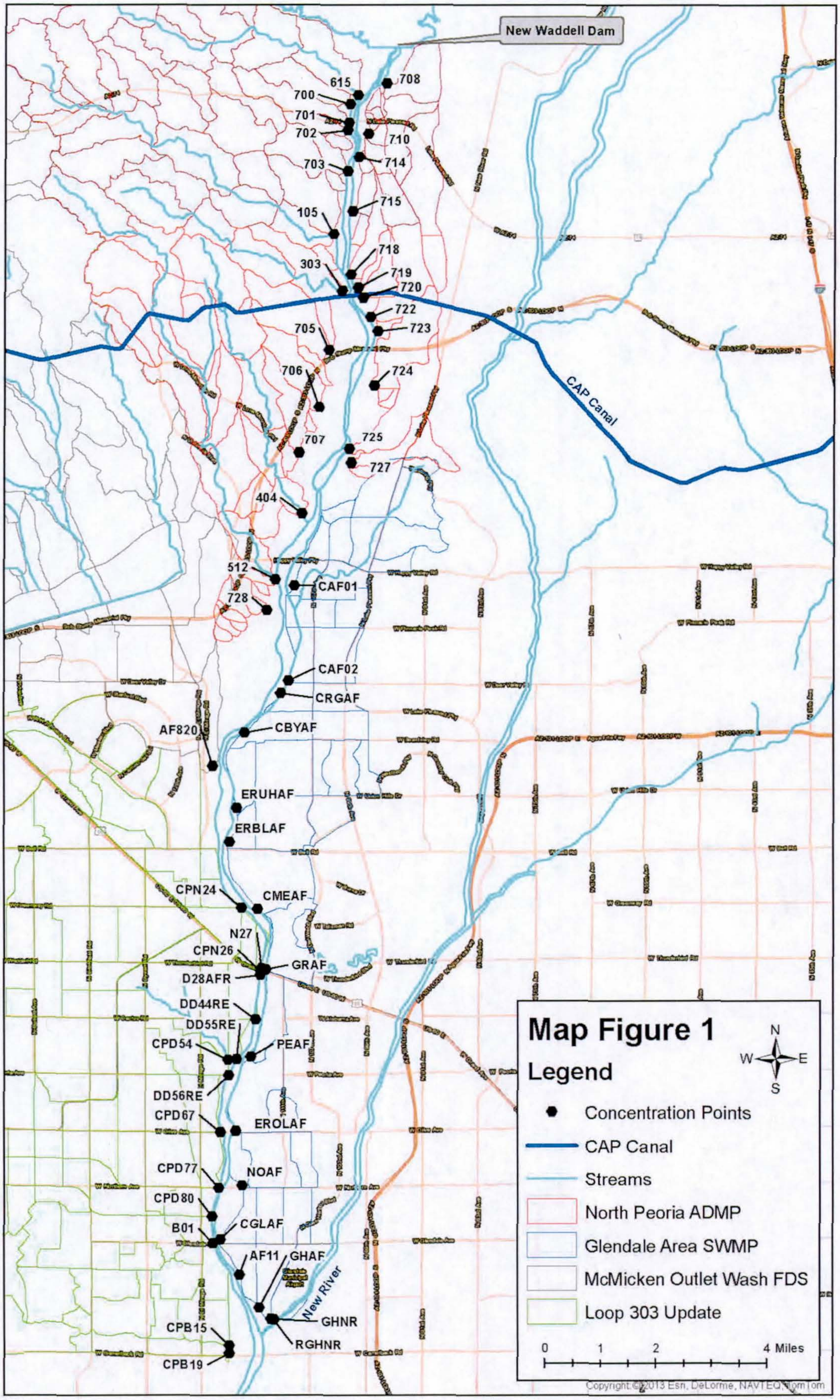
Phase I Model Refinements

HEC-1 models from the Phase I hydrology were refined based on the findings of Task 2.1. In particular, inflows from McMicken Outlet Wash (Wittmann ADMSU area), Glendale-Peoria area, and the Loop 303/White Tanks area were taken from newer modeling collected from the District as part of Task 2.1. The North Peoria ADMP area models were not changed. Comparison of the NOAA Atlas 2 values used in the ADMP modeling and NOAA 14 values showed only minor differences (4.2 inches vs. 4.234 inches) as applied to the model. All of these models used a 100-year 24-hour design storm based on the size of the watersheds being modeled and District modeling criteria.

Concentration points that discharge to the Agua Fria River were identified for each ADMP/model area (see Map Figure 1). DSS ZW records were added to each model and the models rerun into a common DSS file. The resulting hydrographs were then routed through along the River. The SV-SQ records used in the routing model were not changed from the Phase I model. They were constructed from multiple profile runs of the 2001 Agua Fria Watercourse Master Plan (WCMP) HEC-RAS model developed for the Lateral Migration Report (JEF, 2001). The Phase I HEC-1 routing model logic was adjusted as needed to combine all of the tributary inflows appropriately.

Table 1 below shows a summary of the computed peak flow rates and volumes above 9,000 cfs for two scenarios. The first, base scenario, is the combined flows from all the downstream tributaries at several locations between the CAP Canal and the New River confluence. The second scenario assumes 9,000 cfs outflow from New Waddell Dam (the 100-year event outflow according to the 1995 COE New Waddell Dam hydrology) plus the local tributary runoff for the 100-year 24-hour event.

It should be noted that the base scenario presented does not include any outflow from McMicken Dam per the conclusions of the McMicken Outlet Wash FDS.



New Waddell Dam
615
708
700
701
702
710
703
714
715
105
718
303
719
720
722
723
705
706
724
707
725
727
404
512
CAF01
728
CAF02
CRGAF
CBYAF
AF820
ERUHAF
ERBLAF
CPN24
CMEAF
N27
CPN26
GRAF
D28AFR
DD44RE
DD55RE
CPD54
PEAF
DD56RE
CPD67
EROLAF
CPD77
NOAF
CPD80
CGLAF
B01
AF11
GHAF
New River
GHNR
CPB15
RGHNR
CPB19
CAP Canal
Map Figure 1
Legend
N
W
E
S
Concentration Points
CAP Canal
Streams
North Peoria ADMP
Glendale Area SWMP
McMicken Outlet Wash FDS
Loop 303 Update
0
1
2
4 Miles
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Table 1. Summary of initial (dated August 2015) HEC-1 Model Results

		Base Scenario		9000 cfs outflow from New Waddell	
		Peak Flow	Volume above 9000 cfs	Peak Flow	Volume above 9000 cfs
Location	KK ID	cfs	ac-ft	cfs	ac-ft
CAP Canal	AFR719	17,725	779	27,151	4,066
Jomax Rd	AFR707	17,808	910	27,761	4,836
U/S MOW	RGLMMO	17,972	1,232	29,190	6,149
D/S MOW	AFR820	24,676	3,547	36,702	12,700
Greenway Rd	AFRGRN	24,612	3,595	36,733	13,150
Grand Ave	AFRGND	24,572	3,625	36,646	13,446
Olive Ave	AFROLV	24,539	3,666	36,635	13,793
Glendale Ave	AFRGLN	24,461	3,833	36,464	14,786
U/S New River	AFRCML*	24,341	3,863	36,294	15,275
* Excludes any flows from the New River					

Storms of August & September 2014

Given the large storms of monsoon 2014, comparisons have been drawn between those events and the appropriate 100-year flow rate for the Agua Fria River. Two significant events in August and September 2014 were characterized for their magnitude, frequency, and duration. As discussed below neither was long enough in duration to qualify as an appropriate benchmark of the regulatory event for the Agua Fria River downstream of New Waddell Dam. Given the size of the downstream contributing area, its time of concentration, and County hydrologic modeling criteria, the 24-hour storm should be considered as the basis of evaluation of the regulatory runoff from this watershed.

Rainfall

MetStat (2014a, 2014b) has previously analyzed the August and September 2014 storms for the District. They concluded that these storms had approximately a 6-hour duration. The September 8, 2014 event was a 6 hour event that focused on the lower Agua Fria watershed. When viewed in the context of a design storm for a large watershed, this storm was between a 5- and 10-year event. The 24-hour duration storm would be considered when evaluating a design storm for the lower Agua Fria River based on District hydrology manual criteria due to the large drainage area and time of concentration.

Table 2. 100-year 24-hour NOAA 14 point rainfall statistics for lower Agua Fria River watersheds

		NOAA 14 Depth-Frequency Statistics - 24-hour Duration						
	9/8/14	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
NP	2.47	1.82	2.34	2.75	3.32	3.77	4.23	4.72
McMicken	2.29	1.70	2.19	2.57	3.10	3.52	3.96	4.41
Loop 303	2.10	1.46	1.89	2.23	2.70	3.07	3.45	3.86
Glendale	2.73	1.50	1.94	2.29	2.77	3.15	3.54	3.95
all	2.33	1.64	2.11	2.49	3.00	3.41	3.84	4.28

Yellow highlighted numbers represent range of 9/8/14 rainfall for each area.

In order to assess the return interval, or probability of these events, NOAA 14 depth-duration-frequency (DDF) statistics were computed using DDMSW for each polygon area. Additionally, since the NOAA 14 DDF statistics represent point probabilities, they were reduced based on the total drainage area of each polygon. Areal reduction and DDF statistics were also computed for the total drainage area downstream of the Dam to New River. Table 3 shows the results. As can be seen, the storm rainfall depths for the September 8, 2014 storm were just less than or slightly greater than the 100-year return period over the watershed(s) downstream of New Waddell Dam slightly greater than the 100-year **6-hour** return period over the watershed(s)

Table 3. 100-year 6-hour rainfall return periods for September 8, 2014 storm

Drainage Area Name	Size mi^2	9/8/14 Avg. in	8/19/14 Avg. in	NOAA 14 6 hour point values 200-yr in	100-yr in	50-yr in	25-yr in	10-yr in	NOAA 14 6 hour areally reduced values Reduction Factor	200-yr in	100-yr in	50-yr in	25-yr in	10-yr In
North Peoria	67.1	2.47	1.83	3.33	2.99	2.67	2.35	1.95	0.84	2.79	2.50	2.23	1.97	1.64
McMicken	80.3	2.29	1.17	3.15	2.83	2.52	2.22	1.85	0.82	2.59	2.33	2.07	1.83	1.52
Loop 303	67.6	2.10	0.91	2.69	2.41	2.15	1.89	1.57	0.84	2.25	2.02	1.80	1.58	1.31
Glendale SMP	21.0	2.73	0.51	2.85	2.56	2.28	2.01	1.67	0.91	2.59	2.32	2.07	1.83	1.51
All	235.9	2.33	1.22	3.05	2.74	2.44	2.15	1.78	0.80	2.44	2.19	1.95	1.72	1.43

Yellow highlighted numbers represent range of 9/8/14 rainfall for each area.

Runoff

Runoff measurements were also investigated. The only stream gage along the Agua Fria River itself within the study reach is the Agua Fria at Grand Avenue station (ID #5503). The maximum discharge recorded for the August 19, 2014 event was only 80 cfs while on September 8, 2014 a maximum discharge of 687 cfs was recorded. According to the USACE 1995 hydrology report which is the basis of the current effective hydrology for the river downstream of the dam, this correlates to about a 2-year runoff event.

Larger inflows were measured by District stream gages on Dysart Drain and Colter Channel during the September 8th event. Peak discharges of 823 cfs and 158 cfs were recorded respectively. According to flood frequency relationships reported on the District ALERT website, those peak flow rates correspond to about a 10-year and 35-year event respectively. Both were the highest flows recorded by these stations since their installation. Dysart Drain has an 18 year period of record. Colter Channel has a 21 year period of record.

Largest Events Since Completion of New Waddell Dam

Downstream of New Waddell

Examination of the stream gage data for the Agua Fria River at Grand Avenue station shows that the largest two flood events in the period of record occurred on 10/27/2000 and 1/21/2010 when 5,952 cfs and 5,329 cfs were recorded respectively. According to the flood frequency relationship reported in the USACE 1995 hydrology, the October 2000 event corresponds to about a 6-year runoff event. The total runoff volume passing the gage for these events were 371 and 366 ac-ft respectively.

Inflows to Lake Pleasant

The January 2010 event had a large inflow to Lake Pleasant. A peak discharge of 54,800 cfs was recorded at the USGS Agua Fria at Rock Springs gaging station. The total runoff volume for that event was about 59,650 ac-ft. Lake Pleasant's water surface elevation went from almost 1677 feet to just above 1689 feet. CAP reported about 36,000 ac-ft on 1/21/10 and another 38,000 ac-ft on 1/22/10. Most of this entered within an 18 hour period. Negligible flow was released from New Waddell Dam during the 2010 event.

The largest event by volume was the event of January 1993. This storm produced over 100,000 ac-ft of total inflow to Lake Pleasant with about 87,000 ac-ft of inflow occurring within one day.

The implications of large events like 1993 on flow releases to the Agua Fria River downstream of New Waddell Dam are discussed further in the following section.

Characterization of Design Hydrograph for Agua Fria River downstream of New Waddell Dam

Inflows to New Waddell Dam

Since the initial filling of New Waddell in late 1992 (about 23 years ago), there have been three events that have had a peak greater than (or equal to) 40,000 cfs, as measured by the Agua Fria River stream gage at Rock Springs (see Figure 1). Two of these events occurred in the past 6 years. Neither resulted in releases to the lower Agua Fria River. The Jan. 1993 event was the largest volume inflow of these 3 events. It did result in discharges to the lower River. However, those releases were also the result of 'first fill' considerations for dam safety.

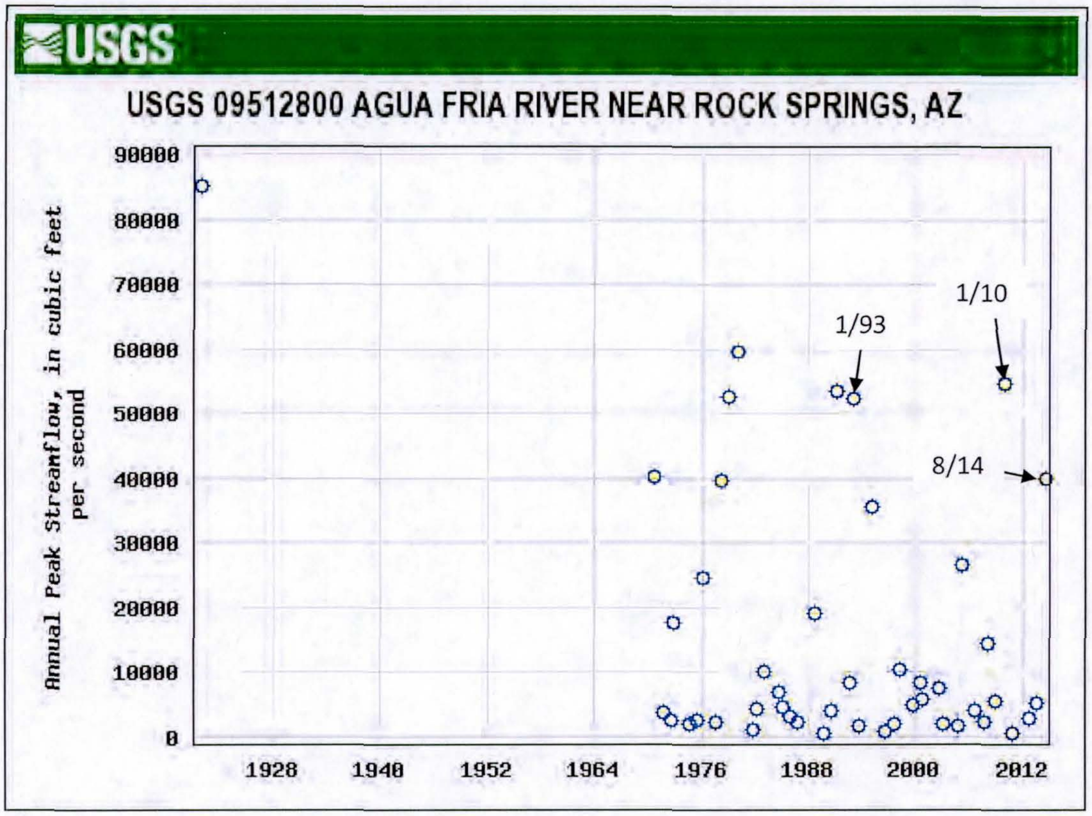

Figure 1. USGS peak flow data for the Agua Fria River

CAP Operations of New Waddell Dam

Meetings were held with Central Arizona Project (CAP) personnel to discuss operation of New Waddell Dam. The purpose was to understand operational requirements and discuss whether the assumptions and conclusions of the USACE (1995) remained reasonable interpretations of the likelihood of releases to the lower Agua Fria River. As mentioned above, in the 24 years since the completion of the dam, one significant release occurred in 1993. While the releases were influenced by 'first fill' dam safety considerations, it was also the largest inflow event since completion of the new dam. Perhaps not coincidentally, one occurrence in approximately 25 years corresponds very well with the conclusions of the USACE (1995) who determined that the probability of controlled releases was about 4 percent (i.e. 25-year event). Therefore, it was decided that releases from the dam could occur in close proximity in time with a significant event on the downstream watershed and should be accounted for in evaluating the 100-year floodplain discharges for the lower Agua Fria River.

Design Hydrograph for the Agua Fria River

Given the above, it is deemed reasonable that an outflow from New Waddell Dam could occur at the same time as a storm on the lower watershed of the Agua Fria. Therefore, a base flow of 9,000 cfs was included in the updated HEC-1 hydrology model of the lower watershed. This model becomes the base model for further analysis in Phase II.

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Agua Fria River Hydrology & Hydraulics Feasibility Studies

FCD 2015C008, Work Assignment No. 1

Task 2.3 Preliminary Hydraulic Modeling

Purpose

This purpose of this memorandum is to document the results of new analyses performed after the Task 2.2 memorandum was submitted. A new topography mosaic dataset was developed by the District that allowed for easy calculation of the storage volumes in the existing gravel mines. Those calculations allowed for further hydrologic and hydraulic modeling (Task 2.3) of the lower Agua Fria River to evaluate the potential use of those mines as flood control storage.

Objectives

Two objectives are outlined in this memorandum. They are:

1) Assess the feasibility of reducing the flow rate in the Agua Fria River using the existing pit volume, and
2) With this reduced flow rate (if any) estimate the commensurate reduction in floodplain limits.

The initial hydrology for this task is the 100-year 24-hour storm event with a base flow of 9,000 cfs from New Waddell Dam developed as part of the Task 2.2 Memorandum. The hydrograph just downstream of Happy Valley Parkway is shown in Figure 1. Its total volume is greater than 55,000 ac-ft.

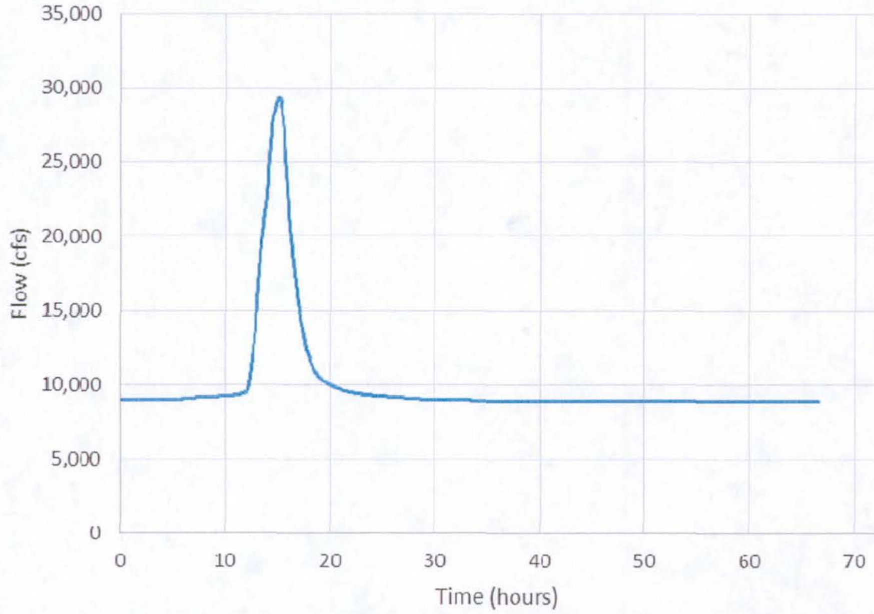

Figure 1. Base hydrograph of the Agua Fria River just downstream of Happy Valley Parkway

Methodology

The following steps were taken to investigate the two objectives:

- Quantify diverted hydrograph volumes at key locations to maintain various flow rates (Figure 2)
- Quantify and map available storage volume by reach (Table 1)
- Run HEC-1 with diversion records to develop potential reduced flow rates (Figure 3)
- Run effective HEC-RAS with reduced flow rates to identify potential floodplain impact (Table 3 & Figure 4)

Results

Diverted Hydrograph Volumes

Figure 2 *shows the diverted volume of water needed to maintain a particular flow rate in the* Agua Fria River. From this figure, it can be seen that as the flow is reduced to 9,000 cfs (maximum controlled outflow to the River from New Waddell Dam) the total volume that needs to be diverted greatly increases. There are two locations that greatly affect the total volume: 1) the flow of the Agua Fria itself at Happy Valley Parkway, and 2) inflows from the McMicken Outlet Wash.

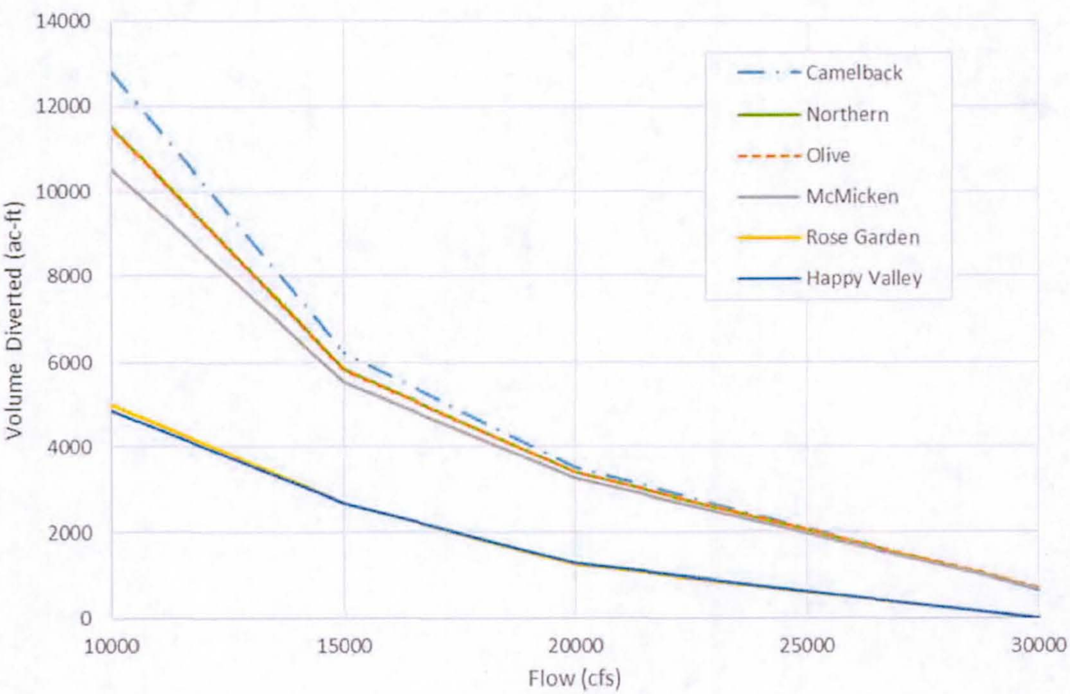

Figure 2. Diversion volume needed to reach target flow rate in Agua Fria River at key locations

Potential Storage Volumes

Potential storage volume associated with sand and gravel mines along the river were quantified using a mosaic of the most recent topography available from the District. The extent and vintage of available topography is presented in the Task 2.1 memorandum. From that topography, potential pit storage volumes were computed in GIS. The potential storage was estimated based on the elevation of the downstream 'lip' of the pits as depicted in the mosaic topography. In some cases, it was evident from more recent aerial photography that the pits have been enlarged since the date of the topographic mapping. Maps of the elevation-volume relationships computed for each pit are provided as an appendix to this memorandum.

Table 1 shows a summary of the available volume for all large sand and gravel mines by reach, where large is defined as mines having storage greater than 1,000 ac-ft. From this table and comparison with Figure 2, it can be seen that there is sufficient storage volume (> 32,000 ac-ft) to capture the hydrograph volume from the Agua Fria River and reduce the main channel flow to 9,000 cfs (around 13,500 ac-ft). However, the locations of these storage areas may not be in the right places to catch the volume entering the river from inflowing tributaries. For example, there is not much storage volume available in the reaches that run between McMicken Outlet Wash and Olive Avenue.

Table 1. Total available storage volume of major pits (storage > 1000 ac-ft) listed by reach

Reach	Storage Volume ac-ft
Happy Valley - Rose Garden	6,279
Rose Garden - McMicken Outlet Wash	4,217
McMicken Outlet Wash - Greenway	1,508
Greenway - Grand	1,772
Olive - Northern	6,704
Northern - Camelback	12,146
Total	**32,626**

HEC-1 Modeling of Pit Storage Volumes

To test if the storage basins are strategically located, an HEC-1 model was developed. In this model, the storage basins were modeled with diversion records where the maximum diverted volume was limited to the maximum available storage in each basin. The generalized rating table used in the HEC-1 model is shown in Table 2, and only off-line storage basins were input as diversion records because it was assumed that the leading inflow of 9,000 cfs from New Waddell Dam would completely fill the inline basins before the peak reached them due to their comparatively low volume (around 4,800 ac-ft of total possible inline storage versus > 50,000 ac-ft of outflow from the Dam).

Table 2. Generalized rating table used in HEC-1 diversion records (DI, DQ)

Upstream Inflow (DI) (cfs)	Diverted Flow (DQ) (cfs)	Continuing Flow (cfs)
0	0	0
9,000	0	9,000
10,000	275	9,725
14,000	2,800	11,200
20,000	8,700	11,300
30,000	19,700	10,300
40,000	31,000	9,000

The generalized rating table was developed using multiple HEC-RAS runs to determine how the weir coefficient responds to increase depth and discharge. A trend line was fit to the weir coefficient results from the HEC-RAS model, and this trend line equation was used to determine the weir coefficient in the weir equation:

$$Q = CLH^{1.5}$$

where Q is the weir discharge in cfs, C is the weir coefficient in $ft^{1/2}/s$, L is the length of the weir in feet and H is the head above the weir crest in feet.

The weir equation was applied to multiple different head values determined from normal depth analyses. The normal depth calculations used different geometries chosen to bracket a range from the widest possible channel width (a real cross-section from the Agua Fria River) to the narrowest feasible section (a 250-foot bottom width) for the Agua Fria River and the general slope of the Agua Fria River of 0.0022.

From Table 2, it can be seen that slightly more than 9,000 cfs remains in the river channel for lower discharges. With increased head, weirs operate more efficiently, but this is counter-balanced by the increased downstream velocity. Actual performance would vary depending on the final weir configuration and the actual channel geometry near the weir.

The results from the HEC-1 analysis are shown in Figure 3 and Table 3. In this figure, the locations where the peak flow is compared with the existing condition are marked with a star. Each star is labeled with the corresponding concentration point name. From these results, it can be seen that downstream of Happy Valley Parkway the peak flow can be reduced to about 11,000 cfs from the initial flow of 29,372 cfs. However, once flow from the McMicken Outlet Wash enters the Agua Fria River, the peak rises above 17,000 cfs and stays above 15,000 cfs until the next set of storage basins are reached downstream of Olive Avenue. The peak then decreases to 10,178 cfs and is further reduced to 9,921 cfs around the alignment of Bethany Home Road. The flow at Camelback Road (AFRCML) of 52,000 cfs is an estimate of the 100-year peak discharge in the Agua Fria River including inflows from Colter Channel and the New River. Note that the existing conditions no-storage flow rates derived from the HEC-1 model are similar to the current FEMA effective flow rates in the river, which are based on the 1995 Corps hydrology.

Table 3. 100-year peak flow comparison of HEC-1 results and current effective FEMA peaks.

Concentration Point	Location	Effective FEMA (cfs)	Existing no-storage (cfs)	Reduced (cfs)
AFR512	downstream of Happy Valley Parkway	31000	29372	29372
RGLMMO	Rose Garden Lane	35000	29180	11276
AFR820	Confluence with McMicken Wash	37500	36692	17386
AFRGND	Grand Avenue	36000	36637	15653
AFRPEO	Peoria Avenue	34500	36674	15547
AFROLV	Olive Avenue	34500	36626	15521
AFRNOR	Northern Avenue	34500	36407	10178
AFRGLN	Glendale Avenue	30000	36455	11952
AFRBH/Pit19	Bethany Home alignment	30000	36416	9921
AFRCML	Camelback Road	54400	54400[1]	52000[2]

[1] New River inflows control peak at this location, no change from Effective FEMA flow and no hydrograph derived at this location.
[2] Estimated based on peak flows, no hydrograph derived at this location.

Detailed examination of Figure 3 also shows that the large pits downstream of Northern Avenue do not provide any appreciable further reduction in flows. This is due to the small volume of tributary inflows in this reach. Dysart Drain does contribute flow, but the net effect on discharges in the river is relatively small. The implications of which basins could be effectively used to reduce peak discharges is further explored in Task 2.4.

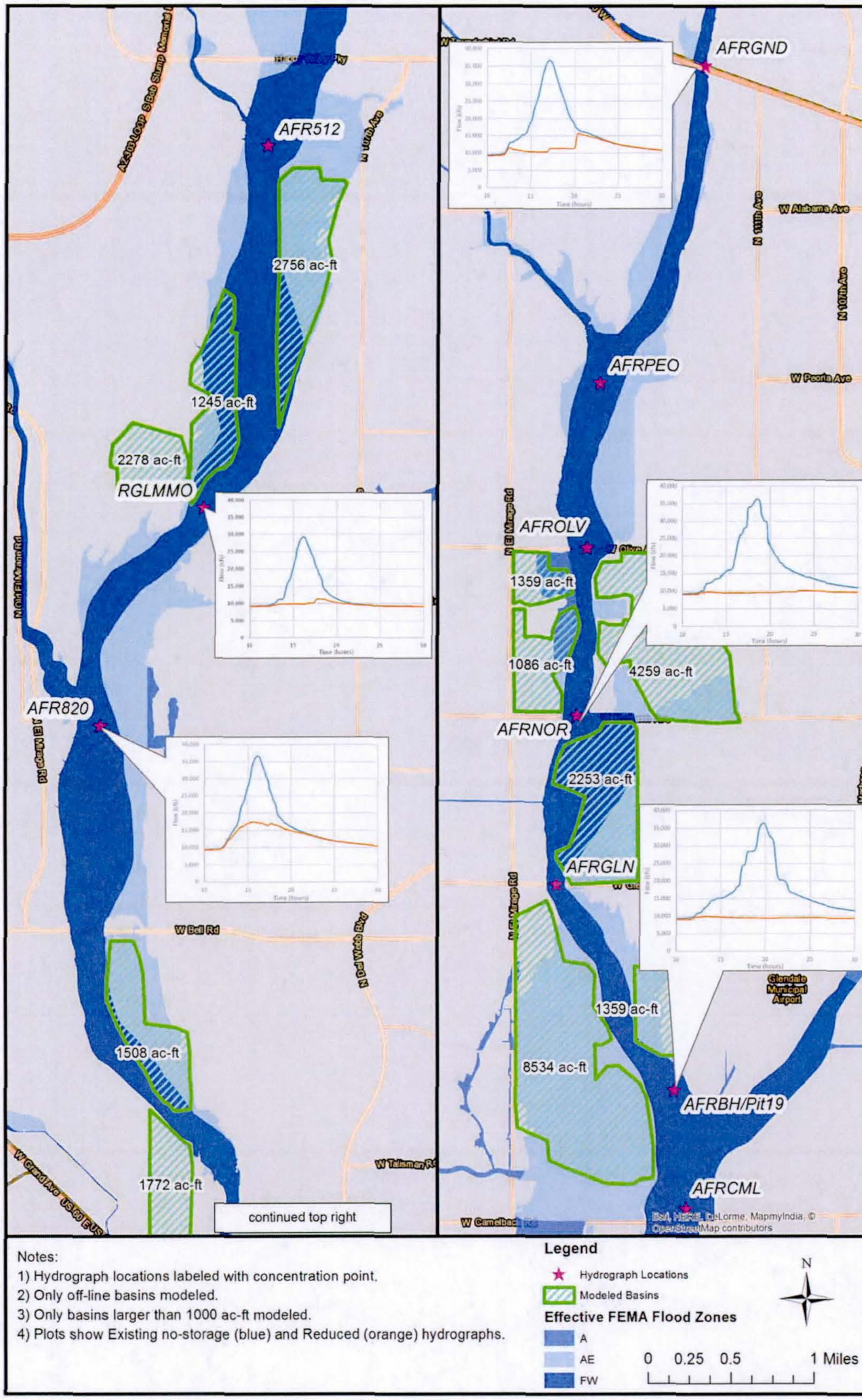

Figure 3. Potential peak flow reduction

HEC-RAS Modeling of Potential Floodplain Implications

Once the revised peak flows were developed, the effect on the floodplain was estimated. Using the cross-section locations from the current effective floodplain model, the cross-sections were recut using the same mosaic topography used to estimate the potential pit storage volumes. The model was refined by:

- Adding blocked obstructions to gravel mines equal to the lowest downstream ground elevation. This ensures the volume is not used for conveyance in the model.
- Adding levees where flow does not appear to break out based on the mosaic topography elevation, regardless of whether it was an engineered structure or not. Reliance on these levee-like features for FEMA approval of a revised floodplain would require compliance with 44 CFR 65.10 – e.g. freeboard, erosion protection, geotechnical stability, etc. Without levee *certification floodplain limits would be greater than shown in Figure 4. The implications of* levee requirements and the NFIP requirements is further discussed in Task 2.4.
- Adding ineffective flow areas at abrupt contractions and expansions locations.
- The exact bridge geometries were not added to this initial floodplain model. Rather, levees were added in the RAS model to ensure flow remained in the main channel at the bridge section unless the channel banks were overtopped. This was considered adequate to assess potential changes to the new floodplain limits.

The results of the preliminary floodplain analysis are shown in Figure 4. Note the relatively small differences between this floodplain and the current effective floodplain. In some cases it is also evident from the newer aerial photography that the floodplain could actually be bigger in areas where active sand and gravel mining has expanded the pits beyond the current floodplain limits, but would fill with water if flow were to enter the pit.

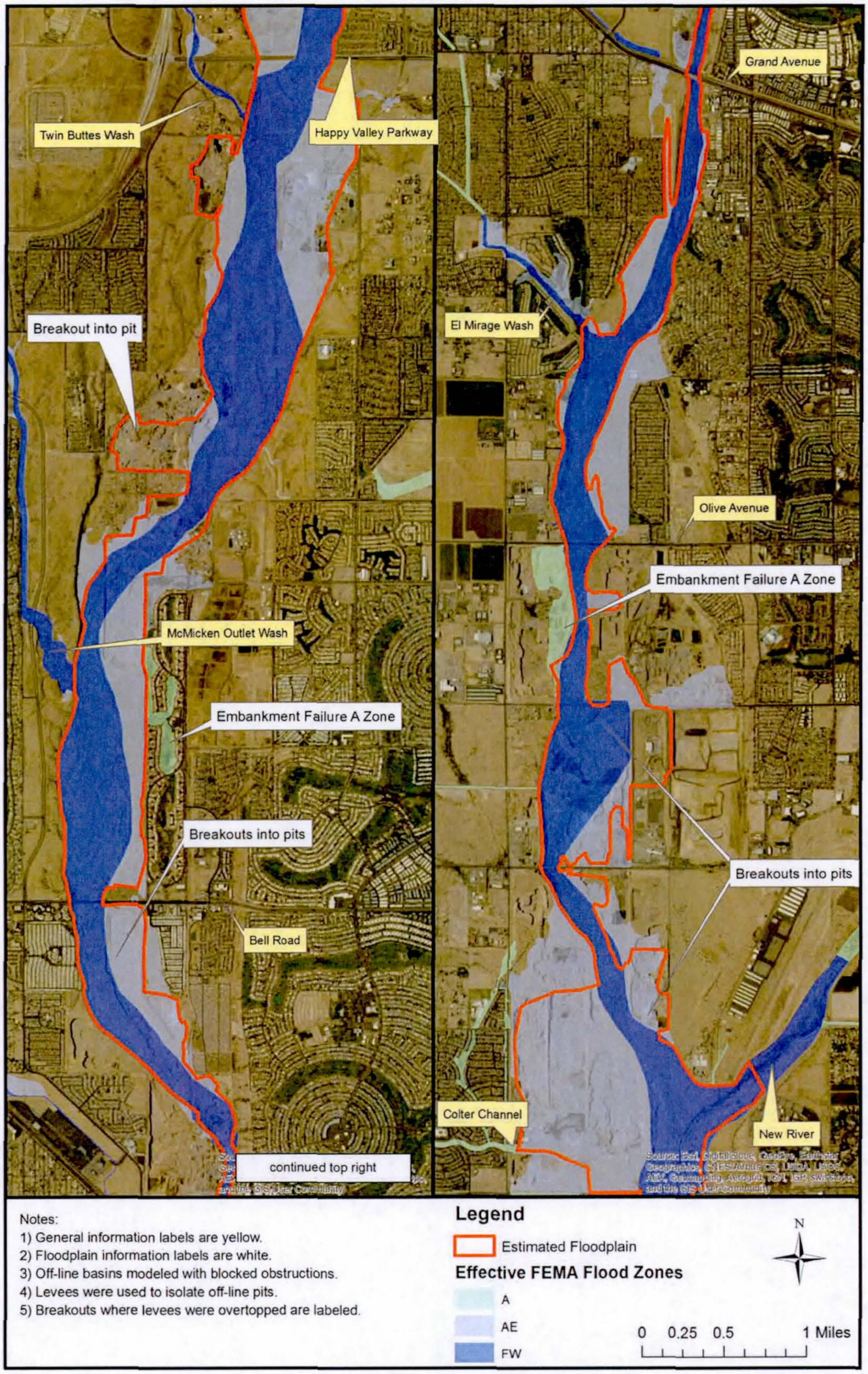

Figure 4. Estimated floodplain with reduced flow rates from Table 3

Analysis of Results

With the available storage in the river, it is possible to reduce the 100-year regulatory flow rate. However, other factors may limit the benefits of this reduction. For example, any storage area used to reduce the flow rate would be considered part of the floodplain. Hence, the reduction in floodplain extents would probably be limited to the reach of river that runs from the confluence with McMicken Outlet Wash to Bell Road and to the reach from Grand Avenue to Olive Avenue. These two reaches can be seen in Figure 5 as well as the location of both the modeled and un-modeled (other) basins.

In these two reaches, the peak flow can only be reduced to about 17,000 cfs based on existing locations of available storage and tributary inflows. Therefore, the floodplain area reduction is not as great in these reaches, unless further engineered channelization were provided. Additionally, these two reaches only represent about 1/3 of the entire of the study area, and have areas where development has already encroached. For example, the Grand Avenue Bridge and the landfill just downstream of Grand Avenue have already significantly decreased the width of the floodplain in that reach of the river. It appears that the main benefit of a reduction in the 100-year regulatory flow rate may be the reduced infrastructure cost to replace/build bridges or utility crossings rather than reduce the floodplain's footprint.

The implication of these results is that reduction in the peak discharge is insufficient alone to significantly reduce the areal extent of the 100-year floodplain. Significant reduction in the floodplain limits would require additional engineering measures such as engineered levee construction and/or channelization. In order to officially change the floodplain limits, FEMA and local criteria for all structures would be necessary. Those requirements include demonstrable operation and maintenance by a public entity, such as a city or the County.

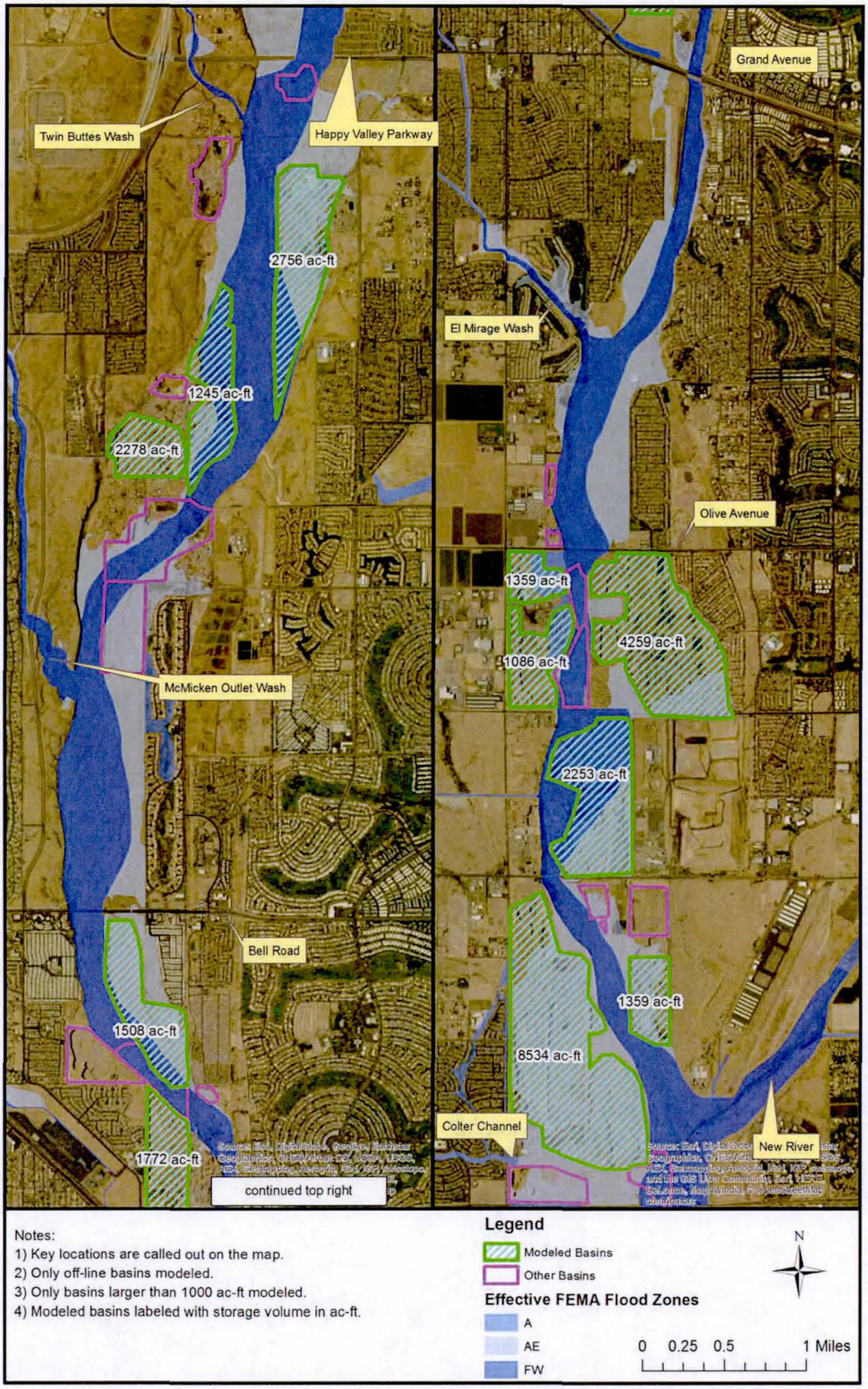

Figure 5. Characteristics of study reach

Agua Fria River Hydrology & Hydraulics Feasibility Studies

FCD 2015C008, Work Assignment No. 1

Task 2.4 Memorandum

This memorandum presents the results of Task 2.4: Issues Assessment. The major implementation considerations are summarized in the list below, and a brief discussion of each item is provided in subsequent paragraphs. The major considerations are:

- FEMA requirements
    - Dedicated storage volume (land rights)
    - Storage areas in floodplain
    - Maintenance plan
    - Controlled inlet structure
- Cost (land acquisition, design and construction of inlet structures and/or levees)
- Land rights acquisition
- Coordination (crosses multiple jurisdictions and affects numerous private landowners)
- Timing/Phasing (of future mining, design, construction and/or to obtain LOMR)
- Environmental permitting (e.g., 401/404 permits)
- Other requirements (e.g., drain time of storage areas)

FEMA Requirements

To officially change the regulatory 100-year flow rate on the Agua Fria River, a letter of map revision (LOMR) will need to be obtained from FEMA. For storage areas, FEMA has three main requirements. Each storage area must:

1) have as-built plans that verify the available storage,
2) have an officially adopted maintenance plan that guarantees the storage will remain, and
3) all maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance.

The above requirements imply that the gravel mines cannot be relied for storage in their current state. A design must be carried out to quantify the storage, prevent erosion when floodwaters enter the mine and prepare engineered plans for the basin. Additionally, since each storage basin requires a maintenance plan, coordination between private landowners and public agencies on who perform maintenance, questions arise such as: can mining still occur on a storage area? And, who will be the ultimate owner of each flood control structure?

Then, to remove any low-lying areas (e.g., gravel mines) from the floodplain, accredited levees would need to be constructed to prevent water from entering in an unsafe manner. For accredited levees, FEMA has numerous requirements outlined in 44 CFR 65.10. Each levee must:

1) meet certain design criteria,
2) have an operation plan,
3) have an interior drainage plan,
4) have a maintenance plan, and
5) meet certification requirements, such as sufficient freeboard, erosion protection, geotechnical stability, etc.

Each of the above items have detailed requirements that are outlined in the accreditation checklist (FEMA, 2008). For example, the certification requirements have two subtasks - a) all submitted data is certified by a professional engineer or a federal agency, and b) certified as-built plans are included with the submittal.

Basically, the existing mines have a large available storage volume, but this volume cannot be leveraged until a LOMR is obtained. However, to obtain a LOMR, all the above requirements for a storage area need to be met; and if low lying areas need to be removed from the floodplain, all the levee requirements need to be met. The FEMA requirements will greatly increase the complexity and cost of any project whose goal is to reduce the discharge and floodplain of the Agua Fria River.

Costs

Since the basins will need to be designed to capture the correct flow and to prevent erosion and failure, the general costs of the designed structures needed to be estimated. To accomplish this task, one area of the project (focus area) was analyzed under two scenarios – 1) existing channel and mine alignment (existing) and 2) encroached channel and pit alignment (encroached). The focus area was chosen because it is at the upstream end of the project where storage is most needed, and it provides a case study that can be applied to other potential storage basins in the area. The two scenarios were chosen because the results of the multiple weir scenarios from Task 2.2 indicate that the length of a lateral weir is generally 1.7 times the channel width (see Figure 1). Since structural concrete is generally more expensive than bank protection or soil cement, a narrower channel was chosen to see if the increased levee height was offset by a shorter weir for a net less expensive result.

The focus area lies within the Agua Fria River and is bounded by Hatfield Road on the north and Rose Garden Lane on the south and is shown in Figure 2. The two levee alignment scenarios are shown on the figure as well as the estimated floodplain that was delineated in Task 2.3 (reported in the Task 2.2 Follow Up Memo). The cost estimate for the existing scenario is shown in Table 1, and the estimate for the encroached scenario is shown in Table 2. The unit costs for these estimates were compiled from recent projects in Arizona, New Mexico, and California, but a lower unit cost was used for earthwork since these basins occur in active mine sites. The final costs for a levee was estimated as $900-$1050 per lineal foot, while a weir is estimated to cost around $1300 per lineal foot.

The encroached scenario has a larger cost because it results in a longer, higher levee. As the width of the channel decreases the depth of flow also increases. This increases the required height of levee, and it was found each additional foot of height increases the cost per lineal foot $50. The encroached scenario resulted in the $1050 per lineal foot estimate. However, it should be noted that each estimate used a constant levee size. In reality, there will be a decrease in levee height after flow has been diverted over the weir, but this kind of detail was not included in the cost estimate. The maximum levee height was used over the entire length of the proposed levees.

Since this focus area is representative of typical engineering structures that would be needed to control erosion (headcut) from water flowing into a potential storage areas, these estimates provide a construction cost that could be applied to other locations in the study reach. Excluding land acquisition costs, the exemplary costs for the reach shown in Figure 2 were extrapolated to the remaining downstream reaches. A cost of about $15 million per pit for the focus area results in a total cost between $66 and $80 million to improve the effective and very effective pits into flood control storage facilities. Permitting, utility relocation, maintenance, and other elements would likely add to actual implementation costs. Additionally, these cost estimates assume that the basins could be drained adequately by percolation. Design and construction of pipe outlets if needed would add significantly to the costs and potential land acquisition needs.

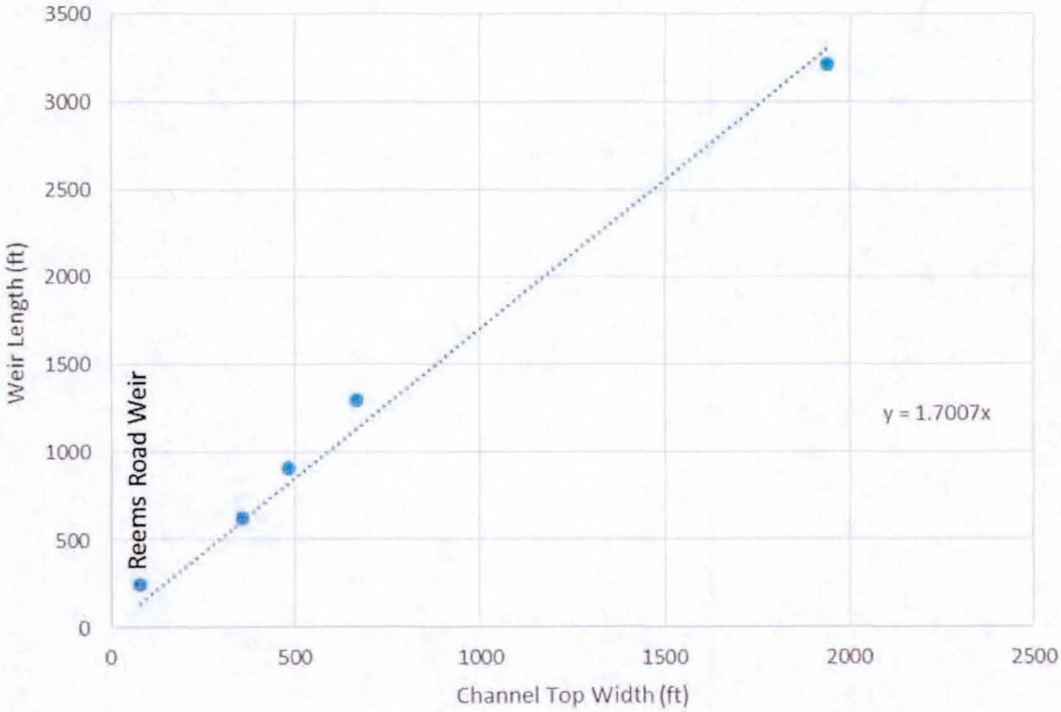

Figure 1.Result of multiple weir scenarios with trend line and trend line equation

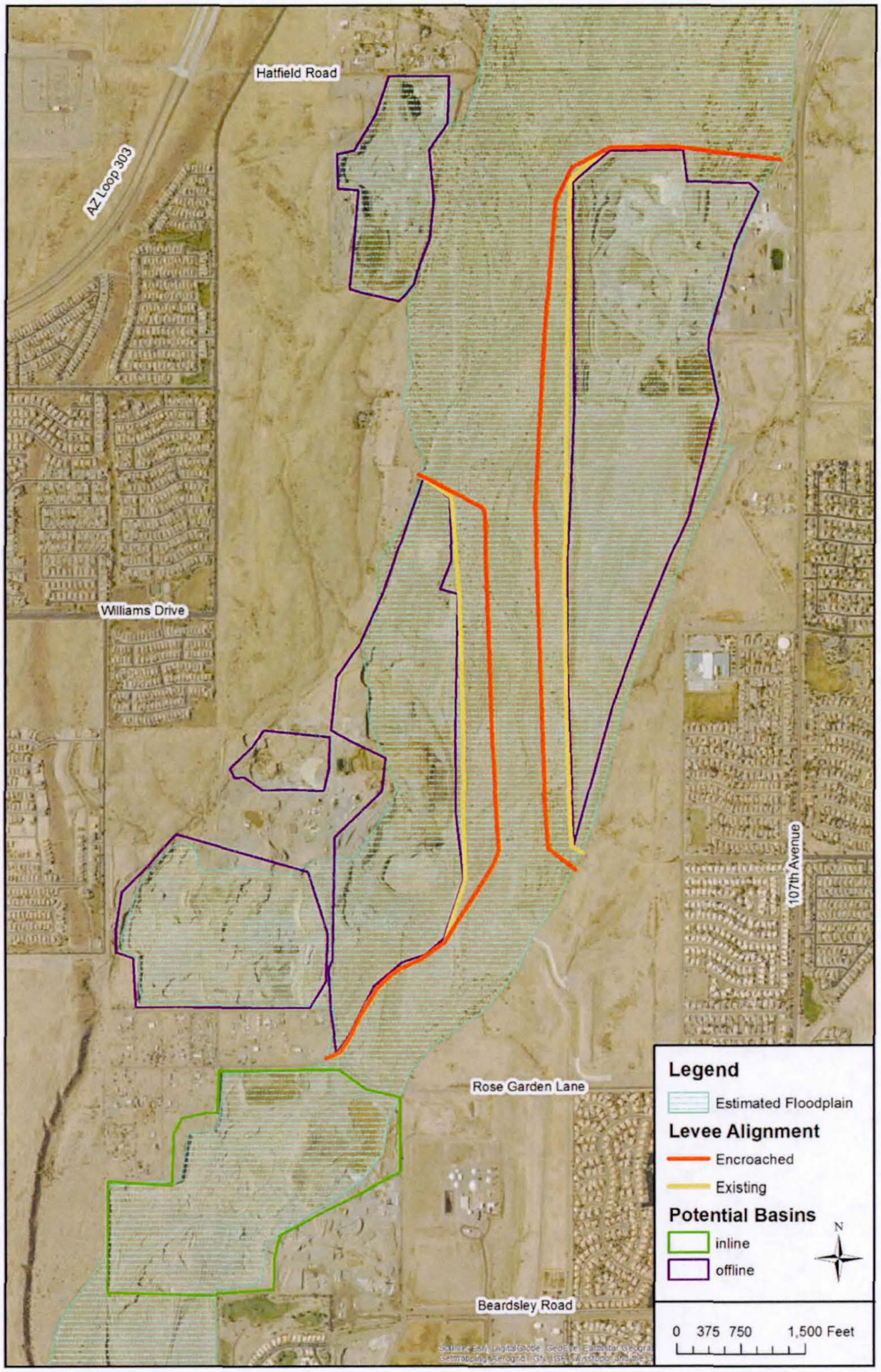

Figure 2. Schematic of focus area just upstream of Rose Garden Land and south of Happy Valley Parkway.

Table 1. Cost estimate for existing scenario (rounded to nearest $100K)

Item	Description	Quantity	Unit	Unit Cost	Total Cost
Embankment	compacted fill behind soil cement	116508	CY	$ 30.00	$ 3,500,000.00
Soil Cement	levee erosion protection	88546	CY	$ 58.00	$ 5,100,000.00
Handrail	only used on top of levee	13981	LF	$ 160.00	$ 2,200,000.00
Structural Concrete	for two lateral weirs and spillways	323000	SF	$ 16.00	$ 5,200,000.00
Excavation, Backfill, and Compaction	related to construction of structures (levee and weirs)	116138	CY	$ 15.00	$ 1,700,000.00
18" D50 Riprap	erosion protection at bottom of spillway	11806	CY	$ 74.00	$ 900,000.00
Earthwork	regrading of side slopes to 3:1	1637500	CY	$ 4.00	$ 6,600,000.00
Contingency	used 20% because there should be minimal utility conflicts near existing gravel mines	1	LS		$ 5,000,000.00
					$ 30,200,000.00

Table 2. Cost estimate for encroached scenario (rounded to nearest $100K)

Item	Description	Quantity	Unit	Unit Cost	Total Cost
Embankment	compacted fill behind soil cement	216877	CY	$ 30.00	$ 6,500,000.00
Soil Cement	levee erosion protection	121640	CY	$ 58.00	$ 7,000,000.00
Handrail	only used on top of levee	16973	LF	$ 160.00	$ 2,700,000.00
Structural Concrete	for two lateral weirs and spillways	155040	SF	$ 16.00	$ 2,500,000.00
Excavation, Backfill, and Compaction	related to construction of structures (levee and weirs)	149287	CY	$ 15.00	$ 2,200,000.00
18" D50 Riprap	erosion protection at bottom of spillway	5667	CY	$ 74.00	$ 400,000.00
Earthwork	regrading of side slopes to 3:1	1748000	CY	$ 4.00	$ 7,000,000.00
Contingency	used 20% because there should be minimal utility conflicts near existing gravel mines	1	LS		$ 5,700,000.00
					$ 34,100,000.00

Coordination

Since each potential storage basin is currently an active mine site, coordination on how to transition the mines to storage areas must be considered. To take advantage of the available storage volume, FEMA requires submittal of as-built plans that verify the initial volume is available and a maintenance plan that guarantees this volume remains in place. These requirements would seem to suggest that the storage area could not remain an active mine site.

Timing/Phasing

Before the floodplain is ultimately changed, there are multiple time frames that should be considered. These are:

1) Design: usually 1-2 years
2) Conditional LOMR approval: about 1 year
3) Construction: 3-5 years (funding available), indefinite (without approved funding)
4) FEMA Approval: 1-2 year (LOMR), 3-5 (Physical Map Revision)

As a result, the best case time frame for final completion is 6 years with a more probable time frame being on the order of 10-15 years, even with an identified funding source.

Additionally, as mentioned under coordination, the continued operation of key pit locations could further slow implementation of the overall project.

Environmental Permitting

Since this project occurs within a major watercourse, it would have to comply with Section 404 of the Clean Water Act. This would require a 404 delineation and subsequent permit for construction activities within the delineation. Complying with Section 404 may slow the project due to the required review and approval by the Army Corps of Engineers. It could also add to the project costs. Once the permit is obtained, it may require other stipulations, such as compensatory mitigation to offset unavoidable adverse impacts due to the proposed project.

The project must also be in compliance with 40 CFR 122, the National Pollution Discharge Elimination System (NPDES), and the Arizona Pollutant Discharge Elimination System (AZPDES). However, since it does not directly produce discharges from stormwater facilities, a general permit may be all that is required.

Other Requirements

In addition to the FEMA requirements enumerated above, there are other regulatory requirements that would need to be fulfilled. For example, a retention basin in Maricopa County is required to drain within 36 hours for "vector control and to allow for the probability of a second severe storm following the previous storm."

Since these basins will ultimately be below the river invert, the water will only drain through percolation or lengthy pipe outlets. Based on the Maricopa County Drainage Policies and Standards, basins that are larger 50 acre-feet can be approved for a longer drain time, but vector control provisions will be one of the requirements for approval of the extended drain time. Percolation tests to estimate the drain time would also be required, or outlet pipe included in the basin design and cost.

Another requirement stems from the Maricopa County Drainage Policies and Standards (FCDMC, 2007), "emergency spillways shall be provided for all stormwater storage basins. For basins with all the design storage volume situated below existing grade (i.e. without a berm/dam), the spillway may be nothing more than grading to ensure that basin overflows will follow the downstream predevelopment drainage pattern in a safe manner."

Since these basins would only drain through percolation, they might be classified as a groundwater recharge facility, which would require a groundwater recharge permit from Arizona Department of Water Resources (ADWR) and an assessment of groundwater recharge capabilities and potential groundwater quality impacts.

The basins are much larger than the typical retention basin and therefore would require a more detailed analysis of freeboard. Based on the design of the Chandler Heights Basin (Kirkham Michael, 2002) *freeboard considers:*

- Settlement of the embankment,
- Flood levels higher than design,
- Malfunction of outlet works, and
- Surface waves being generated to a height greater than the still water surface.

For the basins in this project, the only two additional considerations would be flood levels higher than design and wave action. Both of these could be easily handled during final design but could add to the total costs or reduce the efficacy of the basins themselves (smaller effective storage volumes).

Additionally, slope stability considerations would need to be addressed. Many of the existing mines have vertical or near vertical pit walls. Impoundment of water within these pits as-is would likely result in the collapse of these walls. Therefore, regrading of the pit margins to shallower slopes (e.g. 3H:1V) would probably be required. This might require additional land area and cost to the design and construction for use as a flood control basin.

Summary

In general, all the above considerations can likely be overcome. The FEMA and other regulatory requirements can be met with a detailed engineering analysis during final design. However, they lead to other issues, which have the potential to be the largest stumbling blocks. These are:

1) coordination of land rights between private/public landowners for dedication of the storage volume, and
2) cost (around $15-18 million to convert one pit into a functioning storage basin to FCDMC and FEMA standards).

The total cost could be handled by designing and constructing the project in phases, starting at the upstream end of the reach, so the upfront cost is spread over multiple years. Coordination between the multiple municipal jurisdictions and private landowners along the river will have to be achieved for successful implementation.

References

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Kirkham Michael and Associates, Inc. (Kirkham Michael), East Maricopa Floodway Chandler Heights Basin Design Predesign Study Final Report, January 2002.

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